Chapter 6

GEOTECHNICAL EXPLORATION, MATERIAL TESTING, ENGINEERING ANALYSIS AND DESIGN

This chapter addresses important aspects of geotechnical exploration, material testing, engineering analysis and design for coal refuse disposal embankments and impoundments with consideration of past, current and future mining practices; characteristics of foundation, coal refuse, and soil and rock borrow materials; and procedures for material placement and facility construction. Development of a subsurface exploration program, implementation of a field and laboratory testing program, and selection of geotechnical parameters are key elements in the design of safe facilities for coal refuse disposal. The basic design considerations that must be evaluated for both embankments and foundations are seepage, slope stability and settlement. Another important geotechnical consideration is the analysis of soil-structure interaction for buried pipes (conduits or decant pipes) that are installed within an embankment. Specification, field control and verification of geotechnical properties are essential to the construction of coal refuse embankments that are consistent with design assumptions.

From a geotechnical perspective, the following steps are normally followed in the design and construction of a new or modification of an existing refuse embankment:

- Review of available information
- Site selection and optimization
- Field exploration and in-situ sampling and testing
- Laboratory testing
- Development of geotechnical design parameters
- Analyses and design
- Preparation of plans and specifications
- Construction monitoring (quality control and verification of field conditions)
- Instrumentation
- Embankment and system component performance monitoring
- Maintenance

The level of effort and technical scrutiny required during the above steps will vary depending upon the refuse facility intended use, size and hazard-potential classification.

This chapter describes the scope of geotechnical investigations recommended for support of analyses and design of a refuse disposal facility. References for additional information are provided herein. Based upon the technical guidance provided in this chapter and supplemental information available in the cited references, an experienced geotechnical engineer familiar with the refuse disposal process and the design of water-retention embankment structures should be able to design an economical, safe and environmentally acceptable coal refuse disposal facility. Designers should recognize that investigation programs and studies for specific projects can not realistically be standardized and will vary according to site conditions, material properties, embankment geometry, hazard classification and proposed staging scheme.

In addition to refuse disposal embankments, this chapter is also applicable to other types of embankments that are employed at mine sites including fresh water dams and sedimentation or treatment ponds. Other applications and available design guidance are presented in Section 6.3.6.

In the text that follows, many references are made to ASTM International (ASTM) standards. All references to ASTM standards in this chapter can be found in three volumes of *Section Four – Construction* of the ASTM standards, which are published annually. The most current versions of the standards should be used. The applicable volumes and their citations herein are:

- Volume 05.06 – Gaseous Fuels; Coal and Coke (ASTM, 2008a)
- Volume 04.08 – Soil and Rock (I): D 420 – D 5786 (ASTM, 2008b)
- Volume 04.09 – Soil and Rock (II): D 5877 – latest (ASTM, 2008c)
- Volume 04.13 – Geosynthetics (ASTM, 2008d)

Full citations for these volumes are provided in the References section at the end of this Manual.

6.1 GEOTECHNICAL DESIGN PROCEDURES

This section outlines the steps normally required for designing the geotechnical aspects of a coal refuse disposal facility. The geotechnical design of a new refuse disposal facility provides flexibilities that designers should recognize in their planning. These include:

- Ability to determine the optimum location for the facility
- Flexibility in the design of the starter dam and embankment staging
- Ability to coordinate ongoing and future mining with staging in the vicinity of refuse disposal footprints
- Flexibility in designing the internal drainage system and liner system (if required)
- Flexibility in the selection and design of hydraulic structures

Geotechnical design for an expansion of an existing disposal facility typically imposes constraints that designers must recognize in their planning, particularly limited flexibility in planning embankment staging and design of hydraulic structures while maintaining ongoing disposal operations. Table 6.1 presents guidance for the geotechnical design of refuse disposal facilities with reference to applicable sections of this Manual and to supplemental documents.

6.2 GENERAL CONSIDERATIONS

6.2.1 Unique Characteristics of Refuse Disposal Facilities

A typical coal refuse disposal facility has unique characteristics and objectives compared to most other engineered structures. Some of the basic characteristics of coal refuse disposal facilities and their related significance in geotechnical design are identified in Table 6.2.

TABLE 6.1 TYPICAL GEOTECHNICAL DESIGN PROCEDURE FOR COAL REFUSE DISPOSAL FACILITIES(1)

Design Considerations	Manual Sections for Reference	Supplemental References
I. Obtain and Review Available Information		
Topographic Maps Geologic Maps Soils Maps Aerial Photographs Local Experience Individual Site Mapping Seismicity Maps Mine Maps	6.4	USBR (1992a)
II. Plan Field Exploration		
What is probable configuration? What type of hydraulic structures are likely? Where might important embankments and structures be located? What are significant foundation characteristics? What types of borrow material may be required? What are probable sources of borrow material? What types of sampling will be required? What types of environmental control measures should be considered? Is past or present mining in the area being considered? What changes in the disposal program are likely during the life of the facility?	6.2, Chapter 3 Chapters 3, 5, 9 6.2, Chapter 3 6.2, 6.3, 6.4, 6.6 6.2, 6.3 6.2, 6.3 6.4, 6.5 6.3, Chapters 4, 10 6.3, Chapter 8 Chapters 4, 10	USBR (1992a) Sherard et al. (1963)
III. Field Exploration		
Surficial Reconnaissance Geophysical Surveys Borings and Sampling Test Pits Visual Classification Field Testing Soils and Water Inventory	6.4	Arman et al. (1997) Hvorslev (1948) Legget (1962) USBR (1998, 1992a)
IV. Laboratory Program		
Natural Materials • Index Property Tests • Compaction Tests • Hydraulic Conductivity • Consolidation • Shear Strength • Potential Acidity/Neutralization Potential	6.5	ASTM (2008b,c) Lambe (1951) Bishop and Henkel (1962) USBR (1992a)
Refuse Materials: • Index Property Tests • Compaction • Hydraulic Conductivity and Consolidation • Shear Strength • Leachate Quality	6.5	MSHA (2007)

TABLE 6.1 TYPICAL GEOTECHNICAL DESIGN PROCEDURE FOR COAL REFUSE DISPOSAL FACILITIES[1] (Continued)

Design Considerations	Manual Sections for Reference	Supplemental References
V. Design Considerations and Analyses		
Foundation Preparation Seepage Control Static and Seismic Stability Settlement Rock Excavation	Chapters 6, 7, 8	USBR (1987a,1989) Leonards (1962) USACE (1993)
VI. Construction Operations		
Refuse Transport and Placement Foundation Preparation Borrow Materials Appurtenant Facility Construction Materials Selection Quality Control and Field Testing	Chapter 11	USBR (1987a,1989) Church (1981) USBR (1998) Fell et al. (2005)
VII. Instrumentation and Monitoring		
Visual Observations Movements and Displacements Pore-Water Pressures Hydrology and Hydraulics General Maintenance	Chapters 12, 13	USBR (1998) Dunnicliff (1993) USACE (1995c)

Note: 1. This table is presented as a guide to qualified geotechnical designers. Each site must be evaluated according to conditions at that site. In some cases, studies beyond those identified in this table may be needed.

6.2.2 Site Conditions

In contrast to site selection for a dam that must be built across a valley to form a reservoir of designated size, site selection for a coal refuse disposal facility is generally flexible because of the variety of embankment types that can be constructed. The principal considerations in the selection of a disposal facility site are typically: (1) the potential disposal capacity of the valley/site, (2) the desired preparation plant processing output, (3) the potential influence of previous mining activities, (4) refuse transportation and placement, (5) construction of site development and drainage structures, and (6) other environmental control and safety factors.

6.2.2.1 Topography

Site terrain slopes are very important in disposal facility site selection because of their impact on storage volume, methods and costs of materials handling, methods and costs of drainage control, and hazard potential.

In areas of rugged and steep terrain, such as southern West Virginia, southwest Virginia, and eastern Kentucky, most disposal facilities will be valley-type embankments. Valley slopes are often too steep for side-hill embankments. Ridge tops are generally difficult to access and are limited in area to support large embankments; however, some ridge-top sites can be used in conjunction with mountain-top surface mining operations. Major valley bottoms are too confining for the construction of large heaped or diked embankments. Therefore, the typical disposal facility site is generally a small valley selected by considering:

- Potential effects of past or future mining beneath the site
- Proximity to existing preparation plants
- Surface land ownership
- Cost of establishing a materials-handling system suitable for the topography
- Ability to sequence construction to handle the types and volumes of refuse to be disposed
- Potential to discharge storm water through or around the site
- Stability of existing slopes when modified by construction or imposed loads

In areas of less severe topography, such as in southwestern Pennsylvania, the range of potential site location and embankment types is much greater. However, disposal facilities in these areas are frequently located in small valleys because of their capability to accommodate large volumes of refuse without extensive modification of natural topography. Also, the rolling topography often makes construction of ridge, side-hill or heaped embankments practical.

In relatively flat terrain, as found in portions of Illinois and Indiana, refuse disposal in valleys may not be practical, and the refuse disposal facility will generally be of the heaped (Figure 3.12) or diked-pond embankment type (Figure 3.13. These types of disposal facilities present unique problems for fine refuse disposal in slurry form because volume containment by utilizing the natural topography is not possible. When fine refuse slurry is a small percentage of the total refuse, the most economical disposal facility is a diked-pond embankment constructed from coarse refuse.

TABLE 6.2 BASIC CHARACTERISTICS OF COAL REFUSE DISPOSAL AND DESIGN SIGNIFICANCE

Basic Characteristic	Design Significance
The purpose is safe and economical disposal of refuse.	Greater flexibility in choosing location, configuration and sequence of placement.
The total disposal volume of coarse refuse is normally much greater than that required to serve safety requirements.	Embankment zones may allow different placement specifications (e.g., structural zone).
The refuse disposal occurs over many years and may lead to several unforeseen events and constraints not realized at the time of design.	Construction monitoring and quality control may be specified for the critical construction items with periodical monitoring of routine construction.
The geotechnical properties of refuse may not be available during the design (particularly for a new facility), and changes in the material properties are probable during the life of the facility.	For new facilities, geotechnical design parameters may be estimated based on experience and from facilities with similar characteristics (e.g., similar seam properties, mining technique, and cleaning process). The geotechnical design parameters can be verified when actual samples are available and can be re-evaluated if characteristics change.
The refuse being placed can have adverse chemical characteristics that may lead to undesirable environmental conditions or deterioration of construction materials.	Geotechnical and leachate characteristics of the refuse should be evaluated based on experience and/or by laboratory test, and appropriate amendment or containment/protection requirements should be identified.
After completion of disposal operations, the facility will need to be abandoned in a safe, economical and environmentally acceptable manner.	Planning and design must allow for an acceptable abandonment configuration with materials for effective reclamation.

6.2.2.2 Climate/Weather

Due to the small surface area of coal refuse impoundments, wind, rainfall and temperature conditions are generally not major factors in selecting the most appropriate site for a refuse disposal facility for a given mining operation. However, the variation of weather conditions in different regions of the country can significantly affect disposal facility configuration and design requirements.

For example, in the Appalachian coal region, rainfall is relatively uniform and abundant throughout most of the year. Thus, addition of water to coal refuse for controlled placement in an embankment is seldom a major design and cost consideration. A more important criterion in this region may be to assure that work in valley bottoms, or in critical structural portions of the embankment, can be accomplished during the summer months when conditions are driest.

At the other extreme, such as in the semi-arid Rocky Mountains and Western plains, there is a general lack of precipitation for most of the year. In this situation, the designer must evaluate appropriate control measures and associated costs for adding water to refuse to achieve required placement criteria. Also, the precipitation that does occur is often in the form of high intensity thunderstorms, increasing the required capacity of flood control and diversion structures.

Clearly, the planning of otherwise similar disposal facilities will vary according to geographic location. Regardless of the site location, the size of the watershed draining into a disposal area should be minimized unless other design factors justify the cost of major drainage control systems.

Disposal facilities in regions with extended cold winters and significant snowfalls may require special attention to configuration, materials handling and refuse placement. For example, it may be that critical structural portions of an embankment can be constructed most economically during the construction season when drainage and material properties are most easily controlled. Overall efficiency is then accomplished by establishing areas for placement of refuse in non-critical areas during the remainder of the year.

Blowing dust is generally not considered to be a major design problem with coal refuse disposal. However, dust has been a problem with other types of industrial and mining waste disposal, particularly for disposal of fine-particle materials such as combustion waste.

6.2.2.3 Geology and Surficial Soils

Coal refuse disposal facility design is affected by geology and surficial soil conditions as they relate to the following:

- Extent and effects of past or future mining
- Necessary foundation treatment
- Available borrow material
- Type and size of the starter dam
- Effects of refuse disposal on groundwater quality and the effects of groundwater seepage on embankment design
- Stability and hydraulic conductivity of the existing foundation materials under natural and disturbed conditions

Specific design factors that may be affected by geology and surficial soil conditions include:

- Acceptable embankment slopes as related to the shear strength of the underlying foundation materials.

- Limitations on the location and design of the impounding facility posed by active or inactive landslides.
- Seepage cutoffs through pervious foundation soils for embankments that impound water.
- Embankment construction to account for settlement of underlying soft foundation materials.
- Selection of the starter dam configuration and zoning.
- Starter dam and embankment construction rates to keep excess pore-water pressures to within acceptable limits.
- Seals between foundation rock and the impervious zone of an impounding embankment.
- Situations where badly fractured foundation rock must be grouted to control seepage from an impounding embankment.
- Liner and underdrain systems to address protection of groundwater or seepage into the embankment.
- Selection of decanting and other hydraulic structures to control impounded storm runoff.
- Erosion control measures for surface runoff and drainage structures
- Potential liquefaction of embankment and foundation materials in regions subject to moderate or high seismic loading.

The wide variations of geology and surficial soil conditions that may be present make generalized examples of these situations impossible. However, if mining has occurred beneath a disposal facility, it is particularly important to evaluate the effect of existing or potential subsidence on the embankment and the potential for the contents of the reservoir to break through into the mine. If it is determined that subsidence has occurred or that new or continued subsidence may occur, potential detrimental effects on the structural and hydraulic conductivity characteristics of the refuse embankment must be evaluated. The possible effects on groundwater quality due to leachate infiltrations into fractured foundation bedrock, or due to discharge of fine coal refuse into underground mine voids if the overburden collapses, should also be considered. If mining has not occurred at the planned refuse disposal facility site, construction of the disposal facility may limit the extraction of underlying coal reserves, and this cost factor should be considered in the economic evaluation.

Procedures for investigating and analyzing geology and surficial soil conditions are discussed in Section 6.4, and construction aspects of foundation preparation are discussed in Chapter 11. Designers must understand that an optimum disposal facility can only be achieved if general site conditions are well understood prior to the selection of the facility site and configuration. This knowledge will help to prevent unnecessarily conservative designs as well as designs that may be susceptible to increased maintenance or environmental problems.

6.2.2.4 Miscellaneous Site Considerations

Other considerations that may not be directly related to site conditions, but may influence the design and the volume capacity of the facility include:

- Access to public roads
- Availability of utilities
- Future mine developments
- Mine infrastructure (e.g., conveyor belts, access roads and mine entries)

6.2.3 Embankment Materials

The purpose of a coal refuse disposal impoundment is to provide a means for safe and economical disposal of coal refuse. However, due to safety and operational considerations, other materials are required for development of the facility. For example, since coarse coal refuse is generally susceptible to weathering, crushing and degradation, more resistant granular rock is normally needed for construction of drainage zones and for providing erosion protection. Also, at new facilities, disposal of fine coal refuse requires initial construction of a starter embankment or dam for slurry retention and settling when sufficient coarse refuse is unavailable during disposal facility startup. Starter dams are generally constructed with borrow material, unless suitable coarse coal refuse is available on site. The cost of imported borrow materials is much greater than the coarse refuse, so their use should be limited to addressing specific design requirements. Borrow materials can be obtained directly from mine spoils, from processing of mine spoils, or from a suitable area at or near the site. Embankment materials that must be purchased from a commercial quarry and transported to the site are the most costly alternative.

Other examples where site borrow or imported materials may be needed to supplement the refuse include: (1) fine-grained soils for creating an embankment impervious zone when the material available will not adequately limit seepage, (2) cover soils suitable for revegetation, and (3) soil and rock for supplementing embankment construction when coarse refuse is unavailable or inadequate for slurry retention and storm routing.

The most convenient and economical source for borrow material is typically the area that will eventually be covered by refuse. Use of this material will minimize transportation costs and will increase the capacity of the disposal facility, although it can lead to more extensive seepage control and groundwater protection requirements. Another economical source of borrow material is mine spoil with a matrix of soil and rock. To avoid unnecessary double handling and stockpiling and to assure that the required quantities of materials are excavated from the disposal area, careful planning is essential. This precaution is especially important when the borrow material will be used for final cover.

When selecting borrow materials from the disposal area, it is important that removal of the material will not be detrimental to the long-term performance of the facility. For example, if the natural soils form a desirable impermeable boundary between the refuse and underlying rock, borrow activities should be restricted to areas of thickest soil cover in order to leave a continuous layer of soil.

The required characteristics for materials used in the construction of refuse embankments are discussed in Sections 6.4 and 6.6; transport and placement procedures are discussed in Chapter 11. The following brief discussion of materials generally available for embankment construction is presented to aid initial planning and design.

6.2.3.1 Coarse Coal Refuse

Non-impounding embankments (“refuse piles”) are commonly constructed entirely with coarse coal refuse. Portions of a non-impounding embankment where supplemental borrow material may be needed include: (1) granular underdrain zones for collecting and discharging groundwater seepage away from the refuse, (2) granular zones for controlling seepage or collecting leachates, (3) cover material for promoting vegetation of the embankment surface, and (4) durable, weather-resistant rock for erosion protection in swales, ditches and channels.

Coarse refuse is typically the predominant material used to construct embankments for impounding fine coal refuse slurry. However, materials for impervious zones, filters, and drainage zones for these structures must have specific characteristics and normally must be obtained from suitable borrow areas or from commercial sources.

A summary of published grain-size, specific-gravity, and strength-testing data for coarse coal refuse samples, including their geographic source, is presented in Table 6.3. While coarse coal refuse is generally a well-graded material, significant variation, particularly with respect to the clay-, silt- and fine-sand-size fraction, has been reported depending on geographic location (which generally relates to geologic conditions) or coal mining or preparation processes. Advances in coal mining and preparation processes over the last 15 years have resulted in a trend toward greater percentages of fines in coarse coal refuse. While recent published data (Hegazy et al., 2004) provide evidence of the increase in fines content of coarse refuse in the northern Appalachian area, resulting in classification as silty, clayey sand with gravel to clayey, silty sand with gravel, other regions typically exhibit lower fines content with corresponding classification as a well-graded to silty gravel. Reported strength data are consistent with soil and rock content and gradation.

TABLE 6.3 COARSE COAL REFUSE CHARACTERIZATION – SUMMARY OF AVERAGE/RANGE OF VALUES

Reference	Location	Grain Size				Specific Gravity G_s (gm/cm^3)	Effective Shear Strength	
		D_{30} (mm)	D_{50} (mm)	D_{60} (mm)	Passing No. 200 Sieve (%)		ϕ' (degrees)	c' (psf)
Almes and Butail (1976)	PA, WV, KY, VA	0.7	2.5	4.5	10	1.8-2.4	33-39	0
McCutcheon (1981)	OH	1.9	4.5	7	7	2.0	36	NR
Saxena et al. (1984)	WV	12	16	22	2	2.6	27-40	0-450
Albuquerque (1994)	VA	3.5	7.5	12	1.5	NR(1)	39	0
Hegazy et al. (2004)	PA	0.35	1.23	2.02	19.8	2.0	34	250
Busch et al. (1974)	WV	0.2-4	1-10	3-20	2-19	1.7-2.3	NR	NR
Backer et al. (1977)	UT, NM	1-6	3-15	6-20	4-15	1.7-2.3	NR	NR
Stewart and Atkins (1983)	Eastern PA (anthracite)	1-8	5-16	7-22	1-7	2.2-2.4	NR	NR
Zeng and Goble (2008)	Appalachian region	2	6	9	12	2.5	NR	NR

Note: 1. NR = not reported

Compaction, equipment traffic and weathering cause degradation of coarse coal refuse. Larger pieces of shale will generally crumble to small particles after exposure to the atmosphere for only a short period of time. Thus, the percentage of fines in "aged" refuse can be noticeably greater than in fresh or recently placed coarse coal refuse. Based upon sieve analyses results for fresh and compacted samples, Hegazy et al. (2004) have reported an average increase of fines of about 4 percent due to compaction alone.

6.2.3.2 Fine Coal Refuse

Fine coal refuse, when very wet or in slurry form, is not generally suitable for construction of the structural portion of an embankment. However, fine refuse can be used as the foundation for portions of an embankment when it has had sufficient time to settle and excess pore-water pressures have adequately dissipated. Designers are cautioned that embankments that have fine refuse as a foundation material require comprehensive evaluations and analyses of the following:

- Construction schedule
- Geotechnical properties and strength
- Settlement and seepage properties
- Placement procedures
- Measures for equipment operator safety
- Seismicity, dynamic properties and potential for associated strength loss

Fine coal refuse is the product of extracting, crushing, and cleaning raw coal. The fine coal refuse slurry is typically pumped upstream of an impounding embankment. The coarser material settles out more quickly nearer the discharge location (customarily near the upstream slope of the embankment), forming a fines delta or beach. The finer materials migrate throughout the impoundment, because they take longer to settle. Thus, samples collected from or near the delta are predominantly sand and silt-sized particles, whereas samples collected away from the delta are predominantly silt and clay-sized particles. A summary of published geotechnical data (average and range of values) for fine coal refuse samples, including their source location, is presented in Table 6.4. This table is based on samples collected from slurry impoundments and may reflect the effect of segregation that occurs with settling and deposition. Variations in grain size and plasticity occur due to rock strata, coal extraction and processing, and impoundment depositional characteristics, and thus properties may vary from those reported in Table 6.4. Site-specific testing has characterized fine refuse as plastic clay/silt, low plasticity sandy silt or clay, or low to non-plastic silty/clayey sand typically exhibiting a lower specific gravity (and dry density) and lower peak shear strength than coarse coal refuse.

The grain-size distribution of thickened and dewatered fine coal refuse can be anticipated to be similar to the averages provided in Table 6.4, as this material does not significantly segregate with placement.

6.2.3.3 Combined Refuse

Some coal preparation plants produce combined refuse that does not require impoundments for disposal of fine refuse. At these preparation plants, partially dewatered fine refuse filter cake is produced in addition to coarse refuse. These materials are normally combined, transported and disposed in a non-impounding disposal facility.

Properties of this combined refuse depend on the initial moisture content of the filter cake, the ratio of fine to coarse refuse, and the particle size and moisture content of the coarse refuse. Often, these properties make it difficult to place combined refuse in a controlled manner, and they may limit its potential for use within the structural portion of an embankment. Combined refuse is sometimes mixed with combustion ash for construction of a homogeneous embankment or, in some cases, a zoned embankment (for construction of the downstream structural shell). Table 6.5 presents published geotechnical test data for combined coal refuse from several locations for a range of fines content. The strength data are based on remolded samples compacted to 95 percent standard Proctor maximum dry density, which can be difficult to achieve if the fine portion of the refuse has a high water content.

TABLE 6.4 FINE COAL REFUSE CHARACTERIZATION – SUMMARY OF AVERAGE/RANGE OF VALUES

Reference	Location	Grain Size		Atterberg Limits			Specific Gravity G_s (gm/cm^3)	Effective Shear Strength	
		Passing No. 40 Sieve (%)	Passing No. 200 Sieve (%)	LL (%)	PL (%)	PI (%)		ϕ' (degrees)	c' (psf)
Almes and Butail (1976)	PA, WV, KY, VA	64-100	36-47	20-40	NR[(1)]	<10	1.55-1.65	29-34	0
McCutcheon (1983)	OH	81	46	29	22	7	1.85	36	0
Qiu and Sego (2001)	Western Canada	90	66	40	24	16	1.94	32	200
Hegazy et al. (2004)	PA	65-100	58	31	20	11	1.52	33	230
Genes et al. (2000)	WV	NR	16-90	NR	NR	<12	1.44-2.37	23-36	0
Cowherd and Corda (1998)	NR	NR	24-91	23-39	NR	0-9	1.4-2.1	NR	NR
Huang et al. (1987)	KY, OH, PA TN, VA, WV	NR	27-95	22-44	NR	0-12	1.52-2.14	NR	NR
Busch et al. (1974,1975)	WV	50-98	10-60	34-51	NR	0-13	1.45-2.07	NR	NR
Backer et al. (1977)	UT, NM	60-100	16-98	NR	NR	NR	1.33-2.07	NR	NR
Ullrich et al. (1991)	KY, TN, OH	45-95	25-85	31-44	NR	0-31	1.8-2.5	NR	NR
Zeng and Goble (2008)	Appalachian Region	75-85	40-62	27-36	21-26	3-11	2.02-2.16	NR	NR

Note: 1. NR = not reported

6.2.3.4 Borrow Materials

Borrow materials are those soil and rock materials used in an embankment to meet specific design criteria. Borrow materials are used principally for:

- Starter dam construction
- Filters and drainage zones
- Impervious zones
- Sedimentation pond embankments
- Erosion protection
- Buttresses
- Reclamation cover

For economical designs, most borrow materials are obtained at the site or from suitable nearby mine spoil. Materials for filters, drains, and erosion protection are typically obtained from commercial sources.

TABLE 6.5 COMBINED COAL REFUSE CHARACTERIZATION

Location	Grain Size		Specific Gravity G_s (gm/cm^3)	Total Shear Strength	
	Passing No. 4 Sieve (percent)	Passing No. 200 Sieve (percent)		ϕ (degrees)	c (psf)
Pennsylvania	25-60	7-26	1.9-2.0	26-28	NR
Ohio	25-58	5-11	1.8	36-38	NR
West Virginia	18-58	4-12	2.1	32	NR
Colorado	18-54	3-18	1.8-1.9	33-38	NR

(STEWART AND ATKINS, 1982)

The available mine spoil and borrow materials in most coal mining regions of the U.S. consist either of bedrock or soils derived from bedrock. Typical bedrocks include: (1) soft shales, siltstones and claystones that weather rapidly when excavated and break down to a soil when compacted, (2) harder limestones that usually resist weathering except when exposed to acidic waters from leachates passing through pyritic coal and coal refuse, and (3) hard sandstones that often are resistant to natural weathering and attack from leachates. The soil components are typically present as alluvial deposits in valley bottoms, colluvial deposits that have accumulated toward the base of slopes, residual soils derived from surface weathering of the rock, or partially decomposed weathered rock.

Soft rock is normally suitable for the downstream portion of starter dams not critical to seepage control. The initial particle size of soft rock may prevent its use for constructing impervious zones, and its weathering characteristics usually prevent its use for drainage or erosion protection purposes. When soft rock is used, its design strength characteristics should be based on predicted future compacted or weathered condition, often as a soil. The use of limestone should usually be avoided due to its susceptibility to deterioration from leachates. Limestone can be used in situations where: (1) acidic conditions will not occur, (2) it is arranged within an embankment in a manner that assures separation from acidic leachates, or (3) when it is used in a manner that does not depend upon continued integrity as a granular material. In most coal mining regions, hard sandstone is the best material for erosion protection, filters and drainage zones.

Mine spoil tends to be highly variable in soil and rock content and particle size, and processing may be necessary. This can be accomplished by segregating over-sized fractions or fines, which can be used for other project applications. Sometimes mine spoil can be used for construction without processing of the materials (e.g., in a zoned embankment).

Recent alluvial and older river terrace deposits can vary from relatively clean sands and gravels to “dirty” soils with high contents of fine-grained soils and organic material. Material from clean sand and gravel deposits may be suitable for constructing filter or drainage zones in an embankment, but only after field investigation has determined the quantity of clean material available and laboratory testing has verified the suitability of the grain-size distribution and mineral composition for the intended purposes. Otherwise, alluvial soils typically are not desirable for use in a coal refuse disposal embankment because of the expense associated with excavation and preparation.

Colluvial soils generally consist of a combination of soils derived from fine- and coarse-grained rocks and vary from clays to primarily sandy material. Fine-grained colluvial soils may be suitable for constructing an impervious zone, while all types of colluvium are normally suitable for use as structural components for stability or as cover material to support vegetation. Because they generally have a wide grain-size distribution, colluvial soils normally are not suitable for either filter or drainage zones in an embankment.

Residual soils derived from soft rocks (e.g., shale) are normally fine-grained and suitable for either an impervious zone or the structural portion of an embankment. When available in thick deposits, residual soils can generally be excavated economically. Soils derived from sandstone are too coarse for use in impervious zones, but may be suitable for structural fill or use in drainage zones.

For any borrow material, the deposit must be explored to verify that it is available in adequate quantities to meet the design requirements of the disposal facility. This must be followed by laboratory testing to evaluate the suitability of the borrow materials/mine spoils for embankment construction. Although practically any inorganic, insoluble soil can be incorporated into an embankment when modern compaction equipment and control standards are employed, the following problems may arise:

- Fine-grained soils may have insufficient shear strength or excessive compressibility.
- Clays of medium to high plasticity may expand if placed under low confining pressures and/or at low moisture contents.
- Plastic soils with high natural moisture content may be difficult to adjust for proper moisture for compaction.
- Dispersive clays are not suitable for use in dam embankments.
- Silts may have insufficient erosion resistance.
- Stratified soils may require extensive mixing of borrow material.

Table 6.6 shows a correlation between soil classification and the engineering and design properties of compacted soils (DOD, 2005). This table can be used for evaluation of borrow materials and preliminary design of starter dams. Table 6.7 provides a correlation between soil classification and the relative desirability of soil as compacted fill material for various types of starter embankments. Table 6.8 (Sherard et al., 1963) illustrates a correlation between soil classification and engineering properties related to embankment design and constructability.

6.2.3.5 Coal Combustion Products from Power Plants

The combustion of coal at fossil fuel power plants produces fly ash and bottom ash as residual waste products. Two other products of coal combustion air pollution control technology are fluidized-bed combustion (FBC) waste and flue-gas-desulfurization (FGD) sludge. While no detailed assessment of these and other wastes from the power plants is provided in this Manual, embankments at some refuse disposal sites have been constructed by integrating power plant waste products with coal refuse. The important issues that designers should consider if these waste products are disposed at coal refuse facilities are discussed in this section.

Power plant wastes have certain properties that can be very beneficial if these materials are judiciously integrated at coal refuse disposal sites. The economic viability of disposing dissimilar refuse materials can be especially beneficial when the mining operation is near the coal burning plant. With this in view, a brief description of the engineering properties of power plant waste materials is provided in this section. For more detailed discussion and design considerations, the designer should refer to the Electric Power Research Institute (EPRI) Coal Ash Disposal Manual (DiGioia et al., 1995) and other references such as McLaren and DiGioia (1987), DiGioia and Gray (1979), Gray and Lin (1972). Additionally, research has been published (Daniel et al., 2002) on the material properties of mixtures of fly ash and coal refuse in southwest Virginia. A thorough review of the overall environmental implications of fly ash use is provided in Carlson and Adriano (1993). The USEPA (2000) performed a

TABLE 6.6 CORRELATION BETWEEN USCS CLASSIFICATION AND PROPERTIES OF COMPACTED SOILS

Group Symbol	Soil Type	Range of Max. Dry Weight (pcf)	Range of Optimum Moisture (%)	Typical Value of Compression (% of original height)		Typical Strength Characteristics				Typical Hyd. Cond. (ft/min)	Range of CBR Value	Range of Subgrade Modulus k (lbs/in^3)
				1.4 tsf = 20 psi	3.6 tsf = 50 psi	Compacted Cohesion (psf)	Saturated Cohesion (psf)	ϕ (deg)	tan ϕ			
GW	Well graded clean gravels, gravel-sand mixtures	125-135	11-8	0.3	0.6	0	0	>38	>0.79	5×10^{-2}	40-80	300-500
GP	Poorly-graded clean gravels, gravel-sand mix	115-125	14-11	0.4	0.9	0	0	>37	>0.74	10^{-1}	30-60	250-400
GM	Silty gravels, poorly-graded gravel-sand mix	120-135	12-8	0.5	1.1	–	–	>34	>0.67	$>10^{-6}$	20-60	100-400
GC	Clayey gravels, poorly graded gravel-sand-clay	115-130	14-9	0.7	1.6	–	–	>31	>0.60	$>10^{-7}$	20-40	100-300
SW	Well-graded clean sands, gravelly sands	110-130	16-9	0.6	1.2	0	0	38	0.79	$>10^{-3}$	20-40	200-300
SP	Poorly-graded clean sands, sand-gravel mix	100-120	21-12	0.8	1.4	0	0	37	0.74	$>10^{-3}$	10-40	200-300
SM	Silty sands, poorly-graded sand-silt mix	110-125	16-11	0.8	1.6	1050	420	34	0.67	5×10^{-5}	10-40	100-300
SM-SC	Sand-silt-clay mix with slightly plastic fines	110-130	15-11	0.8	1.4	1050	300	33	0.66	2×10^{-6}	5-30	100-300
SC	Clayey sands, poorly graded sand-clay mix	105-125	19-11	1.1	2.2	1550	230	31	0.60	5×10^{-7}	5-20	100-300
ML	Inorganic silts and clayey silts	95-120	24-12	0.9	1.7	1400	190	32	0.62	$>10^{-5}$	≤15	100-200
ML-CL	Mixture of inorganic silt and clay	100-120	22-12	1.0	2.2	1350	460	32	0.62	5×10^{-7}	–	–
CL	Inorganic clays of low to medium plasticity	95-120	24-12	1.3	2.5	1800	270	28	0.54	$>10^{-7}$	≤ 15	50-200
OL	Organic silts and silt-clays, low plasticity	80-100	33-21	–	–	–	–	–	–	–	≤ 5	50-100
MH	Inorganic clayey silts, elastic silts	70-95	40-24	2.0	3.8	1500	420	25	0.47	5×10^{-7}	≤ 10	50-100
CH	Inorganic clays of high plasticity	75-105	36-19	2.6	3.9	2150	230	19	0.35	$>10^{-7}$	≤ 15	50-150
OH	Organic clays and silty clays	65-100	45-21	-	-	-	-	-	-	-	≤ 5	25-100

Note: 1. All properties are for condition of “Standard Proctor” maximum density, except values of k and CBR which are for “Modified Proctor” maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from USBR data.
3. Compression values are for vertical loading with complete lateral confinement.
4. (-) indicates insufficient data available for an estimate.

(DOD, 2005)

TABLE 6.7 CORRELATION BETWEEN USCS CLASSIFICATION AND RELATIVE DESIRABILITY OF SOILS AS COMPACTED FILL

Group Symbol	Soil Type	Relative Desirability for Various Uses (1 is most desirable. 14 is least desirable)							
		Rolled Earth Fill Dams			Lining			Foundation	
		Homogenous Embankment	Core	Shell	Erosion Resistance	Compacted Earth Lining	Roadway Surfacing	Seepage Important	Seepage Not Important
GW	Well graded clean gravels, gravel-sand mixtures	-	-	1	1	-	3	-	1
GP	Poorly-graded clean gravels, gravel-sand mix	-	-	2	2	-	-	-	3
GM	Silty gravels, poorly-graded gravel-sand mix	2	4	-	4	4	5	1	4
GC	Clayey gravels, poorly graded gravel-sand-clay	1	1	-	3	1	1	2	6
SW	Well graded clean sands, gravelly sands	-	-	3, if gravelly	6	-	4	-	2
SP	Poorly graded clean sands, sand-gravel mix	-	-	4, if gravelly	7, if gravelly	-	-	-	5
SM	Silty sands, poorly graded sand-silt mix	4	5	-	8, if gravelly	5, erosion critical	6	3	7
SC	Clayey sands, poorly graded sand-clay mix	3	2	-	5	2	2	4	8
ML	Inorganic silts and clayey silts	6	6	-	-	6, erosion critical	-	6	9
CL	Inorganic clays of low to medium plasticity	5	3	-	9	3	7	5	10
OL	Organic silts and silt-clays, low plasticity	8	8	-	-	7, erosion critical	-	7	11
MH	Inorganic clayey silts, elastic silts	9	9	-	-	-	-	8	12
CH	Inorganic clays of high plasticity	7	7	-	10	8, vol.-change critical	-	9	13
OH	Organic clays and silty clays	10	10	-	-	-	-	10	14

(DOD, 2005)

detailed review of the use of coal combustion products in mining environments that supported their classification as residual waste, although they recommended continued study of disposal in deep mines and in mine backfill situations where the materials may contact groundwater.

Fly ash is a fine, silt-sized material usually ranging in diameter from 0.5 to 100 microns and consisting largely of spherical, sometimes hollow, glassy particles. Bottom ash consists of primarily coarser material with heavier particles than fly ash. It is generally angular with a porous surface. For fly ash, hydraulic conductivities have been reported in the range of 10^{-7} to 10^{-4} cm/sec and for bottom ash in the range of 10^{-3} to 10^{-1} cm/sec (DiGioia et al., 1995). Depending on the actual material, fine-grained fly ash and coarse-grained bottom ash can be used as an "impervious liner" and drainage filter, respectively, in conjunction with coarse coal refuse and other borrow materials. If fly ash is enriched with nitrogen compounds, it can sometimes be used as a supplement for vegetation growth.

There are two general types of fly ash as defined by ASTM. Class F fly ash contains less than 20 percent calcium oxide and is produced by burning bituminous or anthracite coal. Class C fly ash is produced by burning subbituminous coal or lignite. Both have pozzolanic properties, but Class F fly ash is not appreciably self-cementing. Because of the geographical distribution of coal types, Class F fly ash is principally produced in the eastern U.S., while most Class C fly ash is produced in the western U.S. Class C fly ash is self-cementing due to presence of lime and other chemical compounds. However, much of the fly ash returned to Appalachian mines does not meet either Class C or F criteria (Daniels et al., 2002).

TABLE 6.8 APPROXIMATE CORRELATION BETWEEN ENGINEERING PROPERTIES AND SOIL CLASSIFICATION GROUPS

USCS Group Symbol	Relative Hydraulic Conductivity	Probable Range of k (ft/yr)	Relative Piping Resistance	Relative Shear Strength	Relative Workability(1)
GW	Pervious	1,000 – 100,000	High	Very High	Very Good
GP	Pervious to Very Pervious	5,000 – 10,000,000	High to Medium	High	Very Good
GM	Semi-pervious	0.1 – 100	High to Medium	High	Very Good
GC	Impervious	0.01 – 10	Very High	High	Very Good
SW	Pervious	500 – 50,000	High to Medium	Very High	Very Good
SP	Pervious to Semi-pervious	50 – 500,000	Low to Very Low	High	Good to Fair
SM	Semi-pervious to Impervious	0.1 – 500	Medium to Low	High	Good to Fair
SC	Impervious	0.01 – 50	High	High to Medium	Good to Fair
ML	Impervious	0.01 – 50	Low to Very Low	Medium to Low	Fair to Very Poor
CL	Impervious	0.01 – 1	High	Medium	Good to Fair
OL	Impervious	0.01 – 10	Medium	Low	Fair to Poor
MH	Very Impervious	0.001 – 0.1	Medium to High	Low	Poor to Very Poor
CH	Very Impervious	0.0001 – 0.01	Very High	Low to Medium	Very Poor

Note: 1. Relative workability = ease of moisture-density control.

(ADAPTED FROM SHERARD ET AL., 1963)

Power plants may also generate combustion waste from FBC systems and SO_2 scrubbers by utilizing limestone. FBC waste may be pozzolanic or cementitious.

The physical and engineering properties of coal ash that could be of importance when it is used in combination with coal refuse disposal are grain size, specific gravity, density, optimum moisture content, hydraulic conductivity, shear strength, and compressibility.

Tables 6.9 and 6.10 show summaries of particle-size testing results for Class F and Class C fly ash, respectively.

TABLE 6.9 CLASS F FLY ASH

Gradation Property	Number of Samples	Mean Value (mm)	Standard Deviation (mm)	Coefficient of Variation
D_{85}	84	0.079	0.063	0.800
D_{50}	84	0.023	0.015	0.669
D_{15}	84	0.0075	0.0048	0.648

(MCLAREN AND DIGIOIA, 1987)

TABLE 6.10 CLASS C FLY ASH

Gradation Property	Number of Samples	Mean Value (mm)	Standard Deviation (mm)	Coefficient of Variation
D_{85}	17	0.063	0.020	0.317
D_{50}	17	0.022	0.011	0.500
D_{15}	17	0.0084	0.0082	0.976

(McCLAREN AND DIGIOIA, 1987)

A review of available data (McLaren and DiGioia, 1987) shows that fly ash is a relatively uniform, silt-sized material with a specific gravity slightly lower than most natural soils. Compaction of fly ash is moisture dependent, but the range of optimum moisture contents is greater than that of natural silts and silty clays. The maximum dry and wet densities of compacted fly ash are somewhat less than typical values for natural soils, which makes fly ash useful as a light-weight structural fill.

Of importance when fly ash is used in structural zones of a disposal facility is shear strength. The effective angle of internal friction of fly ash and FBC combustion waste varies with the degree of compaction, but generally ranges from 25 to 40 degrees. Class F fly ash is non-cohesive, and while it may appear to be cohesive when partially saturated, this effect is completely lost when the material is either dried or saturated. In contrast, Class C fly ash can develop considerable cohesive shear strength due to cementitious reactions. This cohesion is the dominant factor in the shear strength of Class C fly ash. Similar to fly ash, the shear strength of bottom ash varies with the degree of compaction. The effective angle of friction for bottom ash in a loose state can vary from 38 to 42.5 degrees, with an average of about 41 degrees. The shear strength of SO_2 sludge varies significantly with the solids content and the amount of stabilizing agent added. The strength of a stabilized sludge is comparable to dense sand and gravel, while the strength of unstabilized sludge is similar to that of loose sand. The angle of friction for stabilized sludge ranges from 38 to 51 degrees depending on the solids content and amount of additives. The angle of friction ranges from 20 to 30 degrees for unstabilized sludge.

Research into the beneficial reuse of fly ash mixed with coarse coal refuse in southwest Virginia was performed by Daniels et al. (2002). Table 6.11 shows variations of maximum compacted dry density, shear strength and hydraulic conductivity reported by them for mixtures of fly ash and coal refuse.

TABLE 6.11 FLY ASH/COAL REFUSE MIXTURE PROPERTIES

Fly Ash Mix Ratio (%)	Maximum Dry Density γ_{dmax} (lb/ft^3)	Effective Angle of Internal Friction ϕ' (degrees)	Effective Cohesion c' (lb/ft^2)	Hydraulic Conductivity k (cm/sec)
0	125	39	0	2.86×10^{-3}
8	123	37.7	0	1.01×10^{-3}
16	120	37	0	2.56×10^{-4}
24	119	37	0	1.71×10^{-4}
32	117	37	0	7.88×10^{-5}
100	85	37	0	5.78×10^{-5}

(DANIELS ET AL., 2002)

Research conducted on coal refuse and Type F fly ash from southern Illinois (Kumar et al., 2001) indicated an increase in strength for mixtures containing up to 15 percent ash. There was a tendency for strength to decrease for mixtures with a higher percentage of ash.

Mixing procedures to blend fly ash with refuse material should be developed based on the strength or stabilization requirements for the embankment. In many cases, the primary beneficial use is associated with moisture control, and for this usage blending with spreading equipment on the embankment surface is acceptable. Where enhancement of the strength of the refuse material is a requirement, or in cases where fly ash amendments are introduced to control acid generation, greater effort to mix or blend the materials may be required or beneficial. Additional discussion concerning blending of amendments is presented in Section 11.5.6.

6.2.4 Scheduling

The procedure for scheduling construction of a disposal facility embankment differs from that used for most other types of constructed embankments for several reasons:

- Refuse disposal occurs continually over many years.
- Disposal must occur on a year-round basis, regardless of weather conditions.
- The rate of refuse disposal is not only determined by the needs of embankment construction, but also by the rate of mining, the quality of the seam, market changes, and scheduled and unscheduled work interruptions.

The many variables affecting refuse disposal scheduling limit the ability to provide a detailed flow chart that accounts for all construction events that could occur during the operational period of a disposal facility. However, the following are questions to be answered in developing the design and establishing a general construction schedule for existing and new disposal facilities:

- What is the expected operational period of the disposal facility?
- What volumes of coarse, fine and combined refuse are anticipated from the prepara-

tion plant on an annual basis, and are there reasons to believe that the relative proportions may change?

- What types of borrow material are available, and must they be developed prior to covering the borrow site with refuse?
- What embankment configurations would enable critical sections to be constructed from available materials during periods of the year most conducive to controlled construction?
- What measures to address potential environmental impacts and protection of surface and groundwater will be required?
- Should hydraulic structures (e.g., spillways, decants, diversion systems) be constructed in stages corresponding to disposal rates?
- Can soil and rock materials excavated for hydraulic structures be used as borrow for embankment construction?
- What measures will be required to economically abandon the disposal facility at the end of its expected operational period?
- If for some reason the disposal facility must be abandoned earlier than expected, can that abandonment be reasonably accomplished?
- Will the disposal rate allow portions of the embankment to be completed in stages so that exposed refuse and eventual abandonment costs are minimized?
- If the rate of refuse production or the operational period of the disposal facility is extended beyond initial projections, can the facility be reasonably expanded?
- Will the timing of future surface or underground mining in the vicinity of the facility impact the design?

The designer must recognize that these schedule-related questions may not address all of the potential issues. However, it is prudent to consider each of the above questions when designing a coal refuse disposal facility.

6.3 DESIGN CONSIDERATIONS

This section presents the basic design considerations for coal refuse embankments and earthen dams. Embankments with and without impoundments are considered separately for two reasons. First, an impounding embankment generally has a greater hazard potential because the impounded water can cause damage for a substantial distance downstream from the disposal facility in the event of a failure. Second, impounded water can contribute directly to a failure if the embankment is improperly designed and/or constructed.

The examples discussed in this section are based on the assumption that coarse refuse is the major structural component of embankments for disposal of coarse and fine coal refuse. Mine spoil, soil and rock borrow materials, which are obtainable at most disposal facility sites, could also be used for the starter dam and in the structural portions of the embankment. The designer should refer to Sherard, et al. (1963) and USBR (1992a; 1998) for additional discussion of design principles for dams and embankments composed primarily of soil and rock materials.

6.3.1 Impounding Embankments

A coal refuse impounding embankment is generally designed based upon the same principles as earthen dams, with the exception that refuse materials are used to the maximum extent possible. Two major aspects of coal refuse impounding embankment construction are: (1) the starter dam for initial disposal of fine refuse and (2) the embankment raising methodology used for long-term refuse disposal. The starter dam is typically constructed with borrow materials, while subsequent crest raisings generally utilize coarse coal refuse.

The borrow material available at a site may range from fine to coarse soils to coarse coal refuse, or rocks and soil from mine spoils and highwall cuttings. For economy, suitable materials at or near the disposal site should be used for starter dam construction. Depending on the type of material available at the site, the following types of starter dam may be designed:

- Homogeneous Embankment
- Zoned Embankment

A homogeneous embankment is generally constructed in situations where the borrow materials vary little in hydraulic conductivity or soil type. A zoned embankment is constructed where two or more types of materials are available for embankment construction. Rockfill, when large quantities of rocks are available from the mine spoil or from spillway construction, can also be used as the stability portion of a zoned embankment.

The principal geotechnical considerations in designing an impounding embankment are:

- <u>Seepage control</u> – internal drainage system, impervious zone, and foundation treatment
- <u>Slope and foundation stability</u> – static slope stability, end-of-construction stability, seismic slope stability and deformation, sloughing and erosion
- <u>Drainage structures</u> – principal and emergency spillways, conduits and surface drainage structures
- <u>Underground mine workings</u> – stability, subsidence and breakthrough potential, sealing of mine openings and boreholes, and potential infiltration into underground mine workings

For earth dams, foundation support, stability and seepage control are important design considerations. Geologic and geotechnical investigations should be performed to identify potentially unstable soils that are incapable of sustaining embankment loadings or susceptible to adverse impacts from seepage. Slope failure and piping (internal erosion) are the most common types of embankment failure associated with seepage. In addition to embankment slope and material strength and unit weight, the stability of an embankment is a function of the depth of the saturation level (seepage phreatic surface) below the embankment face, regardless of the volume of seepage through the embankment. If the phreatic surface rises above the level assumed in the design, embankment stability can decrease to the point where failure occurs. Piping is a process in which particles are carried out of an embankment with seepage, creating voids. This can lead to failure as the voids progressively extend farther back into the embankment as more material is removed and more concentrated seepage flow occurs. Installation of an internal drainage system within the embankment is specifically intended to address seepage-type failures and is fundamental to the selection of the embankment configuration. The subsequent discussions of basic embankment types repeatedly emphasize the importance of seepage control. Cedergren (1989) is an important reference for analyzing seepage conditions in embankments and foundations.

In addition to controlling seepage for embankment stabilization, liner systems may also be needed for mitigation of potential environmental impacts to the groundwater and surface water. Impervious liners composed of fine-grained borrow materials, geomembranes or geosynthetic clay liners can be a critical component of disposal facility design.

Uncontrolled seepage into underground mine workings can affect embankment safety. If the water pressure in an impoundment has the potential for breaking through the overburden into abandoned entries or through zones of soft, weathered rock, the rapid release of water or slurry into a mine could trap mine personnel and equipment and lead to undesirable environmental conditions. Such a situation must be prevented. Even in abandoned workings, the resulting water flow may endanger the population downstream of uncontrolled mine discharge points and cause undesirable environmental conditions. Guidance related to evaluation of breakthrough potential is presented in Section 8.5.

Analysis of seepage, slope stability and structure foundations is generally performed in detail only after the embankment configuration has been selected and the subsurface exploration and laboratory testing programs have been completed. These analyses are introduced in this section and are discussed in more detail in Section 6.6.

6.3.1.1 Homogeneous Embankments

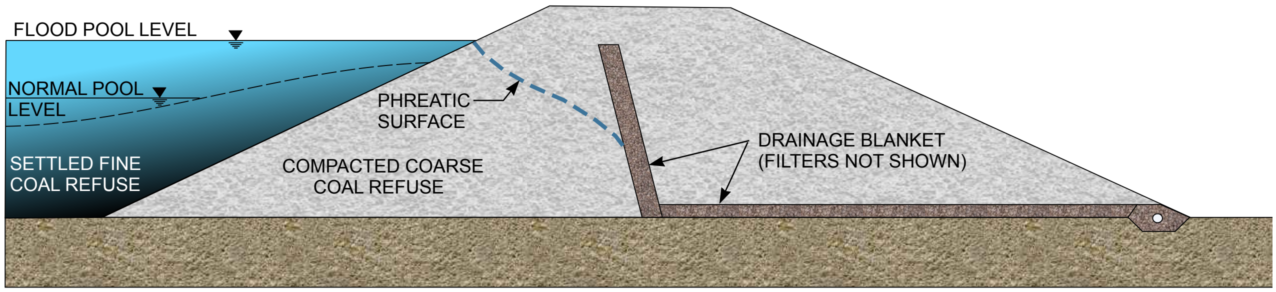
A homogeneous embankment is constructed of only one material; however, to control seepage and saturation of the embankment, a granular internal drain is typically incorporated into the cross section. Homogeneous embankments constructed for coal refuse sites with various types of internal drains are illustrated in Figure 6.1. In all cases, the purpose of the drain is: (1) to maintain stability by keeping the phreatic surface low and (2) to control seepage as it leaves the embankment to minimize the potential for piping. The drainage system may consist of one or more filter zones of intermediate grain-size material to mitigate potential conveyance of embankment material into the collection zone. Selection of the relative gradations of adjacent zones of material and the use of geotextiles to prevent finer material from piping into a coarser downstream zone is discussed in Section 6.6.2. If possible, drainage material should be selected to act as both the filter and the collection zone to avoid the higher costs and more difficult construction associated with placing multiple layers. Selection of the type of internal drainage system is normally based upon the fine refuse and flood pool levels, embankment configuration, and material characteristics, including anisotropy. Table 6.12 summarizes the advantages and disadvantages of the internal drains that are illustrated in Figure 6.1.

When layers of coal refuse are placed during refuse embankment construction, the top surface is often broken into smaller-grained, less-permeable material by the movement of equipment and the effects of weathering. The materials beneath the top surface retain their original grain-size distribution and greater hydraulic conductivity. As a result, anisotropic conditions can develop, leading to an embankment hydraulic conductivity that is greater in the horizontal direction than in the vertical direction. Thus, the effectiveness of a horizontal drainage blanket or toe drain is reduced if the anisotropy is large and the height of the embankment is significant. A chimney drain, or other types of drains that intercept horizontal seepage planes, may be more effective. Section 6.6.2 presents guidance for seepage analyses for the design of internal drains, including the determination of drain dimensions.

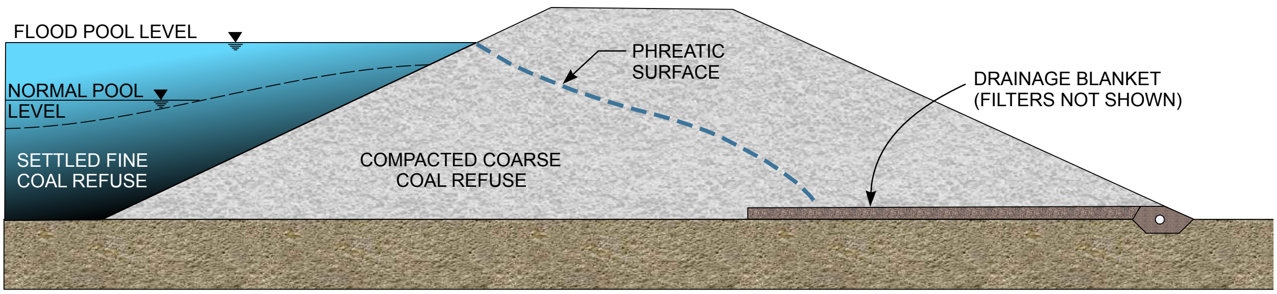
At many coarse refuse embankment dams, the fine coal refuse deposited in the impoundment, creates a “delta” or “beach” on the upstream slope that typically restricts seepage, provided that the normal pool is maintained upstream of the delta. Economies in internal drain construction can be achieved by evaluating the hydraulic conductivity characteristics of the fine refuse material, as well as available borrow material, and using zoned embankment design. For homogeneous embankment dams used for fresh water supply, some savings can also be gained by using outlet drains for discharging the water from the base of a chimney drain, which will eliminate the need for a granular blanket extending beneath the entire downstream portion of the embankment. This alternative is illustrated in Figure 6.1d.

6.3.1.2 Zoned Embankments

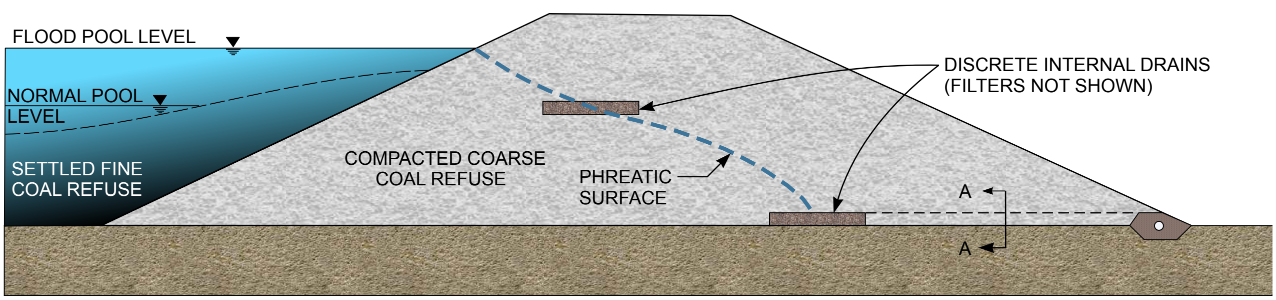
Coarse coal refuse and mine spoil are generally not sufficiently fine grained to keep seepage at a low level. If the design of an embankment requires that seepage be minimized or it is desired to lower the saturation level in the downstream portion of the embankment, a less pervious zone within the



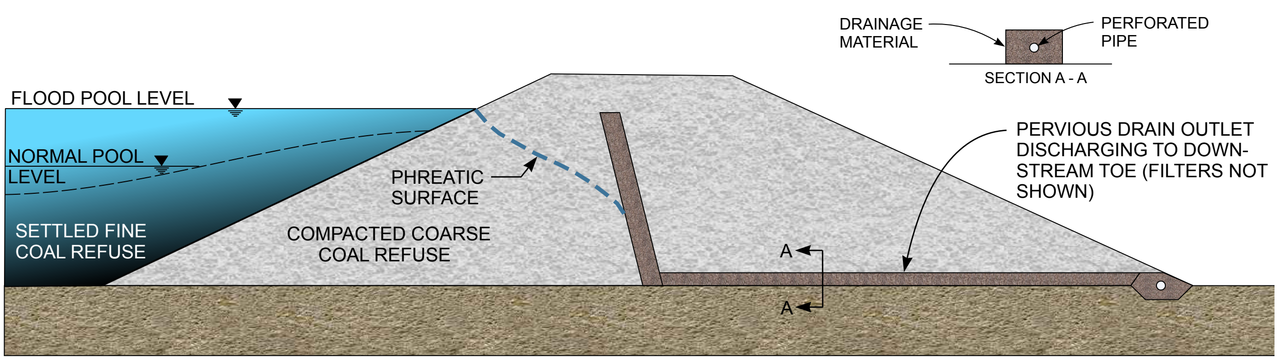
6.1a VERTICAL OR STEEPLY SLOPING CHIMNEY DRAIN



6.1b HORIZONTAL BLANKET DRAIN



6.1c DISCRETE INTERNAL DRAIN



6.1d CHIMNEY DRAIN WITH OUTLET DRAIN EXITS

FIGURE 6.1 DRAIN CONFIGURATIONS FOR HOMOGENEOUS EMBANKMENTS WITH IMPOUNDMENTS

TABLE 6.12 ADVANTAGES AND DISADVANTAGES OF INTERNAL DRAIN TYPES

Internal Drain Type	Figure No.	Advantage(s)	Disadvantage(s)
Steeply Sloping Chimney Drain	6.1a	Positive seepage interception and collection system	Expensive and difficult to construct; requires careful planning and stringent construction control to connect with future stages.
Horizontal Blanket Drain	6.1b	Simple construction	Ineffective in high anisotropy conditions.
Discrete Internal Drain	6.1c	Relatively inexpensive and independent of future embankment raising	Partial seepage interruption; effectiveness depends on anisotropy.
Chimney Drain and Outlet Sections	6.1d	Positive seepage interception	Expensive and difficult to construct.

embankment may be needed. A zoned embankment consists of multiple material zones, generally including an impervious (or low-hydraulic-conductivity, fine-grained material) core or upstream zone of limited width and additional zones of coarse material that provide strength and erosion resistance. Employing such a zoned embankment concept can reduce material requirements for internal drainage structures and structural embankment zones.

At some coal refuse disposal sites, two zones are incorporated into the embankment cross section – a relatively impervious soil zone in the upstream section and coarse material such as mine spoil, granular borrow material or coarse coal refuse in the downstream section. If controlling the volume of seepage is not of primary importance, zoned embankments can be designed with the object of lowering the phreatic surface in the downstream face of the embankment. The primary design consideration in such cases is that the hydraulic conductivity of the downstream zone be sufficiently high to discharge the water seeping through the core or upstream zone without an elevated phreatic surface.

Figure 6.2 illustrates configurations of zoned embankments most commonly used in dam construction. Figures 6.2a and 6.2b show a vertical core and a sloping core, respectively. Figure 6.2c illustrates a zoned embankment consisting of finer soil in upstream portion of the dam and coarse material in the downstream portion. An upstream zone may be preferred for slurry impoundments, provided erosion from pool-level fluctuations or runoff is not significant or can be controlled. Alternately, zoned coarse refuse dams, in which the upstream zone receives more compactive effort to increase material breakdown and to lower hydraulic conductivity, may be specified. However, many coarse refuse dams are not zoned, taking advantage of the fine coal refuse deposited in the impoundment that may be as effective as zoning in limiting seepage, provided that: (1) the impoundment pool is maintained at a low level and does not surcharge the upstream embankment face and (2) the response of the phreatic surface to increased pool levels as a result of storm runoff will not compromise embankment stability.

A description of important features related to impervious zone and shell design for zoned embankments is provided in the following text.

Impervious Zone Design

Although small amounts of seepage are typically present in the low-hydraulic-conductivity portion of an embankment, these embankment components are generally referred to as impervious zones. For design of impervious zones, the primary considerations are the type of impervious zone and the thickness and material used for their construction. The upstream zone of a zoned embankment may serve a similar purpose, particularly for coal refuse impoundments.

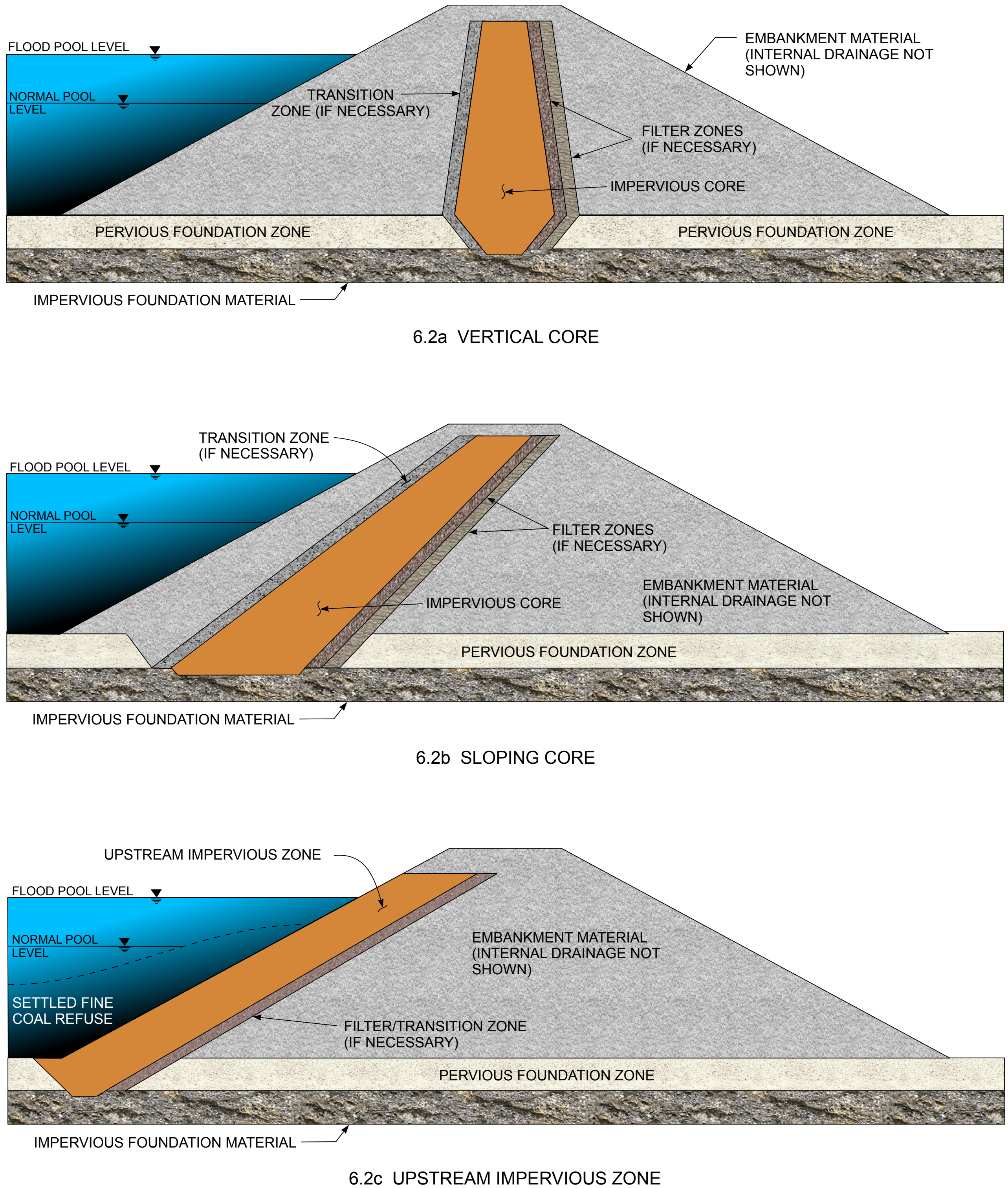


FIGURE 6.2 ZONED EMBANKMENTS WITH IMPOUNDMENTS

Impervious Zone Type

Both vertical and sloping impervious zones have advantages, as described in Table 6.13. The selection of the type of zoning of the material must be determined on an individual site basis. Valuable additional discussion of the design of embankments with impervious zones is provided in USBR (1992a).

TABLE 6.13 RELATIVE ADVANTAGES OF VERTICAL AND SLOPING EMBANKMENT CORES

Vertical Core	Sloping Core
1. Higher confinement pressure, more uniform settlement with increased embankment load, and shear strength is not as critical as with a sloping core.	1. The main downstream portion of the embankment can be constructed first and the impervious core placed later without disrupting construction operations. This advantage allows the downstream portion of the embankment to be constructed year round, even if controlled construction of the core can be done only during short, good-weather periods.
2. Higher pressure is present between the impervious core and the foundation, providing additional protection against leakage along the contact surface.	
3. For a given volume of impervious soil material, the thickness of the vertical core will be slightly greater than that of sloping core.	2. Filter layers between the core and the upstream and downstream portions of the embankment can be made thinner and constructed more easily.
4. If it becomes necessary to grout the foundation after the embankment has been raised to a significant height, the grouting can be conducted from the crest of the embankment through the core and directly into the foundation, as opposed to a sloping core, where the tie between the core and the foundation is beneath the impoundment.	3. The core construction can be staged easily if the embankment is expanded by the downstream staged construction method.
5. A vertical core is not subject to damage from sloughing or erosion of the upstream toe.	

Impervious Zone Thickness

The impervious zone thickness is normally governed by practicalities (Sherard et al, 1963), including: (1) tolerable seepage volume, (2) thickness that will permit proper construction, (3) type, quality and cost of available low-hydraulic-conductivity material, (4) type, quality and cost of available material for any filter layers between the impervious zone and the adjoining downstream soil, and (5) the quantity and quality of available soil/rock for the shell. While subsequent research and testing has provided refinement in the design of impervious zone thickness, the following criteria developed for water retention dams (USBR, 1992a; McCook, 2002) may serve as preliminary guidelines for acceptable impervious zone thickness for other impounding embankments, recognizing that other dimensions/configurations may be suitable pending seepage and stability analyses:

- Cores with a thickness greater than 30 percent of the depth of water head have proven satisfactory for many dams under diverse conditions; for a starter dam, a core of this thickness will probably be adequate for many types of impervious core material and embankment heights.
- Cores with a thickness of 15 to 20 percent of the depth of water head are considered thin, but if adequately designed and constructed filter layers are used, they will probably be satisfactory under most circumstances.
- Cores with a thickness of less than 10 percent of the depth of water head should be considered only in circumstances where a large leak through the core would not lead to embankment failure or unacceptable environmental conditions; very carefully designed and constructed filter layers should be considered.

Impervious zones of limited width, governed by material properties and availability as well as construction practicalities, have proven advantageous in limiting seepage at starter dams and slurry impoundments. As discussed above, the deposition of fine refuse slurry may also be effective at limiting seepage provided that: (1) the impoundment pool is maintained at a low level and (2) the response of the phreatic surface to increased pool levels as a result of storm runoff will not compromise embankment stability.

Impervious Zone Material

Impervious zone material is usually selected based on specific requirements for controlling seepage and the availability of suitable material at the site. Typically, fine-grained soil is used for such construction. The potential for failure resulting from loss of impervious zone material and leakage caused by cracking or differential slippage within the impervious zone will influence the design, the materials used and the construction procedure. The possibility of excessive leakage due to cracking is a particularly important consideration for embankments on soft foundation material, in areas susceptible to subsidence, or in regions of high seismic activity. Brittle soil behavior and cracking problems often can be minimized by placing the impervious zone material at a higher than optimum moisture content. Figure 6.3 provides a classification of materials according to resistance to piping and cracking. If foundation settlements are expected to be high, a suitable internal drainage layer should be placed immediately downstream of the impervious zone to control seepage resulting from possible cracking.

Dispersive clay soils have a preponderance of sodium cations in their pore water in contrast to most clays, which have a preponderance of calcium and magnesium cations in their pore water. A hole through a dispersive clay will increase in size as water flows through (due to the breakdown of the soil structure), whereas a hole in a non-dispersive clay will remain essentially constant in size. Dispersive clays should not be used in dam construction because they are extremely susceptible to piping. The crumb test (ASTM D 6572) can be conducted in the field or laboratory and may indicate if soils are dispersive. The dispersion potential can most reliably be determined using the pinhole test (ASTM D 4647). The dispersion potential of clay for several ranges of measured dispersion is provided in Table 6.14 (Sherard et al., 1976).

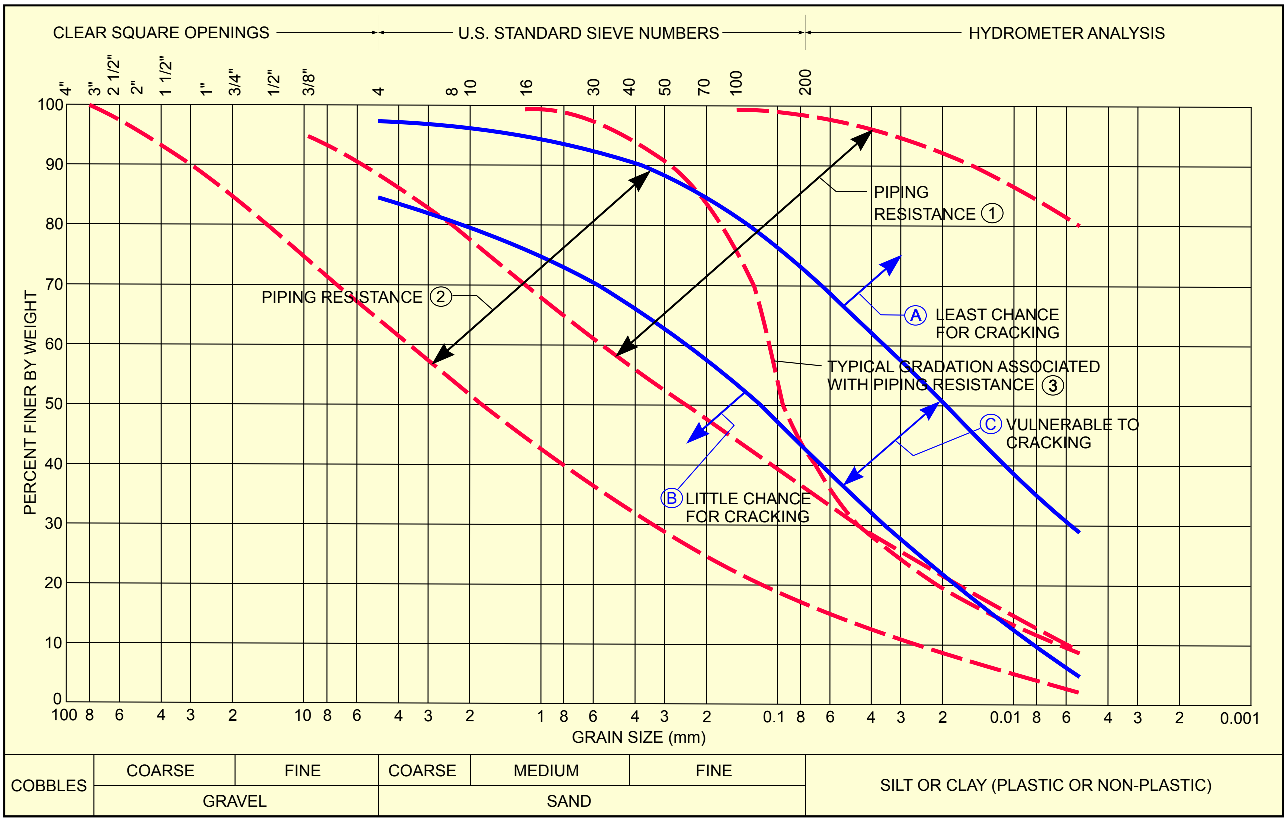
Design of filters for impervious soils used for the core can be critical for the downstream interface with the shell, but is generally less critical for the upstream interface for a coal refuse impoundment because reservoir fluctuations are minimal. Sherard et al. (1985) and McCook (2002) address filter criteria and hydraulic gradient issues associated with impervious soils in dams; this topic is further discussed in Section 6.6.2.

Shell Design

Embankment shell zone material typically is selected from granular material at the site. Coarse coal refuse, mine spoil and rock excavated from the construction of water drainage systems or haul roads are generally suitable for shell construction. The earthfill portion (upstream zone or core) may be zoned or protected by graded filter zones, as discussed in Section 6.6. The use of rockfill is governed by economy and structural stability in addition to slope protection. If a plentiful supply of suitable rock is available at low cost, it is often possible to steepen the slopes of the adjoining earthfill portion by providing additional free-draining rock on the downstream face, resulting in added stability. With careful planning and design, the rockfill section of the starter dam can be utilized as a toe drain for future expansion of the facility by the upstream method of construction. However, this may pose a design constraint if the expansion is by the downstream method.

6.3.1.3 Foundation Seepage Control

A very important consideration in the design of an impounding embankment is the control of seepage through the underlying foundation materials. Unlike a water storage reservoir, there is little need to retain water in a slurry disposal facility. Therefore, minimizing seepage through the foundation



CATEGORY	MATERIAL	CHARACTERISTICS
PIPING RESISTANCE 1	CL AND CH WITH PI>15, WELL GRADED SC WITH PI>15.	GREATEST RESISTANCE TO PIPING. SMALL AND MEDIUM CONCENTRATED LEAKS WILL HEAL THEMSELVES. EMBANKMENT MAY FAIL AS A RESULT OF SLOWLY PROGRESSIVE PIPING CAUSED BY LEAK OF ABOUT ONE-HALF CFS.
PIPING RESISTANCE 2	CL AND ML WITH PI<15, WELL GRADED SC AND GC WITH 7<PI<15.	INTERMEDIATE RESISTANCE TO PIPING. SAFELY RESISTS SATURATION OF LOWER PORTION OF DOWN-STREAM SLOPE INDEFINITELY. MAY FAIL EVENTUALLY AS A RESULT OF EROSION CAUSED BY A SMALL CONCENTRATED LEAK OR BY PROGRESSIVE SLOUGHING. IF A LARGE LEAK DEVELOPS, PIPING CAUSES FAILURE IN A SHORT TIME.
PIPING RESISTANCE 3	SP AND UNIFORM SM AND ML WITH PI<7.	LEAST RESISTANCE TO PIPING. USUALLY FAILS IN A FEW YEARS AFTER FIRST RESERVOIR FILLING IF SEEPAGE IS ABLE TO BREAK OUT ON DOWNSTREAM SLOPE. SMALL CONCENTRATED LEAK ON DOWN-STREAM SLOPE CAN CAUSE FAILURE IN A SHORT PERIOD OF TIME. HIGH DENSITY FROM COMPACTION INCREASES RESISTANCE SIGNIFICANTLY.
CRACKING RESISTANCE A	CH WITH D_{50}<0.02MM AND PI>20.	HIGH POST-CONSTRUCTION SETTLEMENT, PARTICULARLY IF COMPACTED DRY. HAS SUFFICIENT DEFORM-ABILITY TO UNDERGO LARGE SHEAR STRAINS FROM DIFFERENTIAL SETTLEMENT WITHOUT CRACKING.
CRACKING RESISTANCE B	GC, SC, SM, SP WITH D_{50}>0.15MM.	SMALL POST-CONSTRUCTION SETTLEMENT. LITTLE CHANCE FOR CRACKING UNLESS POORLY COM-PACTED AND LARGE SETTLEMENT IS IMPOSED ON EMBANKMENT BY CONSOLIDATION OF THE FOUNDATION.
CRACKING RESISTANCE C	CL, ML AND SM WITH PI<20, 0.15MM>D_{50}>0.02MM.	MEDIUM TO HIGH POST-CONSTRUCTION SETTLEMENT AND VULNERABLE TO CRACKING. SHOULD BE COMPACTED AS WET AS POSSIBLE CONSISTENT WITH STRENGTH REQUIREMENTS.

(DOD, 2005)

FIGURE 6.3 RESISTANCE OF CORE MATERIALS TO PIPING AND CRACKING

is not as critical to the design as it is for storage reservoirs. Where seepage occurs, it must be controlled such that it does not adversely affect the safety of the embankment or result in environmental impacts. The foundation conditions likely to be encountered beneath the starter dam are: (1) pervious foundation, (2) impervious foundation, or (3) impervious stratum at the surface underlain by a pervious stratum. An additional foundation seepage concern at some coal refuse facilities is the potential for fracturing due to subsidence of the ground surface above underlying mines.

Pervious foundations may consist of boulders, gravels, sands or mixtures thereof. For such foundations, measures to minimize seepage quantity and to provide controlled seepage discharge are

TABLE 6.14 CLAY DISPERSION POTENTIAL

Percent Dispersion[1]	Dispersive Tendency
Over 40	Highly Dispersive (do not use)
15 to 40	Moderately Dispersive
0 to 15	Resistant to Dispersion

Note: 1. The ratio between the fraction finer than 0.005 mm in a soil-water suspension that has been subjected to a minimum of mechanical agitation and the total fraction finer than 0.005 mm determined from a regular hydrometer test times 100.

(SHERARD ET AL., 1976)

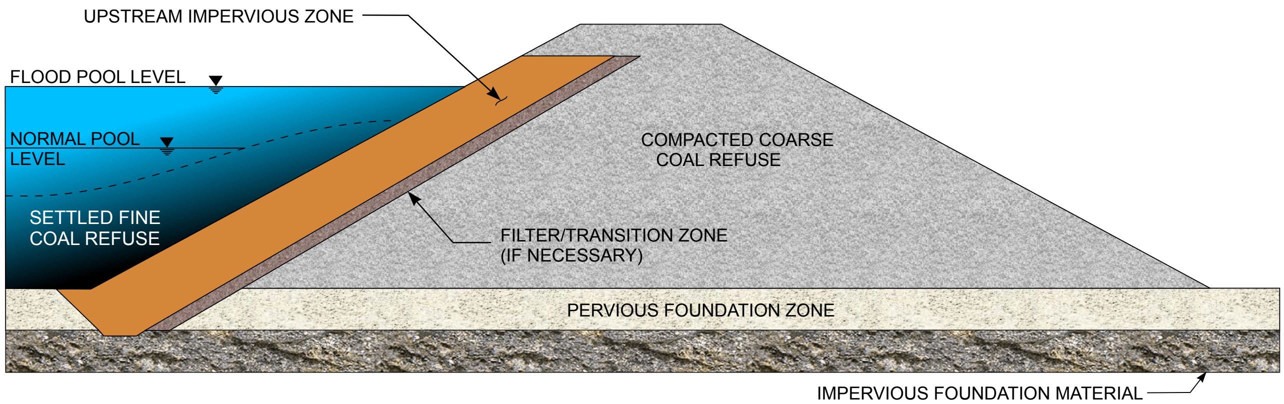
normally required. Control measures may include low-hydraulic conductivity barriers (e.g., cutoff trench and backfill) to decrease or virtually stop seepage, or a collection system can be provided beneath the downstream portion of the embankment to control the discharge of seepage. If seepage control and/or collection systems are not intended to be employed, the safety (including the potential for piping of foundation or embankment materials) and environmental ramifications should be carefully evaluated, and suitable measures should be employed to monitor pore pressures, if necessary.

The most common methods used for controlling foundation seepage are construction of a low-hydraulic-conductivity cutoff through the pervious foundation material and construction of an impervious blanket extending far enough upstream to sufficiently restrict the flow. In some situations, construction of an impermeable liner beneath the impoundment may also be used to address foundation seepage control. These methods are illustrated in Figure 6.4. Normally, if the pervious foundation material is thin and excessive groundwater problems due to excavating in the valley bottom are not anticipated, the low-hydraulic-conductivity cutoff is the least expensive method. Important considerations in the design and construction of seepage cutoffs are presented by Sherard et al. (1963) and USBR (1987a, 1992a). Procedures for designing an impervious blanket are presented by Cedergren (1989).

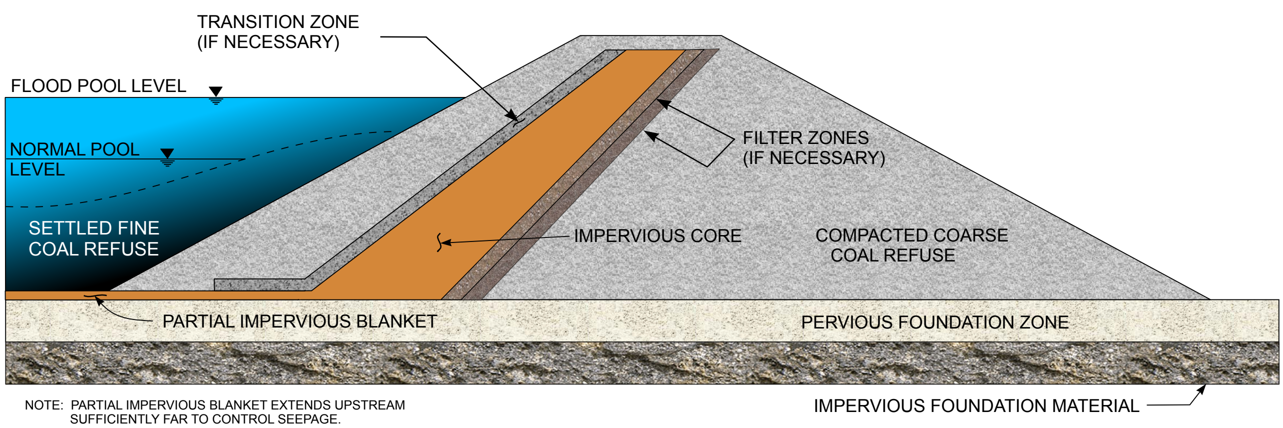
Where seepage through the foundation is allowed to occur, a collection system is almost always provided. The two major negative effects of allowing seepage to occur are: (1) a decrease in the factor of safety against instability of the embankment due to high pore pressures in the foundation and (2) the potential for piping in the foundation. If the foundation soil is not stratified in the horizontal direction, seepage control can be provided by a horizontal blanket drain (Figure 6.1a). This method normally requires analysis of the path of the seepage and assurance during placement of the blanket that it is directly tied to underlying pervious material. Seepage is collected at the downstream end of the blanket and discharged at a predetermined location near the valley bottom.

Other methods of controlling seepage through a pervious foundation include a deep drainage trench constructed near the toe of the embankment and a relief well system, as discussed in greater detail in Section 6.6.2.3.4. Measures associated with the design and management of the impoundment and clarified water level can also aid in controlling seepage. In some cases, impoundment cells for clarified water are developed in the upstream portion of the impoundment. The deposition of fine refuse and resulting longer seepage path aids in restricting seepage beneath the downstream embankment.

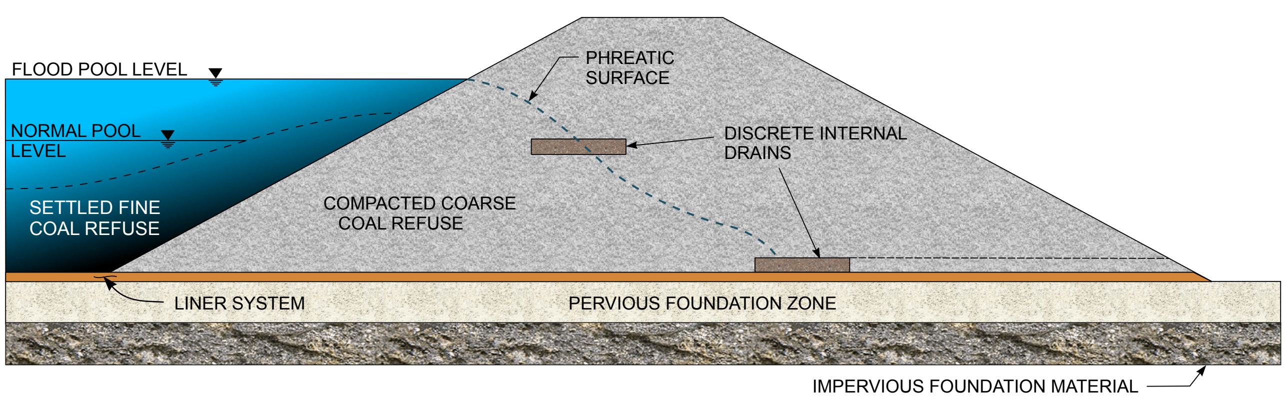
Impervious foundations typically consist of massive rock and predominantly clayey soils. When an impoundment is to be constructed upon impervious foundation materials, seepage beneath the embankment is not a major design consideration if a proper seal is placed between the embankment and the foundation. Methods of foundation preparation that effectively create a seal and references to supplemental technical publications on grouting and rock preparation are presented in Chapter 11.



6.4a PERVIOUS FOUNDATION CUTOFF WITH ZONED EMBANKMENT



6.4b IMPERVIOUS BLANKET



6.4c IMPERVIOUS LINER WITH HOMOGENEOUS EMBANKMENT

FIGURE 6.4 SEEPAGE CUTOFFS FOR PERVIOUS FOUNDATIONS

For disposal facilities where impoundments are constructed upon an impervious foundation, the National Coal Board (1970 and 1972) has suggested an effective concept for allowing controlled drainage from settled slurry. Granular drainage and filter material is placed beneath the impoundment area prior to filling. The drainage and filter material transition to pipes that pass under the embankment to a downstream collection system or to a discharge outlet. In some cases, to assure complete drainage, this zone is constructed over the upstream face of the embankment. D'Appolonia (1988) conducted OSM-sponsored research and implemented a design for an impoundment internal drain structure to improve consolidation of the fine coal refuse and to intercept seepage before it enters the coarse refuse embankment, thus mitigating acid mine drainage potential. This concept is illustrated in Figure 6.5. Another method to improve drainage and consolidation of fine coal refuse is installation of wick drains.

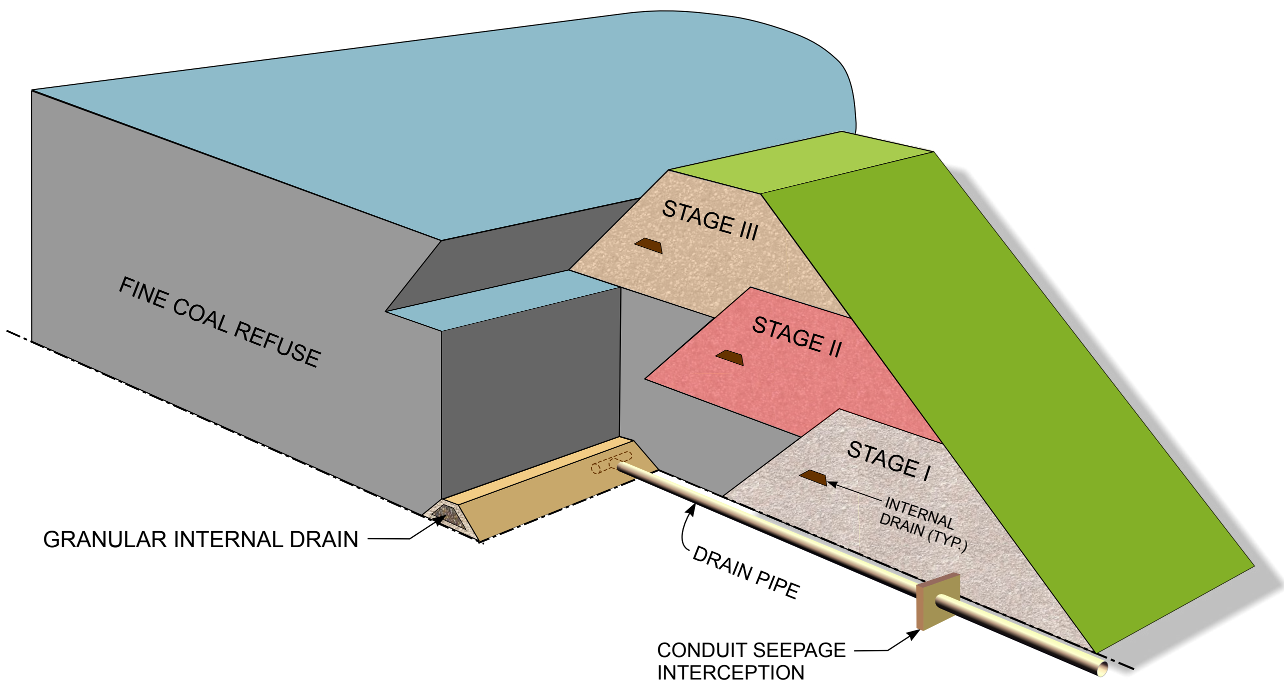


FIGURE 6.5 IMPOUNDMENT INTERNAL DRAIN CONCEPT

For the case of an impervious stratum at the surface overlying more pervious strata below, there is potential for high pore-water pressure to occur downstream of the dam. This may cause blow outs, boiling, piping or instability at or beyond the downstream toe. An upstream cutoff trench, deep drainage using relief wells or construction of berms should be considered in such cases.

6.3.2 Non-impounding Embankments

Non-impounding embankments require many of the same design and construction considerations as impoundments. Careful geotechnical investigation can: (1) identify potentially unstable soils that are incapable of sustaining embankment loadings and (2) decrease the probability of groundwater impacts by identifying certain site characteristics that should either be avoided or recognized during the design phase. Safety may become a major design factor when:

- The disposal facility is located in areas where failure would have a high possibility of taking lives and could seriously damage infrastructure or buildings.
- The facility is located immediately above a stream and significant movement of the embankment could block or restrict the flow, possibly creating a temporary impoundment that could release a flood wave upon breaching of the sloughed material.

- The facility is constructed across a large valley with a significant watershed or it may temporarily impound water during some stage of development, despite the presence of drainage systems.

Embankments should not impede drainage, and cross-valley configurations should be avoided because they can retain water and be subject to classification by MSHA as an impoundment. MSHA (2007) presents factors to be considered in this regard and also cautions that reclamation at abandonment can require significant regrading to address drainage concerns.

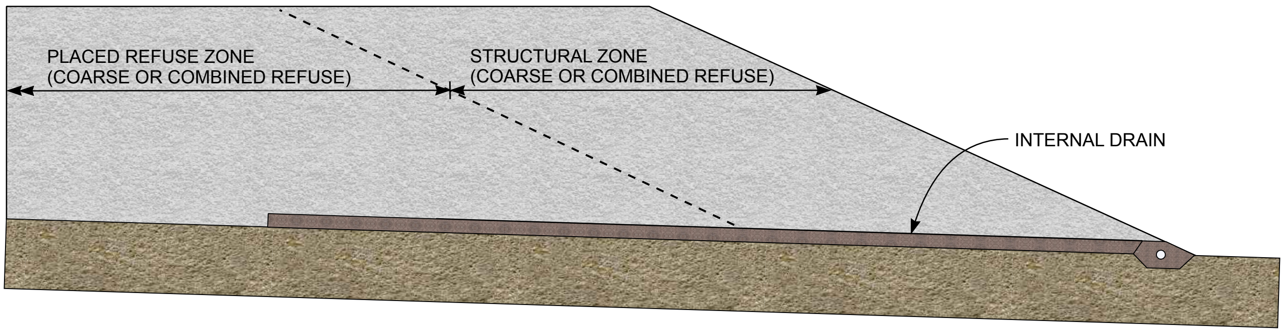
The following discussion of important design considerations for non-impounding embankments does not specifically address cases where a high safety hazard may exist. The designer must recognize special safety hazards and formulate the investigation program and analyses accordingly. To properly plan and design a non-impounding coal refuse embankment, the following questions should normally be answered:

- Considering the required volume and the disposal facility geometry, what is the smallest quantity of constructed material that could form the critical downstream structural portion of the embankment with assured stability?
- If the embankment were to temporarily impound water, could it lead to pore-water pressure conditions that are hazardous to the stability of the embankment?
- Is an impervious liner system needed in order to limit seepage or infiltration of the groundwater?
- Should an internal drainage system be placed under critical portions or all of the disposal facility to control saturation for stability purposes or to collect seepage for treatment before discharge?

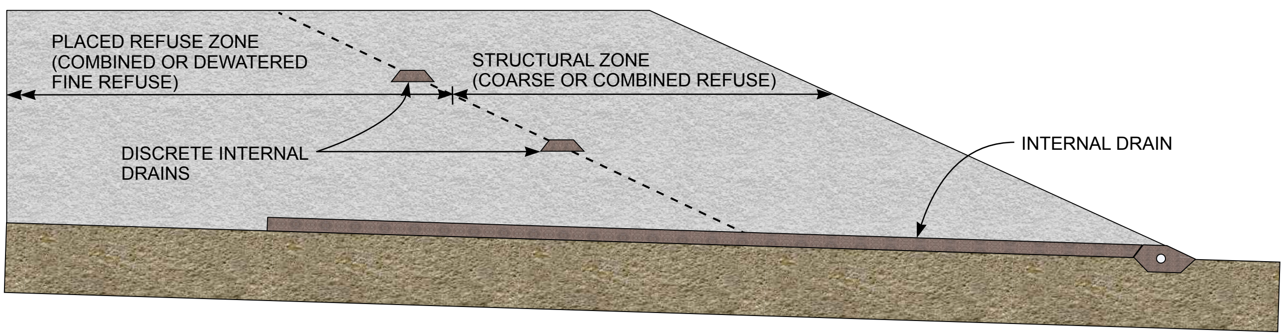
Non-impounding coal refuse embankments typically accommodate disposal of coarse, combined, and dewatered fine coal refuse. Well-graded coarse refuse may be generated without significant excess moisture, such that internal drainage provisions are primarily directed at control of natural springs or infiltration. Consequently, seepage control and slope stability can readily be addressed with minimal internal drainage requirements and normal placement and compaction of materials. Combined refuse and dewatered fine refuse generally contain excess moisture when they are generated, and greater measures are required for seepage control and slope stability. Such embankments may be zoned to establish structural downstream zones for stability and may incorporate internal drainage control structures, allowing for upstream zones where the material consistency and strength are less important. However, applicable regulations for the construction of "refuse piles" must be followed.

Several examples of designs for non-impounding embankments, including provisions for seepage control, are illustrated in Figure 6.6. Figures 6.6a and 6.6b show drainage system concepts previously discussed in Section 6.3.1, where seepage is controlled by internal drainage systems. The previous discussion of internal drain materials and their gradation, and of the need for filters, is also applicable to this type of construction. The full extent of seepage control and drainage systems must be determined on a project-by-project basis considering site conditions, the economics of various methods of providing embankment stability, the cost of leachate treatment, and appropriate regulations.

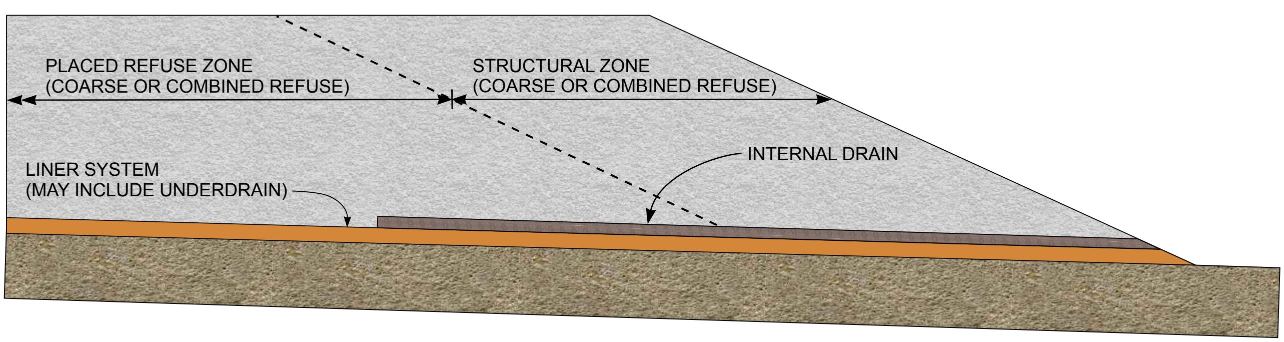
For some disposal facilities, usually depending on site location and condition, seepage impacts on the groundwater regime may need to be mitigated. In such cases, a relatively impervious liner composed of fine-grained borrow materials should be constructed beneath the disposal facility. This can be accomplished as the facility is being constructed by rearranging and compacting native soils or by using borrow soils, as illustrated in Figure 6.6c. At some sites, a geomembrane or geosynthetic clay liner (GCL) may



6.6a BOTTOM INTERNAL DRAIN



6.6b BOTTOM AND DISCRETE INTERNAL DRAINS

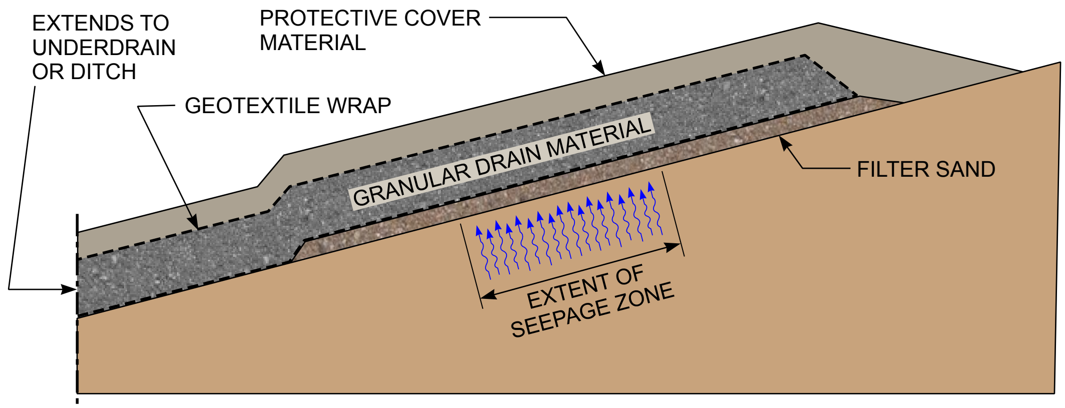


6.6c BOTTOM INTERNAL DRAIN WITH LINER SYSTEM

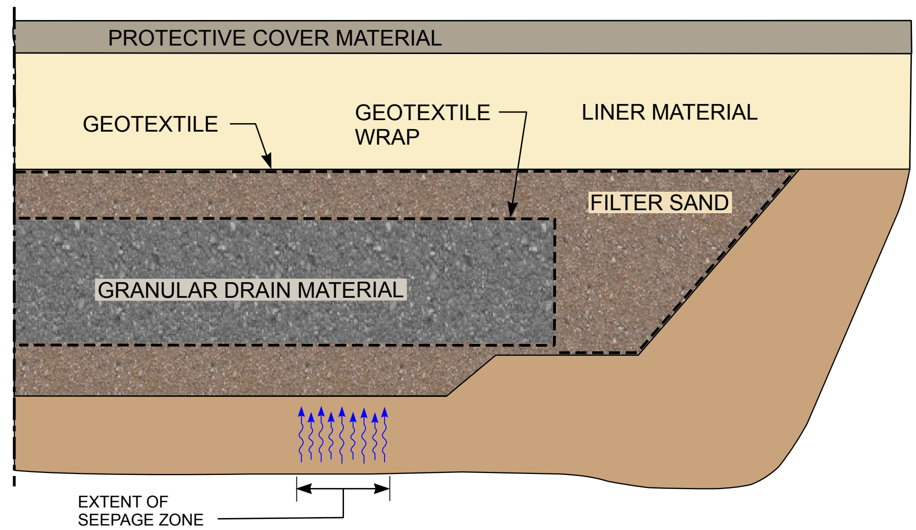
FIGURE 6.6 NON-IMPOUNDING EMBANKMENTS

be used as an impervious liner if satisfactory borrow materials are not available. Selected portions of the impervious zone may be covered by drainage materials to collect and transport leachates to a common point for treatment and/or discharge. The introduction of liner materials may affect the stability of the embankment and thus potentially impact the design slopes or configuration of a disposal facility.

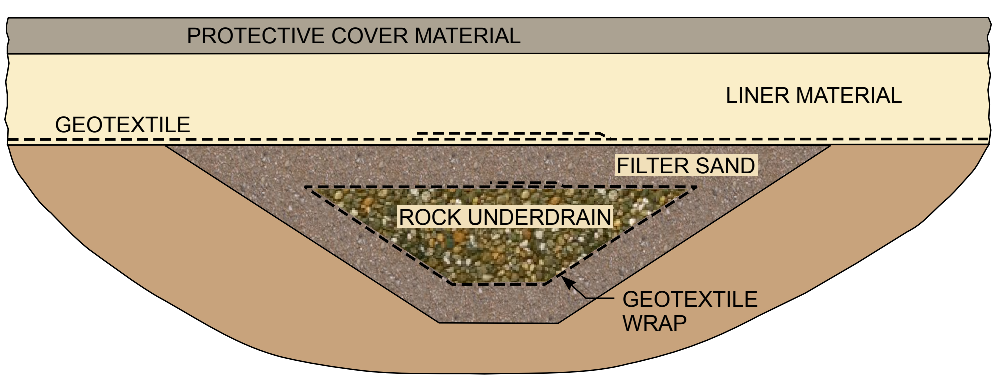
Typically, groundwater seepage or mine discharge is collected and conveyed in a manner such that it cannot enter the refuse. Figure 6.7a shows a seepage collector drain comprising a collection zone and a



6.7a SPRING COLLECTION ZONE WITHOUT LINER SYSTEM



6.7b SPRING COLLECTION ZONE AS PART OF LINER SYSTEM



6.7c ROCK UNDERDRAIN AS PART OF LINER SYSTEM

FIGURE 6.7 EXAMPLE SPRING COLLECTION ZONES AND ROCK UNDERDRAIN conveyance system. The collection zone consists of granular material covered with a granular filter layer and/or geotextile. The collected water is discharged through the granular drain or pipe to the nearest internal drainage system or beyond the downstream toe. The granular filter layer or geotextile situated between the water collecting zone and the overlying refuse and perforated pipes incorporated into the drainage system (if present) should be designed using procedures discussed in Section 6.6. Inclusion of an impervious liner over the collection zone, as shown in Figure 6.7b, may be appropriate for preventing embankment seepage from entering the collected groundwater and to minimize the volume of poor-quality water that may require treatment. Rock underdrains may be placed in valley bottoms to collect outflows from springs. Figures 6.7b and 6.7c show examples of spring collection zones and rock underdrains for a lined facility. Spring collection zones and rock underdrains should be designed for compatibility with surrounding materials using the filter criteria presented in Section 6.6.2.

Underground mine voids may cause subsidence that can impact an overlying embankment, potentially disrupting liners and internal drainage structures. Measures for evaluating and addressing potential subsidence problems are discussed in Chapter 8.

6.3.3 Slurry Cell Embankments

Slurry cell embankments may be classified as impounding embankments or non-impounding embankments depending on the configuration and storage capacity of active, uncovered cells and the potential for multiple cells to be involved in a failure. For facilities with significant coal refuse production rates, it is difficult to keep the active cell capacity below the impoundment classification limit, and thus the aim is to design the facility so that it can be classified as having low hazard potential. Such a system requires a design and construction sequence that minimizes the volume of active cells and provides for timely drainage, covering, and consolidation of completed cells so that the fine coal refuse is not flowable.

For slurry cell systems that are designed and classified as impoundments, the following geotechnical considerations are applicable:

- <u>Seepage control</u> – An internal drainage system and foundation cutoff system designed to intercept seepage and prevent it from impacting the structural zone or toe of the embankment and a foundation treatment and liner system if necessary to address leachate migration from the disposal facility. Slurry cells have limited water and slurry storage and thus represent less significant sources of seepage than conventional impoundments for fine coal refuse slurry disposal.
- <u>Slope stability</u> – Static slope stability is maintained by a structural zone constructed from coarse coal refuse or borrow material with sufficient width to effectively contain and isolate the slurry cells from affecting potential failure surfaces. Seismic stability and deformations may be of less concern because individual cells tend to be shallower deposits of fine coal refuse that are better drained and consolidated by layers of coarse refuse or borrow materials. Sloughing and erosion considerations are essentially the same as for other refuse embankments. Slurry cell disposal requires coarse refuse or borrow material for the cell structures and covering of completed cells and for any structural zones that may be part of a valley-fill configuration. Thus, to address slope stability, slurry cell systems may require more coarse refuse or borrow material for structural elements and cell construction than more traditional types of impounding embankments.
- <u>Drainage structures</u> – Structural foundations and excavation slopes for diversion channels, principal and emergency spillways, conduits and other auxiliary structures associated with impoundments.

- Underground mines – Stability, sealing of mine openings, and infiltration into underground mine workings. Slurry cell systems provide a means to mitigate break-through impacts of an impoundment into underground mine workings.

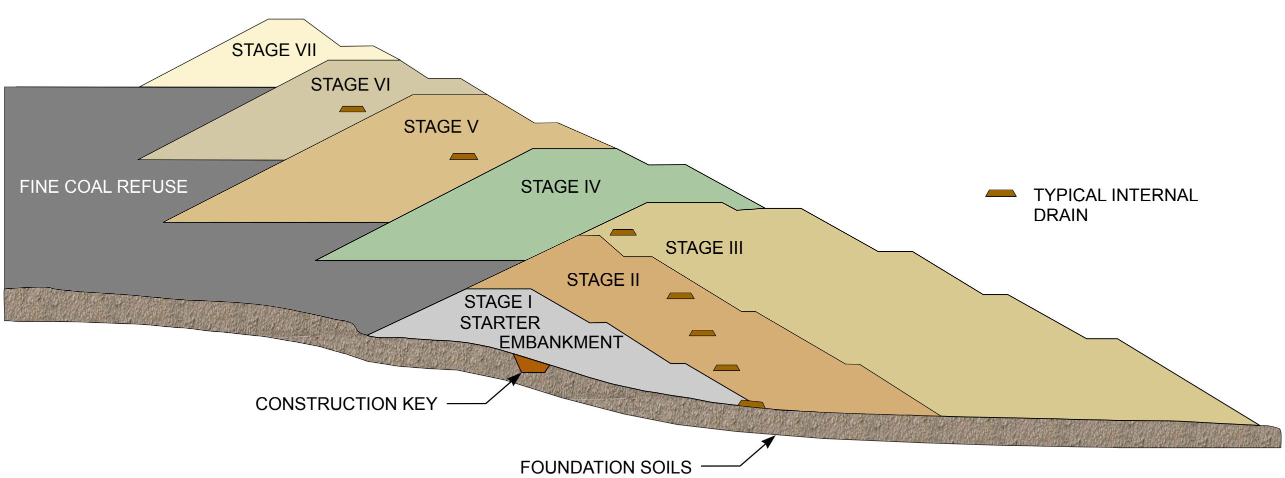
Slurry cell embankments that are designed with limited impounding capacity for water, sediment or slurry so that they do not meet the impoundment size criterion (as discussed in Section 3.4.1) are less sensitive to the impacts of seepage on stability, provided that effective drainage measures are incorporated into the cells. Such embankments may not need principal and emergency spillways and conduits associated with impoundments, thus reducing geotechnical considerations.

6.3.4 Embankment Construction Staging

Embankments are raised in stages using one or more modifications of three basic approaches: (1) upstream method, (2) downstream method, or (3) centerline method. In fact, many disposal plans use a combination of upstream and downstream methods for staging. Figures 6.8 and 6.9 show example configurations of embankments constructed using a combination of the upstream and downstream methods. This section describes common procedures for staging the development of a new embankment or extending the life of an existing embankment. In the previous two sections, general concepts for controlling groundwater and seepage are discussed; they are repeated here only as needed for explaining staging methods. The inclusion of drainage systems in the overall embankment and its staged parts must be designed on a site-by-site basis.

6.3.4.1 Upstream Method

In the upstream method of construction, the crest of the embankment is shifted progressively upstream from the starter dam, as shown in Figure 6.8. The upstream method has several advantages and disadvantages from an operational and safety standpoint, but one important drawback is the engineering and design requirements necessary to address structural performance under earthquake loading conditions. While there has not been a reported failure of a fine coal refuse impoundment due to earthquake loadings in the U.S., other similar tailings impoundments and hydraulic-fill dams have failed during or following seismic activity. The engineering analyses presented in Section 6.6 and Chapter 7 employ current methods for evaluating material properties and loadings for design.



NOTE: TAKEN FROM ACTUAL STAGING FOR VALLEY FILL SITE IN PENNSYLVANIA. THE FIGURE SHOWS UPSTREAM CONSTRUCTION METHOD (STAGES IV - VII) FOLLOWING INITIAL DEVELOPMENT (STAGES I - III).

FIGURE 6.8 UPSTREAM CONSTRUCTION FOLLOWING INITIAL DEVELOPMENT

The major advantages of the upstream method are:

- The structural fill volume is minimized because part of the new embankment is constructed in stages on top of the existing embankment and part is constructed over the deposited fine refuse slurry.
- The downstream face of the constructed embankment is the final face of the completed embankment, and vegetation and other environmental control measures can be performed on a permanent basis.
- The operational requirements such as haul and access roads, culverts, diversion and perimeter ditches can be constructed to serve the entire useful life of the facility.
- The impoundment watershed area decreases with the progress of embankment construction, requiring less volume for storm runoff handling with time.
- The starter dam, if properly designed, can provide support and become part of the internal drainage control for the subsequent embankment development.

When fine coal refuse slurry is deposited in an impoundment, grain-size sorting and layering occurs in both the horizontal and vertical planes during the depositional process. Peripheral discharge of slurry, either by single-point discharge or by multiple discharges, results in the formation of a "beach" around the discharge point. The coarser particles settle close to the discharge point and the finer particles concentrate in the upstream portion of the impoundment. The depositional process may also result in the formation of horizontal layers of coarser and finer fractions of the refuse and affect the engineering properties of the settled fine refuse.

The fine coal refuse particles settling from the slurry accumulate in a very loose state initially and have a high void ratio and moisture content. These loose deposits consolidate with time as water is expelled from the voids between the particles. Consolidation is influenced by such factors as the effective pressure of the overlying material, the hydraulic conductivity of the deposit and surrounding soil, the distance that water must travel to drain and the time during which vertical pressure is applied. Thus, the consolidation of fine coal refuse deposits varies spatially within the impoundment. Even after consolidation under self weight for years, such deposits may have high void ratios and high moisture contents, affecting the shear strength of the fine refuse. This should be borne in mind by the designer, if the upstream method of construction is utilized for embankment raising.

The disadvantage of the upstream method is that construction occurs atop previously deposited, unconsolidated tailings. Under static loading conditions, there is a limiting height to which materials can be placed without the risk of a shear failure. In the initial placement of materials for the embankment stage, normal spreading and compaction procedures cannot be followed until a firm or stabilized base is prepared. Safety of operating personnel during the initial placement of material for an upstream construction stage is very important. Additionally, the height and configuration of an upstream constructed stage will depend on the strength of the material within the zone of shearing, the downstream slope of the embankment, and the location of the phreatic surface within the facility. Under earthquake loading, this type of embankment may be susceptible to failure by strength degradation. However, a stable impoundment can be constructed by following the basic principles of embankment design and through judicious handling of material. A disposal facility constructed using the upstream method must be designed by an engineer experienced in the behavior of soils, coal refuse materials and embankments. Important considerations include:

- The width of the embankment stages or structural fill zones must be adequate to provide downstream slope stability. Upstream slope stability must be demonstrated by adequate factors of safety for failure surfaces that could compromise the crest of the embankment.

- As the embankment is raised, coarse refuse should be placed over settled slurry according to an established schedule, considering the potential for pore pressure buildup and dissipation in the saturated material resulting from the applied loads. To the extent practical, the initial push out for upstream stages should not be placed on submerged fine coal refuse.
- Procedures for placement of materials for the embankment stage should address equipment and operator activity and safety during the initial push-out onto the settled fine coal refuse.
- Reasonable estimates of movement and differential settlements should be used in designing the portion of the embankment constructed over settled slurry.
- The constructed structural portion of each stage of the disposal facility should include a system for controlling seepage, as discussed in Section 6.3.1.
- If the starter dam is intended to become the toe portion of the overall embankment, the downstream zone should be constructed of well-compacted, free-draining material so that it will facilitate the dissipation of pore pressures within the overall embankment.
- In disposal facilities retaining fine refuse slurry, the slurry discharge points should be located adjacent to the embankment and above the pool level. To the extent practical, the discharge point should be periodically moved along the length of the embankment. This will concentrate the coarsest slurry particles adjacent to the embankment, offering the advantage of their greater strength and reducing settlement when the next embankment stage is constructed.
- Analyses of embankment stability should consider dynamic loads and potential for strength degradation. Seismic design and deformation analyses are discussed in Chapter 7.

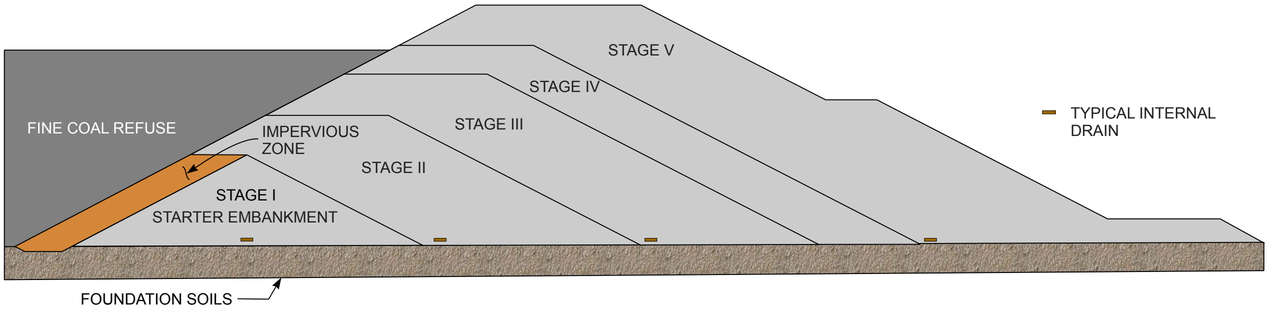
Exploration, testing, engineering analyses and regulatory review associated with addressing the above considerations are typically more complex and lengthy as compared to designs that employ the downstream method.

6.3.4.2 Downstream Method

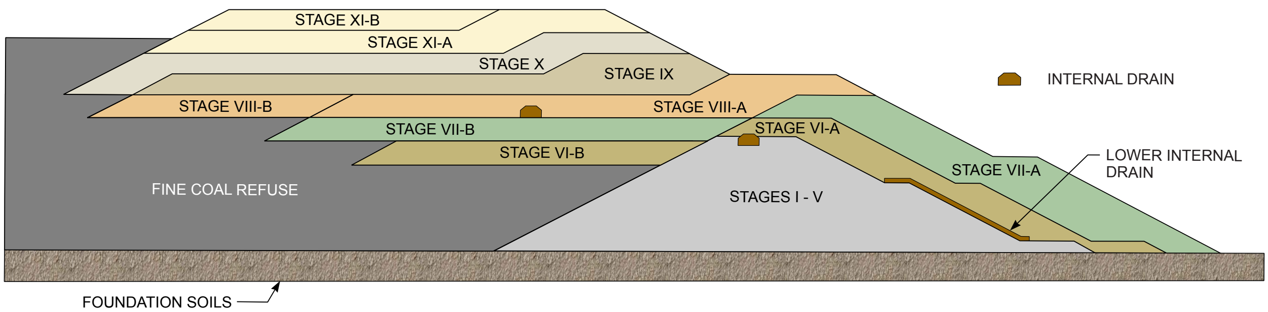
In the downstream method of construction, the crest of the embankment is shifted progressively downstream from the starter dam, as designed for the early stages shown in Figure 6.9. This method can be used for embankments with or without impoundments. The major advantages of the method are:

- The embankment is not built on hydraulically-deposited refuse.
- Placement and compaction control can be exercised as required over the entire fill operation.
- The embankment can generally be raised above its initial design height without serious limitations and complicated design modifications.
- Internal drainage systems can be installed, as required, as the construction progresses.
- The embankment can be designed with minimal concern for strength loss of the fine coal refuse under earthquake loadings. This can result in less complex and lengthy exploration, testing, engineering analyses, and regulatory review.

The major disadvantage of the downstream method is that it requires relatively large volumes of fill for raising the embankment. In the early stages of construction, it may not be possible for the mine to



6.9a EMBANKMENT STAGES I - V CONSTRUCTED USING DOWNSTREAM METHOD



6.9b EMBANKMENT STAGES VI - XI-B CONSTRUCTED USING DOWNSTREAM AND UPSTREAM METHODS

FIGURE 6.9 DOWNSTREAM EMBANKMENT CONSTRUCTION FOLLOWED BY EXPANSION USING THE DOWNSTREAM AND UPSTREAM METHODS

produce a sufficient volume of coarse refuse to maintain the crest of the embankment above the level required for disposal of slurry and routing the design storm. Features such as haul and access roads, culverts, benches, gutters, diversion ditches and perimeter ditches may have to be reconstructed at each stage, resulting in higher cost. Also, the final external face of the embankment is not created until the construction of last stage of disposal, which typically ranges from 5 to 25 years after the start of construction. Consequently, interim faces are exposed to the weather prior to covering by the next stage. These issues may have a considerable financial impact, if the facility has a relatively small drainage area and a decant system is the primary spillway. However, when the construction material is readily available, the downstream method normally results in simpler construction than the upstream method and is also more easily implemented because of the many special requirements of the upstream method.

The arrangement of the drainage and impervious zones within an embankment constructed using the downstream method is usually similar to that of the embankments discussed in Section 6.3.1.1. The zones can be placed in stages corresponding to the height of the embankment.

6.3.4.3 Centerline Method

The centerline method of embankment construction is essentially a variation of the downstream method where the crest of the embankment is not shifted in the downstream direction, but instead is raised vertically above the crest of the starter dam. A major advantage of this method is that the downstream portion of the embankment is built on a firm foundation, and therefore placement and compaction control can be exercised as required over that portion of the embankment. An important consideration in using this method is to maintain an adequate width of structural fill in order to achieve stability. Other design considerations, as discussed in Section 6.3.4.1 for upstream construction, are also applicable to the centerline method. The centerline method also has all the disadvantages listed in Section 6.3.4.2 for downstream method of construction.

The arrangement of the drainage and impervious zones within an embankment constructed using the centerline method is usually similar to that of the embankments discussed in Section 6.3.1.1. The zones can be placed in stages corresponding to the height of the embankment. The centerline method poses many design, construction, environmental and operational problems, and thus is not generally a preferred method of refuse disposal in the coal industry. This method is primarily used in tailings dam construction where cyclones are used for separating the coarser fraction of the tailings. In the coal industry, a combination of the upstream and downstream methods is typically employed to meet storm water and slope stability criteria and material requirements.

6.3.4.4 Embankments Supplemented with Borrow Material

When the production of coarse refuse is not sufficient to construct the planned height of an impounding embankment, or in other situations dictated by mining conditions, borrow material or mine spoil may be used to supplement the planned construction. The embankment can be constructed from borrow material or mine spoil using the upstream or downstream methods, or these materials can be used in combination with coarse refuse to meet the specified gradation in a zoned embankment, provided that specific zones are designated for such materials. The guidance for seepage control and drainage systems discussed in this Manual is appropriate for construction of embankments of this type.

6.3.5 Special Considerations for Existing Embankments

The engineering principles and procedures for analyzing and designing modifications to existing refuse embankments are the same as those for new facilities. If an existing embankment is not performing adequately, is not consistent with current engineering practice, or involves re-activation of an older site, the following issues may need to be considered:

- Current or additional loadings that may impact stability (static and/or seismic loading conditions).
- Potential for liquefaction and foundation failure of the embankment if constructed over fine refuse.
- Potential for piping and/or excessive uncontrolled seepage discharge from the embankment or around the decant discharge pipe.
- Potential safety problems to miners in nearby underground active workings due to overburden breakthrough.
- Potential for excessive erosion along sloped surfaces with inadequate drainage control and vegetative cover.
- For impounding embankments, an inadequate combination of runoff storage and discharge capacity to accommodate the design storm.

Specific conditions that need to be evaluated relative to the modification of existing embankments include:

- Material Placement – The structural portions of embankment should be evaluated relative to shear strength and seepage characteristics.
- Foundation Preparation – The foundation preparation work undertaken prior to construction of the embankment and the potential for sliding along the embankment-foundation interface zone should be evaluated.

- Foundation Condition – It should be determined whether the embankment overlies soft natural soils or settled fine refuse that must be considered in static and seismic stability analyses.
- Piping Potential – It should be determined if existing seepage could lead to a piping failure.
- Underground Mining – It should be determined if the facility overlies abandoned or active mines, particularly ones with openings previously covered by coal refuse that could affect the safety of miners or cause environmental or property damage downstream.
- Drainage Facility – It should be determined if the existing drainage facilities will be structurally safe for expanded operation and will meet design-storm criteria.
- Site Boundary Constraints – It should be determined if physical restrictions (e.g., streams, rivers, mines, utilities, roads, railroads) and property boundary restrictions will limit the extent of any future modifications.
- Operations Capabilities – Existing equipment and manpower capabilities or engineering services (e.g., surveying or engineering support) should be evaluated for adequacy to implement expansion or abandonment plans or facility modifications

If an existing embankment exhibits unacceptable performance or is determined to be inadequate against a stability failure, the following modifications are appropriate:

- Sliding or inadequate factor of safety against slope failure – The slope can be stabilized by: (1) constructing a downstream buttress, (2) removing material from the top, or (3) flattening the slope. Of these options, constructing a downstream buttress is most effective in stabilizing a slope.

  Downstream Buttress – A downstream buttress can be constructed using coarse refuse from normal disposal operations, and borrowed granular soil can be used for internal drainage control. Major limitations in constructing the buttress are physical or property limitations, poor access to the base of the embankment, inadequate supply of coarse refuse to accomplish buttress construction in a reasonable period, and extremely poor foundation conditions in the area of the buttress requiring excavation. Inclusion of drains within the buttress section of the embankment can be very effective in improving the stability of existing embankments. As illustrated in Figure 6.10a, a drain can be placed as a granular blanket over the downstream face of an existing embankment and then covered with coarse refuse to form a buttress.

  Removal of Material from Top – Although this alternative improves slope stability, it is often not practical to remove material from the top of the slope because of the resulting effect of the overall embankment configuration. Also, the cost of removing the material and disposing it at a different location often makes this alternative unattractive. This disadvantage can be partially overcome if the excavated material can be used to construct a buttress at the toe of the slope, additionally enhancing stability.

  Flattening of the Slope – This alternative increases the stability of a non-impounding facility, but it is often not suitable for an impounding embankment constructed by the upstream method. The removal of material reduces the thickness of the structural section of the embankment and may lower the factor of safety. Also, the cost of removing material and disposing it at a different location often makes this alternative unattractive. This disadvantage can be partially overcome if the excavated material can be used to construct a buttress at the toe of slope.

- Phreatic Level Reduction – If high phreatic surface conditions in the embankment contribute to an undesirable factor of safety, it is normally possible to lower the phreatic surface by: (1) installing horizontal drains and (2) reducing seepage.

  Horizontal Drains – One means for providing internal seepage control in an existing embankment is by drilling horizontal drains beyond the phreatic surface to intercept the seepage, as shown in Figure 6.10b. The design requirements and construction techniques for such drains are discussed in Section 6.6.2.3.5. The installation of horizontal drains is often very effective. However, the unknown and potentially variable nature of coal refuse introduces greater risk that the drains will not meet expectations. If horizontal drains are employed, concurrent monitoring of their effectiveness should be performed. Extra drains should be added, or the existing drains should be supplemented with other drainage improvement methods, if conditions warrant. Normally, considerable monitoring of pore pressures conditions is required for evaluation of drain system effectiveness.

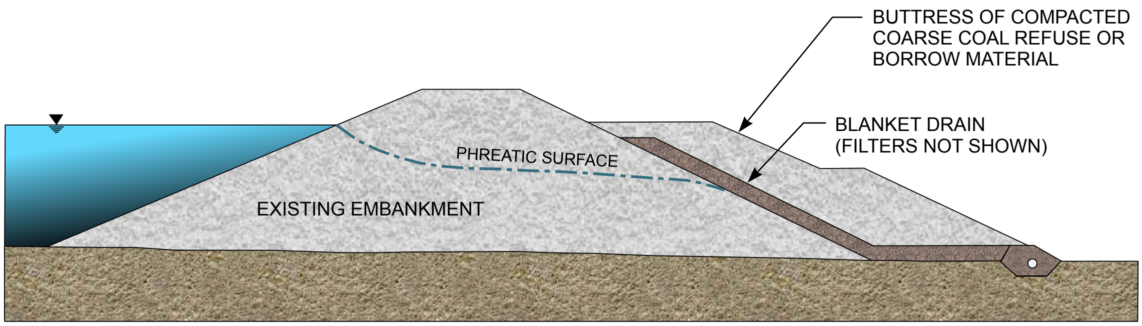
  Reducing Seepage – It is normally possible to reduce the level of seepage by sealing the surface of the upstream face of an impounding embankment. The major disadvantage of this alternative is that the magnitude and rate of improvement is difficult to predict. A "wait-and-see" approach can be taken if the existing factor of safety is not critical on a short-term basis. If the impoundment configuration permits, shifting of the fine refuse discharge point to aid in sealing of the upstream face of the embankment and to force the pool toward the back of the impoundment has been found to be effective in reducing seepage.

- Potential for Piping – The most common measure for reducing the potential for piping of embankment material in a seepage zone is to provide a granular filter around the discharge location so that water can escape without carrying additional fines. Often this measure is coupled with construction of a buttress with a filter zone between the existing embankment and new buttress material, as discussed above and illustrated in Figure 6.10a. The way to minimize the potential for piping is to prevent any past piping from extending into the new construction. Geophysical exploration may be useful for identifying voids. If a void is present, the designer must determine whether a filter system will be adequate and, if not, eliminate the void by a technique such as grouting.

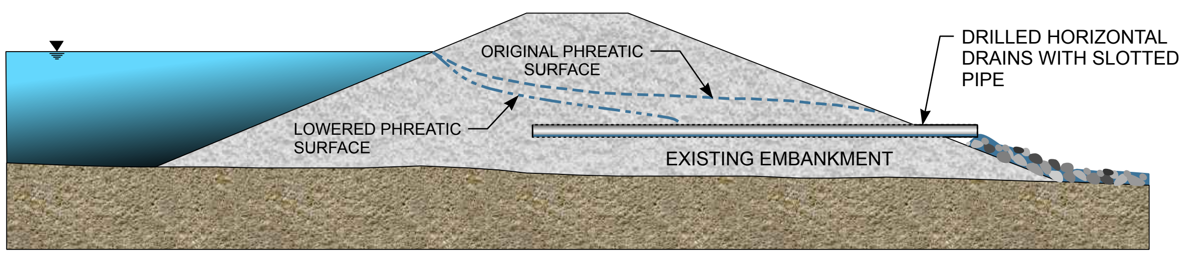
- Erosion Control – Erosion control can often be achieved through relatively minor regrading and planting of vegetation or use of vegetative mats (erosion control blankets). If an existing slope is too steep, regrading the surface to provide horizontal benches can be effective.

6.3.6 Other Impounding Embankments

Other impounding embankments at mine sites include fresh water impoundments, sedimentation ponds, and treatment ponds. The primary distinctions between these structures and slurry impoundments are: (1) the size of the embankment is usually smaller, with the width and height designed to make efficient use of borrow material, typical of an earthen dam; (2) earthen borrow materials that are not part of the on-going disposal operation are typically used for embankment construction; (3) a permanent water pool level is maintained for fresh water impoundments and treatment ponds,



6.10a BLANKET DRAIN ON EXISTING SLOPE



6.10b DRILLED HORIZONTAL DRAINS

FIGURE 6.10 INTERNAL DRAINAGE SYSTEMS FOR EXISTING IMPOUNDING EMBANKMENTS

resulting in steady state seepage conditions; (4) water pool level fluctuation within a sedimentation pond is usually over a limited range, with normal levels maintained at a low level; and (5) without the presence of fine coal refuse slurry, the pool level is typically in contact with the upstream face of the dam and thus is a source of seepage through the embankment. These geotechnical design considerations are important in the application of this Manual to projects with impoundments used for other than the disposal of coal refuse slurry.

The following geotechnical considerations apply to fresh water impoundments, sedimentation ponds, and treatment ponds:

- Seepage control – Internal drainage systems and foundation cutoff systems to prevent seepage from impacting the structural zone or toe of the embankment may be needed. Foundation treatment and a liner system may be needed. Impoundments that maintain a significant water pool depth impose greater hydraulic gradients on the embankment, potentially requiring more seepage control. Internal drains for these structures should have aggregate filters in order to comply with the federal dam safety practices discussed in Section 6.6.2.3. Additionally, the initial filling of such impoundments should be monitored closely for evidence of seepage or other distress, as many dam incidents and failures have occurred under these conditions.
- Slope stability – Static slope stability is maintained by a structural embankment constructed of borrow material, with significant attention given to foundation conditions. Seismic stability and deformations are most critical where loose foundation conditions are present, as the embankment materials are usually designed to be well

compacted and not subject to strength degradation. Sloughing and erosion considerations may be more critical than encountered for slurry impoundments because slopes may be steeper and the water pool level may fluctuate significantly. Fresh water impoundments may impose stresses on the impounding embankment due to rapid drawdown of the reservoir.

- Drainage and outlet structures – Structural foundations and excavation slopes are required for diversion channels, principal and emergency spillways, conduits and other auxiliary structures. Most fresh water impoundments incorporate outlet structures that allow emergency drawdown of the reservoir. Control of seepage along conduits should be a point of emphasis. Many dam failures have been caused by internal erosion due to excessive seepage through poorly compacted backfill around conduits.
- Underground mines – Stability, sealing of mine openings, and infiltration into underground mine workings are concerns. Considering the hydraulic pressures imposed by a fresh water dam, sites with shallow underground mine workings may not be feasible or may require significant remedial measures.

Several design references are available for earthen dams used for fresh water, sedimentation control, and treatment: USBR (1987a); USBR (1992a); Bigatel et al. (1999); NRCS (2005b).

6.4 SITE GEOTECHNICAL/GEOLOGICAL EXPLORATION

Although the characteristics of foundation materials have been discussed in general terms, specific properties required for the design of a disposal facility can be determined only by conducting surface and subsurface geological explorations at the site. The type and extent of explorations to be conducted will depend upon the size of the planned embankment, the complexity of the site geology, the nature of the foundation materials, the specific function of the facility, and, most importantly, whether or not the embankment will impound water. In general, a comprehensive site evaluation and exploration program for a coal refuse disposal facility comprises the following tasks:

1. Review of available topographic maps, geologic soil survey and mine maps, satellite imagery, and aerial photographs of the site and the surrounding area.
2. Surficial geological and geotechnical reconnaissance of the site.
3. Identification of data needs for design, such as topographic maps, mine void locations, and soil, rock and refuse properties.
4. Preparation of a site investigation plan using compiled information to develop a site-specific strategy addressing exploration methods and locations, sampling and in-situ testing requirements, and contingency activities based on anticipated and possible unanticipated subsurface conditions.
5. Conducting a subsurface exploration program consisting of a combination of borings, test pits, in-situ testing and geophysical surveys.
6. Installation of monitoring systems such as piezometers, monitoring wells, and surveying monuments, particularly for exploration at existing facilities.
7. Comparison of sample descriptions to anticipated conditions. Laboratory index testing (i.e., particle-size distribution, Atterberg limits and moisture content) should be performed on the samples to confirm visual classifications.
8. Preparation of subsurface profiles based upon results from the field exploration and laboratory index tests and review of these profiles relative to the initial site investigation objectives and expectations.

9. Selection of samples for performance testing and development of engineering properties for facility design from test results.
10. Laboratory performance testing and verification of the results using correlations with the index test results. If performance test results are inconsistent with the index test data, the inconsistency should be resolved (e.g., performance testing of reserve samples or additional field exploration to obtain replacement samples for testing).
11. Interpretation of performance test results, comparison to anticipated conditions, and selection of engineering properties needed for design.
12. Preparation of facility design, considering constructability issues, with identification of construction-phase exploration and testing requirements to confirm critical performance parameters.

The sequence and potentially iterative nature of these steps are summarized in the flow chart presented in Figure 6.11.

The preceding 12-step procedure is a suggested guideline for developing a thorough and cost-effective field exploration and testing program. The effort required for each of the listed steps will vary. Experience with similar geotechnical conditions will facilitate the development of the site exploration program. Each site exploration program must be sufficiently complete to provide the data required for a geotechnical evaluation of the site and design of the coal refuse disposal facility.

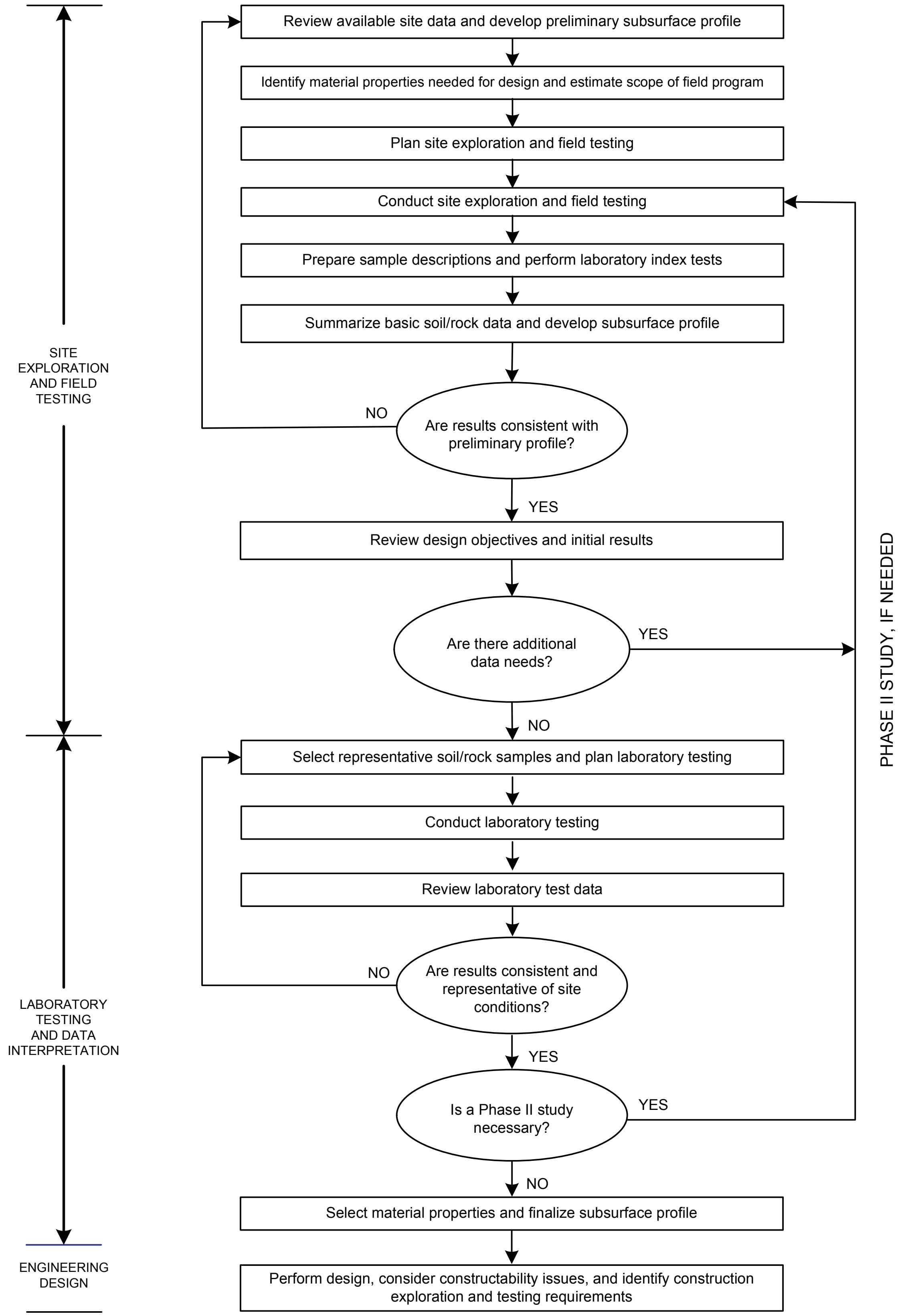
6.4.1 Background Data Sources

In planning a refuse disposal facility and in the initial site explorations, much useful information can be obtained from topographic, geologic and mine maps; agricultural soil surveys; satellite imagery and aerial photographs; and publications available from various government agencies. In addition, information from past investigations and area mining plans may also be searched to augment the information from public agencies. While typically not of sufficient detail for design purposes, these resources are readily available and represent an inexpensive source of valuable planning information.

6.4.1.1 Topographic Maps

Topographic maps are a vital source of information for planning a disposal facility and many published and on-line sources are available. The most important source is the series of standard topographic maps of the U.S. published by the U.S. Geological Survey (USGS). These maps cover quadrangles of 7.5 minutes of latitude and longitude at a scale of 1:24,000 (1 inch = 2,000 feet). More than 55,000 7.5-minute maps are available covering the 48 contiguous states. Other scales and areas of coverage are available from older map series or from other agencies. These topographic maps show the variation of ground surface elevation using contours (i.e., lines of constant elevation). The contour interval (elevation difference between adjacent contour lines) depends upon the scale of the map and the steepness of the terrain. In the Eastern United States the contour interval is usually 10 to 20 feet. In the western mountains, the contour interval is more commonly 50 feet. USGS topographic maps also show cultural and man-made features (roads, dams, buildings and political boundaries), water features (lakes, rivers and canals), wooded areas and areas of past mining activity.

Topographic maps are useful for investigating an existing disposal facility, even though it may have been developed subsequent to the topographic mapping. Comparison of the original topography with the existing condition can provide an insight to the history and development of the site. Knowledge of the topography of the surrounding area is important in planning the expansion or abandonment of an existing disposal facility.



(SABATINI ET AL., 2002)

FIGURE 6.11 FLOW CHART FOR SITE EXPLORATION, MATERIAL PROPERTY TESTING AND FACILITY DESIGN

Another resource is USGS's The National Map web site, a framework for geographic knowledge of the U.S. that provides public access to high-quality, geospatial data and information. The site allows users to access, integrate, and apply geospatial data at global, national, and local scales. It includes a variety of information layers such as boundaries, elevation, geographic names, geology (global seismic networks, and real-time earthquakes), hydrography (real-time gauging stations and wetlands), imagery, land use and land cover, natural hazards (climate and hurricanes), and topographic maps that may be useful to designers.

6.4.1.2 Geologic Maps and Publications

Geologic maps of a proposed disposal facility site and surrounding area can provide valuable engineering information. These maps are prepared by the USGS or by state geological surveys and generally show ground surface outcrops of various rock units using a color code and letter symbols. Geologic maps usually have a column identifying formations and corresponding symbols and one or more geologic sections depicting the regional structure of the rock, identifying the rock units and providing a description of the units and their characteristics. To be most useful for evaluation of individual disposal facility sites, a geologic map should preferably have a scale no larger than the USGS topographic map of the area. Only a small part of the United States has been mapped in this detail, however, and it is often necessary to use maps covering up to several hundred square miles. Geologic maps can be valuable in the initial investigation and evaluation of a site, but proper interpretation of these maps requires knowledge of the fundamentals of geology and an understanding of how geologic information can be used in planning and design.

Specific regional and local information on geologic conditions should also be considered. For example, in steep Appalachian Valleys, joints and fractures from stress relief can affect excavation and abutment stability and impoundment seepage control. Reports on physiography published by the USGS (e.g., Wyrick and Borchers, 1981) can provide insight for planning exploration programs. Furthermore, weathered joints and fractures encountered in eastern Kentucky and southern West Virginia, sometimes referred to as "hillseams," can represent critical foundation or abutment features, and mining publications may be helpful in planning associated with site preparation (e.g., Sames and Moeb, 1989). Additionally, local mining and highway construction experience should be sought, as it can disclose information on bedrock structure and fracture conditions that may influence development plans. Perin (2000) presents a case study that demonstrates the use and limitations of geologic publications, mapping, and exploration for a refuse disposal site in eastern Kentucky.

The USGS provides access to a wealth of information resources including maps, reports, publications, and links to related web sites. Resources of possible interest for refuse disposal planning and design at the time of publication include:

- USGS Library – Access to over 300,000 book, map, and serial records in the USGS Library online catalog.
- USGS Store – Source for USGS maps and books, as well as products from other agencies.
- Publications Warehouse – Search engine for 67,000 bibliographic citations.
- Geologic Information – National clearinghouse for geologic maps, datasets, and related geoscience information with links to USGS geoscience databases and programs and resources for creating digital geologic maps.
- National Water Data – NWISWeb – Comprehensive gateway to water-resources data throughout the U.S.
- National Atlas of the United States® – Comprehensive collection of small-scale geospatial data from federal agencies.

- geodata.gov – Web-based portal for access to maps, data, and other geospatial services from across all levels of government.

6.4.1.3 Agricultural Soil Surveys

Much more widely available than large-scale geologic maps are soil surveys prepared by the Natural Resources Conservation Service (NRCS). A soil survey is a detailed report on the surficial (i.e., upper 5 to 6 feet) soils of a specific area. Soil surveys typically have maps showing soil-type boundaries and photographs, descriptions of soil characteristics, and tables of soil properties and features. The tables section of a soil survey report provides information on soil properties including engineering index properties, physical and chemical properties, and soil and water features. The tables section also has information on soil use, such as crops and pasture, recreation, and engineering. Although data from these surveys are generally not suitable for design analyses, the surveys are valuable tools for initial site reconnaissance studies and for planning detailed field explorations. Printed soil surveys can be obtained from NRCS regional offices or local soil conservation district offices. Surveys are also available from the NRCS web site.

6.4.1.4 Satellite Imagery, Aerial Photographs and Other Imagery

Aerial and satellite photographs and other imagery can be extremely valuable in the investigation and evaluation of a proposed or existing disposal facility site because they reveal much natural and man-made detail that may not be apparent from the ground, no matter how carefully ground reconnaissance is carried out. Also, these data can be used with Computer-Aided Design and Drafting (CADD) and Geographic Information System (GIS) software to provide informative representations of site conditions.

Data and imagery from satellites and aerial reconnaissance flights are increasingly being utilized for site exploration and characterization purposes. These data are available from commercial vendors as well as state and federal government agencies. At the time of publication of this Manual, satellite images are available to the public at a resolution of as small as 2 feet and aerial photographs can be obtained with a resolution of as small as 6 inches. An important use of satellite imagery and aerial photographs is the performance of terrain analysis and lineament studies for identification and location of surface features that are expressions of discontinuities in the underlying bedrock. Such features may reflect bedrock joint/fracture zones that warrant additional focus during exploration programs for impoundments or that require assessment of the potential for breakthrough potential to underground mines.

Photographs represent only a portion of the information available. Sophisticated sensors on satellites and sensory equipment that can be mounted on airplanes can provide a spectrum of light band information that when analyzed in combinations with sophisticated software allow identification of various surface characteristics. Interpretation of these data is referred to as remote sensing, and some applications include classification of land usage, classification of surface cover types, identification of stressed vegetation or tree canopy, and delineation of surface thermal variations.

Some satellite systems that currently (2009) generate data that might be used in site exploration and reconnaissance studies and for preparation of site drawings and figures include:

- Landsat
- IKONOS
- SPOT
- OrbView-3
- QuickBird
- ASTER
- EO-1

There are many forms of satellite data and many vendors that can provide satellite data packages. These data providers can be identified most easily from an Internet search. Also, a wide range of information can be gathered from custom-designed aerial surveys. The types and quality of data available and the associated costs should be carefully reviewed prior to purchase of data or contracting for such data to be obtained.

Generally aerial and satellite imagery data are available or can be obtained in an orthorectified format, which means that the positions of all the data in the photograph or image are accurately located with respect to a known coordinate system. Thus the data can be input to a GIS-based system and automatically shown in true relationship to other orthorectified data such as site boundaries, features, structure, and infrastructure. The USGS through its Earth Resources Observation and Science (EROS) center provides a wide range of such data, some of which can be obtained without charge.

Digital Raster Grids (DRGs) for some parts of the country can be downloaded free from the Internet and printed, and these are generally identical to USGS topographic sheets. They are frequently used as bases for CADD drawings, but they are not attractive for GIS applications because the various types of data shown (e.g., contours, roads, structures, shadings) are combined into a single layer.

Aerial photographs corresponding to USGS quadrangles (referred to as Digital Orthophoto Quadrangles or DOQs) are also frequently used as bases for CADD drawings where proposed construction or property boundaries are shown over a photographic base. They can be used in a similar manner in GIS-based software where individual layers representing roads, structures, utilities, etc. are displayed over a photographic base. Also available are Digital Orthophoto Quarter Quadrangles (DOQQs), which are orthorectified quarter-quadrangle (7.5-minute coverage) photographs.

Digital Elevation Model (DEM) data can be used to generate elevation contour maps of a site. DRG and DEM data can be combined in GIS-based software to generate three-dimensional models of USGS topographic sheets.

The above data can be obtained from the USGS, and some states provide extensive data free over the Internet. Pennsylvania, for example, provides 7.5-minute DRGs (also available with the surrounding border cropped off), DOQs, DOQQs and DEMs for the entire state. Other information such as coal mine maps and environmental data may be available from state web sites.

Soil-type data are available for some parts of the country in digital, orthorectified form suitable for use in GIS-based software.

These sources of aerial and satellite photographic data are increasingly available in an orthorectified (also referred to as georeferenced) format for input to GIS-based software where they can be automatically displayed in accurate relationship to other georeferenced site data. The reliability, availability and costs of various types of data should be carefully evaluated when planning site drawings and pictorial displays.

6.4.1.5 Past Investigations and Area Mining Plans

Information from past investigations and old mine plans in the vicinity of a planned or existing operation can provide valuable information for planning and preliminary design of a new facility. Particularly valuable are maps of past or planned future mining, as well as geologic information on bedrock structure, jointing and fracturing. As discussed in Section 8.2.1, mine maps are available at state agencies (e.g., Virginia Department of Mines, Minerals and Energy; Kentucky Department of Mines and Minerals; West Virginia Office of Miners' Health, Safety and Training; Illinois State Geological Survey) and from MSHA and the Office of Surface Mining (OSM). MSHA district offices maintain maps until a mine closes and OSM stores a copy of the final map after closure at the OSM National Mine Map Repository in Pittsburgh, Pennsylvania. Additional information related to sources and availability of mine maps is presented in Chapter 8.

Underground mine maps can be used to locate mine features with respect to the surface or other underground mines and to determine the dimensions of pillars and mine openings (Shackleford, 2000). Information typically provided on underground mine maps includes (NRC, 2002):

- Pillared, worked-out, and abandoned areas, pillar locations, sealed areas, future projections, adjacent mine workings within 1,000 feet, surface or auger mines, mined areas of the coalbed, and the extent of pooled water.
- Dates of mining, coal seam sections, and survey data and markers.
- Surface features, coal outcrop, and 100-foot-overburden contour or other prescribed mining limit; mineral lease boundaries, surface property or mine boundary lines, and identification of coal ownership.

The accuracy and completeness of underground mine maps varies due to the age and non-uniform standards followed in their development. For example, there are significant limitations to some maps, particularly those for abandoned mines and mines operating before 1969. In addition, the horizontal and vertical (overburden) distances between mined barriers and an impoundment may not be accurately shown. Designers must consider these factors and the resulting impacts in using mine maps for refuse disposal planning and design in the vicinity of active or abandoned mines. Compounding the problem, some maps and records of older mines have been lost or destroyed. Therefore, to confirm map accuracy, site exploration using the geotechnical and geophysical exploration methods described in this chapter may be needed. Sections 8.2 and 8.3 present references and information to assist in locating available mapping and confirming its accuracy.

6.4.1.6 Individual Site Mapping

For most disposal facility sites, site mapping should be performed before planning reaches the design phase. Usually mapping is accomplished using low-altitude, large-scale aerial photography to develop detailed, large-scale topographic maps. The topographic maps can be produced at the scale and contour interval required for final site planning and design. Aerial topographic maps typically have contour intervals of one or two feet for relatively flat terrain and as much as 5 to 10 feet in steeper terrain.

For geographic areas of the size associated with most disposal facility sites, the major cost of obtaining aerial photographs is that of the aircraft. Therefore, it costs very little more to photograph areas adjacent to the anticipated site. This allows flexibility in making final plans and provides additional data for interpreting site conditions that may affect the facility design.

6.4.2 Surficial Reconnaissance and Geologic Mapping

The available topographic and geologic maps, aerial photography, and other documentation that pertain to the site should be supplemented by a surficial reconnaissance and geologic mapping, which consists of walking the disposal facility site and vicinity and observing topography, rock outcrops, mine openings, soil types, vegetative cover, spring discharges, perennial and intermittent watercourses, and any other features that may be relevant to the planned use of the site. This type of site reconnaissance generally requires a geologist or engineer who is familiar with refuse disposal and embankment design and who can recognize the significance of the observed features. If possible, reconnaissance should be conducted during times when vegetation is dormant so that site features are more visible.

Consideration should also be given to conducting the reconnaissance shortly after rainfall so that spring and flow channel conditions that may be relevant to the design can be observed.

The elevation of the site should be compared to known or correlated elevations of mineable coal seams in the vicinity. The site reconnaissance should include a search for evidence of past mining, including but not limited to mine entries, sinkholes, highwalls, haul roads, spoil piles, discolored seepage or watercourses, and areas with no vegetation or distressed vegetation. The presence of any oil and gas wells, pipelines, and other underground or overhead utilities should be noted.

Rock outcrops should be observed for lithology, bedding, and structure. Structural observations include strike and dip of strata, fracture orientation and spacing, and observable folds and faults. Where weathered joints and fractures are encountered, in-fill materials and widths should be recorded. Relatively recent fracturing should be noted as it may be indicative of subsidence. Bedding observations include thickness, sedimentary structures (e.g., planar bedding, cross bedding), and lateral continuity. Distinctive or known marker beds, such as coal seams or other persistent strata such as limestone or dolomite should be noted. Lithologic observations (e.g., color, grain size, mineral presence, weathering, and hardness) should also be noted.

Soils should be observed with respect to density, grain size, stiffness, color, mottling, structure, organic matter, and depth. Slopes should be observed for evidence of recent or older landslides, including slumps, scarps, and bent tree trunks. Mine rock waste (spoil) piles or other conditions related to previous mining such as cliffs, strip pits or ponds should be noted. General vegetation type and density should also be observed and documented.

Surficial reconnaissance may be supplemented by test pits or shallow, hand-augered borings to provide samples for basic laboratory tests for soil classification. Test pits excavated to rock may help disclose bedrock types, coal seam outcrops, and overburden jointing or fracturing. A careful surficial reconnaissance and accompanying tests can produce data sufficient for a preliminary surficial map of a refuse disposal facility site. However, as with any exploration program, it should be recognized that undetected subsurface conditions are a risk, and reasonable contingencies should be considered in the development of designs.

6.4.3 Subsurface Exploration and In-Situ Test Planning

After available information has been collected and evaluated, the designer can begin planning a program for subsurface exploration and in-situ testing. The field exploration methods, sampling requirements, and types and frequency of field tests to be performed should be determined based on project design requirements, geologic conditions, the availability of existing subsurface information, the availability of equipment resources, and local practice. ASTM D420, "Standard Guide to Site Characterization for Engineering Design and Construction Purposes," provides general guidelines for site reconnaissance, exploration planning, equipment and procedures, geophysical exploration, sampling, material classification, in-situ testing, interpretation and reporting.

An overall field exploration and in-situ testing program for obtaining the data needed to define subsurface conditions and perform engineering analyses and design should be developed. Once the field exploration and testing begins, the program may need to be modified in response to site access constraints (e.g., steep terrain may not be accessible to the available drilling equipment) or to address variations in subsurface conditions not anticipated during exploration planning.

Site exploration programs are often conducted in phases. To obtain an overview of the geological issues that can affect a facility, remote sensing, geophysical exploration and widely-spaced geotechnical sampling and testing may be conducted as part of an initial phase. During a second or subsequent phase, localized disturbed and undisturbed sampling and in-situ testing may be conducted to obtain more detailed information for defining geologic features and for determining geotechnical engineering properties for design. The types of subsurface exploration and testing activities in the typical sequence in which they are conducted are:

- Remote sensing
- Geophysical investigations
- Test pits
- Disturbed sampling
- In-situ testing
- Undisturbed sampling

Remote sensing data can be used to identify terrain conditions, geologic formations, escarpments and surface reflection of faults or highly-jointed bedrock zones, buried stream beds, site access conditions and general soil and rock formations. Remote sensing data from satellites (e.g., Landsat images from NASA), aerial photographs from the USGS or state geologic surveys, and data from commercial aerial mapping services may be useful. The designer should be familiar with the use of such data, including limitations.

Geophysical methods offer another means for characterizing subsurface conditions. Geophysical methods can be used for general site characterization, mapping abandoned mine workings and measuring physical properties in boreholes at coal refuse disposal facilities. For general site characterization, geophysical methods can be used to determine the depth to bedrock, map ground stratigraphy, detect sudden changes in subsurface formations, assess the rippability of bedrock, and map variations of physical properties within coal refuse and groundwater. Surface techniques such as electrical resistivity, ground penetrating radar (GPR), electromagnetic conductivity (EM) or seismic refraction can be applied. These techniques may be useful in defining broad variations in the subsurface, but boreholes are needed for verification and interpretation.

Some geophysical methods can be used for detection of abandoned workings or cavities in karst formations from the surface. For this purpose, the most commonly considered geophysical techniques (although only recently applied at refuse impoundment sites) include electrical resistivity, seismic reflection and gravity. The success of these techniques depends on the depth to the mine workings, degree of flooding, and thickness of the coal seam. Downhole geophysical methods can define vertical variations in physical properties. In particular, crosshole and downhole seismic tests induce mechanical waves within the ground mass to provide information on the dynamic elastic properties including the shear (S) wave velocity required for seismic site amplification studies of ground shaking and for soil liquefaction evaluations. The application of surface and borehole geophysical methods suitable for siting and engineering evaluations of refuse disposal sites is presented in Section 6.4.4.

Test pits are small excavations dug to a depth of 10 to 15 feet (i.e., to the typical reach of an excavator or to refusal in rock). Compared to other exploration methods, test pits are more efficient because they can provide information about a relatively large area inexpensively, and they expose a large amount of soil for detailed examination by the field geologist or engineer. Because the side walls of a test pit can collapse rapidly, field personnel should not climb into a hole deeper than about four feet without assessment of soil stability and use of shoring, as appropriate. The locations for test pits are typically selected in the field as the site investigation program progresses. Test pits are generally used to supplement data between borings or to explore areas where only near-surface conditions are important, such as potential source areas for borrow material. The use of test pits for geologic mapping and material sampling is discussed in Section 6.4.3.3.

Disturbed samples can be used to determine soil type, gradation, classification, water content, consistency, relative density, and stratification. The samples are considered to be disturbed because the sampling process modifies their natural structure. Disturbed samples are typically obtained using track- or truck-mounted augers and other rotary-drilling techniques. The most common disturbed sampling method is the Standard Penetration Test (SPT), which is performed using a split-barrel sampler during the drilling of geotechnical borings. Geotechnical borings allow: (1) testing as drilling progresses and recovery of samples, (2) measurement of groundwater levels and collection of groundwater samples, and (3) installation of instrumentation for monitoring the groundwater level or the deformation of the soil and rock at any depth. In planning an investigation, boring locations should be selected so as to optimize the amount of useful data from the drilling program. The basis for choosing boring locations and procedures for drilling borings is discussed in Section 6.4.3.1.

Other in-situ test and geophysical methods can be used to supplement soil boring data. For instance, the cone penetrometer test (CPT), also referred to as the cone penetration test, provides information on subsurface soils without sampling. Stratigraphy and strength characteristics of soils can be determined as the cone penetrometer is advanced. In-situ methods are most effective when they are used in combination with conventional sampling to reduce the cost and the time required for field work. Data from these tests can be correlated with sampling and testing data obtained by conventional means.

Undisturbed samples are obtained for laboratory testing when determinations of the in-place strength, compressibility (settlement), natural moisture content, unit weight, or hydraulic conductivity are needed. They also allow observation of discontinuities, fractures and fissures associated with subsurface formations. Although the sampling equipment is designed to minimize disturbance and these sample types are designated as "undisturbed," in reality they are disturbed to some degree. The degree of disturbance depends on the type of subsurface materials, type and condition of the sampling equipment used, the skill of the drillers, and the storage and transportation methods used.

6.4.3.1 Program Planning

The number and depth of borings and locations of in-situ tests required for a particular subsurface exploration program will depend on the size of the disposal facility site, the nature and uniformity of the site geology, the magnitude of loads to be applied to the natural materials, the groundwater conditions, the presence of past or active underground mining in the vicinity of the embankment, and the complexity of the facility design. For example, explorations for new impounding embankments normally require significantly more borings than those for non-impounding embankments.

The appropriate depth and spacing of borings is difficult to generalize because they depend upon site conditions and disposal facility plans. An exploration program for a specific site should provide sufficient information for identifying, delineating and correlating geologic and soil conditions for designing a safe and environmentally acceptable disposal facility. Often the final location of borings and in-situ test sites must be determined in the field, or additions must be made to the boring program based upon evaluation of the initial data obtained. Excavation of test pits should be considered as a means for supplementation of the boring program. Test pits facilitate examination of shallow subsurface conditions and the recovery of bulk samples.

The spacing and number of borings beneath a dam depends on the complexity of the geology. Some of the more important factors to consider are the character and continuity of the beds, elevation of the strata, presence or absence of joints or faults, and proximity to previous underground mining. The depth, thickness, sequence, extent, and continuity of the various strata should be determined.

USDA (1978) guidance on exploratory borings for fresh-water dams includes the following:

- Centerline of Dam – Minimum of one boring on each abutment and at the outlet structure transect, plus one boring at any abandoned stream channel, plus additional borings for correlation of strata. Boring depths should not be less than the height of the dam unless unweathered rock is encountered and is not underlain by compressible strata or mine workings.
- Outlet Conduit – In addition to the boring at the transect with the centerline of the dam, borings should be located at the vertical riser intake, downstream toe of the dam, outlet of the conduit, and additional locations as needed to define the rock surface. Boring depths should not be less than the height of the backfill over the conduit or 12 feet, whichever is greater, unless unweathered rock is encountered. At the riser intake, the depth should not be less than the planned height of the riser above natural ground or 12 feet, whichever is greater.
- Emergency Spillway – Geologic cross sections based upon three or more borings should be developed at the control section, intake section, and outlet section with additional borings at cross sections as needed for correlation and location of strata contacts and identification of excavation materials. Borings should extend to a depth not less than 2 feet below the bottom of the proposed spillway.
- Foundation Drain – Carefully-logged borings at the centerline of the dam and toe may provide sufficient information, although additional borings or test pits may be necessary where subsurface conditions are highly variable.

Tables 6.15 and 6.16 present guidelines for typical subsurface exploration and in-situ testing programs for new disposal facilities both with and without impoundments. The exploration program to be used for a particular disposal facility and the associated in-situ testing must be developed by a qualified geotechnical engineer familiar with the requirements of the proposed facility. If inadequate data are obtained from the initial boring program, additional borings and/or test pits should be advanced to supplement the original boring data.

An in-situ testing program typically involves SPTs obtained during boring advancement and unconfined compression tests on recovered split-barrel samples performed by field personnel using pocket penetrometers or field torvane equipment. Where soft sediments or fine coal refuse are critical to embankment stability, CPTs provide an effective method for characterizing the consistency and strength of the material. In-situ testing also generally includes hydraulic conductivity testing performed in soil and rock to assess foundation conditions in valley bottom and abutment areas. Installation of piezometers in completed borings can facilitate field hydraulic conductivity testing as well as site groundwater characterization.

Table 6.17 presents guidelines for a typical subsurface exploration and in-situ testing program for an existing disposal facility. Similar to a new disposal facility, the in-situ testing program for an existing disposal facility and the interpretation of subsurface data must be performed by a qualified geotechnical engineer familiar with the facility design parameters. For an existing disposal facility, borings are normally required when it is suspected that the embankment factors of safety are low or when expansion of the embankment is planned, as discussed in Section 6.3.4, and some of the information indicated in Table 6.17 may already be available from previous plans. The subsurface exploration and in-situ testing program is greatly influenced by site conditions, the disposal facility hazard potential, the type of disposed refuse and its current condition, the stage of facility development, and the extent of records of previous construction and placement of materials. Often, the in-situ testing program will be designed to obtain the same information relative to the underlying foundation materials that would be required for a new disposal facility. Extensive data relative to the materials and quality of construction of the existing embankment must generally be obtained.

TABLE 6.15 GUIDELINE SUBSURFACE EXPLORATION PROGRAM FOR A NEW IMPOUNDING EMBANKMENT

Location	Guideline Exploration Program
Abutments	One boring (minimum) should be drilled in each abutment to the maximum extension of the embankment, to the depth of the valley bottom, or to a depth where the boring will overlap geologically with adjacent borings. Multiple borings may be required in each abutment depending on the length of the dam and complexity of the geology.
Other locations	Additional borings, particularly for large embankments, beneath the structural portion of embankment with spacing and depth to provide sufficient overlap between adjacent borings to correlate data and to develop a geologic profile along the embankment.
Valley bottom or lowest portion of embankments not located in valley bottom	A least one and generally multiple borings should be planned beneath the critical structural portion of embankment. The depth of at least one boring should be approximately equal to the planned height of the embankment unless firm bedrock is encountered at shallower depth. Even if rock is encountered, deeper borings may be needed to sufficiently reveal ground water seepage/flow, to evaluate underlying deep mining, or for other special requirements. Test pits should be provided for the purpose of observing and documenting the continuity of shallow soil stratigraphy and for obtaining bulk samples.
Upstream and downstream of crest	At least one boring upstream and one downstream from valley bottom borings for correlating data and developing a geologic profile across embankment. The downstream boring should be near the toe of the embankment where slope stability is expected to be most critical. Borings should penetrate to at least softest layer to be incorporated in stability analyses and preferably into competent rock. Sufficient test pits to observe and document continuity of shallow soil stratigraphy and obtain bulk samples should be provided.
Decant/spillway structures	Borings and/or test pits along probable axes of structures to be founded on natural soil or soft rock should be planned and should be drilled to a depth at least equal to the width of the structures and preferably into very stiff soils or firm bedrock.
Embankment in vicinity of past mining	Unless available mine maps and other information confirm that underground mining is distant enough to preclude potential impacts to the embankment, borings should be drilled at locations and to depths necessary for assessment of the accuracy of the mine mapping and for determination of the potential amount of subsidence, the probability of additional subsidence, and the potential for mine breakthrough. Typically at least one boring (and to assess some subsidence and breakthrough situations, several borings are required) should be drilled to mine elevation to obtain a full profile of the overlying rock and to define the groundwater level in the mine. This is particularly important if the mine is located so that it could adversely affect embankment stability, or if the mine is still in use and water inflow from the impoundment could imperil the miners or the mining operation. Appropriate safety provisions should be taken, including wet drilling, with respect to the potential for encountering a potentially explosive gas mixture within the mine.
Impoundment area	Sufficient borings and/or test pits to observe and document continuity of shallow soil stratigraphy and obtain bulk samples in order to determine the potential for ground water impacts.
Borrow areas	Sufficient borings and/or test pits to characterize materials, estimate available volume, and obtain bulk samples.
General	Additional borings, as determined by a qualified engineer or geologist, to meet special requirements of planned disposal facility or to gain knowledge of special geologic factors affecting design.

6.4.3.2 Subsurface Exploration and In-situ Test Methods and Applicability

This section provides information on various subsurface exploration and in-situ test methods that are currently used for site characterization, sampling and determination of site-specific soil and rock properties for the design of coal refuse disposal facilities. Conventional subsurface exploration and testing programs typically include test pits, rotary drilling, SPTs, and disturbed and undisturbed sample recovery. In-situ testing methods described in this section include SPT, CPT, piezocone penetrometer test (CPTu), seismic piezocone penetrometer test (SCPTu), and vane shear test (VST). Additionally, borehole testing to measure hydraulic conductivity is typically conducted. Standardized test proce-

TABLE 6.16 GUIDELINE SUBSURFACE EXPLORATION PROGRAM FOR A NEW EMBANKMENT WITHOUT AN IMPOUNDMENT

Location/Condition	Guideline Exploration Program
Embankment toe	Sufficient borings and/or test pits should be provided near the planned location of toe to explore conditions where foundation material may have a lower strength than embankment material. Borings must extend past the depth where stability is a consideration and preferably should extend to competent bedrock. Where the depth to bedrock is less than 10 feet, test pits may be substituted for these borings.
Abutments	Sufficient borings and/or test pits should be provided to observe and document subsurface conditions where stability is a consideration and for planned excavations for diversion ditches or access roads.
Potential for groundwater impacts	If seepage through refuse may create an undesirable environmental condition, borings should be drilled sufficiently deep to identify the groundwater level. These borings will provide data for determining need for a drainage collection system and/or an impervious liner between the coal refuse and the underlying foundation. Often one or two additional borings and several test pits will need to be advanced at the general disposal facility site to determine groundwater conditions throughout the area that will be covered by refuse.
Embankment in vicinity of past mining	Unless available mine maps and other information confirm that underground mining is sufficiently distant to preclude potential impacts to the embankment, borings should be drilled to evaluate the potential effects of subsidence, the stability of the structural portion of embankment, the potential for breakthrough, or if leachates from facility could adversely affect groundwater quality in the mine.
Borrow areas	Sufficient borings and/or test pits should be provided to characterize materials, estimate available volume, and obtain bulk samples.
General	Additional borings, as determined by a qualified engineer or geologist, should be provided to meet special requirements of the planned disposal facility or to gain knowledge of geologic factors that could affect design.

dures for these in-situ testing methods have been developed by ASTM, and these are identified in the sections that follow. ASTM D 420, "Standard Guide to Site Characterization for Engineering Design and Construction Purposes," summarizes the various methods for site characterization. For in-situ test methods that do not have standardized ASTM procedures, references for additional details are cited.

Conventional subsurface exploration methods typically involve the retrieval of soil samples and rock core. Soil samples may be either disturbed (but representative) or undisturbed. Disturbed samples are those obtained using equipment that destroys the macro structure of the soil, but does not alter its mineralogical composition. Specimens from these samples can be used for determining the general lithology of soil deposits; for identifying soil components and general classification purposes; and for determining grain size, Atterberg limits, and compaction characteristics of soils. Undisturbed samples are obtained in cohesive or fine-grained soil strata for use in laboratory testing to determine engineering properties. Undisturbed samples of granular soils can be obtained, but specialized and costly procedures are required such as freezing or resin impregnation and block or core type sampling. The term "undisturbed" refers to the relative degree of disturbance to the in-situ properties of the soil. Undisturbed samples are obtained with specialized equipment designed to minimize the disturbance to the in-situ structure and moisture content of the soils. Specimens obtained by undisturbed sampling methods are used for determining the strength, soil layering, hydraulic conductivity, density, consolidation, dynamic properties, and other engineering properties. Common methods used to obtain disturbed and undisturbed soil samples are presented in Table 6.18.

6.4.3.3 Test Pits

To evaluate materials near the ground surface and to supplement the information gained from the borings, test pits or test trenches are frequently employed. Test pits are usually excavated by a backhoe or bulldozer and can range from a few cubic feet to a few cubic yards in volume. In addition to providing

TABLE 6.17 GUIDELINE SUBSURFACE EXPLORATION PROGRAM FOR AN EXISTING EMBANKMENT

Location/Condition	Guideline Exploration Program
Embankment centerline	Typically, three or more borings should be drilled in a line perpendicular to embankment axis. Borings should extend to a depth below the phreatic surface in the embankment unless a water surface is not encountered for a depth significantly below that which could reasonably be expected to affect the stability of the embankment. Piezometers should be installed in selected borings to monitor the level of water surface within the embankment and foundation. Special efforts should be made to evaluate materials and water levels encountered as a function of depth within these borings to determine if horizontal impervious zones are present that could affect seepage through the embankment.
Downstream embankment face	A line of borings parallel to the axis of the embankment should be drilled to a depth below the phreatic surface. Normally these borings should be drilled in the downstream face of embankment where the phreatic surface level is most critical to stability. This location should be determined based on a profile developed from data from the first line of borings perpendicular to the embankment axis.
No information on foundation preparation or foundation stability is a concern	If no information is available relative to procedures originally used to prepare the embankment foundation or if the designer believes that stability along the existing foundation may be critical, at least one and preferably multiple borings should be drilled in the embankment axis and downstream face and should extend through the embankment and into competent rock.
Downstream valley bottom	At least one boring should be drilled at the highest section of embankment where stability is likely to be critical. Multiple borings may be required in this area if expansion of the disposal facility to a greater elevation is planned.
Facility expansion	Additional borings should be drilled at locations that will appropriately allow for analyses associated with enlarging the embankment, as determined by designer.
Settled fine refuse	If the embankment impounds settled fine refuse slurry and an expansion of the embankment over slurry is contemplated, borings should be extended into the slurry in order to obtain samples for laboratory testing. The exploration program may also entail the use of cone penetration testing and geophysical surveys. If the embankment was built using the upstream method, the extent to which fine refuse underlies the embankment should be determined by drilling borings and/or use of geophysical methods.
Embankment in vicinity of past mining	If past mining has been completed beneath any portion of the existing disposal facility, borings should be drilled at locations and depths necessary to determine the potential subsidence, the probability of additional subsidence, and the potential for mine breakthrough. Generally, at least one boring should be drilled to below the mine elevation to obtain a full profile of overlying rock and to define the groundwater level at the mine elevation. This is particularly important if the mine is located such that it could adversely affect the stability of the existing embankment or if the embankment impounds water and the mine is still in use (i.e., where water inflow from the impoundment could be unsafe to miners or mining operations). Appropriate safety provisions should be taken to prevent possible gas release through any boring drilled to an abandoned mine.
Decant/spillway structures	If changes in the existing disposal facility include plans for decant or spillway structures founded on soil, soft rock or refuse materials, borings should be drilled and/or test pits advanced along the probable structure axis to provide data for a sufficient depth to properly design foundations or bedding requirements.
General	The required boring program and any modifications should be determined by a qualified geotechnical engineer. Flexibility will be required on the part of the engineer because the boring program will frequently need to be modified as the field investigation proceeds in order to resolve issues arising from data obtained from earlier borings.

access to larger samples than possible from borings, test pits permit direct visual examination of the soil in place. In-situ density and field shear strength tests also may be conducted in test pits at various depths. Test trenches permit observation of lateral variations of soil conditions over the trench length. This may be particularly useful in residual soils produced by in-place weathering of rock where several degrees of weathering and initial rock quality may be present and in colluvial soils where large variations in gradation may be observed. If compressible cohesive soils are present, test pits can provide access for undisturbed sampling of soil blocks, as described in ASTM D 7015, "Standard Practices for Obtaining Undisturbed Block (Cubical and Cylindrical) Samples of Soils."

6.4.3.4 Boring Methods and the Standard Penetration Test (SPT)

A variety of drilling methods can be used for subsurface geotechnical exploration, including displacement, wash, and auger borings, and percussion drilling. Augering in soil and coring in rock are preferred because these methods permit recovery of representative samples for classification and testing. Depending on the terrain, either truck- or skid-mounted drilling rigs are used. Truck rigs are generally more powerful and drill faster, but skid rigs are more maneuverable in rough or heavily wooded terrain and are less difficult to mobilize.

For drilling in soil, augers ranging from 6 to as much as 18 inches in diameter are used. Soil samples can be obtained from: (1) the auger cuttings, (2) a split-barrel drive sampler (disturbed sample), or (3) a thin-walled tube (undisturbed sample). Usually, the boring is advanced by augering a set distance (2 to 10 feet) or until there is a change in soil layer, and either a split-barrel or thin-walled tube sample is then taken.

Where undisturbed samples are not required, split-barrel samples are obtained in accordance with ASTM D 1586, "Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils." The SPT is accomplished by placing a hollow, thick-walled-tube sampler at the bottom of the boring

TABLE 6.18 COMMON SAMPLING METHODS

Sampler	Sample Type	Appropriate Soil Types	Method of Penetration
Split-barrel (Split-spoon)	Disturbed	Sands, silts, clays	Hammer driven
Thin-walled Shelby tube	Undisturbed	Clays, silts, fine-grained soils, clayey sands	Mechanically pushed
Continuous push	Partially Undisturbed	Sands, silts, and clays	Hydraulic push with plastic lining
Piston	Undisturbed	Silts and clays	Hydraulic push
Pitcher	Undisturbed	Stiff to hard clay, silt, sand, partially weathered rock, and frozen or resin impregnated granular soil	Rotation and hydraulic pressure
Dennison	Undisturbed	Stiff to hard clay, silt, sand and partially weathered rock	Rotation and hydraulic pressure
Continuous auger	Disturbed	Cohesive soils	Drilling with hollow-stem augers
Bulk	Disturbed	Gravels, sands, silts, clays	Hand tools, bucket augering
Block	Undisturbed	Cohesive soils and frozen or resin-impregnated granular soil	Hand tools

(ADAPTED FROM MAYNE ET AL., 2002)

and driving it 18 inches into the underlying soil by blows from a 140-pound hammer dropping 30 inches. The number of blows required to drive the sampler each 6-inch interval is recorded. The first 6-inch interval is regarded as a seating value, and the blows for the second and third increments are summed to give the SPT N-value or blow count resistance of the soil. If the sampler cannot be driven 18 inches, the number of blows for each 6-inch increment and for each partial increment is recorded on the boring log. For partial increments, the depth of penetration is recorded in addition to the number of blows. The SPT can be performed in a wide variety of soil types, as well as weak rocks, but it is not particularly useful for the characterization of gravel deposits or soft clays. The SPT provides a semi-quantitative measure of the stiffness or density of the soil in place. When the sampler is removed from the boring, a representative soil sample is recovered for classification and for laboratory tests that are applicable to disturbed soil samples, including moisture content, grain size analysis and Atterberg limits. The advantages and limitations of the SPT are summarized in Table 6.19.

A properly conducted boring program entails close supervision by an experienced engineer or geologist. This supervision includes: (1) careful and detailed classification of the materials recovered from the boring, (2) preparation of a detailed log for each boring, noting the classification of the material and its condition, and (3) other significant observations such as water levels in the boring.

The boring log is the basic record for geotechnical exploration and provides a detailed record of the work performed and the subsurface conditions at the boring location and can be recorded on paper or on electronic data loggers. If recorded on paper, field boring logs should be written or printed legibly and should be as clean as is practical considering site conditions and weather. All appropriate portions of the boring logs should be completed in the field as the work is being performed.

A wide variety of drilling log forms are in use, but the specific form(s) to be used for a given type of boring will depend upon local practice. A boring log should provide a description of exploration procedures and subsurface conditions encountered during drilling, sampling and coring. The following information should be provided on a boring log:

- Survey data including boring location in reference to site coordinates, surface elevation, and bench mark location and datum, if available.
- An accurate record of any change in the planned boring locations.
- Identification of the soil and bedrock encountered, including density, consistency, color, moisture, structure, and geologic origin.
- The depths of the various generalized soil and rock strata encountered.
- Sampler type, depth, penetration, and recovery.
- Sampling resistance in terms of hydraulic pressure or blows per depth of sampler penetration; size and type of hammer; height of drop.
- Soil sampling interval and recovery.
- Rock core run numbers, including depths and lengths, core recovery, and rock quality designation (RQD).
- Drilling method used to advance and stabilize the hole.
- Comparative resistance to drilling.
- Observed loss of drilling fluid.
- Water level observations.
- The date and time that the boring was started, completed, and when water level measurements were made.
- Closure of borings.

TABLE 6.19 ADVANTAGES AND LIMITATIONS OF THE STANDARD PENETRATION TEST

Advantages	Limitations
Obtain both a sample and a number	Disturbed sample (index tests only)
Simple, rugged and suitable in many soils	Analysis required if varying numerical results are obtained
Can be used in weak rock	Not applicable for soft clays and silts
Available throughout the U.S.	High variability and uncertainty

(ADAPTED FROM MAYNE ET AL., 2002)

6.4.3.5 Undisturbed Soil Sampling

Undisturbed samples are usually obtained when the structure of the soil (e.g., strength and compressibility) is important to its behavior. Relatively undisturbed samples are commonly obtained by pushing a thin-walled tube into the soil at the bottom of the boring and removing the soil sample from the boring in the protective tube. The sampler typically has an approximate outside diameter of 3 inches and inside diameter of 2 ⅞ inches. Thin-walled samplers vary in outside diameter between 2 and 3 inches and typically come in lengths from 28 to 36 inches. Larger diameter tubes are used where higher quality samples are desired and sampling disturbance must be kept to a minimum. The procedure for thin-walled tube sampling is described in ASTM D 1587, "Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes."

Undisturbed sampling is generally practical only for fine-grained, cohesive soils that contain few rock particles. Undisturbed tube sampling of coarse gravels and coarse refuse with large particles is not practical because of the resistance to pushing the tube and potential damage to the tube. Special types of tubes and procedures are sometimes employed to obtain suitable undisturbed samples of very soft or sensitive soils, such as very wet fine refuse. These are described in Mayne et al. (2002). If these types of samples are desired, the ability of the potential drillers to obtain them should be verified.

The thin-walled tubes used for undisturbed sampling are manufactured using carbon steel, galvanized carbon steel, stainless steel, and brass. Carbon steel tubes are often used, but are unsuitable if the samples are to be stored in the tubes for more than a few days because of rusting. In stiff soils, galvanized carbon steel tubes are preferred because carbon steel is stronger, less expensive, and the galvanizing provides additional resistance to corrosion. Thin-walled tubes are manufactured with a beveled front edge to reduce pushing resistance and sample disturbance. Thin-walled tubes can be pushed with a fixed head or piston head. ASTM D 4220, "Standard Practices for Preserving and Transporting Soil Samples," provides guidance for field preparation, transport and storage of undisturbed samples prior to laboratory testing.

6.4.3.6 Rock Coring

Where borings must extend into weathered and unweathered rock, rock drilling and sampling are required. For disposal sites, defining the top of rock by drilling can be difficult, especially when large boulders are present and where the top of rock profile is irregular. In all cases, care must be taken in determining the top of rock because improper identification may lead to a miscalculated thickness of rock overburden above a mine or inaccurate determination of material quantities.

Destructive rock drilling is a relatively quick and inexpensive means for advancing a boring when an intact rock sample is not required. Destructive drilling can be used to determine the elevation of the top of rock or the elevation of the top of a mine void. Types of destructive drilling include air-track drilling, down-the-hole percussive drilling, rotary tricone (roller bit) drilling, rotary drag bit drilling and, in very soft rocks, augering with carbide-tipped bits. When destructive drilling is employed, caution should be exercised in determining the top of soft rock because drilling proceeds rapidly, and weathered and soft rock can be easily penetrated, resulting in an inaccurate top-of-rock elevation.

When formations are encountered that are too hard to be sampled by soil sampling methods (typically more than 50 blows per inch with a 2-inch-diameter, split-barrel sampler), core drilling procedures should be employed, as described in ASTM D 2113, “Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigation.” Seismic refraction or other geophysical methods can be used to assist in determining the top-of-rock elevation. Seismic-refraction data can also provide information between borings. The depth of rock coring will vary depending on site conditions, but as a minimum coring should extend to a depth sufficient to account for the presence of pervious or soft strata that could affect the stability of the embankment.

Core barrels may be single-, double-, or triple-tube types. A double-tube core barrel is commonly used because the inner and outer core tubes better isolate the rock core from the drilling fluid stream and the inner tube isolates the core from the rotating outer tube. In triple-tube core barrels, the inner tube may be longitudinally split to allow observation and removal of the core with reduced disturbance.

Rock coring can be accomplished with either conventional or wireline equipment. With conventional drilling equipment, the entire string of rods and core barrel are raised to the surface after each core run for rock core retrieval. Wireline drilling equipment allows the inner tube to be uncoupled from the outer tube and raised rapidly to the surface by means of a wire-line hoist. The main advantage of wireline drilling is the increased drilling production resulting from the rapid removal of the core from the hole. Wireline coring also provides improved quality of recovered core, particularly in soft rock, because this method minimizes rough handling of the core barrel during retrieval of the barrel from the borehole and when the core barrel is opened. Wireline drilling can be used on any rock coring project, but typically is used where boreholes are more than about 75 feet deep and rapid removal of the core from the hole has a greater effect on cost.

Although NX (2.154-inch-diameter) core is the size most frequently used for engineering explorations, both larger and smaller sizes are sometimes used. Larger core sizes will usually produce greater recovery and less fracturing during drilling.

The length of each core run should be limited to a maximum of 10 feet. Core run lengths should be reduced to 5 feet or less just below the rock surface and in highly fractured or weathered rock zones. Shorter core runs generally reduce the degree of damage to the core and improve core recovery in poor quality rock.

The core bit provides the grinding action at the bottom of the core barrel assembly that cuts the core from the rock mass. Diamond, carbide-tipped and sawtooth core bits are the most commonly used. Core bits are generally selected by the driller and are often approved by the geotechnical engineer. Bit selection should be based on drill bit performance for the expected formations and the proposed drilling fluid.

Clear water is most often used as the drilling fluid in rock coring because it is readily available, does not react with most rock types and does not require special disposal procedures. If collapsing holes or zones where there is loss of drill water are encountered, a drilling mud may be required for stabilizing the borehole. Drilling mud should be used with care because it will clog open joints and fractures and can adversely affect hydraulic conductivity measurements and piezometer installations. A settling basin should be used to remove drill cuttings and to allow recirculation of the drilling fluid. Unless contaminated with oil or other substances, drilling fluids can be discharged onto the ground surface.

Rock core should be carefully removed from the core barrel, placed in a rock core box appropriately sized for the diameter of core drilled, and visually classified. The rock core recovery and RQD should be calculated and recorded. ASTM D 6032, "Standard Test Method for Determining Rock Quality Designation (RQD) of Rock Core," should be followed in determining the RQD, which is a normalized measure of the degree of rock fracturing. The rock core should be preserved and transported following the guidance in ASTM D 5079, "Standard Practices for Preserving and Transporting Rock Core Samples." Additional guidance for visual classification, core handling and labeling, and other field practices is provided in Mayne et al. (2002). The application of RQD in geotechnical design is presented in Section 6.6.

6.4.3.7 Cone Penetrometer Test (CPT) and Piezocone Penetrometer Test (CPTu or PCPT)

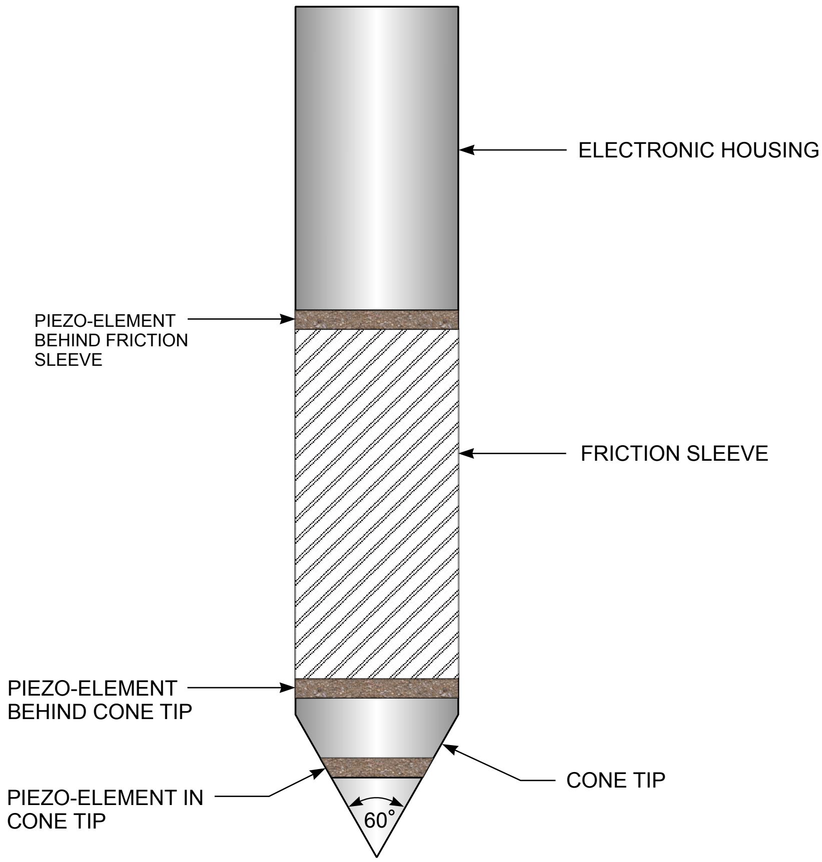
An alternative or supplement to the SPT is the Cone Penetrometer Test (CPT), also referred to as Cone Penetration Test, an in-situ test that is fast, economical, and provides continuous profiling of soil strata and soil properties. The test is described in ASTM D 5778, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils," and consists of pushing a cylindrical steel probe into the ground at a constant rate of 2 centimeters per second and measuring the resistance to penetration. The standard cone penetrometer has a conical tip with an apex angle of 60 degrees, a 10-cm^2 projected area for the cone, and a 150-cm^2 surface area for the friction sleeve. The ASTM standard also permits a larger diameter unit that has a 15-cm^2 tip and 200-cm^2 sleeve. The measured point or tip resistance is q_c and the measured side or sleeve resistance is f_s. An illustration of a typical cone penetrometer is provided in Figure 6.12.

The CPT can be used in very soft clays to dense sands, but it does not work well in gravels or rocky terrain. The advantages and limitations of using the device are summarized in Table 6.20. Because the CPT provides more accurate and reliable parameters for analysis, it is an excellent complement to traditional soil borings with SPT measurements. The CPT is not practical for coarse refuse where larger rock fragments can impede the penetrometer, but the method has been used to characterize the consistency and to estimate the engineering properties of settled fine refuse in impoundments.

A piezocone penetrometer test (CPTu or PCPT) is performed by advancing a cone penetrometer with transducers for measuring pore-water pressures. In clean sands, the measured pore pressures are nearly hydrostatic because the high hydraulic conductivity of the sand permits immediate dissipation of excess pore-water pressures mobilized by advancement of the cone. In clays, the advancement of the cone may result in the development of elevated pore-water pressures. If the advancement of the penetrometer is halted, the decay of pore-water pressures can be monitored with time and used to calculate an in-situ rate of consolidation and soil hydraulic conductivity. Details related to test methods, cone types and calibration, data reduction, and cone maintenance are provided in ASTM D 5778 and Lunne et al. (1997).

Piezocone penetrometer testing can be a viable technique for determination of gradational variability, strength, hydraulic conductivity and consolidation properties of fine coal refuse at existing refuse disposal sites. However, the measured cone resistance q_c must be corrected for pore-water pressures acting on unequal areas of the cone tip. This correction is most important for soft to stiff clays and silts and for very deep soundings where the hydrostatic pressures are high. Usually in sands, the correction is minimal because q_c is much greater that any mobilized pore pressures.

Because soil samples are not obtained with the CPT, indirect assessment of soil behavior is typically inferred from an examination of the test data. The data can be processed for use in empirical chart classification systems, or the raw readings can be interpreted by eye to determine soil strata changes. For example, clean sands are generally indicated by a total tip resistance q_T greater than 50 tsf, while for soft to stiff clays and silts, q_T is less than 20 tsf. The total tip resistance q_T is a function of the pore



(ADAPTED FROM FHWA, 1992)

FIGURE 6.12 PIEZOCONE PENETROMETER

pressure behind the cone tip q_c and some factors related to cone geometry. This value is automatically calculated and plotted during the test.

Generally, pore-water pressures associated with penetration in loose sands are approximately equal to hydrostatic pressures, in contrast to penetration in dense sands where the pore-water pressure is typically less than hydrostatic. In soft to stiff intact clays, pore-water pressures associated with advancement of the penetrometer are generally several times the hydrostatic pressure. Notably, negative pore-water pressures are observed in fissured overconsolidated materials. The sleeve friction,

TABLE 6.20 ADVANTAGES AND LIMITATIONS OF THE CONE PENETROMETER TEST

Advantages	Limitations
Fast and continuous profiling	No soil samples are obtained
Economical and productive	Unsuitable for gravel or boulder deposits(1)
Results not operator-dependent	Requires skilled operator to run
Strong theoretical basis for interpretation	Electronic drift, noise, and calibration
Particularly suitable for soft soils	High capital investment

Note: 1. Except where special rigs are provided and/or additional drilling support is available.

(MAYNE ET AL., 2002)

often expressed in terms of a friction ratio ($FR = f_s/q_T$), is also a general indicator of soil type. In sands, FR usually falls in the range of 0.5 to 1.5 percent; in clays FR normally falls between 3 and 10 percent. A notable exception is that in sensitive and quick clays, a low FR is observed. An approximate estimate of clay sensitivity is $10/FR$ (Robertson and Campanella, 1983).

6.4.3.8 Field Vane Shear Test (FVST)

The field vane shear test (FVST) is used to evaluate the in-situ undrained shear strength of soft to stiff clays and silts, mine tailings and organic muck. The test is conducted in accordance with ASTM D 2573, "Standard Test Method for Field Vane Shear Test in Cohesive Soil," by inserting a four-bladed vane (Figure 6.13) into cohesive soil at the bottom of a boring and rotating the device about a vertical axis. The torque required to turn the device is measured. A variety of vane sizes, shapes, and configurations are available depending upon the consistency and strength characteristics of the soil. Vanes can have a blade diameter D ranging from 1.5 to 4.0 inches, a vane height H ranging between 1.0 and 2.5 D, and a blade thickness ranging from 0.006 to 0.125 inches. The end of the vane is usually rectangular or tapered at 45 degrees.

ASTM D 2573 provides relationships for converting the measured peak torque to a value of peak undrained vane shear strength S_{uv} based on the geometry of the vane. For a rectangular vane with $H/D = 2$:

$$S_{uv} = 6\,T_{max} \,/\, (7\pi D^3) \tag{6-1}$$

where:

T_{max} = maximum measured torque corrected for apparatus and rod friction (length times force)

Relationships for other vane geometries are presented in ASTM D 2573.

After the test to determine S_{uv} is completed, the undrained steady-state (residual) shear strength S_{ur} can be determined by quickly rotating the vane another 5 full revolutions to fully remold the soil and then repeating the shear test. The ratio of peak to remolded undrained strengths is the sensitivity S_t. Table 6.21 provides a summary of the advantages and limitations of the FVST. Additional guidelines related to application of the FVST are presented in Mayne et al. (2002).

ASTM D 2573 recommends a loading rate of no faster than 0.1 degree per second (15 minutes for 90 degrees of rotation). At this rotation rate, the time required to reach undrained peak strength typically ranges from 2 to 5 minutes, but in very soft clays the time to failure may be as much as 10 to 15 minutes.

Chandler (1988) and Morris and Williams (2000) investigated the applied loading rate. Chandler applies a theoretical method by Blight (1968), and indicates that for typical vanes and a time to failure of 1 minute, the test will be undrained if the coefficient of consolidation (c_v) is less than 0.035 cm^2/sec (3.3 ft^2/day). Blight defined time to failure as the time from the beginning of vane rotation. Morris and Williams (2000) proposed a revision to Blight's theoretical method, which accounts for the pore pressure increase due to vane insertion as well as vane rotation and defines time to failure as the time from vane insertion. Morris and Williams indicate that, for a vane diameter of 63 mm (2.5 in), a time to failure of 2 minutes will result in an undrained test for materials with c_v as high as 1450 m^2/year (42 ft^2/day). This value of c_v should encompass coal refuse and natural materials with a plasticity index (plastic limit minus liquid limit) of 10 or higher.

To minimize drainage in fine coal refuse during the FVST, the rotational loading should be applied as soon as possible after vane insertion, and the loading rate should be increased significantly from the

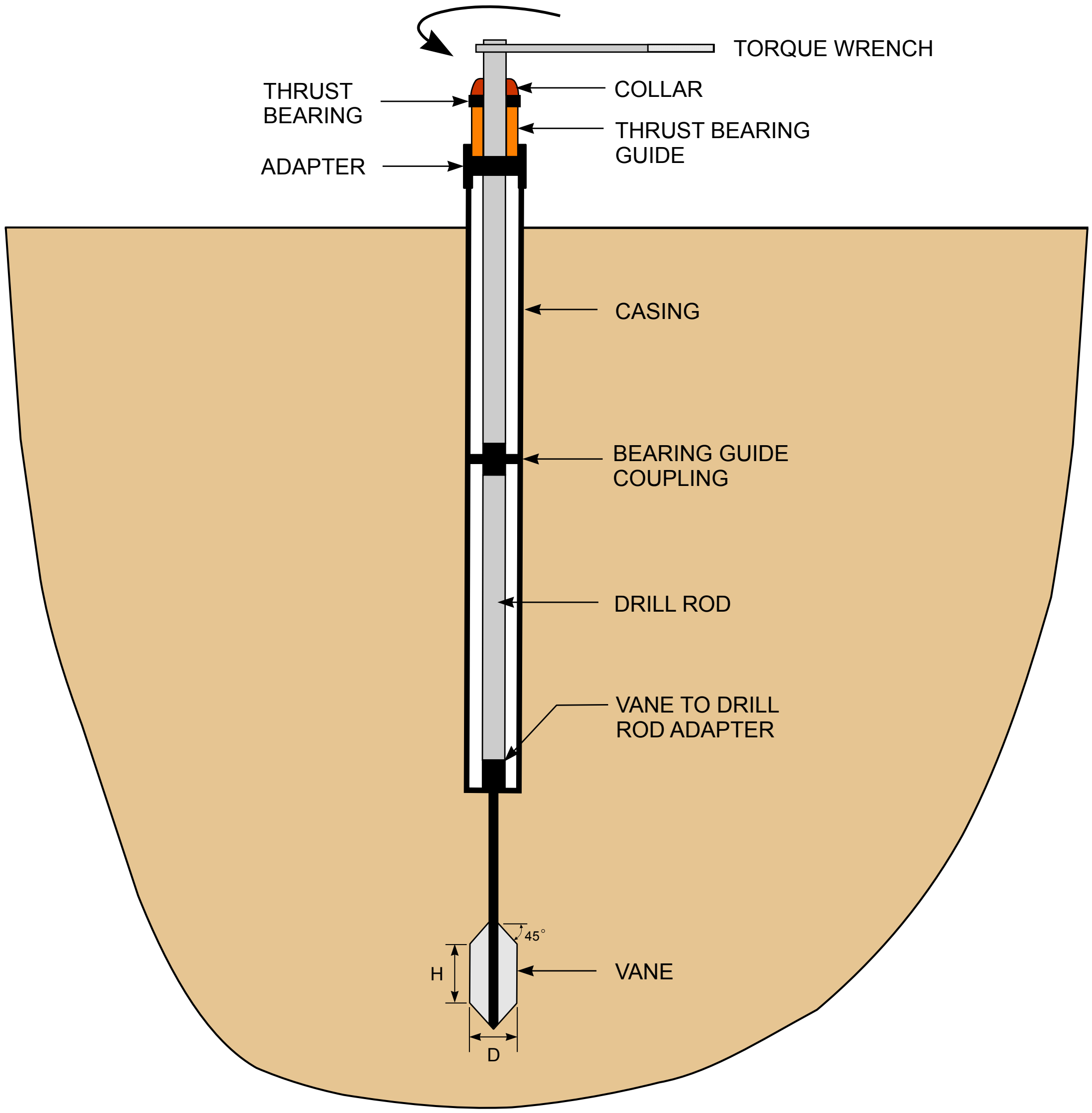


FIGURE 6.13 FIELD VANE SHEAR TEST SETUP

ASTM D 2573 recommendation in order to achieve failure within about 1 minute. For a soft material, if only the peak strength is being measured, a loading rate of 2 to 10 degrees per second is reasonable, but if the undrained, steady-state (residual) strength is being measured as well as the undrained peak strength, then the following procedure is recommended:

1. Initially apply the torque at a rate of about 10 degrees per second.
2. After the peak strength has been reached, increase the rate of rotation to at least 60 degrees per second (6 seconds per revolution or faster) for at least 5 complete revolutions to remold the material.
3. Avoiding a rest period, slow the rate of rotation to about 10 degrees per second to measure the steady-state strength.

The rotation and torque should be measured and recorded as the test is conducted, which can be accomplished with a gear box and stylus recording system or other type of data acquisition system. The rod and apparatus friction corrections (per ASTM D 2573) should be performed for the rates of rotation actually used in steps 1 and 3.

TABLE 6.21 ADVANTAGES AND LIMITATIONS OF THE FIELD VANE SHEAR TEST

Advantages	Limitations
Assessment of undrained strength (S_{uv})	Limited application to soft to stiff clays
Simple test and equipment	Slow and time-consuming
Measurement of in-situ clay sensitivity (S_t)	Raw S_{uv} needs correction (empirical)
Long history of use in practice	Can be affected by sand lenses and seams

(MAYNE ET AL., 2002)

Studies by several researchers have demonstrated the importance of correcting the measured vane strength for use in stability analyses involving embankments on soft ground, bearing capacity analyses, and for analyses associated with excavations in soft clays. The correction to obtain the mobilized shear strength is given by:

$$S_{u\,(mobilized)} = \mu_R\, S_{uv} \tag{6-2}$$

where:

μ_R = empirical correction factor related to plasticity index (PI) based on back calculation from failure case history records of full-scale projects (dimensionless)

Bjerrum (1972) recommended values of μ_R to correct the measured peak field vane strength (with a time to failure of a few minutes or less) to a value of $S_{u\,(mobilized)}$ (during a full-scale slope failure corresponding to a time to failure of several weeks to several months) that may be appropriate for stability failures.

Chandler (1988) combined Bjerrum's case history data and other data sets to develop a more specific strain-rate correction factor:

$$\mu_R = 1.05 - b\,(\mathrm{PI})^{0.5} \tag{6-3}$$

where b is a dimensionless rate factor that depends on the time to failure (t_f) in minutes (for a full-scale slope stability failure):

$$b = 0.015 + 0.0075 \log t_f \qquad (10\text{ min} < \mathrm{b} < 10{,}000\text{ min}) \tag{6-4}$$

Values of μ_R as a function of PI and t_f are presented in Figure 6.14. Slope stability failures should generally be considered to have t_f values of 10,000 minutes (7 days) because of the construction methods involved.

These strain rate correction factors are for peak undrained strengths and are based on case histories for natural clays, not fine coal refuse. However, until more research is available, the same correction factors should be applied to fine coal refuse and to remolded undrained (steady-state) strength as well as to undrained peak strength.

The FVST is applicable only to soft to medium stiff clay-like materials that are relatively impermeable such that they remain undrained during the test. If drainage occurs, the measured torque and resulting calculated strength will exceed the actual value. No published guidance is available on limitation

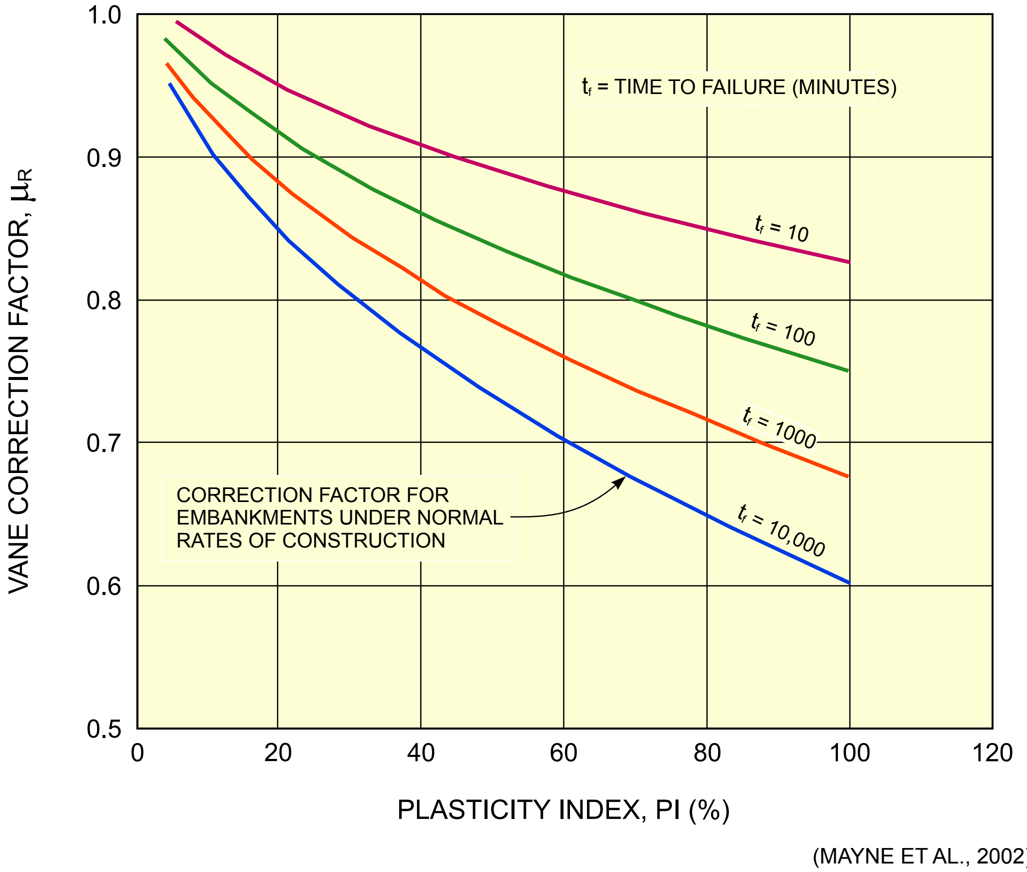


FIGURE 6.14 VANE CORRECTION FACTOR

relative to PI; this Manual recommends that the FVST should not be used for materials with a PI of 7 or less because these materials are likely to drain during the test. FVST should be used with caution in materials with higher PI that contain thin layers of sand-like material because the sand-like layers may allow drainage. For material with PI between 7 and 10, FVST should only be used if supporting data are provided to demonstrate that the test was undrained. Material samples should be recovered from each test zone for geotechnical index testing (moisture content, grain size distribution, and Atterberg Limits at a minimum). Also, CPT and piezocone measurements performed adjacent to an FVST can be used to measure the rate of pore-pressure dissipation, so that it can be confirmed that the zone in which the FVST is run is relatively impermeable. The piezocone data may be used to estimate the value of c_v in order to confirm that the rotation rate is sufficiently rapid that the test can be considered undrained.

The FVST is not intended for stiff clay-like materials because these materials will normally exceed the torque capacity of the FVST device. The FVST cannot be used for testing sand-like material or coarse refuse because (1) these materials will drain during the test and (2) the shear strength of these materials will exceed the capacity of the FVST device.

6.4.3.9 Directional (Longhole) Drilling

Directional or longhole drilling refers to: (1) in-mine drilling operations used to identify geological and mining conditions in advance of mining and (2) surface drilling through an outcrop to determine cover and coal barrier thickness. Development in the 1990s of systems with instrumentation to measure drill bit location, high-thrust drilling equipment, powerful downhole motors and high-strength drill tubing has allowed the implementation of this technique. In combination with hydraulic fracturing techniques used to increase connectivity between boreholes, directional drilling has been employed to reduce in-situ methane gas contents in low-hydraulic-conductivity coal and to fracture a massive sandstone roof in advance of longwall mining (Brunner and Schwoebel, 1999). Depending on conditions, drilling rates of 300 feet per shift and drilling accuracies of approximately 1 degree in azimuth and 0.5 inches in pitch can be achieved. The longest reported in-mine horizontal borehole exceeds 5,000 feet in length (Brunner and Schwoebel, 1999). Directional drilling has also been used to locate old abandoned workings, drain accumulations of mine water, and degasify gob areas (Kravits and Schwoebel, 1994). While conventional coring should not be attempted in a long, directionally-drilled borehole, spot cores can be taken at selected locations (Kravits and Schwoebel, 1994). As such, the technique may have applicability for accurately locating and determining the thickness of horizontal in-mine barriers and can be a means of validating geophysical methods for locating and sizing barriers in mines.

6.4.3.10 Field Hydraulic Conductivity Tests

The hydraulic conductivity of soil or rock is often measured in place during subsurface exploration to determine if seepage through foundation materials will be an important design consideration. The hydraulic conductivity of soils near the ground surface can be measured in hand-dug pits or cased holes. At greater depths the hydraulic conductivity can be determined in the borings used for sampling, provided that the borehole is cased and the hydraulic conductivity test will not affect sampling of an important soil layer immediately below the test level. Table 6.22 summarizes various methods for measurement of hydraulic conductivity in the field. Additional guidance in selecting field test methods is provided in ASTM D 4043, "Standard Guide for Selection of Aquifer-Test Method in Determining of Hydraulic Properties by Well Techniques."

The most commonly performed field hydraulic conductivity test involves a sudden change (increase or decrease) of water level in a borehole and measurement of the response in terms of water level versus time. The test procedure and methods of analysis are presented in ASTM D 4044, "Standard Test Method (Field Procedure) for Instantaneous Change in Head (Slug) Tests for Determining Hydraulic Properties of Aquifers." Hydraulic conductivity values determined by this procedure often fail to correspond well with values predicted from laboratory testing due to: (1) characteristic differences (e.g., gradation and density) between the soils tested in the field and in the laboratory, (2) clogging of the boring face by soil particles suspended in the water, (3) varying directional hydraulic conductivity of layered soil that cannot be duplicated in the laboratory with disturbed soil samples, or (4) failure to conduct the field test in saturated soils, resulting in measurement of the rate of saturation rather than hydraulic conductivity. Because field hydraulic conductivity tests inherently account for the effects of geologic variations, they are generally more representative of in-situ conditions than laboratory tests. However, the evaluation and interpretation of the test data require knowledge of the test conditions and of the possible effects of these test conditions on the results.

The field hydraulic conductivity test for rock is similar to that for soil, as described in ASTM D 4630, "Standard Test Method for Determining Transmissivity and Storage Coefficient of Low-Permeability Rocks by In Situ Measurements Using the Constant Head Injection Test." The test is performed in a rock boring using water pumped under pressure from the ground surface. Two types of tests can be performed: a single-packer test or a double-packer test.

In the single-packer test, a pipe is inserted into a boring with a packer at the lower end of the pipe. The packer is expanded mechanically or pneumatically from the ground surface to seal the annulus between the walls of the boring and the pipe, and water is pumped down the pipe into the boring below the packer. Since the depth and diameter of the hole below the packer, the applied water pressure, and the rate of flow of water through the system are known, the average hydraulic conductivity of the rock below the packer can be calculated.

TABLE 6.22 FIELD METHODS FOR MEASUREMENT OF HYDRAULIC CONDUCTIVITY

Test Method	Applicable Soils	Reference
Various field methods	Soil and rock aquifers	ASTM D 4044
Pumping tests	Drawdown in soils	ASTM D 4044
Slug tests	Soils at depth	ASTM D 4044
Constant head injection	Low-hydraulic-conductivity rocks	ASTM D 4630
Pressure pulse technique	Low-hydraulic-conductivity rocks	ASTM D 4630

(ADAPTED FROM MAYNE ET AL., 2002)

In the double-packer test, a selected zone within the boring is tested by placing one packer at the bottom and another packer at the top of the test zone. Water is then pumped through the pipe into the annular space between the packers. The hydraulic conductivity is computed by the same procedure as for the single-packer test.

During hydraulic pressure testing of rock, the water pressure applied to the test zone must not exceed the pressure caused by the weight of overburden above the test zone. Excess pressures may force water into joints, bedding planes or fractures and cause additional fracturing of the rock by lifting the overburden. This “jacking” of the rock can seriously increase the amount of leakage that will occur later and may also decrease the stability of the rock mass and its ability to resist the loads applied by an embankment or other surface loading.

Hydraulic pressure testing to measure rock hydraulic conductivity is an essential part of subsurface exploration where an impoundment is planned and where groundwater leakage could create unsafe uplift pressures or piping of the embankment or foundation soils. In addition to posing a threat to the safety of the embankment and natural slopes, excessive leakage can increase stream and groundwater pollution down gradient from the refuse disposal facility. The hydraulic conductivity values obtained from hydraulic pressure testing allow the prediction of quantities and locations of leakage from the impoundment. If rock zones where excessive leakage could occur are observed, grouting of the rock formations may be required. Houlsby (1990) and Weaver and Bruce (2007) discuss procedures and materials for grouting rock formations to reduce water flow.

6.4.3.11 Groundwater-Level Measurements

For new embankments, determination of the groundwater level in the planned construction area is important to construction requirements, particularly where excavations are planned. An understanding of the groundwater regime is essential in determining the direction and rate of possible seepage from an impoundment and may aid in estimating the overall hydraulic conductivity of the foundation materials.

Using piezometers to measure the phreatic surface level within an existing embankment is often the most important of the field tests used for evaluation of an existing coal refuse disposal facility, whether there is an impoundment or not. ASTM D 4750, “Standard Test Method for Determining Subsurface Liquid Levels in a Borehole or Monitoring Well (Observation Well),” describes procedures that should be followed in measuring groundwater levels. Piezometers or standpipes should generally be installed in exploration boreholes. Common techniques for installing piezometers and standpipes are presented in Chapter 13 along with monitoring procedures. The accuracy of data obtained from piezometers is directly related to the care taken in their installation. Therefore piezometer installation should always be under the supervision of a qualified engineer or geologist.

6.4.3.12 Water Flow and Quality Tests

In addition to phreatic surface level measurement with piezometers, valuable information for an existing impounding refuse disposal facility can be gained from: (1) monitoring flows from nearby springs or seep areas with weirs and (2) evaluating water quality aspects of these flows. As an example, measuring the volume of flow from a spring below an impoundment during both wet and dry periods, and as the impoundment level changes, will provide an indication of the rate of seepage from the impoundment and the effect on the overall groundwater system. Likewise, simple field measurements of temperature and acidity of seepage will allow comparison with similar measurements for the water in the impoundment and/or the inflow groundwater. Flow measurements and related water quality data are generally less important in the design of a new disposal facility, but the data are useful for future evaluation of the effect of the impoundment on the local and regional groundwater and surface water quality.

The type of weir to be used for flow monitoring and related construction requirements are determined by the magnitude of flow to be measured and the type of material in which the weir will be placed. In the case of very small seeps, the flow volume can be estimated simply by observation. If the flow initially passes over a "natural weir," such as a rock outcrop or through an existing pipe, estimates can be made without installing special instrumentation. Construction of weirs and methods for accurately measuring flows from weirs and pipes are discussed in detail in Chapter 13.

Field testing of water can be performed using portable equipment to obtain indicator parameters such as pH, specific conductance and temperature, as well as some other mining-related parameters. Where measurements of additional constituents and characteristics of seepage water (including sulfate, chloride, iron, manganese, acidity, alkalinity, and dissolved and suspended solids) are desirable, additional water sampling and laboratory testing can be performed.

6.4.3.13 Backfilling of Boreholes

Boreholes at coal refuse disposal facility sites should not be left open, particularly if they are located beneath embankments or impoundments, or if they can potentially provide a pathway for fluid flow that is detrimental to site safety. Open boreholes can be backfilled with drill cuttings, cement grout, bentonite, and other materials depending on the objectives. Where there is no concern related to fluid migration or the impact of seepage on ground conditions, backfilling with cuttings or other materials may be acceptable. As a practical matter, boreholes in cohesionless soils may collapse when not supported, and it may not be possible (or necessary) to backfill such boreholes. Grouting of an open borehole is generally performed for the purpose of constructing a barrier that will prevent the vertical migration of fluids between geologic units. At active mine sites, the purpose may be to maintain the barrier between the mine workings and other strata. Materials employed for backfilling boreholes include cement, bentonite slurries, dry bentonite, and fast-setting cement grouts. Placement can be accomplished by tremie, pumping, and surface pouring. Site-specific considerations for a grouting program include: (1) whether to grout, (2) where to grout, and (3) the method of deployment.

ASTM D 5299, "Decommissioning of Ground Water Wells, Vadose Zone Monitoring Devices, Boreholes, and Other Devices for Environmental Activities," presents guidance on methods and materials for closing of boreholes. While this standard is primarily oriented to environmental activities, it can be used to decommission boreholes where no contamination is observed. Attributes of common borehole plugging materials are discussed in the standard.

Grouting of boreholes that penetrate mines requires special provisions for supporting the borehole plug above the mine void (and potentially in the mine floor, if the boring is advanced through the mine). Frequently, sacrificial casing is left in the mine void to support the plug, although a grout basket has been used to allow sealing the borehole and to permit grouting.

6.4.4 Geophysical Methods

Applied geophysics is a rapidly evolving field, and the applicability of geophysical techniques to coal refuse disposal facilities will continue to advance with respect to the aspects of data gathering, processing, interpretation and presentation of the geophysical data. Two basic deployments of geophysics are available: (1) surface surveys and (2) measurements from boreholes. Airborne geophysical techniques are not discussed herein, as their application in terms of identifying features of interest with respect to coal refuse disposal facilities is still in the experimental stage. Nevertheless it is worth noting that in some cases airborne electromagnetic (EM) surveys have been used to map flooded, abandoned coal workings (Love et al., 2005), and aeromagnetic surveys have been used for many years to map abandoned well casings, which can be a significant hazard to coal mining (Frischknecht et al., 1985).

Numerous sources of information related to geophysics are available in the general literature. A good source of information is the Environmental and Engineering Geophysical Society (EEGS) in Denver, Colorado, which annually holds the Symposium on the Application of Geophysics to Engineering and Environmental Problems (SAGEEP). The SAGEEP proceedings are an excellent source of up-to-date information on the application of engineering geophysics to the types of problems that could be encountered at a coal refuse disposal facility. Other comprehensive compilations of geophysical techniques for subsurface exploration for engineering applications include Ward (1990), USACE (1995a), Sabatini et al. (2002), Wightman et al. (2003), and Sirles (2006). The Federal Highway Administration (FHWA) presents summaries of geophysical techniques at their web site. This material is substantially based on the USACE (1995a) work.

MSHA (2008) is a summary report of mine void detection demonstration projects that were performed to evaluate the use of geophysical techniques for detection of underground mine workings. These projects include actual field demonstrations of void detection at mine sites using seismic methods, electrical resistivity, electromagnetics, and radar.

The following ASTM standards provide guidance for conducting geophysical exploration:

- D 6429, “Standard Guide for Selecting Surface Geophysical Methods”
- D 6430, “Standard Guide for Using the Gravity Method for Subsurface Investigation”
- D 6431, “Standard Guide for Using the Direct Current Resistivity Method for Subsurface Investigation”
- D 6432, “Standard Guide for Using the Surface Ground Penetrating Radar Method for Subsurface Investigation”
- D 5753, “Standard Guide for Planning and Conducting Borehole Geophysical Logging”
- D 6639, “Standard Guide for Using the Frequency Domain Electromagnetic Method for Subsurface Investigations”
- D 7128, “Standard Guide for Using the Seismic-Reflection Method for Shallow Subsurface Investigation”
- D 6820, “Standard Guide for Use of the Time Domain Electromagnetic Method for Subsurface Investigation”

While ASTM guides provide useful background information on geophysical techniques, they may be dated in terms of defining procedures for data acquisition, processing, interpretation and presentation. For example, ASTM D 6431, “Standard Guide for Using the Direct Current Resistivity Method for Subsurface Investigation,” discusses the technique in terms of acquisition with a four-electrode system and processing of one-dimensional data sets with computer programs developed in the 1970s. Modern resistivity surveys are commonly conducted with multi-electrode arrays, and the data are routinely processed and interpreted in terms of 2D profiles or 3D blocks.

6.4.4.1 Surficial Geophysical Techniques

In general terms, surface geophysical testing can be used to create a general image of subsurface conditions that can be checked by intrusive means such as borings or test pits. Data from geophysical testing should always be correlated with information from direct methods of exploration. Often, a combination of geophysical and direct exploration methods provides the best approach for interpreting subsurface conditions. When conducted at the outset of a subsurface investigation program, geophysical exploration can prove to be cost-effective through reduction of the number of borings needed for characterization of a site.

Conventional applications of surface geophysics include: (1) establishing the stratification of subsurface materials, (2) mapping the top of bedrock, depth to groundwater, and extent and quantity of soil deposits, and (3) determining the rippability of hard soil and rock. Over the past several years some improvements in traditional geophysical techniques have enhanced capabilities for the detection of abandoned mine workings and the presence of karst-related voids.

As summarized in Table 6.23, surface geophysical testing offers some advantages and limitations that should be understood before a technique is selected for a specific application.

Table 6.24 presents an overview of surficial geophysical methods and techniques in relation to the physical parameters measured and exploration objectives for coal refuse disposal facilities. The following text describes the most commonly used geophysical techniques listed in the table.

6.4.4.1.1 Seismic Refraction

The seismic refraction technique consists of measuring the first arrival of P- and/or S-waves at varying distances from a seismic source. The most common application of this technique is the determination of the depth to bedrock, which requires that the upper layer velocity (soil/weathered or soft rock) is less than that of the lower layer (competent rock). If a high-velocity surface layer is present, the technique is not effective. For this reason, the technique is generally not applicable for detection of mine voids.

TABLE 6.23 ADVANTAGES AND LIMITATIONS OF SURFACE GEOPHYSICAL TESTING

Advantages	Limitations
1. Many geophysical tests are non-invasive and thus offer significant benefits in cases where conventional drilling, testing, and sampling are difficult (e.g., deposits of gravel, talus deposits or access constraints). 2. Geophysical testing generally covers a relatively large area, thus providing the opportunity to characterize large areas with relatively limited testing. It is particularly well suited to projects having large areal extent (e.g., new refuse disposal facility). 3. Some types of geophysical measurement can assess the characteristics of soil and rock at very small strains (0.001%), thus providing information on truly elastic properties. 4. Most geophysical methods are relatively inexpensive when considering cost relative to the relatively large areas over which data can be obtained. 5. A properly performed geophysical survey can reduce the number of borings required for site characterization.	1. Geophysical testing, when applied to locating changes in soil and/or rock properties, will be effective only if a target of interest has a physical contrast with the surrounding ground. 2. Results are generally interpreted qualitatively. Useful results can only be obtained by an experienced engineer or geologist who is familiar with the particular testing method. 3. Specialized equipment is required, as compared to more conventional subsurface exploration methods. 4. Results from surface geophysical testing should be validated using direct methods of exploration such as borings.

(ADAPTED FROM SABATINI ET AL., 2002)

Seismic waves are usually created using a sledge hammer for depths up to about 50 feet and with explosives for depths up to about 100 feet. Other sources such as vibrators are sometimes used. Initially, the seismic waves travel solely through the soil to arrive at geophones (vibration transducers) located away from the source. The seismic waves also propagate through the overburden and refract along the bedrock surface. While the waves are traveling along this surface, they continually refract seismic waves back to the ground surface that are also detected by the geophones and recorded with a seismograph. The result is the generation of travel-time curves, as shown in Figure 6.15. The method can often be a low-cost (compared to boreholes) method of bedrock mapping and overburden estimation. When measurements are obtained with a high degree of redundancy, the result is a reliable acoustic image of the subsurface in terms of layers and variations of seismic velocity within the individual layers.

Seismic refraction data can also be useful for determining the rippability of rock materials using heavy construction equipment. Companies such as Caterpillar have prepared graphs comparing rippability versus P-wave velocity for various equipment types, an example of which is shown in Figure 6.16.

The design of a seismic refraction survey involves locating the profiles where data are desired and determining the length of the array of geophones (the geophone spread) and the geophone spacing.

TABLE 6.24 SURFACE GEOPHYSICAL METHODS IN RELATION TO TYPICAL INVESTIGATION OBJECTIVES

Geophysical Method	Dependent Physical Property	Applications (see key below)									
		1	2	3	4	5	6	7	8	9	10
Seismic refraction	Elastic moduli; density	P	P	P	S	X	X	X	M	M	S
Seismic reflection	Elastic moduli; density	S	M	S	S	X	P	X	X	P	P
Resistivity	Resistivity	P	X	X	P	S	P	X	P	M	S
Spontaneous potential (SP)	Potential differences	X	X	X	X	P	X	X	X	X	X
Electromagnetics (EM)	Conductivity; inductance	M	X	X	P	S	S	P	S	P	S
Ground penetrating radar	Permittivity; conductivity	M	X	X	X	M	S	M	X	S	S
Gravity	Density	X	X	X	X	X	P	X	X	X	X
Magnetics	Magnetic susceptibility	X	X	X	X	X	M	P	X	X	X

Key to Techniques and Applications		
Technique Applicability	Applications	
P – primary technique	1 – depth to bedrock	6 – location of mine workings or other subsurface voids
S – secondary technique	2 – rippability of rock and hard soil	7 – abandoned well detection
M – may be used but probably not the best approach	3 – elastic properties of coal refuse, soil and rock	8 – variations of coal refuse composition
X – not applicable	4 – hydrogeological investigations	9 – location of faults/fractures/ geologic structures
	5 – location of seepage pathways in a dam	10 – determination of soil/bedrock stratigraphy

(ADAPTED FROM SIRLES, 2006)

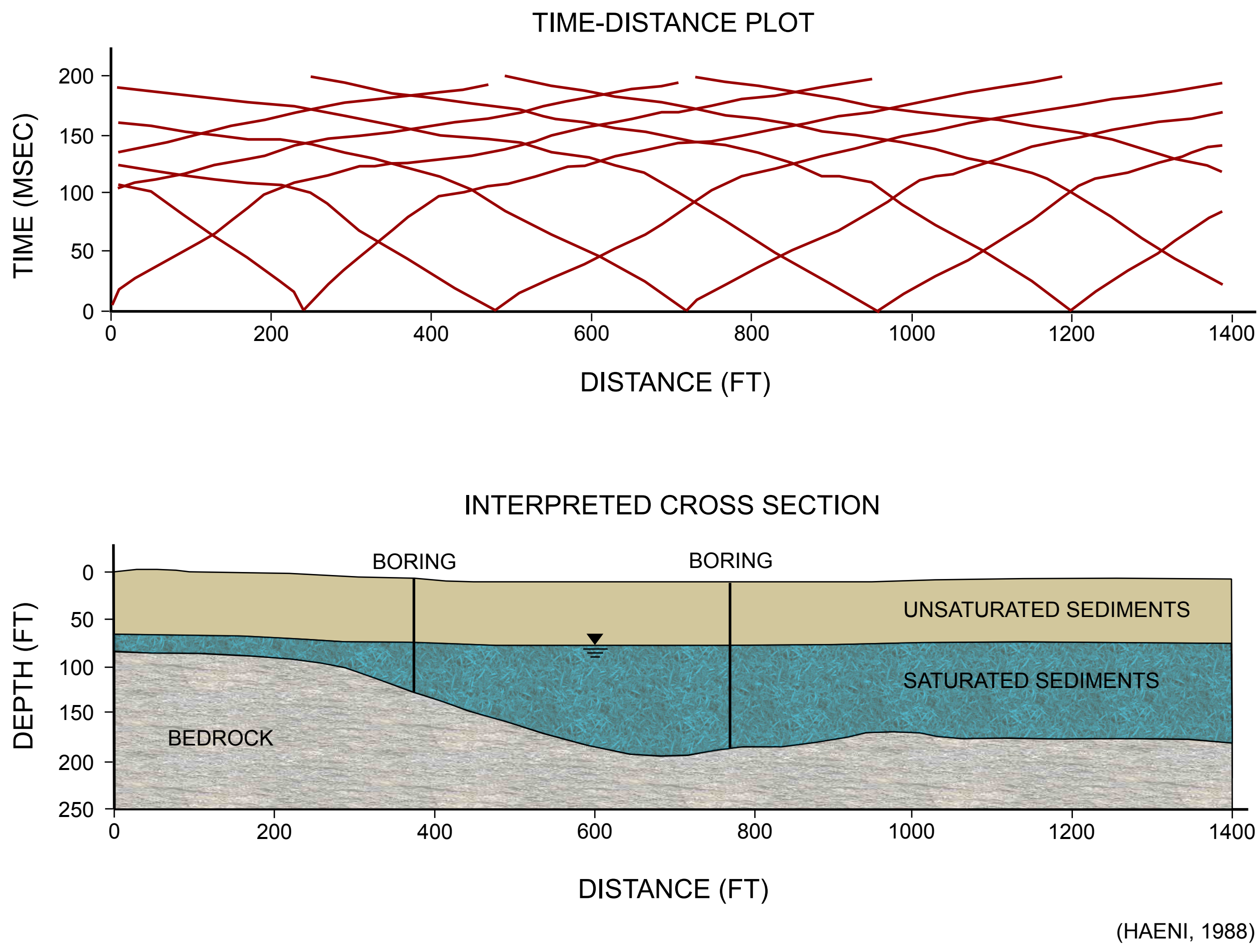


FIGURE 6.15 TYPICAL SEISMIC REFRACTION DATA WITH INTERPRETATION

The length of the spread depends on the required depth of penetration. As a rule of thumb, the length of the spread needs to be at least four times the depth of interest. Seismic "noise" originates from ambient vibrations that can be caused by sources such as traffic or wind. Obtaining multiple recordings from the same location and summing (stacking) the results or covering the geophones with sand bags can reduce the effect of this type of interference. Improved results with the seismic refraction technique can also be obtained when multiple refractions from the same refractive interface generated by multiple shots are received at a given geophone. This multiplicity of data is obtained by using a single geophone spread with multiple shot points. Commercial engineering seismographs designed for the seismic refraction technique usually allow for simultaneous recording from spreads with either 12 or 24 geophones, although some commercial equipment will allow for recording with 96 or more geophones.

There are several steps in the processing and interpretation of seismic refraction data. Firstly, it is necessary to pick the first arrival time for P-wave analysis and to tabulate this information. If the purpose is to obtain the S-wave velocity, then it is necessary to pick the onset of the S-wave arrival, which is more difficult than picking the P-wave arrival, because the S-waves lie within the wave train of the P-wave. Analytical procedures for processing the data commonly use either the generalized reciprocal method (GRM) or the delay-time method for the inversion and interpretation of refraction data. Both methods are suitable for resolving multilayer profiles with structural complexities (dips up to about 20 degrees). The acquisition of redundant data optimizes the accuracy of both interpretive techniques. The end result is a cross section of the ground with velocities assigned to each layer, as depicted in Figure 6.15.

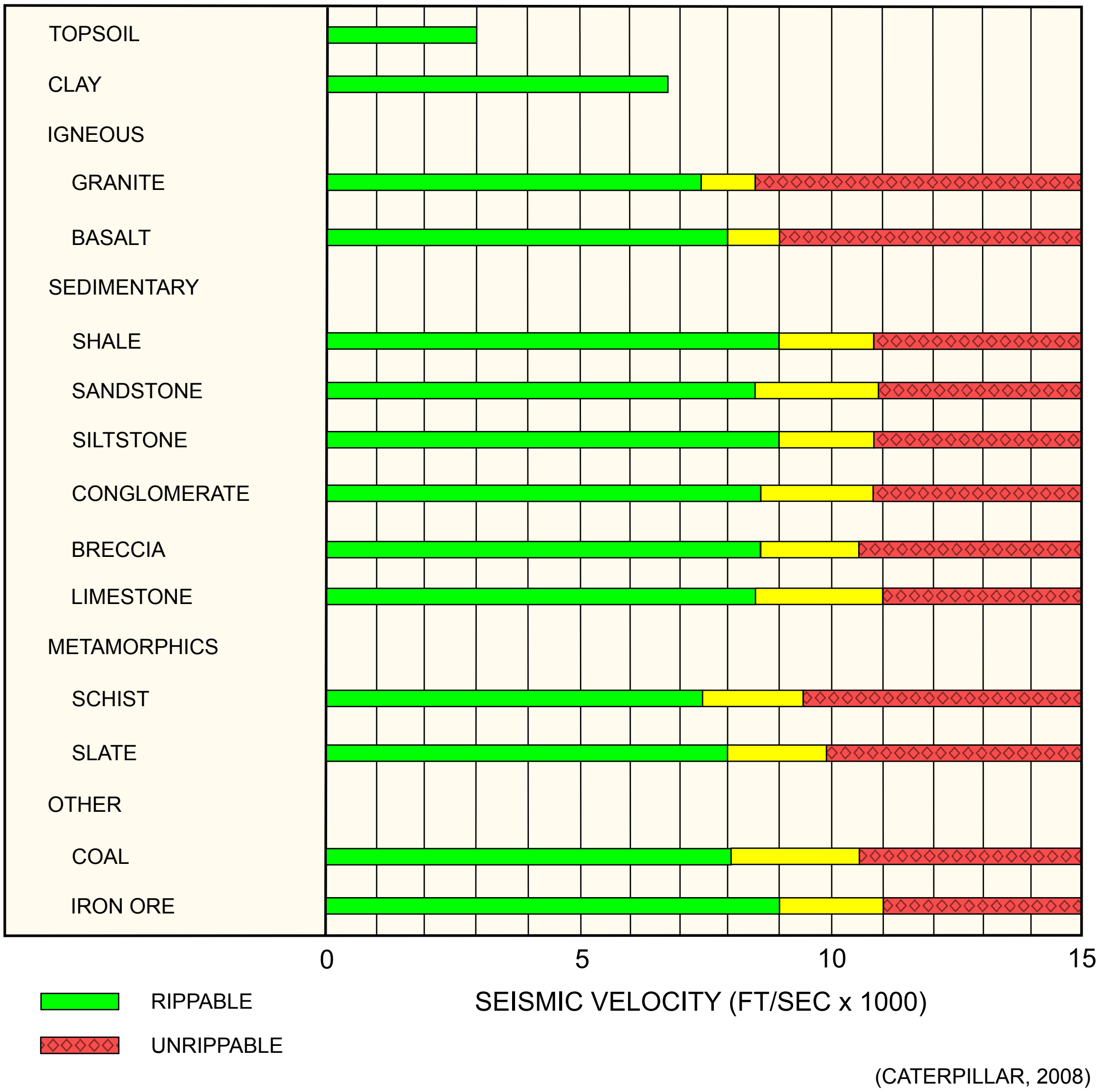


FIGURE 6.16 CORRELATION OF RIPPABILITY WITH P-WAVE VELOCITY FOR CATERPILLAR D9

When S-wave arrivals are picked from a seismic refraction record, it is possible to use the S-wave velocity to calculate the elastic properties of the identified layers. In practice, it can be difficult to identify S-waves, even when a horizontally-polarized source and horizontal geophones are used. Although there are numerous published examples (Johnson and Clark, 1992; Ellefsen et al., 2005) of the successful calculation of S-wave velocity from refraction data, in practice the most reliable methods for measuring elastic properties of the subsurface are from crosshole or downhole surveys, as discussed in Section 6.4.4.2.

6.4.4.1.2 Seismic Reflection

Application of the seismic reflection technique involves measuring the travel time required for a seismic wave generated at or near the surface (P-wave or S-wave, depending on the survey setup) to return to surface or near-surface geophones after reflection from acoustic interfaces between subsurface layers (Figure 6.17). Seismic reflection is the most powerful of all geophysical techniques for mapping subsurface layering and is by far the most commonly applied method for oil and gas exploration. It is also the most sophisticated of all geophysical methods and requires highly specialized equipment and processing software for its successful application. For this reason, the tech-

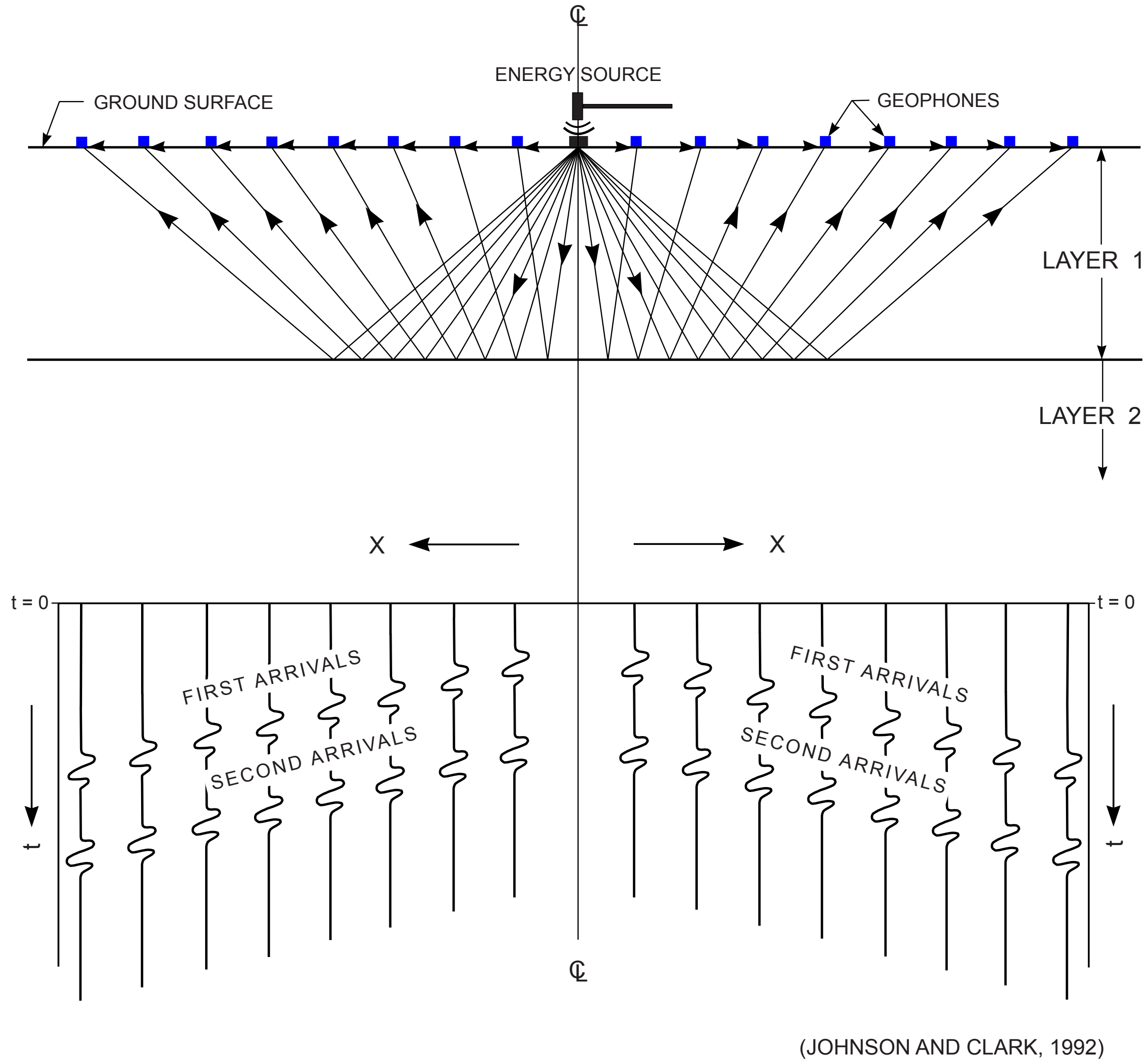


FIGURE 6.17 SEISMIC REFLECTION PRINCIPLE AND SCHEMATIC OF REFLECTION DATA RECORD

nique requires highly experienced practitioners, and no attempt has been made herein to describe the details of the data acquisition, processing and interpretation. Seismic reflection is not commonly used in environmental and engineering projects because of its relatively high cost. Contrary to most other geophysical methods, shallow seismic reflection studies are more expensive than the deep surveys conducted for oil and gas exploration because of the need for closely spaced measurements. Nevertheless, the method offers the potential for defining subsurface structure better than other methods. In cases where it is important to know the location of faults or other lithologic breaks or when the target is a deep abandoned mine working, the high-resolution seismic reflection technique may be the only practical means to obtain the desired data.

Seismic reflection has been applied to mapping the continuity of coal seams in advance of longwall mining, particularly in Europe where mines are commonly at depths greater than 1000 feet and it is difficult and expensive to characterize a coal seam with borings (D'Appolonia, 1982). The method has also been successfully applied to the mapping of mine voids (Clark et al., 1994; Johnson et al., 2002), but the experience base is limited and few practitioners are equipped to properly conduct this type of survey. An example of a high-resolution seismic reflection survey performed over shallow mine workings is presented in Figure 6.18.

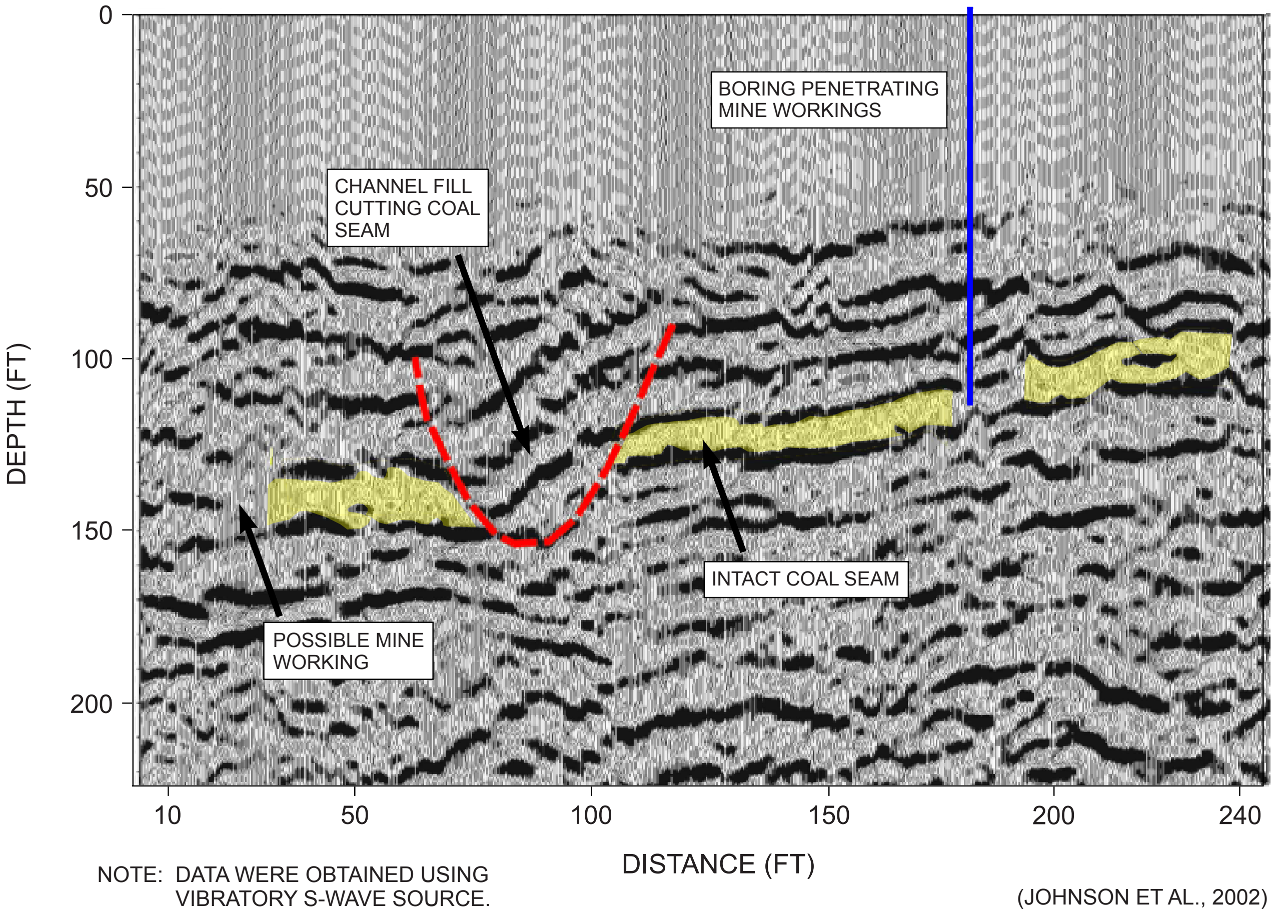


FIGURE 6.18 SEISMIC REFLECTION SURVEY PROFILE OVER ABANDONED COAL MINE WORKINGS

6.4.4.1.3 Electrical Resistivity

The purpose of electrical resistivity surveying is to determine the subsurface resistivity distribution by making measurements at the ground surface. From these surface measurements, the true resistivity of the subsurface can be estimated, and variations or anomalies in the observed resistivity may indicate limits of surface deposits such as coal refuse, the bedrock surface, or flooded mine workings. Resistivity is typically described in units of ohm-meters or ohm-feet. Ground resistivity is affected by various physical parameters such as the mineral and fluid content, porosity, and the degree of saturation.

The measurement of electrical resistivity is normally performed using four electrodes, two that induce current into the ground and two that measure potential difference (voltage). Figure 6.19 provides some examples of electrode configurations commonly used for electrical resistivity measurements. Electrical resistivity surveys have been performed for many decades as part of hydrogeological, mining and geotechnical investigations, but the use of this technique has recently increased due to improvements in both data acquisition and data processing technologies. Multi-electrode systems have greatly improved the efficiency of data acquisition, as measurements can now be made automatically without moving the current insertion and voltage measurement points. Also, the DC resistivity method had been limited by the need to perform complex calculations to model subsurface electrical properties. With the availability of high-speed personal computers and improved 2D and 3D processing software (Gan; 2004, 2005), the technique has seen increased interest from the mining industry, including as a means for detection of subsurface openings.

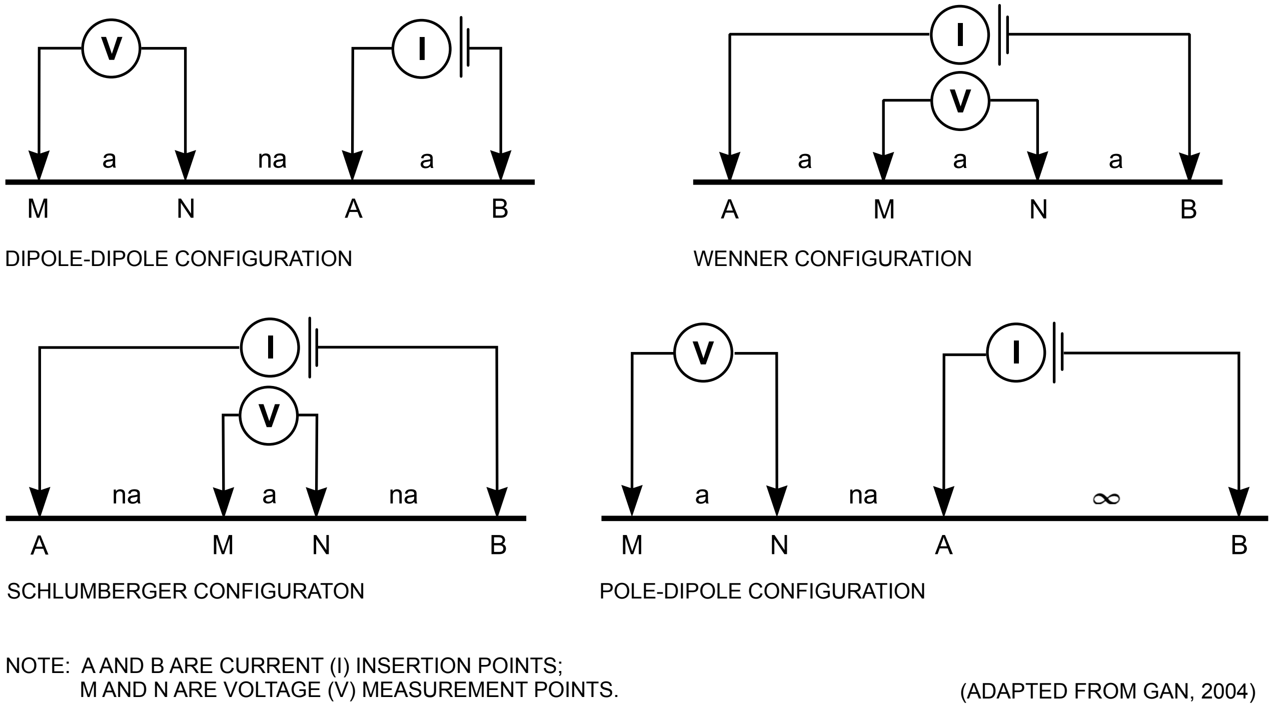


FIGURE 6.19 COMMONLY USED ELECTRODE CONFIGURATIONS FOR GROUND ELECTRICAL MEASUREMENTS

At coal refuse disposal sites, electrical resistivity surveys have proven useful for estimating the amount of coal refuse disposed at existing facilities by enabling location of the base of the refuse. Furthermore, the method also has the potential to differentiate zones with varying physical characteristics related to coal content within the coal refuse. An example of the application of electrical resistivity to generate profiles across a fine coal refuse deposit is presented in Figure 6.20. In the figure, the depth to the base of the coal refuse, which is in contact with bedrock, is clearly visible (yellow line), and variations in resistivity within the fine coal refuse can be attributed to physical differences such as the level of coal content.

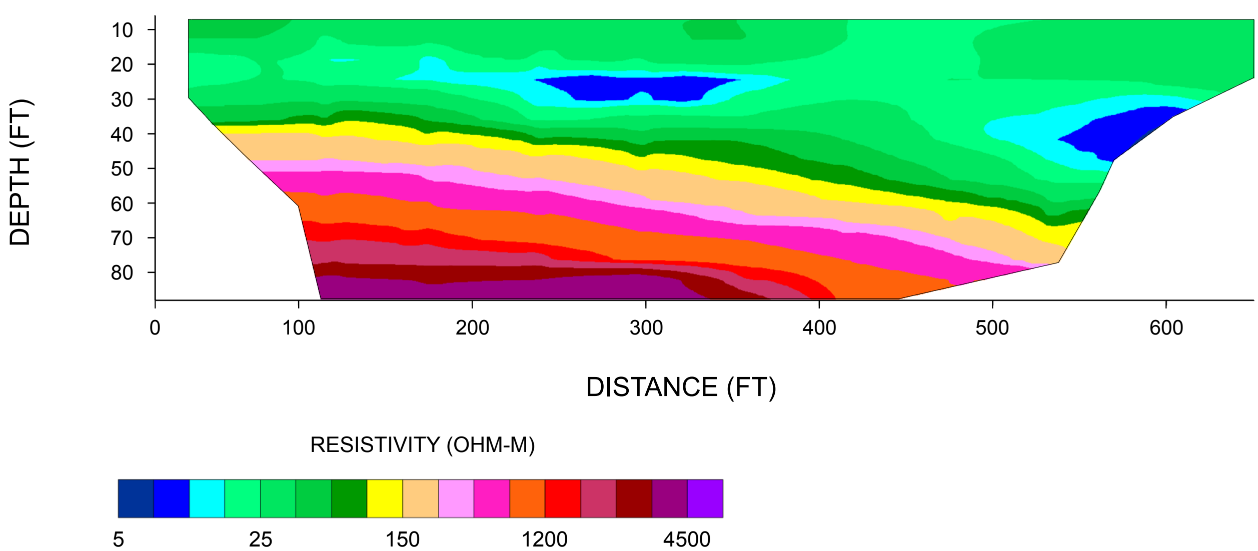


FIGURE 6.20 RESISTIVITY PROFILE ACROSS FINE COAL REFUSE IMPOUNDMENT

Another application of the electrical resistivity method at coal refuse sites is the detection of abandoned mine workings, as shown in Figure 6.21. The ability to detect voids is enhanced when the void has a physical contrast with the surrounding rock. If the void is empty (no water), it will be difficult to detect with electrical measurements. Air does not transmit an electrical current, and, unless the coal has an unusually low resistivity, it may be difficult to distinguish a void. The resistivity contrast between flooded mine voids and typical coal will approach two orders of magnitude (Johnson, 2003), thus allowing for detection of mine workings as resistivity lows. Project experience with electrical resistivity demonstrates that commercially available technology can be effective, especially for the detection of flooded mine workings at depths up to about 100 feet (Figure 6.21). D'Appolonia (2006) conducted a demonstration project for MSHA to illustrate the application of the method at the perimeter of an impoundment where abandoned workings in the 40- to 60-foot-depth range contained limited water. For deeper workings, the method has the potential to be effective, but theoretical models and practical experience indicate that the target size/depth ratio needs to be favorable and that the length of the resistivity profile required for acquiring deep images is often limited by surface interference. Therefore, the method is usually most effective for mine subsidence applications.

6.4.4.1.4 Spontaneous Potential (SP)

The spontaneous or self-potential (SP) method consists of measuring naturally occurring electrical potentials (voltage differences) in the subsurface. One of the sources of these electrical potentials is the movement of water through a porous medium, which produces electro-filtration or streaming potentials. These potentials can be used for the evaluation of seepage (Figure 6.22). As water flows through a capillary system, it collects and transports positive ions from surrounding materials. The positive ions accumulate at the exit point of the capillary, leaving a net positive charge. The untransported negative ions accumulate at the entry point of the capillary, leaving a net negative charge. If the streaming potentials developed by this process are of sufficient magnitude to be measured, the entry point and the exit point of zones of concentrated seepage can be determined due to the negative and positive (respectively) SP anomalies.

SP is measured with a pair of non-polarizing electrodes and a high-impedance voltmeter. One of the electrodes is placed in the ground at a convenient location and remains in place throughout the survey. This is referred to as the "base" electrode. It is connected to a multi-meter via an insulated, single-conductor wire mounted on a reel. This wire may be hundreds of meters long. The second

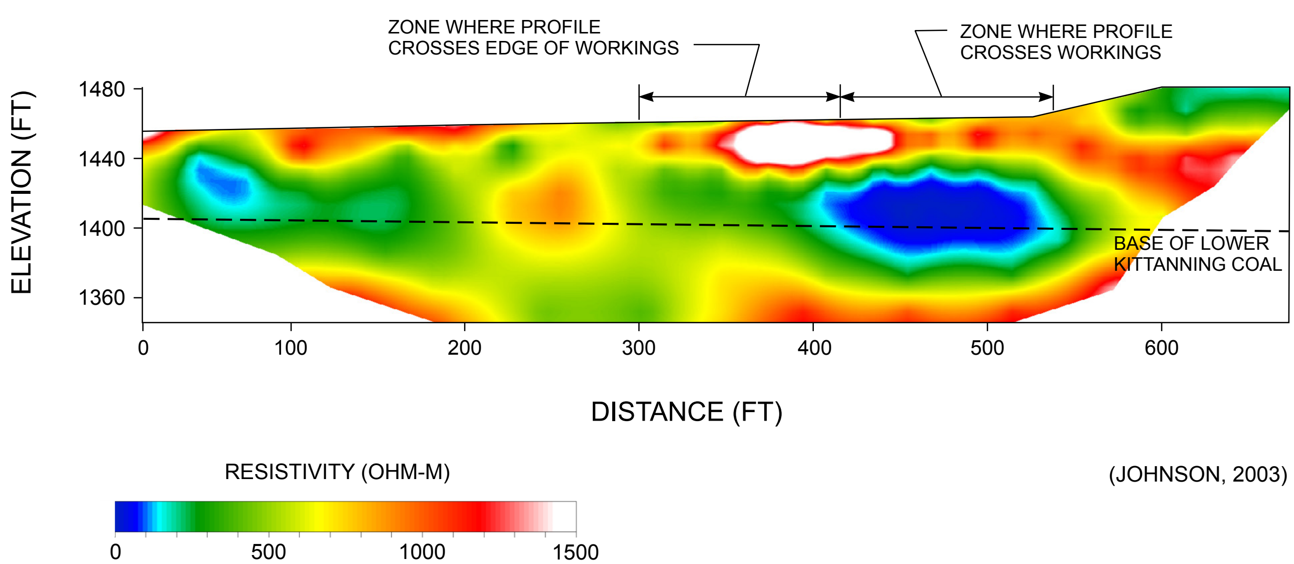


FIGURE 6.21 ELECTRICAL SURVEY OF FLOODED MINE WORKINGS

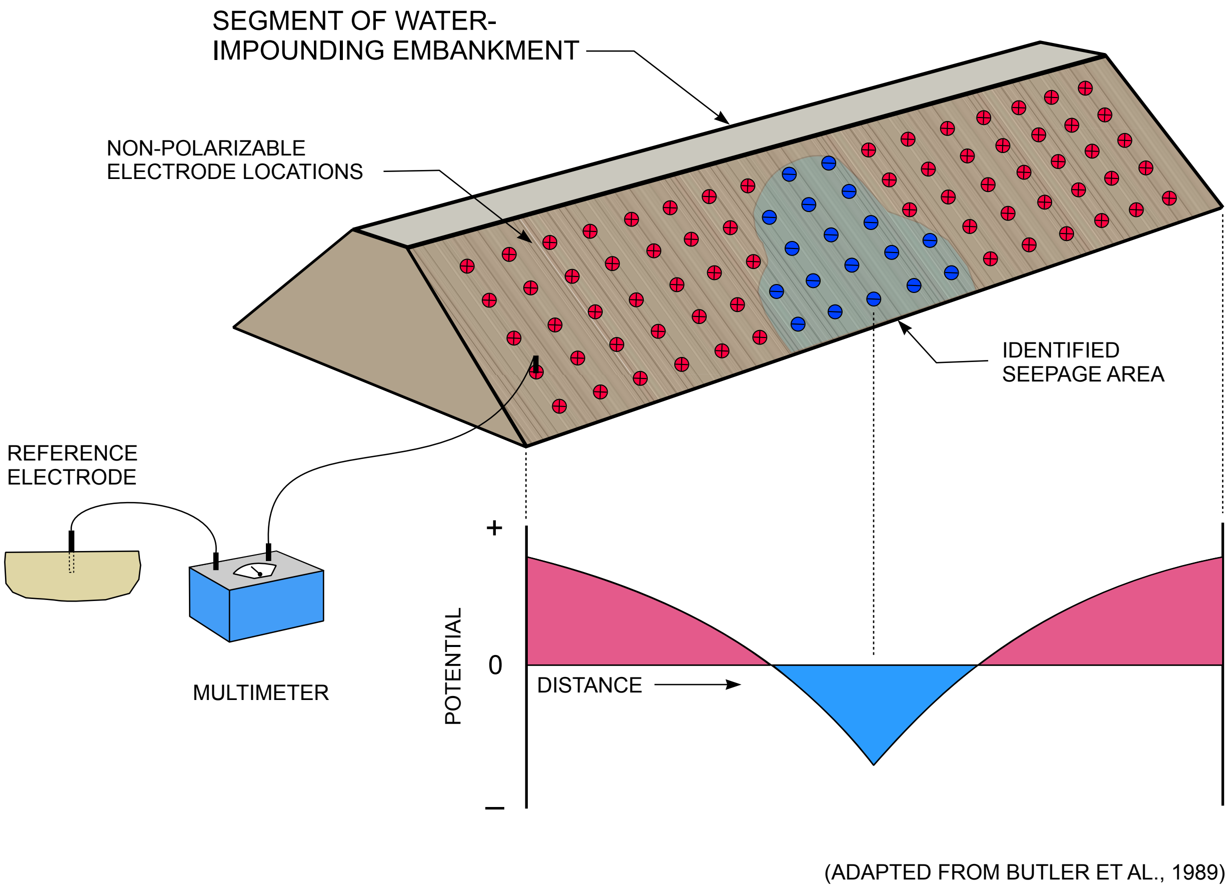


FIGURE 6.22 ILLUSTRATION OF USE OF SP METHOD TO IDENTIFY AN AREA OF SEEPAGE

electrode, or "measuring electrode," the reel, and the multi-meter are then moved from point to point in a planned grid. At each point of the grid, the electrical potential between the base and the measuring electrode is recorded.

If the potential differences are plotted in profile or contoured to identify zones with negative potential, areas of seepage can be identified. Additional information and examples of SP surveys used to locate seeps in embankment dams and tailings impoundments are provided by Butler et al. (1989), Butler et al. (1990), Bérubé (2004), Song et al. (2005), and Mainali (2006). Recent advances in the SP technique allow for predictive modeling of the SP anomalies associated with seepage, enhancing the interpretability of SP results (UBC-GIF, 2005; Bérubé, 2004).

6.4.4.1.5 Electromagnetics (EM)

Similar to electrical resistivity measurements, electromagnetic methods (EM) allow mapping of the distribution of subsurface electrical properties, except that the EM methods are designed for measurement of variations in conductivity, not resistivity. Resistivity and conductivity are different parameters related to the same physical property and are simply the inverse of one another. As noted above, the unit most commonly used to measure ground resistivity is the ohm-meter (ohm-m). The term "mho" reflects its inverse relationship to the ohm, but was discontinued in favor of the term "siemen" (symbol S) in the late 1970s. The corresponding unit of conductivity is the inverse of an ohm-meter, referred to as a mho (or siemen) per meter, or S/m. The most common unit of conductivity is the μS/cm, which is 0.0001 S/m. With this conversion, 10,000 μS/cm = 1 S/m = 1 ohm-m.

An advantage of all EM systems as compared to the resistivity method is that it is not necessary to insert electrodes in the ground and thus the surveying is more rapid. Disadvantages of EM methods are: (1) they are generally not as good as the DC resistivity method in resolving variations of electrical properties with depth and (2) they are more subject to cultural interference from electrical lines and metallic objects. For these reasons, EM methods are most commonly used to rapidly measure lateral variations of soil electrical properties, as well as to delineate the distribution of metal objects.

Electromagnetic (EM) techniques can be grouped into active methods, where an active EM signal is induced in the ground by human activity, and passive systems, where measurements are made of natural variations of the earth's EM field. Active systems are further grouped into frequency domain and time domain. Passive systems include very low frequency (VLF), and magnetotelluric methods. McNeill (1990) provides a discussion of various EM techniques.

EM methods have potential application for characterization of coal refuse sites (e.g., mapping abandoned workings) as long as the workings are flooded. For example, time-domain EM (TDEM) measurements have been used to map flooded workings, as shown in Figure 6.23. Where this technique has been attempted over workings that are not flooded, the method was less successful (MSHA, 2008). EM techniques that measure bulk ground conductivity are commonly used at operating or abandoned refuse disposal facilities to determine the migration of contaminated groundwater or to delineate the extent of waste deposition. EM techniques can also be used to characterize variations in the physical properties of existing coal refuse deposits related to coal content. Results of this type of survey with a commonly-used conductivity meter (Geonics EM-31) over an existing coal refuse deposit are shown in Figure 6.24. The plan location of the resistivity profile in Figure 6.20 is shown in Figure 6.24. In this example, the EM survey provides mapping of the near-surface horizontal variations of the coal refuse, while the resistivity profile in Figure 6.20 shows vertical variations.

6.4.1.1.6 Ground Penetrating Radar (GPR)

Ground penetrating radar (GPR) has evolved over the past two decades into one of the most commonly applied techniques for imaging the shallow subsurface. The method offers the highest resolution of geophysical techniques commercially available today. In many cases, the time required for the acquisition of GPR profiles is minimal, and subsurface profiles can normally be generated in real time, making this tool very cost-effective. GPR works best in non-conductive soils, such as dry sand or sand saturated with fresh water.

The typical result of a GPR survey is a profile that presents radar wave amplitude as a function of distance along the line and two-way travel time. To determine the depth to a reflector, it is necessary to know the average propagation velocity from the ground surface. The velocity of a radar pulse in an earth material is dependent on the relative dielectric constant (ε_r) of the material according to the following relationship:

$$V = c / (\varepsilon_r)^{0.5} \tag{6-5}$$

where:

V = velocity in propagating material (m/sec)

c = speed of light (m/sec)

ε_r = relative dielectric constant (dimensionless)

This velocity can sometimes be estimated from the characteristics of the subsurface lithology. Table 6.25 presents typical velocities in terms of two-way travel time (nanoseconds/meter) for various earth materials along with their approximate relative dielectric constants.

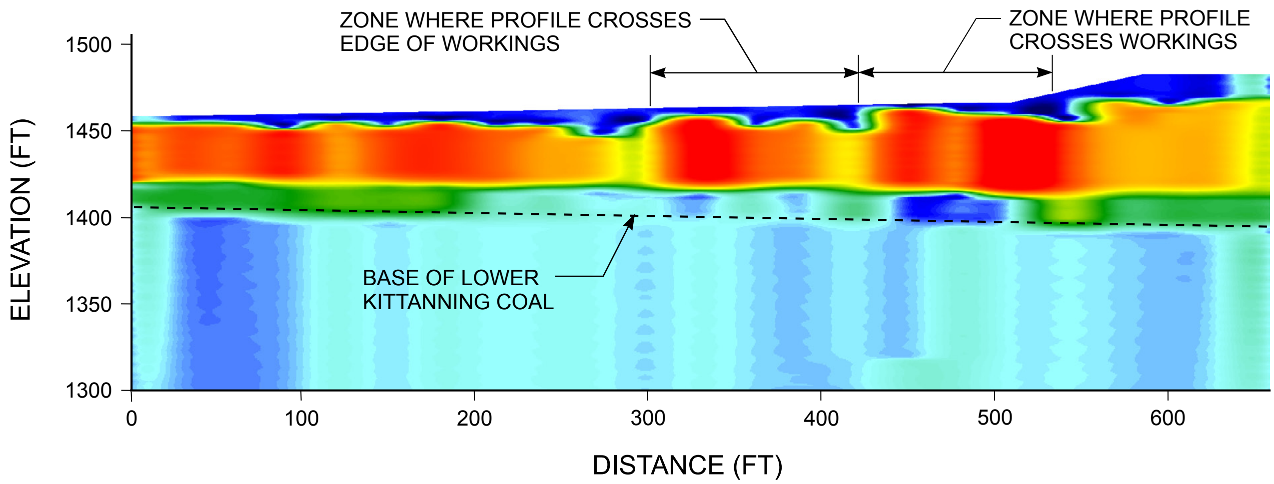


FIGURE 6.23 DELINEATION OF FLOODED MINE WORKINGS WITH TDEM METHOD

Until the advent of commercial systems with separate transmitting and receiving antennas, depth estimation based on subsurface material properties or from observations from reflectors of a known depth was the only means to interpret a GPR profile. Modern systems with the ability to record reflections at varying distances from the transmitting antenna allow for the calculation of the subsurface velocity profile by means of a normal moveout (NMO) correction based on hyperbolic reflections from subsurface features.

Depth of penetration depends on the selection of an appropriate antenna frequency. An antenna frequency of one gigahertz would be suitable for mapping rebar in concrete, but would only have at most a few feet of penetration in typical soil. Most GPR surveys in soil use antennas with frequencies

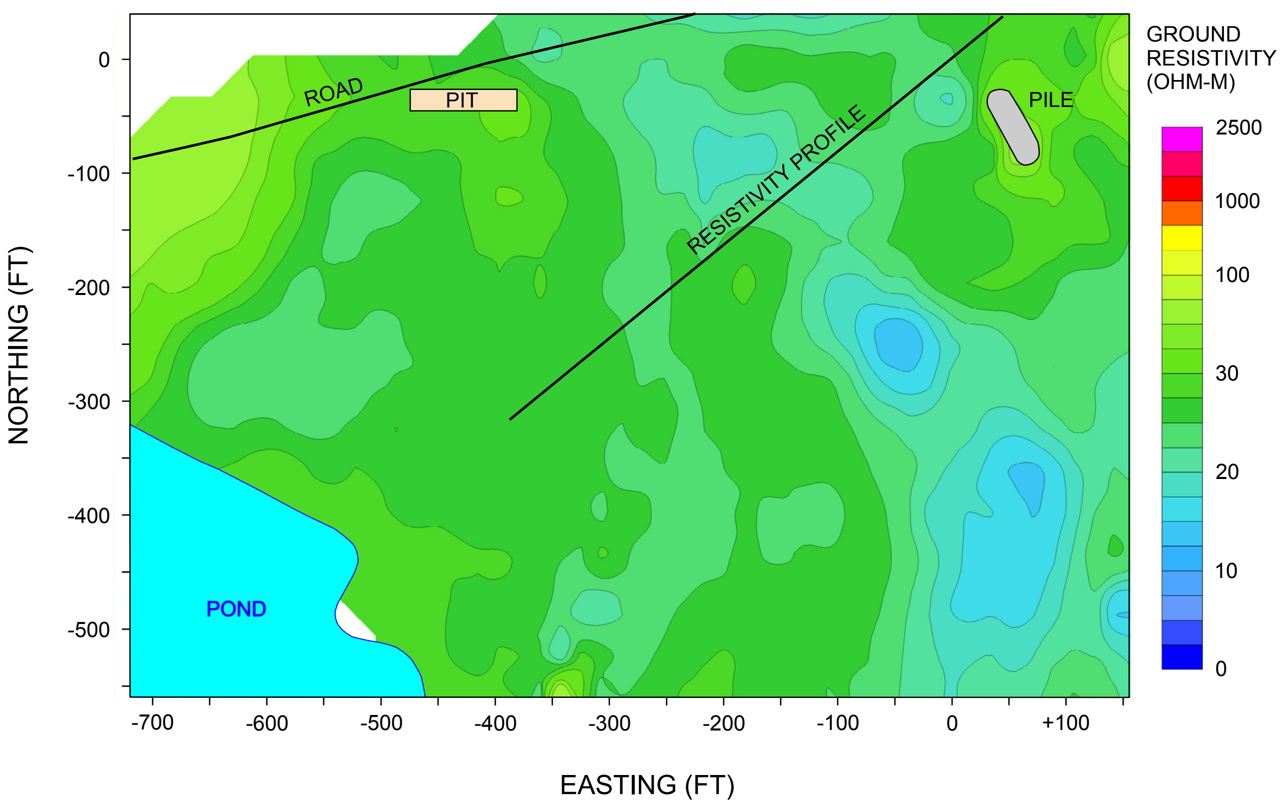


FIGURE 6.24 RESISTIVITY OF COAL REFUSE FROM EM-31 MEASUREMENTS

TABLE 6.25 PHYSICAL PROPERTIES AND TYPICAL GROUND PENETRATING RADAR VELOCITIES FOR COMMON EARTH MATERIALS

Material	Approximate Conductivity (mS/m)	Approximate Relative Dielectric Constant ε_r (dimensionless)	Two-Way Travel Time (sec $\times$ 10^{-9}/m)
Air	0	1	6.6
Fresh Water	10^{-1} – 30	81	59
Fresh-Water Ice	10^{-1} – 10	4	13
Permafrost	10^{-2} – 10	4 – 11	13 – 15
Limestone	10^{-6} – 1	6 – 8	22
Granite	10^{-6} – 1	5.6 – 8	18.7
Dry Sand	10^{-4} – 1	4 – 6	13 – 16
Saturated Sand (fresh water)	10^{-1} – 10^{2}	30	32 – 36
Saturated Silt (fresh water)	10 – $l0^{2}$	10	21
Saturated Clay (fresh water)	10^{2} – 10^{4}	8 - 25	18.6 – 23
Average “Dirt”	10^{-1} – 10^{2}	16	20 – 30

(BENSON ET AL., 1984)

between about 100 and 400 MHz, with the greatest penetration achieved with the 100-MHz antenna, but with a substantial loss of resolution as compared to the 400-MHz antenna.

Another factor affecting the depth of penetration of the GPR signal is attenuation. Attenuation is caused by spreading and scattering losses, as well as electrical losses. Scattering and electrical losses are due primarily to the conductivity of the subsurface materials, which in soils relates mainly to clay and moisture content. In dry sand, penetration can reach as much as 50 to 70 feet. In wet, saturated clay penetration may be as little as 3 to 7 feet.

The main limitation of the GPR technique is depth of penetration under conditions commonly encountered in areas with coal workings. The soils commonly encountered in coal mining areas are clays weathered from the claystones associated with the sedimentary sequences that include the coal, and these soils can severely restrict the effective penetration of the radar waves. Thus, use of GPR is generally limited to the identification of near-surface features such as buried waste, pipes, etc. Nevertheless, where abandoned mine workings are shallow, GPR can sometimes be used to detect these workings. An example of a GPR record with identified mine workings is shown in Figure 6.25.

6.4.4.1.7 Gravity

At mine sites the gravity method can detect shallow abandoned mine workings by measurement of minute changes in the earth's gravity field resulting from the lack of near-surface mass associated with mine openings. The measurement of the gravity field for this application is referred to as microgravimetry and requires the use of specialized gravimeters with a sensitivity of one microgal (approximately one billionth of the earth's gravity field). An air-filled mine void would in theory be detectable with commercial equipment at a depth of about 30 feet. In practice, it is time-consuming to acquire the data, and accurate elevation control is needed, and it is desirable to have a topographic survey crew accompany the geophysicist to measure the precise elevation of the instrument at each reading point. For a target as shallow as 30 feet, the surface width of the gravity anomaly is about 100 feet. Thus, a survey requires a significant amount of accessible space, which is often not avail-

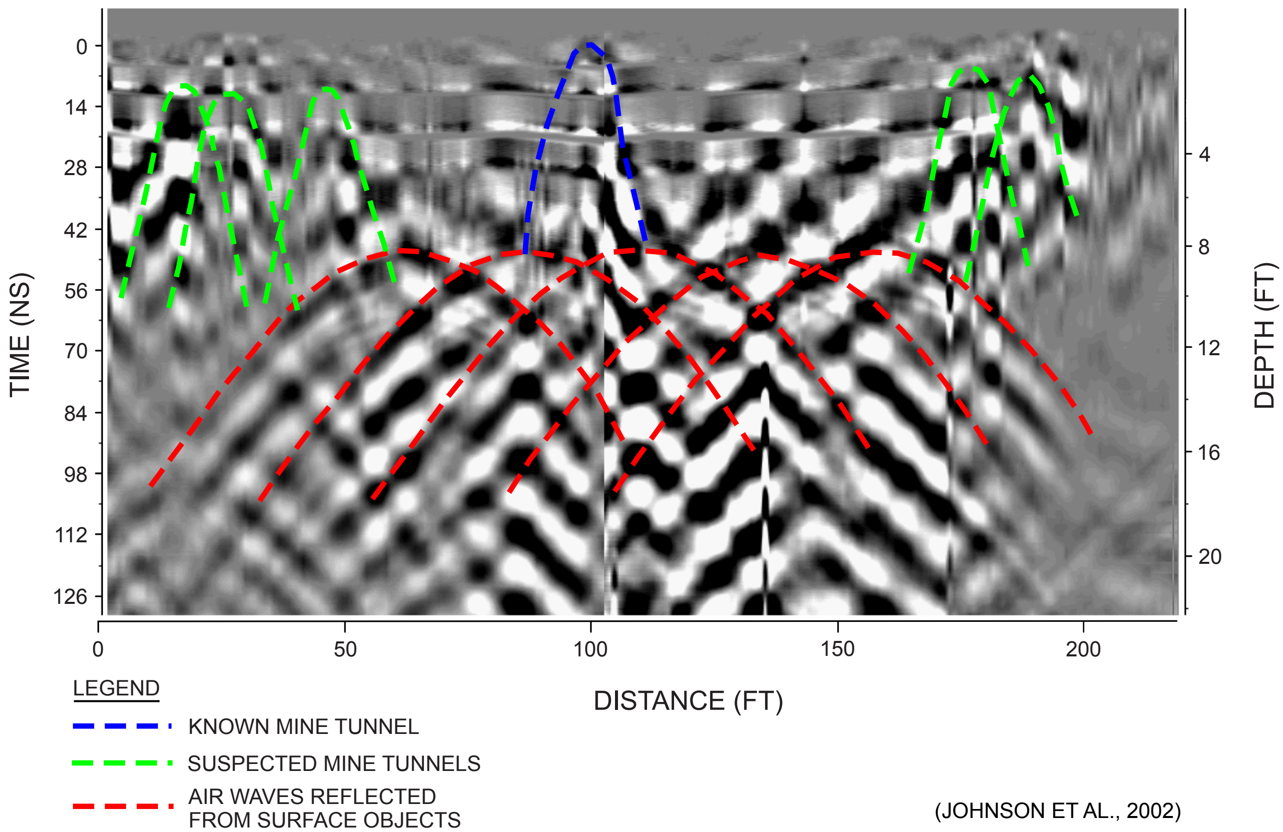


FIGURE 6.25 GPR RECORD OF SHALLOW COAL MINE WORKINGS

able. Furthermore, it is often difficult to correct the gravity data for variations caused by surrounding topography, instrumental drift, and elevation. In particular, micro-topographic changes can significantly affect gravity readings. Unless the target is in a flat, open area and the depth does not exceed about 40 to 50 feet, the gravity method will probably not be practical. Nevertheless, if the mine workings are expected to be very shallow and air-filled, the gravity method is one of the few geophysical methods that can provide conclusive evidence of the presence of a mine void. A theoretical gravity response over an air-filled mine void is presented in Figure 6.26.

6.4.4.1.8 Magnetics

The primary application of the magnetic method in a coal refuse environment is the detection of metal, including abandoned metal well casings. Measurements are made by an instrument called a magnetometer, and the unit of magnetic intensity is the nanotesla (n*T*), sometimes referred to as a gamma. Differences in the normal value of the earth's magnetic field correspond to magnetic anomalies that can be measured with a magnetometer. Surveys for well casings can be conducted from the ground or from the air, as previously noted. As shown in Figure 6.27, well casings produce very strong anomalies, detectable even when the magnetometer is located at an elevation of 250 feet above the well casing. The potential for detecting abandoned mines is minimal, unless mine openings are associated with metal, as might be the case if old mine rails are present. Coal has a low magnetic susceptibility when compared to most other rocks. A void in a coal seam, therefore, will not produce a significant disturbance to the earth's natural magnetic field, but a sensitive magnetometer can detect old mine rails at a depth of several tens of feet.

6.4.4.2 Borehole Geophysical Techniques

Borehole logging includes numerous geophysical techniques involving the lowering of sensing devices into a borehole and continuously recording physical parameters associated with the sur-

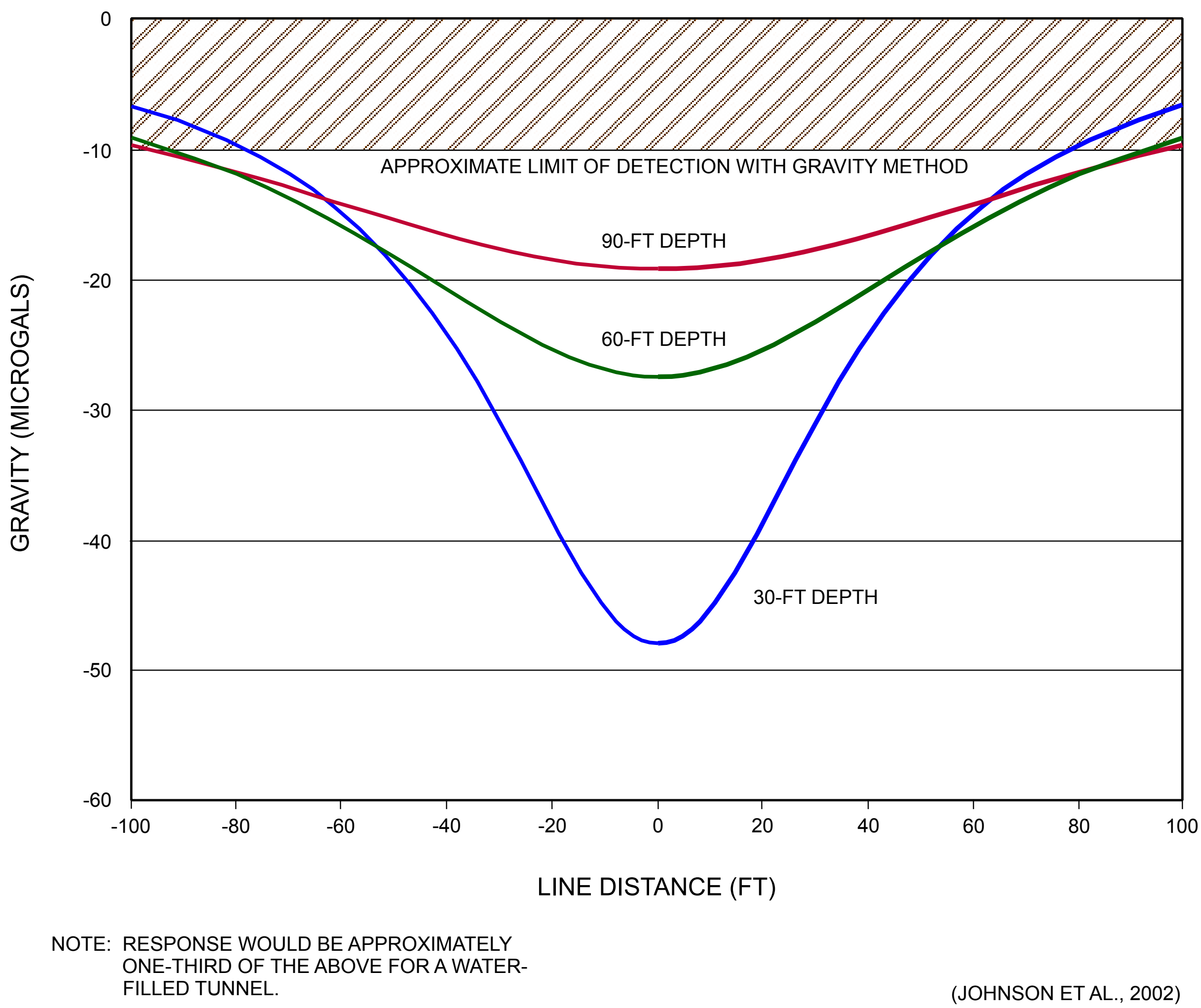


FIGURE 6.26 THEORETICAL RESPONSE OF GRAVITY GEOPHYSICAL METHOD OVER 20-FOOT-DIAMETER, AIR-FILLED TUNNEL

rounding rock, soil, pore fluids, or other physical parameters. The FHWA lists 23 borehole logging techniques on their web site.

A general grouping of the most commonly applied borehole techniques is provided in ASTM D 5753, "Standard Guide for Planning and Conducting Borehole Geophysical Logging." This general grouping of borehole geophysical techniques is shown in Table 6.26, where a division is made in terms of: (1) acoustic logs intended to determine ground variations related to seismic velocity, (2) electrical and induction logs that identify lithologic and groundwater variations on the basis of resistivity/conductivity, (3) nuclear logs that relate to variations in natural or induced radioactivity, and (4) miscellaneous techniques that define the physical characteristics of boreholes and/or voids penetrated by boreholes.

For coal refuse facilities, most of the commonly used borehole geophysical methods are suitable for general site characterization in combination with conventional drilling and sampling. Techniques with particular relevance to coal refuse disposal facilities are those that provide the S-wave velocity as a function of depth (crosshole and downhole seismic methods), because this information is useful for the evaluation of the potential for seismically-induced liquefaction. Borehole logging and techniques that have potential for characterization of abandoned mine workings (video, laser imaging, sonic imaging) are also directly applicable to coal refuse disposal facility evaluations.

A borehole technique with the potential for imaging abandoned mine workings is borehole GPR. Mine voids within approximately 10 ft can be detected from borehole GPR. Research has shown that crosshole GPR tomography can identify tunnels, but this is not a commonly applied technique and the difficulties of this technique in mapping coal mine voids is well described by the Colorado School of Mines in an experimental mine detection study for MSHA (CSM, 2007).

Information on commonly applied general borehole geophysical techniques can be found in USEPA (1993), USACE (1995a), Keys (1997), Sabatini et al. (2002), Wightman et al. (2003), and Sirles (2006). The following discussion focuses on techniques for measuring S-wave velocity that are primarily used for determining the seismic properties of coal refuse and soils for liquefaction analyses, but these techniques may also be useful for characterizing mine voids.

6.4.4.2.1 Crosshole and Downhole/Uphole Seismic Surveys

Crosshole and/or downhole/uphole seismic testing in boreholes is conducted for determining soil and rock properties (P- and S-wave velocities). The information obtained from these tests can be used to compute shear modulus, Young's modulus, and Poisson's ratio for use in static/dynamic analyses. The basic relationships are as follows:

$$E = \rho V_p^2 \left[(1 + \upsilon)(1 - 2\upsilon) / (1 - \upsilon) \right] \qquad (6\text{-}6)$$

$$G = V_s^2 \rho \qquad (6\text{-}7)$$

where:

V_p = compressional (P-wave) velocity (length/time)

V_s = shear (S-wave) velocity (length/time)

E = Young's modulus (force/length^2)

G = shear modulus (force/length^2)

ρ = mass density of soil (mass/length^3)

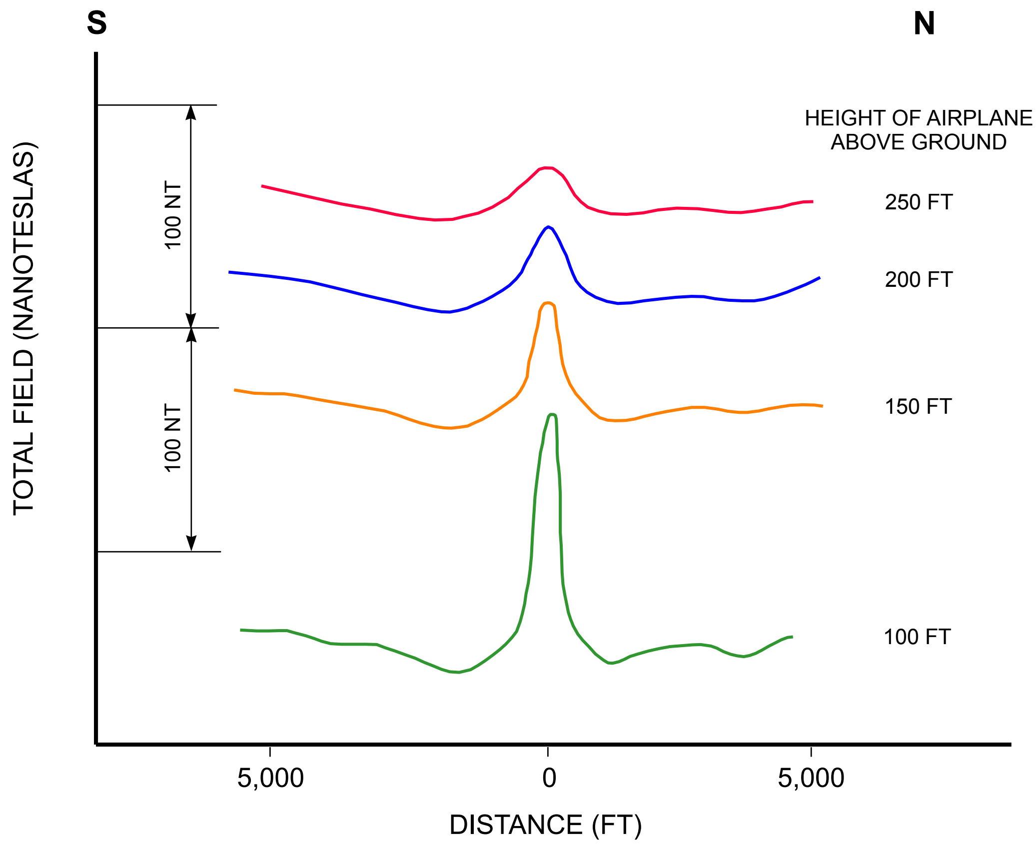
υ = Poisson's ratio of soil (dimensionless)

If both V_p and V_s are known, the Poisson's ratio of the soil can be determined from the following relationship between E and G:

$$G = E / 2(1 + \upsilon) \qquad (6\text{-}8)$$

With borehole seismic surveys, one or more boreholes are drilled into the soil to the desired depth of exploration. Wave sources and/or receivers (borehole geophones normally oriented to record both horizontal and vertical components of wave motion) are then lowered into the boreholes. There are three basic approaches to borehole seismic surveys:

- Crosshole Survey – In a crosshole survey, the energy source is located in one borehole and detectors are placed in another borehole at the same depth as the energy source. The energy source is usually a mechanical pulse instrument composed of a stationary part and a hammer. The pulse instrument is held against the side of the borehole by a pneumatic or hydraulic bladder. Travel times between the source and receivers are measured, allowing determination of wave velocities.
- Uphole Survey – Geophones are laid out on the ground surface in an array around the borehole. The energy source is set off within the borehole at successively decreas-



(FRISCHNECHT AND RAAB, 1984)

FIGURE 6.27 MAGNETIC INTENSITY RECORDED AT VARIOUS HEIGHTS ABOVE WELL CASING

ing depths starting at the bottom of the hole. The travel times from the source to the surface are analyzed to determine the wave velocity versus depth.

- Downhole Survey – In a downhole survey, the energy source is located on the surface and a detector (geophone) is placed in a borehole. The travel time is measured with the geophone placed at progressively increasing depth, and a wave-velocity profile is generated.

Crosshole seismic surveys involve measurement of the travel time of seismic energy transmitted between two or preferably three boreholes to derive information relative to the elastic properties of the intervening materials. The travel times of the seismic waves are derived from the identified first-arrivals of the P- and S-waves on the seismic trace for each shot-receiver position and are used with the known distance (s) between the shot/receiver boreholes to calculate the apparent velocities (P- and S-wave) for each depth interval. The borings are usually cased and grouted to the surrounding soil/rock. PVC casing is normally used for the tests, so that the casing is not a seismic pathway. A typical field setup for a crosshole seismic survey is shown in Figure 6.28.

Crosshole geophysical testing is described in ASTM D 4428, "Standard Test Methods for Crosshole Seismic Testing." Crosshole measurements are generally preferred to downhole measurements because they provide higher resolution and greater accuracy. However, the distances between the energy source and the detector must be measured precisely. An inclinometer survey is generally performed in crosshole test boreholes to correct the data for deviation of the boreholes from vertical. To calculate P- and S-wave velocity, the wave arrivals must be processed with a computer program that accounts for situations where the waves may be refracted between the boreholes according to Snell's Law. The data are then used to develop vertical profiles of the various elastic moduli.

TABLE 6.26 APPLICABILITY OF COMMON BOREHOLE GEOPHYSICAL METHODS

Borehole Geophysical Method	Dependent Physical Property	Application (see key below)									
		1	2	3	4	5	6	7	8	9	10
Acoustic Logs											
In-hole acoustic velocity	Elastic moduli, density	A-2	A-2	X	X	A-2	X	A-2	X	X	X
Crosshole S-wave velocity	Elastic moduli, density	A-4	X	X	X	M	X	X	X	X	M
Downhole S-wave velocity	Elastic moduli, density	A-4	X	X	X	M	X	X	X	X	M
Electric and Induction											
Spontaneous potential	Potential difference	A-2	X	X	X	A-2	X	X	X	X	X
Single-point resistance	Resistance	A-2	X	X	X	A-2	X	A-2	X	X	X
Multi-electrode resistivity	Resistivity	A-2	X	A-2	X	A-2	X	A-2	X	X	X
Induction	Conductivity	A-4	X	A-4	X	M	X	X	X	A-4	X
Nuclear											
Gamma	Gamma radiation	A-6	A-6	X	X	X	X	X	X	X	X
Gamma-gamma	Density	A-6	X	A-6	X	A-6	A-6	X	X	X	X
Neutron	Hydrogen content	A-6	X	A-6	X	A-6	X	X	X	X	X
Fluid logs											
Borehole fluid characteristics	Resistivity/ conductivity	X	A-5	X	A-5	A-5	X	X	X	X	X
Fluid flow	Velocity	X	A-5	X	X	X	X	X	X	A-1	X
Temperature	Temperature	X	A-5	X	A-5	A-5	X	X	X	A-1	X
Miscellaneous											
Borehole deviation	Inclination	X	X	X	X	X	X	X	X	A-6	X
Video	Visual characteristics	M	X	X	M	A-6	X	X	A-6	4	A-6
Laser imaging	Physical dimensions	X	X	X	X	X	X	X	X	X	A-3
Sonic imaging	Physical dimensions	X	X	X	X	X	X	X	X	X	A-2
Caliper	Physical dimensions	X	X	X	X	X	X	X	A-3	A-3	M

Applications Key

1. Lithology and correlation
2. Hydraulic Conductivity
3. Porosity
4. Fluid properties
5. Depth to groundwater
6. Bulk density
7. Rock structure
8. Borehole parameters
9. Elastic properties of coal refuse, soil and rock
10. Characterization of mine workings or other subsurface voids

Technique Applicability and Required Hole Conditions

A-1 Applicable (cased, fluid-filled hole)
A-2 Applicable (uncased, fluid-filled hole)
A-3 Applicable (uncased, dry hole)
A-4 Applicable (open or fluid-filled hole, non-conductive casing)
A-5 Applicable (screened or uncased, fluid-filled hole)
A-6 Applicable (any hole condition, but fluid must be clear for video)
M May be applicable, but probably not the best approach
X Not applicable

(ADAPTED FROM USACE, 1995)

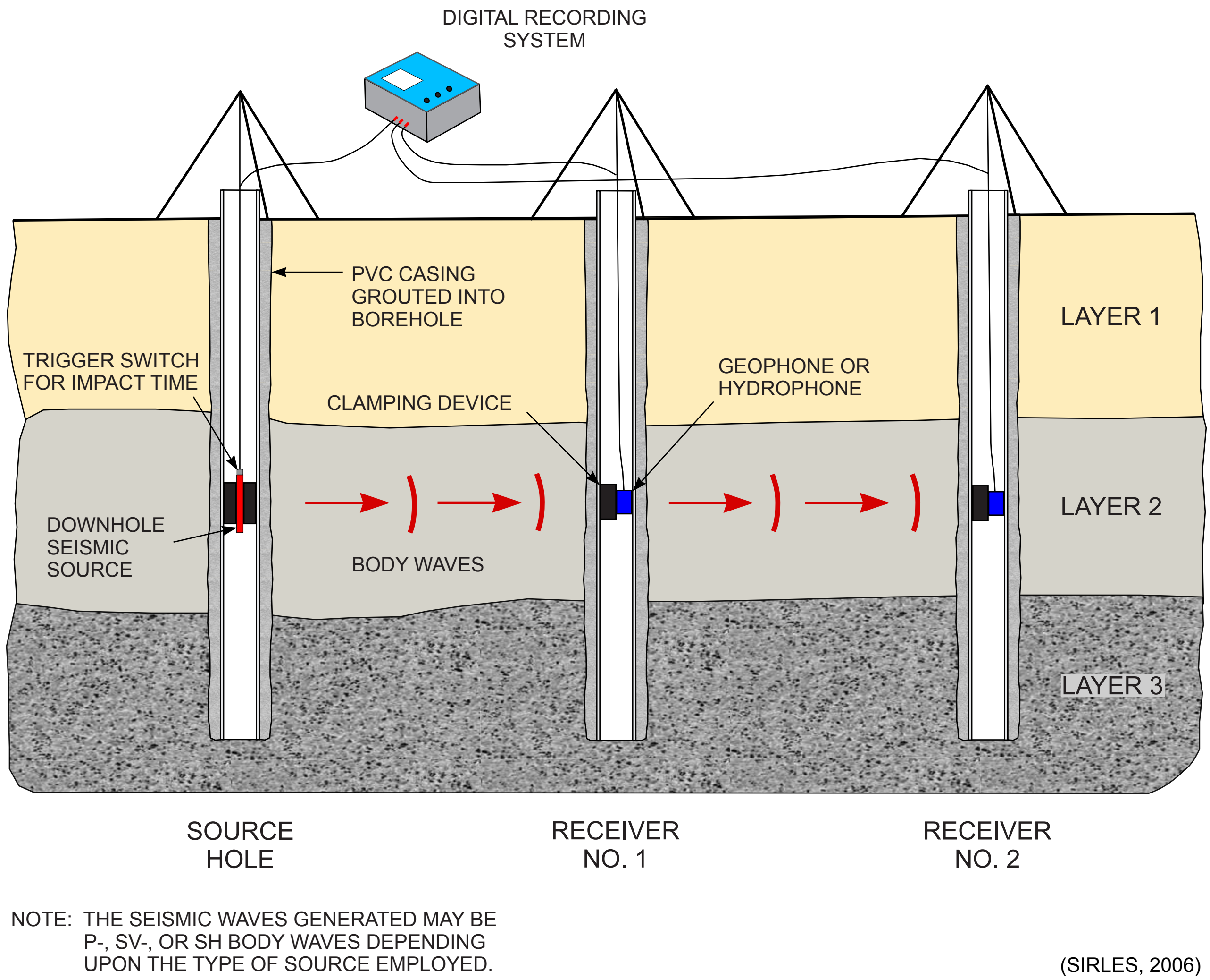


FIGURE 6.28 FIELD SETUP FOR CROSSHOLE SEISMIC SURVEY

The uphole and downhole techniques are more economical alternatives to the crosshole technique because only one borehole is required. Downhole measurements are not as accurate as crosshole measurements, especially if the layers of interest are thin. However, if critical thin layers are not present, downhole and uphole measurements may be the preferred means for determining the variation of the P- and S-wave velocities with depth. Downhole surveys are generally preferred to uphole surveys, because it is usually more practical to induce a strong seismic signal at the surface than it is in a borehole. Figure 6.29 depicts the deployment for a downhole seismic survey. In terms of analysis, the downhole or uphole methods differ from the crosshole method in that it is necessary to calculate incremental velocities on the basis of differences in travel time between geophones at varying depths rather than from direct pathways.

Seismic tomography employing surveys from boreholes can be used as a tool for detecting abandoned mines. A vertical seismic profile (VSP) can be developed by deploying geophone sensors in a borehole and a seismic source at multiple surface locations. An emerging technology whereby a drill bit is used as a downhole source and seismic waves are recorded at the surface is referred to as reverse vertical seismic profile (RVSP). Although these techniques are generally available and are used in the oil and gas industry, they are rarely applied to investigations at coal refuse facilities because of their relatively high cost. Nevertheless, there is some experience in the application of these techniques as documented by the Colorado School of Mines (2007) and Gritto (2003).

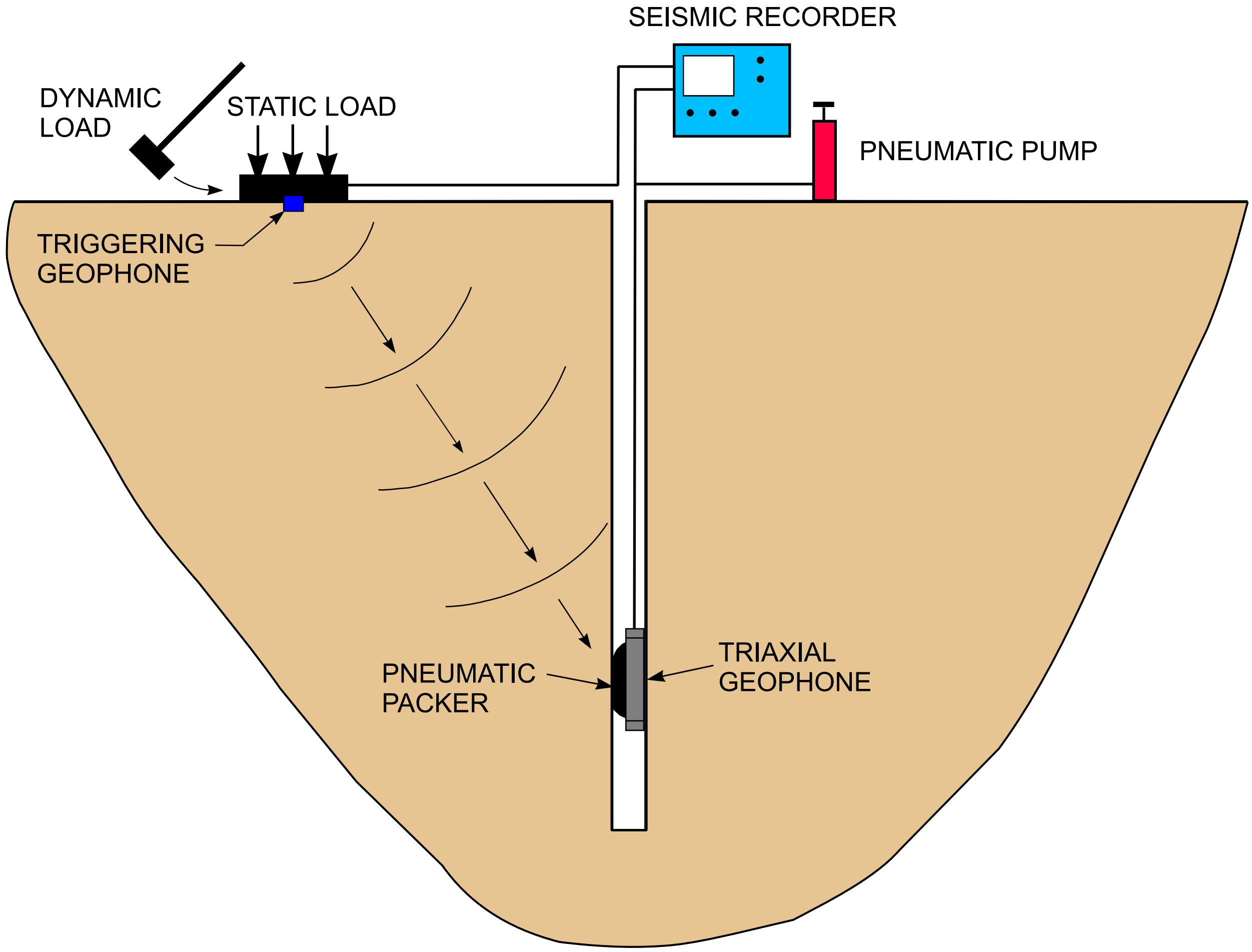


FIGURE 6.29 FIELD SETUP FOR DOWNHOLE SEISMIC SURVEY

A specialized application of a cross borehole technique to identify mine voids is seam wave seismic. Because coal typically has a relatively low seismic velocity compared to the rock formations that confine coal seams, seismic waves can become "trapped" in a coal seam and propagate over long distances with relatively little attenuation if the coal is continuous. Conversely, obstructions to a coal seam such as abandoned mine workings will prevent the propagation of a seam wave. D'Appolonia (1982) describes the use of this technique, but since the publication of this report, seam wave technology has only rarely been used due to the expense and difficulty in interpreting the results. Additional discussion of this technology is provided by Marshall Miller & Associates (MM&A, 2006) and Pennsylvania State University (2006).

6.4.4.2.2 Video and Laser/Sonar Imaging

Imaging of portions of abandoned mine workings can be critical to an understanding of the orientation and condition of these mine workings with respect to an existing or planned coal refuse facility. Video imaging with a borehole camera is a mature technology that also allows for the identification of fractures and collapse zones from a borehole. A disadvantage of using a borehole video camera in an abandoned mine is that it is difficult to image very far into the mine because of lack of light. Another disadvantage is that only visual data are obtained, and it is often difficult to determine distances because of a lack of scale.

If a mine is not flooded, an alternative means to image abandoned workings is a laser, range-based geometric scanner inserted through a dry borehole. Once deployed into a mine void space, a pan and tilt sequence is initialized, producing a scan of the void. The collected data set is then converted in the field into a 3D point cloud model of the void. The point cloud model is then converted into a 3D mesh model of the underground space. These data can subsequently be processed to produce plan views, sectional views, and volume estimations. Figure 6.30 is a 2D image of a mine entry obtained from a borehole laser scanner, and Figure 6.31 is a 3D laser image of mine workings.

If a mine is submerged, it is still possible to image mine openings with a submersible, sonar, range-based scanner inserted into a borehole. Data collected can be oriented using an on-board compass. Through correlation of several scans at varying elevations, it has proven practical to prepare 3D models of the flooded space.

6.5 MATERIAL PROPERTY DETERMINATION THROUGH TESTING

Accurate and reliable laboratory soil, rock and materials testing requires selection of the appropriate tests and care in sample preparation and performing the tests. Laboratory test results must be carefully interpreted, based upon the: (1) sampling and testing procedures, (2) types of soil and rock at the site, (3) geologic history of the site, and (4) types of coal refuse present and their possible use in refuse disposal facility construction. Suggested references for soil, rock and materials testing procedures include the most current ASTM standards, Mayne et al. (2002), Bardet (1997), and Head (1980, 1982, 1986)

Careful work and attention to detail in laboratory testing are important if accurate and representative results are to be obtained. This is true for all soils, but it is especially important for soils whose structure or fabric, and consequently their tested engineering characteristics, can be affected by disturbance. When undisturbed sample tests are to be conducted on fine-grained soils, sample disturbance must be minimized during sampling, transport, storage and testing. Similarly, some rock types (e.g., mudstones and claystones) can degrade following stress relief and exposure to the air following drilling. Such materials should be carefully stored and transported so that their in-situ moisture condition is preserved to the extent possible.

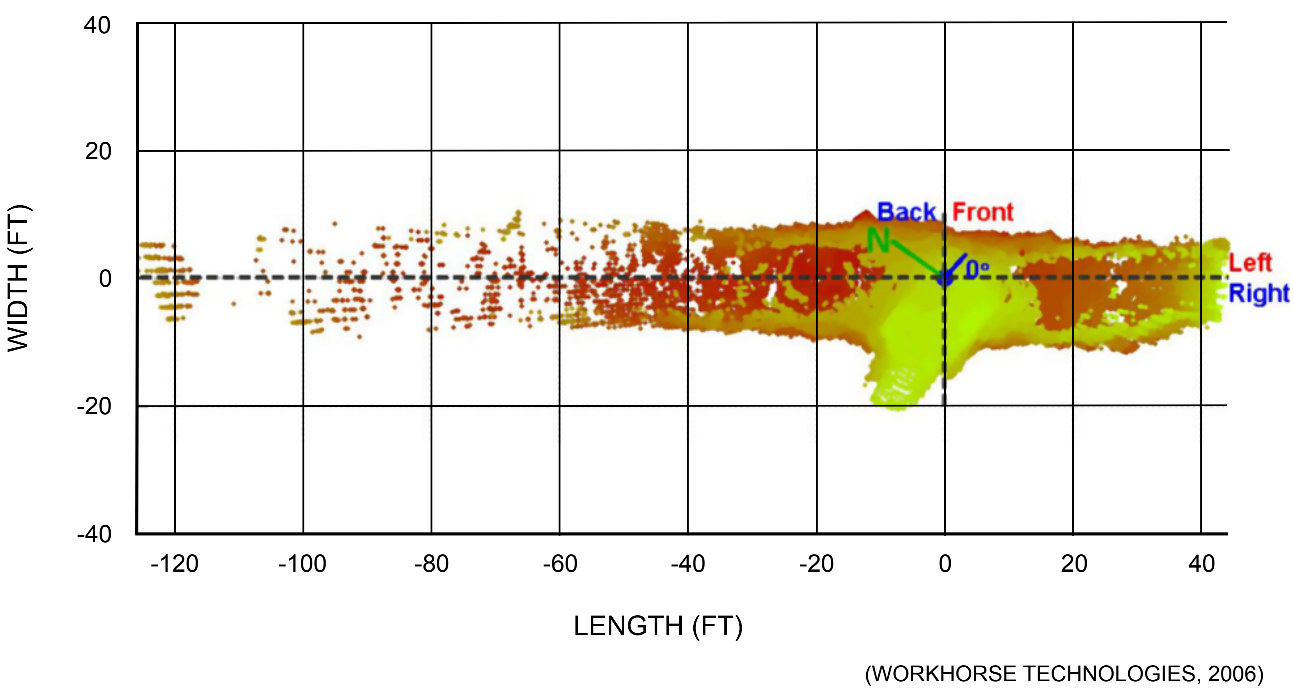
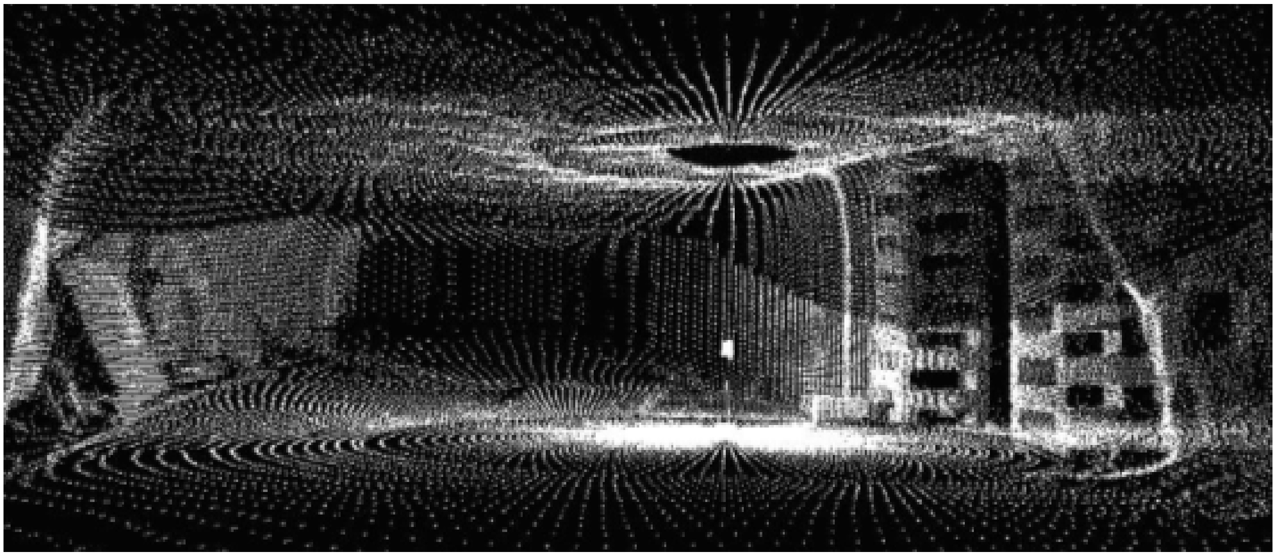


FIGURE 6.30 IMAGE OF MINE ENTRY FROM BOREHOLE LASER SCANNER



(WORKHORSE TECHNOLOGIES, 2006)

FIGURE 6.31 LASER IMAGE OF MINE WORKINGS

Laboratory tests that provide essential data for the analysis and design of an earthen dam or coal refuse embankment are summarized in Tables 6.27 and 6.28. Table 6.27 lists typical soil laboratory tests for site characterization, and Table 6.28 summarizes typical laboratory soil tests applicable to the types of soil, rock and refuse materials used in the construction of embankments and other earth/refuse structures. In most investigations, all of the classification or index property tests listed in the tables should be performed on representative samples. The need for other tests depends upon the purpose and subject of the investigation. The use of test data in the analysis and design of earthen dams and coal refuse impoundments is discussed in Section 6.6.

Standard testing procedures used in soil and rock mechanics are generally applicable to coal refuse, although modifications may be appropriate because of the characteristics of coal refuse. Tests for ash content, pyrite content and leachate water quality, as indicated in Table 6.28, are parameters not included in a typical embankment testing program. However, these parameters may be important in any portion of a disposal facility to be constructed from coal refuse. The ash content is an indication of the amount of coal remaining in the refuse and can be directly correlated with measurements of specific gravity and density. In cases where significant amounts of coal may remain in the refuse, there is a possibility of spontaneous combustion. Knowledge of pyrite content and leachate water quality can facilitate placement procedures that will minimize the potential for environmental impacts.

6.5.1 Selection of Samples for Testing

Samples used for laboratory testing include: (1) bulk and disturbed samples, (2) undisturbed samples, and (3) reconstituted samples. Reconstituted samples are samples created to have characteristics similar to in-situ properties and to meet specified test criteria (e.g., maximum particle dimension cannot be greater than some proportion of the minimum dimension of the test apparatus). The following text describes the three basic types of samples and their possible use for laboratory testing.

6.5.1.1 Bulk and Disturbed Samples

Representative bulk and disturbed samples of soils and refuse materials (for existing refuse disposal facilities) are collected from refuse delivered from the preparation plant, test pit excavations and disturbed sampling (e.g., split-barrel samples) for use in conducting laboratory index (e.g., classification, moisture content, Atterberg limits) and property characterization (e.g., compaction tests and

TABLE 6.27 TYPICAL SOIL AND ROCK LABORATORY TESTS FOR COAL REFUSE DISPOSAL SITE CHARACTERIZATION

Test Category	Test Description	ASTM Designation
Visual Identification	Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)	D 2488
Index Properties	Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass	D 2216
	Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer	D 854
	Standard Test Method for Particle-Size Analysis of Soils	D 422
	Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)	D 2487
	Standard Test Methods for Amount of Material in Soils Finer than the No. 200 (75-µm) Sieve	D 1140
	Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils	D 4318
Corrosivity	Standard Test Method for pH of Soils	D 4972
	Standard Test Method for Measuring pH of Soil for Use in Corrosion Testing	G 51
	Standard Test Method for Sulfate Ion in Water	D 516
	Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method	G 57
	Standard Test Methods for Chloride Ion in Water	D 512
Organic Content	Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils	D 2974
Compaction Test	Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft^3) (600 kN-m/m^3)	D 698
	Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft^3) (2,700 kN-m/m^3)	D 1557
	Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table	D 4253
	Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density	D 4254
Hydraulic Conductivity	Standard Test Method for Permeability of Granular Soils (Constant Head)	D 2434
	Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter	D 5084
Consolidation-Properties	Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading	D 2435
	Standard Test Method for One-Dimensional Consolidation Properties of Saturated Cohesive Soils Using Controlled-Strain Loading	D 4186

TABLE 6.27 TYPICAL SOIL AND ROCK LABORATORY TESTS FOR COAL REFUSE DISPOSAL SITE CHARACTERIZATION (Continued)

Test Category	Test Description	ASTM Designation
Static Strength Properties	Standard Test Method for Unconfined Compressive Strength of Cohesive Soil	D 2166
	Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils	D 2850
	Standard Test Method for Consolidated-Undrained Triaxial Compression Test for Cohesive Soils	D 4767
	Standard Test Method for Direct Shear Test of Soils under Consolidated-Drained Conditions	D 3080
	Standard Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil	D 4648
Cyclic/Dynamic Strength Properties	Standard Test Method for the Determination of the Modulus and Damping Properties of Soils Using the Cyclic Triaxial Apparatus	D 3999
	Standard Test Methods for Modulus and Damping of Soils by the Resonant-Column Method	D 4015
	Standard Test Method for Load-Controlled Cyclic Triaxial Strength of Soil	D 5311
Rock Properties	Standard Test Method for Determination of the Point Load Strength Index of Rock	D 5731
	Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures	D 7012
	Standard Test Method for Splitting Tensile Strength of Intact Rock Core Specimens	D 3967
	Standard Test Method for the Slake Durability of Shales and Similar Weak Rocks	D 4644

(ADAPTED FROM SABATINI ET AL., 2002)

strength and compressibility testing of reconstituted samples) testing. Table 6.18 provides a summary of common sampling methods for obtaining bulk and disturbed soil samples.

Field personnel directing field sampling activities need to be aware that the quantity of material needed depends on the laboratory tests to be performed, the relative amount of coarse (> 3 inches) particles present, and the size limitations of the test equipment. ASTM D 420, "Standard Guide to Site Characterization for Engineering Design and Construction Purposes," provides general guidelines for minimum sample weights. These guidelines are presented in Table 6.29. More specific guidance on minimum sample weight is provided in the instructions for individual test procedures.

For moisture-sensitive, fine-grained soils, samples should be retained in sealed containers, and bulk samples should be labeled, indicating information such as test pit number, depth below the ground surface, and date sampled.

TABLE 6.28 TYPICAL LABORATORY SOIL TESTS FOR VARIOUS MATERIALS[1]

Test	ASTM Test Method	Type of Material[2]						Use in Design
		Fine-Grained Soil	Coarse-Grained Soil	Rock	Coarse Refuse	Fine Refuse	Combined Refuse	
Classification or Index Property Tests								Evaluation of feasible configuration Correlation of materials Selection of samples for other tests Selection of borrow areas Specification of construction procedures Determination of filter and drainage requirements
Moisture Content	D 2216	a	a	–	a	a	a	
Unit Weight	–	c	–	–	–	c	c	
Specific Gravity	D 854	b, c	b, c	b	b	b	b	
Atterberg Limits	D 4318	b, c	b, c	–	–	b, c	–	
Particle-Size Analysis	D 422, D 2217	b, c	b, c	b	b	b, c	b	
Soil Classification	D 2487	a	a	–	–	–	–	
Compaction Tests								Evaluation of sample preparation for other tests Specification of placement requirements
Standard Proctor	D 698	b	–	–	b	b	b	
Modified Proctor	D 1557	b	–	–	b	b	b	
Relative Density	D 4253, D 4254	–	b	b	b	–	–	
Hydraulic Conductivity	D 2434, D 5084	c, d	–	–	d	c, d	c, d	Seepage analyses Determination of pore pressure for stability
Consolidation	D 2435, D 4186	c, d	–	–	–	c, d	–	Settlement analyses
Shear Strength								Stability analyses Structure foundation design
Direct Shear	D 3080	c, d	d	d	d	c, d	c, d	
Triaxial compression	D 2850, D 4767	c, d	d	–	d	c, d	c, d	
Unconfined compression	D 2166	c, d	–	–	–	–	–	
Vane Shear	D 4648	c	–	–	–	c	–	
Direct Simple Shear	D 6528	–	–	–	–	–	–	

TABLE 6.28 TYPICAL LABORATORY SOIL TESTS FOR VARIOUS MATERIALS[1] (Continued)

Test	ASTM Test Method	Type of Material[2]						Use in Design
		Fine-Grained Soil	Coarse-Grained Soil	Rock	Coarse Refuse	Fine Refuse	Combined Refuse	
Rock Property and Behavior Tests								
Point Load	D 5731	–	–	c	–	–	–	Design of rock slopes
Unconfined Compression	D 7012	–	–	c	–	–	–	Stability of underground mine roofs, pillars and barriers
Slake Durability	D 4644	–	–	c	–	–	–	Evaluation of rock degradation
Indirect Tensile Strength	D 3967	–	–	c	–	–	–	
Miscellaneous Tests								
Ash Content	D 2415	–	–	–	b	b	b	Evaluation of burning potential
Pyrite Content	D 4239, D 2492	–	–	b	b	b	b	Corrosion analyses
Leachate Water Quality	D 1068, D 858, D 516	–	–	–	b[3]	b[3]	b[3]	

Note: 1. The testing program for all significant coal refuse embankments should be established by a qualified, experienced geotechnical engineer. The types and numbers of tests needed will vary depending of the purpose of the testing program and the condition being evaluated. Use of these guidelines should not be substituted for evaluation of specific site conditions by a qualified engineer. Additional discussion is provided in Section 6.5.

2. a – tests normally conducted on all samples
b – tests normally conducted on representative disturbed samples
c – tests normally conducted on representative undisturbed samples
d – tests should be conducted on specially-prepared samples to simulate as-constructed behavior

3. Discussion related to conducting water quality tests as part of the geotechnical investigation is provided in Section 6.4.4.

6.5.1.2 Undisturbed Samples

Undisturbed samples are obtained from cohesive soil strata for laboratory testing to determine properties such as strength, stratification, hydraulic conductivity, density, consolidation, dynamic behavior, and other engineering characteristics. Specialized procedures are required for obtaining undisturbed samples of granular soils, thus their application at coal refuse disposal sites is limited to locations where void ratio, density and strength tests are needed for seismic design. Undisturbed samples are obtained with specialized equipment designed to minimize the disturbance to the in-situ structure and moisture content of the soils. Table 6.18 provides a summary of common methods for obtaining undisturbed soil samples. The importance of sample preservation during undisturbed sample recovery and transport is described in Section 6.4.3.5.

TABLE 6.29 GUIDELINES FOR MINIMUM SAMPLE WEIGHTS

Test and Soil Characteristics	Minimum Sample Weight
Visual classification	2 ounces to 1 pound
Soil constants and particle-size analysis of non-gravelly soil	1 to 5 pounds
Soil compaction tests and sieve analysis of gravelly soils	40 to 80 pounds
Aggregate properties	100 to 400 pounds

(ADAPTED FROM ASTM, 2008b,c)

6.5.1.3 Reconstituted Samples

Occasionally due to lack of adequate sample volume, difficulties encountered in the field in retrieving undisturbed samples, or dimensional requirements imposed by specific test methods, samples must be created or reconstituted in the laboratory for testing to establish engineering properties needed for design. The need to use reconstituted samples is more common for granular soils because undisturbed sampling of sands and gravels is difficult and costly and because the particle sizes in the coarser fraction of a sample may exceed particle-size limits in some tests. For example, the relative density of a saturated sand or of fine coal refuse can be estimated by in-situ testing, but an acceptably undisturbed sample for cyclic triaxial testing in the laboratory is difficult and costly to obtain. As a result, samples may need to be prepared in the laboratory to reasonably recreate the in-situ relative density or void ratio of the soil or sand-like refuse material. For predominantly coarse-grained soils, samples can be reconstituted by compaction in a mold. For sand-like refuse material, samples can be reconstituted by molding moist material or settling from a slurry. For clay-like fine refuse, the depositional history cannot be readily recreated in the laboratory, so reconstituted samples should not be used.

Reconstituted samples may also be necessary when the coarse particle-size fraction of a sample (usually a bulk sample) exceeds a dimensional limitation associated with the desired test. For example, ASTM D 3080, "Standard Test Method for Direct Shear Test of Soils under Consolidated Drained Conditions," prescribes that the maximum particle size not exceed 0.1 times the tested sample diameter (for circular samples) or sample width (for square samples). With this criterion, the maximum sample particle size cannot exceed 0.2 inches (corresponding approximately to a No. 4 sieve) for a 2-inch-diameter sample and 0.4 inches (corresponding approximately to a ⅜-inch sieve) for a 4-inch-square sample. For this example, if the maximum particle size exceeded 0.4 inches, the particle-size distribution for the sample used for direct shear testing would need to be adjusted to accommodate the maximum-particle-size criterion. ASTM test methods identify such gradational limitations and describe sample preparation techniques and test result evaluation methods to account for the removal of over-size particle fractions.

6.5.2 Classification and Index Property Tests

To catalog soils and coal refuse materials that will form the foundation, embankment cross section, and impoundment of a coal refuse disposal facility, samples from the field testing program should be examined and accurately classified. The system of classification used by most geotechnical engineers and government agencies is the Unified Soil Classification System (USCS) as described in ASTM D 2487, "Standard Practice for Classification of Soils for Engineering Purposes (Unified Classification System)." This classification system for engineering purposes is based on laboratory determination of particle-size distribution and Atterberg limits. ASTM D 2488, "Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)," provides a companion procedure for preliminary classification of soils based on visual and manual techniques available to field and laboratory personnel.

As shown in Table 6.30, the USCS divides soils into two main classes: coarse-grained and fine-grained soils. Highly organic soils form an additional division. Coarse-grained soils are soils composed of predominantly gravel- and/or sand-sized particles (greater than 50 percent retained on a No. 200 sieve) that can be separated into eight groups based primarily on the coarseness, gradation, and percentage of fines and secondly on the plasticity of fines. Fine-grained soils are soils composed of predominantly silt- and/or clay-sized particles (greater than 50 percent passing a No. 200 sieve) that can be separated into six groups, based primarily on plasticity and secondly on coarseness, gradation, and percentage of coarse fractions, if present. Generally, the system is arranged so that any sample can be classified by visual observation and simple tests that often can be conducted in the field by the engineer or geologist supervising an exploration program. However, laboratory tests on representative samples are needed to confirm the visual classification of soil properties or to classify borderline cases. Table 6.30 provides numerical criteria used for classification based upon laboratory test results. Figure 6.32 presents the plasticity chart used to classify fine-grained soils based on the liquid limit (LL) and plasticity index (PI) of tested samples.

Table 6.8, as adapted from Sherard et al. (1963), presents an approximate correlation between the USCS classification and the engineering and design properties of soils. Although the table is not a substitute for detailed laboratory tests, it can be used to help determine which tests should be conducted and to preliminarily evaluate available embankment materials.

There are no categories in the USCS for coal refuse materials, and current practice is to classify and test them in the same manner as other soil materials. Each of the following discussions of classification tests concludes with information on coal refuse properties as compared to properties of other soils. The basis for the discussion includes published data, as cited, and project experience, although it should be recognized that substantial variation can occur due to site-specific conditions and mining and coal preparation practices.

6.5.2.1 Moisture Content

Moisture content tests are typically conducted on disturbed and undisturbed samples obtained at a site to: (1) better characterize in-situ conditions and evaluate other tests, (2) evaluate borrow material suitability through comparison of natural moisture content and the moisture content required for proper compaction, and (3) provide information for calculating the void ratio of saturated samples.

Void ratio is defined as the ratio of void space to the volume of the solid particles:

$$e = V_v / V_s \tag{6-9}$$

where:

V_v = volume of voids (length^3)

V_s = volume of solids (length^3)

Properly obtained samples of fine-grained soils sealed in plastic, wax or airtight jars at the time of sampling can be accurately tested for moisture content at a later time in the laboratory. Testing of coarse-grained soils may not be accurate if water is lost by drainage during sampling. The field engineer should note whether moisture content measurements for coarse-grained soil samples may have been affected by the sampling.

As described in ASTM D 2216, "Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass," the procedure for measuring moisture content is to weigh

TABLE 6.30 SOIL CLASSIFICATION CHART (LABORATORY METHOD)

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[1]			Soil Classification: Group Symbol	Soil Classification: Group Name[2]
GRAVELS ≥ 50% of coarse fraction retained on No. 4 Sieve	CLEAN GRAVELS < 5% fines	$C_u \ge 4$ and $1 \le C_c \le 3$[5]	GW	Well-graded Gravel
		$C_u < 4$ and/or $1 > C_c > 3$[5]	GP	Poorly-graded Gravel[6]
	GRAVELS WITH FINES > 12% fines[3]	Fines classify as ML or MH	GM	Silty Gravel[6,7,8]
		Fines classify as CL or CH	GC	Clayey Gravel[6,7,8]
SANDS ≥ 50% of coarse fraction retained on No. 4 Sieve	CLEAN SANDS < 5% fines[4]	$Cu \ge 6$ and $1 \le Cc \le 3$[5]	SW	Well-graded Sand[9]
		$Cu < 6$ and $1 > Cc > 3$[5]	SP	Poorly-graded Sand[i]
	SANDS WITH FINES > 12% fines[4]	Fines classify as ML or MH	SM	Silty Sand[7,8,9]
		Fines classify as CL or CH	SC	Clayey Sand[7,8,9]
SILTS AND CLAYS LL < 50	Inorganic	PI > 7 and plots on or above "A" line[10]	CL	Lean Clay[11,12, 13]
		PI < 4 or plots below "A" line[10]	ML	Silt[11,12,13]
	Organic	LL after oven drying < 0.75 LL before oven drying	OL	Organic Clay[11,12,13,14]
			OL	Organic Silt[11,12,13,15]
SILTS AND CLAYS LL ≥ 50	Inorganic	PI plots on or above "A" line	CH	Fat Clay[11,12,13]
		PI plots below "A" line	MH	Elastic Silt[11,12,13]
	Organic	LL after oven drying < 0.75 LL before oven drying	OH	Organic Clay[11,12,13,16]
			OH	Organic Silt[11,12,13,17]
Highly fibrous organic soils	Primarily organic matter, dark in color, with organic odor		PT	Peat and Muskeg

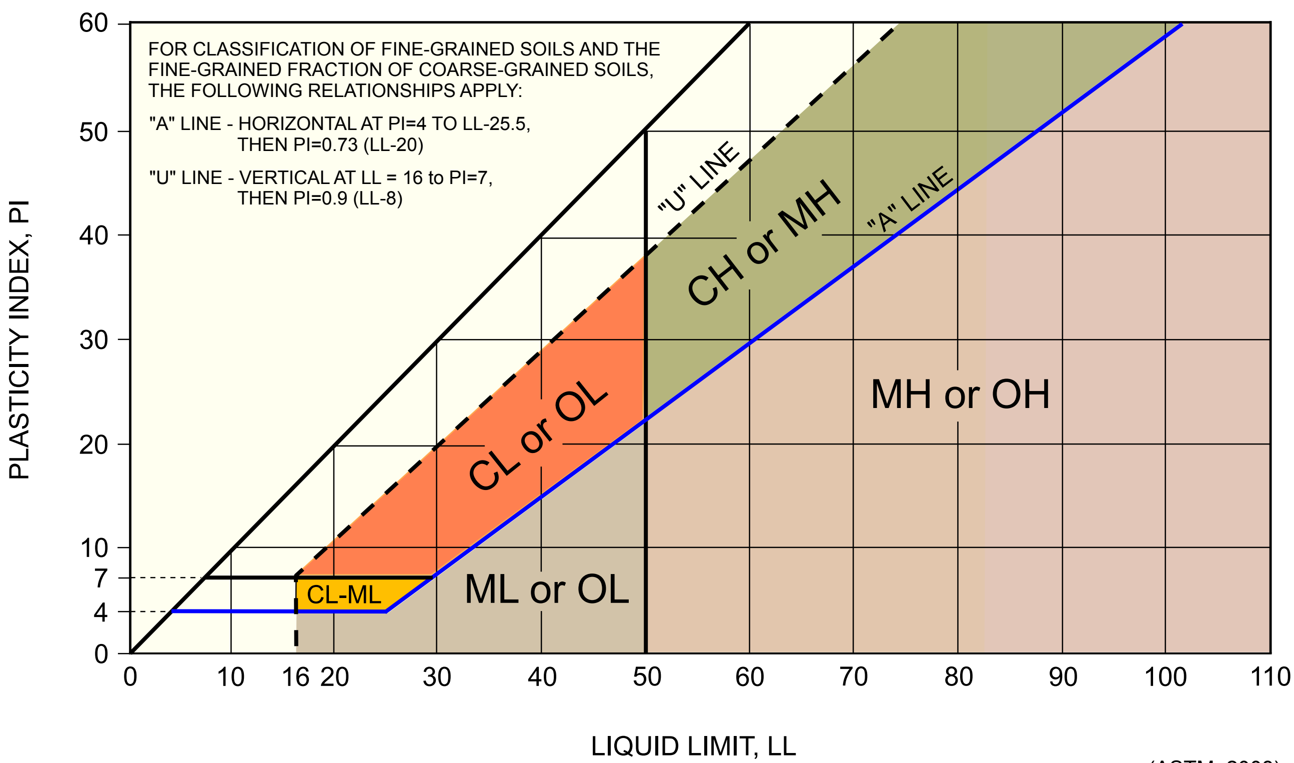
Note:
1. Based on the material passing the 3-in (75-mm) sieve.
2. If field sample contained cobbles or boulders, or both, add "with cobbles" or "with boulders" to group name.
3. $C_u = D_{60}/D_{10}$ = uniformity coefficient (UC);
   $C_c = (D_{30})^2 / (D_{60} \times D_{10})$ = coefficient of curvature
4. If soil contains ≥ 15% sand, add "with sand" to group name.
5. Gravels with 5 to 12% fines require dual symbols:
   GW-GM – well-graded gravel with silt
   GW-GC – well-graded gravel with clay
   GP-GM – poorly-graded gravel with silt
   GP-GC – poorly-graded gravel with clay

TABLE 6.30 SOIL CLASSIFICATION CHART (LABORATORY METHOD)
(Continued)

Note 6. If fines classify as CL-ML, use dual symbol GC-GM or SC-SM.
7. If fines are organic, add "with organic fines" to group name.
8. If soil contains ≥ 15% gravel, add "with gravel" to group name.
9. Sands with 5 to 12% fines require dual symbols:
   SW-SM – well-graded sand with silt
   SW-SC – well-graded sand with clay
   SP-SM – poorly-graded sand with silt
   SP-SC – poorly-graded sand with clay
10. If Atterberg limits plot in the orange area in Figure 6.32, soil is a CL-ML (silty clay).
11. If soil contains 15 to 29% plus No. 200 sieve, add "with sand" or "with gravel," whichever is predominant.
12. If soil contains ≥ 30% plus No. 200 sieve, predominantly sand, add "sand" to group name.
13. If soil contains ≥ 30% plus No. 200 sieve, predominantly gravel, add "gravelly" to group name.
14. PI ≥ 4 and plots on or above "A" line.
15. PI < 4 or plots below "A" line.
16. PI plots on or above "A" line.
17. PI plots below "A" line.

(ADAPTED FROM ASTM, 2008b,c)

a wet sample, dry it in a constant-temperature oven at 105° C until the weight is constant (approximately 24 hours for small samples of fine-grained soils), and then weigh the dry sample. In soil mechanics practice, the moisture content is defined as the ratio of the weight of water (wet weight minus dry weight) to the dry weight. This is sometimes referred to as the dry-weight-basis moisture content. In other disciplines, moisture content may be defined on a wet-weight basis, i.e., moisture content is defined as the ratio of the weight of water to the wet weight of soil.



(ASTM, 2008)

FIGURE 6.32 PLASTICITY CHART

If a soil sample contains a significant amount of organic material, this method of measuring moisture content is not always satisfactory, because heating the sample to 105° C may drive off some of the organic material in addition to the water. Alternatively, ASTM D 2216 permits oven drying at 60° C for the moisture content of organic soils and organic materials. Most coal refuse is not significantly affected by oven drying at 105° C, although this may need to be considered when working with lower grade coals such as lignite.

6.5.2.2 Specific Gravity

The specific gravity of a soil is the ratio of the weight of a given volume of soil solid particles to the weight of an equal volume of distilled water at 4° C. As used in geotechnical engineering, the term specific gravity refers to the average specific gravity of the individual soil particles in a sample rather than bulk specific gravity. Specific gravity is determined in the laboratory in accordance with ASTM D 854, "Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer." The test is performed by weighing a calibrated bottle containing soil particles suspended in distilled water and comparing this to the weight of the same bottle containing an equal volume of distilled water only. Specific gravity is used to determine relationships between soil weight and soil volume. Specific gravity is used for: (1) computing the void ratio of a soil, (2) hydrometer analyses, and (3) predicting the unit weight of a soil. Occasionally, the specific gravity may be useful in soil mineral classifications (e.g., iron minerals have a higher specific gravity than silica).

Soils typically have a specific gravity ranging from 2.4 to 2.8. For many design purposes specific gravity can be estimated without testing. For coal refuse facilities, specific gravity is an important design parameter because coal refuse often contains a significant amount of materials with specific gravity in the range of 1.3 to 1.6. As a result, coarse coal refuse can have a specific gravity ranging from as low as 1.5 to as high as 2.8. The most common range is between 1.9 and 2.4. Similarly, specific gravity measured for fine coal refuse typically ranges from 1.4 to 2.3. Published data on specific gravity and unit weight of coarse and fine coal refuse from sites in the northern Appalachian region illustrating some of the variability in these parameters is presented in Tables 6.31 and 6.32, respectively, as compiled by Hegazy et al. (2004). The database for these summaries from Hegazy et al. (2004) was developed from geotechnical investigations of existing coal refuse disposal sites in western Pennsylvania and England. In-situ samples were collected from the sites using both disturbed methods (bucket samples from test pits and fine coal refuse deltas and split-barrel samples from boreholes) and undisturbed sampling methods (Shelby- and Dennison-tube samples).

The coefficient of variability (COV) is the ratio of the standard deviation of a set of data divided by the mean. Typically, values of COV below 10 percent are thought to be low, between 10 and 30 percent moderate, and above 30 percent high. Values of total unit weight γ_T, dry unit weight γ_D, and specific gravity G_s for coarse and fine coal refuse are provided in Tables 6.31 and 6.32. For coarse coal refuse, Table 6.31 indicates low variability for γ_T and γ_D and moderate variability for G_s. For fine coal

TABLE 6.31 IN-PLACE UNIT WEIGHT AND SPECIFIC GRAVITY OF COARSE COAL REFUSE

Property	Dimension	Average	Standard Deviation	Coefficient of Variation
γ_T	lb/ft^3	124	5.8	0.048
γ_D	lb/ft^3	115	5.5	0.047
G_s	Dimensionless	2.02	0.31	0.154

(ADAPTED FROM HEGAZY ET AL., 2004)

refuse, Table 6.32 indicates low variability for γ_T and moderate variability for γ_D and G_s. Figure 6.33 shows the effect of carbon content on the specific gravity of coal refuse materials.

Designers should recognize that values of specific gravity and unit weight for coal refuse are lower than for commonly encountered soils. The lower specific gravity of coal refuse results in lower densities, higher moisture contents at a given void ratio, and the potential for reduced stability with respect to seepage forces. These characteristics are discussed further in Section 6.6.4.

6.5.2.3 Atterberg Limits

The Atterberg Limits define the boundaries between four states of consistency (hardness or softness) of fine-grained soils. In order of decreasing moisture content, these states are: liquid, plastic, semi-solid and solid. The boundaries or limits between these states are:

- Liquid limit (*LL*) – boundary between the liquid and plastic states
- Plastic limit (*PL*) – boundary between the plastic and semi-solid states
- Shrinkage limit (*SL*) – boundary between the semi-solid and solid states

The plasticity index (*PI*) is defined as *LL* minus *PL*. The liquid and plastic limits and plasticity index are determined in accordance with ASTM D 4318, "Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils." The shrinkage limit is determined in accordance with D 4943, "Standard Test Method for Shrinkage Factors of Soils by the Wax Method."

TABLE 6.32 UNIT WEIGHT AND SPECIFIC GRAVITY FOR FINE COAL REFUSE

Property	Dimension	Average	Standard Deviation	Coefficient of Variation
γ_T	lb/ft^3	86	7.7	0.096
γ_D	lb/ft^3	62	9.1	0.162
G_s	Dimensionless	1.52	0.25	0.165

(ADAPTED FROM HEGAZY ET AL., 2004)

The liquid limit is the moisture content at which the soil sample flows and closes a standard width groove when the sample is jarred in a specified way. For practical purposes, it is the moisture content at which the soil has essentially no shear strength (the soil becomes liquid). The test for the *LL* is conducted using a standard liquid-limit device and grooving tool. The plastic limit is the moisture content at which the soil begins to crumble when rolled by hand into ⅛-inch-diameter threads. The shrinkage limit (*SL*) is the moisture content sufficient to fill the pores of the soil at the minimum volume it will attain by drying. Other useful parameters from these tests are the plasticity index (*PI*) and the liquidity index (*LI*). The plasticity index is an indicator of soil plasticity, and the *LI*, which equals $(w - PL)/PI$, is an indicator of stress history and soil sensitivity. The liquidity index is approximately 1 for normally-consolidated soils and zero for over-consolidated soils. An *LI* greater than 1 indicates high sensitivity.

Many properties of fine-grained clays and silts can be correlated to the Atterberg limits, and the plasticity chart shown in Figure 6.32 can be useful in interpreting the correlation. Procedures for using the plasticity chart for various soil types are discussed by Terzaghi et al. (1996).

Caution and considerable judgment should be used when applying the plasticity chart to coal refuse, because the chart is based on the behavior of natural, fine-grained soils. Tests conducted on only the

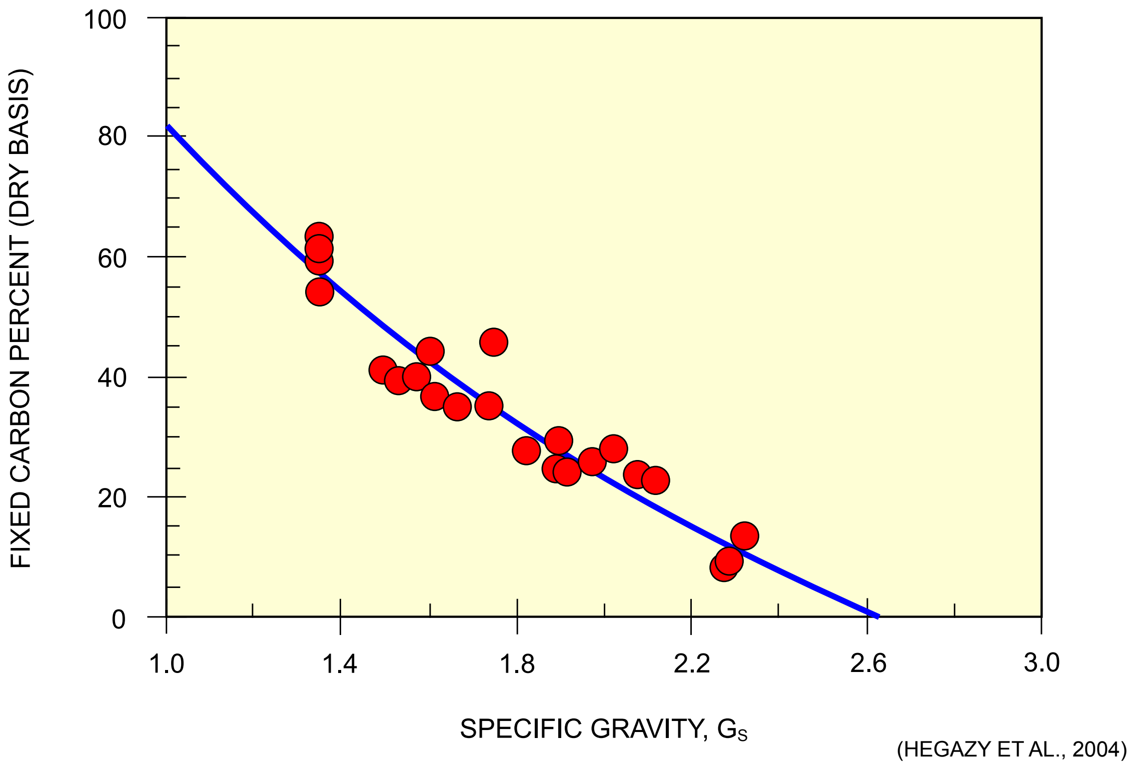


FIGURE 6.33 EFFECT OF CARBON CONTENT ON SPECIFIC GRAVITY OF COAL REFUSE MATERIAL

fines portion of coarse refuse have obtained *LL*'s in the range of 25 to 35 percent and *PI*'s typically below 12 percent. Table 6.4 cites published data characterizing the properties of fine coal refuse. Tests conducted on fine refuse samples from impoundments show *LL*'s in the range of 20 to 40 percent and *PI*'s generally below 15 percent, although higher *PI*'s have been reported. Factors affecting plasticity are discussed in Section 6.2.3.2. A summary of statistical properties related to Atterberg limits for fine coal refuse from northern Appalachian sites is provided in Table 6.33. Other data suggest that the plasticity of fine refuse from an impoundment increases with distance from the slurry discharge point, as the content of clay-size refuse particles increases.

6.5.2.4 Particle-size Distribution

The particle-size distribution of a coarse-grained soil is an important factor in its engineering behavior. The particle-size distribution of a coarse-grained soil can be used to classify the soil, to estimate hydraulic conductivity and to provide a qualitative indication of soil strength and deformation characteristics. For fine-grained soils, plasticity and moisture content are better indices of soil behavior than particle-size distribution. For both fine and coarse material, the particle-size distribution is needed for checking filter criteria requirements as part of prevention of piping of fines into coarser filter and drainage zones. Figure 6.34 shows typical gradation curves for several types of materials including coarse and fine coal refuse. Of particular note is the correlation between types of soil (clay, silt, sand and gravel) and sieve sizes and particle diameters.

For comparative purposes between soil types and for certain design applications, the uniformity of a soil can be expressed by the uniformity coefficient, which is the ratio of D_{60} to D_{10}, where D_{60} is the particle diameter at which 60 percent of the soil weight is finer and D_{10} is the particle diameter at which 10 percent of the soil weight is finer. Sand and gravel soils having uniformity coefficients less than 6 and 4, respectively, are considered to be "uniform." For example, the uniformity coefficients of

TABLE 6.33 STATISTICAL PROPERTIES FOR ATTERBERG LIMITS AND MOISTURE CONTENTS OF FINE COAL REFUSE AT NORTHERN APPALACHIAN SITES

Property	Average (%)	Standard Deviation	Coefficient of Variation
LL	31.2	5.2	0.17
PL	20.1	3.4	0.17
PI	11.2	3.1	0.28
w	33.0	11.5	0.35

(HEGAZY ET AL., 2004)

the two sandy soils shown in Figure 6.34 are about 6.1 and 1.4. These soils are termed "well-graded sand" and "uniform sand," respectively.

As described in ASTM D 422, "Standard Test Method for Particle-Size Analysis of Soils," the particle-size distribution of the portion of a soil sample coarser than a No. 200 sieve (0.074-mm-square openings) is generally determined by sieve analysis. This procedure consists of shaking a dry soil sample or washing a wet soil sample through a stack of wire screens of decreasing opening size. The diameter of an individual soil particle is defined as the minimum side dimension of a square hole through which the soil particle will pass.

The particle-size distribution of the portion of a sample finer than a No. 200 sieve can be measured with a hydrometer test as described in ASTM D 422. In this test, a sample of soil is vigorously mixed with water and a deflocculating agent to form a suspension. The suspension is then allowed to sit

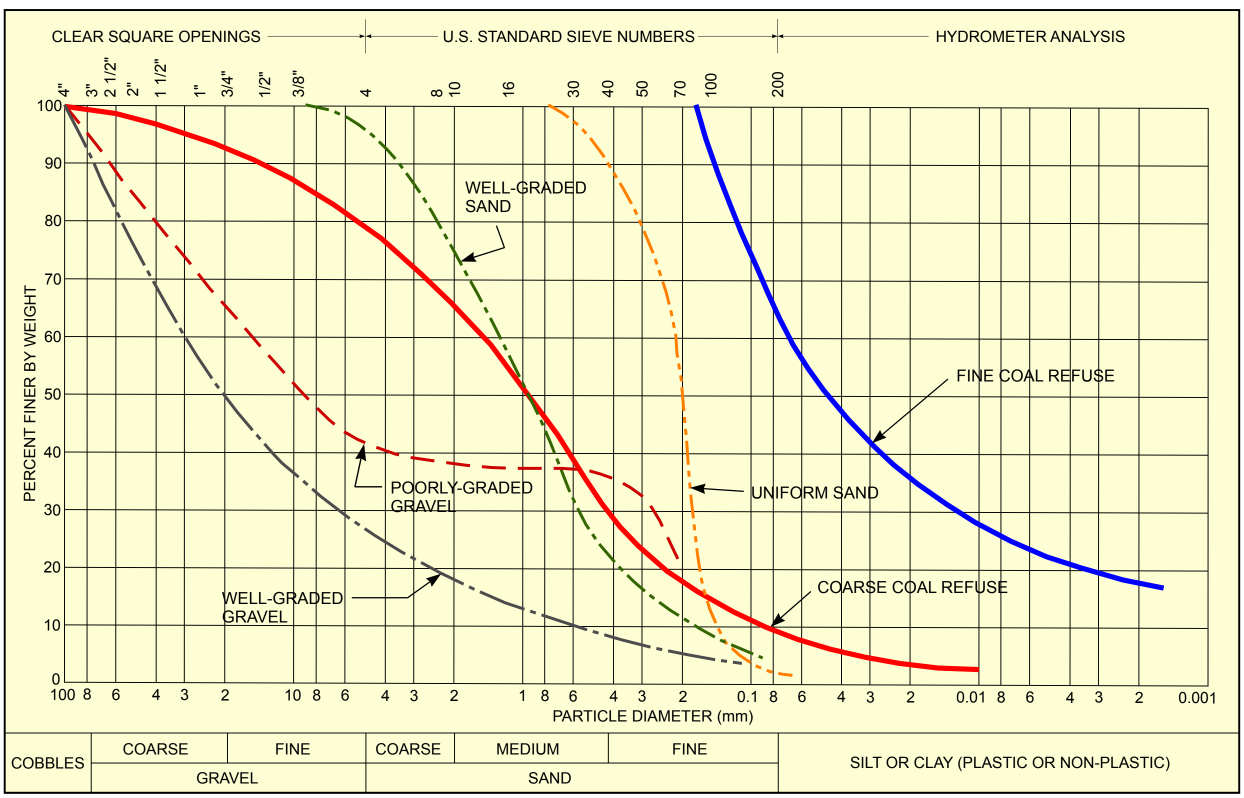


FIGURE 6.34 PARTICLE SIZE DISTRIBUTION

undisturbed in a 1000-ml glass cylinder to permit settlement of the suspended soil particles. Periodically, the change in the specific gravity of the suspension is measured with a calibrated hydrometer, allowing the approximate distribution of particle sizes to be calculated. The specific gravity of the suspension changes with time because the larger particles settle faster than smaller particles of the same specific gravity. The test result is only approximate because the calculation is based upon the assumption that the particles are spherical, of equal specific gravity, and do not interfere with each other during settlement. In actual tests, the particles are not spherical, there are variations in specific gravity, and considerable interference between particles likely occurs during settlement. Because the distributions determined by the hydrometer test are primarily used for comparative purposes, the accuracy of the test is generally not of major concern.

The effect of differences in specific gravity of various particles in the hydrometer test is greater for coal refuse than for other soils, because coal refuse consists of coal with specific gravity as low as 1.3 and rock fragments with specific gravity as high as 2.8. For most analyses of coal refuse, it is appropriate to consider the average specific gravity of the entire sample. However, it should be understood that the variation in specific gravity of coal refuse particles adds greater than normal inaccuracy to the portion of the gradation curve determined by the hydrometer method.

Coarse coal refuse delivered to disposal facilities typically has the grain-size distribution of a well-graded silty sand and gravel. Coal preparation usually limits the upper size to about 5 inches, although this size has more characteristically been less than 3 inches. Uniformity coefficients for coarse refuse range from about 20 to several hundred. Because many coal refuse particles are extremely friable and highly susceptible to both chemical and mechanical deterioration, the particle-size distribution changes during preparation, transportation and placement at the disposal facility. As delivered to the point of disposal, coarse refuse typically contains between 5 and 30 percent of particles finer than the No. 200 sieve (0.075 mm). The clay-size fraction (finer than 0.002 mm) is usually very small and often less than 2 percent. Sampling programs should be designed to evaluate the potential for particle degradation through collection of both as-delivered samples and after-placement samples.

Particle-size degradation occurs at the surface of a coarse refuse embankment or disposal facility, due to both chemical and mechanical deterioration. This behavior is described in Andrews et al. (1980) where the environmental effects of slaking of surface mine spoils in the eastern and central U.S. were evaluated. The study showed that degradation of surface mine spoil occurred over periods of years depending on the rock type(s) found in the spoil and their depth of burial. Field examination showed that degradation was more predominant with finer-grained, rock-type spoils (e.g., shales, mudstones and claystones) and was most prevalent in the upper 5 to 10 feet depending on the amount of compaction (if any) during spoil placement. For embankments constructed using coarse refuse, particle-size degradation is also affected by hauling equipment and mechanical compaction that occurs as the material is placed.

Fine coal refuse delivered to disposal facilities from preparation plants is typically a slightly clayey, sandy silt. Generally, 50 to 80 percent of the material will pass the No. 200 sieve, most of which is silt-size. Fine refuse segregates after being deposited, with the larger and heavier particles settling out of suspension near the discharge point. Farther from the discharge point, the settled refuse is predominantly finer-grained, and samples containing nearly 50 percent clay-size particles have been reported. A summary of statistical properties related to particle-size distribution for fine coal refuse samples from sites in northern Appalachia is provided in Table 6.34.

6.5.2.5 Chemical Characterization

Some soil and rock materials encountered at refuse disposal sites and the refuse being placed can have adverse chemical characteristics that may lead to undesirable environmental conditions or deteriora-

TABLE 6.34 PARTICLE-SIZE DISTRIBUTION FOR FINE COAL REFUSE

Particle Size or Percent Passing	Dimension	Average	Standard Deviation	Coefficient of Variation
D_{10}	mm	0.010	0.015	1.50
D_{30}	mm	0.037	0.055	1.49
D_{50}	mm	0.127	0.128	1.01
D_{60}	mm	0.196	0.209	1.07
Passing No. 200 (0.075-mm) sieve	%	57.7	25.0	0.43

(HEGAZY ET AL., 2004)

tion of construction materials. Therefore, the chemical characteristics of these materials (e.g., corrosivity, organic content, ash content, pyrite content, and leachate water quality) should be determined. When deleterious conditions are encountered, appropriate amendment or containment/protection requirements should be implemented. Table 6.35 lists laboratory test methods that can be used for chemical characterization of soil, rock and refuse materials.

6.5.3 Compaction and Density Tests

Any soil placed as part of a structural fill, including coal refuse in embankments, is normally compacted to increase density and shear strength and to decrease compressibility and hydraulic conductivity. This makes relatively steep slopes possible, reduces seepage from impoundments, and reduces the potential for spontaneous combustion by reducing the flow of air and water into the embankment. In the field, compaction is accomplished with various types of rollers, including sheepsfoot, static and vibrating steel drum, and rubber-tired. The type of roller that is most appropriate depends upon the material being compacted, as discussed in Section 11.4.3.

6.5.3.1 Fine-grained Soils and Coal Refuse

Standard laboratory test procedures to control and evaluate the degree of compaction that can be achieved in the field during placement have been established. The test most commonly used for fine-grained soils and coal refuse utilizes impact compaction. The test procedure entails placing soil or refuse in several layers in a standard mold and compacting each layer by dropping a hammer of specified weight a specified distance for a specified number of times per layer.

The two standardized impact compaction tests are the standard Proctor and the modified Proctor tests. The standard Proctor compaction test was developed in the 1930s and was designed to approximate the compactive energy applied by field compaction equipment then available. As field equipment became larger and more efficient, the modified proctor compaction test was developed to approximate the greater compactive energy of the newer equipment. The test procedures for the standard and modified Proctor compaction tests are described in ASTM D 698, “Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft^3 (600 kN-mm^3))” and ASTM D 1557, “Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft^3 (2,700 kN-mm^3)),” respectively.

These test methods are suitable for soils and coal refuse that have 30 percent or less retained on the ¾-inch sieve and that have more than 15 percent by dry weight passing a No. 200 sieve. If more than 30 percent is retained on a ¾-inch sieve for either test, the unit weight and moisture content should be corrected in accordance with ASTM D 4718, “Standard Practice for Correction of Unit Weight and Water

TABLE 6.35 TEST METHODS FOR CHEMICAL CHARACTERIZATION OF SOIL, ROCK AND REFUSE MATERIALS

Item	Description	ASTM Test Method
Corrosivity	Standard Test Method for pH of Soils	D 4972
	Standard Test Method for pH of Soil for Use in Corrosion Testing	G 51
	Standard Test Method for Sulfate Ion in Brackish Water	D 4130
	Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method	G 57
	Standard Test Methods for Chloride Ion in Water	D 512
Organic Content	Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils	D 2974
Ash Content	Standard Test Method for Ash in Coal Tar and Pitch	D 2415
Pyrite Content	Standard Test Method for Forms of Sulfur in Coal	D 2492
Leachate Water Quality(1)	Standard Test Method for Leaching Solid Material in a Column Apparatus	D 4874
	Standard Test Method for Shake Extraction of Mining Waste by the Synthetic Precipitation Leaching Procedure	D 6234

Note: 1. State regulatory agencies may require specific test procedures. Other standard test method references include EPA Method 1312, "Synthetic Precipitation Leaching Procedure (SPLP)" and EPA Method 1320, "Multiple Extraction Procedure (MEP)."

Content for Soils Containing Oversize Particles." Alternatively, a 12-inch-diameter compaction mold using standard Proctor compactive effort can be employed using the U.S. Army Corps of Engineers procedure, "Compaction Test for Earth-Rock Mixtures," described in Section 11.5.1 (USACE, 1986). If less than 15 percent by dry weight passes the No. 200 sieve, the density of the soil may not be affected by changes in moisture, and the guidelines presented in Section 6.5.3.2 for coarse-grained soils may be applicable. Coarse coal refuse may contain less than 15 percent by dry weight passing a No. 200 sieve, but it generally does respond to changes in moisture content when compacted. Accordingly, in practice, the standard Proctor test is typically used to evaluate the compaction and density of coarse refuse.

Normally, a series of compaction tests is performed on several soil samples prepared at varying moisture contents. From the test results plotted as shown in Figure 6.35, the maximum density attainable and the moisture content at which the maximum density is attained (the optimum moisture content), can be determined. The greater compactive energy of the modified Proctor compaction test produces higher maximum densities at lower optimum moisture contents than the standard Proctor compaction test. The 100-percent-saturation (or zero-air-voids) curve to the right of the compaction curves in Figure 6.35 can be determined using the relationship:

$$\gamma_d = \gamma_w G_s / (1 + w G_s) \tag{6-10}$$

where:

γ_d = dry density of soil (force/length^3)

γ_w = unit weight of water (force/length^3)

G_s = specific gravity of solids (dimensionless)

w = moisture content expressed as a decimal value (dimensionless)

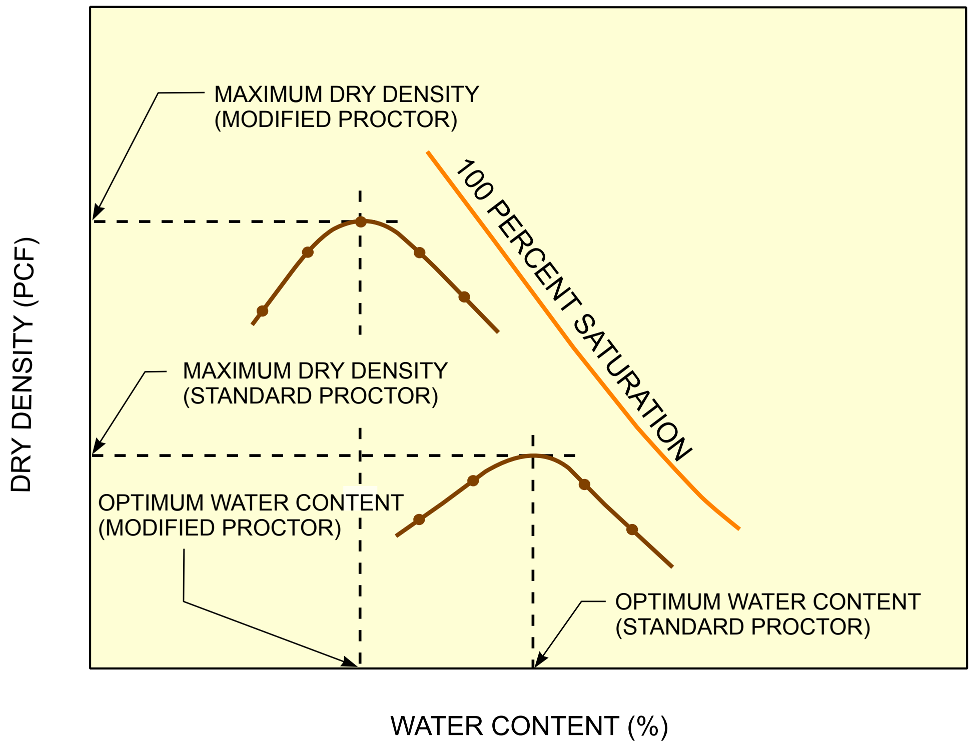


FIGURE 6.35 TYPICAL MOISTURE-DENSITY RELATIONSHIPS FOR DYNAMIC COMPACTION

Equation 6-10 provides a check on the compaction test results to confirm that no compaction test result plots to the right of the 100-percent-saturation curve and that the shape of the dry density-moisture relationship wet of optimum moisture content generally parallels the 100-percent-saturation curve. Additionally, Equation 6-10 demonstrates the need to know the specific gravity of a material when determining compaction and density.

Specifications for construction of compacted fills usually require that the density attained in the field be equal or greater than a certain percentage of the maximum density attained in the laboratory compaction tests (for structural embankment zones, normally 95 percent of the maximum density at the optimum moisture content from the standard Proctor test). To help achieve good density control, specifications also usually require the fill to be placed at a moisture content near the optimum moisture content (normally in the range from 2 percent below optimum to 3 percent above optimum).

D'Appolonia (1973) reported a measured in-place dry unit weight for uncompacted coarse refuse of 80 to 110 pounds per cubic foot (pcf), with a median value of 94 pcf based upon data from 200 tests. For compaction testing performed at mine sites in northern Appalachia, Hegazy et al. (2004) reported a median dry density for compacted coarse refuse of 115 pcf. While these results demonstrate dry densities encountered in some specific situations, there can be considerable variation depending upon geographic location and mining and coal processing activities.

Because of the degradation of coarse coal refuse due to equipment traffic during placement and weathering after placement, the percentage of fines in "aged" coarse coal refuse will likely be greater than in fresh or recently placed coarse coal refuse. Saxena et al. (1984) report that particle breakdown due to a combination of weathering and compaction produces better graded materials with higher density and strength and lower compressibility and hydraulic conductivity. Hegazy et al. (2004) presented the results of sieve analyses performed for fresh coarse coal refuse samples and repeated after compaction to determine the effect of compaction on the fines content (i.e., the percent passing the No. 200 sieve). For the tests plotted in Figure 6.36, the average increase of fines due to compaction was approximately 4 percent.

In applications sensitive to fines content, breakdown due to weathering and compaction can be evaluated through slake durability testing of fresh and weathered compacted samples (Section 6.5.9.4).

Specifications for clay core materials designed to restrict seepage through an embankment are normally based upon Proctor test results but these specifications often require that the material to be placed slightly wet of optimum. This results in lower strengths, but eliminates the potential for brittle soil behavior. The resulting core should be sufficiently flexible to allow for small amounts of differential movement without the development of cracks that would allow the passage of large volumes of water and cause piping. Sherard et al. (1963), Sherard (1973), and Lo (1990) report a number of dam failures caused by cracking, and they discuss the related importance of compaction specifications and control.

Slurry-placed fine coal refuse typically has a very low in-situ density because of low specific gravity and moisture contents near the liquid limit. Dry densities near 50 pcf have been reported. Coarser or dryer portions of the fine refuse may have dry densities of 70 pcf or higher.

6.5.3.2 Coarse-Grained Soils and Coal Refuse

Relative density is the dry density of a soil in relation to the minimum and maximum dry densities that can be achieved by specific laboratory procedures. The relative density test is applicable to free-draining, cohesionless soils with low fines content (i.e., less than 15 percent non-plastic fines passing the No. 200 sieve) that do not have a well-defined moisture-density relationship. The maximum dry density is determined in accordance with ASTM 4253, "Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table," and the minimum dry density and rela-

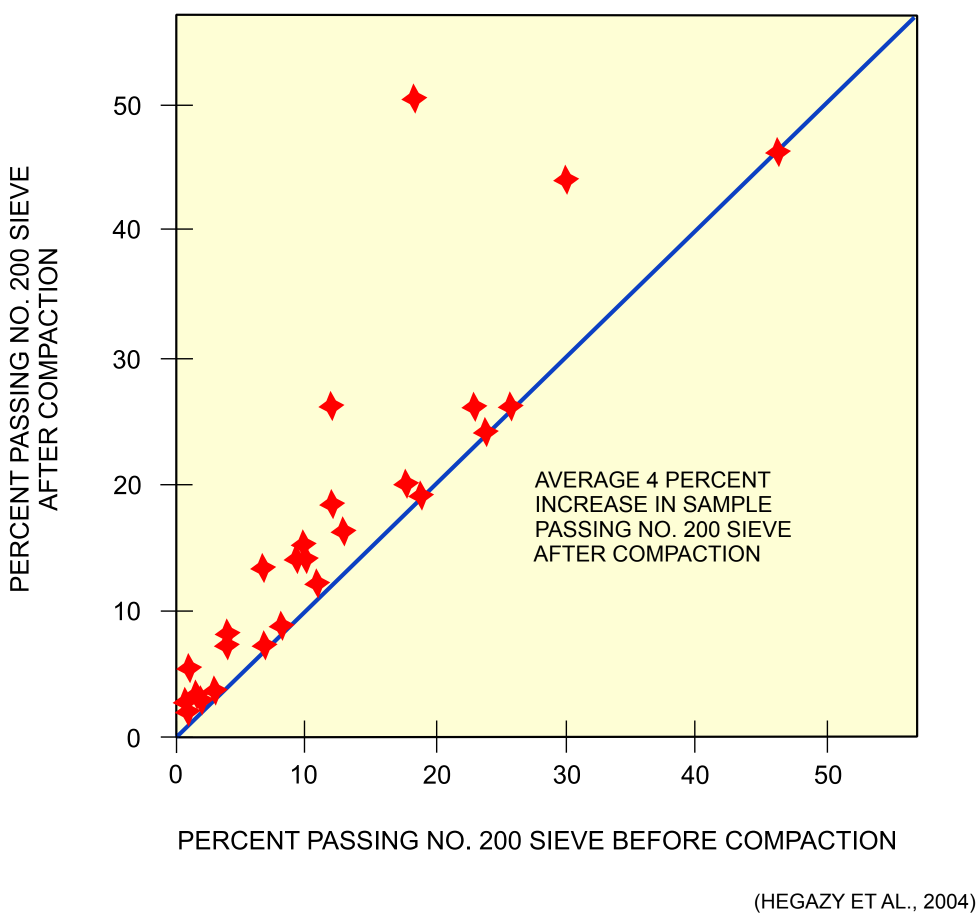


FIGURE 6.36 PARTICLE-SIZE DISTRIBUTION OF COARSE COAL REFUSE AFTER COMPACTION

tive density are determined in accordance with ASTM 4254, "Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density."

For coal refuse embankments, the relative density test is normally applicable to materials used for granular drainage and filter zones that require compaction. Relative density tests are also conducted for evaluating the capability of a saturated, coarse-grained soil to resist seismic loadings. Seismic issues are discussed in Chapter 7.

The minimum density (zero percent relative density) in accordance with ASTM 4254 is obtained by placing dried soil as loosely as possible in a mold of known volume. The preferred method for placing soil in the mold requires using a pouring device that limits the height of free fall to no more than ½ inch. The weight of the known volume of soil is then measured and used in the determination of the minimum test dry density. The minimum density is the weight of soil divided by the volume of the mold.

The maximum dry density (100 percent relative density) in accordance with ASTM 4253 can be obtained using either dry soil (method A) or wet soil (method B). The soil is densified using either an electromagnetic, vertically vibrating table or an eccentric or cam-driven, vertically vibrating table. If method A is used, dry soil is placed in a mold using a scoop or funnel, a surcharge base plate is placed atop the level soil surface, the filled mold with surcharge weight is attached to the vibrating table, and the table is vibrated for 8 to 12 minutes depending on the frequency of vibration.

If method B is used, the mold is attached to the vibrating table, the table is turned on, and wet soil is placed in the mold over a 5- to 6-minute period during which care is taken to avoid excessive vibration that causes the soil to boil excessively. The table is then turned off, a surcharge is placed atop the soil, and the table is vibrated for 8 to 12 minutes as for Method A. After the table is turned off (both methods), the surcharge is removed and the height of the sample in the mold is measured. The material in the mold is then weighed. If the sample is wet, it is oven dried and weighed again after drying. The weight of the known volume of dried soil is used to determine the maximum dry density. The difference between the wet and dry weights can be used to determine the moisture content of the material tested.

The relative density is calculated by the relationship:

$$D_r = \frac{\gamma_{dmax}\,(\gamma_d - \gamma_{dmin})}{\gamma_d\,(\gamma_{dmax} - \gamma_{dmin})} \times 100 \tag{6-11}$$

where:

D_r = relative density (dimensionless)

$\gamma_{d_{max}}$ = maximum dry density (force/length^3)

$\gamma_{d_{min}}$ = minimum dry density (force/length^3)

γ_d = measured dry density of the sample (laboratory or in situ) (force/length^3)

The relative density D_r is used as a basis to confirm whether placement of coarse-grained soils in the field meets the minimum specified compaction criteria. The minimum acceptable relative density for coarse-grained soils typically ranges between about 70 to 85 percent depending upon performance requirements.

6.5.4 Hydraulic Conductivity Tests

The hydraulic conductivity (or permeability) of a soil is a measure of the rate at which water will flow through a soil under a particular pressure (or head). It is represented by the coefficient of hydraulic conductivity k, which is normally expressed in units of distance per time. Hydraulic conductivity is essentially the volume of flow per unit time per unit of cross-sectional area for a unit pressure gradient. While laboratory measurement of hydraulic conductivity can be valuable in developing design criteria, field tests are generally more representative of in-situ materials if site conditions and access allow performance of the tests.

Hydraulic conductivity is measured in the laboratory by percolating water through a soil sample of known cross-sectional area and length. The constant head hydraulic conductivity test is conducted for coarse-grained soils in accordance with ASTM D 2434, "Standard Test Method for Permeability of Granular Soils (Constant Head)." This test method is most suitable for granular soils with a hydraulic conductivity greater than 10^{-4} cm/sec that might be used for drains or filter media and that do not have more than 10 percent passing a No. 200 sieve. Soils tested using this procedure are typically compacted in a permeameter to a density comparable to the relative density used for field placement.

For soils that have a hydraulic conductivity less than about 10^{-4} cm/sec, laboratory hydraulic conductivity tests should be conducted in accordance with ASTM D 5084, "Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter." This standard permits hydraulic conductivity testing by either the constant- or falling-head methods and is suitable for soils with hydraulic conductivities not less than about 10^{-9} cm/sec. Thus, the method and equipment are suitable for testing a wide range of soils. The test specimen is sealed within a flexible membrane and enclosed within a pressure cell similar to that used for triaxial testing (Section 6.5.7.4). This setup permits back pressuring to saturate the test sample and application of high hydraulic pressures (gradients) needed for testing low hydraulic conductivity soils and fine coal refuse within a reasonable time frame of several days to a few weeks.

Table 6.8 shows the probable range of hydraulic conductivity for various USCS soil classification groups. For coal refuse, hydraulic conductivity data are less well documented. Based upon a very limited number of field tests and observations, hydraulic conductivity values for coarse coal refuse range from 10^{-6} to 10^{-2} cm/sec (about 1.0 to 10,000 ft/year). Hegazy et al. (2004) report the results of falling head and rising head slug tests that were performed to estimate the hydraulic conductivity of the coarse refuse at coal refuse disposal facilities in western Pennsylvania. The average horizontal hydraulic conductivity k_h was 3×10^{-5} cm/sec (about 30 ft/year), and the standard deviation and coefficient of variation were 2.7×10^{-5} and 0.9, respectively. Density is inversely related to hydraulic conductivity; thus equipment trafficking across the refuse surface and weathering due to exposure following placement tend to increase the density and lower the hydraulic conductivity. These environmental and construction processes can result in a vertical hydraulic conductivity on the order of 10 times less than the horizontal hydraulic conductivity, as discussed in Section 6.6.2.1. For impounding coal refuse disposal facilities, it is important to be aware of the effects of density and grain-size distribution on the hydraulic conductivity of the refuse materials.

As with coarse refuse, the hydraulic conductivity of fine coal refuse varies greatly and is difficult to predict. Typical values based on the results of piezocone dissipation tests performed at northern Appalachian sites are presented in Table 6.36. The range of estimated k_h indicates that fine coal refuse behaves similarly to very fine sands, silts and mixtures of sand, silt and clay. Some designers have reported the presence of moderate plasticity clay with very low vertical hydraulic conductivity at refuse disposal sites. In general, conservative assumptions should be made relative to the hydraulic conductivity of coal refuse, with consideration of field test data when available. Generally, conservative values are assumed based upon judgment and hydraulic conductivity values from the high end of the range determined from available test data, material classification and representative anisotropy values, resulting in either higher (more conservative) phreatic levels or seepage rates.

TABLE 6.36 ESTIMATED HORIZONTAL HYDRAULIC CONDUCTIVITY OF FINE COAL REFUSE BASED ON PIEZOCONE DISSIPATION TESTS

Test	Depth (m)	t_{50} (seconds)	c_h (cm^2/s)	k_h (cm/s)
PCPT1	7.9	800	15×10^{-3}	1.21×10^{-5}
PCPT1	19.2	30,000	0.4×10^{-3}	0.03×10^{-5}
PCPT1	22.9	40	301×10^{-3}	24.3×10^{-5}
PCPT3	13.3	128	94×10^{-3}	7.59×10^{-5}
PCPT4	21.9	300	40×10^{-3}	3.24×10^{-5}

Note: Pool level was approximately 3 m below the ground surface.

(HEGAZY ET AL., 2004)

6.5.5 Geosynthetic Materials Tests

Geosynthetic materials are polymeric sheet materials used with soil, rock, or other geotechnically-related material as an integral part of a civil engineering project, structure, or system. Geotextiles, geomembranes and geosynthetic clay liners (GCLs) are types of geosynthetic materials that may be used in the design and construction of a coal refuse disposal facility to convey or limit seepage or to act as a filter medium.

A geotextile is a permeable geosynthetic made of textile materials. At refuse disposal facilities, geotextiles are used as filters in drainage applications, as well as for material separation applications such as beneath spillway linings and haul roads.

Geomembranes are continuous polymeric sheets with very low hydraulic conductivity (typically less than 10^{-12} cm/sec) in contrast to GCLs, which are sheets of very low hydraulic conductivity, composite barrier material. The geomembrane polymeric types used for refuse disposal applications include: (1) chlorinated polyethylene (CPE), (2) chlorosulfonated polyethylene (CSPE), also called “Hypalon,” (3) high-density polyethylene (HDPE) and (4) polyvinyl chloride (PVC). Of these types, PVC and HDPE are the most commonly used because they are lowest in cost and widely available.

GCLs consist of dry bentonite clay soil between two geotextiles or on a geomembrane carrier. Geomembranes and GCLs are manufactured in sheets and delivered to the site in rolls. The geotextiles used above and below the dry clay may or may not be connected with threads or fibers to increase the in-plane strength. Geomembrane rolls are seamed in the field using thermal methods or solvents, and GCLs are overlapped in the field to create a continuous barrier. At refuse disposal sites, geomembranes and GCLs are used as hydraulic barriers to limit seepage from impoundments into the groundwater and underground mines.

Although not required by MSHA, some state agencies that regulate coal refuse disposal facilities may require linings to control seepage. While placement of a low-hydraulic conductivity, compacted soil liner is permitted, some sites have insufficient material for constructing such a liner. Therefore, geomembrane and geosynthetic clay liners are more commonly used at these sites.

Leakage, rather than hydraulic conductivity, is the primary design concern for geomembrane-lined containment structures. Leakage can occur through poor field seams, poor factory seams, pinholes from manufacture, and puncture holes from handling, placement, or in-service activity. Leakage of geomembrane liner systems can be minimized by specification of an appropriate liner material and implementation of QA/QC procedures during installation.

The quality of geosynthetic material installation can be controlled by testing. Tables 6.37, 6.38 and 6.39 identify applicable quality control test methods published by ASTM or the Geosynthetic Research Institute (GRI) for geotextiles, geomembranes and GCLs, respectively. Laboratory testing such as the gradient ratio test described in ASTM D 5101, "Standard Test Method for Measuring the Soil-Geotextile System Clogging Potential by the Gradient Ratio," can be employed to check for clogging of geotextiles used for filtration. "Geotextile Filter Performance via Long Term Flow (LTF) Tests" (GRI Method GT1) should also be considered. In the design of a slurry impoundment, if a geotextile is to be used instead of a granular filter in a location where clogging of the geotextile would adversely affect the safety of the embankment, testing should be performed with site-specific materials to demonstrate that significant clogging of the geotextile will not occur during the design life of the filter. Additionally, as discussed in Section 6.6.2.3.2, sufficient field instrumentation to monitor the phreatic level near critical drain and filter installations is recommended.

6.5.6 Consolidation Tests

Applying loads to coal refuse and underlying foundation materials causes compressive strains that are either immediate or time-dependent. Immediate strains are usually the result of elastic deformation of solids and compression of voids that are not held open temporarily by trapped pore water. Time-dependent strain is referred to as consolidation and is a function of the movement of pore water.

Immediate strain is common to soils with a low degree of saturation and/or a high hydraulic conductivity. This type of deformation is usually not an important part of the design of an embankment because the resulting movements are generally complete by the end of embankment construction. Situations where immediate strain should be considered include:

- Horizontal and vertical movement of a pipe within the embankment that could cause opening of joints, cracking of the pipe material or changes in the slope of drainage pipes. Special pipes are available that are flexible or have joints that allow longitudinal movement and slight bending.
- Movement beneath rigid structures, such as concrete spillways, that can be cracked or hydraulically affected by differential movement.

The behavior of saturated, fine-grained soils under stress can be very important in disposal facility design. Compressive strain in these soils can occur only through drainage of water from the pores because, at the stresses experienced during construction, both the soil particles and the pore water are essentially incompressible. Therefore, most of the strain occurs slowly, possibly over several months or years. This slow compressive strain is termed consolidation. With saturated fine-grained soils, the major portion of the total strain is due to consolidation. Until the drainage occurs, the excess pore-water pressures can reduce the effective strength of the material and cause instability.

Consolidation can be an important design consideration because, in addition to causing damage to pipes and structures, it can affect the gradient of surface drainage structures and the integrity of a cap following abandonment. The problems it creates may not become apparent until after the disposal facility begins operation, when corrections are most expensive. If an embankment is constructed over soft clay deposits or previously settled fine refuse, the movements resulting from consolidation can be especially large and the excess pore-water pressures can cause instability. Additional problems that can result from consolidation include:

- Settlement of the embankment crest below the design elevation
- Differential settlement that disrupts internal drains
- Cracking of the embankment, particularly in areas where large differential settlements occur over small distances, such as where soft foundation materials abut harder soil or rock at the base of a valley wall

TABLE 6.37 METHODS FOR QUALITY CONTROL TESTING OF GEOTEXTILES

Description	Test Method
Standard Test Method for Biological Clogging of Geotextiles or Soil/Geotextile Filters	ASTM D 1987
Standard Practice for Sampling of Geosynthetics for Testing	ASTM D 4354
Standard Test Methods for Water Permeability of Geotextiles by Permittivity	ASTM D 4491
Standard Test Method for Trapezoid Tearing Strength of Geotextiles	ASTM D 4533
Standard Test Method for Tensile Properties of Geotextiles by the Wide-Width Strip Method	ASTM D 4595
Standard Test Method for Grab Breaking Load and Elongation of Geotextiles	ASTM D 4632
Test Method for Determining the (In-plane) Flow Rate per Unit Width and Hydraulic Transmissivity of a Geosynthetic Using a Constant Head	ASTM D 4716
Standard Test Method for Determining Apparent Opening Size of a Geotextile	ASTM D 4751
Standard Test Method for Index Puncture Resistance of Geotextiles, Geomembranes and Related Products	ASTM D 4833
Standard Test Method for Measuring the Soil-Geotextile System Clogging Potential by the Gradient Ratio	ASTM D 5101
Standard Test Method for Permittivity of Geotextiles Under Load	ASTM D 5493
Standard Test Method for Hydraulic Conductivity Ratio (HCR) Testing of Soil/Geotextile Systems	ASTM D 5567
Standard Test Method for Biological Clogging of Geotextile of Soil/Geotextile Filters	ASTM D 1987
Geotextile Filter Performance via Long Term Flow (LTF) Tests	GRI Test Method GT1
Fine Fraction Filtration Using Geotextile Filters	GRI Test Method GT8

- Restrictions on the rate of fill placement over previously deposited fines
- Settlement of abandoned facilities that disrupts cap integrity and positive surface drainage control

The conventional laboratory test for measuring the consolidation characteristics of a fine-grained soil consists of trimming an approximately 1-inch-thick undisturbed soil sample in a metal ring that prevents lateral expansion. This test is described in ASTM D 2435, "Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading." Porous stones above and below the sample allow excess pore water to drain from the soil as it is subjected to a series of load and unload cycles. Application of a constant vertical load to the sample permits measurement of compression with time. When compression stops, the sample is said to be 100 percent consolidated under the applied load. The load is subsequently removed and the sample undergoes a small rebound. The load is then increased and held constant until the compression stops again. This procedure is continued for several cycles of loading and unloading, and a relationship is developed between the applied load and the compression produced. It is convenient to express the relationship between sample compression and load, as shown in Figure 6.37, such that the logarithm of the applied load is plotted on the horizontal axis and the compression is plotted on the vertical axis in terms of the void ratio of the sample (as the sample is compressed, the void ratio decreases).

TABLE 6.38 METHODS FOR QUALITY CONTROL TESTING OF GEOMEMBRANES

Description	ASTM Test Method
Standard Practice for Sampling of Geosynthetics for Testing	D 4354
Standard Practice for Determining the Integrity of Field Seams Used in Joining Flexible Polymeric Sheet Geomembranes	D 4437
Standard Practice for Determining the Integrity of Factory Seams Used in Joining Manufactured Flexible Sheet Geomembranes	D 4545
Standard Test Method for Index Puncture Resistance of Geotextiles, Geomembranes and Related Products	D 4833
Standard Test Method for Determining Performance Strength of Geomembranes by the Wide Strip Tensile Method	D 4885
Standard Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method	D 5321
Standard Test Method for the Determination of Pyramid Puncture Resistance of Unprotected and Protected Geomembranes	D 5494
Standard Test Method for Large Scale Hydrostatic Puncture Testing of Geosynthetics	D 5514
Standard Test Method for Multi-Axial Tension Test for Geosynthetics	D 5617
Standard Practice for Geomembrane Seam Evaluation by Vacuum Chamber	D 5641
Standard Practice for Pressurized Air Channel Evaluation of Dual Seamed Geomembranes	D 5820
Standard Test Method for Determining Tearing Strength of Internally Reinforced Geomembranes	D 5884
Standard Guide for Selection of Test Methods to Determine Rate of Fluid Permeation Through Geomembranes for Specific Applications	D 5886
Standard Test Method for Measuring Core Thickness of Textured Geomembrane	D 5994
Standard Test Method for Determining the Integrity of Field Seams Used in Joining Geomembranes by Chemical Fusion Methods	D 6214
Standard Practice for the Nondestructive Testing of Geomembrane Seams using the Spark Test	D 6365
Standard Test Method for Determining the Integrity of Non-reinforced Geomembrane Seams Produced Using Thermo-Fusion Methods	D 6392
Standard Guide for the Selection of Test Methods for Flexible Polypropylene (FPP) Geomembranes	D 6434
Standard Guide for Selection of Techniques for Electrical Detection of Potential Leak Paths in Geomembrane	D 6747
Standard Practice for Leak Location on Exposed Geomembranes Using the Water Puddle System	D 7002
Standard Test Method for Strip Tensile Properties of Reinforced Geomembranes	D 7003
Standard Test Method for Grab Tensile Properties of Reinforced Geomembranes	D 7004
Standard Practice for Ultrasonic Testing of Geomembranes	D 7006

< PREVIOUS VIEW

TABLE 6.38 METHODS FOR QUALITY CONTROL TESTING OF GEOMEMBRANES (CONTINUED)

Description	ASTM Test Method
Standard Practices for Electrical Methods for Locating Leaks in Geomembranes Covered with Water or Earth Materials	D 7007
Standard Test Method for Determining the Tensile Shear Strength of Pre-Fabricated Bituminous Geomembrane Seams	D 7056
Standard Specification for Non-Reinforced Polyvinyl Chloride (PVC) Geomembranes Used in Buried Applications	D 7176
Standard Specification for Air Channel Evaluation of Polyvinyl Chloride (PVC) Dual Track Seamed Geomembranes	D 7177
Standard Practice for Leak Location using Geomembranes with an Insulating Layer in Intimate Contact with a Conductive Layer via Electrical Capacitance Technique (Conductive Geomembrane Spark Test)	D 7240

As shown in Figure 6.37, the steeper slope of the void ratio versus log effective stress plot is defined as the compression index C_c. The flatter slope of the void ratio versus log effective stress plot is defined as the recompression index C_{cr}. Typically, C_{cr} is in the range of 0.1 to 0.2 C_c. The compression index is typically used for estimating consolidation settlement of normally consolidated soils and fines. Procedures for determining C_c and C_{cr} are presented in ASTM D 2435. Presenting the test results in the form shown in Figure 6.37 simplifies computation of the estimated compression for a given increment of applied load. Most standard texts on soil mechanics include comprehensive discussions of this test and procedures for using the test data to predict settlement.

Consolidation testing also provides information regarding the time rate of settlement as a function of consolidation stress. The time rate of consolidation settlement is defined by the coefficient of vertical consolidation c_v and the coefficient of horizontal consolidation c_h. The parameter c_v can be used for estimating vertical pore pressure dissipation with time, which is useful for evaluating the rate of consolidation settlement of fills that are large in areal extent. The parameter c_h can be used for estimating horizontal pore pressure dissipation with time, which is important in the design of wick drains. Procedures for determining c_v and c_h are presented in ASTM D 2435. Values for c_v and c_h determined from laboratory consolidation results tend to be conservative (i.e., underpredict the rate of pore-pressure dissipation) and can vary significantly from in-situ values. More reliable values of c_v and c_h can usually be obtained by conducting a dissipation test during piezocone testing (Section 6.4.3.7) or by monitoring piezometers.

TABLE 6.39 METHODS FOR QUALITY CONTROL TESTING OF GEOSYNTHETIC CLAY LINERS

Description	ASTM Test Method
Standard Guide for Storage and Handling of Geosynthetic Clay Liners	D 5888
Standard Test Method for Determining the Internal and Interface Shear Resistance of Geosynthetic Clay Liner by the Direct Shear Method	D 6243
Standard Guide for Acceptance Testing Requirements for Geosynthetic Clay Liners	D 6495
Standard Test Method for Tensile Strength of Geosynthetic Clay Liners	D 6768

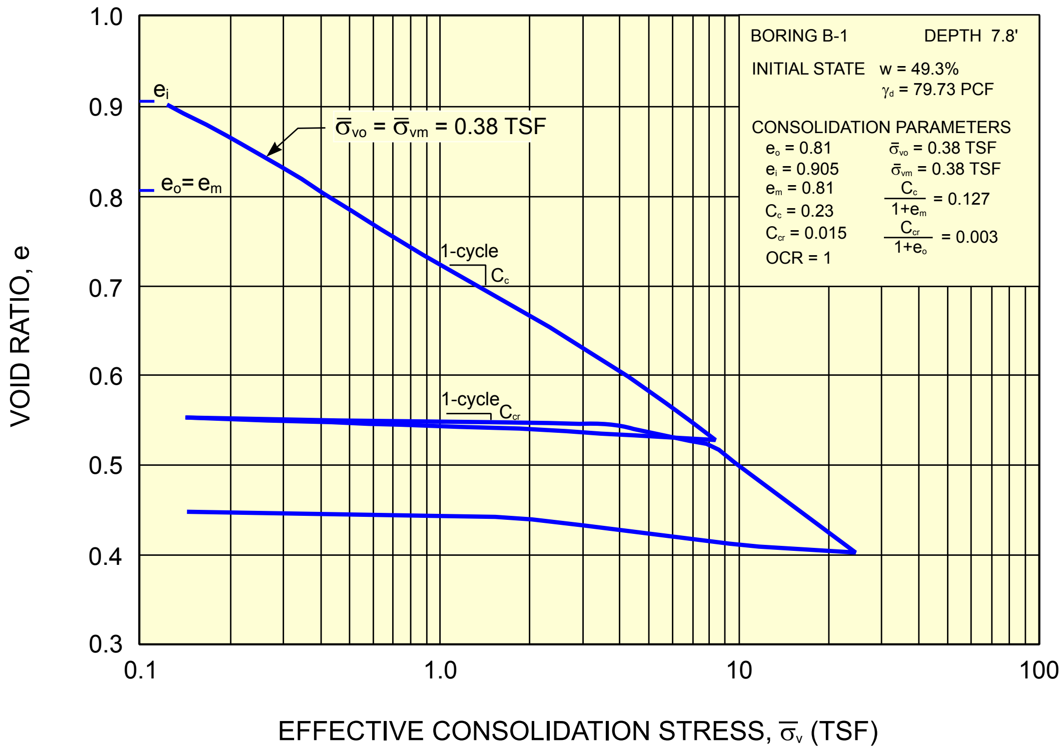


FIGURE 6.37 RESULTS OF CONSOLIDATION TEST ON FINE COAL REFUSE

Coarse coal refuse is normally placed at a moisture content below saturation and is usually sufficiently coarse-grained that consolidation is not a consideration. On the other hand, consolidation of fine coal refuse is important, especially in the design of disposal facilities developed using the upstream method of construction. Consolidation effects should be considered if the incremental increase in height of an embankment constructed on settled fine coal refuse is greater than several feet and the embankment supports drainage structures or seepage barriers that could be impacted by differential settlement. Consolidation parameters may also be estimated from the results of triaxial shear strength tests.

Published data for typical ranges of consolidation parameters for coal refuse are limited. Almes and Butail (1976) report that C_c for saturated fine coal refuse varies between 0.2 and 0.3 at moisture contents ranging between 30 and 45 percent. Hegazy et al. (2004) present values for the horizontal coefficient of consolidation c_h determined from piezocone testing in fine coal refuse deltas at disposal sites in western Pennsylvania. The values of c_h range from 15 x 10^{-3} cm^2/sec to 300 x 10^{-3} cm^2/sec. These results are typical of coefficient of consolidation values reported for sandy silt to silty clay soils (Bardet, 1997).

6.5.7 Shear Strength and Related Tests

The shear strengths of soil and coal refuse materials used to construct an embankment, or used as the embankment foundation, are needed for stability analysis of the embankment. Stability analyses are discussed more extensively in Section 6.6.4. Shear strengths are also needed for determination of the allowable bearing pressure for structures founded on or within the embankment and for the stability of slopes cut during embankment construction.

Embankment and foundation stability may be evaluated using either "total stress" or "effective stress" methods. The method selected depends on the:

- Embankment material or materials
- Foundation conditions

- Magnitude of pore-water pressures within the embankment
- State of construction or use for which embankment stability is to be evaluated

Total stress is a combination of the stress between the individual soil grains, termed "effective stress," and the pressure of the pore water, termed "pore pressure." Because pore water has no shear resistance, all shear resistance is represented by the effective stress. The shear stress at failure (ultimate shear strength) on any surface within an embankment is directly related to the stress normal to the failure surface, because the failure mechanism involves friction of one body moving on another and apparent bonding. This relationship can be expressed as:

$$\tau_{max} = c' + (\sigma - u) \tan \phi' = c' + \sigma' \tan \phi' \tag{6-12}$$

where:

τ_{max}	=	shear stress on surface at failure (force/length^2)
c'	=	effective cohesion (force/length^2)
ϕ'	=	angle of effective internal friction (degrees)
σ	=	total stress acting normal to the failure surface (force/length^2)
u	=	pore-water pressure acting on the failure surface (force/length^2)
σ'	=	effective stress acting normal to the failure surface (force/length^2)

The following paragraphs describe the above two approaches and their application to refuse embankment design.

For shear strength tests conducted for a total stress analysis, water is not allowed to drain from the sample during shearing. This method of stability analysis and related types of analyses (Section 6.6.4) are generally considered most appropriate for evaluating relatively short-term conditions that would occur: (1) during and immediately after construction, (2) immediately following rapid changes in the impoundment level, (3) during pushouts, and (4) during seismic loadings. Tests typically used to develop strength parameters for a total-stress analysis include the vane shear test, the unconfined compression test and the unconsolidated-undrained (UU) triaxial test. For these tests, the strength does not increase with increasing normal stress if the soil is saturated. Total stress analysis parameters can also be calculated from the consolidated-undrained (CU) triaxial shear test.

Shear strength tests for an effective-stress analysis either allow water to drain from the samples during testing or provide for measurement of the pore pressures under loading and confining conditions that are intended to simulate actual field conditions. Effective-strength parameters apply to all soil types, including gravels, sands, silts, and clays. This method of stability analysis and related types of analyses (Section 6.6.4) are generally considered most appropriate for evaluating long-term conditions after the temporary effects of construction on pore pressures have dissipated and seepage rates become steady. The tests typically used to develop effective-stress strength parameters include: (1) consolidated-undrained triaxial shear tests with pore-water pressure measurements ($\overline{\text{CU}}$), (2) consolidated-drained triaxial tests at slow strain rates (CD), or (3) drained direct shear tests. For these tests, the strength increases with increasing normal stress. For long-term analyses, the drained test strength parameters are the effective cohesion intercept c' and effective friction angle ϕ' from the effective stress Mohr-Coulomb envelope. The shear strength τ_{max} is given by:

$$\tau_{max} = c' + \sigma' \tan \phi' \tag{6-13}$$

For analysis purposes, c' is often assumed to be zero because laboratory tests are affected by loading rate and duration effects. In this situation, the cohesion component of strength can be likened to a bond that weathers with time (Mesri and Abdel-Ghaffar, 1993).

Sample preparation for laboratory shear strength tests is an essential aspect of a testing program, if the tests are to accurately reflect field conditions. Undisturbed, disturbed and remolded or compacted samples may be tested depending on the soil and material conditions to be modeled in the total- or effective-stress analyses. For shear strength tests on proposed embankment construction materials, the materials are compacted to the densities and moisture contents that are anticipated to occur within the embankment. Tests for the shear strength of coarse-grained foundation soils are performed on samples that are reconstituted in the laboratory to simulate the in-situ conditions. Tests for the shear strength of fine-grained foundation soils are performed on undisturbed samples obtained from borings or test pits.

Triaxial and direct shear test results are presented either as a series of Mohr's circles or stress paths that reflect sample failure conditions. A Mohr's circle presentation is typically used for UU triaxial and direct shear test results, while a stress path presentation is used for $\overline{CU}$ and CD triaxial test results. An example Mohr's circle presentation is shown in Figure 6.38, and a stress path presentation is provided in Figure 6.39. As shown in Figure 6.38, a Mohr's circle for each test is drawn on a plot of shear stress τ versus normal stress σ by drawing a circle that connects the maximum and minimum principal stresses on the sample at failure (i.e., σ_{1f} and σ_{3f}, respectively). Failure is then defined by a straight or curved line drawn tangent (or nearly tangent) to the series of circles on the plot. The inter-

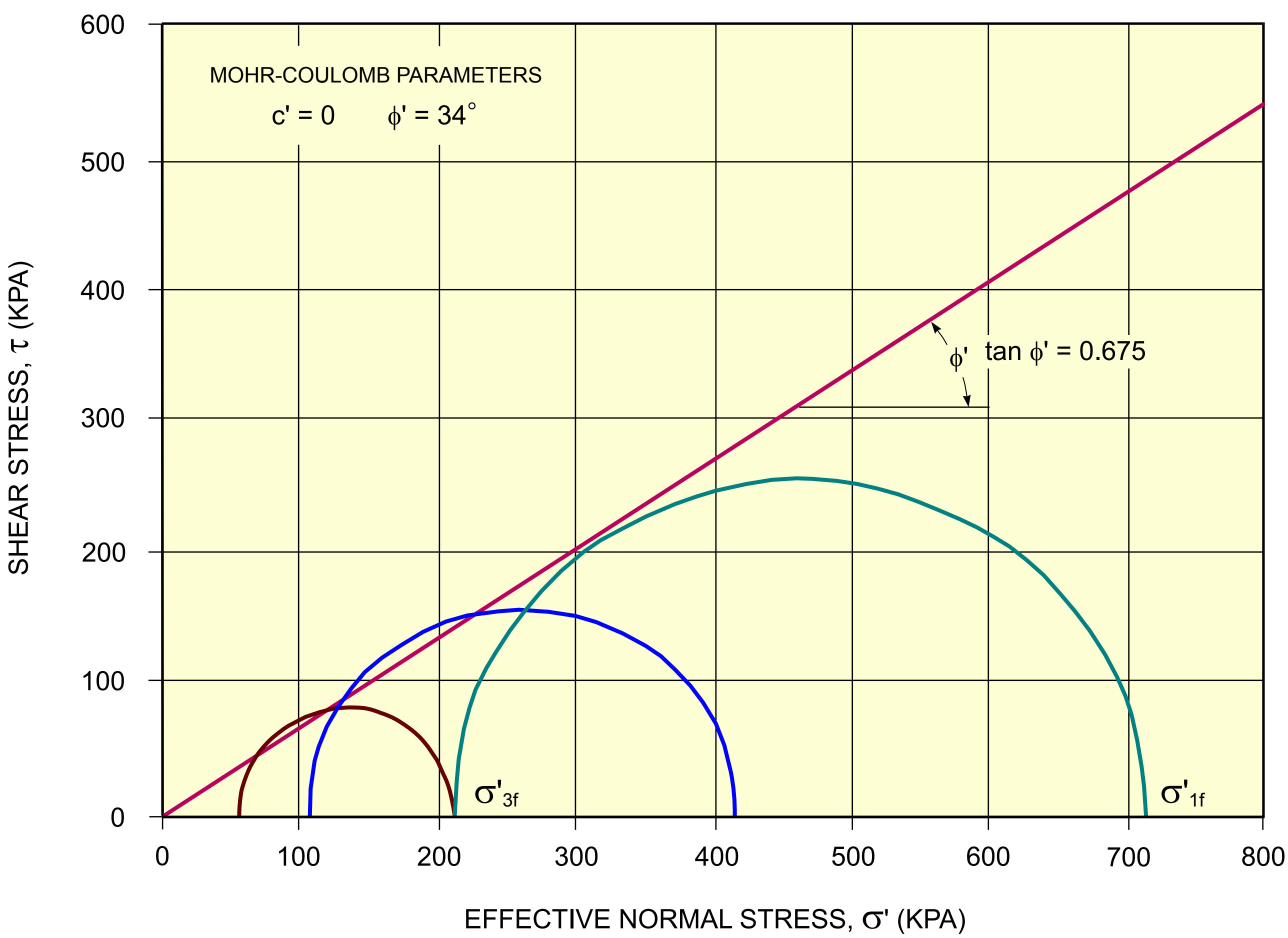


FIGURE 6.38 EFFECTIVE STRESS MOHR'S CIRCLE FOR $\overline{CU}$ TRIAXIAL TEST

cept of the line with $\sigma = 0$ is referred to as the total stress cohesion intercept c and the angle of the line to the horizontal is the total stress angle of friction ϕ.

As shown in Figure 6.39, the stress path for each test is constructed on a plot of $(\sigma_1' + \sigma_3')/2$, or p, versus $(\sigma_1' - \sigma_3')/2$, or q, where values of p and q are plotted for each load increment in each test. Failure is defined by a straight or curved line connecting values of (q/p)max for each stress path. The intercept of this line with the p axis (a) can be represented as $a = c' \cos \phi'$, where the angle to the horizontal of the line connecting values of (q/p)max for each stress path is α and $\tan \alpha = \sin \phi'$. Then the effective angle of friction $\phi' = \arcsin (\tan \alpha)$ and $c' = a/\cos \phi'$.

Many soils exhibit stress-strain behavior that varies with confinement. This behavior is referred to as stress dependency and can be characterized by the stress path method. A stress path is a numerical and graphical representation of the past, present and future state of stress on a representative soil element because it captures the geologic stress history of the element, the current stresses acting on the element, and the anticipated future changes in stress on the element. The stress path of a material is determined by plotting the effective strength from $\overline{\text{CU}}$ and CD triaxial tests for each load increment of the tests. Using the stress path method, the test results are then analyzed with respect to the approximate field stress and strain conditions before, during, and after construction (Lambe, 1967; Lambe and Marr, 1979).

Determining the appropriate strength parameters for evaluating the stability of any embankment, regardless of size or location, should be performed by a person experienced in the engineering behavior of soil, rock and refuse materials. The complexity of laboratory shear strength test procedures for modeling expected field conditions requires that laboratory shear strength testing be conducted with

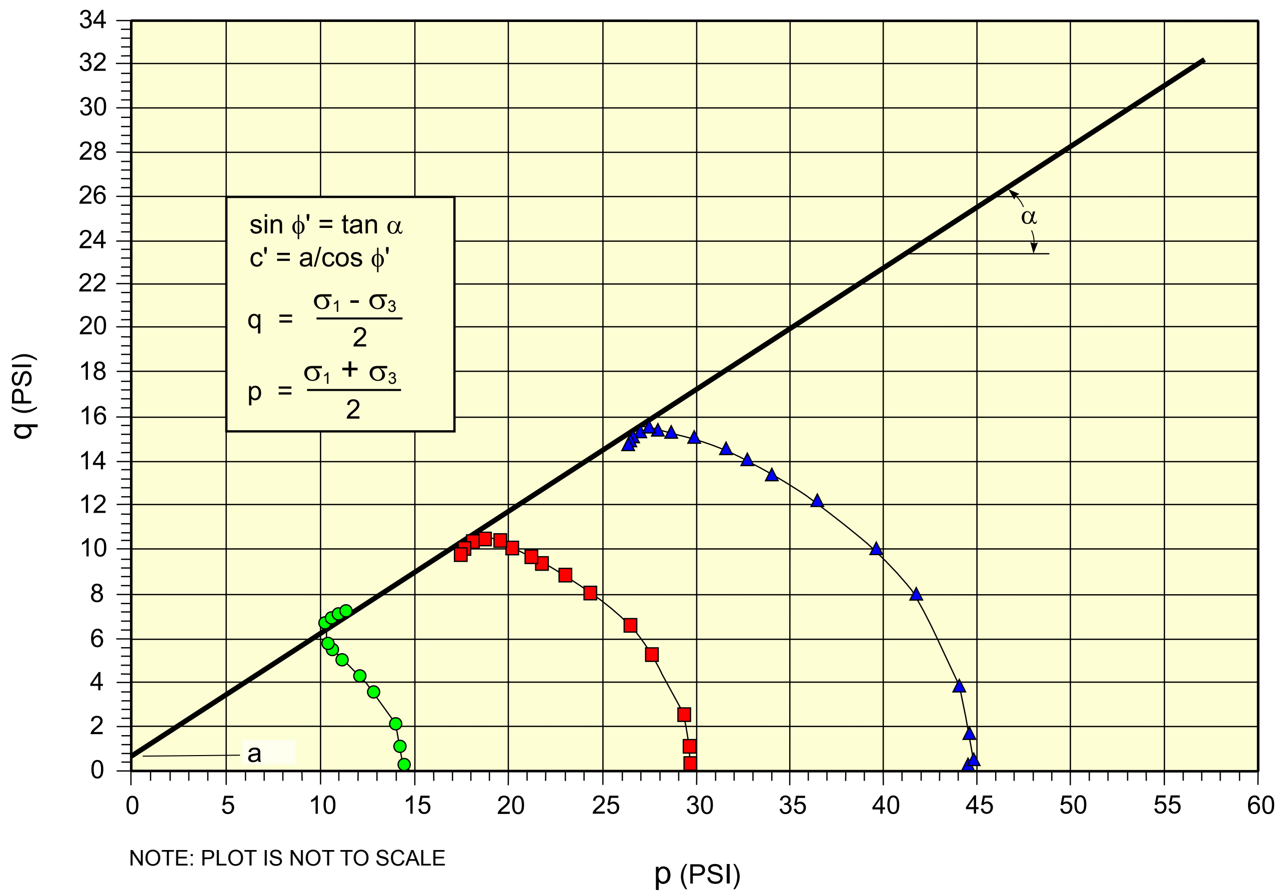


FIGURE 6.39 EFFECTIVE STRESS PATHS AND FAILURE ENVELOPE FOR $\overline{\text{CU}}$ TRIAXIAL TEST

greater care and more professional scrutiny than routine tests. Shear strength testing must be tailored to site conditions by a qualified geotechnical engineer familiar with the type of embankment to be constructed and foundation conditions. The tests should be conducted by laboratories with appropriate equipment and skilled technicians.

Standard shear strength test methods are: (1) vane-shear, (2) direct-shear, (3) unconfined-compression, and (4) triaxial-compression. The applicability of these tests to various soil types is presented in Table 6.40. Descriptions of these tests are provided in the following sections. Head (1982, 1986) and Bardet (1997) discuss other methods for determining shear strength.

6.5.7.1 Vane-Shear Test

The laboratory vane-shear test is used to determine the undrained shear strength s_u using the test method described in ASTM D 4648, "Standard Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil." Similar to the field vane-shear test (Section 6.4.3.8), the laboratory vane-shear test is conducted on very soft to stiff, fine-grained, undisturbed, remolded or reconstituted, cohesive soil by inserting a four-bladed vane into a soil sample and rotating it such that shearing occurs along a cylindrical surface. The undrained shear strength is determined from the resistance to rotation. The miniature vane is similar to the field vane-shear device, except that it has a smaller blade diameter (0.5 inch) and blade height (1 inch). After s_u is determined, the residual (minimum) shear strength s_{ur} is determined by quickly rotating the vane 10 full rotations (to fully remold the soil) and then conducting a second shear test. The ratio of peak to remolded undrained shear strength is the sensitivity S_t. The laboratory vane test is typically conducted on a vertically oriented sample because that is the direction in which the soil sample is taken in the field. If the sample is rotated 90 degrees from the vertical, the laboratory vane test can be used to measure soil anisotropy. Laboratory vane shear testing of fine coal refuse is not recommended. Instead, the strength of fine coal refuse should be determined in situ using the CPT, PCPT methods or the field vane-shear test, as described in Section 6.4.3.8, or by laboratory testing using the direct-shear or triaxial-compression test methods described in Sections 6.5.7.2 and 6.5.7.4, respectively.

6.5.7.2 Direct-Shear Test

The direct-shear test is the oldest and simplest form of shear test. Direct-shear tests are used for testing reconstituted cohesionless soils and undisturbed cohesive soils. The test method is particularly useful if the residual strength at large strain is desired. The test is conducted in accordance with ASTM D 3080, "Standard Test Method for Direct Shear Test of Soils under Consolidated Drained Conditions."

As shown schematically in Figure 6.40, the direct-shear test is performed by placing a ½-inch-minimum-thickness specimen into a cylindrical (2-inch-minimum-diameter) or square-shaped (typically 3 or 4 inches) shear box that is split along a horizontal plane. The test specimen is confined top and bottom by porous stones, and the shear box is placed in a container to permit submergence and saturation of the specimen during testing. A vertical (normal) load is applied over the specimen and allowed to consolidate. The test is conducted by holding the upper or lower part of the box stationary and applying a horizontal load on the other part of the box to shear the specimen along a predefined horizontal plane. The shearing load applied at failure divided by the cross-sectional area of the soil sample is considered to be the shear stress at failure. The normal load divided by the cross-sectional area of the sample is considered to be the normal stress at failure. Direct shear tests of cohesionless soils are considered drained tests because of the high sample hydraulic conductivity. Depending on the rate of shearing, direct-shear tests of cohesive soils can be either undrained or drained.

After the maximum shear strength has been determined, the residual shear strength (c_r' and σ_r') can be determined by performing repeated and rapid cycles of shearing (usually a minimum of five full forward and reverse cycles) along the plane of failure mobilized during the initial portion of the test.

TABLE 6.40 LABORATORY TESTS FOR DETERMINING SOIL SHEAR STRENGTH

Type of Test	Relative Frequency of Use[1]				Preparation Prior to Applying Load	Drainage Conditions During Test	Parameters Determined	Remarks
	Coarse-grained Soils	Fine-grained Soils	Coarse Refuse	Fine Refuse				
<u>Direct Shear</u>								
• Drained	1	3	1	1	Consolidated Under Normal Load	Drained	Effective Stress	Difficult to control rate of test to assure drained condition
• Undrained	NA	3	NA	NA	Consolidated Under Normal Load	Undrained	Approximate Total Stress	Difficult to conduct quickly enough to assure no drainage
<u>Triaxial</u>								
• Unconsolidated-Undrained (UU)	NA	2	NA	3	Uncon-solidated	Undrained	Approximate Total Stress	Also called Quick (Q) test
• Consolidated-Undrained ($\overline{CU}$)	1	1	1	1	Consolidated Under Isotropic Pressure	Undrained	Total Stress and Effective Stress	Pore pressures are measured to give effective stress condition
• Consolidated-Drained (CD)	2	1	2	1	Consolidated Under Isotropic Pressure	Drained	Effective Stress	Also called Slow (S) test
<u>Unconfined</u>	NA	2	2	NA	Unconsolidated	Undrained	Approximate Total Stress	Sample must have sufficient cohesion to maintain shape without support

Note: 1. 1 = frequently used; 2 = occasionally used; 3 = applicable, but seldom used; NA = not applicable

The repeated cycles of loading are intended to simulate large straining in the field that would be typical of a slope failure. Once the cycles of repeated shearing are complete, excess pore pressures in the test specimen are allowed to dissipate under constant normal load. When the excess pore-water pressures equilibrate (i.e., consolidation is complete), the test specimen is sheared as previously described.

As shown in Figure 6.41a, a series of direct-shear tests (typically three minimum) is conducted using varying normal stress σ'. Test results are plotted in the form of shear stress τ versus horizontal displacement δ. A plot of the peak or failure shear stress τ versus σ' is used to determine the angle of internal friction and cohesion intercept, as shown in Figure 6.41b. For most soils, a line drawn through the points for each test is approximately straight. This line is termed the failure envelope. The angle that the line makes with the horizontal axis is a measure of the component of strength due

< PREVIOUS VIEW

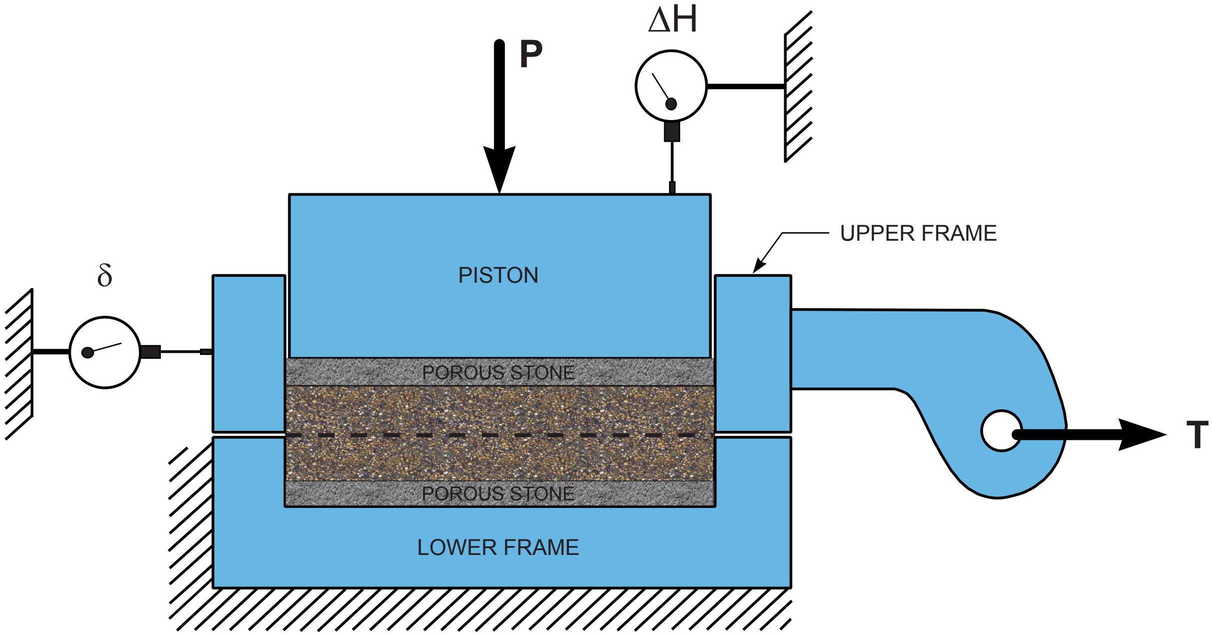


FIGURE 6.40 DIRECT SHEAR TESTING ARRANGEMENT

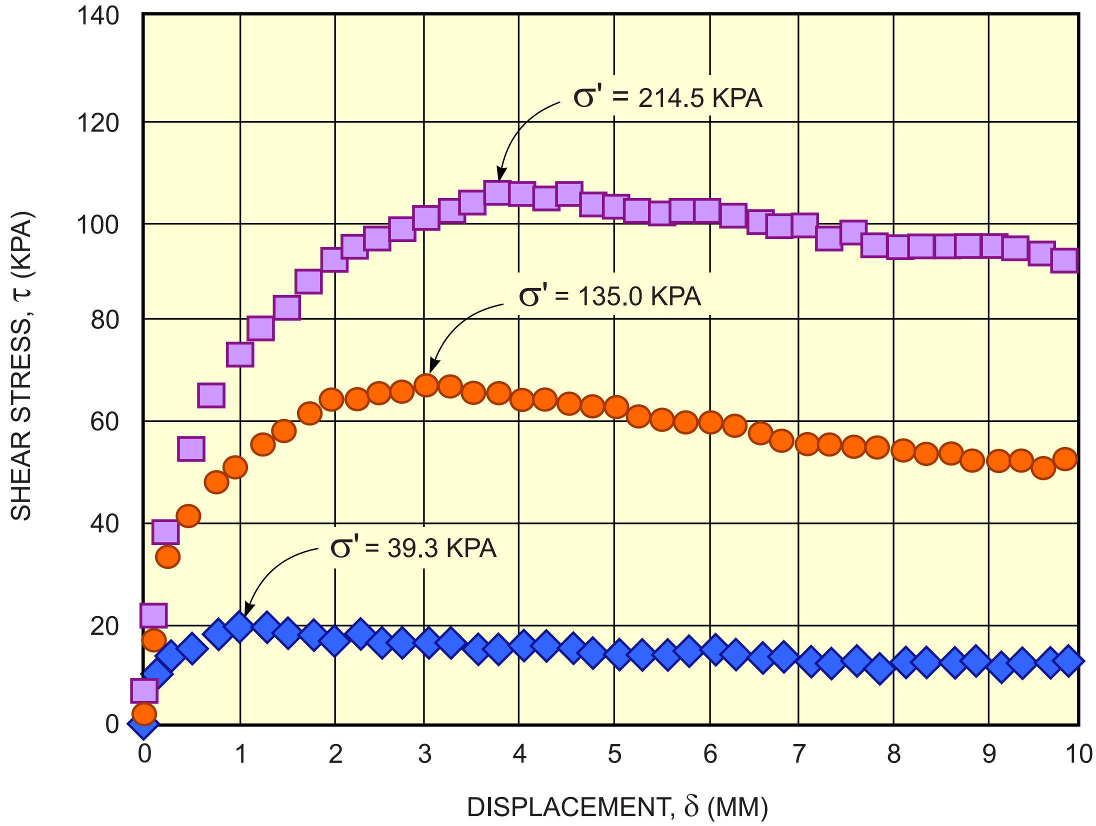
to friction between the soil particles and is termed the angle of effective internal friction ϕ'. When the line intercepts the vertical axis at a value greater than zero, the intercept is referred to as the effective cohesion c'.

Direct-shear tests are simple and can be performed relatively quickly. However, the test has several inherent shortcomings due to the forced plane of shearing:

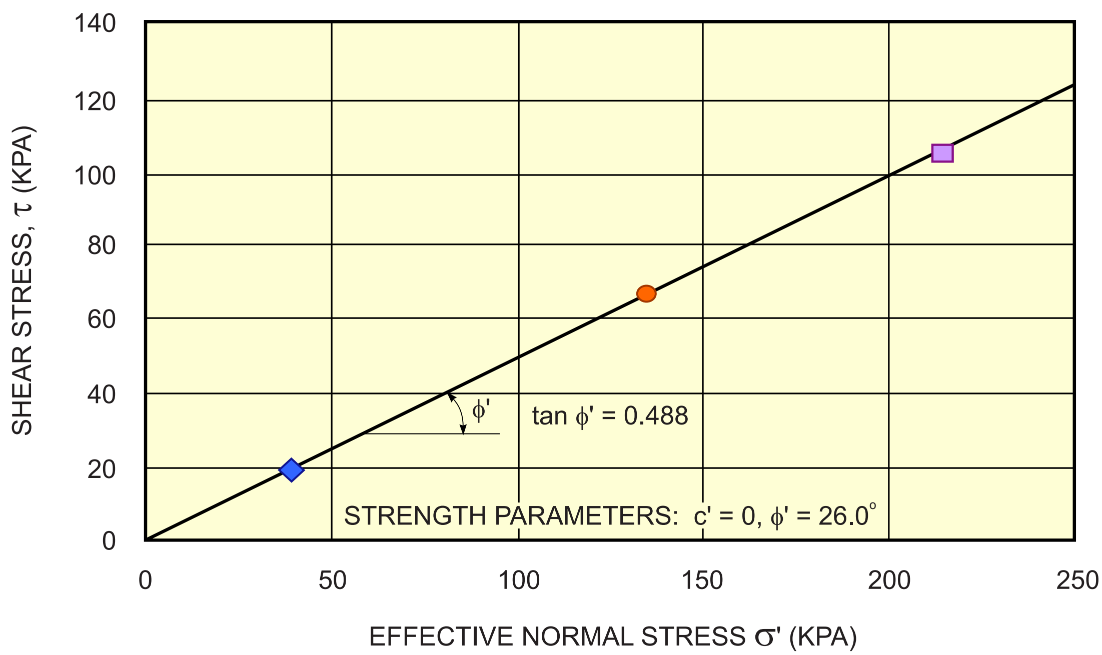
- The failure plane is predefined and horizontal and may not be the weakest plane in the sample.
- There is little control over the drainage of the soil.
- The height cannot be defined for calculating shear strains, so a stress-strain modulus cannot be determined from the test.
- Stress conditions on the failure surface are non-uniform and failure develops progressively (the entire strength of the specimen is not mobilized simultaneously at all points on the failure surface), so measured strength values are lower than would be obtained under uniform stress conditions.
- Stress conditions are known only at failure.

6.5.7.3 Unconfined-Compression Test

The unconfined-compression test is conducted on cohesive soils in accordance with ASTM D 2166, "Standard Test Method for Unconfined Compressive Strength of Cohesive Soil." Cohesive soil specimens are tested to failure by rapidly applying an axial load. Measurements of axial force and axial deformation are made during the test, and the test results are presented as a plot of axial stress versus axial strain as shown in Figure 6.42. The maximum measured force over the sample cross section q_u is the axial stress, and the peak value of axial stress divided by 2 is the undrained shear strength s_u. For a total stress analysis, the unconfined-compression test provides an approximate measure of the short-term, undrained strength of the soil at the density and moisture content of the sample. It provides no information about long-term strength properties appropriate for an effective stress analysis.



6.41a SHEAR STRESS VS. DISPLACEMENT



6.41b SHEAR STRESS VS. EFFECTIVE NORMAL STRESS

NOTE: DATA BASED UPON DIRECT SHEAR TEST OF TRIASSIC CLAY IN RALEIGH, NC

(MAYNE ET AL., 2002)

FIGURE 6.41 RESULTS OF DRAINED DIRECT SHEAR TEST ON CLAY

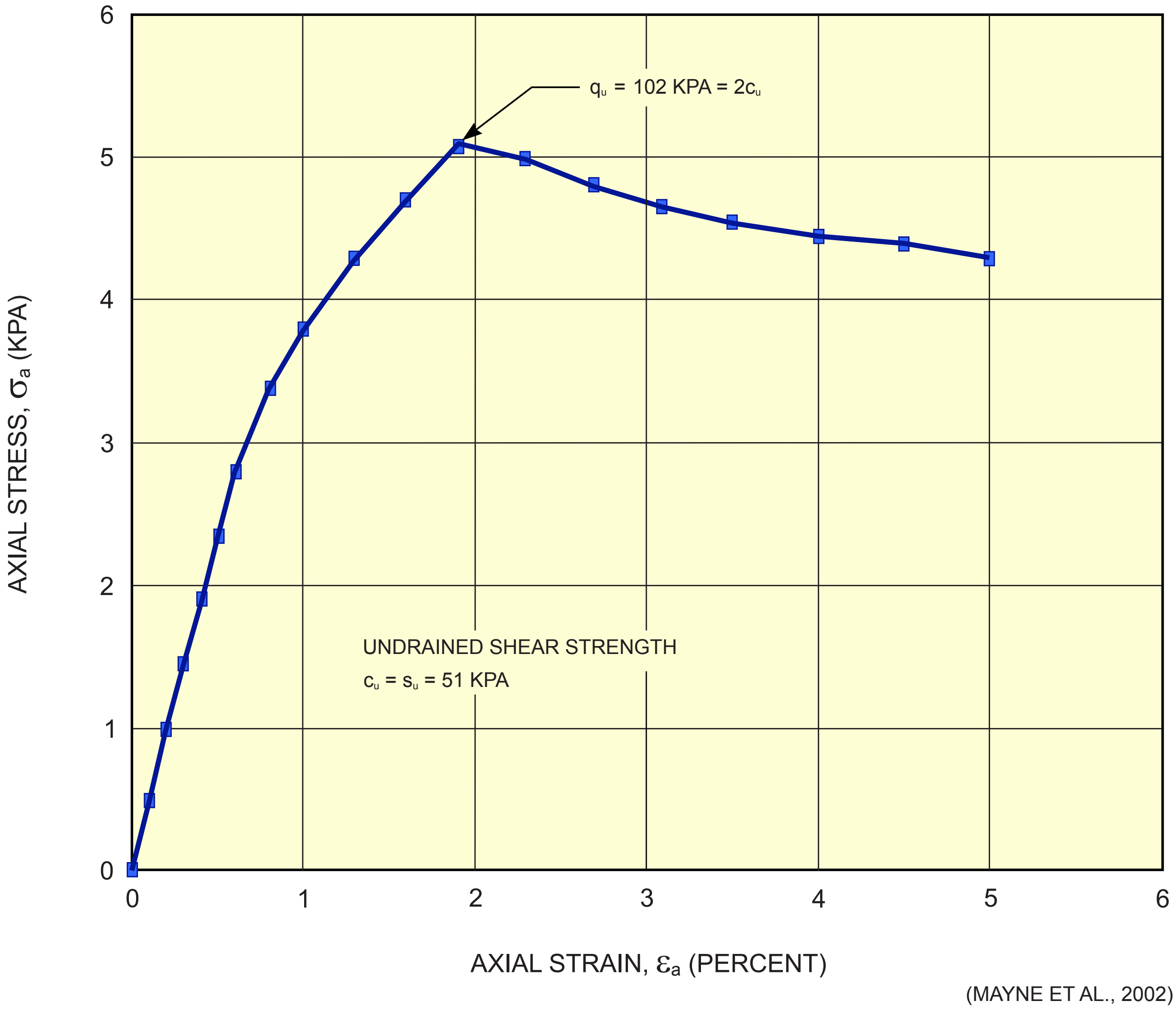


FIGURE 6.42 UNCONFINED COMPRESSION TEST RESULTS

The unconfined-compression test can be performed using undisturbed, remolded or compacted soil samples. The stress-strain curves and failure modes observed during testing provide an index value of the soil properties in addition to strength. For example, bulging or yielding of the sample signifies a relatively soft clay, while a sudden brittle failure indicates a desiccated clay or cemented material. The stress-strain curves developed from these tests should be used with caution when determining the soil modulus for input to numerical analyses (e.g., finite element analysis) because they are very sensitive to minor variations of the modulus.

Test specimens with inclined fissures, sand and silt lenses or slickensides have a tendency to fail prematurely along these weaker planes in unconfined compression tests. If these failure modes occur, more sophisticated testing, such as triaxial tests, may be needed to obtain a more realistic determination of the in-situ strength.

6.5.7.4 Triaxial-Compression Test

The triaxial test is used to determine strength and stress-strain behavior of undisturbed, remolded or reconstituted soil samples. To conduct a triaxial test, cylindrical samples are consolidated, usually isotropically, and then sheared in axial compression. Undrained and drained testing can be conducted, and pore-water pressures can be measured during undrained shear tests. Triaxial tests provide a reliable means for determining: (1) the undrained strength of cohesive soils, (2) the angle of friction and cohesion intercept of undisturbed, reconstituted and compacted soils, and (3) the soil modulus at intermediate to large strains.

Test specimens are typically 1.4 to 2.8 inches in diameter with a height to width ratio between 2 and 2.5. Selection of the sample diameter is governed by limitation of the maximum particle size in the test specimen to not more than one-sixth of the sample diameter. Thus, the maximum particle size for a 1.4-inch-diameter sample is about ¼ inch and for a 2.8-inch-diameter specimen about ½ inch. If the soil to be tested has large-size particles, then sufficiently large-diameter specimens should be used so that the sample diameter is more than six times the maximum particle size. Otherwise, the tested sample should be modeled or scalped using the procedures recommended by Becker et al. (1972) and summarized in Duncan and Wright (2005)

Modeling entails creating a modeled particle-size distribution that parallels the original gradation, where the maximum particle size does not exceed one-sixth of the diameter of the tested sample. Using this approach, Becker et al. (1972) determined that strength test results using the modeled gradation were essentially the same as the strength of the original gradation that had been tested using sufficiently large diameter samples to meet the one-sixth criterion provided the test specimens were prepared to the same relative density (Section 6.5.3.2).

Scalping entails using that portion of the sample that remains after sieving to remove particle sizes that exceed the one-sixth criterion. As with modeling, Becker et al. (1972) determined that strength test results using a scalped gradation were essentially the same as the strength of the original gradation provided that the test specimens were prepared to the same relative density. However, if either modeling or scalping is used to achieve an acceptable particle-size distribution for testing, additional testing to determine the minimum and maximum densities of both the original and modeled or scalped materials must be conducted to verify that the relative density of the modeled or scalped material is approximately equal to the relative density of the original material.

Of the options for achieving an acceptable particle-size distribution for testing, scalping is probably the simpler and less costly approach. Scalping also does not result in an appreciable shifting of the fines content, which could affect the strength test results if a substantial portion of over-size material must be removed for testing. While these procedures can be used for coal refuse, they should be applied with caution, particularly in cases where the characteristics (e.g., material type, angularity, surface roughness) of the smaller particles differ materially from those of the larger particles that were removed from the sample.

To conduct the triaxial test, a sample is enclosed by a thin rubber membrane and placed inside a cylindrical pressure chamber that is usually filled with water. The sample is subjected to a uniform confining pressure σ_3 by compression of the fluid in the chamber acting on the membrane. The range of σ_3 for a triaxial test series is generally selected so that the confining pressures are higher, lower and about equal to anticipated in-situ value of σ_3 at the end of construction. Using a range of σ_3 where all confining pressures are less than or equal to the anticipated in-situ value of σ_3 can result in overestimation of the strength and compressibility of the tested material, as compared to the in-situ material. Depending on the type of triaxial test conducted, a backpressure may be applied to the specimen through the end platens to saturate the specimen. The test sample is sheared to failure by applying an axial stress, typically referred to as the deviator stress (σ_1 - σ_3), through a vertical loading ram. Axial stress can be applied at a constant deformation rate (strain controlled) or by means of dead weight increments or hydraulic pressure (stress controlled) until the sample fails.

Triaxial tests can be used to simulate various in-situ loading conditions. The types of triaxial tests typically employed for this purpose include:

- Unconsolidated-undrained (UU or Q) test
- Consolidated-undrained (CU test)
- Consolidated undrained ($\overline{\text{CU}}$ or R) test with pore-pressure measurement
- Consolidated-drained (CD or S) test

UU (also referred to as quick or Q) triaxial tests are conducted on cohesive soils in accordance with ASTM D 2850, "Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils." The UU test provides only an approximation of the short-term or undrained strength for a total stress analysis and typically is conducted in order to provide data for preliminary analyses and for designing the final test program. The UU test does not provide data about the effective stress or long-term properties of the material. In a UU test, the specimen is not allowed to consolidate during application of σ_3 or to drain during the testing, so the strength measured is the undrained shear strength s_u. The rate of axial deformation during shear is comparable to the rate used for unconfined compression tests. The results of undrained tests depend on the degree of saturation S_r of the specimens. If $S_r \approx 100$ percent, testing of similar samples will provide similar values of s_u, because the shear strength of the test sample will not increase with increasing confining pressure. However, if $S_r \leq 95$ percent, increasing σ_3 may result in increasing values of s_u until the air voids compress and the sample becomes completely saturated.

If water is allowed to completely drain from a test sample when σ_3 is applied, the sample becomes uniformly consolidated. Two types of tests can be performed on consolidated samples. In the consolidated-undrained (CU) test, the consolidated sample is sheared by application of an axial test load without allowing any additional water to drain during the loading. Because the sample does not drain during loading, the results are suitable only for total stress analyses. The rate of strain used for the CU triaxial test is similar to that for the UU triaxial test.

If the pore pressure is measured while the sample is loaded, effective stress parameters can be calculated. This test is called a consolidated undrained with pore-pressure measurement ($\overline{\text{CU}}$ or R) triaxial test. The $\overline{\text{CU}}$ triaxial test permits determination of both total stress and effective stress (c′ and ϕ') parameters. The rate of strain used for the $\overline{\text{CU}}$ triaxial test is much slower than for the UU or CU triaxial tests so that excess pore pressures equilibrate throughout the test specimen and pore-water pressures can be reliably measured at the ends of the test specimen. The $\overline{\text{CU}}$ test is performed in accordance with ASTM D 4767, "Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils." The rate of strain used for the $\overline{\text{CU}}$ triaxial test is prescribed in ASTM D 4767 and is much slower than the rate of strain for UU triaxial tests. This permits equalization of excess pore pressures during undrained shearing so that the excess pore-water pressures measured at the end of the test specimen are not less than 95 percent of the excess pore-water pressure along the sample shear plane.

Consolidated-drained (CD) triaxial (also referred to as slow or S) tests also yield the effective stress parameters c' and ϕ'. The primary difference between the $\overline{\text{CU}}$ and CD triaxial tests is that the sample is allowed to drain during the CD test. The rate of strain used for CD tests is usually much slower than the rate used for $\overline{\text{CU}}$ tests, so that the development of excess pore pressures in the test sample is less than a few percent of the initial effective confining pressure σ_3'. The CD test measures only the effective stress parameters c′ and ϕ'.

Triaxial test results are typically presented in Mohr's Circle or p-q plots, as described previously.

Typical values of the shear strength from triaxial testing of fine coal refuse, as reported by Hegazy et al. (2004) for sites in northern Appalachia, are summarized in Table 6.41. Drained-shear-strength parameters were determined using consolidated isotropic undrained compression (CIUC) triaxial tests with pore-pressure measurements and consolidated isotropic drained compression (CIDC) tri-

TABLE 6.41 SUMMARY OF FINE COAL REFUSE SHEAR STRENGTH PARAMETERS BASED ON TRIAXIAL TEST RESULTS

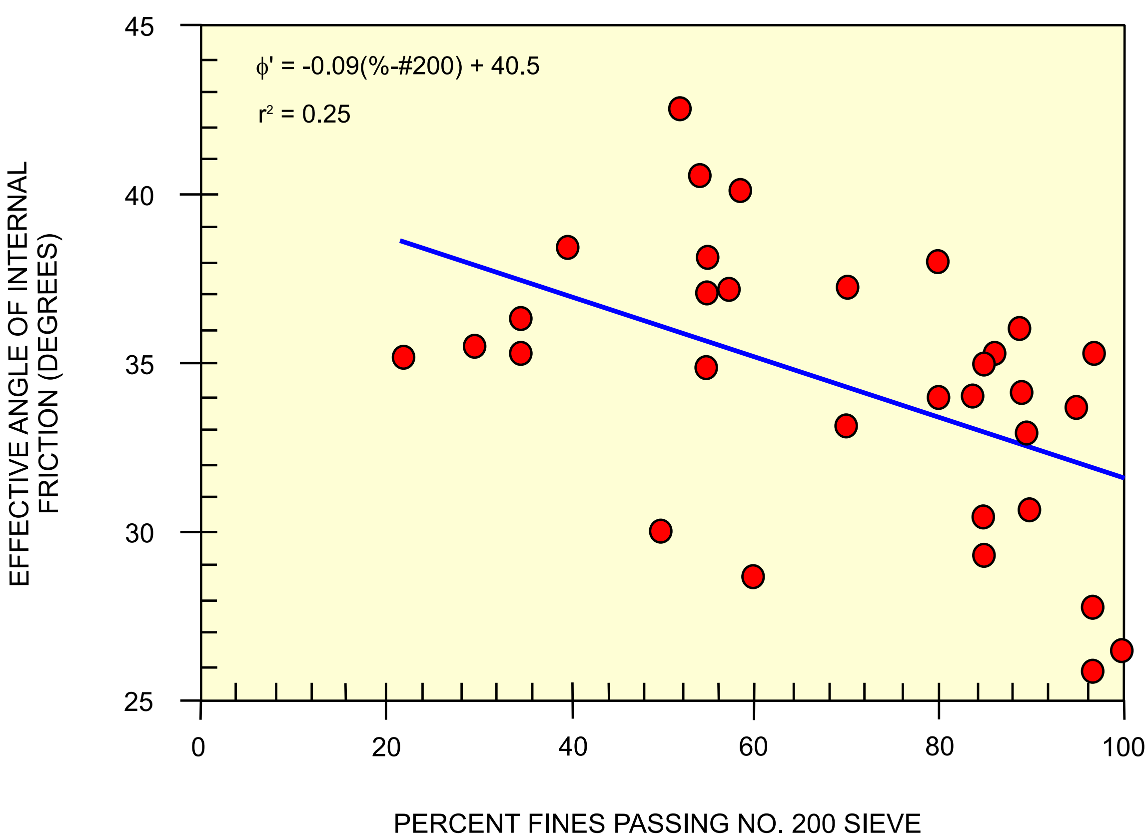
Parameter	Average	Standard Deviation	Coefficient of Variation
φ'	33 degrees	4	0.12
c'	11 kPa	14	1.30
φ' (c' = 0)	35 degrees	4	0.11

(HEGAZY ET AL., 2004)

axial tests. In this study, fine coal refuse samples from Shelby tubes were collected from beneath the upstream stages of a refuse embankment or from working platforms built over the fine coal refuse in the impoundment. Table 6.41 indicates that the variability of ϕ' is low to moderate, while the variability of c' is relatively high. The shear strength parameters presented in the table are peak values and were found to decrease with increasing fines contents, as shown in Figure 6.43.

Residual shear strength values at large strains determined in accordance with the cited ASTM standards are sometimes considered for the design of impoundments with upstream construction.

The test results in Table 6.41 and Figure 6.43 are presented for illustration purposes only, and strength characteristics will vary depending on the geology and coal extraction, processing, and disposal practices.



(HEGAZY ET AL., 2004)

FIGURE 6.43 VARIATION OF STRENGTH OF FINE COAL REFUSE WITH FINES CONTENT

6.5.8 Seismic Property Characterization

Laboratory testing for determination of the seismic properties of soil or refuse materials generally involves the evaluation of potential strength loss associated with earthquake loading. This section presents testing methods for: (1) cyclic-triaxial testing, (2) cyclic loading followed by monotonic loading, and (3) resonant-column testing. Chapter 7 presents details of seismic stability and deformation analyses, including the application of the laboratory strain-based approach developed by Castro (1994), sometimes referred to as the residual- or steady-state-strength approach. Section 7.4.3.2 presents guidance for laboratory testing and application of the laboratory tests below.

6.5.8.1 Cyclic-Triaxial Test

The cyclic-triaxial test can be used to evaluate the cyclic strength (or liquefaction potential) of primarily cohesionless, free-draining soils in undrained shear. The samples tested are either undisturbed, or they are reconstituted to simulate the relative density of the in-situ soil. The test apparatus consists of a regular triaxial cell and a cyclic (often sinusoidal) loading machine attached to the loading piston. The sample is isotropically consolidated in the triaxial cell and then subjected to a cyclic axial load in extension and compression. The cyclic loading generally causes an increase in pore-water pressure and a decrease in the effective confining pressure with increasing cyclic deformation of the sample. Failure occurs when the excess pore-water pressure equals the initial effective confining pressure (sometimes called initial liquefaction) or when some limiting cyclic or permanent strain is mobilized. Details regarding the test method are described in ASTM D 5311, "Standard Test Method for Load Controlled Cyclic Triaxial Strength of Soil."

There are limitations to use of cyclic triaxial tests for representing field conditions during earthquake loading, including:

- Non-uniform stress conditions imposed on the test sample by the end platens can cause a redistribution of the void ratio.
- There would be a continuous reorientation of the principal stresses in the field whereas the reorientation angle is either 0 or 90 degrees in the laboratory test.
- The laboratory test sample is isotropically consolidated, whereas the material sampled would be in an at-rest lateral earth pressure (K_o) condition in the field (i.e., lateral stress = K_o times vertical stress).
- Cyclic shear stress is applied on a horizontal plane in the field but on a 45-degree plane in the triaxial test.
- The mean normal stress in the field is constant while the mean normal stress in the laboratory varies cyclically.

Despite these limitations, the cyclic triaxial test has been used with reasonable success since the early 1960s.

6.5.8.2 Cyclic Loading Followed by Monotonic Loading

The purpose of this type of testing is to evaluate the post-earthquake, residual, steady-state, undrained strength of clay-like materials for post-earthquake stability analyses. A limited number of loading cycles is applied to a test sample to model the straining induced by earthquake loading. Monotonic loading is then applied in order to determine the post-earthquake strength. The initial portion of the test consists of a cyclic triaxial test (Section 6.5.8.1) followed monotonic loading using the $\overline{CU}$ triaxial test (Section 6.5.7.4). Selection of the cyclic stress ratio and number of cycles to be applied during cyclic loading depends on requirements discussed in Section 7.4.3.2. Typically, there is a holding period between the end of cyclic loading and the beginning of monotonic loading to permit equilibration of excess pore-water pressures so that measurement of pore-water pressures at the ends of the sample during $\overline{CU}$ testing are representative of pore-water pressures along the sample shear plane. In reality, the time needed to transition between cyclic and $\overline{CU}$ testing is sufficient to permit pore-water pressure equilibration in the sample. Application of this type of testing program for seismic design is discussed in Section 7.4.3.2.

6.5.8.3 Resonant-Column Test

Evaluation of the response of foundation and embankment soils to seismic ground amplifications requires information regarding shear modulus (G_{max} or G_o) and damping D. While field geophysical methods such as the crosshole, downhole, and surface wave techniques can provide direct in-situ measurements of shear wave velocity (Section 6.4.4), the resonant-column test permits an evaluation of the variation (decrease) of shear modulus with increasing shear strain γ_s and the increase of D with γ_s under controlled effective stress states. The test may yield lower values than those obtained from field testing due to the effect of soil aging.

The resonant-column test is conducted in accordance with ASTM D 4015, "Standard Test Methods for Modulus and Damping of Soils by the Resonant-Column Method." The undisturbed or reconstituted test specimen is sealed in a flexible membrane and enclosed in a pressure cell similar to that used for triaxial testing (Section 6.5.7.4). This setup permits the use of back pressure to saturate the test specimen. The resonant-column device excites one end of the test sample in a fundamental mode of vibration by means of torsional or longitudinal motion. Both solid and hollow specimens can be used in the apparatus. Either a sinusoidal torque or a vertical compressional load is applied to the top of the sample through the top cap. The deformation of the top of the sample is measured, and the excitation frequency is adjusted until the sample resonates. The wave velocity or modulus is computed from the resonant frequency and the geometric properties of the sample and driving apparatus. Damping is determined by switching off the current to the driving coil at resonance and recording the amplitude of decay of the vibrations. The decay of the amplitude with time is used to determine the logarithmic decrement (the percentage decay over one log cycle of time), which is directly related to the viscous damping ratio. Typical test results are presented in Figure 6.44. Figure 6.44a shows the decrease in shear modulus with increasing strain amplitude, and Figure 6.44b shows the increase in damping with increasing strain amplitude for a clay soil.

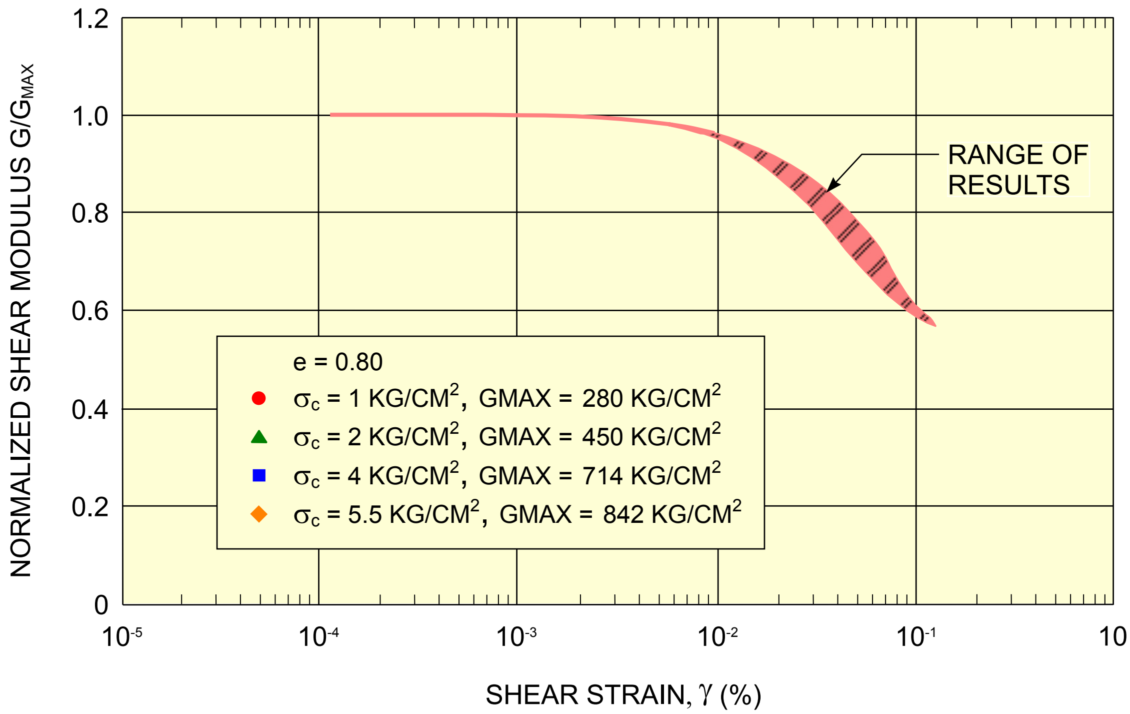
The resonant-column test is generally limited to small to intermediate shear strains by the applied force requirements and resonant frequencies. At larger strains, hollow samples must be used to maintain a relatively constant shear strain across the sample. For these reasons, resonant column testing is primarily used to estimate shear modulus associated with small strains.

6.5.9 Rock Property Tests

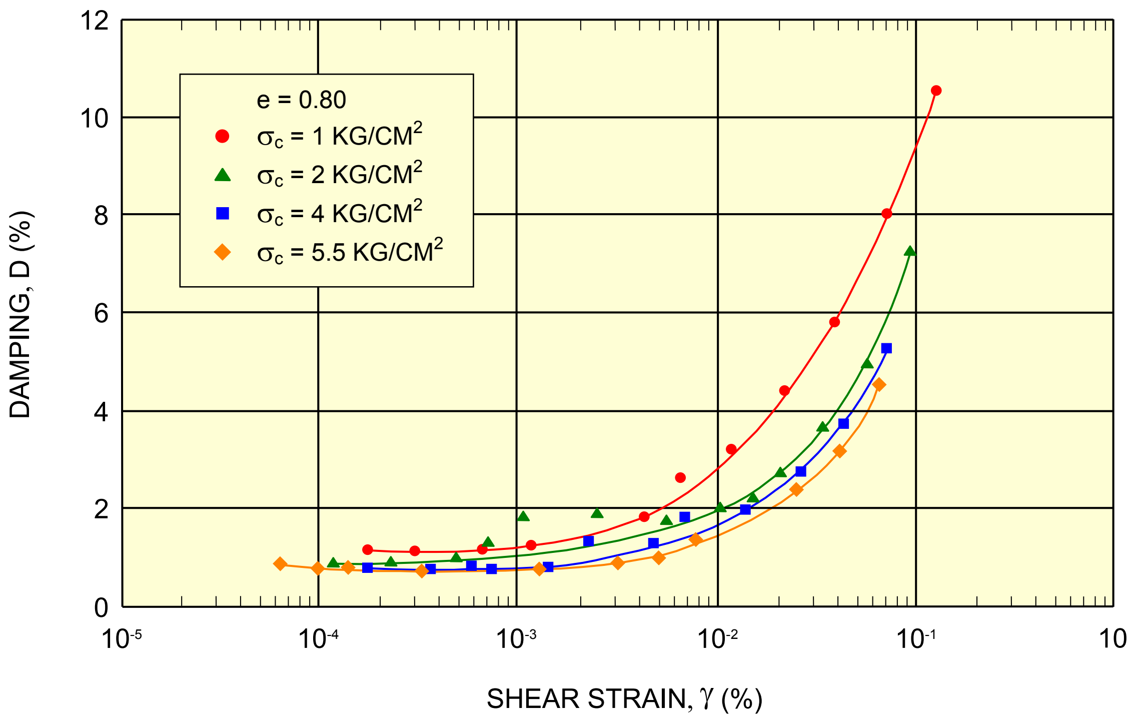
Common laboratory tests for engineering properties of intact rocks and index testing of rock fragments include measurements of strength, stiffness, and durability. Table 6.42 presents a summary list of ASTM standards and procedures for laboratory rock testing. Additional discussion of the testing of coal is presented in Section 8.4.2.2.

6.5.9.1 Point-Load Index Test

Determination of rock strength is typically determined in the laboratory using specially prepared rock core and specialized test equipment. Because of the extensive sample preparation and equipment requirements, the point-load test was developed so that rock specimens from drilled core, cut blocks or irregular lumps could be tested using portable equipment suitable for the field or laboratory. The point-load test is conducted using an apparatus (Figure 6.45) that applies a concentrated load through a pair of spherically truncated, conical platens. The distance between the opposing specimen-platen contact points is recorded, and the load is steadily increased until the specimen fractures and the failure load is recorded. The point-load test is conducted in accordance with ASTM D 5731, "Standard Test Method for Determination of the Point Load Strength



6.44a DECREASE IN SHEAR MODULUS WITH STRAIN



6.44b INCREASE IN DAMPING WITH STRAIN

(ELLISON AND CHO. 1976)

FIGURE 6.44 TYPICAL RESULTS FOR RESONANT COLUMN TEST ON FINE COAL REFUSE

TABLE 6.42 STANDARDS AND PROCEDURES FOR LABORATORY TESTING OF INTACT ROCK

Test Category	Name of Test	ASTM Test Method
Point Load Strength	Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classification	D 5731(1)
Compressive Strength	Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures	D 7012(1)
Creep Tests	Standard Test Method for Creep of Rock Core Under Constant Stress and Temperature	D 7070
Tensile Strength	Standard Test Method for Direct Tensile Strength of Intact Rock Core Specimens	D 2936
	Standard Test Method for Splitting Tensile Strength of Intact Rock Core Specimens	D 3967(1)
Direct Shear	Standard Test Method for Performing Laboratory Direct Shear Strength Tests of Rock Specimens under Constant Normal Force	D 5607(1)
Hydraulic Cond.	Standard Test Method for Permeability of Rocks by Flowing Air	D 4525
Durability	Standard Test Method for Slake Durability of Shales and Similar Weak Rocks	D 4644(1)
	Standard Test Method for Testing Rock Slabs to Evaluate Soundness of Riprap by Use of Sodium Sulfate or Magnesium Sulfate	D 5240
	Standard Test Method for Evaluation of Durability of Rock for Erosion Control under Freezing and Thawing Conditions	D 5312
	Standard Test Method for Evaluation of Durability of Rock for Erosion Control under Wetting and Drying Conditions	D 5313
Deformation and Stiffness	Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures	D 7012
	Standard Test Method for Laboratory Determination of Pulse Velocities and Ultrasonic Elastic Constants of Rock	D 2845
Specimen Preparation	Standard Practices for Preparing Rock Core as Cylindrical Test Specimens and Verifying Conformance to Dimensional and Shape Tolerances	D 4543
	Standard Practice for Preparation of Rock Slabs for Durability Testing	D 5121

Note: 1. The rock test procedure associated with this ASTM standard is described in the Chapter 6 text. Additional discussion of the testing of coal is presented in Section 8.4.2.2.

(ADAPTED FROM MAYNE ET AL., 2002)

Index of Rock." The test is used to classify and characterize rock that has a compressive strength greater than 2,200 psi.

ASTM recommends that test samples conform to size and shape requirements. In general, for diametral tests, core specimens with a length-to-diameter ratio of 1.0 are adequate, while for axial tests, core specimens with length-to-diameter ratio of 0.3 to 1.0 are suitable. Specimens for the block and the irregular lump test should have a length of 50 ± 35 mm and a length-to-width ratio between 0.3 and 1.0 (preferably close to 1.0). Samples are typically tested at their natural moisture content.

Size corrections are applied to obtain the point-load strength index $I_{S(50)}$ of a rock specimen. A strength anisotropy index $I_{a(50)}$ is determined when $I_{S(50)}$ values are measured perpendicular and parallel to

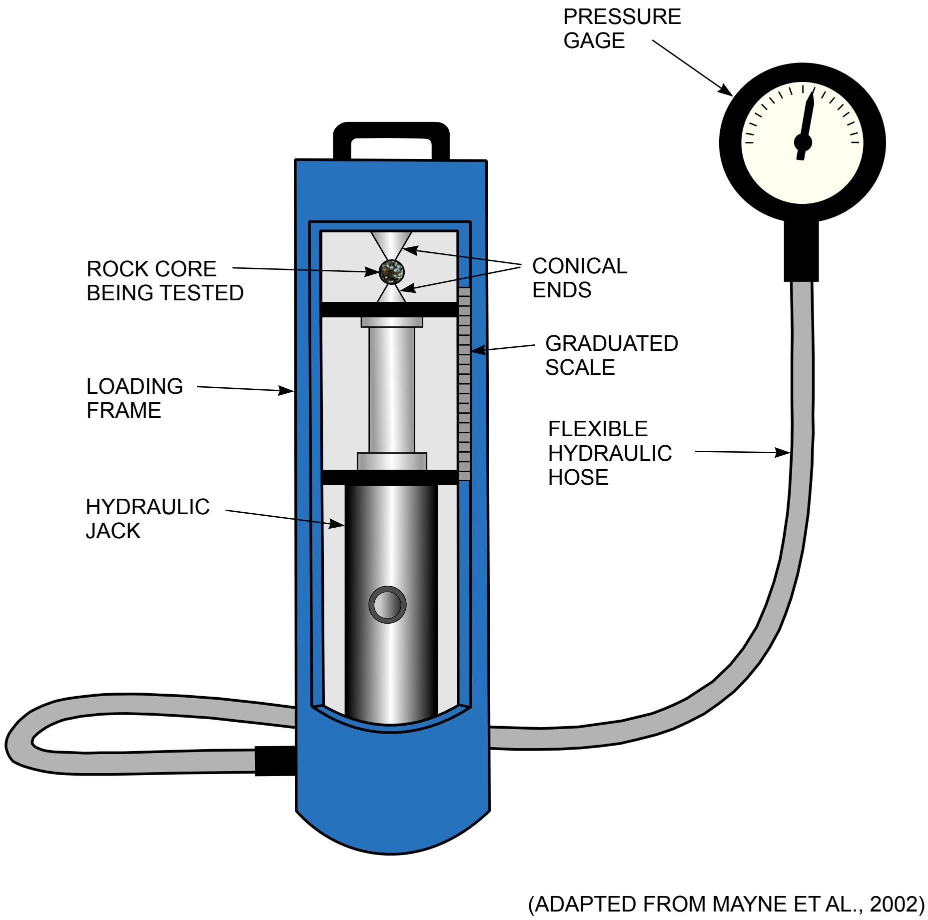


FIGURE 6.45 POINT-LOAD TEST APPARATUS

planes of weakness. The test can be performed in the field or in the laboratory. The point-load index is used to estimate the unconfined compressive strength q_u using a relationship of the form:

$$q_u = K\, I_{s(50)} \qquad (6\text{-}14)$$

The value of the constant K has been reported to vary from 15 to 50 (especially for anisotropic rocks) depending upon the specific rock formation. Rusnak and Mark (2000) determined that $K \approx 21$ for rocks associated with coal seams in the eastern, mid-western and western U.S. Additional discussion of the limitations of testing of coal is presented in Section 8.4.2.2.

6.5.9.2 Unconfined Compressive Strength Test

The unconfined compressive strength serves as an initial index of the competency of intact rock and represents the most direct method for determining rock strength. The test method is described in ASTM D 7012, “Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures.” In this test, cylindrical rock specimens are tested in compression without lateral confinement. The test procedure is similar to the unconfined compression test for soils and concrete. The test specimen should be a rock cylinder of length-to-width ratio in the range of 2 to 2.5 and should have flat, smooth, and parallel ends cut perpendicular to the cylinder axis in accordance with ASTM D 4543, “Standard Practices for Preparing Rock Core Specimens and Determining Dimensional and Shape Tolerances.” The peak stress during unconfined loading is the uniaxial compressive strength q_u. The results may be affected by: (1) moisture content, (2) rate of loading, (3) condition of both ends of the rock sample, and (4) the presence of inclined fissures, intrusions, and other anomalies that may cause premature failures on the associated planes. These conditions should be noted so that other tests such as triaxial or direct shear tests can be performed, as appropriate. Because the tests are of necessity conducted using intact rock samples, the test results may not be representative of rock mass behavior due to the effects of in-situ discontinuities (e.g., bedding planes and joints), weathering and moisture. For these reasons, unconfined compressive strength testing should be limited to rock types and strata where tests on intact rock are reasonably representative of the rock mass. This is especially a problem in determining the compressive strength of coal through laboratory testing of small-diameter core (e.g., 2 to 3 inches) because the presence of defects in the coal results in unconservative strengths as compared to the strengths determined from tests on larger-diameter core (e.g., 6 to 12 inches) and in-situ tests on field-scale test volumes (Pariseau, 2006). Additional discussion of the limitations of testing of coal is presented in Section 8.4.2.2.

The stress-strain behavior of intact rock samples can be measured during an unconfined-compression test in accordance with ASTM D 7012, "Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures." For this test, specimen deformations are measured using strain gauges applied to the test specimen or linear voltage displacement transducers attached to the top and bottom load platens.

6.5.9.3 Indirect Tensile Strength Test

Rock is relatively weak in tension; thus, the tensile strength To of intact rock is approximately 5 percent of its compressive value (Mayne et al., 2002). Because of the difficulties involved in proper end preparation (Jaeger et al., 2007), the direct tensile strength testing of rock specimens is not a common laboratory procedure. Therefore, the tensile strength of rock is usually determined by indirect methods such as the indirect tensile (Brazilian) test. The indirect tensile strength of intact rock core σ_T is determined in accordance with ASTM D 3967, "Standard Test Method for Splitting Tensile Strength of Intact Rock Core Specimens." Core specimens with length-to-diameter ratios between 2 and 2.5 are placed in a compression loading machine with the load platens placed diametrically across the specimen. The maximum load to fracture the specimen is recorded and used to calculate the indirect tensile strength.

Alternatives to the indirect tensile strength test are the direct tensile strength test and the bending test. The direct tensile strength of intact rock core is determined in accordance with ASTM D 2936, "Standard Test Method for Direct Tensile Strength of Intact Rock Core Specimens." The core specimens for direct tensile testing are prepared as described in ASTM D 4543 for compressive strength testing, but the test is conducted by cementing the specimen ends to the test load apparatus and applying tensile loads on the sample until it fails in tension. While there is no ASTM standard for bending tests, the test specimens need to be long relative to their thickness which makes their preparation difficult and expensive, especially when samples are taken from very jointed rock strata (Pariseau, 2006).

Thus, the indirect tensile strength test is significantly more convenient and practical for routine measurements than the direct tensile strength test and the bending test, and it provides very similar results to those obtained from direct tension tests (Jaeger et al., 2007). In many situations, the indirect tensile test provides a more fundamental measurement of rock material strength than compression tests because the failure mode is more representative of failures that occur (e.g., overstressing of roof strata) in underground mines. It should be noted that the point-load index test discussed in Section 6.5.9.1 is actually a variation of the indirect tensile strength test with results correlated to compressive strength.

6.5.9.4 Rock Durability

Rock used for slope protection and aggregate used for drainage purposes must have high durability to attain suitable long-term performance when exposed to construction and in-service environments.

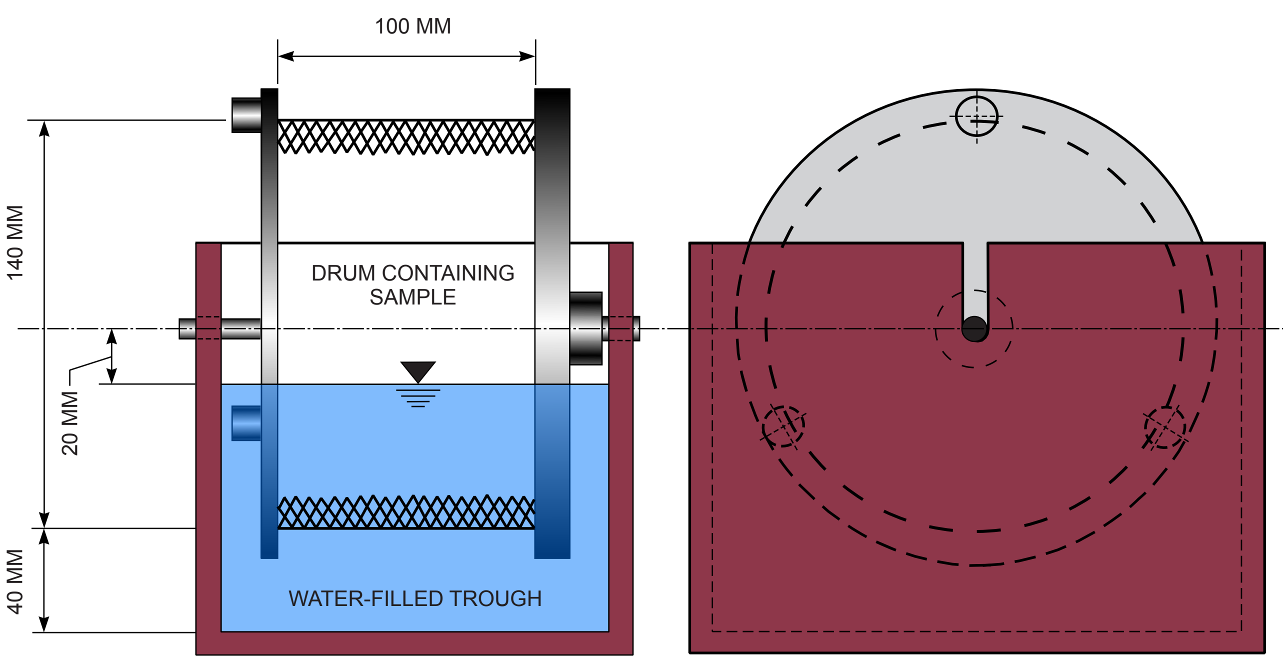
Material deterioration mechanisms include cracking, spalling, delaminating or splitting, disaggregating, dissolving and disintegration (USACE, 1990a). The following ASTM test methods can be used to evaluate the durability of these types of materials:

- D 5240, "Rock Slab Testing For Riprap Soundness, Using Sodium/Magnesium Sulfate"
- D 5312, "Rock-Durability For Erosion Control Under Freezing/Thawing"
- D 5313, "Rock-Durability For Erosion Control Under Wetting/Drying"

Additional rock durability test methods that have been used for evaluation of the slake durability of surface mine spoils and earthen embankments constructed using shale and other slake prone sedimentary rocks are described in Andrews et al. (1980) and Michael and Superfesky (2007). Some of these test methods involve use of acid solutions.

Rock materials used for constructing earthen dam and coal refuse embankments are generally obtained from borrow sources near the construction area. Because coal is found in geologic settings where shales and other weak rocks are encountered as part of mining operations, the durability of these rock materials as compacted fill needs to be evaluated. The most problematic rock types are certain shales, mudstones, claystones and other weak rocks that degrade rapidly soon after they are exposed to atmospheric conditions. These materials can degrade rapidly, affecting the stability of an embankment fill, a rock cut, or foundation on which a refuse embankment is constructed.

The slake durability test was devised in the early 1970s (Franklin and Chandra, 1972) to provide a qualitative measure of these materials in service. As described in ASTM D 4644, "Standard Test Method for Slake Durability of Shales and Similar Weak Rocks," representative rock fragments are subjected to cycles of wetting and drying, and the weight loss is measured after two cycles. Figure 6.46 provides an illustration of the slake durability test apparatus.



(MAYNE ET AL., 2002)

FIGURE 6.46 ROTATING DRUM ASSEMBLY AND SETUP OF SLAKE DURABILITY EQUIPMENT

Ten dried fragments of rock of known weight are placed in a drum fabricated with 2.0-mm-square mesh wire cloth. The drum is rotated in a horizontal position along its longitudinal axis while partially submerged in distilled water to promote wetting of the sample. The specimens and the drum are dried at the end of the rotation cycle (10 minutes at 20 rpm) and weighed. After two cycles of rotating and drying, the weight loss and the shape and size of the remaining rock fragments are recorded, and the slake-durability index (SDI) is calculated. As a qualitative measure of durability, Gamble (1971) proposed the classification system presented in Table 6.43 using the SDI determined after two cycles. Both the SDI and the description of the shape and size of the remaining particles are used to determine the durability of soft rocks. The application of the test for design and construction with mine spoil is presented in Section 8.8.2.

Limestone and calcite cemented sandstone can degrade relatively quickly where acidic leachates are present and should not be used in the construction of filters, underdrains and internal drains. A fizz test of aggregate and rock can be conducted by placing a dilute solution of hydrochloric acid on the material. If an effervescent or fizzing reaction occurs, the material is generally considered to be unacceptable. Laboratory testing may also be performed on aggregate materials to evaluate the calcium-carbonate portion in terms of percent of total content. The presence of sulfide minerals in aggregates can lead to the formation of sulfuric acid and sulfate minerals and should also be avoided. Generally, aggregate materials should contain less than 1 to 5 percent calcium carbonate and much less than 1 percent sulfide materials in order to be acceptable for drain applications.

6.5.10 Structural Material Tests

This section identifies the quality control tests that should be considered in developing plans and specifications for disposal facilities when using portland cement concrete and controlled low-strength material (CLSM).

6.5.10.1 Concrete

Portland-cement concrete is used at coal refuse disposal facilities for construction of drainage control structures and structure foundations. Structural design of reinforced concrete is beyond the scope of this Manual. For additional guidance, the most current version of ACI 318, "Building Code Requirements for Structural Concrete and Commentary," should be used. Table 6.44 presents ASTM test methods for quality control of fresh portland-cement concrete.

6.5.10.2 Controlled Low-Strength Materials (CLSM)

CLSM is used as a replacement for compacted earth fill and consists of various mixtures of pozzolan (e.g., fly ash), portland cement, aggregates (typically fine aggregate such as sand and/or bottom ash), select cohesionless soil, water, and occasionally chemical admixtures (Howard and Hitch, 1998). A low percentage of bentonite or attapulgite can also be included in CLSM for reduced hydraulic conductivity and enhanced plasticity, where such characteristics are important. CLSM has been used for general backfills, shallow cutoff trenches, structural fills, insulating and isolating fills, pavement bases, erosion control, pipe bedding, cradles and backfill, and void filling. It is the latter two applications that have found the greatest application in mining and mine refuse disposal. CLSM is processed and mixed much like fresh concrete and delivered as a fluid in ready-mix trucks or as a dry material in a dump truck. The quality control test methods used for CLSM are adapted from those used for fresh concrete and grout, accounting for the various constituents in the CLSM mix and its lower compressive strength (i.e., 1,200 psi or less). Table 6.45 presents test methods used for quality control of CLSM.

CLSM must be designed in accordance with performance criteria appropriate for the specific application. At coal refuse disposal facilities, CLSM is used primarily for bedding and backfilling of pipe. When pipe is used as part of a decant or other structure that extends through the embankment cross section, the bedding and backfill should have adequate strength to provide support for the pipe and

TABLE 6.43 SLAKE DURABILITY CLASSIFICATION

Durability Classification	Percent Retained after Two 10-Minute Cycles
Very High	> 98
High	95 to 98
Medium High	85 to 95
Medium	60 to 85
Low	30 to 60
Very Low	< 30

(GAMBLE, 1971)

TABLE 6.44 QUALITY CONTROL STANDARDS FOR PORTLAND-CEMENT CONCRETE

Description	ASTM Test Method
Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens	C 39
Standard Test Method for Slump of Hydraulic Cement Concrete	C 143
Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method	C 173
Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method	C 231

low hydraulic conductivity and suitable stiffness and shrinkage characteristics to limit the potential for seepage along the pipe. The bedding should also perform as intended relative to the desired pipe behavior (i.e., flexible versus rigid system). Typical CLSM mix designs have a target maximum compressive strength in the range of 50 to 200 psi for flexible pipe installations and up to 1,200 psi for structural applications. When CLSM is used in flexible pipe installations, a lower strength CLSM is specified in order to produce high quality, soil-like bedding, cradle, and backfill zones to preserve the flexible behavior of the pipe-backfill system while capitalizing on the benefits of CLSM (i.e., ease of placement, better protection of the pipe under construction traffic loadings, improved haunch support and seepage control due to more intimate contact with the pipe, resistance to piping and erosion, and better quality control). Testing of the uniaxial strength of CLSM samples for design of flexible pipe installations should follow ASTM C109 with the addition of strain measurements for determination of the modulus of elasticity in accordance with ASTM D7012. Table 6.46 provides a mix design guide for CLSM. It should be noted that the CLSM strength is also dependent on the cement, ash and mixing water characteristics. Application-specific design and testing is recommended for use of CLSM as pipe backfill, with strengths evaluated at 7, 14 and 28 days (similar to concrete) and also at 56 and 90 days.

TABLE 6.45 QUALITY CONTROL STANDARDS FOR CLSM

Description	ASTM Test Method
Standard Test Method for Preparation and Testing of Controlled Low Strength Material (CLSM) Test Cylinders	D 4832
Standard Practice for Sampling Freshly Mixed Controlled Low-Strength Material	D 5971
Standard Test Method for Unit Weight, Yield, Cement Content, and Air Content (Gravimetric) of Controlled Low Strength Material (CLSM)	D 6023
Standard Test Method for Flow Consistency of Controlled Low Strength Material (CLSM)	D 6103

TABLE 6.46 CLSM MIX DESIGN GUIDE

Properties and Criteria	Type A	Type B	Type C	Type D
Mix Design(1) (per yd^3)				
Cement (lb)	100	50	150 to 200	300 to 700
Pozzolans (lb)	2000	300	300	100 to 400
Bottom ash or coarse aggregate or fine aggregate (lb)	0	2600	2600	(2)
Air entrainment				
Slump (in) – ASTM C 143	7 (min.)	7 (min.)	7 (min.)	7 (min.)
Density (lb/ft^3) – ASTM D 6023	NA(3)	NA	NA	30 to 70 or as specified(4)
Water absorption of aggregate – AASHTO T 85	–	–	–	20% max.
Compressive strength (psi)	125 (max.)	125 (max.)	800 (max.)	90 to 400

Note: 1. Quantities may vary in order to adapt mix to density and strength requirements or to adapt to site conditions and material characteristics.
2. Requires use of suitable light-weight aggregate or air-entraining admixture.
3. Not applicable.
4. Approximate value; use of air-entraining agent may reduce this value.

(ADAPTED FROM PennDOT, 2007)

6.5.11 Verification of Test Results

Unverified or unvalidated test data or engineering properties (e.g., parameters that are assumed from published or other available sources or adopted from other sites without evaluation of their appropriateness) should not be used in the design of refuse disposal facilities. The only exceptions are analyses for facilities with very simple embankments with low hazard potential and for embankments where very conservative slopes are acceptable. Therefore, it is important to verify test data, particularly those for shear strength.

If site access, time and budget were not an issue, the design engineer could obtain as many samples as deemed necessary and conduct as many laboratory or in-situ tests as desired to obtain a complete assessment of subsurface soil and rock conditions. Engineering properties would be quantified, and any inconsistent data would be set aside and additional testing would be initiated, as needed. Unfortunately, site access, schedule and budgets are major factors that designers must consider in making critical decisions throughout the design process to obtain the most reliable and realistic soil, rock and coal refuse property data. As described previously, a critical step in determining these properties lies in the selection of specific tests and proper interpretation of the test results. For a number of reasons (e.g., cost, sampling difficulties or lack or representative values), it may be difficult to determine values for specific parameters of interest. Fortunately, designers can sometimes use established and/or site-specific correlations to obtain values for the desired parameters. Correlations are useful for evaluating test results that do not appear to be representative, and they can also be very useful in preparing test programs, performing preliminary calculations, and serving as a quality assurance check on the test results.

Engineering property correlations come in many forms, but all have a common theme. Specifically, a useful correlation is developed from a large database of results based on past experience. In the best case, the correlation and experience have been developed or "calibrated" using specific site construction materials, or the correlation may be based on reportedly similar construction materials. The reliance upon or use of correlations to obtain soil, rock and coal refuse properties may be justified in the following cases: (1) specific data are simply not available and the only possibility is indirect comparison to other properties, (2) a limited amount of data for the specific property of interest are available and the correlation will complement these limited data, or (3) the validity of certain data is in question and a comparison to previous test results allows the accuracy of the data to be evaluated. It is strongly emphasized that correlations should never be used as a substitute for an adequate site exploration program, but rather should complement and verify available test data. Examples of each of the three cases listed above are provided in the following:

- Specific data are unavailable – Several examples of this type exist. Most notable is the strength of uncemented clean sands. Undisturbed sampling may be problematic and prohibitively expensive, and correlations to SPT, CPT, and other in-situ tests results have been shown to be quite reliable.
- Limited data are available – If few high-quality consolidation tests are performed and compression properties are found to correlate well with the results of Atterberg limits tests, it may be concluded that Atterberg limits tests can be used for assessment of compression properties.
- Data validation – If results from tests on two similar materials are inconsistent, comparing the results to previous data for similar soils may allow determination as to whether the data are simply inconsistent or if some of the data are inaccurate.

There are several sources of correlation data for geotechnical materials and properties. Many geotechnical textbooks and references provide correlations, including Holtz and Kovacs (1981), DOD (2005) and Carter and Bentley (1991). The Electric Power Research Institute (EPRI) commissioned the preparation of useful documents that include several correlations for laboratory and in-situ tests (Kulhawy and Mayne, 1970). For information regarding the engineering properties of coal refuse materials, designers should refer to Hegazy et al. (2004), Chen (1979), DiGioia and Gray (1979), and Coates and Yu (1977).

The following comments on applicability and use of correlations are noted:

- A correlation is only as good as the data upon which it is based.
- There may be some scatter in the correlation data. The effects of data scatter should be accounted for by using upper and lower bound (i.e., best case/worst case) scenarios and factors of safety in the design calculations.
- A correlation will be most accurate if it is calibrated to site soil/material conditions.
- If calibration to site conditions and design-phase laboratory test data cannot be conducted in sufficient detail, consideration should be given to an expanded program of field inspection, performance monitoring and maintenance (Chapter 12) and instrumentation (Chapter 13).

6.6 GEOTECHNICAL ANALYSES

Sections 6.1 through 6.5 have addressed planning and design considerations, site exploration, and material property characterization required for development of a functional, safe and environmentally acceptable coal refuse disposal facility. This section discusses geotechnical analyses associated with refuse disposal facility design. Analytical procedures for seepage, settlement, slope stability, rock excavation, and conduit design are presented. These discussions are necessarily limited in both scope and depth. More extensive treatments of these subjects are available in references cited herein.

6.6.1 Analytical and Numerical Methods in Geotechnical Engineering

Many of the important areas of geotechnical engineering analysis are governed by differential equations for solving problems of elastic equilibrium, consolidation and steady-state seepage.

< PREVIOUS VIEW

Limit analysis using classical techniques such as Rankine's earth pressure theory, Terzaghi's bearing capacity equation, or various methods of slices is another important approach for the solution of geotechnical problems involving the failure of soil masses for slope stability analysis. These "traditional" methods of analysis, as applied to the design of coal refuse disposal facilities, are discussed in the following text. Seismic analysis and design issues for refuse disposal facilities are addressed in Chapter 7.

The availability of powerful personal computers, augmented by improved tools for problem setup, constitutive modeling of material properties, and post-processing of computational results, has facilitated the use of numerical methods to solve complex geotechnical problems. Sophisticated methods, such as the finite element (FE) method, are available to perform complex analyses related to seepage, deformation, and slope stability. The FE method is a numerical method for obtaining solutions to differential equations, given appropriate boundary condition data. Some of the most useful features of the FE method are that it can:

- Simulate one-, two- and three-dimensional problems
- Accommodate complex geometries and construction sequencing
- Model soil property variability (e.g., nonlinear stress-strain behavior and anisotropic hydraulic conductivity)

Nearly all areas of geotechnical analysis can be solved using the FE method. However, the engineer must recognize the inherent uncertainty and variability in material properties. The possible effects of simplifying assumptions for material behavior in FE models must be carefully evaluated. An understanding of empirical relationships and the ability to apply the lessons of experience are key factors in successfully using FE models and interpreting the results obtained.

6.6.2 Seepage Analysis

6.6.2.1 Basic Assumptions, Conditions Requiring Seepage Analysis

All earth and coal refuse structures are to some degree pervious to water and therefore susceptible to seepage. Seepage is a concern in earthen dams and coal refuse embankments for three reasons: (1) the pore-water pressures in an embankment and its foundation affect the stability of the embankment, (2) an excessive hydraulic gradient on an embankment slope, at drain or fine/coarse material interfaces, or at the toe can lead to piping, internal erosion, and destructive uplift pressures, and (3) water lost through or under a structure may require treatment or may be a valuable water source worth recirculating through the preparation plant. Seepage control systems are incorporated into embankments to minimize the potential for excessive or uncontrolled seepage. Practical concepts for seepage control in coal refuse embankments and their foundations are discussed in Section 6.3. This section discusses analytical methods for assessing the need for seepage control and for selecting materials and design dimensions to achieve that control. Additionally, operational controls to mitigate seepage, such as the deposition of slurry within impoundments, are also discussed.

Under steady-state conditions, seepage through an earth or coal refuse embankment and its foundation can be estimated from the relationship for flow through porous media, which is based on the following assumptions:

- The flow occurs through incompressible media.
- The flow is caused by gravity forces only.
- The media through which flow occurs is always saturated.
- The boundary conditions of the flow are known.

For unsaturated flow and transient seepage analyses, additional assumptions may be required.

Whether or not the boundary conditions are actually known depends on the specific flow condition being analyzed. For flow through a pervious foundation under a relatively impervious embankment, the boundary conditions are readily apparent. However, not all boundary conditions are easily determined, and many boundary conditions must be approximated or determined by iterative analysis procedures (e.g., the seepage and phreatic condition in an embankment with internal drains).

Steady-state flow through porous media such as soils or coal refuse is normally laminar and therefore follows the Darcy equation:

$$Q = k\,i\,a \qquad (6\text{-}15)$$

where:

Q = flow rate (volume/time)

k = coefficient of hydraulic conductivity (length/time)

i = hydraulic gradient (dimensionless)

a = cross-sectional area through which flow occurs (area)

Selection of the coefficient of hydraulic conductivity should be based on field and laboratory testing, although evaluation of other factors may also be required, including:

- Published information (e.g., soil surveys) along with the results of site-specific index and classification tests for natural soils and coal refuse.
- Potential variability of existing fill or mine spoil, especially when there are no available data related to in-situ material or placement.
- Potential for anisotropic conditions (if not documented based upon site-specific data), particularly for coal refuse embankments where the horizontal hydraulic conductivity may be an order of magnitude greater than the vertical hydraulic conductivity.
- Sensitivity of flow rate and hydraulic gradient (and phreatic surface) to variation of assumed parameters.
- Observed site groundwater levels in natural soils and existing embankments (to assist in validating assumed values, particularly if multiple zones or anisotropic conditions are present).

Seepage analysis for coal refuse embankments should be based on conservative estimates of hydraulic conductivity, with values selected based on available test data (preferably field tests), material classifications and anisotropy ratios resulting in conservative (higher) phreatic levels and seepage rates. The ratio of horizontal to vertical hydraulic conductivity (anisotropy ratio) can vary due to the material type, placement and compaction in lifts, and for embankment dams has been reported to range from 1 to 100 (Fell et al., 2005). The U.S. Army Corps of Engineers (USACE, 1993) suggests an anisotropy ratio between 2 and 10 or greater for embankment dams constructed in compacted lifts, and MSHA (2007) recommends a ratio of at least 9 for coarse coal refuse or as based upon site-specific conditions and materials. For an existing embankment, back calculation of monitored pool and piezometer levels to estimate anisotropy is recommended.

If fine coal refuse deposits are accounted for in the seepage analysis, an anisotropy ratio of between 1 and 10 is usually applied. In typical situations where coarse coal refuse is more permeable than the fine refuse, the anisotropy ratio of the fine refuse deposit may have little effect on the computed phreatic levels and seepage rates in downstream coarse refuse embankments. It is likely that the anisotropy ratio of hydraulically-placed fine refuse is quite variable (beyond the range cited above) such that design of seepage control and collection measures critical for stability of the facility should be effective over the likely range of anisotropy.

For impounding coal refuse embankments, seepage analyses should be performed for critical intermediate stages along with the final stage of development. The pool level used in the seepage analysis should be the maximum normal pool level for the stage (usually based on the decant invert level) or, if there is potential for saturation of the embankment due to retention of the design storm, the maximum design storm pool level.

Unsteady-state seepage analysis can be employed for evaluation of the extent of saturation of an embankment or to assess the impact of rapid drawdown on the upstream embankment slope. Also, unsaturated flow analysis is sometimes coupled with saturated flow analysis (generally using numerical models) to evaluate internal drainage systems, particularly where there is significant anisotropy.

Turbulent flow regimes may occur in rockfill and rock drains used in embankments. Based on earlier work by Cedergren (1989), the U.S. Army Corps of Engineers (1993) provides guidance for estimating the reduction in hydraulic conductivity of narrow size range aggregate caused by turbulent flow at large hydraulic gradients in underdrains (Figure 6.47). As indicated in Cedergren (1989), the reduction in hydraulic conductivity is of relatively little importance for flat-lying underdrains if hydraulic gradients are less than 0.02. Furthermore, the effect of increasing the hydraulic gradient 100 times (from 0.01 to 1.0) reduces the hydraulic conductivity to one-tenth of the laminar value. Alternatively, Leps (1973) developed the following empirical relationship for turbulent flow through rock which is sometimes applied:

$$Q = a\,W m^{0.5}\, i^{0.54}\, [e / (1 + e)] \qquad (6\text{-}16)$$

where:

- Q = flow rate (length^3/time)
- a = flow cross section area (length^2)
- W = empirical constant for rockfill material, dependent on the shape and roughness of rock particles and the viscosity of water (length-time units) (Wilkins, 1956)
- m = hydraulic mean radius (length)
- i = hydraulic gradient (dimensionless)
- e = void ratio (dimensionless)

Leps (1973) provides the following guidance on the determination of $Wm^{0.5}$ based on rock size:

Rock size (in)	3/4	2	6	8	24	48
$Wm^{0.5}$ (in/sec)	10	16	28	32	58	84

While Equation 6-16 applies to uniformly-sized rock, Leps suggests that it can be adapted for graded materials by using the 50-percent rock size to compute $Wm^{0.5}$, provided that the minus 1-inch material is less than 30 percent, and preferably less than 10 percent by weight, of the rockfill. If the percentage of minus 1-inch material is greater than 30, Equation 6-16 may not be applicable.

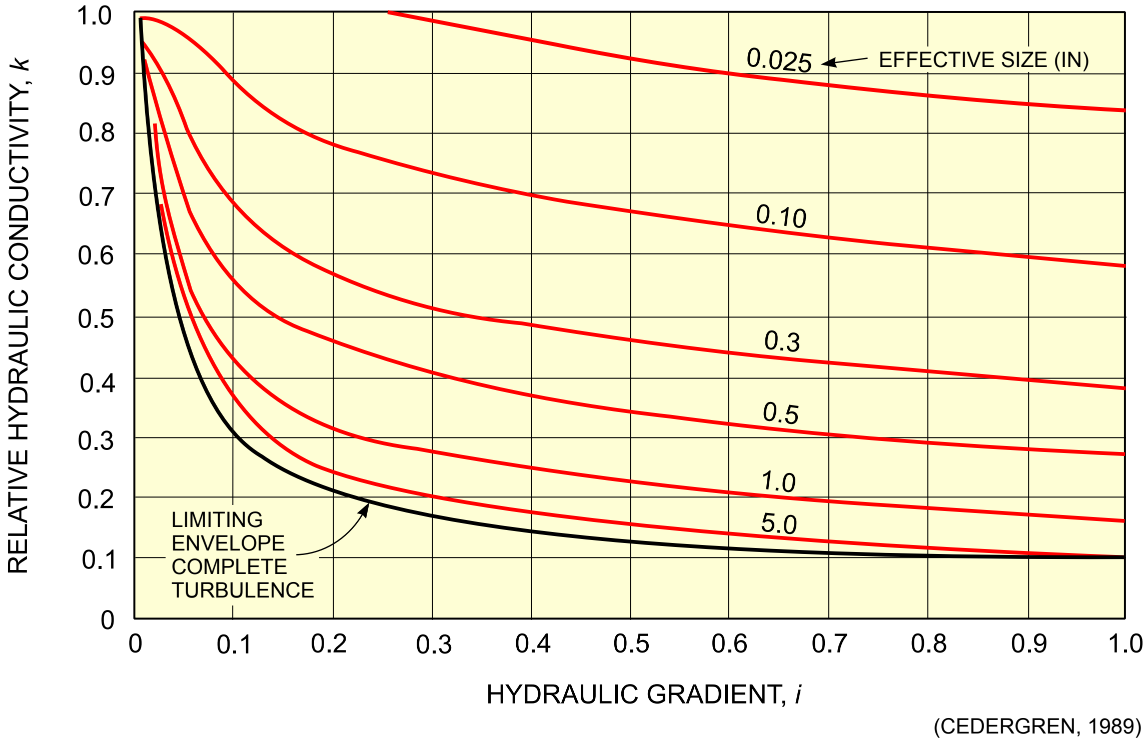


FIGURE 6.47 APPROXIMATION FOR ESTIMATED REDUCTION IN HYDRAULIC CONDUCTIVITY UNDER TRUBULENT FLOW

6.6.2.2 Seepage Analysis Methods

There are several approaches for conducting seepage analyses. Prior to the widespread availability of computers, graphical hand-solutions (i.e., hand-drawn flow nets) and hand calculations were commonly used for modeling embankment and foundation seepage. However, the use of numerical methods (primarily finite element modeling) has become the most common method for seepage analyses. These approaches are discussed in the following sections.

Seepage in homogeneous coal refuse embankments can be readily analyzed using graphical and analytical methods, provided that the hydraulic conductivity and any anisotropy are known. Zoned embankments, variable foundation conditions, and the presence of the settled fine coal refuse introduce additional complexity, and numerical methods are better suited for detailed analysis of these situations. Coal refuse embankments are developed in stages, each with a specific configuration and internal drainage system designed on the basis of the projected fine coal refuse level and pool level. Thus, in addition to the final impounding stage, intermediate stages will likely need to be analyzed as part of the design of internal drainage structures and to establish the stage configuration in conjunction with the stability analysis.

With either the graphical, analytical or numerical modeling approach, it is essential that the boundary conditions for the analysis be accurately defined. These boundary conditions typically include:

- Appropriate reservoir/impoundment pool level
- Foundation conditions, including possible artesian conditions
- Location(s) and capacity(s) of internal drains
- Geometry of intermediate and post-construction cross sections
- Maximum and minimum potential inflow rate(s) to the reservoir/impoundment

Selection of the appropriate elevations for water and fine coal refuse in an impoundment are important if meaningful results are to be realized. Typically, normal pool or the maximum decant inlet level for a specific stage is selected. If the reservoir pool is above the settled fine coal refuse level and against the upstream slope of the impoundment, the effect on the phreatic surface elevation of the fine coal refuse will be limited, even if it has low hydraulic conductivity relative to the coarse refuse. If the impoundment is designed to retain a design storm that would cause saturation of the embankment due to an elevated reservoir level, then seepage associated with that reservoir level should be analyzed. Cedergren (1989) provides a method for estimating the approximate time for embankment saturation to occur for simple embankment geometry and homogeneous hydraulic conductivity. Unsteady seepage analyses can be performed for complicated geometries and zoned embankments, although it is recommended that the designer exercise caution and consider redundant seepage control measures in critical situations.

Site conditions may necessitate the evaluation of other features and boundary conditions. As one of the primary purposes of seepage analyses is to determine seepage conditions for use in stability analyses, the locations of the seepage cross sections should be consistent with the locations of stability sections (for 2D analyses). Also, the size, lateral extent and continuity of drainage features should be evaluated when deciding whether and how to incorporate them into the analysis. For example, a blanket drain that is limited in lateral extent to the center of a valley may not significantly affect the phreatic level at the abutments of an embankment. In such a case, it may be necessary to also analyze the influence of a higher saturation level in cross sections close to the abutments.

6.6.2.2.1 Graphical (Flow Net) Approach

A flow net is a set of orthogonal lines graphically representing the seepage conditions in an embankment. The seepage at any cross section of an earth or coal refuse embankment can be determined by constructing a flow net using free-hand trial-and-error sketching. One group of flow net lines represents paths of flowing water, and the other group represents lines of equal head (equipotential lines). The flow net must satisfy the flow boundary conditions. The flow lines and equipotential lines must intersect at right angles, and for each element the mean dimension parallel to the flow lines must equal the mean dimension parallel to the equipotential lines (or remain in the same proportion over the extent of the flow net). These dimensional requirements can be checked by drawing a circle in elements and determining if it is tangent to the mid-sides of the adjacent flow and equipotential lines. Figure 6.48 shows a typical flow net analysis.

To estimate the rate of seepage flow from a two-dimensional flow net of unit thickness, the Darcy equation can be written in the following form:

$$q = k\,h\,(N_f/N_d) \tag{6-17}$$

where:

- q = flow rate per unit width (length^2/time)
- k = coefficient of hydraulic conductivity (length/time)
- h = total head loss across the system (length)
- N_f = number of flowpaths through the system (dimensionless)
- N_d = number of potential drop (divisions of head loss) across the system (dimensionless)

Examples of the application of this equation are provided in numerous references, including Cedergren (1989), Harr (1962) and USACE (1993).

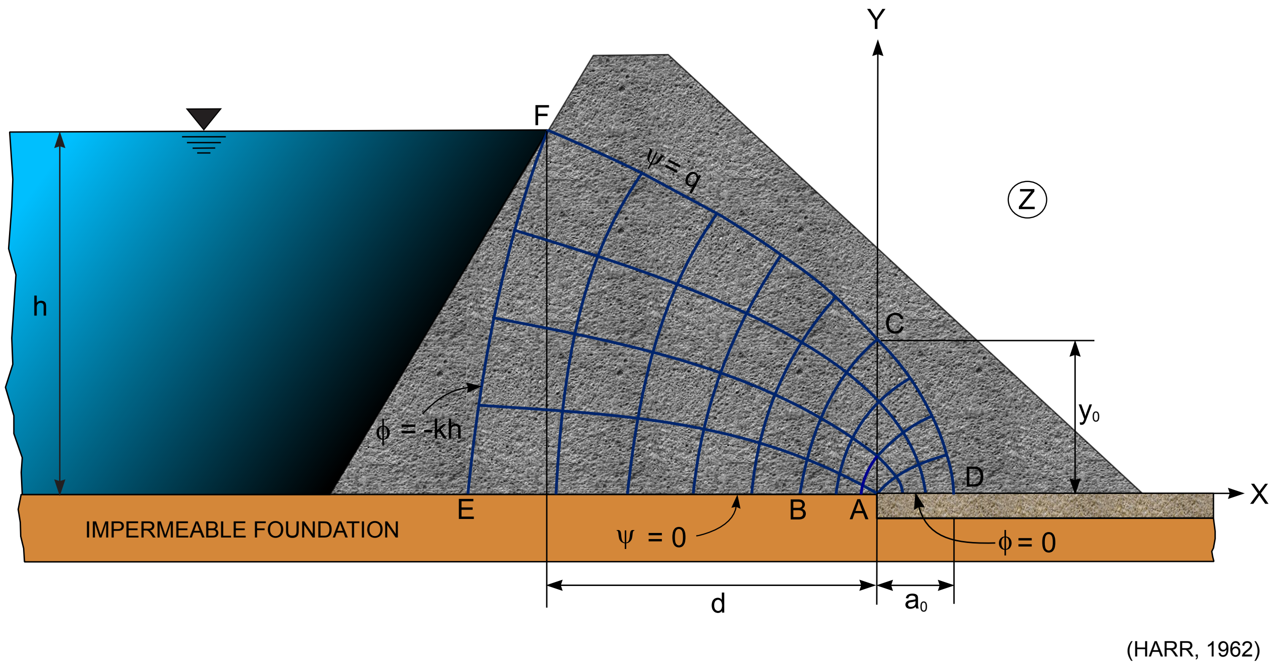


FIGURE 6.48 FLOW NET ANALYSIS

An example of the flow net graphical approach is illustrated in Figure 6.49a for a homogeneous dam section. Figure 6.49b illustrates the graphical transformations necessary to develop flow nets for a homogeneous dam embankment with anisotropic hydraulic conductivity. When a dam is composed of a number of different soils with varying anisotropic hydraulic conductivity, the graphical approach becomes considerably more complicated. In practice, the use of graphical methods may require that a number of simplifying assumptions be made to render the problem manageable. The construction of flow nets and their application are described in USDA (1973a) and Reddi (2003).

6.6.2.2.2 Analytical Solutions

The seepage and associated phreatic surface for an embankment dam with an underdrain resting on an impervious base, generally referred to as Kozeny's basic parabola, can be determined analytically (Harr, 1962). The phreatic surface and minimum width of drain a_0 (Figure 6.48) can be estimated from the following equations:

$$x = (-ky^2/2q) + (q/2k) \qquad y_0 = (d^2 - h^2)^{0.5} - d \tag{6-18}$$

$$q = k\left[(d^2 + h^2)^{0.5} - d\right] \tag{6-19}$$

where:

k = coefficient of hydraulic conductivity (length/time)

q = seepage rate per unit length of drain (length^2/time)

Also, the d, h, x, and y dimensions (length) are as shown on Figure 6.48 and $a_0 = y_0/2$.

Harr (1962) provides analytical solutions for embankments with foundations at great depth (as well as finite depth) and with variable upstream slopes. These procedures allow for treatment of anisotropy using transformed sections (simple expansion or contraction of spatial coordinates to convert an anisotropic zone to an isotropic zone).

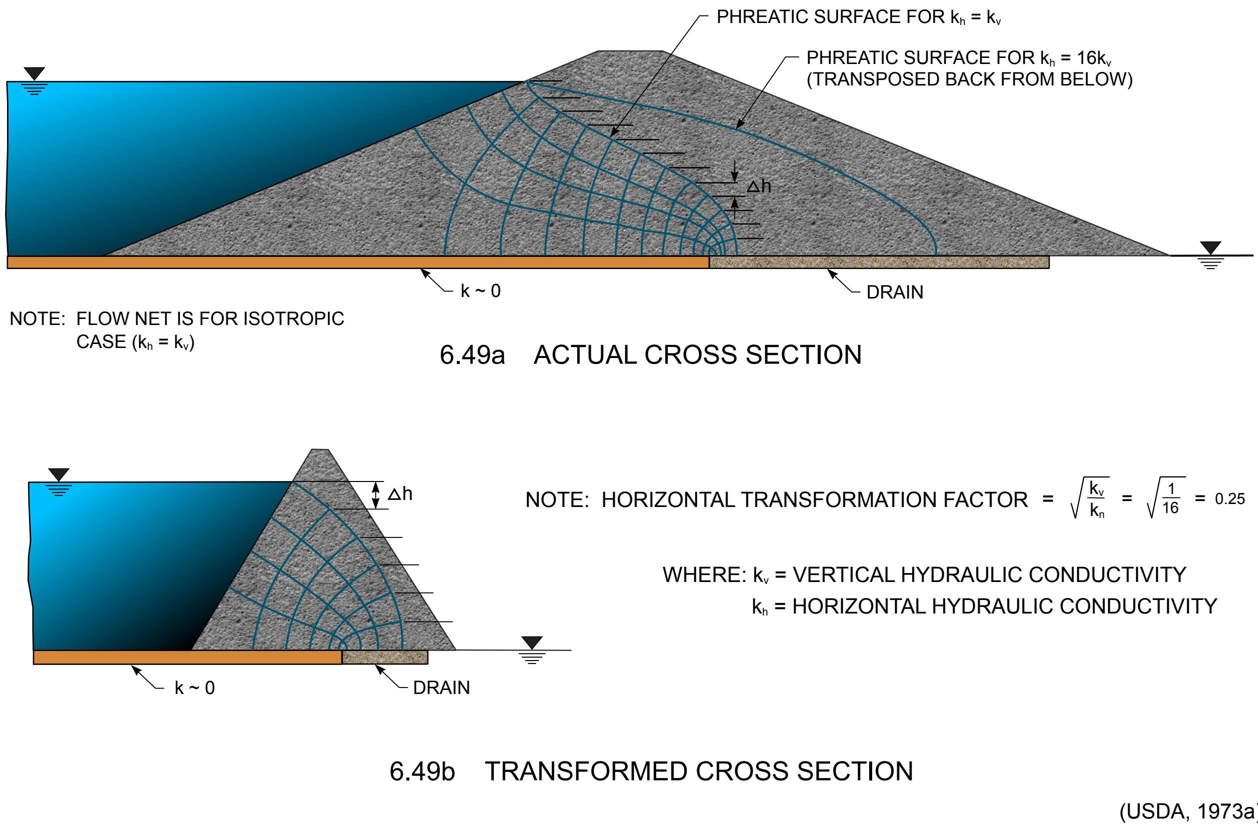


FIGURE 6.49 FLOW NET ANALYSIS FOR ANISOTROPIC CONDITIONS

Kashef (1986) and Mishra and Singh (2005) point out simplifying assumptions for Kozeny's basic parabola and their impact on seepage rate and drain length. The results are compared with numerical methods for zoned embankments.

Coal refuse embankments frequently employ limited width and height internal drains positioned above the base of the dam to intercept seepage and control the phreatic surface. Van Zyl and Robertson (1980) have analyzed a drain of limited width on an impervious base. They estimate the effective width of the drain as approximately $0.1h$ for the condition where $d/h > 2$, provided the drain filter has sufficient capacity. They also recommend that the drain width be sized 50 percent larger than the width determined based on the capacity of the filter, with a minimum dimension of 2 meters.

Analytical solutions provide a means to estimate seepage and phreatic levels for some specific boundary conditions. They can be useful for initial development of flow nets and for interpreting results obtained with numerical modeling, as subsequently discussed.

6.6.2.2.3 Numerical Modeling Approach

As noted previously, the advent of powerful computers has facilitated the use of numerical modeling for evaluating seepage. There are a number of finite element programs that can be used to model seepage for two- and three-dimensional, anisotropic, unconfined and confined flow. The use of such computer programs has several advantages over the use of graphical methods to analyze seepage. Frequently, computer programs support the use of hydraulic conductivity functions (relationships between hydraulic conductivity and pore-water pressure) and thus allow the inclusion of unsaturated flow into the seepage model. Computer programs can more easily accommodate anisotropic hydraulic conductivities and complicated soil/rock geometries, including varying anisotropy ratios. Transient analyses are also possible using numerical methods if water content data for each material in the model are available.

Computer programs utilizing numerical methods frequently provide several alternatives for modeling boundary conditions, including the ability to model internal drainage systems of limited width and height located above the base of the embankment. However, not all of these alternatives will result in a realistic model. Therefore considerable judgment must be used if realistic results are to be achieved. As noted previously, accurate geometry and realistic representation of boundary conditions are vitally important when setting up a seepage model. The resulting model should be checked using a more simple approach (e.g., flow nets) to determine whether the resulting flow paths and related directional orientation of velocity vectors, predicted total head, and predicted pressure heads are realistic.

The application of numerical methods for seepage analyses at fine coal refuse impoundments has been presented in publications by Snow et al. (2000) and Thacker (2000). These studies present models for steady-state flow conditions with anisotropic hydraulic conductivity for various material property zones, and they use specified heads in the embankment or as boundary conditions to simulate impoundment levels or drains. For numerical models of this type, flow vectors and hydraulic gradients should be checked to verify that values are reasonable and that mass balance is maintained.

6.6.2.3 Seepage Control Measures

6.6.2.3.1 Granular Drain and Filter Requirements

Drains and filters made of aggregate materials are used within and under coal refuse embankments to collect seepage and to lower pore-water pressures in parts of embankment where internal erosion or unstable conditions might otherwise develop. The use of drains and filters in coal refuse embankments is also discussed in Section 6.3. Technical requirements for drains are discussed in the following pages.

For a drainage system to be effective, the following criteria must be satisfied:

- The hydraulic conductivity must increase in the direction of flow.
- The particles in the system must be stable against the seepage forces.
- Material segregation during construction must be prevented so that grain-size characteristics are uniform throughout the drain.
- The materials must not be susceptible to clogging over time.
- The materials must be durable and not decompose over time.

Filter criteria must be satisfied for transitions between a drain and adjacent portions of an embankment and between zones of finer and coarser materials within the embankment. As water percolates through soil or refuse materials, seepage forces in the direction of flow are exerted on the soil/material particles. Where water flows from a fine soil into a coarse soil, it may transport fine particles into the voids in the coarser material. This erosion of the fine material can lead to piping, a phenomenon where a path of much higher hydraulic conductivity that could ultimately cause a failure is developed within an embankment. When the difference in grain-size distribution for adjacent embankment zones is too great to prevent piping, a filter or layer of material of intermediate particle size needs to be placed between the two zones. When adjacent embankment and drainage system zones differ greatly in particle size, two or more filter zones with graduated particle sizes may be needed.

The U.S. Army Corps of Engineers (USACE, 1993) guidance for selecting materials for successive layers of a filter and drainage system, stated as rules, is provided in the following:

To prevent particles of a fine soil from washing into an adjacent coarser soil:

Rule 1: The criteria presented in Table 6.47 should be followed.

Rule 2: Ideally, grain-size curves for the base and filter soils should be approximately parallel. For gap-graded base materials (sometimes the case with coarse coal refuse), filter criteria that exclude the coarser portion of the base soil should be applied. Also adjustments can be made to exclude particles larger than the No. 4 (4.75-mm) sieve. The filter should be designed to protect the fine matrix. For gap-graded and broadly-graded materials, considerable care and judgment must be employed in identifying the grain-size portion of the base soil to be filter-protected so that sufficient capacity to transmit seepage flow from the base soil is provided (USBR, 2007a). Alternatively, tests can be conducted in the laboratory to select the appropriate filter (Sherard and Dunnigan, 1985).

For hydraulic conductivity consistent with adequate discharge capacity:

Rule 3: $$\frac{D_{15\ filter\ soil}}{D_{15\ base\ soil}} \geq 3 \text{ to } 5 \qquad (6\text{-}20)$$

Rule 4: The portion passing the No. 200 sieve should be less than or equal to 5 percent, and the fines should be cohesionless.

To prevent segregation during placement:

Rule 5: The restrictions on filter grain size presented in Table 6.48 should be followed.

Rule 6: $D_{max} \leq 3$ inches.

The USACE (1993) provides guidance for filters within gap-graded and broadly-graded soils. The latter condition may be particularly relevant because the grain-size distribution associated with coarse coal refuse embankments is frequently either gap-graded or broadly-graded. Additionally, the above guidelines (Rule 6) may require adjustment in some project-specific situations where large mine rock overburden is used in downstream zones.

Where a perforated drainpipe is the final element of a drainage system, a well-graded gravel that will not wash through the perforations or slots should be placed around the pipe. The D_{85} size of the gravel should be greater than the width of the perforations or slots. Experience has shown that drainpipes placed with perforations directed downward are less likely to clog. Geotextiles should not be placed directly against a perforated or slotted pipe due to the potential for clogging. Instead, a zone of sand or aggregate should be placed around the pipe and then the filter (geotextile or another granular layer) should be placed around the aggregate (France, 2004)

The discharge capacity of all portions of a drainage system must be checked for adequacy, particularly for thin horizontal drains. This can be accomplished by very simple approximate calculations using the Darcy equation presented earlier in this section. Cedergren (1989) contains several useful examples of this calculation.

The NRCS (1994) and the USBR (2007a) also provide guidance on filter criteria, including the use of perforated pipes in drains. These criteria may be applied to coal refuse disposal facilities. McCook

TABLE 6.47 FILTER CRITERIA AS A FUNCTION OF BASE SOIL AND PERCENT PASSING NO. 200 SIEVE

Base Soil Category	Base Soil Description	Percent Finer Than No. 200 (0.075-mm) Sieve[1]	Filter Criteria[2]
1	Fine silts and clays	> 85	$D_{15\ filter\ soil} \leq 9\ D_{85\ \text{base soil}}$ $D_{15\ filter\ soil} \geq 0.2$ mm
2	Sands, silts, clays, and silty to clayey sands	40 to 85	$D_{15\ filter\ soil} \leq 0.7$ mm
3	Silty and clayey sands and gravels	15 to 39	$D_{15\ filter\ soil} \leq 0.7\ \text{mm} + [4(D_{85\ base\ soil}) - 0.7\text{mm}] \times [(40 - A)/(40 - 15)]$[3,4]
4	Sands and Gravels	< 15	$D_{15\ filter\ soil} \leq 4$ to $5\ D_{85\ base\ soil}$[5]

Note:
1. Category designation for soil containing particles larger than 4.75 mm is determined from a gradation curve of the base soil that has been adjusted to 100 percent passing the No. 4 (4.75-mm) sieve.
2. Filters should have a maximum particle size of 3 inches (75 mm) and a maximum of 5 percent passing the No. 200 (0.075-mm) sieve with a PI = 0. To provide sufficient hydraulic conductivity, filters must have $D_{15\ filter\ soil} \geq 4\ D_{15\ base\ soil}$, but not less than 0.1 mm.
3. A is the percent passing the No. 200 sieve using the adjusted grain-size curve.
4. When 4 $D_{85\ base\ soil} < 0.7$ mm, use $D_{15\ filter\ soil} = 0.7$ mm.
5. In Category 4, the $D_{15\ filter\ soil} \leq 4\ D_{85\ base\ soil}$ criterion should be used for filters beneath riprap subject to wave action and for drains that may be subjected to violent surging and vibration.

(USACE, 1993)

(2006) presents an overview of design and construction issues for pipes in drain systems, comparing criteria published by the USACE, NRCS and USBR and methods for estimating inflow capacity. Slotted pipe generally is less prone to clogging and has a much larger inflow capacity than circular perforated pipe. However, perforated pipe probably has higher strength than slotted or screened pipe, especially when the perforations are relatively small in diameter and widely spaced. Care should be exercised in selecting pipe materials, recognizing that: (1) slotted single-wall corrugated tubing has been vulnerable to crushing in some drain installations and (2) single-wall corrugated pipe is seldom used in the design of significant- and high-hazard-potential dams. Pipes in drains should generally be designed to withstand the maximum height of backfill over the pipe, and they should be protected against damage during construction. Care should be exercised in evaluating the load-carrying capacity of perforated pipe, as the perforations can affect the strength of the pipe. Since the outer rings of

TABLE 6.48 RESTRICTIONS ON FILTER PARTICLE SIZE TO LIMIT SEGREGATION

Minimum $D_{10\ \text{filter}}$ (mm)	Maximum $D_{90\ \text{filter}}$ (mm)
< 0.5	20
0.5 to 1.0	25
1.0 to 2.0	30
2.0 to 5.0	40
5.0 to 10	50
10 to 50	60

(USACE, 1993)

corrugated pipe carry the majority of the load, the effect of perforations on the inner rings is thought to be negligible (< 1 percent). Double-wall, corrugated and slotted HDPE and PVC pipe is available in a variety of diameters.

It should also be noted that all materials used in drainage systems for a coal refuse embankment should be resistant to corrosion from exposure to seepage leachates. Guidelines for selecting such materials are presented in Chapter 11.

Drain Capacity and Grade

Drains should be sized with sufficient dimensions and grade (typical minimum slope of 1 percent) to convey the estimated drain demands based on the seepage analyses multiplied by a safety factor. It is recommended that drains have the capacity to convey 10 times the estimated drain demand (Cedergren, 1989). It is also recommended that drains installed in trenches should include a minimum width determined on the basis of capacity requirements and constructability considerations (typically 3 feet) of the drain core material surrounded by specified minimum thicknesses for the surrounding filter aggregate(s). For drains at the ground surface or the coal refuse working surface, the minimum width of the core material should be determined on the basis of constructability. Additional discussion of drain and filter dimensioning is available from the USBR (2007a).

Detailed review of the hydraulic gradients in the vicinity of a drain may lead to adjustments in drain dimensions. McCook (2002) provides a discussion of critical hydraulic gradients and notes that cohesive soils such as clays will sustain significant gradients when confined. Indraratna and Radampola (2002) provide further guidance relative to critical gradients at drain filters.

Drain Configurations

The above criteria for drain capacity and grade apply to aggregate drains of various configurations, including trench drains, finger drains, blanket drains, chimney drains and discrete longitudinal drains. In determining the thickness of drainage layers, the layer inclination and the method of placement in addition to hydraulic conductivity requirements must be considered. Drainage layers placed by machine should generally have a thickness of at least 18 inches. Layers placed very carefully by hand in confined spaces or machine-placed layers for which there is construction oversight and QC compliance monitoring should have a minimum thickness of 12 inches. If adjacent materials could migrate into drainage layers during placement, the thicknesses should be increased. Where machine placement is used for constructing steeply inclined drainage layers within an embankment, each layer should be sufficiently thick to permit efficient operation of the construction equipment. Filters and drainage structures that are so thin that a small amount of contamination during construction would reduce the effective size to below design requirements are generally considered to be inadequate.

Another factor that may impact the thickness of drainage layers is the source and grain size of the granular material. Granular drains should be compatible with the selected filter material. The minimum thickness of a well-graded rock drain must be more than the maximum size of the rock, and gradation requirements should be strictly monitored so that there is no concentration of large rock. If uniformly-graded rock is used for a drain, then an increase in the minimum thickness should be considered. A minimum thickness equal to twice the maximum rock size should provide predictable flow capacity.

6.6.2.3.2 Geotextile-Wrapped Drains

At some coal refuse embankments, geosynthetic materials have been used to separate granular drain material from soils or coal refuse as a replacement for or supplement to granular filters. In addition to some state dam safety agencies, the Nation Dam Safety Review Board currently recommends that geotextiles not be used as filters in locations where they would be critical to the safety of an embankment dam, citing concerns about the long-term capability of the geotextile to function without deterioration or clogging. Critical drain applications and areas of concern include internal drains for controlling the phreatic surface for embankment stability and drains and filters that are designed to minimize the potential for piping of susceptible soils. FEMA, in a publication planned for 2009, is addressing the use of geotextiles in embankment dams.

MSHA has generally permitted the use of geotextiles as filters in slurry impoundments provided that testing using site-specific materials demonstrates acceptable behavior with respect to clogging and there is sufficient instrumentation to monitor the phreatic level. In addition to monitoring of the phreatic level, the seepage quantity from geotextile-wrapped drains should be monitored as an additional indicator of how the geotextile is performing. In recognition of the potentially more critical seepage conditions that can exist in other dams with significant hydraulic head and narrow cross section (e.g., fresh water dams), and until the use of geotextiles is accepted in this application, MSHA advocates that granular filters be used in such significant- and high-hazard-potential dams where filters are critical to safety.

As discussed in Section 6.5.5, a geosynthetic is a planar polymeric material used with soil or rock as an integral part of a civil engineering project, structure, or system. Geotextiles are a subcategory of geosynthetics and are made from woven or nonwoven fabric that allows the passage of water. At refuse disposal facilities, geotextiles have often been used as filters in internal drains. The geotextiles restrict movement of soil particles as water flows into the drain. Typically, non-woven geotextiles are used for filtration purposes. However, woven monofilament geotextiles have performed well in filter applications, and knit geotextiles have been used around perforated pipes as part of a two-stage filter in combination with a primary sand filter layer.

AASHTO M288, "Standard Specification for Geotextile Specification for Highway Applications," (AASHTO, 2008) provides reference information concerning material properties, applications related to highway use (including subsurface drainage) and construction guidance for the use of geotextiles in drainage applications. FHWA Publication No. HI-95-038, "Geosynthetic Design and Construction Guidelines," (Holtz et al., 1998) is a comprehensive reference providing information on the retention criterion, hydraulic conductivity criterion, clogging resistance criterion and survivability criterion, as discussed in subsequent paragraphs.

Geotextiles, like graded granular filters, require engineering design if they are to perform as desired. Unless flow requirements, piping and clogging resistance, and constructability requirements are accurately specified, the geotextile/soil filtration system may not perform properly. Also, as with graded granular filters, construction using geotextiles must be monitored to verify that installation is performed correctly and that the geotextiles are not damaged during installation.

Design of geotextile filters is comparable to design of graded granular filters. A geotextile is similar to a soil in that it has openings (voids) and filaments and fibers (particles). However, the geometric relationship between filaments and openings is more complex than the relationship between particles and voids. In geotextiles, opening size can be measured directly, whereas for soils pore size is a function of particle size. Because pore size can be directly measured, relatively simple relationships have been developed between the pore sizes and particle sizes of the soil to be retained. Three filtration concepts are used in the design process:

1. If the size of the largest opening in the geotextile filter is smaller than the larger particles of embankment material, the filter will retain the embankment material.
2. If the smaller openings in the geotextile are sufficiently large enough to allow smaller particles of embankment material to pass through, then the geotextile will not clog.
3. The number of openings in the geotextile should be such that adequate flow can be maintained even if some of the openings become clogged.

The above simple concepts and analogies to soil filter design criteria have been used to establish design criteria for geotextiles (Holtz et al., 1998). Specifically, these criteria require that geotextiles must:

- Be capable of retaining soil or other embankment material (retention criterion)
- Allow water to pass (hydraulic conductivity criterion)
- Be functional throughout the life of the structure (clogging resistance criterion)
- Survive the installation process (survivability criterion)

Retention Criterion

For steady-state flow conditions:

$$AOS \le B\,D_{85\ soil} \tag{6-21}$$

$$AOS \approx O_{95} \tag{6-22}$$

where:

AOS	=	apparent opening size (length)
O_{95}	=	opening size in the geotextile for which 95 percent are smaller (length)
B	=	coefficient (dimensionless)
D_{85}	=	soil particle size for which 85 percent are smaller (length)

B ranges from 0.5 to 2 and is a function of the type of soil to be filtered, its density, the uniformity coefficient C_u (for granular soils), the type of geotextile (woven or nonwoven), and the flow conditions. For sands, gravelly sands, silty sands, and clayey sands (with < 50 percent passing the No. 200 sieve), B is a function of C_u as defined below:

$C_u \le 2$ or ≥ 8:	$B = 1$
$2 \le C_u \le 4$:	$B = 0.5\ C_u$
$4 < C_u < 8$:	$B = 8\ C_u$

where:

$$C_u = D_{60}/D_{10} \tag{6-23}$$

For silts and clays, B is a function of the type of geotextile:

Woven	$B = 1$	$O_{95} \le D_{85}$
Nonwoven	$B = 1.8$	$O_{95} \le D_{85}$
For both	AOS or $O_{95} \le 0.3$ mm	

Due to their random pore characteristics and, for some types, their “felt-like” nature, nonwoven geotextiles will generally retain finer particles than a woven geotextile of the same apparent opening size. Therefore, a value of B = 1 is more conservative for nonwoven geotextiles than it is for woven geotextiles.

Hydraulic Conductivity Criterion

For non-critical applications (less severe conditions):

$$k_{geotextile} \geq k_{retained\ material} \tag{6-24}$$

For critical applications (severe conditions):

$$k_{geotextile} \geq 10\ k_{retained\ material} \tag{6-25}$$

Guidelines for determining critical nature or severity for drainage applications are provided in Table 6.49. Geotextile permittivity ψ can be defined as:

$$\psi = k/t \tag{6-26}$$

where:

k = Darcy coefficient of hydraulic conductivity (length/time)

t = geotextile fabric thickness (length)

Geotextile permittivity should meet the following requirements:

$\psi \geq$ 0.5/sec for < 15 percent passing No. 200 sieve

$\psi \geq$ 0.2/sec for 15 to 50 percent passing No. 200 sieve

$\psi \geq$ 0.1/sec for > 50 percent passing No. 200 sieve

Clogging Resistance Criterion

For actual flow capacity, the hydraulic conductivity criterion for non-critical applications is conservative because an equal quantity of flow through a relatively thin geotextile takes significantly less time than flow through a thick granular filter. Where flow reduction is judged not to be a problem, such as in clean, medium to coarse sands and gravels, Equation 6-24 may be used. Even so, some pores in the geotextile may become blocked or plugged with time. Therefore, for critical or severe applications, Equation 6-25 should be used to provide an additional level of conservatism. FEMA, in a document to be published in 2009, suggests even greater conservatism with $k_{geotextile}$ = 10 to 100 $k_{retained\ material}$ and indicates that the French (Degoutte and Fry, 2002) use 100 $k_{retained\ material}$ for dams.

The required flow rate through the system q should also be determined, and the geotextile and drainage aggregate should be selected to provide adequate capacity. As indicated previously, flow capacities should not be a problem for most applications. In some situations, however, such as where geotextiles are used in multiple stage filters around a perforated or slotted pipe (pipe wraps), portions of the geotextile may not function effectively. For these applications, the following criteria should be used together with the hydraulic conductivity criteria:

$$q_{required} = q_{geotextile}\ (A_g/A_t) \tag{6-27}$$

where:

A_g = geotextile area available for flow (length^2)

A_t = total geotextile area (length^2)

TABLE 6.49 GUIDELINES FOR EVALUATING CRITICAL NATURE OR SEVERITY FOR DRAINAGE APPLICATIONS

Category	Critical	Less Critical
A. Critical Nature of the Project		
1. Risk of loss of life and/or structural damage due to drain failure	High	None
2. Repair costs versus installation costs of drain	Much Higher	Equal or Less
3. Evidence of drain clogging before potential catastrophic failure	None	Yes
B. Severity of the Conditions		
1. Soil to be drained	Gap-graded, pipable or dispersible	Well-graded or uniform
2. Hydraulic gradient	High	Low
3. Flow conditions	Dynamic, cyclic or pulsating	Steady-state

(ADAPTED FROM CARROLL, 1983)

Clogging resistance for less critical/less severe conditions should be designed to meet:

$$O_{95\ geotextile} \geq 3\,D_{15\ retained\ material} \tag{6-28}$$

This relationship applies to soils with $C_u > 3$. For $C_u < 3$, a geotextile with the maximum *AOS* value should be selected. In situations where clogging is possible (e.g., gap-graded or silty soils), the following optional qualifiers may be applied:

- For nonwovens – porosity of the geotextile $n \geq 50$ percent
- For woven monofilament and slit-film wovens – percent open area (POA) $\geq$ 4 percent

Most common non-woven geotextiles have porosities much greater than 70 percent. While most woven monofilaments easily meet the criterion of $POA \geq 4$ percent, tightly woven slit films do not and are therefore not recommended for subsurface drainage applications.

For critical/severe conditions, geotextiles should be selected in accordance with the retention and hydraulic conductivity criteria described previously. A filtration test should be conducted to check for clogging using samples of on-site materials and hydraulic conditions such as long-term flow tests or the gradient ratio test, which is described in ASTM D 5101, "Standard Test Method for Measuring the Soil-Geotextile System Clogging Potential by the Gradient Ratio." Dam safety agencies have expressed concern about the use of geotextiles in critical filters and drains (e.g., structures critical for the control of internal erosion and piping failures) due to susceptibility to:

- Excessive clogging from buildup of fines at the face of the geotextile or from biological, fungal or mineral matter buildup
- Separation at interfaces, junctions, connections and boundaries
- Undetected damage during construction.

These performance problems are related to the following mechanisms: (1) inability to support the seepage discharge face, (2) excessive clogging or piping, (3) stress-induced distortion, (4) environmental degradation, (5) slope instability, and (6) rupture. Designers should consider these concerns and performance issues when using geotextiles and, as part of the planning process, should discuss the acceptability of any proposed application with MSHA and state agencies.

Talbot et al. (2000) describe concerns about the use of geotextiles in dams, including the propensity for excessive clogging because seepage forces move the geotextile away from the base soil and into the voids of the adjacent drain. Thus, separation can occur between the base soil and geotextile allowing fine particles from the base soil to accumulate at the interface, thus “blinding” the filter. Blinding can also occur if there are open voids in the base soil or if the base soil surface is irregular and has poor contact with the geotextile. These concerns can be addressed through: (1) use of fine gravel (about ¾-inch to 1½-inch maximum size) for the drainage layer to improve contact with the base soil (Giroud, 1997; van Zyl and Robertson, 1980), (2) grading the base soil surface smooth and placing the geotextile in contact with the base soil with a minimum of wrinkles, and (3) minimizing vertical or steeply inclined slopes (FEMA, to be published in 2009).

The potential for particulate clogging can be addressed through application of filter criteria, and chemical and biological clogging can be addressed based on evaluation of the drain environment and/or testing, as discussed subsequently. Stress-induced distortion, environmental degradation, and stability can be addressed with design and laboratory testing procedures.

Chemical clogging of geotextiles at coal refuse disposal sites will most likely be associated with iron and manganese oxidation and deposition, although it is possible that calcium carbonate precipitation may also be encountered. These processes are affected by chemical reactions and biological activity that varies depending on whether the environment is anaerobic and aerobic. Factors that may lead to problems with clogging of drains are discussed by Smith and Hosler (2006). Research into predictive methods has generally concentrated on biological clogging and the role of microbial activity, and more work is necessary for prediction of potential chemical clogging. Long-term hydraulic conductivity testing procedures can be used to evaluate the potential effect of chemical clogging.

Koerner and Koerner (2005) provide drainage reduction factors for determination of the design flow rate or transmissivity of geotextiles. These reduction factors address soil clogging and blinding, reduction in void space, and chemical and biological clogging. Koerner and Koerner recommend reduction factors to the design flow capacity of a geotextile of between 1.2 and 1.5 for landfill filters to account for chemical clogging, and they suggest adoption of a high reduction factor where total suspended solids in the permeating liquid is greater than 5,000 mg/l. Similarly, they recommend reduction factors of between 2 and 5 to the design flow capacity of a geotextile to account for biological clogging and suggest adoption of a high reduction factor where biochemical oxygen demand (BOD) is greater than 5,000 mg/l.

The potential for biological clogging can be examined in accordance with ASTM D1987, “Standard Test Method for Biological Clogging of Geotextile or Soil/Geotextile Filters.” However, before using this test, Mackey and Koerner (1999) recommend that to facilitate interpretation of the results of the test the physical, chemical and biological processes at the site be evaluated and understood. If clogging is a concern, a higher-porosity geotextile can be used, and/or the drain design and operation can include an inspection and maintenance program for flushing the drainage system. For nonwoven geotextiles, Luettich et al. (1992) recommend using the largest porosity available, but not less than 30 percent; for woven geotextiles they recommend using the largest POA, but not less than 4 percent. Because of concerns for clogging, geotextiles with the largest opening sizes that satisfy piping requirement should generally be used.

Survivability Criterion

For filtration and drainage applications, if a geotextile is to survive the construction process, certain minimum strength and endurance properties are required. Table 6.50 provides these minimum requirements for less critical/less severe applications. It is important to understand that these minimum survivability parameters are based on empirical data from geotextiles that have performed satisfactorily in drainage applications. These parameters serve as guidelines for selecting geotextiles for most projects. The guidelines are not intended to replace site-specific evaluation, testing, and design.

Geotextile material should be covered subsequent to installation as soon as possible to prevent degradation from sunlight or in accordance with the manufacturer's recommendations. Geotextile endurance

TABLE 6.50 GEOTEXTILE STRENGTH PROPERTY REQUIREMENTS FOR DRAINAGE GEOTEXTILES(1,2,3,4)

Property	ASTM Test Method	Units	Geotextile Class 2(5)	
			Elongation	
			< 50%(6)	≥ 50%(6)
Grab Strength	D 4632	N (lb)	1100 (247)	700 (157)
Sewn Seam Strength(7)	D 4632	N (lb)	990 (223)	630 (142)
Tear Strength	D 4533	N (lb)	400(8) (90)	250 (56)
Puncture Strength	D 4833	N (lb)	400 (90)	250 (56)
Burst Strength	D 3786	kPa (lb/in^2)	2700 (392)	1300 (189)

Note:
1. Acceptance of geotextile material shall be based on ASTM D 4759, "Standard Practice for Determining the Specification Conformance of Geosynthetics."
2. Acceptance shall be based upon testing of either conformance sampler obtained using Procedure A of ASTM D 4354, "Standard Practice for Sampling of Geosynthetics for Testing," or on manufacturer's certifications and testing of quality assurance samples obtained using Procedure B of ASTM D 4354.
3. Values apply to minimum strength; use value in weaker principal direction. All numerical values represent minimum average roll value (i.e., test results from any sampled roll in a lot shall meet or exceed the minimum values in the table). Lot samples according to ASTM D 4354.
4. Woven slit film geotextiles will not be allowed.
5. AASHTO Geotextile Class 2 is the default geotextile selection. The engineer may specify AASHTO Class 3 geotextiles for trench drain applications based on one or more of the following:
   a) The engineer has found Class 3 geotextiles to have sufficient survivability based on field experience.
   b) The engineer has found Class 3 geotextiles to have sufficient survivability based on laboratory testing and visual inspection of a geotextile sample removed from a field test section constructed under anticipated field conditions.
   c) Subsurface drain depth is less than 2 m, drain aggregate diameter is less than 30 mm, and the compaction requirement is ≤ 95 percent of ASTM D 698, "Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft^3 (600 kN-mm^3))."
6. As measured in accordance with ASTM D 4632, "Standard Test Method for Grab Breaking Load and Elongation of Geotextiles."
7. When seams are required. Values apply to both field and manufactured seams.
8. The required MARV tear strength for woven monofilament geotextiles is 250 N (56 lb).

is related to longevity. Geotextiles have been shown to be basically inert materials in most environments and applications. However, some applications may expose the geotextile to chemical or biological activity that could dramatically affect filtration properties or durability. For example, in drains, granular filters and geotextiles can become chemically clogged by iron or carbonate precipitates and biologically clogged by algae and mosses. Biological clogging is a potential problem when filters and drains are periodically inundated then exposed to air. Excessive chemical and biological clogging can significantly affect filter and drain performance, and monitoring with piezometers is recommended.

6.6.2.3.3 Seepage Control along Conduits

Seepage along conduits that pass through coal refuse embankments should be controlled. Two methods for seepage control are filter diaphragms and anti-seep collars (also known as cutoff collars). Since the mid-1980s, the use of anti-seep collars has become less common and filter diaphragms have become more widely used (Van Aller, 1998). Many dam safety agencies require the use of filter diaphragms because of the many instances where seepage problems have occurred along conduits even when anti-seep collars were used. The primary advantage of filter diaphragms is the relative ease of construction as compared to anti-seep collars, particularly with respect to compaction around conduits. The presence of anti-seep collars complicates compaction around conduits, and accordingly they are more likely to function poorly due to construction flaws. Also, filter diaphragms are considered to be a better measure for mitigating the consequences of embankment cracking associated with the presence of a conduit. Filter diaphragms should be constructed with suitable filter materials, and careful placement is required during construction. The following sections present basic design considerations for both filter diaphragms and anti-seep collars.

Filter Diaphragms

A filter diaphragm is used for intercepting seepage through backfill pores or cracks and to prevent internal erosion of the backfill materials along buried conduit installations. To meet filtration and drainage requirements, filter diaphragms may consist of a single zone or multiple zones of granular material. The guidance for dimensioning filter diaphragms provided in the following text is taken from the USDA (1985) and NRCS (2007b). Van Aller (1998) discusses various aspects of filter diaphragm design, and McCook (2002) discusses site-specific conditions that should also be considered.

Filter diaphragms should be located approximately parallel to the centerline of the embankment and approximately perpendicular to the direction of seepage flow, and should extend horizontally and vertically into adjoining portions of the embankment and foundation. In homogeneous dams, filter diaphragms should be located using the following criteria:

- Downstream of the cutoff trench
- Downstream of the centerline of the dam if there is no cutoff trench
- Upstream of a point where the embankment cover (from the upstream face of the diaphragm to the downstream face of the embankment) is at least one-half of the difference between the elevation of the top of the filter diaphragm and the maximum potential impoundment water level

In zoned embankments, the filter diaphragm should be located downstream of the core zone and/or cutoff trench, in accordance with the minimum cover guidance for homogeneous dams. In instances where the downstream shell of a zoned embankment is more pervious than the diaphragm material, the diaphragm should be located at the downstream face of the core zone.

Provisions should be made for discharging seepage and groundwater collected by the filter diaphragm. These provisions could include tying the diaphragm into other drainage systems in the foundation, tying into internal embankment drains or designing/constructing a separate outlet for the filter diaphragm. Such an outlet should be designed with the assumption that the hydraulic conductivity in the zone upstream of the filter diaphragm is 100 times the hydraulic conductivity in the compacted embankment material. This zone should have a cross-sectional area equal to the area of the filter diaphragm, and the length of the seepage path should equal the distance from the embankment upstream toe to the filter diaphragm. This higher hydraulic conductivity is intended to account for partially-filled cracks and openings in the upstream zone. An advantage of having a separate outlet for the filter diaphragm is that the seepage outflow can be monitored separately from other seepage flows and will provide feedback as to the performance of the diaphragm system over time.

For rigid conduits, the NRCS (2007b) recommends that filter diaphragms should extend the following minimum distances from the conduit surface (Figure 6.50):

1. Horizontally and vertically upward 3 times the outside diameter of circular conduits or the vertical dimension of rectangular box conduits except that:
   a. The vertical extension need be no higher than the maximum potential impoundment level.
   b. The horizontal extension need be no further than 5 feet from the sides and slopes of any excavation for installation of the conduit.
2. Vertically downward from the conduit:
   a. Filter diaphragms should extend from the pipe support 1.5 times the outside diameter of circular conduits or outside vertical dimension of box conduits or to the top of rock, whichever is shallower.
   b. Alternatively, for conduit settlement ratios δ of 0.7 and greater, filter diaphragms should extend the greater of 1 foot beyond the bottom of the trench excavation for the conduit or 2 feet. The diaphragm should be terminated at the surface of bedrock when it occurs within this distance. Additional control of general seepage through an upper zone of weathered rock may be needed. The conduit settlement ratio δ is defined in NRCS (2007a) and Technical Release-5 (TR-5) by the USDA (1958) and requires a complex computation. On firm foundations with $\delta = 0.7$ or greater (the conduit settlement ratio for pipe on rock is 1.0), the filter diaphragm should extend below the pipe to rock, or at least 2 feet.

For flexible conduits, NRCS recommends that filter diaphragms be designed to extend in all directions a minimum of 2 times the outside diameter from the surface of the conduit, except that the diaphragm need not extend beyond the limits in 1a and 1b described above or beyond a bedrock surface beneath the conduit.

Filter diaphragms should have a minimum thickness of 3 feet, but the thickness specified should be appropriate for the level of quality control and supervision during construction. If a multi-zone system is employed to satisfy filter criteria, a minimum thickness of 1 foot should be used for any single zone. Greater thickness may be required as dictated by: (1) flow capacity requirements, (2) the need to tie the filter diaphragm into embankment internal or foundation drains, or (3) the need to accommodate construction methods. An example design for a filter diaphragm associated with a decant pipe is shown in Figure 6.51.

Some state regulatory agencies have developed specific guidance on filter diaphragms. Also, McCook (2002) discusses site-specific conditions that may warrant enlarging the filter diaphragm relative to

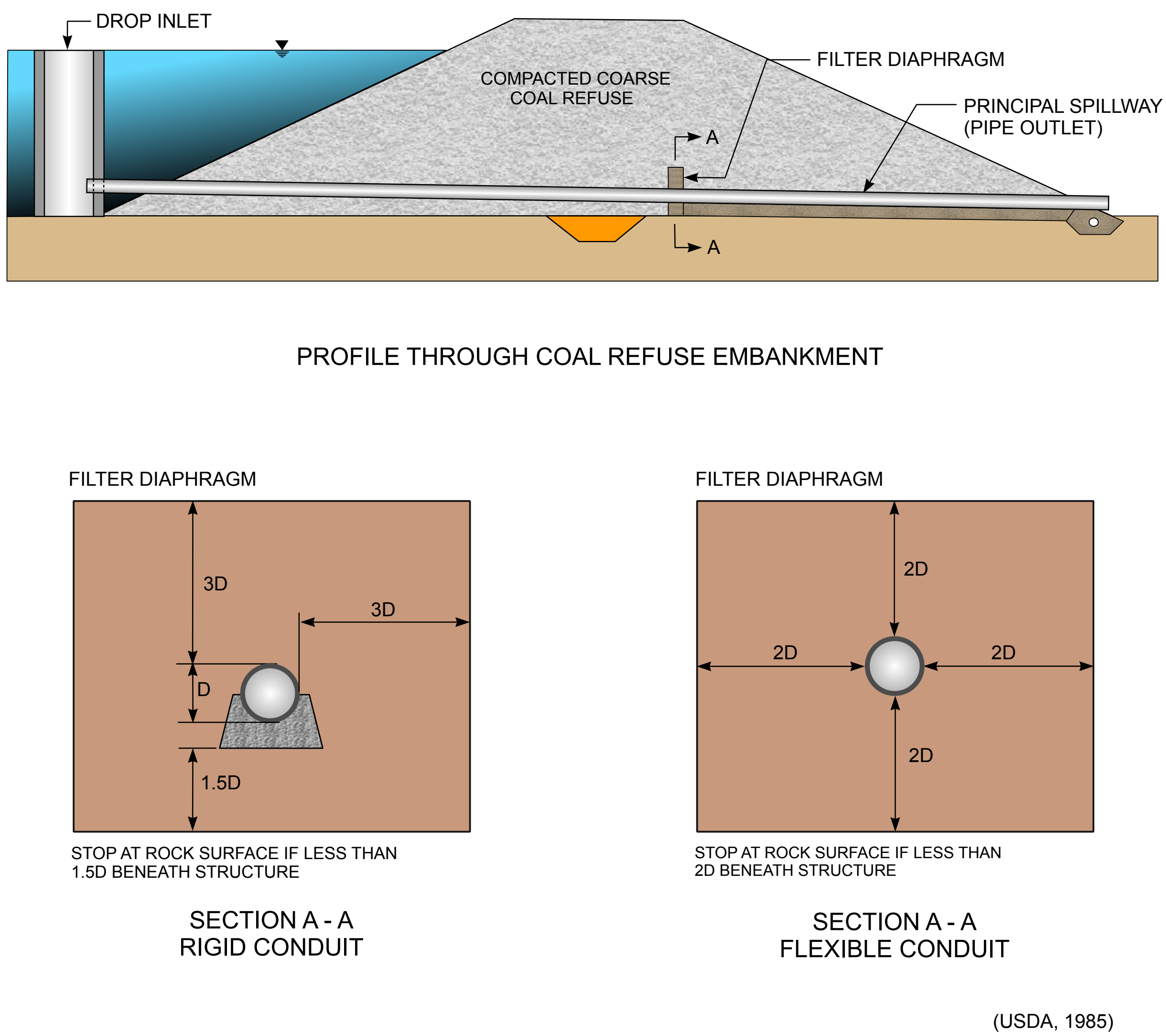


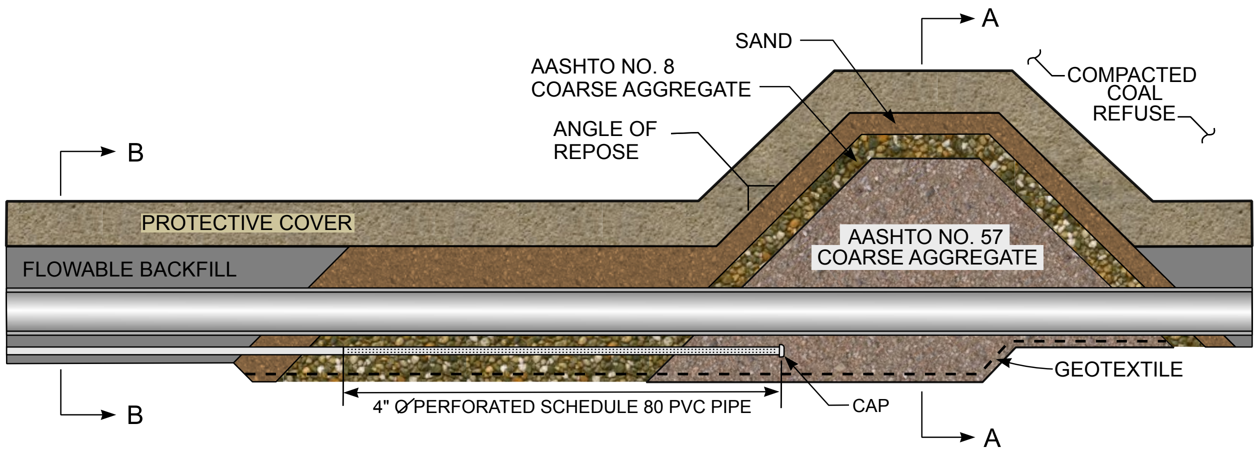
FIGURE 6.50 FILTER DIAPHRAGM DESIGN FOR CONDUIT

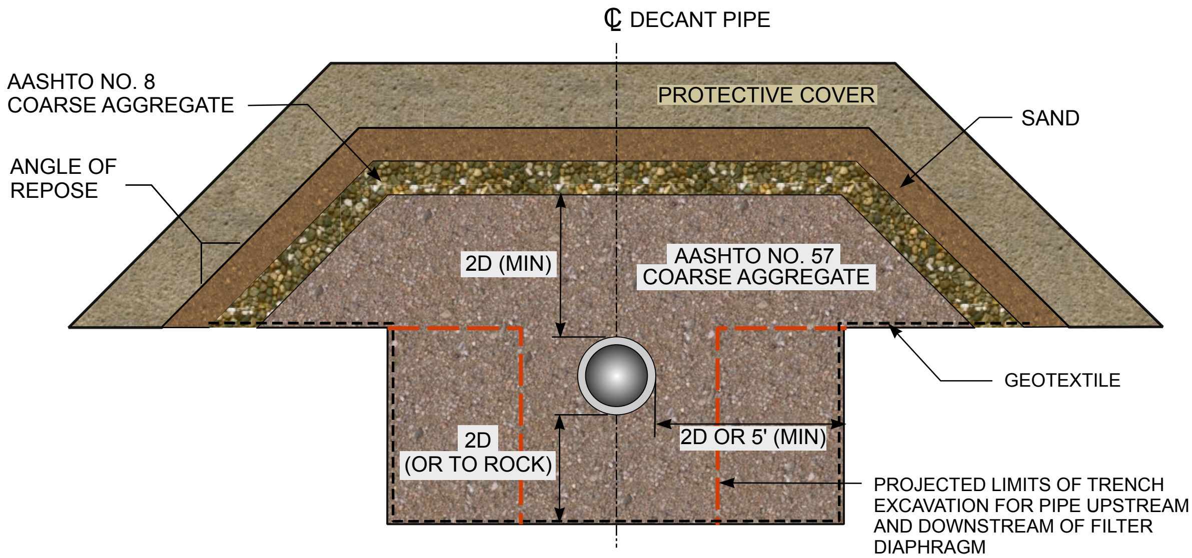
the minimum guidance cited above. These conditions include foundations with varying rock surfaces, soft soils, and situations where there is potential for differential settlement and related strain.

Anti-seep Collars

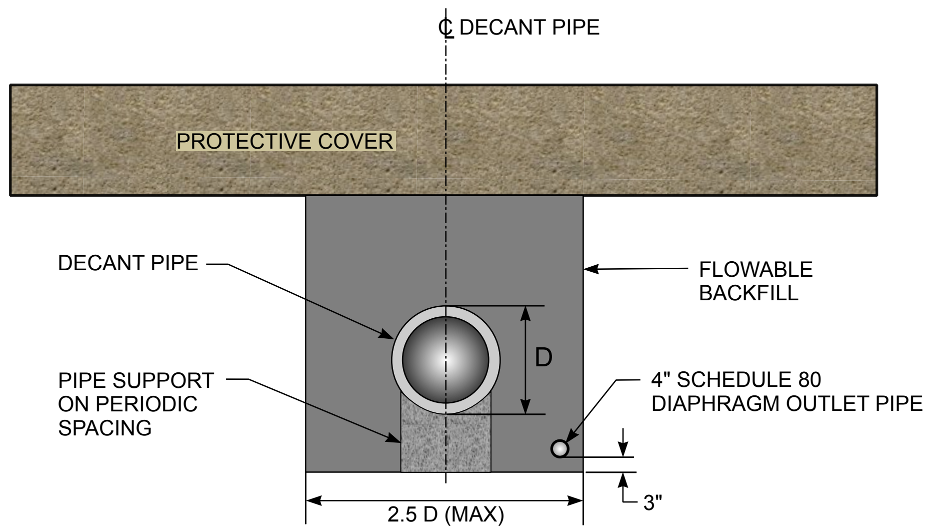
Cutoff or anti-seep collars are intended to minimize seepage along the contact between the outside surface of a conduit and an embankment. Van Aller (2004) and FEMA (2005a) provide compelling reasons to use filter diaphragms rather than anti-seep collars. For low-hazard-potential structures with dam heights of 35 feet or less and storage volume less than 3,000 acre-feet, the NRCS (2002) allows consideration of anti-seep collars. In coal refuse embankment dams, there may be site-specific reasons to use anti-seep collars. For example, during early stages of construction, installation of anti-seep collars may be preferable to an internal drainage structure that, in the case of downstream construction, could ultimately be located relatively far upstream and perhaps beneath the impoundment during later stages of construction. In such situations, the pipe and collar backfill should be designed so as to minimize the potential for concentrated seepage zones and internal erosion.

Cutoff collars have been fabricated using concrete, steel, and plastic for consistency with conduit materials. The intent of their use is to increase the length of percolation along the conduit contact





SECTION A - A



SECTION B - B

FIGURE 6.51 EXAMPLE DESIGN CONFIGURATION FOR FILTER DIAPHRAGM AND DECANT PIPE surface by 20 to 30 percent for significant- and high-hazard-potential dams (USBR, 1987a). For a conduit on an earth foundation, the cutoff collar should completely encircle the conduit. Where the foundation is sound rock and good contact along the base is expected, cutoff collars will need to extend only to a depth sufficient for keying into the rock foundation. Cutoff collars should be separated from rigid conduits using watertight fillers (gasket or seal) to avoid introducing concentrated stresses into the walls of the conduit. Non-rigid collars can be attached or clamped to flexible conduits using gaskets that accommodate conduit deformation such that a watertight connection is achieved.

For small, low-hazard-potential dams, the NRCS (2002) recommends the use of filter diaphragms unless it is determined that anti-seep collars will adequately serve the purpose. If anti-seep collars are used, the NRCS recommends increasing the flow path along the conduit by at least 15 percent, with 10- to 25-foot spacing between collars.

6.6.2.3.4 Relief Wells

If a pervious layer underlies a relatively impervious layer beneath the toe of an embankment slope, pore-water pressures can build up in the pervious layer to produce an artesian effect if drainage downstream is impeded and the layer is recharged upstream by groundwater, seepage from the impoundment, or rainfall. If artesian water pressure must be reduced for stability, relief wells can be used, as shown in Figure 6.52.

Water collected by relief wells is usually conducted through a horizontal overflow pipe at the ground surface and discharged into a lined drainage ditch near the toe of the embankment. With this collection method, the phreatic surface can be lowered to the ground surface. If a greater lowering of head is required, the outfalls from the wells and the collection drain can be lowered below the ground surface provided that they can be connected to the collection drain through a discharge pipe. To allow for inspection and maintenance, relief well casings should extend to the ground surface.

The spacing required for relief wells depends on the geology and groundwater hydrology at the site. As with embankment seepage modeling, an analysis should be performed to evaluate the impact of relief wells on foundation pressures. Numerical modeling is well suited to such an analysis, but requires an understanding of groundwater conditions, including geometry, pre-construction piezometric levels and the connection of the pervious strata with recharge features such as the impoundment or local aquifers. In instances where the model can be sufficiently simplified, flow nets can be used successfully. As a check, an estimate of the required spacing of relief wells can be made using methods presented by Leonards (1962). Given the limitations inherent in this type of modeling, the adequate function of the relief wells should be monitored with piezometers. A relief well system can be readily expanded if the initial configuration fails to produce the desired reduction in piezometric head.

Attention should be given to relief well design details so that the wells meet performance requirements and can be properly inspected and maintained. The chemical content of the water to be recovered should be evaluated to determine if any special precautions are needed for preventing corrosion of any part of the relief well system. The diameter of the internal perforated pipe depends on the anticipated flow volume, but should be no less than 6 inches. To facilitate inflow and to prevent clogging, relief wells should be surrounded by a filter designed according to the requirements discussed earlier in this section.

The annulus around the portion of the relief well above the pervious layer should be sealed with an impervious material (such as hydrated bentonite or a cement-bentonite mixture) or concrete to prevent upward flow of water around the pipe. It may be necessary to temporarily lower the water level in the relief well during the construction of this seal.

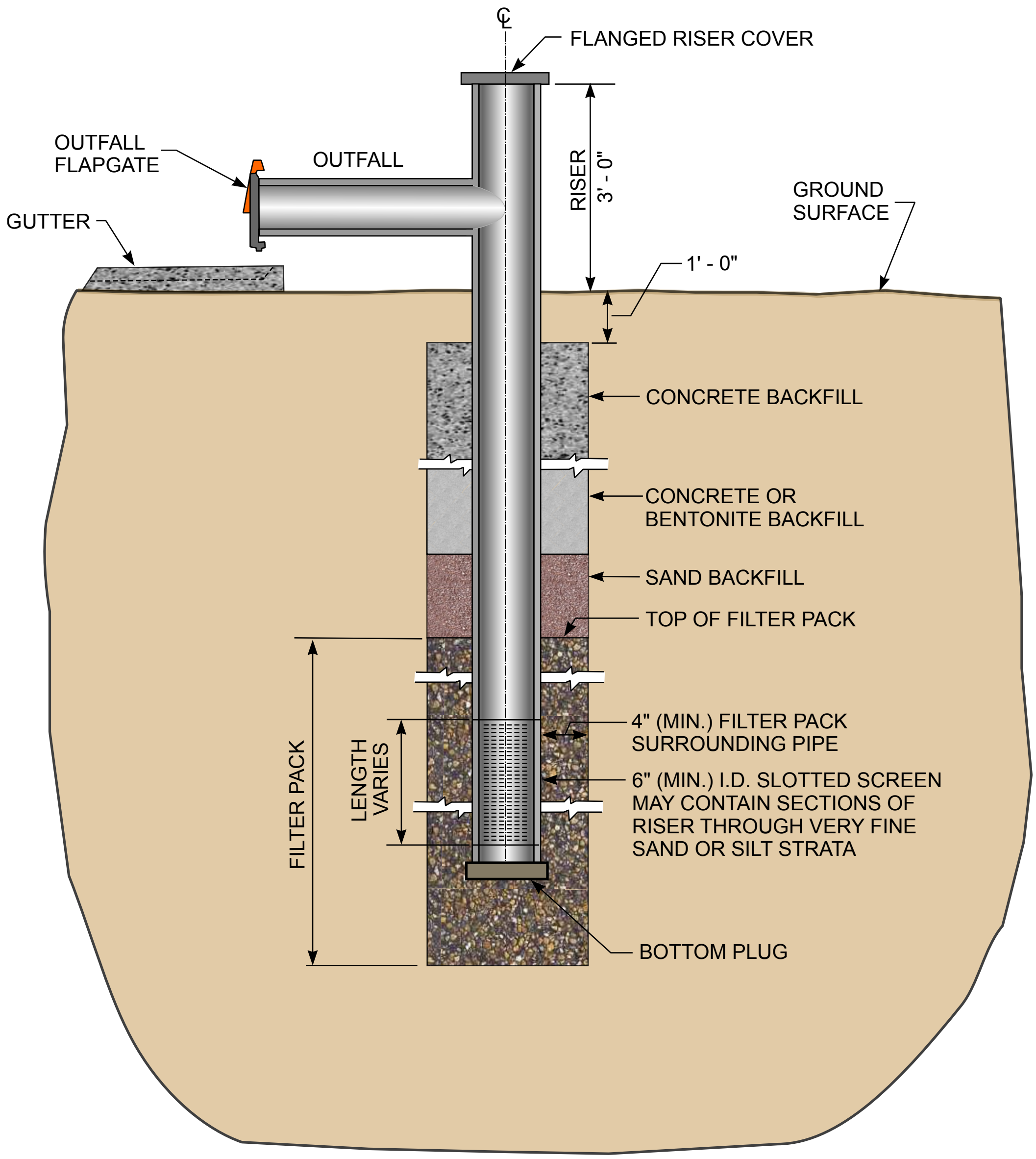
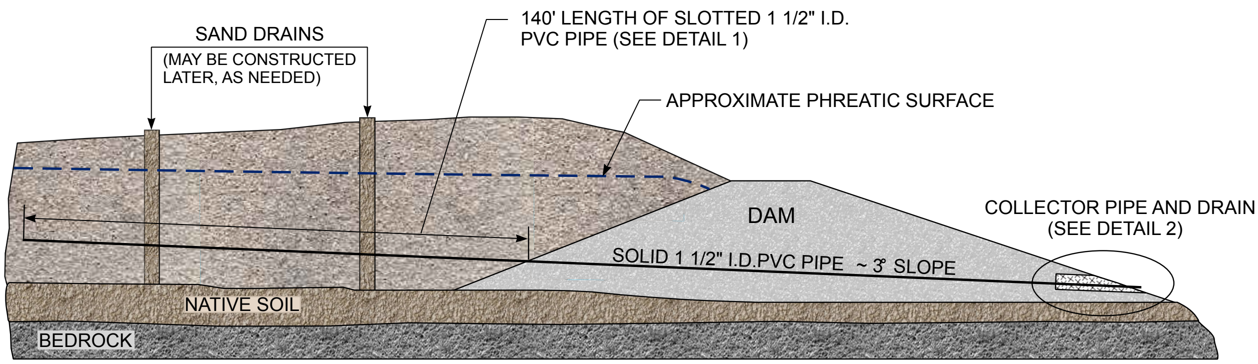


FIGURE 6.52 TYPICAL RELIEF WELL INSTALLATION

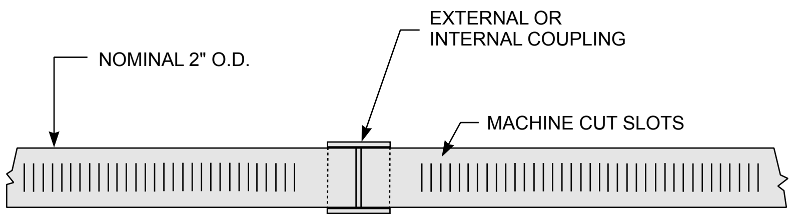
6.6.2.3.5 Horizontal Drains

Horizontal drains perform a similar function to relief wells, but provide more effective drainage either in the foundation under the main body of an embankment or within the embankment. Most often, horizontal drains are used to reduce excessive pore-water pressure within or beneath an existing embankment. A typical horizontal drain installation and details are shown in Figure 6.53. Horizontal drains are normally sloped toward the discharge end.

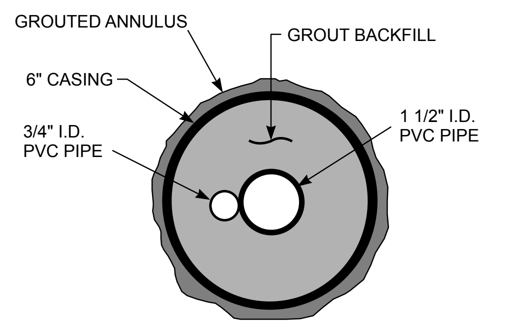
Although horizontal drains 600 feet long and longer have been installed, lengths of 400 feet or less are more common. The drains, which are usually slotted plastic pipe, are normally installed inside steel drill rods, which are subsequently retracted. Because there is no soil filter around the slotted pipe, the slots must be sized to prevent infiltration of fines based upon the rules discussed in Section 6.6.2.3.1. In some cases, porous plastic pipe has been placed around the slotted pipe to limit the infiltration of fines. The spacing required between horizontal drains is difficult to determine accurately. Pore-water pressure should be monitored with piezometers to check the performance of an installation. A system



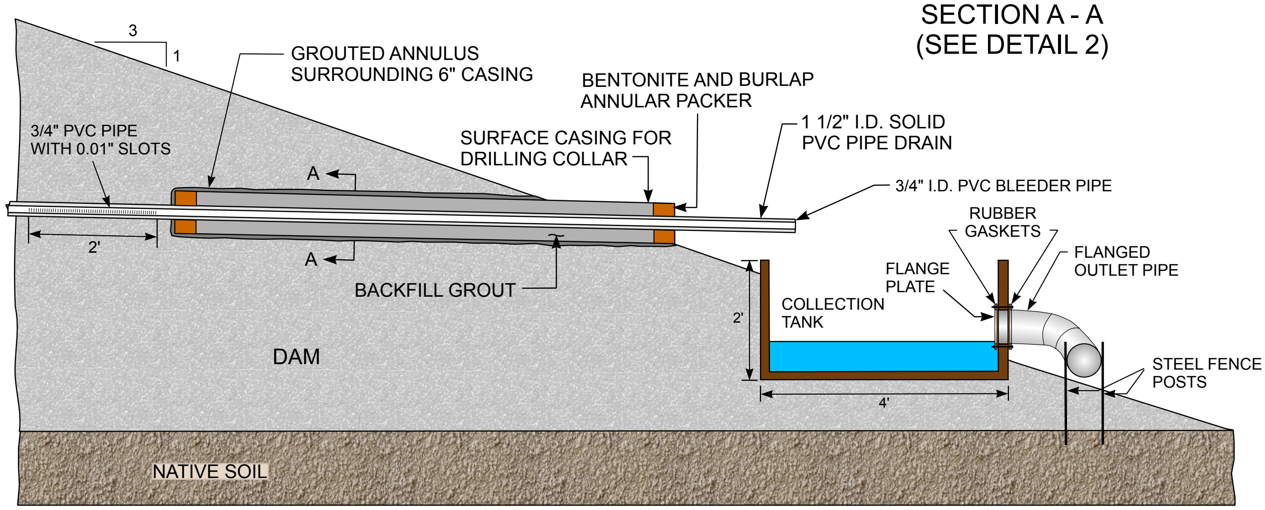
EMBANKMENT CROSS SECTION



DETAIL 1 - 1 1/2" I.D. PVC PIPE WITH 0.02" MACHINE-CUT SLOTS



SECTION A - A
(SEE DETAIL 2)



DETAIL 2 - DRAIN COLLAR AND DRAINAGE COLLECTION

FIGURE 6.53 EXAMPLE HORIZONTAL DRAIN INSTALLATION of horizontal drains can be readily expanded if the capacity of the initial installation fails to produce the required reduction in piezometric head.

The installation of horizontal drains can be dangerous in situations where large volumes of seepage could lead to a piping failure at the drain borehole during installation. Horizontal drain systems should be installed by an experienced contractor under expert supervision.

6.6.2.3.6 Impoundment Liners

To protect the groundwater some state agencies may require that seepage from fine coal refuse be limited. This can be accomplished by using a layer of low-hydraulic-conductivity soil or geomembrane liners. Geomembranes are manufactured, low-hydraulic-conductivity synthetic materials that function as barriers to liquids and vapors. When used as a liner on the bottom and sides of a refuse disposal impoundment, a geomembrane can impede leachate migration from overlying refuse into the underlying soil and groundwater and can be used to collect the leachate for treatment. When a geomembrane is used as a cap in the final cover over the impoundment, it prevents precipitation from infiltrating the coal refuse, thus minimizing or eliminating leachate generation.

Minimum requirements for geomembrane liners are often specified in state regulations. Minimum strength properties are provided in Table 6.51. However, if the application is on a slope or there is a possibility that differential settlement could occur, increasing stress and strain on the geomembrane, a more conservative choice of membrane thickness may be appropriate. The strength of the liner is usually reduced at seams. Standard tests for shear and peel strength should be performed on both factory and field seams. A determination must be made as to the minimum percentage of geomembrane material strength that the seam itself must possess. This minimum seam strength criterion is typically incorporated into the quality assurance/quality control (QA/QC) program for liner installation. For evaluation of survivability, the minimum seam strength should be determined early in the design stage if it is likely that the geomembrane will be subjected to ultraviolet radiation for a significant period of time. If that is the case, a material with a high resistance to ultraviolet light deterioration should be used.

TABLE 6.51 RECOMMENDED MINIMUM PROPERTIES FOR GENERAL GEOMEMBRANE INSTALLATION SURVIVABILITY

Property and Test Method		Required Degree of Survivability			
		Low(1)	Medium(2)	High(3)	Very High(4)
Thickness – ASTM D 1593	(mils)	20	25	30	40
Tensile – ASTM D 882 (1-in strip)	(lb/in)	30	40	50	60
Tear Resistance – D 1004 (Die C)	(lb)	5	7.5	10	15
Bursting Strength – D 3787	(lb)	20	25	30	35
Impact Resistance – D 3998	(ft-lb)	10	12	15	20

Note:
1. Low refers to careful hand-placement on very uniform, well-graded subgrade with light loads of a static nature – typical of vapor barriers beneath building floor slabs.
2. Medium refers to hand- or machine-placement on machine-graded subgrade with medium loads – typical of canal liners.
3. High refers to hand- or machine-placement on machine-graded subgrade of poor texture with high loads – typical of landfill liners and covers.
4. Very high refers to hand- or machine-placement on machine-graded subgrade of very poor texture with high loads - typical of reservoir covers and liners for heap leach pads.

(ADAPTED FROM KOERNER, 2006)

6.6.2.3.7 Foundation Seepage Cutoffs

In addition to measures to control seepage in embankments, foundation treatments such as cutoff trenches backfilled with compacted low-hydraulic-conductivity materials are also routinely incorporated into refuse facility designs. Foundation cutoff trenches are discussed in Section 6.3.

6.6.2.3.8 Impoundment Water and Slurry Deposition Management

Management of impoundment water and slurry deposition at a coal refuse disposal facility are operational measures for controlling and mitigating the effects of seepage. Maintenance of a low level of clarified water in an impoundment reduces the hydraulic head and the source volume for seepage through the embankment. Additionally, deposition of coal refuse slurry at or near the upstream embankment face and the resulting build up of a delta of fine refuse against the embankment will reduce seepage into the embankment if the fine refuse has a lower hydraulic conductivity than the embankment material. To develop and maintain an effective fine refuse delta across the upstream face of the embankment, periodic relocation of the slurry discharge point or use of multiple discharge points may be required. At some facilities, small cells have been constructed near the upstream end of a refuse impoundment for storage of water for recirculation to the preparation plant. Such provisions reduce the likelihood of water being impounded directly against a coarse refuse embankment and associated seepage concerns.

6.6.2.4 Seepage Measurements

When seepage rates or piezometric levels are an important factor in the performance of a disposal facility, estimates of the phreatic surface, magnitude and rate of dissipation of pore pressures, and quantity of seepage collected in internal drains should be made. These predictions should be checked during and after construction by instrumenting the refuse embankment (and foundation if necessary) and measuring changes in the phreatic surface and pore pressures. Procedures for monitoring groundwater levels and pore-water pressures are discussed in Chapter 13. Field piezometer data provide a basis for updating performance predictions and for possible modification of design or construction procedures. Weirs or parshall flumes at the outlets of internal drains can be used to measure the seepage collected from an embankment for comparison to results of seepage analyses associated with the internal drain design. In addition, instrumentation can provide a check on the in-situ embankment hydraulic conductivity, as compared to that assumed in the analyses and/or determined by laboratory or field hydraulic conductivity tests. Monitoring of seepage conditions is also important for detection of unanticipated changes in the saturation level or seepage quantity and can provide an early indication of a problem such as clogging of an underdrain.

6.6.3 Settlement Analysis

6.6.3.1 Conditions Requiring Deformation Analysis

Settlement of a coal refuse embankment occurs as a result of embankment compression, foundation compression, plastic deformation, differential settlements, mine subsidence or a combination of these effects. Settlement is usually important if an embankment will impound water or if it will serve as the foundation for future construction. The design of any impounding embankment must limit settlement of the crest so that the freeboard is not reduced below the allowable limit. Embankments should be cambered so that the crest is elevated relative to the abutments to compensate for the increased settlement that typically occurs near the center of the embankment where the foundation overburden and embankment height are the greatest. It is also important that the embankment does not settle so much that the hydraulic conductivity characteristics of the embankment are significantly changed or the potential for piping or internal erosion due to cracking is created. Embankment and foundation settlement can also affect the performance and structural integrity of conduits and other structures. The settlement that can be tolerated by a structure depends upon its function.

The magnitude of settlement under self-weight experienced by a coal refuse embankment cannot be accurately estimated from fundamental stress-strain properties. The most useful information for computing embankment settlement is performance data from instrumented earth and rockfill dams. These data indicate that settlement of well-compacted earth dams due to embankment compression ranges from less than 1 percent for dams constructed of non-plastic soils to more than 4 percent for dams constructed of highly-plastic, fine-grained soils. Measured settlements of rockfill dams have ranged from essentially no settlement for well-compacted and sluiced rockfill placed in thin lifts to 10 percent or more for unsluiced rock placed in thick lifts. Therefore, the amount a coal refuse embankment will settle due to compression can range from 1 percent or less for well-compacted materials to 8 to 10 percent for uncompacted materials. However, a coal refuse embankment is often constructed over 20 to 30 or more years, as compared to one to three years for an earth or rockfill dam. Thus, nearly all coal refuse embankment settlement will likely occur during construction, and additional settlement after abandonment will be minimal.

If an embankment foundation consists of dense glacial till, dense sand and gravel, or rock, it will deform only slightly under the weight of the embankment because, at the stress levels experienced, both the foundation materials and any existing pore-water are essentially incompressible. Thus, settlement of such embankments due to foundation settlement will be minimal. If the foundation consists of saturated fine-grained soils, there may be significant settlement from consolidation due to the time-dependent expulsion of pore water from the soil. In such cases, the development of excess pore-water pressures can also affect foundation stability and these excess pressures need to be taken into account in embankment stability analyses.

Upstream construction of coal refuse disposal embankments over fine coal refuse deposits typically results in consolidation settlements that can give rise to elevated pore pressures if construction occurs rapidly. To prevent localized instability, construction procedures should be carefully planned. Monitoring of pore pressure may be needed as part of monitoring of embankment stability and controlling the rate of construction. These issues are discussed in Chapter 11.

Consolidation occurs slowly, sometimes over several months or years. Consolidation settlement can be a very important design factor because, in addition to causing damage to drainage pipes and structures, it can affect aspects of the entire disposal facility. Problems created by consolidation may not become apparent until after the disposal facility begins operation, at a time when the settlement could create a safety hazard and when corrections are most expensive. If consolidation is associated with embankment construction over soft clay deposits, the total settlement could be substantial. When a slurry impoundment is to be eliminated by backfilling the remaining reservoir, the settlement of the cap material must be considered in the design. Sufficient cap material must be placed such that following long-term settlement positive surface drainage will be maintained and a depression that will collect water is not created.

The magnitude of the foundation consolidation settlement depends on the weight of the embankment, the depth and thickness of the compressible strata in the foundation, and the compression indices of these compressible strata. Compression indices can be obtained from laboratory consolidation tests on undisturbed samples taken from the compressible strata. Compression indices from field settlement records for other disposal facilities underlain by similar compressible strata with similar moisture contents and index properties can also be used to predict the potential range of settlement.

The rate at which settlements occur is a function of the rate of change in vertical stress (directly proportional to the rate of construction), the hydraulic conductivity of the compressible material, and the drainage characteristics of the foundation. The rate of settlement is much more difficult to estimate than the magnitude of settlement. Computations based on laboratory consolidation test data may be inaccurate because the rate at which foundation settlements occur is often controlled by minute geological characteristics that may not be detected even by carefully conducted foundation studies.

Consolidation and conventional laboratory tests used for measuring the consolidation characteristics of a soil are discussed in Section 6.5.6.

Plastic deformation of an embankment and its foundation represents only a portion of the observed settlement at a coal refuse embankment. Plastic deformation is the lateral spreading of an embankment at the base coincident with settlement. Although difficult to determine, it is important to allow for the amount of plastic deformation that will occur because it can cause extension of pipes constructed through the embankment. Although the magnitude of such an extension may be as much as several percent of the total length of the pipe, damage can usually be avoided if the extension occurs uniformly over the length of the pipe. However, if the extension is concentrated or if the pipe cannot tolerate significant extension, it may separate and leak or fracture and collapse. Such an outcome could seriously affect embankment stability. To minimize the risk of this type of failure, pipes should either be placed in the foundation below the plastic zone or should be designed to tolerate the anticipated extension.

Differential vertical settlements can also cause damage to a coal refuse embankment and to interior structures. Differential foundation settlements, particularly if they occur over small distances, can result in embankment cracks and lead to subsurface erosion. Differential settlements can also cause damage to pipe drains and decant lines installed within or beneath an embankment. When considering the potentially damaging effects of foundation settlements beneath a planned coal refuse embankment, it should be assumed that some areas of the foundation may have localized settlements at least twice as great as the predicted overall settlement. Therefore, if substantial foundation settlements are expected, placing pipelines within or beneath an embankment should be avoided. If a decant pipe is placed in an embankment, it should be located as far as possible from the areas where the settlement is anticipated to be greatest.

Embankments constructed above or adjacent to underground mine voids can experience settlement and lateral distortion caused by subsidence. Depending on the characteristics of the overburden rock, the depth and type of mining, and whether the mining is pre-existing or is occurring during disposal operations, the subsidence can have a significant effect on the stability of the embankment and the retained coal refuse. Additional discussion of subsidence issues can be found in the following Manual sections:

- Section 5.4 for mine subsidence considerations during facility design
- Section 8.4 for analysis of mine subsidence
- Section 8.5 for mine breakthrough potential evaluations

Sherard (1973) reports that earth dam cores of almost all soil types have experienced cracking. Therefore, designers should carefully consider the surface topography in which, and the foundation condition and materials on which, the embankment will be constructed. For dams in narrow valleys, cracking patterns such as those shown in Figure 6.54 have been observed. Transverse vertical cracks often develop within about 15 to 50 feet from the end of the crest, and in the central portion where the embankment is the highest, vertical cracks often develop on or near the crest, as shown in Figure 6.54.

Based on the results of laboratory testing reported by Leonards and Narain (1963) and statistical evaluation of earth dam performance by Biarez et al. (1970) and Londe (1970), as reported by Sherard (1973), the tensile strain at which first cracking occurs is in the range of 0.1 to 0.3 percent. In most of these case histories, the embankments were placed with a moisture content from 1 to

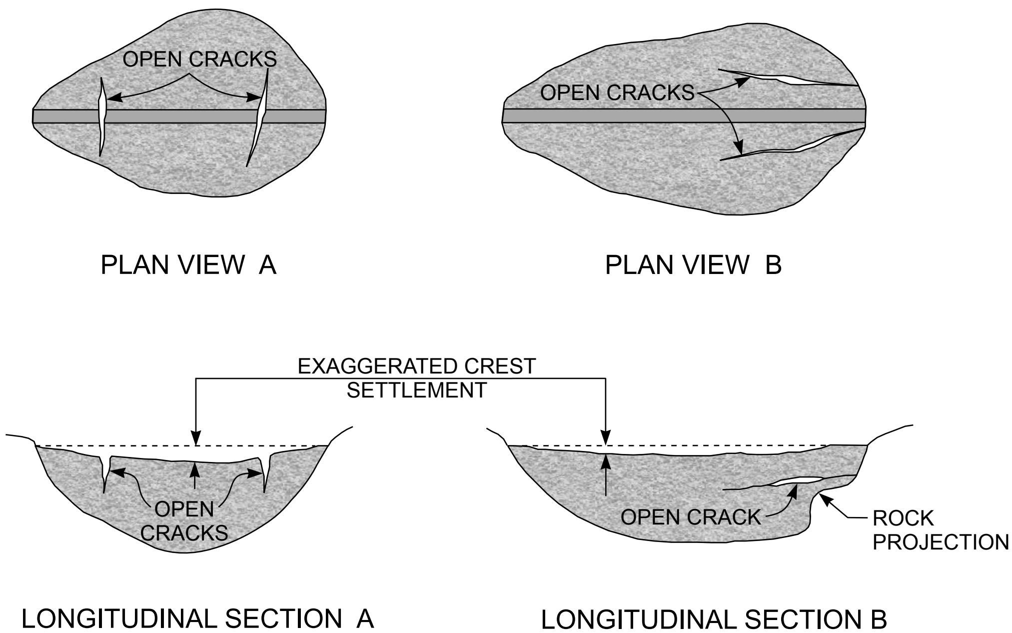


FIGURE 6.54 CRACKING PATTERNS OBSERVED FOR DAMS IN NARROW VALLEYS

4 percent dry of optimum (Wilson and Squier, 1969). Tensile strains in earth dams should be considered, and precautionary measures to limit the magnitude of tensile strains should be incorporated, as necessary. Precautionary measures could include: (1) incorporating more crack resistant soils in the dam cross section where cracking is most likely, (2) taking added care in foundation preparation to avoid/minimize large changes in foundation grades, or (3) increasing the width of the embankment cross section. Regardless, designers should assess the potential effects of factors that can cause differential settlement and incorporate appropriate measures to limit the development of cracks during construction and long-term operations.

Broadly-graded coarse coal refuse, placed at or above optimum moisture content and at relatively low construction rates that allow settlement to occur as fill is gradually added, is generally considered to be less susceptible to cracking than earthen materials, which are frequently subject to longer-term consolidation, after completion of construction.

6.6.3.2 Settlement and Deformation Analysis

Embankment and foundation settlement and deformation are typically evaluated using principles of elastic behavior and/or Terzaghi consolidation theory. For most settlement and deformation analyses associated with coal refuse disposal facilities, stresses in foundations and embankments and changes in stress due to load application (e.g., embankment loading on the foundation) can be estimated using elastic theory. Closed-form equations and graphical methods for estimating stresses for a variety of geometric loading cases, homogeneous and layered subsurface profiles, and isotropic and anisotropic subgrade conditions are available in Poulos and Davis (1974). Methods for estimating settlement due to elastic compression and consolidation are described in numerous foundation engineering textbooks (Terzaghi et al., 1996; Hunt, 1986; Holtz and Kovacs, 1981) and design manuals (CGS, 2007; DOD, 2005).

Since the early 1970s, finite element (FE) methods that permit realistic deformation analysis of earthen embankments and foundations have been developed. As summarized by Duncan (1996), some of the special features of FE methods with application to embankments and foundations are:

- Versatile tool for analysis of stresses and movements in earth masses, including:
  - Stresses, deformations and pore pressures in embankments and foundations
  - Conditions during construction such as consolidation and embankment compression due to self-weight
  - Potential for cracking and hydraulic fracturing
- Can model nonlinear stress-strain behavior, non-homogeneous conditions, and changes in geometry such as embankment construction
- Software are available with graphical pre- and post-processors to facilitate data input and evaluate analysis results.

FE analyses require input data such as: (1) definition of the in-situ stress conditions of the foundation materials, (2) the stress-strain properties of the foundation and embankment materials, and (3) the sequence of construction.

For problems that involve a natural soil or rock deposit or an existing fill, the state of stress in the soil mass prior to the beginning of construction or loading must be specified because:

- For incremental analyses, the changes in stress calculated during each increment are added to the stresses at the beginning of the increment in order to evaluate the stresses at the end. To begin this process, it is necessary to know the initial in-situ stresses.
- The stiffness of the soil is a function of the stresses in the soil.

The in-situ stresses can be measured, but are usually estimated. For level ground where at-rest pressure conditions would be expected, the vertical stresses are usually assumed to be equal to the overburden pressure, and the horizontal stresses are assumed to be the at-rest lateral earth pressure coefficient K_o times the overburden pressure. The value of K_o is usually estimated based upon empirical relationships (Jaky, 1944; Mayne and Kulhawy, 1982). For initially sloping ground, one procedure that has been used is performing a gravity turn-on analysis (i.e., applying vertical forces representing the weight of the material to an initially unstressed mesh) and then changing the horizontal stresses to K_o times the calculated vertical stresses.

The stress-strain properties of modeled materials play a critical role in finite element analyses. For most deformation analyses of embankments and foundations where constituent materials are not stressed close to failure and strains are small, stress-strain behavior can be represented by a linear elastic model. However, for rock foundations, the modulus should reflect the deformation characteristics of the rock mass through modification of the deformation of intact (i.e., unfractured) rock using a rock mass classification system such as that proposed by Bieniawski (1989). The Bieniawski geomechanics classification system involves modification of the modulus of intact rock as determined from unconfined compression tests (Section 6.5.9.2) using the following relationship:

$$E_m = 2\,RMR - 100 \qquad (6\text{-}29)$$

where:

E_m = modulus of the rock mass (force/length^2)

RMR = rock mass rating; this parameter accounts for the effects of intact rock strength, rock quality designation (RQD), joint spacing, joint condition, joint orientation and groundwater (dimensionless)

RMR increases with rock quality with a range from 0 to 100. Equation 6-29 is valid for $RMR \geq 55$. For softer rocks, the following relationship, which was proposed by Serafim and Pereira (1983), can be used:

$$E_m = 10^{(RMR\text{-}10)/40} \qquad (6\text{-}30)$$

For soils it is often necessary to use stress-strain relationships that account for nonlinear behavior and the variation of soil modulus with confining pressure. Table 6.52 summarizes the types of stress-strain models typically used and their respective advantages and limitations.

TABLE 6.52 STRESS-STRAIN RELATIONSHIPS USED FOR FINITE ELEMENT DEFORMATION ANALYSES OF EMBANKMENTS AND FOUNDATIONS

Stress-Strain Relationship	Advantages	Limitations
Linear Elastic	Simplicity	Can only model real soil behavior at low stress levels and small strains
Multi-linear Elastic	Can model any shape stress-strain curve for ductile materials	Must be developed on a case-by-case basis to approximate stress-strain behavior
Hyperbolic	Can model nonlinear behavior; parameters have physical significance and can be evaluated by triaxial testing	Inherently elastic; does not model plastic deformations in a fully logical manner

(ADAPTED FROM DUNCAN, 1996)

Analyses should simulate as closely as possible the actual construction or loading sequence associated with the structure being analyzed. This can be accomplished by adding elements to simulate fill placement, removing elements to simulate excavation, and applying loads in increments. Other processes that can be modeled in FE analyses include raising or lowering phreatic levels and consolidation.

Comparison of the results of FE analyses with field measurements shows there is a tendency for calculated deformations to be larger than measured deformations. Duncan (1996) offered the following reasons for this difference:

- Soils/materials in the field tend to be stiffer than laboratory test samples at the same density and moisture content due to aging effects.
- Average field densities are higher than the specified minimum dry density, which is often used as the target for preparing samples for laboratory testing.
- Soils/materials sampled in the field for laboratory testing are disturbed by the sampling process.
- Most field conditions approximate plane strain whereas triaxial tests are routinely used for laboratory characterization.
- 2D finite element analyses overestimate deformations of embankments constructed in V-shaped valleys with steep walls.

Analysis of embankments using the FE method has demonstrated the considerable potential of the approach and has identified the sources of uncertainty that engineers should be aware of. These uncertainties are primarily associated with difficulties in accurately predicting the in-place density and moisture content of soils/materials in the field and difficulties anticipating the sequence of operations that will be followed during construction.

6.6.3.3 Deformation Control Measures

In a slurry impoundment, fine coal refuse typically has a low unit weight (as compared to typical soils), low hydraulic conductivity and low coefficient of consolidation. It is placed by peripheral discharge of low-solids-content slurry, either by single-point discharge or using multiple discharge locations. Because of this method of placement, fine coal refuse tailings can be very soft and susceptible to long-term settlements.

The rate of consolidation at existing refuse disposal sites can be expedited by the installation of prefabricated vertical (PV) drains into the tailings at close vertical spacing to drain excess pore-water pressures. PV drains consist of a high-flow polymeric core wrapped in a non-woven geotextile. The drains are installed using static, vibratory or jetting methods. Guidelines for the engineering design of PV drains are presented in Rixner et al. (1986). Brown and Greenaway (1999) describe instances where PV drains were used to expedite consolidation of uranium mill tailings before construction of a clay cap required for abandonment. Prefabricated drains have also been placed horizontally on the surface of a coal refuse impoundment prior to upstream construction to speed consolidation and dissipation of pore pressures (Thacker et al., 1988). The effectiveness of the drain installations was demonstrated by the control of pore pressures and consolidation. Drilled horizontal drains have also been used to lower pore-water pressures in tailings impoundments.

Drainage measures to accommodate dissipation of pore pressures and to consolidate fine materials such as fine coal refuse can be incorporated into facility design, as discussed in Section 6.3.

Adaptations of shallow and deep soil mixing technologies developed for improving loose and soft soil deposits have been used for the in-place solidification of fine coal refuse. Bazán-Arias et al. (2002) describe the use of these methods to stabilize fine coal refuse using custom-designed equipment to blend a cement/fly ash slurry with coal refuse for supporting a highway embankment over a slurry pond. QC testing of cured samples resulted in a 28-day, unconfined compressive strength greater than 100 psi and peak shear strength parameters of $\phi' > 45°$ and $c' > 13$ psi.

Lime has also been used to stabilize fine coal refuse. Lime stabilization causes the refuse to behave as an overconsolidated material. However, when load is applied that exceeds the apparent maximum past consolidation pressure, the "stabilized" refuse tends to collapse and return to its previous "unstabilized" behavior. Therefore, it is recommended that potential use of lime stabilization of fine coal refuse be carefully evaluated through laboratory and field performance testing before implementation in the field.

6.6.3.4 Deformation Measurements

When predicted settlements are an important factor in the design of a coal refuse disposal facility, estimates of the magnitude and the rate of settlement should be made using sophisticated analyses with the best available data. The performance of a coal refuse disposal facility should be monitored through installation of an instrumentation system and comparison of observed data to predicted deformations. Depending on site conditions and project requirements, deformations and deformation rates can be monitored by:

- Surface monuments (vertical and horizontal displacements)
- Settlement gages and extensometers (vertical displacements)
- Inclinometers (lateral displacements in slopes)
- Piezometers (piezometric heads in the embankment and foundation)

As described in Chapter 13, instrumentation should be installed to verify that acceptable levels of performance are being achieved and to provide a check on design assumptions.

6.6.4 Slope Stability Analysis

6.6.4.1 Conditions Requiring Stability Analysis

The design of a new coal refuse disposal facility or the expansion or modification of an existing facility requires that the stability of compacted embankments and natural soil and rock slopes and foundations be assessed. The most critical cross sections and cases must be analyzed. Factors considered in selecting the most critical cross sections include: (1) slope of the embankment, (2) height of the embankment, (3) foundation conditions, (4) pore-pressure conditions, and (5) presence of lower strength material zones within the embankment/impoundment (e.g., upstream construction cross sections). Critical cross sections can include upstream or downstream embankment slopes. Important cases include the following:

- Long-term or final embankment configuration
- Intermediate stages of development that may include critical cross sections related to slope, height, foundation, embankment material properties, or phreatic levels or pore-pressure conditions
- Short-term, end-of-construction conditions for stages on soft, compressible materials
- Rapid loading of fine coal refuse deposits during initiation of upstream construction that possibly leads to an unacceptably low factor of safety against bearing failure of the fine refuse underlying the area of embankment raising
- Rapid drawdown of the impoundment
- Seismic loading and strength loss or increases in pore pressure

Phreatic levels and pore-pressure conditions should be determined on the basis of seepage analyses and, for existing facilities, by correlations with piezometric measurements.

In selecting an upstream construction cross section for analysis, the interface between the coarse refuse embankment and fine refuse deposits should be determined based on subsurface exploration (borings and cone penetration tests are recommended) if an existing facility is being analyzed. For new facilities, analysis cross sections should be determined based on staging calculations considering upstream construction procedures and fine coal refuse behavior. Staging calculations will identify the approximate level of the settled fine refuse at the initiation of upstream construction. The following facility-specific issues must be considered:

- Type of settled fine coal refuse that will support the upstream stage (e.g., sandy or clayey fine refuse)
- Presence and location of impoundment pool relative to the extent of the upstream push out
- Staging area for coarse coal refuse to be used for the push out
- Presence of excess pore-water pressures in areas where the fine refuse is not fully consolidated
- Equipment and lift thickness that can be used for the push out
- Monitoring program that can be implemented and used in controlling the push out

The behavior of fine coal refuse in response to upstream construction may include consolidation, mixing with the coarse refuse during the initial push out, and development of a zone of assimilation. Appropriate material strengths and levels of excess pore-water pressure need to be used in analyses. Experience at similar facilities can provide valuable insight into material behavior, construction procedures to be employed, and the resulting zone of mixing. The strength in this zone of mixing should be determined based on the relative properties of the coarse and fine refuse materials at the site. In some cases, an exploration program at an existing similar facility might be undertaken for determining the desired properties. Typically, this zone of mixing does not play a significant role in the static analysis of downstream embankment slopes, but can be critical for seismic analysis and upstream slopes. The following guidance has been developed for deciding whether potential upstream failure surfaces are critical to the seismic stability and deformation of the embankment (MSHA, 2007):

- Potential upstream slope stability failure surfaces that terminate on the crest of the embankment stage should provide an acceptable factor of safety such that the integrity of the embankment and impounding capacity of the facility are maintained (i.e., if a portion of the embankment becomes unstable, a sufficient section of the crest will remain intact to prevent release from the impoundment).
- Potential deformation of the crest of the embankment should not result in the threat of a release from the impoundment (i.e., sufficient freeboard must be available to compensate for the maximum amount of crest settlement).

Another critical case where fine coal refuse characteristics are important is related to recovery or re-mining of fine coal refuse within an impoundment. This situation typically involves the development of an excavation plan with interim and final coal refuse slopes that must have acceptable factors of safety.

Additional conditions requiring slope stability analyses may arise due to situations in which the shear strength decreases or stress level in an embankment increases. Duncan and Wright (2005) cite the following causes for a decrease in shear strength:

- Increased pore pressure (reduced effective stress) due principally to a rise in groundwater level or increased seepage during periods of heavy rainfall
- Cracking near the crest of a slope due to tension and factors such as soil desiccation
- Swelling of highly plastic and heavily overconsolidated clays
- Development of slickensides due to shear in highly plastic clays resulting from shear on distinct slip planes
- Decomposition of rock in fills due to inadequate breakdown during compaction and weathering as a result of wetting and drying
- Creep of highly plastic clays under sustained load
- Leaching of chemical constituents in the soil matrix
- Strain softening of brittle soils leading to progressive failure
- Weathering of rocks and indurated soils due to physical, chemical and biological processes
- Cyclic loading and loss of strength due to liquefaction (Chapter 7)

Possible causes for an increase in shear stress include:

- Increased loads at the top of a slope
- Water pressure in tension cracks at the crest of a slope
- Increase in soil weight due to increase in moisture content

- Excavation at the bottom of a slope
- Rapid drawdown of an impoundment (upstream slope)
- Earthquake loading

Traditionally, slope stability has been evaluated using limit equilibrium analyses whereby the forces tending to decrease stability are compared to the forces tending to increase stability. These types of analyses are generally conducted using limit equilibrium slope stability computer programs. Since the early 1970s, finite element methods of analysis have improved to the point where realistic stability/deformation analysis of soil slopes is possible. The following section provides an overview of stability analyses for coal refuse disposal facility embankments.

6.6.4.2 Methods of Stability Analysis

6.6.4.2.1 Limit Equilibrium Stability Analysis

The stability of refuse embankments is usually solved by limit equilibrium methods of analysis. These analyses are conducted by calculating the minimum factor of safety (FS) for a slide surface through the slope as follows:

$$FS = \frac{\text{Available shear strength}}{\text{Equilibrium shear stress}} = \frac{s}{\tau} \tag{6-31}$$

If a large number of potential slide plane surfaces are assumed, the surface with the minimum factor of safety is a numerical representation of the relative safety of the slope. If $FS = 1$, a slope is in a state of "just-stable" limit equilibrium. Because of the uncertainty related to the geometry of the actual slide plane surface, the controlling soil properties, the pore-pressure distribution in the slope, and other factors that may affect the stability of a slope, slopes for water impounding embankments should be designed with FS equal to at least 1.5. A higher factor of safety should be used where factors that affect slope stability (e.g., limited testing has been performed) are less certain. The available shear strength (s) is defined in terms of the angle of friction (ϕ or ϕ') and cohesion (c or c') of the soil along the slide plane surface using soil properties determined from in-situ or laboratory tests.

Two approaches can be used to satisfy static equilibrium of a slope. The first and much less commonly used approach is to assume a single free-body bounded by the face of the slope and slide plane surface. Examples of this approach are the infinite slope, log spiral and Swedish slip circle methods. The second approach involves dividing the slope into a number of vertical slices that extend between the face of the slope and slide plane surface. Examples of this approach are the ordinary method of slices, simplified Bishop method and Spencer's method. Regardless of the approach used there are more unknowns (e.g., forces, location of forces, FS) than equilibrium equations, so the problem is statically indeterminate. Therefore, assumptions must be made to render the problem determinate. Examples of such assumptions include inclination of interslice forces, the location of the normal force at the base of a slice and the relationship of interslice shear force to the interslice normal force.

Some slope stability analysis methods are based solely upon force-equilibrium principles, while other methods involve satisfaction of all conditions of equilibrium. The characteristics of various equilibrium methods for slope stability analysis are summarized in Table 6.53. If force-equilibrium methods are used (Lowe and Karafiath, 1960; USACE, 2003), the factor of safety is affected significantly by the assumed inclinations of the side forces between slices. Thus, force-equilibrium procedures are not as accurate as methods that satisfy all conditions of equilibrium (Janbu, 1973; Spencer, 1967; Morgenstern and Price, 1965). The maximum range of results for methods that satisfy all conditions of equilibrium is generally less than 12 percent. Thus, with an average accuracy of about plus or minus 6 percent, a factor of safety calculated using procedures that satisfy all conditions of equilibrium can

TABLE 6.53 CHARACTERISTICS OF EQUILIBRIUM PROCEDURES FOR SLOPE STABILITY ANALYSIS

Procedure	Application
Infinite Slope	Homogeneous cohesionless slopes where stratigraphy restricts slip surface to shallow depths and parallel to slope face. Very accurate where applicable.
Logarithmic Spiral	Applicable to homogenous slopes; accurate. Potentially useful for developing slope stability charts and used in some software for design of reinforced slopes.
Swedish Circle	Applicable to slopes where ϕ = 0 and for relatively thick zones of weaker materials where slip surface can be approximated by a circle.
Ordinary Methods of Slices (Fellenius, 1922)	Applicable to non-homogeneous slopes and c - ϕ soils where slip surface can be approximated by a circle. Very convenient for hand calculations, but inaccurate for effective stress analyses with high pore pressures.
Simplified Bishop (Bishop, 1955)	Applicable to non-homogeneous slopes and c - ϕ soils where slip surface can be approximated by a circle. More accurate than OMS, especially for effecttive stress analyses with high pore pressures. Calculations can be performed by hand or spreadsheet.
Force Equilibrium Methods (Lowe and Karafiath, 1960; USACE, 2003)	Applicable to virtually all slope geometries and soil profiles. The only procedure suitable for hand calculations with non-circular slip surfaces. Less accurate than complete equilibrium procedures, and results are sensitive to assumed inclination for interslice forces.
Janbu Generalized Procedure of Slices (Janbu, 1973)	Satisfies all conditions of equilibrium. Applicable for any shape of slip surface. Numerical problems are encountered more frequently than with some other methods.
Spencer (Spencer, 1967)	Satisfies all conditions of equilibrium. Accurate procedure applicable to virtually all slope geometries and soil profiles. Simplest complete equilibrium procedure for computing factor of safety.
Morgenstern and Price (1965)	Satisfies all conditions of equilibrium. Accurate procedure applicable to virtually all slope geometries and soil profiles. Rigorous, well-established complete equilibrium procedure.
Chen and Morgenstern (1983)	Satisfies all conditions of equilibrium. An updated Morgenstern and Price procedure. Rigorous and accurate procedure applicable to any slip surface shape and slope geometry, loads and soil profiles.
Sarma (1973)	Satisfies all conditions of equilibrium. Accurate procedure applicable to virtually all slope geometries and soil profiles. Convenient complete equilibrium procedure for computing seismic coefficient required to produce a given factor of safety. Side force assumptions are difficult to implement for all but simple slopes.

(DUNCAN AND WRIGHT, 2005)

be considered to be acceptably accurate, because for practical purposes key parameters such as slope geometry, unit weight, shear strength, and pore-water pressure cannot be defined with an accuracy of plus or minus 6 percent. Therefore, any method that satisfies all conditions of equilibrium should be sufficiently accurate for impoundment design and analysis. Additional information relative to selection of a slope stability analysis method can be found in Duncan and Wright (1980).

For slopes composed of nearly homogeneous materials, both analysis and observation of actual failures indicate that the failure surface can be approximated with sufficient accuracy by a circular arc. For such cases, procedures that do not satisfy all conditions of equilibrium may be acceptably accurate. For non-homogeneous slopes or embankments, embankments supported on a foundation with a weak zone and impoundments lined with geomembranes or geosynthetic clay liners, limit-equilibrium procedures that are suitable for any shape slip surface and that satisfy all conditions of equilibrium must be used. Designers should be especially alert to the presence of a weak layer or layers upon which sliding may occur. In such cases, the factor of safety for wedge-type surfaces coincident with the weak layer must be evaluated in addition to the usual failure surfaces. If movement has already occurred in a zone of material that is included in a stability analysis, then residual shear strength may be applicable.

An implicit assumption in equilibrium analyses of slope stability is that the stress-strain behavior of the constituent material is ductile (i.e., it does not have a brittle stress-strain curve where the shearing resistance drops off after reaching a peak). This limitation results from the fact that limit-equilibrium methods provide no information regarding the magnitudes of the strains within a slope, nor any indication about how they may vary along the slip surface. Therefore, unless the strengths used in the analysis can be mobilized over a wide range of strains (i.e., the soil exhibits ductile stress-strain behavior) there is no guarantee that the peak strength can be mobilized simultaneously along the full length of the slip surface. Where multiple embankment or impoundment zones are traversed by the slip surface, strain compatibility for each material should be evaluated. For instance, coarse coal refuse typically mobilizes peak strength at a lower strain than fine coal refuse and cohesive foundation soils. Thus, the stability analysis should be based on strength at compatible strains, particularly if there is a drop-off in strength with large strains. If the shearing resistance of one material drops off after the peak is reached, progressive failure can occur, and the shearing resistance that can be mobilized at some parts of the failure surface may be smaller than the peak strength. For this situation, a reliable approach is to use the residual strength rather than the peak strength in the analysis.

For coal refuse, earth and rockfill embankments, the following critical embankment conditions should be evaluated:

1. High pore-water pressures are present in both the embankment and foundation. This condition occurs most often during or at the end of construction, particularly if construction is rapid, the slope materials have low hydraulic conductivity, and construction conditions are wet. For a coal refuse embankment, the rate of construction usually is not fast enough to cause high pore pressures in the foundation materials. An exception is when a thick layer of saturated clay underlies the embankment. For this case pore pressures during construction should be estimated, and piezometers should be installed to facilitate maintaining pore pressures within acceptable limits during construction. The rate of construction can also be an issue if an upstream construction pushout is constructed rapidly. Stability checks may be required both during construction and at the end of construction when an embankment is constructed over settled fine refuse using the upstream method.

2. Steady seepage has developed within the embankment and may have saturated a large part of the downstream slope. This condition occurs most often after long-term operation of an impounding embankment at full storage level, particularly if the slope materials had a high hydraulic conductivity. For compacted embankments, placement of refuse is usually slow enough that excess pore pressures will adequately dissipate. For situations where wet materials are placed in thick lifts, (e.g., filter cake or combined refuse in upstream embankment zones) excess pore pressures can develop, although generally the rate of construction is slow enough that the excess pore pressures will dissipate. Some slurry impoundments are designed to

store the runoff from the design storm and release it relatively slowly. In such cases, the water level may rise above the level of the slurry delta and be in direct contact with relatively permeable upstream slope material. If storm water is impounded against the upstream slope long enough for steady-state pore-water pressures to develop, this condition could represent a critical stability scenario.

3. The impoundment water level drops very quickly after steady seepage has developed within the embankment. This condition is generally referred to as rapid drawdown and can be critical to the stability of the upstream slope of the embankment. USBR (1987a) provides general guidance for water-detention and storage dams considering susceptibility of earth fill materials (based on USCS classification) to rapid drawdown loading (drawdown rate of 6 inches or more per day following prolonged storage at high reservoir levels). For most coal refuse embankments that impound slurry and runoff water, rapid drawdown is either not possible or is not an issue because the embankment material is generally free draining. For embankments constructed of low-hydraulic-conductivity material (less than 10^{-4} cm/sec) and designed to store storm runoff for subsequent release through a decant pipe, the potential effects of rapid drawdown should be considered. Another situation where rapid drawdown may need to be considered is during remining of an impoundment for recovery of additional coal from the fine coal refuse.

4. The embankment is subjected to earthquake loading during embankment construction, operation, or following abandonment. Issues related to the analysis and design of embankments that are subjected to earthquake loading are discussed in Chapter 7.

Analyses for the first three critical stability conditions listed above must reflect the rate of construction and pore-pressure conditions. The following analyses are typically employed:

- Total-Stress Analysis is used in situations where the pore-water pressure (u) that would act on the potential failure surface at failure is unknown and cannot be reliably estimated. The embankment stability is analyzed in terms of total stress (i.e., the stress between the individual soil grains plus the pore pressure). This method of stability analysis is generally considered most appropriate for evaluating relatively short-term loading conditions such as end-of-construction and rapid drawdown of the impoundment.

- Effective-Stress Analysis is used in cases where the pore-water pressure (u) that would act on the potential failure surface at failure is known or can be reliably estimated. The embankment stability is analyzed in terms of effective stress (i.e., the total stress minus the pore pressure). This method of stability analysis is generally considered most appropriate for evaluating long-term conditions after the transient effects related to construction and seepage have ended.

Selection of the appropriate conditions for analysis requires knowledge of soil behavior under drained and undrained conditions and evaluation of the conditions that will control drainage in the field. Shear strengths, water and pore pressures, and unit weights for slope-stability analyses are summarized in Table 6.54.

A useful guide for determining whether total- or effective-stress methods of analysis are applicable relates to whether the soils comprising the foundations and refuse embankment are free draining or impermeable. Free-draining soils are those able to drain completely during the construction or loading period. Impermeable soils are those that cannot drain completely during the construction or

TABLE 6.54 SHEAR STRENGTHS, WATER PRESSURES, AND UNIT WEIGHTS FOR SLOPE-STABILITY ANALYSES

Soil Type	Parameter	Condition		
		End-of-Construction	Multi-stage Loading[1]	Long-Term
All soils	External Water Pressures	Include	Include	Include
All soils	Unit weights	Total	Total	Total
Free-draining	Shear Strength	Effective stress c' and ϕ'	Effective stress c' and ϕ'	Effective stress c' and ϕ'
Free-draining	Internal Pore Pressures	u from steady-state seepage analyses	u from steady state seepage analyses	u from steady state seepage analyses
"Impermeable"	Shear Strength	Total stress c and ϕ from in-situ or UU or CU lab tests	Total stress c and ϕ from in-situ or UU or CU lab tests	Effective stress c' and ϕ'
"Impermeable"	Internal Pore Pressures	No internal pore pressures, set u = 0 in computer input	No internal pore pressures, set u = 0 in computer input	u from steady state seepage analyses

Note: 1. Multi-stage loading includes rapid drawdown, staged construction, and any other condition where a period of consolidation under one set of loads is followed by a change in load under undrained conditions.

(DUNCAN, 1996)

loading period. Duncan (1996) recommends using the dimensionless time factor T from consolidation theory to estimate the degree of drainage that can occur during construction or loading using the relationship:

$$T = c_v t / D^2 \quad (6\text{-}32)$$

where:

c_v = coefficient of consolidation (length^2/time)

t = time

D = drainage path (length)

If $T > 3$, the material can be treated as drained. If $T < 0.01$, the material can be treated as undrained. If $0.01 < T < 3$, both drained and undrained conditions should be evaluated. Duncan (1996) suggests that, if the data needed to calculate T are not available, soils with hydraulic conductivity $k > 100$ feet/year can be considered drained and soils with $k < 0.1$ foot/year can be considered undrained.

Undrained conditions should be analyzed in terms of total stress in order to avoid having to rely on estimated, and sometimes unreliable, pore pressures for undrained loading conditions. Undrained strength can be determined using in-situ tests (e.g., vane shear), UU triaxial tests, or CU tests in conjunction with a strength normalizing procedure such as the SHANSEP (stress history and normalized soil engineering parameters) procedure (Ladd and Foott, 1994). For multi-stage construction (e.g., upstream pushouts), the undrained strength can be estimated using CU triaxial test results together with values of consolidation pressure determined from consolidation analyses (Ladd, 1991).

Drained conditions can be analyzed in terms of effective stresses using c' and ϕ' from drained triaxial or direct shear tests or from $\overline{\text{CU}}$ tests. Direct-shear or $\overline{\text{CU}}$ tests are more often used when testing clays,

TABLE 6.55 GUIDELINE FRICTION VALUES AND EFFICIENCIES FOR VARIOUS GEOSYNTHETIC AND SOIL COMBINATIONS

Soil-to-Geomembrane Friction Angle

Geomembrane	Soil Types		
	Concrete Sand (ϕ = 30°)	Ottawa Sand (ϕ = 28°)	Mica Schist Sand (ϕ = 26°)
PVC			
Rough	27° (0.88)[1]		25° (0.96)
Smooth	25° (0.81)		21° (0.79)
CSPE	25° (0.81)	21° (0.72)	23° (0.87)
HDPE	18° (0.56)	18° (0.61)	17° (0.63)

Geomembrane-to-Geotextile Friction Angle

Geotextile	Geomembrane			
	PVC		CSPE	HDPE
	Rough	Smooth		
Nonwoven, Needle-Punched	23°	21°	15°	8°
Nonwoven, Melt-Bonded	20°	18°	21°	11°
Woven, Monofilament	11°	10°	9°	6°
Woven, Slit Film	28°	24°	13°	10°

Soil-to-Geotextile Friction Angle

Geotextile	Soil Types		
	Concrete Sand (ϕ = 30°)	Ottawa Sand (ϕ = 28°)	Mica Schist Sand (ϕ = 26°)
Nonwoven, Needle-Punched	30° (1.00)	26° (0.92)	25° (0.96)
Nonwoven, Melt-Bonded	26° (0.84)	–	–
Woven, Monofilament	26° (0.84)	–	–
Woven, Slit Film	24° (0.77)	24° (0.84)	23° (0.87)

Note: 1. Efficiency values in parentheses are based on the relationship $E = \tan\delta / \tan\phi$

(ADAPTED FROM KOERNER, 2006)

because the time required for testing is shorter than for conducting CD tests. Values of c' and ϕ' from $\overline{\text{CU}}$ tests have been found to be nearly identical to values obtained from drained tests. Values of ϕ' for natural deposits of cohesionless soils are usually estimated using correlations with SPT or CPT data.

In performing embankment stability analyses, the strength behavior of coal refuse must be carefully evaluated. There is evidence that the failure strength envelope of coarse refuse has considerable curvature in the stress range associated with high embankments due to crushing of the refuse particles. Thus, the cohesion and friction angle of the coal refuse may vary depending upon location in the embankment. Application of a bi-linear failure model may be appropriate for such cases.

For sites where impoundments are lined with geomembranes or geosynthetic clay liners (GCLs) to control seepage, the potential for slope failure between the liner and subgrade and between the liner and soil cover placed over the liner may need to be evaluated. Table 6.55 provides guideline friction angles that may be appropriate for preliminary design for various interfaces. However, final design should be based on more refined published data and manufacturers' information for the specific geosynthetic materials under consideration, as well as laboratory interface testing between on-site soils and selected geosynthetic materials to simulate site-specific conditions.

When GCLs are used on slopes, their friction properties are important. Sodium bentonite, which is often used in GCLs, is a clay with a saturated, drained residual internal angle of friction of approximately 6° to 9°. However, significantly greater friction angles may be appropriate in GCLs that are needle-punched or stitched. Manufacturers should be consulted for design data, as these data may be product specific.

An important part of slope stability analysis is determining the slip surface with the lowest *FS*. Most computer programs that use an assumed circular failure surface systematically change the position of the center of the circle and the length of the radius to find the most critical (lowest *FS*) circle. For more complex geometries typical of most real-world situations, local minima may exist. Therefore, multiple searches should be conducted using multiple starting points and search strategies so that the overall minimum value of FS is determined. The results of slope stability analyses should be carefully examined to verify that the upstream and downstream limits of the search are not so restrictive as to exclude potentially critical failure surfaces. Locating a critical noncircular surface is more complex, and a variety of approaches have been developed. Methods such as random generation of kinematically admissible slip surfaces, coupled dynamic programming minimization techniques, and optimization have been used successfully to model slopes that do not have extremely complicated geometries. Regardless of the computational procedure used, tests of reasonableness should be applied to the results, and multiple searches should be performed to be certain that the critical slip surface has been located. Failure surfaces through weaker embankment or foundation layers should always be considered.

Most slope stability problems can be modeled in two dimensions because the geometry of a slope is typically relatively constant along its length. However, some slopes are: (1) curved in plan or contain corners (e.g., some diked embankments), (2) subjected to loads of limited extent at the top, or (3) constrained by physical boundaries such as a dam in a narrow-walled valley. For these situations, consideration may be given to conducting a three-dimensional (3D) limit-equilibrium analysis. Duncan (1996) reports that the factor of safety for 3D analysis is greater than the factor of safety for 2D analysis (i.e., $FS_{3D} > FS_{2D}$) provided that: (1) FS_{2D} is calculated for the most critical two-dimensional cross section of the slope and (2) the procedure used for 2D limit-equilibrium analysis satisfies all conditions of force and moment equilibrium. If 2D analyses of refuse embankment stability meet these criteria, a 3D analysis is not generally warranted.

Some limit-equilibrium computer programs include features that permit a probabilistic analysis of slope stability. Some common characteristics of these programs include:

- Simulation techniques (e.g., Monte Carlo) that allow the program to repeatedly sample values from probability distributions of the uncertain variables
- Modeling of input parameters as random variables (e.g., material properties, phreatic surface location, seismic load coefficient)
- Defining the probability density function of the random variables in terms of statistical distributions commonly used in geotechnics (e.g., normal, exponential, lognormal)

- Using truncated distributions to define maximum and/or minimum values
- Defining correlation coefficients between correlated values (e.g., c and ϕ)
- Presentation of results in a variety of forms (e.g., histograms, cumulative plots, scatter plots)

Thus, probabilistic slope stability analysis accounts for variability and uncertainty associated with traditional limit-equilibrium methods. Figure 6.55 illustrates the results of a stability analysis of a cohesive soil slope using Spencer's method. The figure shows a histogram of frequency distribution of the *FS* computed for 5,000 random analyses. The red bars at the left of the distribution represent analyses where *FS* is less than 1. From the histogram, the mean *FS* = 1.072, and the maximum and minimum *FS* are 1.298 and 0.860, respectively. However, Figure 6.55 illustrates another advantage of probabilistic analysis. The ratio of the 658 analyses where *FS* is less than 1 to the 5,000 total analyses is the probability of failure p_f. For this analysis, p_f = 13.2 percent, which provides a measure of the potential for failure separate from the factor of safety. For this case, the analysis shows that a low average *FS* results in a high probability of failure.

Note that no value of p_f is recommended for design of embankment dams at the time of publication of this Manual, although proposed risk evaluation criteria and guidelines for significant- and high-hazard-potential dams are available (Von Thun, 1996). El-Ramly et al. (2003) present a probabilistic stability analysis of a tailings dike along with a general spread sheet model, and they note that computed values of pf for existing dams demonstrating satisfactory performance may not meet recommended values. However, a probabilistic analysis is useful in understanding the contribution of parameters affecting stability and in comparing conditions and configurations for establishing reliable design. Probabilistic acceptance criteria will become established as more analyses are conducted and results published for both failed and satisfactorily performing slopes.

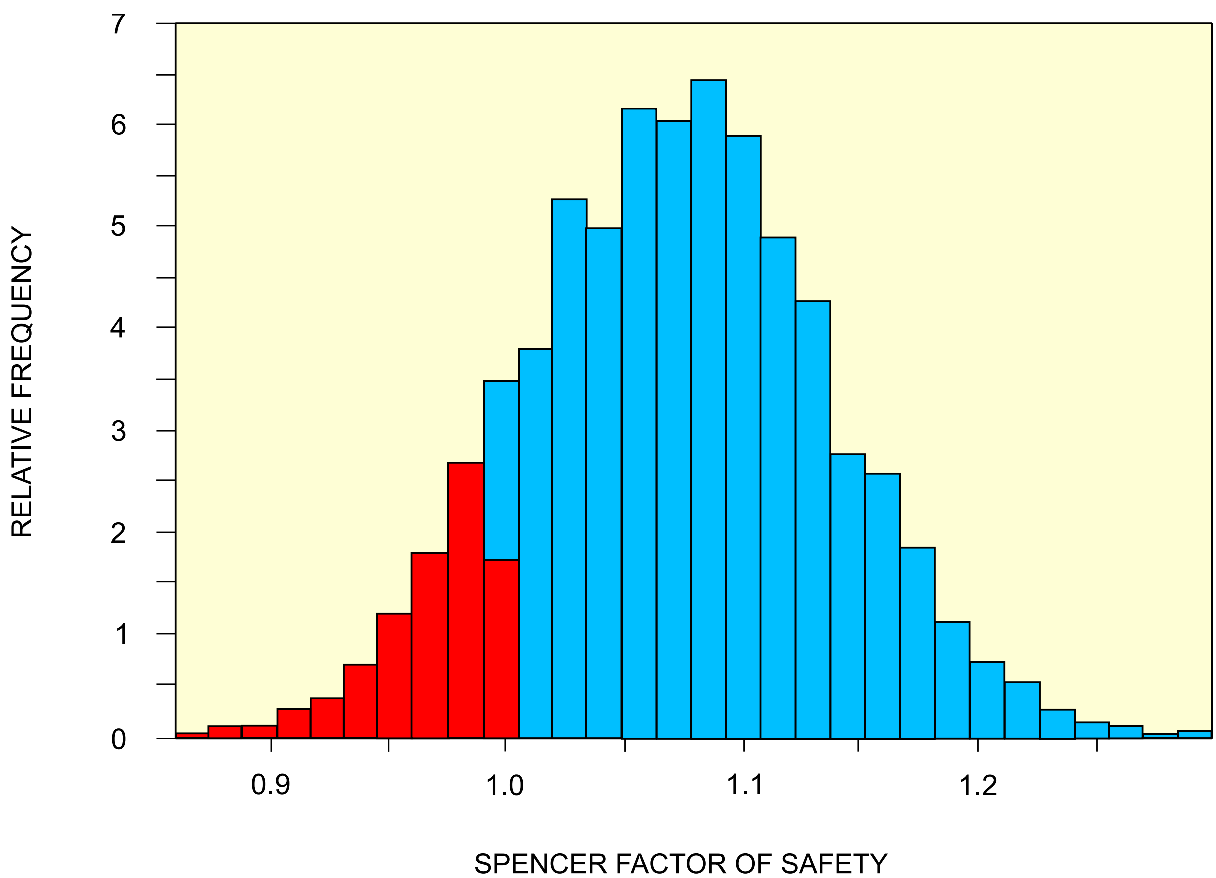


FIGURE 6.55 HISTOGRAM OF FACTOR OF SAFETY DISTRIBUTION

Additional information relative to probabilistic methods for the stability analysis of slopes is provided in Hoek (2000) and Baecher and Christian (2003).

6.6.4.2.2 Stability/Deformation Analysis Using Finite Element Methods

Slope stability analyses are traditionally performed using classical techniques such as the method of slices. These approaches are based on the assumption of rigid-plastic stress-strain behavior with no deformation occurring prior to failure. However, if elastic deformations occur prior to failure, the finite element method can be used to solve the elastic differential equations, with failure defined by placing a limit on stresses using the Mohr-Coulomb (typical) or other failure criterion. The stress-strain behavior of the embankment material should be modeled using the simplest representation possible that is appropriate for the problem analyzed (Duncan, 1996). However, while simple linear elastic, multi-linear elastic or hyperbolic models may be appropriate for analyzing stress states well prior to failure, more complex stress-strain models (e.g., elasto-plastic and elasto-viscoplastic) are required to analyze slope behavior near failure.

The elasto-plastic and elasto-viscoplastic FE approach to slope stability analysis offers the following advantages over traditional methods (Griffiths and Lane, 1999):

- No a priori assumptions are needed relative to the shape or location of the failure surface. Failure occurs "naturally" through zones in the soil mass where the shear strength is unable to support the gravity-induced shear stresses.
- Because there is no concept of slices in the finite element approach, there is no need for assumptions about slice side forces, and the finite element method preserves global and local equilibrium until failure is reached.
- The finite element method can indicate progressive failure up to and including overall shear failure.

Slope stability analysis using the finite element method requires the following steps:

- Gravity loads are applied to the slope.
- An elastic analysis is performed to compute stresses.
- Stresses in each element are compared with the Mohr-Coulomb failure criterion.
- If stresses exceed the Mohr-Coulomb criterion, they are redistributed to neighboring elements that still have reserve strength.
- Slope failure occurs if stress redistribution cannot be accomplished to satisfy the Mohr-Coulomb criterion and global equilibrium. Failure is indicated by significantly increased nodal displacements.

Finite element analyses are not commonly used for the design of refuse embankments, but have application for the analysis of embankments where unexpectedly large deformations are observed or where unusually soft foundations may lead to unacceptable plastic deformations or differential settlements. Additionally, finite element analyses may prove to be a useful tool for evaluation of the effects of deformation on embankment stress and for the evaluation of mine subsidence effects.

6.6.4.3 Acceptable Factors of Safety

Selection of an acceptable factor of safety for the analysis or design of an embankment slope depends on the degree of uncertainty in calculating *FS* and the hazards or consequences should a slope fail. The calculation of the factor of safety for a coal refuse disposal facility embankment involves evaluation of many factors, including:

- Uncertainties about the type and extent of sampling and testing to determine material strengths
- The variation of materials in the embankment and impoundment
- Uncertainties of embankment geometry
- The type of embankment (e.g., new or existing, impounding or non-impounding, constructed by upstream method or downstream method)
- The level of uncertainty concerning the location of the phreatic level and related excess pore-water pressures
- Embankment location (e.g., in a V-shaped valley, high seismic region)
- The accuracy of the stability analysis method(s) used
- Potential for future loads
- Potential for future changes in disposal operations and practices

The hazards or consequences associated with refuse embankment failure include:

- Potential for loss of life and/or property damage on site
- Potential for loss of life and/or property damage off site
- Economic cost to mining operations if disposal operations are lost
- Economic cost associated with restoring or safely abandoning disposal operations
- Economic cost of restoring environmental damage

The magnitude of the factor of safety to be used for coal refuse disposal facility design should be determined by the designer considering state and federal regulatory requirements and the completeness and accuracy of designer's knowledge of the uncertainties and conditions identified previously. If these uncertainties and conditions are well defined, or if conservative assumptions have been made, a lower *FS* may be acceptable. Conversely, when many assumptions are made relative to forces and material strengths, a higher *FS* is appropriate.

In selecting an acceptable *FS* for the stability of embankment slopes for coal refuse disposal facility design, the practices of major engineering organizations, both governmental and private, should be considered. Most earth dams in the U.S. are designed based upon extensive laboratory test information and clearly identified loads and geometry. For these structures, a $FS \geq 1.5$ is generally considered to be acceptable for permanent or sustained loading conditions. For special circumstances, such as soft embankment foundations, a higher *FS* is sometimes adopted to limit foundation or embankment deformations. For temporary loading conditions during construction, a lower *FS* is often acceptable. For transient loads, such as earthquakes, a *FS* of 1.2 is generally acceptable depending on the design earthquake magnitude, as discussed in Chapter 7. Minimizing the volume of material used for construction is not usually a design goal for refuse embankments as it is for earthen dams; thus, it may be possible to achieve somewhat higher factors of safety for refuse embankments without introducing special materials or operations, and the designer may be able to accommodate concerns about uncertainty of engineering properties or loads.

Table 6.56 presents recommended minimum factors of safety for the design of coal refuse embankments based on values adopted by the cited federal agencies (USSD, 2007). These values are generally consistent with factors of safety cited in the MSHA Impoundment Inspection and Plan Review Handbook (MSHA, 2007) and have been adopted by many state regulatory agencies.

The values of *FS* provided in Table 6.56 apply if the following conditions are met:

TABLE 6.56 RECOMMENDED MINIMUM FACTORS OF SAFETY FOR DESIGN OF COAL REFUSE EMBANKMENTS

Condition	Design Basis	Factor of Safety	Source[1]
Long-term stability analysis with maximum storage pool	Design based on shear strength measured in laboratory and/or field testing program reflecting long-term site development with steady-state seepage and maximum storage (operational) pool.	1.5	USACE, USBR, NRCS, FERC
Long-term stability analysis with maximum surcharge pool	Design based on shear strength measured in laboratory and/or field testing program reflecting long-term site development with steady-state seepage and maximum surcharge (design storm) pool.	1.4	USACE, FERC
Intermediate-stage static analysis with maximum storage pool	Design based on shear strength measured in laboratory and/or field testing program reflecting intermediate-stage critical configurations using long-term analysis with steady-state seepage and maximum storage (operational) pool.	1.5	
Intermediate-stage static analysis with maximum surcharge pool	Design based on shear strength measured in laboratory and/or field testing program reflecting intermediate-stage critical configurations using long-term analysis with steady-state seepage and maximum surcharge (design storm) pool.	1.4	
Short-term, end-of-construction, undrained static analysis	Design based on intermediate- or long-term configurations and supported by shear strength measured in laboratory and/or field testing program using undrained strength parameters, as appropriate.	1.3	USACE, USBR, FERC, TVA
Rapid-drawdown analysis with maximum storage pool	Design based on intermediate or long-term configurations and supported by shear strength measured in laboratory and/or field testing program using drained or undrained analysis.	1.3	USACE, USBR
Seismic analysis	Discussed in Chapter 7.	–	

Note: 1. From USSD (2007).

- The critical failure surface has been determined from stability analyses based on systematic searches and evaluation of defined planes of weakness.
- Stability analysis parameters are, with reasonable certainty, known to be representative of the actual conditions that will exist in the embankment.
- Sufficient control will be provided during construction to verify that materials placed within the embankment conform to the standards assumed or required by the disposal facility design.
- When pore-water pressure in an embankment and its foundation is a significant factor in the stability of the embankment, piezometers are installed and monitored, and the observed data are compared to design assumptions.

If an existing coal refuse disposal embankment is found to have a low *FS*, it may not be possible to modify the site sufficiently within a short time period to satisfy minimum *FS* criteria. In such cases, a slight, temporary reduction in the *FS* may be acceptable provided that:

- Monitoring of pore pressures in the embankment and movements of the embankment surface is performed on a scheduled basis.

- A plan to improve the *FS* is developed and implemented (e.g., the impoundment level is lowered, the embankment upstream face is sealed to reduce seepage, pore pressures are measured, or other steps to reduce the impounding capacity or to implement abandonment are initiated).

When test data for embankment and foundation materials are limited or available test data are inconsistent, either conservative values of shear strength and pore-water pressure should be used in the stability analysis or an increased *FS* should be used in the embankment design.

6.6.4.4 Stability Control Measures

The most effective measures for stabilizing coal refuse embankments and other slopes are: (1) drainage control and (2) buttress fills. Drainage control is probably the most frequently used and often the most effective measure because slope failures are very often the result of increases in groundwater level, phreatic surface level or pore pressures. Also, drainage control is often the least expensive and most easily implemented of the options that are typically available. The types of drainage control measures (Section 6.6.2.3) that can be employed include:

- Surface control using ditches designed with a gradient and lining that limits infiltration into the slope and conveys surface flows to a drainage outlet.
- Lowering of the impoundment pool level, if feasible, and consideration of partial liner systems placed on the upstream slope to limit seepage.
- Occasional movement of the slurry discharge point so that fines are distributed along the upstream embankment slope, limiting seepage and pushing the free water further away from the embankment.
- Horizontal drains drilled at an upslope gradient into the downstream face of the embankment to intercept water up to 400 feet from the face. As discussed in Section 6.6.3.3, prefabricated drains have been used to control pore pressures and improve consolidation of impoundment fines.
- Relief wells to intercept artesian water pressure at the downstream toe of the embankment. The wells may be furnished with pumps to convey flow to a drainage outlet.
- Trench drains extending below the toe of the downstream embankment slope or into a bench on the slope. Flow is conveyed by gravity and typically in pipes to a drainage outlet.
- Finger drains excavated perpendicular to and typically at a shallow depth into the downstream face of the embankment to intercept water flow at a shallow depth.
- Blanket drains placed at the surface of downstream embankment slopes where seepage or piping is occurring. Blanket drains, especially when incorporated into a buttress, also add weight to help increase the effective stress.

Structural buttress and gravity berm fills constructed at the toe of an embankment also serve to increase stability. A structural buttress fill constructed with well-compacted, high-strength material improves stability by providing both strength and weight. A gravity berm fill using uncompacted material improves stability by providing weight to reduce shear stresses at the toe of the embankment slope. The effectiveness of either type fill can be improved by placing a layer of free-draining material between the embankment face and the fill to convey water draining from the face of the embankment slope to a drainage outlet.

Other types of stabilization measures such as structural and ground improvement options may be considered, but these choices typically require greater time to implement and are more expensive and less effective than drainage or toe buttress stabilization measures.

6.6.4.5 Stability Measurements

Parameters that influence the slope stability factors of safety should be monitored during and after each stage of construction by instrumentation to provide a basis for checking the design factor of safety so that modifications to the design or to construction procedures can be made, if needed. Depending on site conditions and project requirements, embankment performance monitoring instrumentation may include:

- Surface monuments for monitoring vertical and horizontal displacements
- Inclinometers for monitoring lateral displacements at depth
- Piezometers for monitoring piezometric head

In addition, monitoring of density test results is important to stability because it provides an indirect method to monitor the strength of the compacted fill. Thus, the minimum compaction requirement serves to ensure that the shear strength of the compacted fill is consistent with the strengths obtained from laboratory testing and used in the stability analysis.

An extensive discussion of instrumentation for coal refuse disposal facilities is provided in Chapter 13.

6.6.5 Rock Excavations

Excavations into rock at coal refuse disposal facilities are usually associated with: (1) spillway channels, (2) diversion ditches, and (3) haul roads. Less frequently, rock excavations are associated with decant structure installation and obtaining borrow material.

The degree of assurance that an excavated rock slope will remain stable and has a sufficient factor of safety for each of these purposes will depend on facility design requirements. This section provides a discussion of the stability of rock excavations, methods for minimizing the potential for a failure, and procedures for improving stability.

6.6.5.1 Conditions Requiring Stability Analysis

Major factors that must be considered when evaluating the stability of rock excavations include:

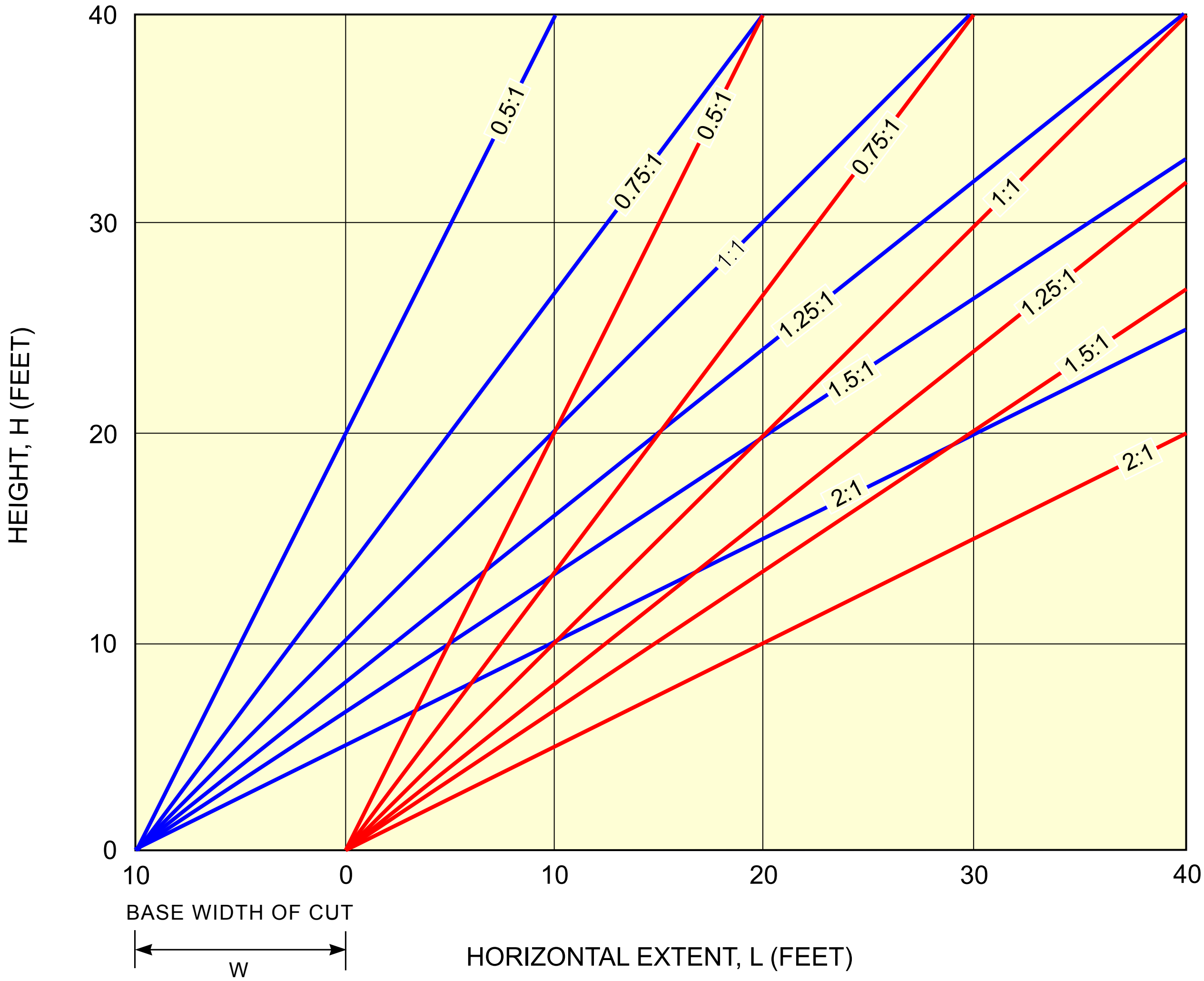
- Existing ground surface slope
- Overburden soil thickness, type and quality
- Bedrock surface conditions
- Rock type, quality and configuration (particularly the orientation of discontinuities)
- Groundwater conditions

6.6.5.1.1 Existing Ground Surface Slope

The steepness of the existing ground surface is an important factor in both the cost of excavation and the resulting stability. Figure 6.56 provides a guide for estimating both the vertical and horizontal extent of rock excavations as a function of existing slope, cut width, and cut slope. In steep areas where existing slopes are only marginally stable, deep excavations are very expensive because of the volume of material to be excavated and protection requirements.

6.6.5.1.2 Overburden Soil Thickness, Type and Quality

Usually the top portion of an excavation into rock extends through overlying soils. The need for a shallower slope and erosion protection in the overburden soil portion of the excavation must be considered in evaluating the limits of excavation and related costs.



LEGEND

EXISTING SLOPE (S_E)

SLOPE OF CUT (S_C)

(1) $S_E > S_C$

(2) $L = W \dfrac{S_C}{S_E - S_C}$

(3) $H = \dfrac{L}{S_C}$

FOR W ≠ 10 FEET, MULTIPLY THE VALUES OF L AND H OBTAINED FROM THE GRAPH BY COEFFICIENT C = W/10.

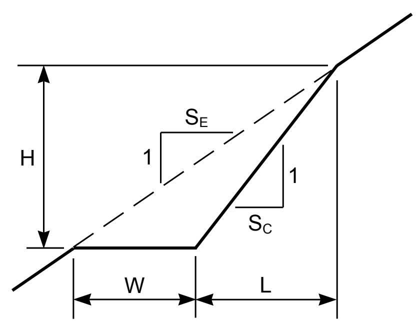


FIGURE 6.56 HORIZONTAL AND VERTICAL EXTENT OF CUTS FOR BASE WIDTH OF 10 FEET

6.6.5.1.3 Bedrock Surface Conditions

As shown in Figure 6.57, the bedrock surface can be difficult to define because of weathering, downhill slope creep and loosening from stress relief. Except where rock is very massive, conditions normally change with depth from: (1) overburden soil consisting of colluvial and residual material to (2) soft, heavily weathered rock and then to (3) sound and competent rock. Heavily weathered, decomposed rock should be treated as soil to a depth determined by an experienced geologist or engineer. Particular caution is required for soft rocks (e.g., shale, siltstone, claystone and mudstone) that weather very rapidly when exposed to air, rainfall and freezing conditions.

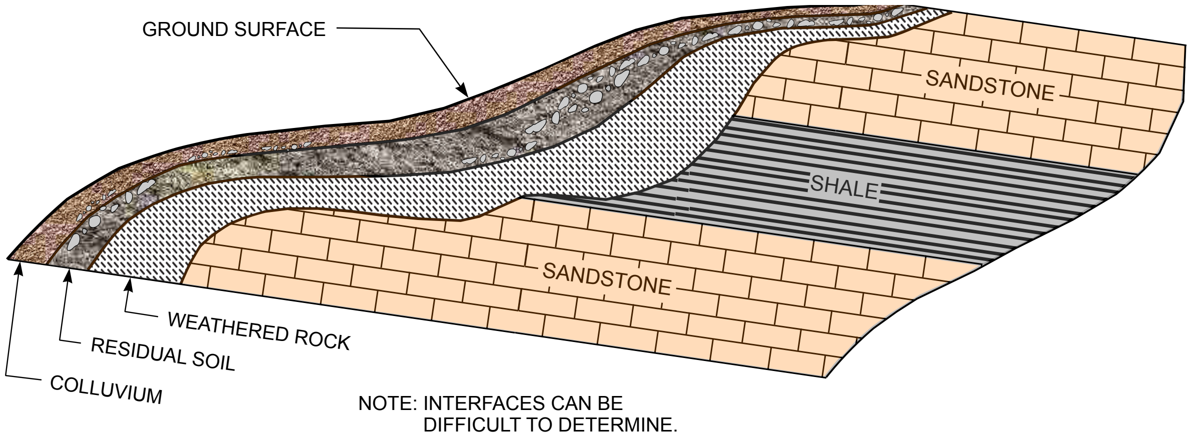


FIGURE 6.57 SCHEMATIC GEOLOGICAL PROFILE OF A BEDROCK SLOPE

6.6.5.1.4 Rock Type, Quality and Configuration

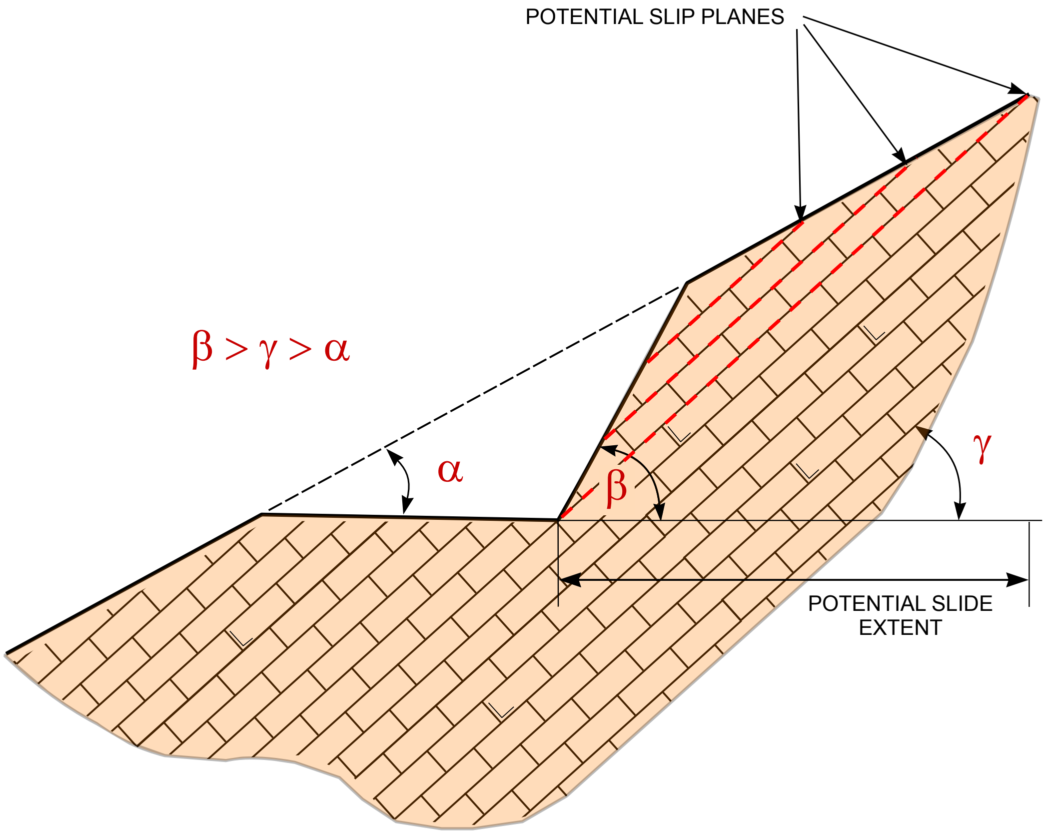
Generally the design of a cut into competent rock is technically simple and requires only application of the designer's judgment or general experience. However, when rock conditions are such that analyses are required for demonstrating adequate stability, detailed evaluation is frequently more complex than the evaluation of the stability of a soil slope. A rock mass may consist of: (1) a layered system of individual beds of different type rocks with variable bed thickness and strength and (2) discontinuities such as bedding planes, joints, fractures and faults that have much lower strengths than the intact rock. These discontinuities control rock slope stability, but can be difficult to define for purposes of analysis. The types and characteristics of rock most often found in coal mining areas are:

- Soft Rocks – Shale, Siltstone, Claystone and Mudstone – Rapid weathering of these types of rocks affects their strength, which eventually decreases to that of soil. Where these rocks are exposed, it must be expected that weathering will occur during the useful life of the facility. When exposed soft rock underlies massive, more competent rock, particular care must be taken to avoid instability as the soft rock weathers and provides less support. Also, erosion of soft rocks can be a problem, particularly when they are thinly-bedded and exposed to surface water flows for long periods.

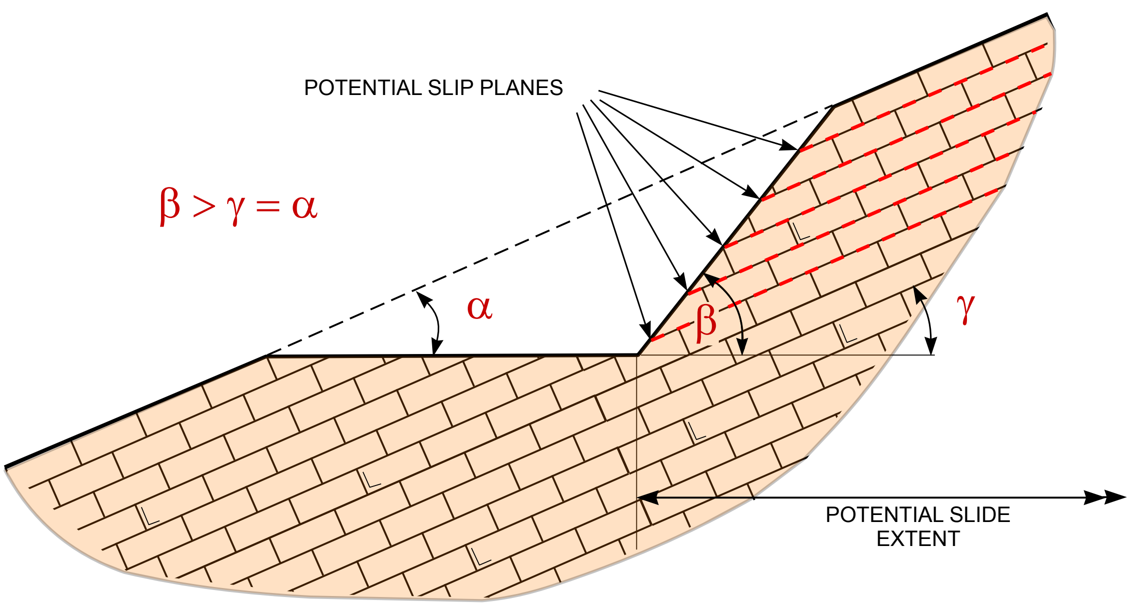
  Excavation of soft rocks can usually be accomplished with dozers, shovels, scrapers or backhoes. Even when competent, these rocks can be ripped without blasting using a bulldozer.

- Harder Rocks – Limestone and Sandstone – These types of rocks are more resistant to weathering. Strength is usually high if defects such as fractures or interbedded units of softer rocks do not occur at critical locations. Defects and weak cementation can also adversely affect the erosion resistance of these rocks, which is otherwise good to excellent.

  The exploration and evaluation of rock defects is very important. The characteristics of rock defects reported as part of an exploration program generally include type, quality, thickness, strike and dip, continuity, extent, frequency, and relative strength. The orientation of rock defects relative to the excavation is critical. For example, Figure 6.58 illustrates two conditions where orientation of bedding planes dipping into a cut and "daylighted" by the excavation could cause bedrock sliding along these planes. The extent of a potential slide could be small (Figure 6.58a) or large (Figure 6.58b) depend-



6.58a EXTENT OF POTENTIAL SLIDE (SMALL)



6.58b EXTENT OF POTENTIAL SLIDE (SEVERE)

FIGURE 6.58 UNFAVORABLE ORIENTATION OF BEDDING PLANES ing on the interrelationship of the dip angles of the existing slope, cut and bedding planes. Figure 6.59 shows three cases of favorable bedding plane orientation. Even with a favorable bedding plane orientation there may be potential for failure if the rock is highly fractured (Figure 6.59b) or if a capable fault is present (Figure 6.59c).

Thinly layered strata of limestone and sandstone are usually rippable and are not extremely difficult to excavate. However, thick layers of competent rock will require blasting.

Because sedimentary strata are layered, harder more competent rock may overlie softer, less competent rock. Weathering of the softer rock will occur faster than weathering of the harder rock and can lead to overhangs of the more competent and more massive harder strata. For permanent excavations, slope conditions should be observed over time, and large and potentially dangerous overhangs should be eliminated by occasionally scaling the slope to make it more uniform in geometry. If loose rock should develop, it should be removed or the potential rockfall zone should be barricaded.

6.6.5.1.5 Groundwater Conditions

It is important to recognize that the flow of groundwater through a rock formation can cause distress. Excessive pressure along bedding planes or in fractures, can contribute to rock movements. Also, flowing water can cause erosion of defects either chemically from solutioning or mechanically from fines washing from joints and fractures. Both types of erosion can weaken the strength of a rock mass. Finally, freezing water in fissures and cracks can widen existing fractures and accelerate the weathering process or can act as a dam to create higher water pressures behind the face of the slope.

6.6.5.1.6 Other Factors

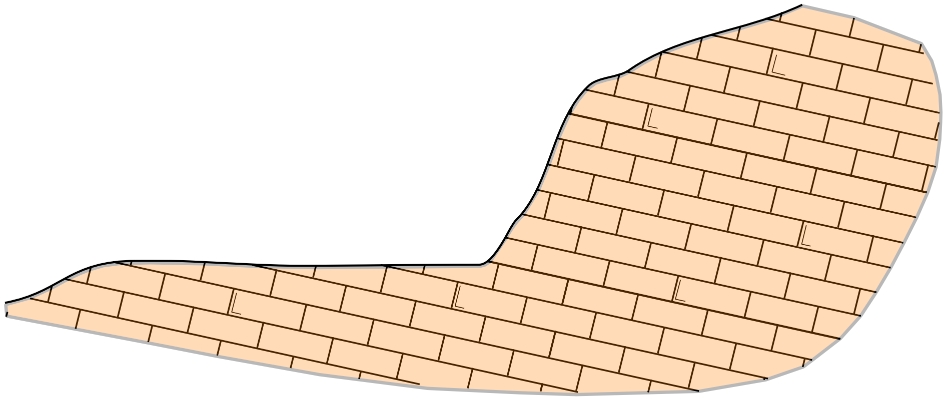
Other factors that can affect the stability of rock cuts include:

- Erosion from precipitation associated with large storms or long wet periods
- Infiltration associated with precipitation
- Vegetation growth
- Shocks and vibrations from blasting or earthquakes
- Changes in loading conditions

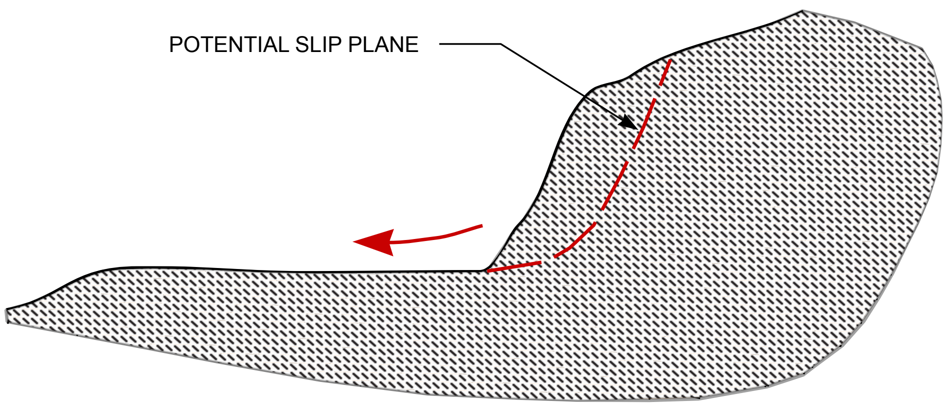
6.6.5.2 Slope Stability Analysis

The stability of cuts in rock can be analyzed using two basic approaches. Analyses similar to those used for soil slopes (Section 6.6.4) can be used if the rock mass can be assumed to act as a homogenous medium. This condition could apply when the rock strength is not governed by particular planes of weakness. In most cases, however, discontinuities govern rock behavior, and principles of rock mechanics can be used for stability analyses, as discussed by Wyllie and Mah (2004), Hoek (2000), and Coates and Yu (1977).

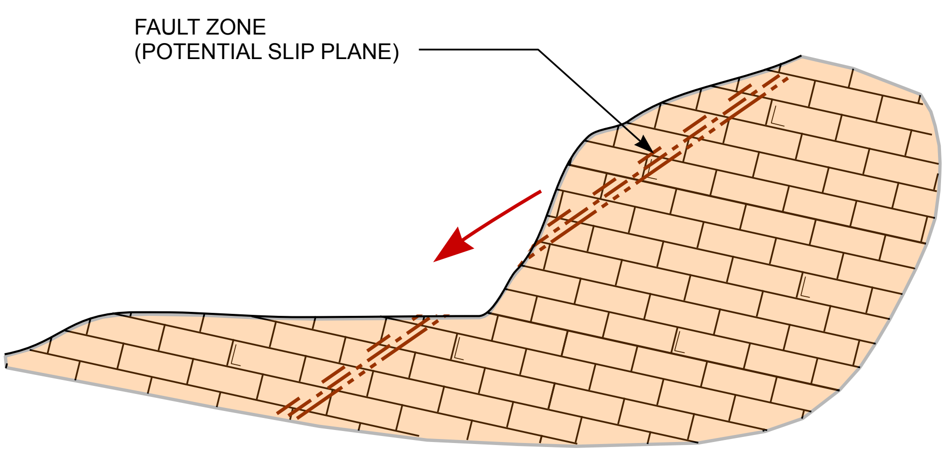
Most small rock excavations, where failure would not represent a severe hazard, can be designed based on experience with similar geological conditions. The slopes for these conditions usually range from 2 horizontal to 1 vertical (2H:1V) to 0.5H:1V for soft rocks and from 1H:1V to nearly vertical for hard rocks unless the orientation of weakness planes require flatter slopes. The uniformity of a rock slope is often broken by benches or berms for controlling runoff and catching loose rock fragments and debris.



6.59a ROCK IS COMPETENT



6.59b ROCK IS HIGHLY FRACTURED



6.59c FAULT ZONE IS GOVERNING FEATURE

FIGURE 6.59 FAVORABLE ORIENTATION OF BEDDING PLANES

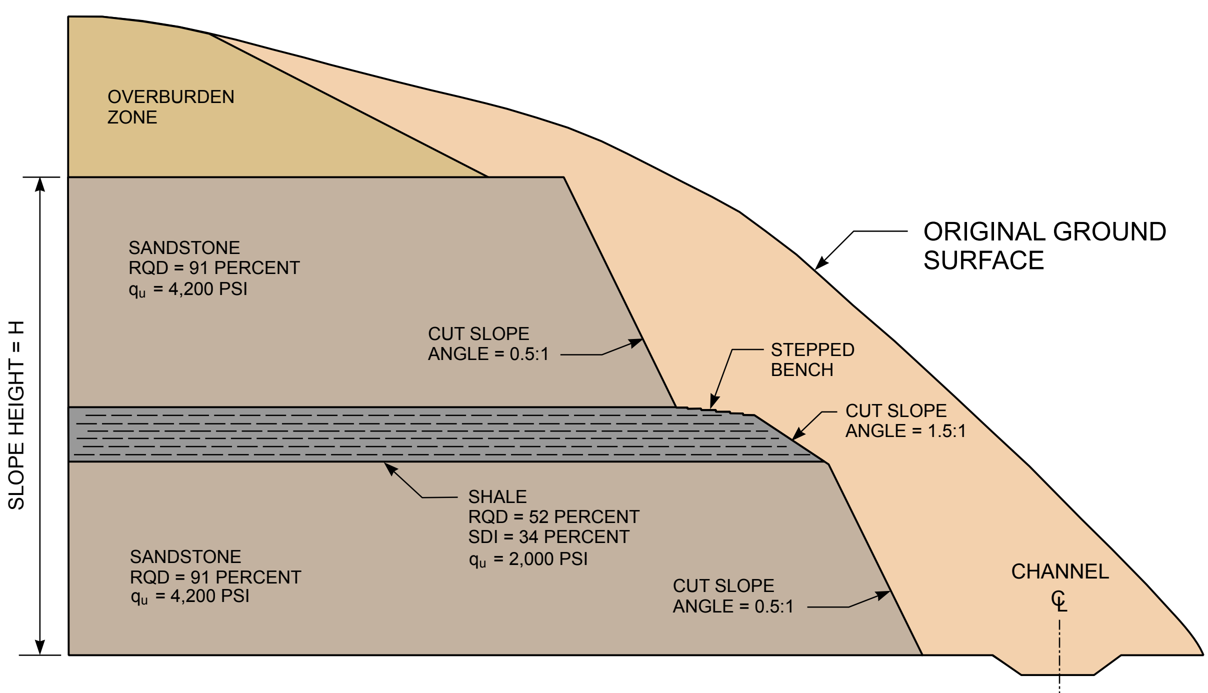
Guidance for rock cut slopes above roadways is presented by ODOT (2006), based on the slake durability index (SDI), rock quality designation (RQD) values, and unconfined compressive strength. Construction bench and debris width guidance is also provided.

6.6.5.3 Stability Control Measures

An approach to stability control developed by ODOT (2006) is illustrated in Figure 6.60. The advantage of this technique is that ample horizontal area is provided for debris collection and vegetation growth after construction. The edges of individual steps gradually break down and debris is collected at the benches so that vegetation growth helps to protect against sloughing and erosion. The individual benches and steps do not have to be spaced equally or be level. To minimize the cost, the steps should be cut during initial excavation.

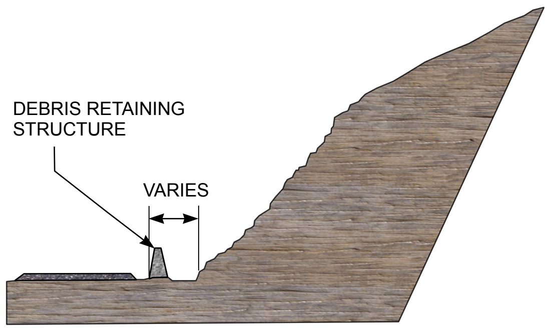
Several remedial alternatives are available when difficulties with the stability of rock excavations are anticipated or encountered. Benching of the rock slope can be a simple means of improving stability. Other methods, as shown in Figure 6.61, include:

- Construction of a bench with optional retaining berm or wall at the toe of slope to catch falling rock debris.
- Construction of a retaining wall against all or a portion of the slope.
- Prevention of rock weathering and sloughing by anchoring a wire mesh (with optional shotcrete overlay to protect against weathering) to the rock slope.
- Containing loose rock on the cut face with rock bolts, possibly combined with mesh.
- Supporting the rock face with tiebacks anchored below potential slide planes.
- Placing concrete or masonry support to replace weathered soft rock supporting overlying layers of harder rock.

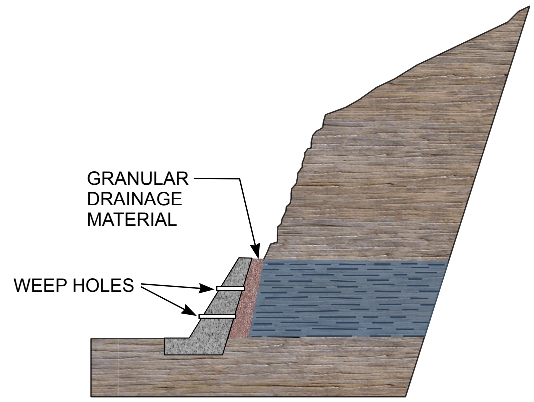


(ADAPTED FROM ODOT, 2006)

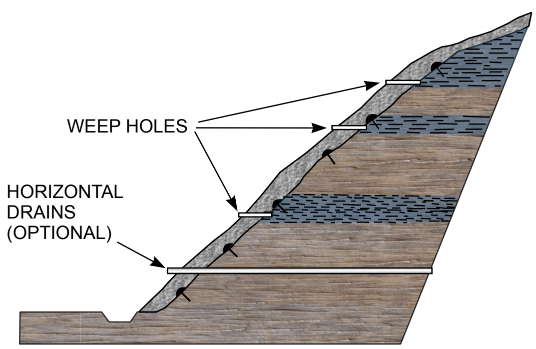
FIGURE 6.60 STEPPED SLOPE EXAMPLE FOR CUTS IN WEATHERED ROCK



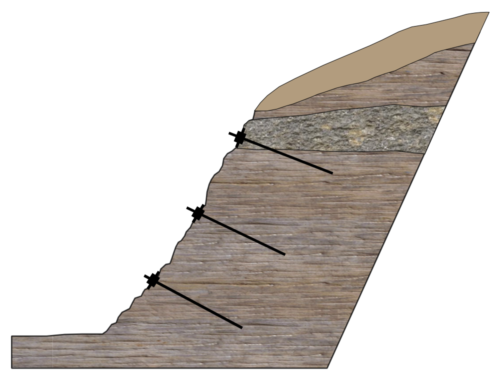
ROCK-FALL CATCHMENT



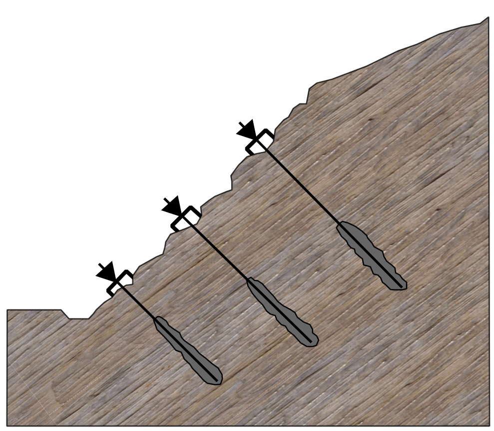
RETAINING WALL



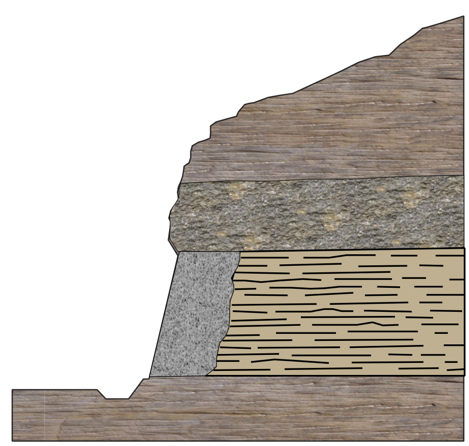
WIRE MESH WITH OR
WITHOUT GUNITE



ROCK BOLTS



TIE-BACK ANCHORS



CONCRETE OR MASONRY
SUPPORT

FIGURE 6.61 EXAMPLES OF ROCK-CUT CONTROL

All of these support techniques are costly and require contractor expertise and monitoring by experienced engineering personnel for successful implementation.

Where stabilization of highwalls is not practical, safety zones where access is restricted should be considered. The extent of safety zones can be established using computer programs such as the Colorado Rockfall Simulation Model (Jones et al., 2000) to delineate the area that could be affected by rockfalls and to estimate the size of the berm that should be constructed to retain materials that fall from the rock face.

6.6.5.4 Stability Measurements

When the stability of a rock slope is important to the design of a coal refuse disposal facility, estimates of deformations should be made for comparison to field observations. Depending on site conditions and project requirements, rock slope performance can be monitored by:

- Surface monuments for monitoring vertical and horizontal displacements
- Inclinometers for monitoring subsurface lateral displacements within slopes

Instrumentation for mine refuse disposal sites is presented in Chapter 13.

6.6.6 Conduit Structural Design for Earthen Fill Loads

At coal refuse disposal facilities, conduits are used to convey water from an impoundment through, under, or around a refuse embankment dam in a controlled manner. They are also used to convey surface water under haul roads and to control surface flows at the periphery of impoundments and refuse embankments. Conduits through refuse embankments serve the following purposes:

- Convey stored waters to the coal preparation plant
- Provide emergency reservoir evacuation capability
- Provide a primary or secondary outlet for passing storm water flow

A conduit through a refuse- or water-impounding embankment creates a discontinuity in the embankment cross section. Therefore, the structural design and construction of conduits for these situations must address potential problem conditions that have led to the distress and/or failure of impounding embankments. Some of these potential problem conditions include:

- Differential settlement between the conduit and the surrounding embankment due to difference in stiffness between the conduit and surrounding material.
- Differences in compaction between the backfill around the conduit and the remainder of the embankment.
- Structural defects in the conduit resulting from deterioration, overstressing due to lack of proper backfilling under conduit haunches or excessive fill heights, cracking due to foundation compression or lateral extension, or joint separation due to poor design and construction.
- Water leaking from the conduit that can lead to increased seepage, especially under pressurized flow conditions, in the embankment.
- Seepage flowing into an open conduit joint or crack in a conduit leading to internal erosion in the embankment.

There are special design considerations for conduits through impounding embankments that are unlike those for conduit applications that do not involve dams or impoundments. These include:

- Limiting the number of joints to maintain water tightness and to minimize the potential for internal erosion, backward erosion piping, and structural deterioration.
- Limiting flow velocity to minimize the potential for cavitation and erosion.
- The need for seepage control structures along the outside of the conduit.
- Accommodating fluctuating flows due to seasonal conditions and operating requirements.
- Accommodating flow interruptions to meet inspection and maintenance requirements.
- Maintaining operating tolerances for gate and valve functionality.

The following sections describe the types of conduits typically used at coal refuse embankments, including their structural characteristics and methods used for structural design and conduit installation (FEMA; 2005a, 2007). Hydraulic design issues for coal refuse embankment conduits are presented in Chapter 9.

6.6.6.1 Conduit Types

The materials used for conduits at coal refuse disposal facilities include reinforced precast concrete, thermoplastics, and welded steel pipe. High-density polyethylene (HDPE) is currently popular due to its light weight, durability, corrosion resistance, lower cost and the use of fuse-weld pipe joints to make the constructed pipe virtually jointless except at upstream extensions of decant pipes. Precast, prestressed concrete cylinder pipe also has a long history of satisfactory performance and resistance to structural deterioration. However, product cost, heavy weight and the frequency of joints are limitations that must be considered. Coated steel pipe is also used at refuse disposal facilities. A general discussion of the characteristics of concrete, thermoplastic and metal conduits including advantages and disadvantages is presented in the following sections.

6.6.6.1.1 Concrete

Types of precast concrete pipe typically used for conveying water through dams and impounding refuse embankments include reinforced concrete pipe (RCP), reinforced concrete cylinder pipe (RCCP), and prestressed concrete cylinder pipe (PCCP). RCP is used to convey flows under gravity head. RCCP and PCCP have seals at the pipe joints and thus are able to convey flow under pressure head. Precast concrete pipes are typically circular in cross section. Rectangular (or box) precast conduits are seldom used in dams and impounding refuse embankments because watertight joints cannot be reliably constructed.

The advantages of using precast concrete pipe for conduits include:

- The pipe is manufactured in a controlled environment to tight tolerances.
- Installation is relatively quick.
- Varying settlement along the pipe length can be accommodated by articulated joints between sections.

The disadvantages of using precast concrete for conduits include:

- There is a potential for opening of joints due to embankment settlement or elongation because longitudinal reinforcement does not extend across joints.
- Shipping and handling limitations result in short pipe section lengths and numerous joints over the length of the pipe, leading to an increase in the number of potential leakage locations.

- Gasketed joints are the only defense against leakage.
- Compaction of backfill is difficult under the pipe haunches.

While reinforced cast-in-place concrete pipe has a long history of successful performance for water-impounding dams, it is seldom used at coal refuse disposal facilities due to the extended timeframes over which the lengthy decant conduits are constructed, the difficulty of delivering adequate volumes of fresh concrete (to what are often remote locations) during pipe construction, the complexity of formwork and related construction, and the susceptibility to cracking and other distress associated with settlement.

6.6.6.1.2 Thermoplastic

Thermoplastics are solid materials that change shape when heated, and they commonly include polyethylene (PE) and polyvinyl chloride (PVC). Thermoplastic pipe is produced by an extrusion process, in which molten polymer material is continuously forced through an angular die by a turning screw. The die shapes the molten material into a cylindrical shape. After a number of additional processes, the final product is cut into pipe lengths that are suitable for delivery and handling.

The thermoplastic material most commonly used in refuse facilities is solid-wall HDPE. HDPE pipe is relatively inert chemically and thus is not particularly prone to corrosion or deterioration, has a long service life, and requires little maintenance. This is especially important for small-diameter pipes that are not easily renovated and cannot be easily inspected. HDPE is typically available in sizes up to 63 inches in diameter. Manufacturers can fabricate HDPE custom pipe fittings in addition to common fittings such as bends, flanges, reducers, and transitions.

The advantages of using thermoplastic pipe include:

- Its light weight facilitates relatively quick and simple installations.
- It resists corrosion and is not affected by most naturally occurring soil and water conditions.
- Its smooth interior surface limits friction loss and is resistant to the adherence/buildup of minerals such as calcium carbonate.
- The pipe is relatively watertight pipe if pipe joints are properly heat fused.
- It is resistant to biological attack.

The disadvantages of using thermoplastic pipe include:

- Its susceptibility to damage or displacement by construction and compaction equipment.
- Proper compaction of backfill at pipe haunches is difficult.
- Heat fusion of pipe joints requires special equipment and an experienced operator.

6.6.6.1.3 Metal

Metal conduits used at coal refuse disposal facilities are typically limited to welded, coated steel pipe. Steel pipe with diameters of 24 inches and smaller (36 inches and smaller at some shops) is manufactured in standard wall thicknesses and diameters. Pipe that is greater than 24 inches in diameter can be custom manufactured to any desired diameter. Standard diameters for steel pipe with diameters greater than 24 inches are listed in Manual M11 published by the American Water Works Association (AWWA, 2004) . Minimum plate (wall) thickness for larger-diameter pipe is one-quarter inch. Available plate thicknesses increase by multiples of one-sixteenth inch.

Steel pipe can be protected with a variety of linings and coatings. Frequently, the interior lining is not the same as the exterior coating because of the difference in exposure conditions between the interior and exterior surfaces. Typically, the interior surface is lined with the same coating regardless of location. The exterior surface coating will depend on location, encasement, and whether or not the pipe is submerged. The exterior surface is usually uncoated if it will be encased in concrete. Interior coatings and linings for mitigation of corrosion should be selected consistent with the anticipated fluid velocities within the pipe. Cement mortar should only be used on the interior surfaces of steel pipe where flow velocities will be low.

The advantages of using steel for conduits include:

- It is manufactured to tight tolerances in a controlled environment.
- It has a long service life, if proper linings and coatings are used.
- Welded joints provide watertightness.
- It can be constructed on compressible foundations.
- It has high compressive and tensile strength.
- It is flexible and deformable under stress.
- Its high modulus of elasticity provides resistance to buckling loads caused by external water pressures.
- A wide variety of special sections can be fabricated.
- It is easy to connect additional steel pipe in the future by tapping and welding.
- Flanges provide a rigid connection to gates and valves.

The disadvantages of using steel for conduits include:

- The material cost is high.
- Proper selection of linings and coatings and associated protection and maintenance measure is necessary in order to prevent corrosion.
- Proper compaction of backfill at pipe haunches is difficult.
- Special linings are required at impoundments where aggressive/corrosive water may be present (e.g., acidic mine drainage).

Corrugated Metal Pipe (CMP) applications are generally limited to surface drainage culverts and other near-surface installations where maintenance or replacement can readily be accomplished.

6.6.6.2 Soil-Structure Design

Conduits should be designed to withstand a variety of loads and pressures including:

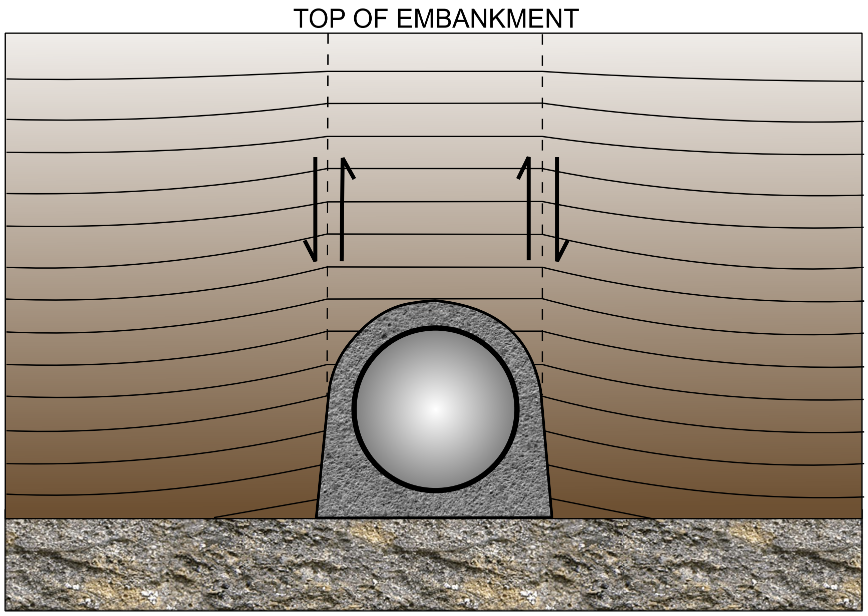
- Internal fluid and vacuum pressures
- External hydrostatic loadings and buckling pressures
- Embankment loads
- Surface surcharge loads
- Construction loads from handling or equipment trafficking
- Operational and maintenance loadings
- Load combinations

Designers should also consider the effects of vertical and horizontal displacements that might occur due to settlement and spreading of the embankment and foundation during construction. These displacements could result in loads on the conduit that exceed the design loads listed previously. Excessive displacements, both vertical and lateral, can occur when conduits have foundations that are either weak, compressible, or both. Poorly compacted embankments or embankments supported on compressible foundations can deform due to shear forces, and this deformation can lead to lateral spreading of conduits (Rutledge and Gould, 1973).

FEMA (2005a) recommends that conduits be analyzed for the following loading conditions:

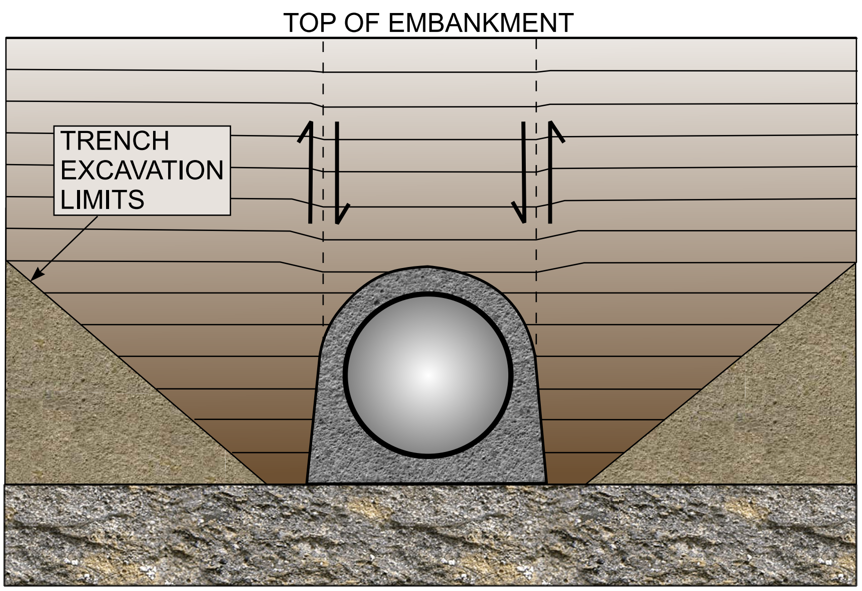
- Usual – Includes: (1) normal operating conditions with the reservoir at or near normal pool, involving combinations of vertical soil load (due to the weight of the fill above the pipe), (2) horizontal soil load, (3) external and internal hydrostatic pressure loads, and (4) the vertical foundation reaction (typically assumed to be equal to the vertical soil load plus the weight of the conduit).
- Unusual – Includes loads resulting from high reservoir levels and elevated discharges associated with flood conditions. Because floods typically have a short duration, conduits may not be significantly affected by increased external hydrostatic pressure. The difference between usual and unusual loading conditions could be limited to increased internal hydrostatic pressure.
- Extreme – Usual loading conditions plus earthquake loading. Depending on the consequences of failure of (or severe damage to) a conduit and vertical concrete riser intake (particularly, if of appreciable height) under seismic loading, a range of earthquake loadings may need to be considered, including the seismic loading associated with the maximum credible earthquake, if conduit operation is required for preventing overtopping during the design storm. Conduits are "low-profile" structures and tend to have a relatively high natural frequency. Unless a conduit is founded on deep layers of soil where peak ground accelerations could be magnified, unamplified peak ground accelerations are typically assumed to act on the conduit. If the natural frequency of the conduit is greater than 33 Hz, a pseudostatic analysis generally provides acceptable results. Other factors that may affect loading conditions are the type of foundation, method of bedding, flexibility of the pipe, and soil properties. Generally, conduits designed with an adequate static factor of safety are unlikely to buckle without a substantial stiffness reduction of the embedding soil under earthquake loading. Davis and Bardet (1998) describe a pseudostatic analysis and estimate backfill stiffness reduction, demonstrating that critical conditions would be associated with high pore-pressure buildup under low vertical effective stress (e.g., low depth of cover).
- Construction – Pertains to loads resulting from construction activities such as construction vehicles or equipment moving or working near or over the conduit.

The Marston theory (Moser, 2001) is typically used to calculate loads on a conduit that is partially or fully projecting above the original ground surface. The vertical load on the conduit is considered to be a combination of the weight of the fill directly above the conduit (i.e., prism load) and the frictional forces from adjacent fill acting upward or downward at the boundaries of the prism of earth above the conduit. This combined loading is also known as the "projection" condition. As illustrated in Figure 6.62, there are two projection loading conditions. The embankment condition shown in Figure 6.62a occurs when fill adjacent to a conduit settles more than the fill directly above the conduit. As a result, downward frictional forces act on the prism of earth above the conduit and can increase the resultant load on the conduit by as much as 50 percent of the weight of the fill above the conduit (i.e., 1.5 times the prism load, which has historically been adopted by the NRCS (2005a)). The trench condition shown in Figure 6.62b occurs when the fill



6.62a CONDUIT CONSTRUCTED PRIOR TO FILL PLACEMENT

NOTE: FRICTION FACTORS INCREASE EMBANKMENT LOAD ON THE CONDUIT AS ADJACENT FILL SETTLES MORE THAN EARTHFILL OVERLYING THE CONDUIT.



6.62b CONDUIT CONSTRUCTED IN TRENCH EXCAVATED INTO EMBANKMENT

NOTE: FRICTION FACTORS DECREASE EMBANKMENT LOAD ON THE CONDUIT AS FILL OVER CONDUIT SETTLES RELATIVE TO ADJACENT EMBANKMENT.

(ADAPTED FROM FEMA, 2005a)

FIGURE 6.62 DEFORMATIONS ASSOCIATED WITH CONDUITS IN EMBANKMENTS adjacent to the conduit settles less than the overlying fill. For this case, the differential settlement results in an arching condition that can reduce the load on the conduit by as much as 50 percent of the weight of the fill above the conduit (i.e., 0.5 times the prism load).

FEMA (2005a) recommends that conduits in embankment dams not be installed in trenches with vertical side walls or steep side slopes because a reduction of contact between the fill and the conduit is possible and planes of reduced in-situ stress can develop due to the effects of arching. These phenomena can lead to concentrated seepage paths and conditions more prone to internal erosion of susceptible backfill along and around the conduit. With respect to plastic pipe, FEMA (2007) presents guidance for determining pipe loading (i.e., application of the Marston theory or reliance on the prism load immediately above the pipe) for applications with depths up to about 50 feet and recommends other guidance from pipe manufacturers for situations of deeply buried pipes. FEMA's recommendations are written in the context of earth embankment dams designed for water supply or flood protection. Trench installation of conduits has been successfully employed in the mining industry. However, unnecessarily deep trenches, trenches in potentially unstable ground, and installation details that are difficult to construct within a trench should be avoided. If trench installation of conduits is considered, the backfill should be designed to provide consistent lateral support for the pipe (e.g., through reliable compaction effort or the use of CLSM to enable positive contact of the backfill with the conduit and trench sides). Also, the design of trench installations for flexible conduits should take into account imposed impoundment pressures resulting in external hydrostatic loads and concentrated seepage. The need for a seepage barrier to limit seepage and pressures should be evaluated in conjunction with downstream filter and drainage provisions (e.g., filter diaphragm) along the conduit.

Caution should be exercised when the height of the overburden on a conduit is greater than about 100 feet. Under such fill heights some of the previously mentioned guidance for estimating loads on conduits can result in very high stresses that require inordinately thick conduit walls and/or special reinforcement. More rigorous earth-structure interaction analyses (e.g., finite element methods) may be warranted and/or desirable in such cases and where relatively uniform firm lateral or subgrade support is not present.

The Marston theory is considered to be a very conservative approach for estimating earth loads associated with the conduit fully-projecting condition. However, more sophisticated soil-structure-interaction analytical methods that allow for 2D, 3D, and time-dependent analysis of conduits are available. These methods also allow modeling of the effects of nonlinear, stress-dependent soil (or coal refuse) stress-strain behavior. Computer programs such as CANDE (Culvert ANalysis and DEsign), FLAC (Fast Lagrangian Analysis of Continua) and Plaxis can be used to model excavation, conduit construction, conduit backfilling, and embankment construction over conduits. These programs permit calculation of backfill and conduit stresses and displacements for each stage of construction. They can also be used to evaluate the effects of conduit stiffness, backfill characteristics, foundation movements and other factors that can affect conduit performance and are not considered by Marston theory. Parameters that are typically required for conducting such an analysis include:

- Embankment/fill geometry and unit weight
- Trench geometry
- Undeflected pipe geometry
- Ground/embankment water levels
- Conduit internal water pressure
- Short- and long-term elastic modulus of conduit material
- Compressive strength of conduit material

- Tensile strength of conduit material (metal conduits)
- Geometry, spacing and material properties of reinforcing steel (concrete conduits)
- Bedding factor (concrete conduits)
- Stress-strain properties of compacted backfill envelope (e.g., soil or CLSM) and embankment fill

Concrete conduits are typically designed as rigid structures, whereas plastic and metal conduits are designed as flexible structures. Concrete conduits deflect minimally under load and derive much of their external load capacity from the high strength of the conduit, although bedding and backfill support also contribute to this capacity. Flexible conduits deflect under load and derive their external load capacity from the ability of the conduit to deflect and develop soil support at the sides of the conduit. These very different behaviors must be accounted for during design.

Precast concrete cylinder pipe (RCCP, PCCP) is customarily structurally designed as a rigid conduit by the manufacturer. Internal, external and combinations of loads are applied to a unit length of pipe, and thrusts and moments at various points around the perimeter of the pipe are calculated. The required reinforced concrete design parameters, including concrete thickness, reinforcing steel amount, steel cylinder thickness, and prestress tension, are then determined. Detailed reinforced concrete design procedures with examples for RCCP are provided in Manual M9 (AWWA, 1995). Design procedures, with examples for PCCP, are presented in Standard C304-07, "Design of Prestressed Concrete Cylinder Pipe" (AWWA, 2007).

Thermoplastic and welded steel pipe are designed as flexible structural elements. Internal, external and combinations of loads are applied to a unit length of pipe, and thrusts and moments at various points around the perimeter of the pipe are determined using design guidelines such as *National Engineering Handbook Chapter 52: Structural Design of Flexible Conduits* (NRCS, 2005a). FEMA (2007) has also published a technical manual for plastic pipe used in embankment dams. The NRCS document includes detailed procedures and guidelines for design, inspection, maintenance, and repair of thermoplastic pipe. Detailed design procedures, including the application of methods for evaluating constrained pipe wall buckling, ring deflection, soil reaction modulus, and long-term deflection considerations and limits are provided in engineering manuals prepared by flexible pipe manufacturers (e.g., Performance Pipe, 2003). Additional references include Watkins and Anderson (1999).

Both thermoplastic and welded steel pipe, when backfilled with CLSM or concrete, provide resistance to seepage along the external perimeter of the pipe (FEMA, 2005a). Use of CLSM or concrete encasement, for example, can eliminate the problem of poor backfill compaction in the haunch area of circular pipes. The design of CLSM for pipe support is discussed in Section 6.6.6.3.3. The design of concrete encasement will need to address the structural interaction between the rigid concrete and flexible conduit, and it should demonstrate that the resulting stresses are acceptable.

Because of the critical nature of conduits extending through embankments, careful attention must be paid to construction quality control. Major conduit installation work should be monitored on a full-time basis by a person familiar with the installation requirements. This should include careful monitoring of conduit joint construction and sampling and testing of construction materials. In the post-construction period, major conduits should be periodically inspected (typically through internal camera surveys and, particularly for flexible conduits, inside diameter/deflected shape measurements) to verify acceptable performance of the conduit and to allow early detection of defects and signs of overstressing. Early detection is critical to timely planning and implementation of remedial measures for poorly functioning or distressed conduits. In multi-stage coal refuse disposal facilities, data from monitoring of early stages provides valuable information for projecting or verifying conduit performance under increased fill depths.

The structural adequacy of riser inlets attached to the main conduit barrel should be evaluated. Typically, risers extend upward to relatively shallow depths below the impoundment surface during facility operation and then are decommissioned when no longer needed. Careful consideration should be given to the method of supporting and subsequently capping and decommissioning riser pipes, particularly when the capped riser will eventually be deeply buried. Decommissioned riser pipes represent a potentially weak point in the conduit system. Loads imposed on the riser pipes can be transferred to the main conduit (“barrel pipe”)and cause unacceptable deflection or stress at the location of the riser pipe connection. At present, there are few analytical tools available for analyzing the interaction of deeply buried risers (particularly in cases where the inlet sections are neither vertically nor horizontally oriented) with the surrounding fine coal refuse. Until better analytical tools are available, the support (or encasement) of inlet risers should be carefully evaluated. It may be beneficial to monitor the performance of the riser pipe where it joins the main conduit using internal camera surveys and by making inside diameter/deflected shape measurements at key stages during the life of the facility.

6.6.6.3 Conduit Installation and Design Details

6.6.6.3.1 Installation

Conduits are constructed or installed by placement at or near a level ground surface (embankment condition) or within a trench (trench condition). If the fill adjacent to the conduit settles more than the fill overlying the conduit, which is typical for an embankment installation, downward frictional forces are induced that can increase the load on the conduit by up to 50 percent of the weight of the fill above the conduit. However, if the fill placed directly above the conduit settles more than the adjacent fill, which can happen for conduits constructed in a trench, arching will occur and can reduce the load on the conduit by up to 50 percent of the weight of the fill above the conduit. FEMA (2005a, 2007) presents guidance and recommended limitations for pipe loading and installation conditions for conduits and plastic pipe, respectively, in dams, although other references are applicable for plastic pipe at depths greater than about 50 feet.

Construction of conduits in trenches with vertical side walls or steep side slopes is not recommended (FEMA, 2005a) because a reduction of contact between the fill and the conduit is possible and planes of reduced in-situ stress can develop due to the effects of arching. As noted previously, FEMA’s recommendations are written in the context of earth embankment dams designed for water supply and flood protection. If trench installation of conduits is considered, the backfill should be designed to provide consistent lateral support for the pipe. Also, the design of trench installations for flexible conduits should account for imposed impoundment pressures resulting in external hydrostatic loads and concentrated seepage. Unnecessarily deep trenches, trenches in potentially unstable ground, and installation details that are difficult to construct within a trench should be avoided.

6.6.6.3.2 Rigid Conduit Support

Precast concrete circular pipe should be constructed on concrete or CLSM cradles to eliminate difficulties related to compaction beneath the haunches of the pipe. The cradle should be concrete or higher strength CLSM and should provide vertical, longitudinal, and lateral structural support to the pipe. The cradle should extend for the full length of the pipe and should also extend above the bottom of the pipe at least 25 percent of the pipe outside diameter (i.e., Class A bedding per the American Concrete Pipe Association (1987). A more substantial cradle may be desirable or may be required for enhanced support, especially on soil subgrade. For applications that allow for conduit support on soil, direct or indirect design methods are available (American Concrete Pipe Association, 2007; USACE, 1998a).

The design of the conduit bedding and backfill is dependent upon the compressibility of the foundation and the purpose for which the conduit will be used. Precast concrete pipe should not be used for pressurized applications in significant- and high-hazard-potential embankments unless measures are taken to address the potential impact of failure of a pipe joint or joint gasket and resulting leakage of pressurized water into the embankment. Conduits should be constructed on rock or firm foundations whenever possible. If a pipe is founded on a compressible foundation, the design should accommodate the anticipated magnitude and distribution of settlement, because settlements can lead to joint failure.

Many approaches have been used to design rigid conduits on compressible foundations. A preferred approach utilizes a joint design for the cradle that allows articulation and spreading of both the pipe and the cradle. Cradle joints are placed at the locations of the pipe joints, and cradle reinforcement is not allowed to pass through the joints. Spaces between joints are filled with a compressible material, such as high-density sponge rubber or bituminous fiberboard, to keep extraneous material out of the joints and allow articulation. USDA (1958) provides design guidelines for pipe cradles.

A concrete cradle should bond to the conduit. In embankment installation configurations, the cradle should extend at least 4 inches beyond each side of the pipe, and the sides of the concrete cradle should always be sloped at 1H:10V or flatter through low-hydraulic-conductivity zones to allow construction equipment to compact earthfill directly against the cradle. There should be no sharp or protruding corners associated with the cradle that could cause undesirable stress concentrations in the fill. Measures should be taken to support the conduit on grade until the concrete cradle has been placed and cured.

Concrete bedding and cradles beneath circular conduits are typically designed to reach about 25 percent of the conduit outside height in order to provide support to the conduit and to facilitate compaction under the haunches. The bedding is often constructed with joints opposite the circular conduit joints so as to not interfere with potential conduit movement. Guidance for the use of bedding in conjunction with fully circular pipes is provided in USDA (1958) and USACE (1998a). The choice of whether to use cradles or bedding is typically a function of the height of the embankment, quality of the subgrade and the benefits gained from using a cradle versus bedding in reducing the structural requirements for the pipe. Cradles are often used in higher embankments where more lateral support is required. Regardless of whether a cradle or bedding is used, seepage control measures such as filter diaphragms should be employed, even for low-hazard-potential embankments and favorable conditions. A discussion of seepage control measures along conduits is provided in Section 6.6.2.3.3.

6.6.6.3.3 Flexible Conduit Support

As discussed in Section 6.6.6.1, the use of HDPE pipe at coal refuse disposal facilities has become prevalent. Concrete cradles and bedding are not generally used for flexible pipe installations because flexible pipes require some deflection to develop resistance from the surrounding backfill, and cradles and bedding limit this deflection. Thus, if flexible pipe is constrained by a rigid cradle, it may become overstressed. Therefore, HDPE pipe should not be supported or constrained by a rigid cradle or encased in normal-strength concrete that will limit the flexible pipe from deforming in its intended manner. However, HDPE pipe may be used as a liner within a structural encasement if the system is properly designed as a rigid conduit, with appropriate stiffness capable of withstanding potential external hydrostatic loads, and the heat of hydration of the concrete and wall thickness of the HDPE liner are balanced to limit the potential for separation of the liner from the encasement. Simply encasing a plastic liner pipe in concrete will not necessarily preclude development of significant hydrostatic pressure at the interface between the liner pipe and encasement. The potential for external hydrostatic loading is further addressed in FEMA (2007).

As an alternate to compacted soil backfill, flowable or controlled low-strength material (CLSM) backfill has been successfully used to provide support and encasement of flexible conduits. For plastic conduits, the flowable backfill/CLSM should be designed to have a relatively low cement content (low unconfined compressive strength), but have substantial fractions of fine aggregate and pozzolan so that a mix design with a soil-like matrix, low heat of hydration, limited shrinkage potential, and relatively low hydraulic conductivity is achieved. As described in Section 6.5.10.2, flowable backfill consists of cement and other pozzolans (e.g., fly ash), sand and water. The target strength should be that of a very stiff to hard soil (in the range of 50 psi to 200 psi maximum) so that the conduit-backfill system behaves as intended. Mixes designed to produce a compressive strength much lower than 50 psi may not have sufficient cement content to produce a conduit backfill with uniform characteristics (e.g., strength and compressibility) and erosion and weathering resistance. Pre-testing of prospective flowable backfill mixes is recommended so that a flowable material with relatively uniform strength and stiffness properties is achieved.

The hydraulic conductivity of the flowable fill material should be less than or equal to the hydraulic conductivity of the adjoining portions of the embankment. The addition of a small percentage of bentonite or attapulgite to the CLSM mix is an option in cases where a hydraulic conductivity much lower than 10^{-5} cm/sec is desired or required. For conduit installations in embankment dams, measures should be in place to intercept and drain potential seepage along the conduit. Typical measures include filter diaphragms, as described in Section 6.6.2.3.3. FEMA (2007) has expressed caution about the use of CLSM in significant- and high-hazard-potential embankment dams, pending research to evaluate its performance. As noted previously, FEMA's concerns are presented in the context of earth embankment dams designed for water supply and flood protection. CLSM backfill installations of conduits are becoming more common for embankments used in transportation infrastructure and in the mining industry, and they are attractive because they provide effective support while addressing seepage issues.

The reaction modulus for CLSM backfill should be selected based on the mix design, as discussed above, and available test data (e.g., unconfined and uniaxial compressive strength tests), conservatively considering the CLSM to be equivalent to a very dense sand or gravel material. For deep conduit installations, the influence of earthen materials adjacent to the CLSM may also need to be incorporated into the evaluation of deflection and buckling. When used in the modified Iowa Formula for buried pipe deflection calculations, the reaction modulus is analogous to the soil reaction modulus E' (Howard, 1977). Flowable fill employing a mix design to achieve a minimum 50 psi at 28 days, and incorporating the recommendations presented in Section 6.5.10.2, would presumably have a reaction modulus equal to or greater than highly compacted granular backfill given its compressive strength and low compressibility. Modeling based on testing of such mixes by McGrath and Hoopes (1998) determined an E' of 3,000 psi after 28 days for CLSM with high air content. Reaction modulus values as high as 3,800 psi have been applied in the past for embankment dams in the mining industry and have yielded reasonably good predictions of actual deflections for deeply-buried, thick-walled HDPE pipe. However, the actual width of flowable fill on each side of the pipe installation relative to the pipe diameter and the consistency of the native sidefill soil or trench wall will influence the extent to which the flowable fill support/reaction predominates over the support/reaction of the sidefill soil or trench wall (as applicable).

Note that the modified Iowa Formula and the recommended modulus of soil reaction E' values for application in the formula were developed from data on pipe installations up to about 50 feet deep, presuming that the prism load acts on the buried conduit. Therefore, for burial depths up to and somewhat greater than 50 feet, the prism load and recommended limiting soil reaction values should be adopted when applying the modified Iowa Formula. However, for burial depths much greater than 50 feet, if the vertical load on the conduit is truly increasing in proportion to the fill height, it follows that there would be some attendant increase in the soil reaction modulus. Currently, guidance for quantifying a soil reaction modulus or reaction modulus of non-soil backfill for simplified analysis of deeply buried conduits is limited.

6.6.6.3.4 Watertightness

If the joints between conduit sections separate or deteriorate, the conduit may develop leaks that can lead to internal/backward erosion (piping) failure mechanisms. Accordingly, government agencies such as the USACE and the Bureau of Reclamation require that joints in conduits in embankment dams be watertight (FEMA, 2005a). The degree to which joints must be watertight depends on the anticipated hydrostatic head both inside and outside of the conduit. As shown in Figures 6.63 and 6.64, RCCP and PCCP employ a rubber gasket confined between steel spigot rings and mastic in the joints to provide a waterstop. The watertightness of HDPE conduit is achieved by fuse welding the conduit joints. Similarly, the watertightness of steel conduit is achieved by welding the conduit joints. Details related to conduit joint construction and measures for achieving watertightness are provided in FEMA (2005a).

Conduits constructed on compressible foundations are vulnerable to joint spreading. Special attention should be given to evaluating the compressibility and shear strength of materials in compressible foundations. Section 6.6.3 provides a discussion of methods for predicting settlement. Technical Release 18 (TR-18) published by the USDA (1969) uses the predicted vertical strain beneath a conduit, the shear strength of foundation soils, and the geometry of the embankment and foundation to predict horizontal strain in the conduit. If the joints between the ends of conduit sections separate or develop other defects, a conduit may develop leaks. This leakage can lead to the development of internal erosion or backward erosion piping failure mechanisms.

Excessive lateral movement of an embankment/foundation system can occur when thin weak layers in the foundation are loaded beyond their shear strength. These movements may result in slope instability, and they can also cause damage to a conduit if it is located over the weak layer. Slope flattening and berms can be used to prevent such movements, but these remedies will result in a longer conduit. To minimize differential settlement and movement of conduit joints, foundations under conduits should have relatively uniform compressibility characteristics.

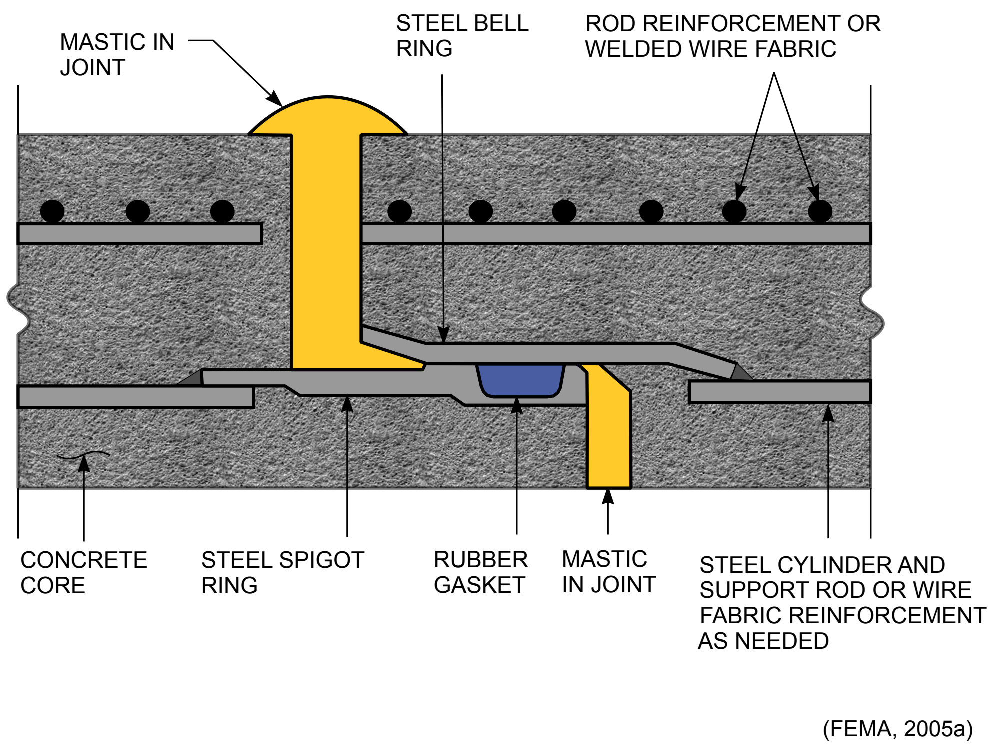


FIGURE 6.63 REINFORCED CONCRETE CYLINDER PIPE (RCCP) DETAILS

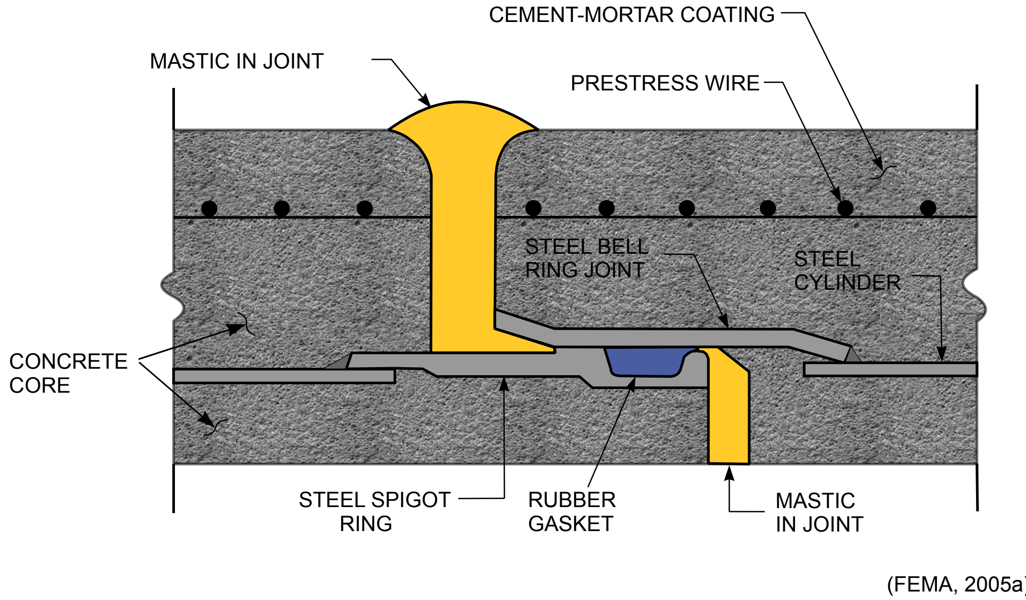


FIGURE 6.64 PRESTRESSED CONCRETE CYLINDER PIPE (PCCP) DETAILS (LINED CYLINDER)

Variable foundation conditions can result in abrupt changes in conduit settlement that can lead to large relative movements and failure. A properly designed joint will limit vertical and transverse displacement of conduit sections relative to each other as an embankment settles and will also accommodate rotation and longitudinal movement while maintaining watertightness. For conduits on a compressible foundation, the maximum potential joint elongation that may occur as a result of the compressibility of the foundation should be determined as accurately as possible. The potential joint elongation is a function of:

- Shear strength of the foundation
- Estimated settlement of the foundation
- Configuration of the embankment dam
- Lengths of conduit sections

If the predicted elongation exceeds the magnitude that the joints can accommodate, design changes such as using shorter lengths of conduit, replacing compressible foundation soils with compacted backfill, and flattening the slopes of the embankment should be considered.

Embankment settlement may not always be as predicted by analyses and can sometimes vary over relatively short distances, resulting in abrupt joint displacements. Figure 6.65 illustrates that measured settlement can be significantly different from the calculated settlement. These variations in actual settlement can lead to joint displacements. Abrupt joint displacements are generally more likely for conduits that are constructed using precast concrete pipe than for conduits constructed with RCCP or PCCP. The reason for the difference is that RCCP and PCCP conduits are constructed with longitudinal reinforcement extending through the joints, allowing the conduits to bridge over weak foundation areas and to spread the effects related to variation in settlement. Welding of the conduit joints for HDPE and steel pipe results in higher strength and strain characteristics and increased resistance to distress caused by abrupt displacement. However, conduit foundations should be designed to provide uniform support.

Special precautions should be taken at joints where conduits connect to structures. The joint between an intake structure and a conduit is susceptible to differential settlement because of the potential

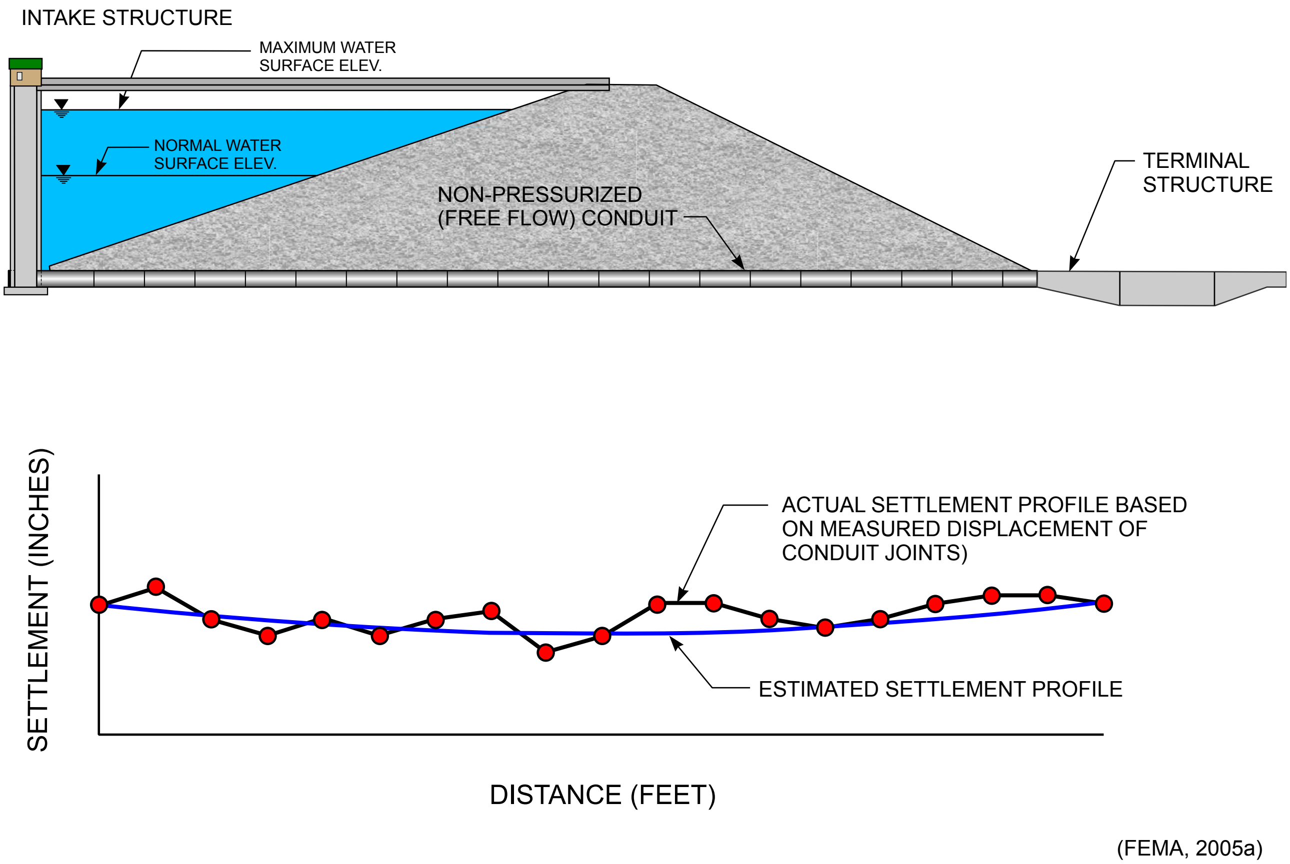


FIGURE 6.65 COMPARISON OF PREDICTED AND OBSERVED CONDUIT SETTLEMENTS

disparity in the weights and foundation conditions between the two structures. If both structures are constructed over relatively soft materials, the intake structure will tend to settle much more than the conduit. If the intake structure is constructed over engineered fill of low compressibility or on deep foundation elements, the conduit may tend to settle more than the intake structure. Either situation can lead to excessive deformation of the conduit joints in the vicinity of the intake structure. Under such circumstances, provisions to allow for relative movement may need to be incorporated into the design.

6.6.7 Blasting Impacts

Blasting must be conducted in such a manner as to prevent injury to site personnel and unacceptable impacts to structures associated with a coal refuse disposal facility, including refuse or earthen embankment dams and their impoundments. Impacts to off-site structures and properties must also be limited in accordance with applicable guidelines. Typically, embankment dams have very low natural frequencies (on the order of 1 hz) and thus are not particularly susceptible to damage due to blast vibrations, which have a much higher frequency range. If it is believed that blast effects could have a deleterious effect on site structures (such as for embankments developed by the upstream construction method), the impact of ground motion for the anticipated magnitude and frequency range of blast vibrations can be considered using the procedures for seismic stability described in Chapter 7. Some structures associated with fresh water dams, such as large concrete spillway channels, tall riser intake structures or pipelines under low confinement could possess natural frequencies similar to blast frequencies and thus be impacted by blasting. However, typical concrete structures and pipelines used at coal refuse facilities are normally not very susceptible to damage from blasting vibrations.

Structures are affected by blasting in relation to the peak particle velocity and frequency content of the ground motion induced by a blast. Simplified relationships are commonly used to determine the charge size relative to the horizontal distance to the monitoring point in order to meet acceptable motion (velocity) criteria. One such relationship for the peak particle velocity V resulting from a blast is (ISEE, 1998):

$$V = K(D/W^{0.5})^{-1.6} \tag{6-33}$$

where:

K = site-specific constant determined from calibration test

D = distance to blast (length)

W = weight of charge (force)

Topographic and geologic variations between the blast location and observation point and the position of the blast horizon relative to the foundation of the structure can significantly affect the attenuation or amplification of blast-induced ground motions.

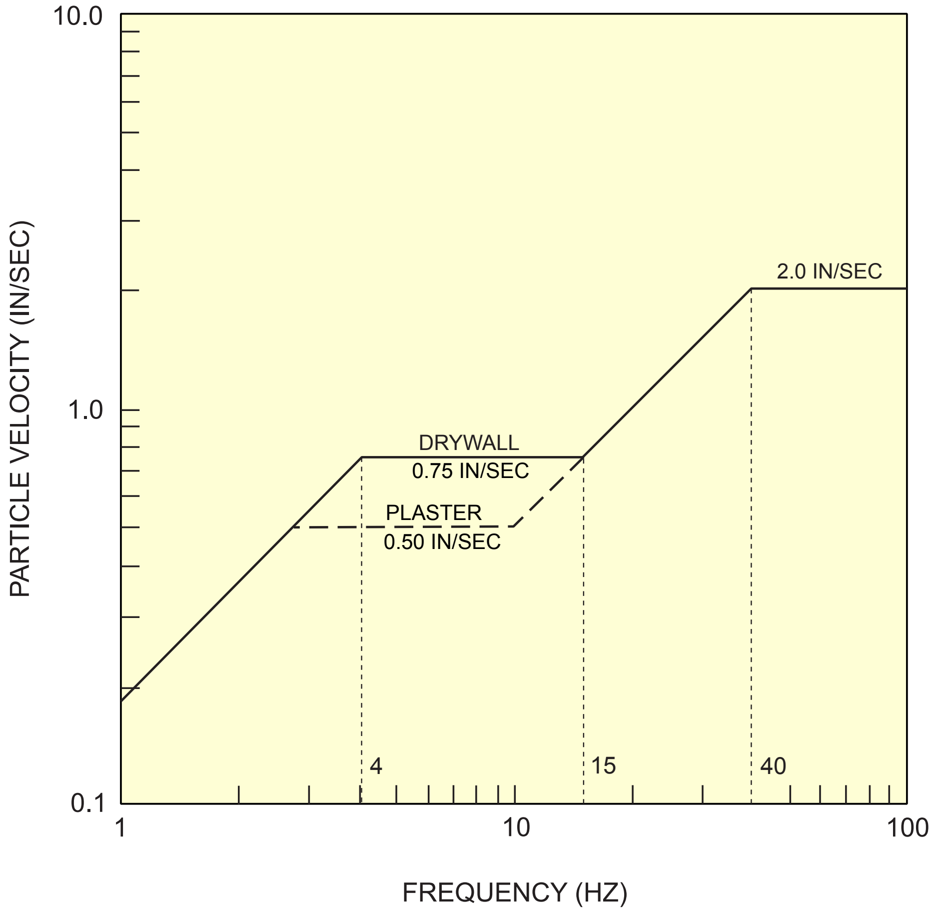
Acceptable vibration criteria are published in a number of sources: state regulatory programs typically provide guidance for peak particle velocity for common structure types, and general guidance can be found in Nichols et al. (1971), Siskind et al. (1980), ISEE (1998), and Hartman (1992). Blasting for excavation of rock materials is generally controlled to a peak particle velocity (PPV) of 4 inches per second for mass concrete structures (Hartman, 1992) and to 2 inches per second for typical steel and concrete superstructures. Notably, these PPV thresholds are generally very conservative and correlate to the possible onset of visible cosmetic damage. The noted structure types can typically tolerate much higher PPVs before structural damage occurs. Table 6.57 provides additional information on levels of damage to houses for specific particle velocities, and Figure 6.66 shows acceptable limits of vibration for houses, as recommended in Siskind (1980).

When blasting is planned within 500 feet of an active underground mine, the Surface Mining Control and Reclamation Act of 1977 requires approval of an operator's blasting plan by OSM, or the appropriate state agency, and MSHA. Blasting regulations are provided in 30 CFR § 780.13 and 30 CFR §

TABLE 6.57 COMMON RESIDENTIAL VELOCITY CRITERIA AND EFFECTS

Velocity	Damage Level
0.5 in/sec	Recommended limit to prevent threshold damage in plaster-on-lath construction near surface mines due to long-term, large-scale blasting operations (USBM, 1980).
0.75 in/sec	Recommended limit to prevent threshold damage in sheetrock construction near surface mines (USBM, 1980).
1.0 in/sec	Office of Surface Mining (OSM) regulatory limit for residences near surface mine operations at distances of 300 to 5,000 ft (long-term, large-scale blasting).
2.0 in/sec	Widely accepted limit for residences near construction blasting and quarry blasting. Also allowed by OSM for frequencies above 30 hz (USBM; 1971, 1980).
5.4 in/sec	Minor damage to the average house subjected to quarry blasting vibrations (USBM, 1971).
9.0 in/sec	About 90 percent probability of minor damage from quarry blasting. Structural damage to some houses, depending on vibration source, characteristics and house construction.
20.0 in/sec	For close-in construction blasting, minor damage to nearly all houses, structural damage to some. For low-frequency vibrations, structural damage to most houses.

(ADAPTED FROM ISEE, 1998)



(ADAPTED FROM SISKIND ET AL., 1980)

FIGURE 6.66 ACCEPTABLE LIMITS OF VIBRATION FOR HOUSES

816.61 through 816.68 and 816.79. State criteria may also be applicable. The blasting plan should be prepared by a professional licensed in the state where the blasting is to be performed. Potential concerns when an impoundment is present include fracturing of abutments, impacts to pipes and other rigid structures, and possibly impacts to upstream construction. The potential impacts should be evaluated, and monitoring of particle velocity with a seismograph may be appropriate. Monitoring of specific structures and features may be warranted and could include inspection of impounding embankment crests and slopes for evidence of cracks or displacements, review of piezometer data for evidence of water level fluctuations, and observation of concrete joints or crack apertures to verify the general integrity and to note any movements.