Chapter 9

HYDROLOGY AND HYDRAULICS

Coal refuse impoundments and embankments must handle the runoff from precipitation that occurs over the contributing watershed area. If not properly controlled, runoff can jeopardize the collection and conveyance system (channels and conduits). For impoundments, runoff can cause the embankment to be overtopped with the potential for failure. The principles of hydrology and hydraulics can be used to determine and design the required combination of flow capacity and freeboard and to select durable channel lining systems. The discussion of technical issues in this chapter is based on the assumption that the reader is experienced in the technical areas of hydrology and hydraulics and is familiar with the selection of hydrologic and hydraulic design parameters and the use of related computer software. A number of traditional design concepts are reviewed herein, and reference is made to additional resource materials.

The design of coal refuse disposal facilities requires a somewhat specialized approach. There are many possible combinations of disposal facility configuration, facility staging, environmental considerations and unique characteristics and properties associated with each site. Therefore, one of the major aims of this chapter is to relate fundamental engineering principles to the unique requirements of refuse disposal facility site design. While primarily focused on slurry impoundments, the contents of this chapter are also applicable to other mining dams and impoundments.

The hydrologic and hydraulic information and design procedures presented in this chapter fall into five interrelated categories, as follows:

- Basic definitions and principles – Sections 9.1 and 9.2 define basic terms and conditions applicable to coal refuse disposal facilities that relate to hydrologic and hydraulic features. Table 9.1 presents a complete summary of hydrologic and hydraulic planning and design procedures. The table also serves as an outline of this chapter and a summary of supplemental references. The fundamental interrelationships of runoff, reservoir storage, and outflow are established. The major elements that may affect these interrelationships at coal refuse disposal facilities are also discussed.
- General design considerations – Section 9.3 identifies regional and site conditions that affect the suitability of various hydraulic conveyance structures for coal refuse disposal facilities. In Section 9.4, these broad concepts are extended to consider the effect of disposal facility configuration upon selection of suitable hydraulic convey-

ance structures. Characteristics that distinguish coal refuse disposal facilities from conventional embankment dams are emphasized.

- Design-storm criteria – Section 9.5 presents design storm precipitation criteria for coal refuse disposal facilities. Factors such as location, facility size, and hazard potential are discussed. Design storm criteria for short-term conditions and for minor hydraulic structures are also addressed.
- Procedures for analysis – Sections 9.6, 9.7 and 9.8 discuss analytical procedures for evaluation and design of coal refuse disposal facility hydraulic structures. Methods for determining runoff based on predicted precipitation are first established, followed by reservoir storage and outflow capacity requirements. Various components of outflow structures are discussed in detail. Procedures for routing storm runoff through an impounding disposal facility and optimizing reservoir storage and outflow are presented.
- Dam-breach analysis – Section 9.9 discusses procedures for evaluation of dam breach and potential downstream inundation for the determination of hazard potential and for Emergency Action Plan (EAP) preparation.

9.1 GENERAL CONSIDERATIONS

The hydrologic and hydraulic design and analysis procedures discussed in this chapter apply to both existing and new coal refuse disposal facilities. The sequence presented in Table 9.1 is normally followed either for modifying an existing disposal facility or for constructing a completely new disposal facility. It should be recognized that sequencing of a modification to an existing coal refuse disposal facility should be continually coordinated with the ongoing mining and coal preparation operations.

The designer of a new coal refuse disposal facility normally has flexibility in site selection, staging of the embankment growth and long-term planning of related hydraulic structures. Given this flexibility, design flood requirements can typically be met throughout the entire life of the disposal facility. Often the designer is able to optimize the relationships between refuse disposal operations, embankment design, hydraulic structure construction, and the overall mining and coal preparation operations.

A designer modifying an existing disposal facility should first determine its conformance with current design storm criteria and should then assess options for any necessary upgrade of the runoff collection and control system. Sometimes a facility has limited storage or hydraulic conveyance capability, may not satisfy current design and regulatory requirements, and cannot be easily modified in a short period of time. An effective solution may be to perform a staged modification program, as part of continued refuse disposal operations, which may in fact provide materials necessary for increasing freeboard and constructing diversions, thus improving hydraulic capacity. Under such conditions, the modifications to the facility are usually required to meet or exceed MSHA's short-term hydrologic design criteria, as subsequently described in Section 9.5.2.

9.2 HYDROLOGY AND HYDRAULICS PRINCIPLES

Hydrology is the study of climatic and physical conditions that govern natural flows in rivers, streams and channels. Hydrologic analyses are used to determine the probable and possible direct runoff to a particular site from natural causes such as precipitation or snow melt. Hydraulics is the study of water flows in channels and conduits. Hydraulic engineering is used in the design of decant systems, outlet works, spillways, ditches, channels, diversion structures, and other systems for controlling flowing waters. An integrated application of hydrology and hydraulics is necessary for the development of safe, economical and environmentally acceptable coal refuse disposal facilities.

TABLE 9.1 HYDROLOGIC AND HYDRAULIC DESIGN PROCEDURES FOR COAL REFUSE DISPOSAL FACILITIES

Design Considerations	Applicability		Manual Sections for Reference	Supplemental References
	All Facilities	Impounding Facilities		
I. Determine Importance of Hydrologic and Hydraulic Considerations				
Type of facility	X		Chapter 3, 9.4	USBR (1987a)
Impounding vs. non-impounding			9.4	USBR (1987a)
Site conditions	X		Chapter 5, 9.5	USBR (1987a)
Downstream conditions	X	(Of particular concern)	9.3, 9.5	USBR (1987a)
Startup, operation, abandonment requirements	X		Chapters 4, 6, 9	
II. Establish Preliminary Facility Configuration and Hydraulic Systems				
Select structure type	X		Chapter 3, 9.4	
Balance availability of materials for embankment construction with facility staging	X		Chapter 5, 9.3, 9.4	
Determine size and potential hazard classification based on dam breach analysis and downstream inundation		X	9.5, 9.9	FEMA (2004a)
Determine appropriate design storm for long-term operation	X		9.5	MSHA (2007)
Determine if separate design consideration should be given to short-term conditions with lesser design storm at any time during the operational period of the facility	X		9.4, 9.5	MSHA (2007)
Calculate watershed contributing to major hydraulic systems	X		9.3	
Determine approximate inflow rates and volumes to be controlled by major hydraulic systems from design storm criteria	X		9.6	NWS (2006a,b) NRCS (2004b)
Evaluate alternative combinations of spillway outflow and impoundment storage capacities		X	9.6 to 9.8	USBR (1987a) NRCS (2004b) Brater et al. (1996)
Determine preliminary spillway type, location and approximate size (for all stages of operation)		X	Chapter 5, 9.6 to 9.8	USBR (1987a) Brater et al. (1996)
Determine preliminary decant type, location and approximate size (for all stages of operation)		X	Chapter 5, 9.6 to 9.8	USBR (1987a) Brater et al. (1996)

TABLE 9.1 HYDROLOGIC AND HYDRAULIC DESIGN PROCEDURES FOR COAL REFUSE DISPOSAL FACILITIES (CONTINUED)

Design Considerations	Applicability		Manual Sections for Reference	Supplemental References
	All Facilities	Impounding Facilities		
Determine magnitude of storm that can be controlled and compare with appropriate design storm for facility size and potential hazard classification	X		9.5 to 9.9	USBR (1987a) NRCS (2004b)
Evaluate modifications to be made to improve the facility's hydraulic system	X		Chapter 6, 9.5 to 9.8	USBR (1987a)
Evaluate advantages and disadvantages of modifying the facility for continued use or to a satisfactory configuration for abandonment	X		Chapter 6, 9.4 to 9.8	
Assign appropriate long-term design storm or abandonment criteria	X		9.5	FEMA (2004c)
III. Determine Design Inflow Rates and Volumes for Major Hydraulic Systems				
Determine if key parameter curves are suitable for final design for any or all stages, including abandonment	X		9.4, 9.6	
Determine inflow hydrograph parameters, if required, for any stage of development	X		9.6	NRCS (2004b) USBR (1987a)
IV. Design Major Hydraulic Systems				
Design major diversion system to insure against failure during appropriate design storm				
• Collection of inlet area	X		9.7	Chow (1959)
• Establish control section of flow (inlet, transport section or outlet)	X		9.7	USBR (1987a) Henderson (1966) Brater et al. (1996)
• Determine requirements to prevent failure by overtopping, erosion or clogging	X		9.6 to 9.8	USBR (1987a)
• Determine downstream outlet and/or discharge requirements to avoid unacceptable damage at design flow	X		9.7, 9.8	Chow (1959) USBR (1987a) Brater et al. (1996)
Determine optimum combination of storage and outflow for each stage of development (for impoundments)				
• Perform reservoir routing analysis of inflow hydrograph		X	9.6 to 9.8	USBR (1987a)
Design the spillway system for the appropriate design storm for each stage of development				

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TABLE 9.1 HYDROLOGIC AND HYDRAULIC DESIGN PROCEDURES FOR COAL REFUSE DISPOSAL FACILITIES (CONTINUED)

Design Considerations	Applicability		Manual Sections for Reference	Supplemental References
	All Facilities	Impounding Facilities		
• Establish control section for all flow conditions to assure adequate capacity		X	9.6 to 9.8	Chow (1959) USBR (1987a) Henderson (1966) Brater et al. (1996)
• Design the inlet including provisions to prevent clogging		X	9.8	USBR (1987a)
• Design the outlet to prevent unacceptable damage at magnitude of flow		X	9.8	FHWA (2006) USBR (1987a)
Design the decant system for normal operating conditions and to evaluate impoundment storage of design storm				
• Establish flow control for all storage levels to assure adequate capacity		X	9.6, 9.8	USBR (1987a) Brater et al. (1996)
• Design the inlet, including provisions to avoid clogging		X	9.8	USBR (1987a) Brater et al. (1996)
• Design the transport section, considering structural stability, corrosion resistance, and capacity		X	9.8	USBR (1987a) Brater et al. (1996) FHWA (2005b)
• Design the outlet to prevent unacceptable damage		X	9.8	USBR (1987a) FHWA (2006)
Perform dam breach analysis and evaluate downstream inundation		X	9.9	FEMA (2004c)
V. Design Minor Hydraulic Systems				
Surface drainage ditches that are not critical to safety during design storm	X		9.6, 9.8	USBR (1987a) FHWA (2005a) FHWA (2006)
Minor roadway culverts	X		9.6, 9.8	FHWA (2005b) FHWA (2006)
Weirs to separate seepage from large flows, if required, for environmental control	X		9.8	Henderson (1966)

9.2.1 Basic Design Principles

The fundamental principle governing the hydrologic and hydraulic design of a coal refuse disposal facility is that runoff, natural drainage and process water must be conveyed past the embankment, stored within the facility impoundment(s), or handled by a combination of these two methods. The hydrologic characteristics of the applicable watershed (rainfall, tributary area, land use cover conditions, soil type, slope, etc.) determine the runoff hydrograph, while the physical dimensions and hydraulic characteristics of the facility and hydraulic structures determine the required conveyance and storage capacity. Table 9.2 presents a summary of the application of the basic design principles to coal refuse disposal facilities.

TABLE 9.2 HYDROLOGIC AND HYDRAULIC DESIGN CONSIDERATIONS FOR COAL REFUSE DISPOSAL FACILITIES

Embankment Type[1]	Runoff, Outflow and Storage Considerations
Valley-Fill and Side-Hill Non-Impounding Embankments	If placement of the embankment is started at the upper end of the valley, runoff from the natural watershed can be diverted around the embankment and no water has to be stored. Precipitation on the embankment can be directed downstream. If placement is started by forming a downstream embankment, it will have a temporary character with interim diversion ditches sequentially replaced as the fill is raised (sometimes the final diversion ditches are installed initially). Precipitation and runoff on the embankment are directly discharged downstream with the intervening drainage between the final diversion ditch and the interim diversion ditches.
Ridge and Heaped Non-Impounding Embankments	Ridge and heaped embankments that are constructed above the natural topography only have inflow associated with direct rainfall onto the disposal area. Precipitation and runoff on the embankment can be directed downstream.
Cross-Valley Impounding Embankment	The cross-valley impounding embankment presents a variety of alternatives for handling hydrologic events. Inflow may include precipitation from upstream of the embankment, including the drainage area above diversion ditches, unless the ditches are designed not to fail from the design storm. For a cross-valley impoundment, the three possibilities for handling design storm inflow are: 1. If the embankment crest elevation is maintained sufficiently high above the pool level, all runoff from the design storm can be stored, such that outflow is not a requirement during the design storm. The impounded water can then be lowered gradually by flow through a decant system. 2. If a spillway of adequate size is constructed with its crest at the normal pool level of the impoundment, all of the storm runoff can be passed directly through the disposal area and the storage requirement will be minimal. 3. If the spillway crest is located above the normal pool level, but the storage volume between the pool and spillway elevations is less than the inflow volume, the spillway must be designed to conduct a volume equal to the difference between inflow volume and storage volume in an appropriate time interval.
Side-Hill Impounding Embankment	A side-hill impounding embankment can have all of the alternatives of a cross-valley impoundment except that the smaller watershed and the potential for diversion significantly reduce the storage and outflow requirements associated with the design storm.
Diked-Pond Embankment	Normally, a diked-pond embankment will have inflow equal to the precipitation falling directly into the impoundment. Total storage with limited or no outflow during the design storm is normally the best solution, although the drawdown requirement must be met by either a spillway, decant pipe, or pumping.
Incised Pond	An incised pond has a water surface below the normal ground surface, and inflow runoff, storage and outflow generally are not critical to safety.

Note: 1. Embankment types are discussed in Chapter 3.

In general, for non-impounding coal refuse disposal facilities or the downstream or perimeter portions of impounding facilities or slurry cells, runoff is conveyed around the facility without retention and storage. On the other hand, impounding embankments are designed to temporarily store runoff from upstream areas and to convey excess flows past the embankment with decant pipes and spillways.

Design criteria for impounding and non-impounding coal refuse disposal facilities include the total volume of runoff from the design storm, as discussed in Section 9.5. For a non-impounding coal refuse disposal facility, the peak runoff rate caused by a flood or the design storm is of prime concern. For an impounding facility, both the peak runoff rate and the total volume of runoff are of concern. In the first case, the hydraulic facilities must be sized to pass the peak runoff rate, while in the latter case, the impoundment and hydraulic structures must be designed to store and pass the total volume of runoff.

The runoff and outflow elements are influenced by a number of critical factors, as discussed in the following section.

9.2.2 Definition and Discussion of Key Runoff Elements

Sources of impoundment inflow are shown in Figure 9.1. These sources also include ancillary flow contributions such as process water (water or water-slurry mixture pumped from the mine or the coal processing plant), indirect runoff from adjacent watersheds, or other diverted flows such as from underground mines. The sources of impoundment inflow can be categorized as follows:

Major Sources

- Direct precipitation – rain or snow falling directly onto the disposal site
- Runoff – from precipitation falling on areas upstream or upgradient from the site and within the watershed associated with the facility

Minor Sources

- Springs from groundwater flow
- Base flow in a stream passing through or by the site that is relatively independent of the most recent rainfall events, but directly related to infiltration associated with earlier rainfall events
- Process water and other pumped flows

Minor sources of flow are typically much smaller than the major sources of runoff. The volumes associated with minor sources can be determined with relative accuracy. However, the amount of runoff resulting from a storm will vary depending upon site location. Geographic location, climatic conditions and watershed characteristics all contribute to storm runoff, as discussed in the following sections.

9.2.2.1 Watershed Boundary and Area

The watershed is all of the catchment area that drains toward a particular point of interest. Watershed boundaries are typically determined from site-specific topographic maps (Section 6.4.1.1) or USGS topographic quadrangle maps, as shown in Figure 9.2.

9.2.2.2 Precipitation

Runoff results from precipitation falling on the watershed, melting of snow already on the ground and outflow from upstream impoundments in the watershed. Snowmelt is usually a minor portion of runoff in small watersheds, such as those usually associated with a coal refuse disposal facility. The effects of upstream impoundments should be considered on an individual site basis.

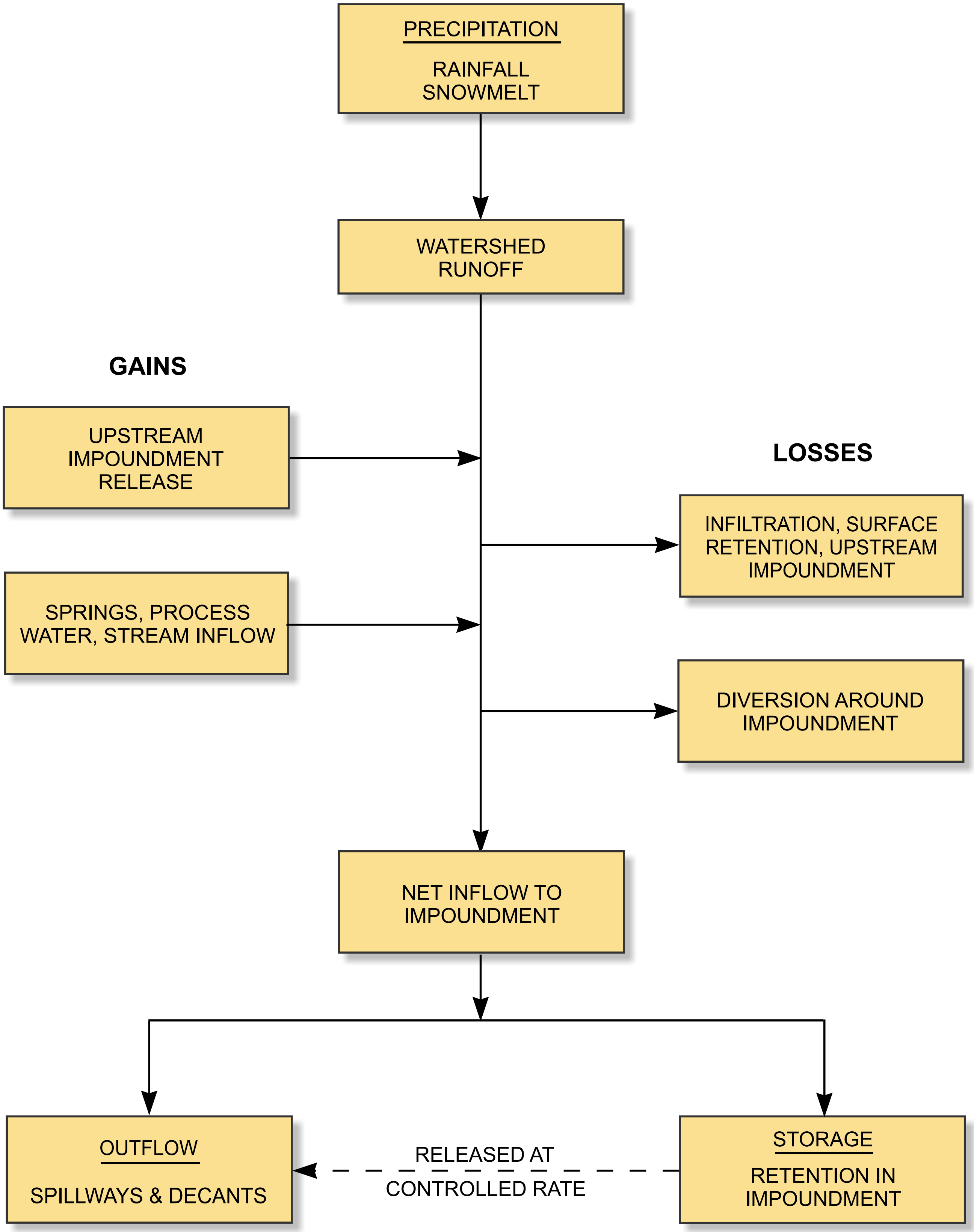


FIGURE 9.1 RUNOFF AND IMPOUNDMENT INFLOW SOURCES

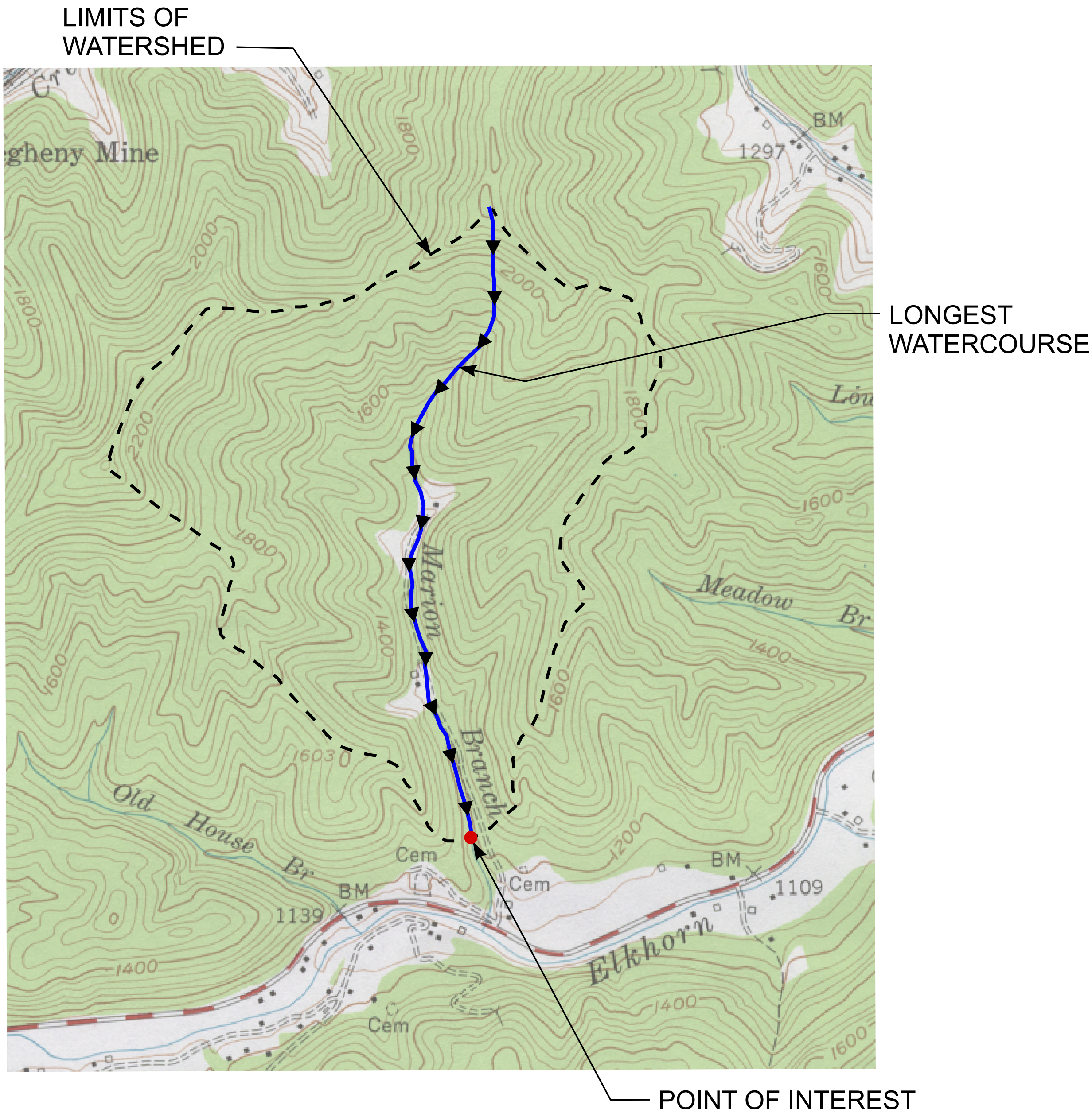


FIGURE 9.2 WATERSHED BOUNDARY DELINEATED ON USGS TOPOGRAPHIC MAP

9.2.2.2.1 Rainfall Curves

Calculation of the design storm rainfall (Section 9.5) involves determination of the total amount and distribution of rainfall for the entire storm duration. The relationships between total precipitation (cumulative rainfall depth), storm duration, and storm intensity (slope of the rainfall distribution curve) have a direct effect on the runoff rate and volume (Sections 9.6.1 and 9.6.2). For example, a sudden short rainfall can result in a high runoff rate and a small total volume of runoff, while a prolonged rainfall of low intensity can produce a large total volume of runoff with a relatively low runoff rate. Coal refuse disposal facilities should be designed to accommodate all possible precipitation/runoff conditions associated with the design storm.

9.2.2.2.2 Rainfall Intensity

The relationships between rainfall intensity, duration of the rainfall event and frequency (i.e., intensity-duration-frequency or I-D-F) can be used to determine the peak runoff, and are useful in the design of hydraulic structures such as culverts, channels and ditches (Sections 9.6.3 and 9.6.4). Only the most intense portion of the rainfall, not the entire storm history, governs the selection of culvert size, the most efficient ditch or channel configuration, and the required erosion protection associated with the runoff flow velocity.

9.2.2.3 Watershed Characteristics

A portion of the precipitation falling on a watershed is retained in the soil and by vegetation or may be retained in upstream impoundments. The portion of the precipitation that flows to the point of interest is termed the runoff. The watershed characteristics that determine the difference between the amount of precipitation falling on the watershed and the amount that becomes runoff include: (1) the types of surficial soils and their effect on infiltration; (2) the condition of the ground surface (e.g., wet, dry, snow-covered or frozen) prior to the precipitation (termed the antecedent moisture condition); (3) the type and density of vegetation; (4) development features such as paved surfaces, channeling, storm sewers, etc.; and (5) the presence of dams, lakes, ponds or swamps upstream from the disposal facility that can either store water and release it at a slow rate or fail and release large volumes of stored water at a high rate.

The runoff hydrograph at the point of interest will vary as a function of the intensity distribution of precipitation and the geometric shape and slope conditions of the watershed area. Inflow and outflow hydrographs for a typical impoundment are shown in Figure 9.3. The figure also shows the net inflow and volume of impoundment storage.

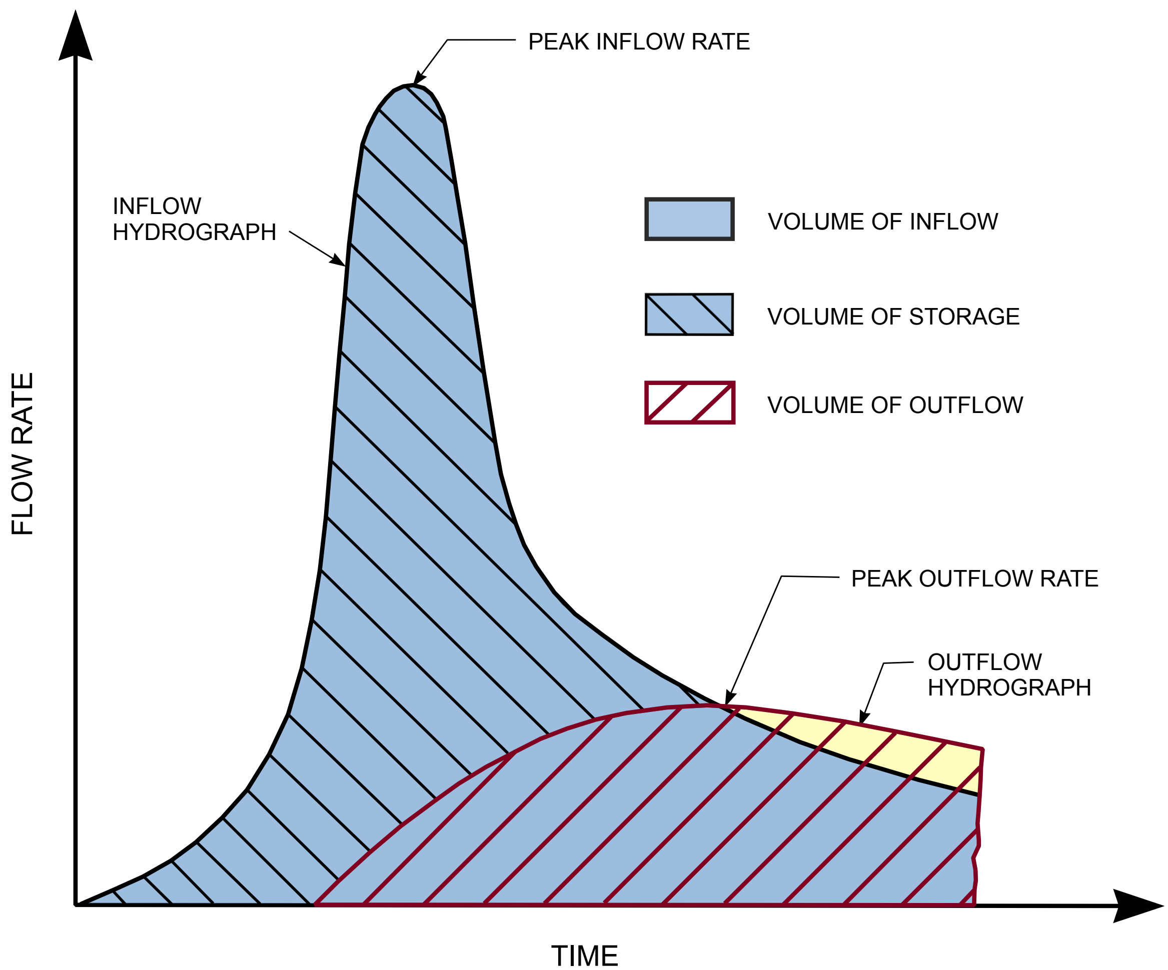


FIGURE 9.3 TYPICAL IMPOUNDMENT INFLOW AND OUTFLOW HYDROGRAPHS

9.2.3 Key Storage and Outflow Elements

The principal factors governing the storage capacity of a reservoir or impoundment are the physical dimensions of the embankment and ground surface and the current level of water/slurry. The outflow capacity is determined by the types and sizes of the hydraulic structures.

9.2.3.1 Impoundment Capacity

The storm storage capacity of an impoundment is the volume of runoff that can be temporarily retained during the applicable design storm. If a refuse disposal facility has minimal storage capacity, the outflow hydrograph will be about the same as the inflow hydrograph, and the hydraulic structures must be designed to transport the peak runoff rate. The primary benefit of impoundment storage is that the outflow rate can typically be reduced, permitting use of smaller hydraulic structures. An additional benefit is that the downstream flooding risk is not exacerbated by an increase in runoff from disturbed areas in the watershed. The potential difference in peak flow rates is illustrated by the inflow and outflow hydrographs shown in Figure 9.3.

Figure 9.4 shows a typical impoundment capacity curve relating storage volume to pool elevation. The figure also shows the relationship between reservoir surface area and pool elevation. Such curves are used to evaluate the storage conditions at any given pool elevation and are prepared as part of the design of an impounding structure (Section 9.7).

Two terms used to describe the limits of acceptable pool elevation are “surcharge” and “freeboard.” Surcharge is the vertical distance between the usual operations level of the impoundment and the maximum allowable water surface elevation. Normal freeboard is the vertical distance between the pool elevation and the top of the embankment at its lowest point (where the dam would begin to be overtopped). Design storm freeboard is the vertical distance between the maximum water surface elevation during the design storm and the top of the embankment. The minimum design storm freeboard is an impoundment design criterion and should be such that waves do not overtop the embankment crest during the design storm. Freeboard also serves to compensate for uncertainty in hydrologic parameters.

9.2.3.2 Decants, Principal Spillways and Auxiliary Spillways

Decants are conduits that extend through an embankment and discharge under controlled conditions at or beyond the embankment toe. As the term “decant” would imply, impoundment water typically enters the conduit by flowing over the top edge of the upstream end. At coal refuse disposal facilities, decants are generally not intended to discharge at high flow rates, but are designed to remove clarified process water, pass base stream flows or to drain the impoundment of stored water after a storm. However, a decant must be sufficiently large that stored water from the design storm can be drained within a reasonable period of time, so that the storage volume needed for a subsequent storm is available. Several types of decant systems are shown in Figure 9.5.

Principal spillways are generally designed to control the discharge associated with large design storms, to limit discharges and associated impacts downstream, and to limit the frequency and duration of flow through the emergency (auxiliary) spillway. Principal spillways are most often associated with fresh water impoundments and sedimentation and treatment ponds, and state regulatory agencies typically provide specific design storm criteria that govern the size and capacity of these structures. In some coal refuse facility designs, the decant may also function as a principal spillway. Principal spillways are designed to: (1) release runoff at a controlled rate, (2) provide settling time for the removal of sediment or process water solids prior to discharge, (3) provide runoff detention, and (4) function as decants to control the impoundment operational pool level. Decant systems are generally not considered in design storm flood routing analyses for determining maximum impoundment pool level, as they do not have significant discharge capacity. If considered in the flood routing analysis, the decant pipe should be of sufficient size that clogging is unlikely (typically, greater than 12 inches in diameter) and should be equipped with a properly designed trashrack.

Auxiliary (emergency) spillways are open channels generally used to discharge that portion of the runoff volume that cannot be stored in the impoundment or routed through the principal spillway. Auxiliary spillways typically are capable of discharging: (1) moderate flows from storms much smaller than the design storm (Section 9.5.1) with little or no damage or (2) large flows resulting from the design storm, where some localized damage may occur, but without the threat of failure of the entire impounding embankment. Typical auxiliary spillway systems are shown in Figure 9.6.

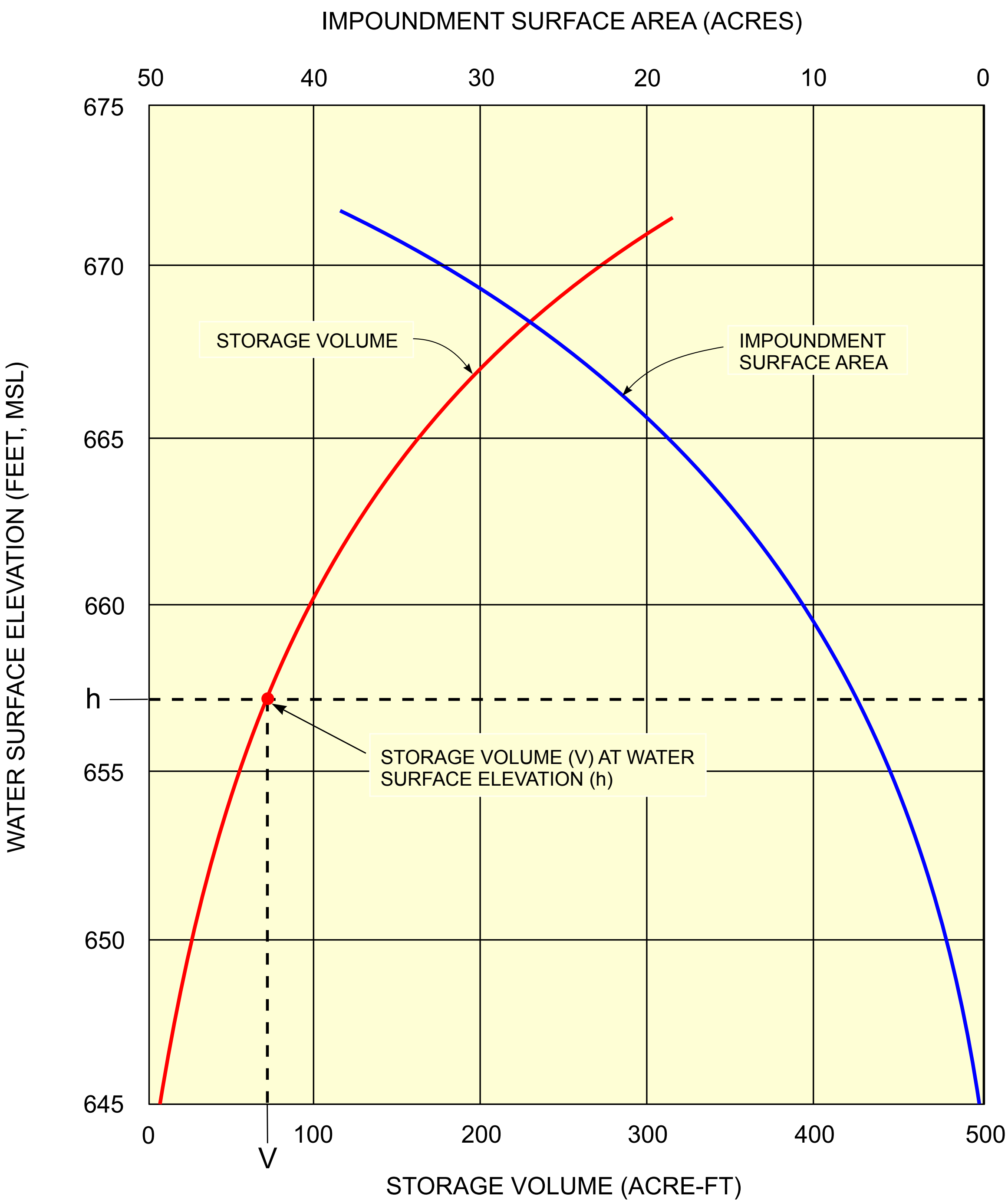
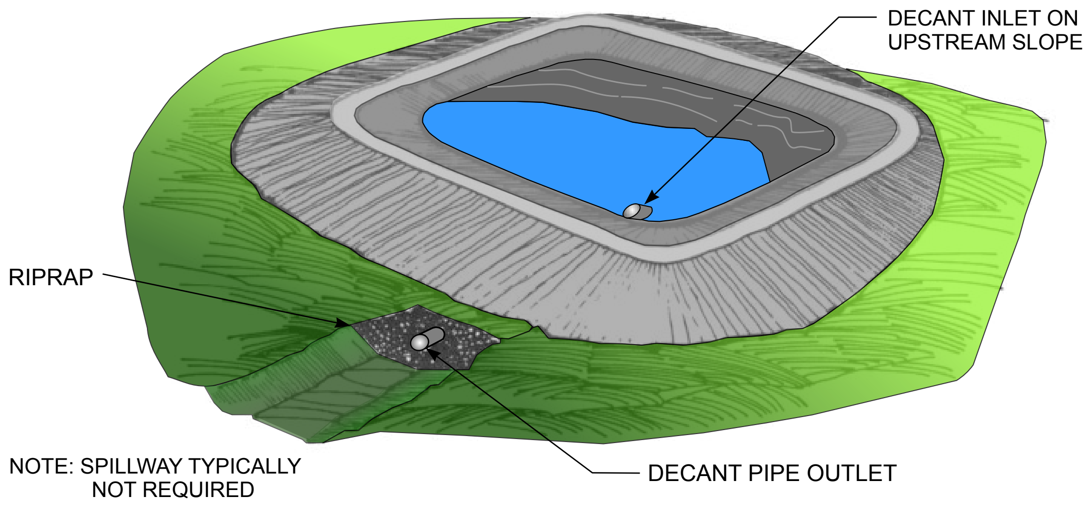
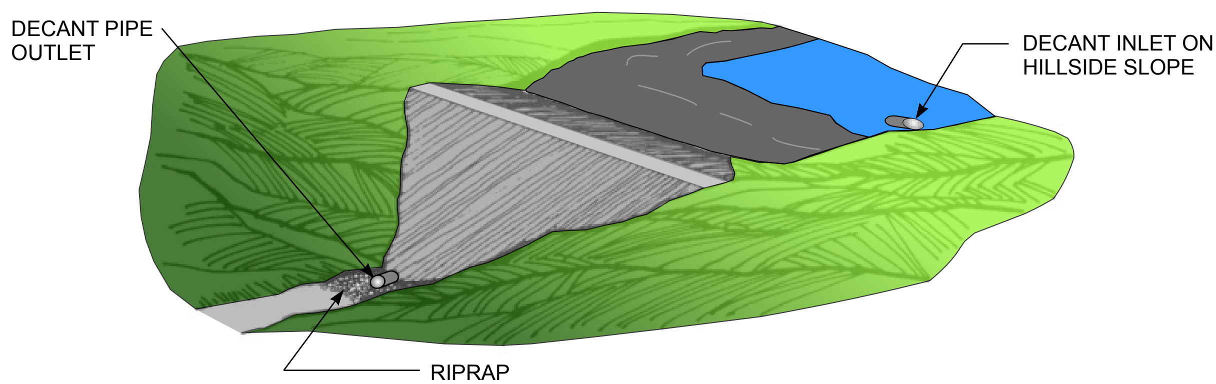


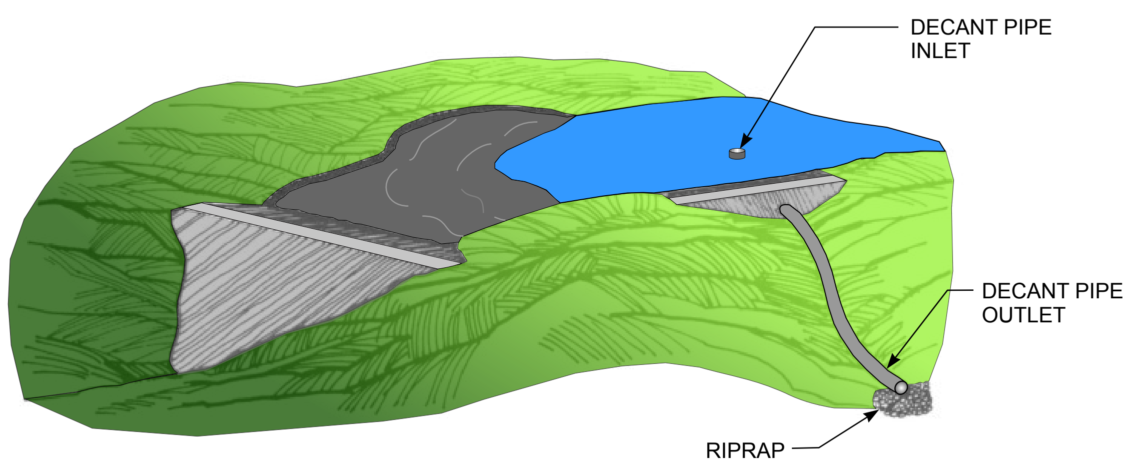
FIGURE 9.4 TYPICAL IMPOUNDMENT AREA AND STORAGE VOLUME CURVES



9.5a DECANT FOR DIKED IMPOUNDMENT



9.5b DECANT THROUGH REFUSE EMBANKMENT



9.5c DECANT THROUGH SADDLE

FIGURE 9.5 TYPICAL DECANT SYSTEMS

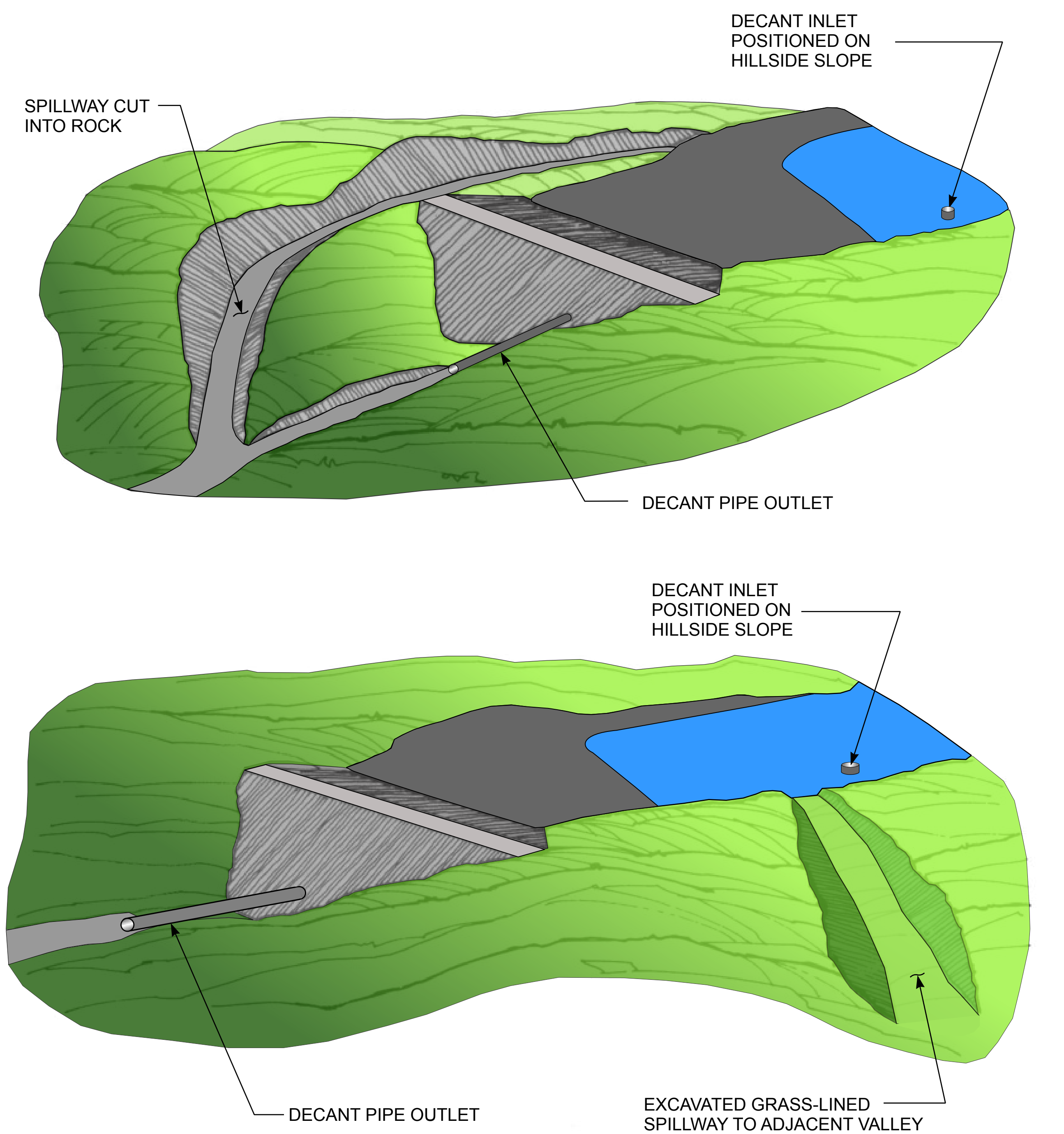


FIGURE 9.6 TYPICAL DECANT AND SPILLWAY SYSTEMS

The design of auxiliary spillways, principal spillways and decants normally requires evaluation of three basic components: the inlet, the transport section and the outlet. Key processes in the design of these systems include:

- Determining which component controls the outflow rate for various flow conditions.
- Sizing each component to function properly for the anticipated range of flow conditions.
- Specifying materials for each component that will not erode excessively under the anticipated flow velocities.

- Designing inlet or transport sections that will not become clogged or otherwise fail, causing major downstream damage or failure of the impounding embankment.
- Arranging the outlet location so that the release of water does not lead to failure of the impoundment embankment or major downstream damage.

The design of outflow systems is further discussed in Section 9.7.

9.3 GENERAL CONSIDERATIONS FOR COAL REFUSE DISPOSAL FACILITIES

9.3.1 Special Characteristics

Table 9.3 lists characteristics that distinguish the design of typical coal refuse disposal facilities from many other structures with appurtenant hydraulic structures. In addition to the special characteristics indicated in Table 9.3, the hydrologic and hydraulic design of coal refuse disposal facilities is also governed by the considerations discussed in the following sections.

9.3.2 Site Conditions

Site selection impacts the cost and difficulty in providing adequate hydraulic appurtenant structures for use during the disposal period and subsequent abandonment of a coal refuse disposal facility. Based upon hydrologic and hydraulic considerations, the best site will almost always have the smallest possible upstream watershed. In some cases, however, hydrologic/hydraulic considerations are secondary to preparation plant location and materials handling requirements. Even if this appears to be the case, the designer should evaluate the needed hydraulic structures considering downstream hazard potential, environmental control and construction costs prior to finalizing the location for a disposal facility. Large initial costs associated with the construction of hydraulic structures may be justified if this allows materials transportation costs to be lowered.

TABLE 9.3 FACILITY CHARACTERISTICS INFLUENCING HYDRAULIC SYSTEM DESIGN

Characteristic	Significance In Design
The facility is designed for disposing coal refuse, with active operations taking place for an associated period of time, and not to collect water for flood prevention, water supply, power, or recreation.	Greater flexibility in choosing location, configuration and construction sequence for appurtenant hydraulic structures.
The facility covers a large area, with the gradient or drainage slope primarily in one direction.	Providing diversion facilities not subject to localized failures or controlled overtopping during large storms is often not economically practical.
The placement of refuse occurs over many years, during which time the facility configuration is constantly changing.	Hydraulic systems must be designed so that they can be expanded or decommissioned and replaced as the facility grows.
The growth rate of the facility is estimated based upon projected quantities of refuse production.	Actual quantities must be evaluated periodically to determine if the rate of construction is adequate.
Water passing over or through the coal refuse can be destructive or environmentally unacceptable.	Proper design requires that potentially adverse environmental effects (e.g., corrosion of construction materials), and the cost of water collection and treatment, be considered in the evaluation of alternative hydraulic systems.
When placement of refuse is completed, the facility typically has no continuing utility, and the hydraulic systems are decommissioned, the impounding capability is eliminated, and the site is abandoned in accordance with mine reclamation requirements.	The sequence of constructing hydraulic systems must provide an arrangement that will function until decommissioning at a specified future date. Planning must allow for the possibility that decommissioning, elimination of impounding capability, and abandonment may be required for a configuration either larger or smaller than originally anticipated.

The site conditions described in the following sections may affect decisions related to the selection and design of facility hydraulic structures.

9.3.2.1 Topography

The importance of topography on the geotechnical aspects of site selection and disposal facility configuration is discussed in detail in Section 6.2.2.1. As discussed in this section, the significance of topography is generally limited to the planning, design and construction of hydraulic conveyance structures.

9.3.2.1.1 Steep Terrain

In areas of steep and rugged terrain, many disposal facilities must necessarily be located in valleys formed by small streams. Two very significant problems may be encountered with respect to placing diversion ditches, spillways and conveyance channels in these areas:

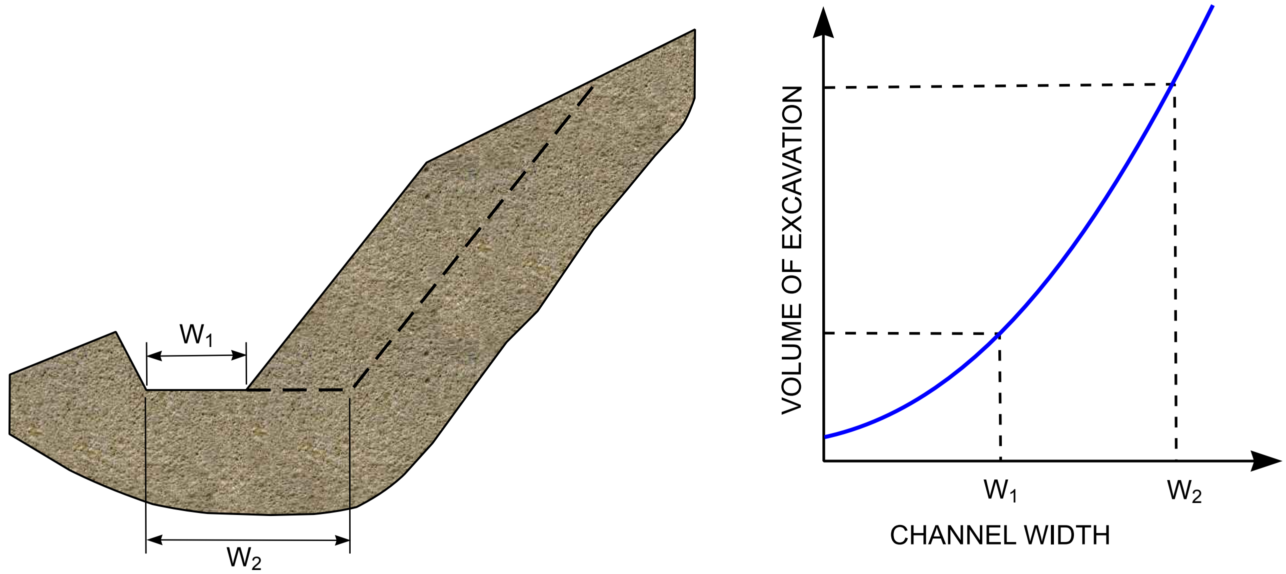
- Channels cut into the side of the valley will require the excavation of large amounts of material, as illustrated in Figure 9.7a. With increased channel width, the cut becomes more extensive and the slope of the cut must often be decreased to achieve stability. These conditions combine to increase excavation quantities and costs disproportionately to the flow capacity gained.
- The potential for sloughing of overburden soil or weathered rock into the channel, thus restricting its flow capacity, is increased, as illustrated in Figure 9.7b. Major sloughing will often occur during a heavy rainstorm when a large flow capacity is desired. The possibility of main spillway channels becoming obstructed by sloughing must be considered in the geotechnical analysis and design of the cut slope.

Diversion ditches for non-impounding coal refuse embankments are designed based on the design storm (100-year-recurrence-interval storm). For impounding coal refuse facilities with more extreme design storms such as the Probable Maximum Precipitation (PMP), it is usually not feasible to design perimeter diversion ditches large enough to pass the maximum flow. While diversion ditches for impounding facilities still perform an important function, the hydraulic design of the impoundment generally is based on the assumption that during large floods the diversion ditches will be overtopped and the resulting overflow will enter the impoundment. However, such overtopping should not be permitted to occur if flows in excess of the diversion ditch design storm could cause erosion of the dam and spillway.

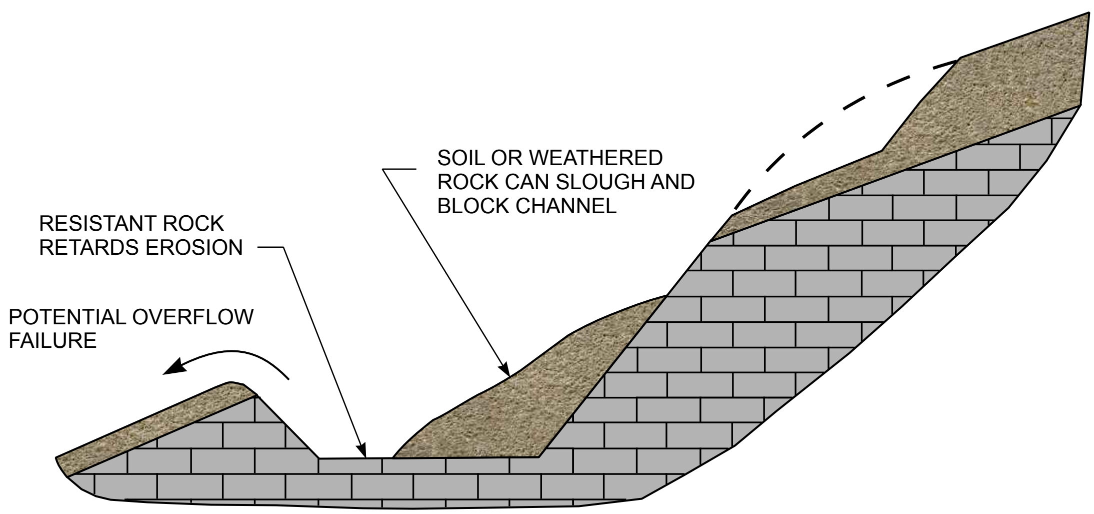
Although there are undesirable aspects to cutting channels into steep slopes, there may also be significant advantages. Bedrock is normally found near the surface in rugged terrain. Thus, channels cut in such areas will often be resistant to erosion without special protection. A channel should be located where its base will be on the most resistant material. If possible, channels should be constructed in sound rock, particularly where flow velocities will be erosive and where failure of the channel would create an unsafe general condition or large repair costs. Another advantage that may be realized from cutting channels into such slopes is the concurrent production of borrow materials suitable for use as resistant drainage material.

9.3.2.1.2 Gently Sloping Terrain

In gently sloping terrain, the disadvantages associated with hillside channel excavation are not as pronounced as in steep terrain. As shown in Figure 9.8a, the volume of excavation is a nearly linear function of channel width. In addition, achieving stability of the uphill cut slope is not as difficult as for channels cut into steep hillsides. However, as illustrated in Figure 9.8b, these areas often do not have rock near the surface. Therefore, the channels are more susceptible to erosion unless flow



9.7a EXCAVATION VOLUME VS. CHANNEL WIDTH



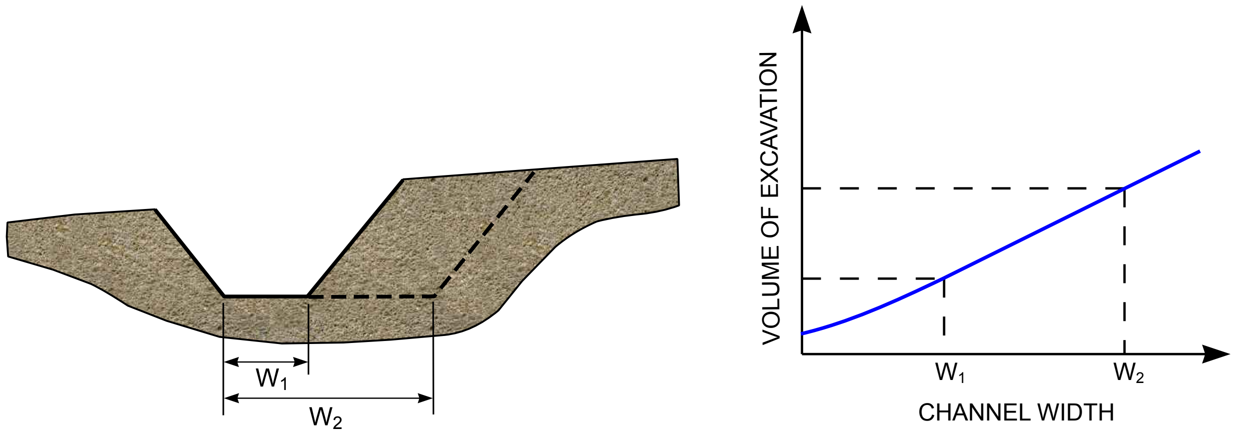
9.7b CHANNEL EROSION AND OVERBURDEN SLOUGHING

FIGURE 9.7 CHANNEL CONSTRUCTION IN MODERATELY AND STEEPLY SLOPING TERRAIN

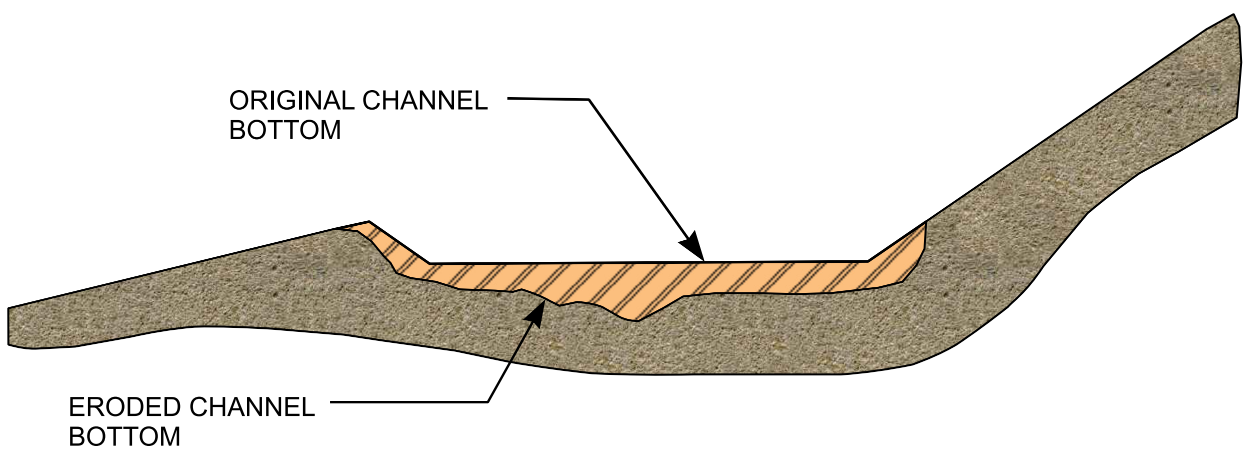
velocity can be kept low or some type of stabilization system (e.g., channel lining) is provided. A key to economical design in this case is to minimize the length of channel sections where flow velocities exceed the natural erosion resistance of the channel.

9.3.2.1.3 Effects of Slope on Facility Staging

The combination of site topography and the constantly increasing size of coal refuse disposal facilities often creates design problems not normally encountered with other water-impounding facilities. For example, the previously mentioned difficulties involved in excavating wide auxiliary spillway channels in steep slopes are multiplied when the design requires that multiple auxiliary spillway channels be excavated as the height of the embankment is increased in subsequent stages. Problems



9.8a EXCAVATION VOLUME VS. CHANNEL WIDTH



9.8b CHANNEL EROSION

FIGURE 9.8 CHANNEL CONSTRUCTION IN GENTLY SLOPING TERRAIN

can also occur when it becomes necessary to tie an embankment into an excavated rock face, as opposed to the original soil cover on the natural slope. Tying the embankment material into the steep and broken rock increases the potential for future problems related to seepage, leakage and embankment stability.

For cases where multiple auxiliary spillways are required with construction of succeeding stages, the designer may wish to consider a series of cascading spillways. A new embankment stage with its associated spillway channel can be configured to extend a sufficient distance downstream to allow the outflow to drop into the spillway channel of the preceding stage with the addition of a plunge pool. For such configurations, the hydraulic design of the channels and plunge pool and the erosion resistance of the rock must be carefully evaluated.

In some cases, the topography may permit an open-channel spillway to be located away from the embankment, such as through a saddle in a ridgeline, so that the flow is discharged into an adjacent watershed, as shown in Figure 9.6. This arrangement can be beneficial in that potential issues associated with flow escaping from the spillway channel and adversely affecting the downstream face of the embankment are avoided.

9.3.2.2 Weather and Climate

Weather and climatic conditions should be considered as part of the planning associated with the design and construction of hydraulic structures. Specific examples include:

- In most coal regions of the United States, construction of channels, ditches, concrete spillways and decant systems should be scheduled to the extent possible during normal construction seasons and should be avoided in winter, when freezing conditions and snowfall may interrupt construction. Accordingly, the staging of any disposal facility should be planned so that there is adequate flexibility to allow extensions, replacement or modification to hydraulic systems during favorable weather even though coal refuse is handled and disposed on a year-round basis.
- Many western coal fields are in arid or semi-arid climates where the growth of vegetation is a very slow process. In these areas, using vegetation as a means of erosion protection in excavated channels may not be practical.

9.3.2.3 Geology

Normally, geologic conditions do not change drastically within a small geographic area, and thus they generally do not directly affect disposal facility site selection alternatives. However, soil and rock conditions at a site are always important to the design of hydraulic structures and often are the deciding factor in choosing among several hydraulic system alternatives with similar cost and utility characteristics. The following are important geologic and geotechnical factors that must be considered in design:

- If excavated channels in steep slopes are being considered, the designer should evaluate the stability of the cut slopes. If excessive costs will be required to achieve stability, either by benching or by constructing retaining systems, an alternative system may be more cost effective.
- If an excavated channel is to be located along a hillside, it should have a sufficient capacity that overtopping or discharge that could cause cascading water to flow onto a critical portion of the embankment does not occur. If a bend in the channel is required, the effects of flow, erosion, and water superelevation caused by the change of direction should be carefully evaluated. The outlet end of a spillway channel should be located sufficiently far downstream that the discharge will not erode the downstream face of the embankment.
- Channels should be designed to resist potential erosion effects, so that post-construction stabilization is not required.
- If hydraulic structures are to be constructed in or over soft soils or soft coal refuse, the amount of settlement that could occur should be estimated in order to determine whether special construction will be required. Similar considerations may arise in situations where differential settlement may occur, such as where hydraulic structures are constructed across rock abutments and onto fill materials. This is especially important for conduits through an embankment. Where possible, such conduits should be founded on and properly bedded in firm materials that will not settle significantly. Where settlement is unavoidable, the initial slope, camber, joints and conduit material should be selected such that anticipated settlements can be accommodated without damage to the system.

The effect of geologic conditions on runoff during storms is discussed in Section 9.6.1.

9.3.3 Construction Materials

The selection of construction materials for hydraulic structures should account for the following:

- The potential for corrosion of construction material is high at many coal refuse disposal facilities because of the chemical characteristics of water seeping through refuse materials. Choosing corrosion-resistant materials with higher initial cost may be far less expensive over the long term than repairing a deteriorated structure several years after its original installation, especially if the structure will be buried under many feet of refuse.
- Any conduit or structure beneath or within an embankment should be designed for the external pressure of the maximum potential height of the embankment above it and for deformations that may result from embankment construction.
- Channel lining material should be selected to be resistant to the maximum anticipated flow velocities with provisions for drainage and resistance to uplift pressures.
- Filter criteria for all materials used in the embankment and appurtenant structure construction should be evaluated so that the potential for erosion and piping within the embankment or loss of structural support and/or failure of the hydraulic conveyance structures is minimized.

9.4 DESIGN CONSIDERATIONS FOR DISPOSAL FACILITY EMBANKMENT TYPES

In addition to the general design considerations discussed in Chapter 5, there are specific hydrologic and hydraulic design considerations for each type of coal refuse disposal facility embankment. The following discussion of facility-dependent hydrologic and hydraulic design considerations is a general summary of the most common considerations for each type of disposal facility.

Some of the primary hydraulic system functions common to all refuse disposal facilities are listed below. The type and configuration of the coal refuse disposal facility determines the significance of each function.

- Collection of runoff from the watershed above the embankment and from the surface of the embankment.
- Control, conveyance and discharge of collected water to a downstream location.
- Control of the embankment slope utilizing benches at 50-foot or lower vertical intervals to reduce potential erosion.
- Erosion protection of the embankment surface during initial, interim and reclamation stages, especially along the embankment face.
- Protection of streams or wetlands from encroachment or other potential environmental impacts that may require mitigation.
- Protection of downstream water quality from sediment-laden runoff, leachate from internal drain collection systems, or collected seepage.

The specific impact of the above hydrologic and hydraulic design considerations is discussed in the following sections. While typical figures are presented to assist in recognizing specific conditions, they do not depict all design situations.

9.4.1 Non-Impounding Embankments

Non-impounding embankments are used for the disposal of coarse, combined, and dewatered fine coal refuse. A non-impounding coal refuse disposal facility is designed such that no fine coal refuse slurry, process water or direct or indirect runoff can accumulate within or upstream of the disposal facility limits. General types of non-impounding coal refuse embankments include valley, ridge, side-hill and heaped fills.

9.4.1.1 Valley-Fill Embankments

Valley-fill refuse embankments, as illustrated in Figure 9.9, are often constructed by starting disposal at the upper end of a valley and extending the embankment in stages down the valley in such a manner that an impoundment is never created. Often these types of embankments are located in large valleys so that large refuse disposal volumes can be placed. The potential runoff in such valleys during a large storm event can be high, and to prevent excessive erosion large diversion channels may be needed. A key design objective associated with the collection of watershed and embankment surface runoff and the discharge of the collected water at a downstream point is to provide the optimum balance between channel cross section and slope, thereby minimizing the cost associated with channel erosion protection.

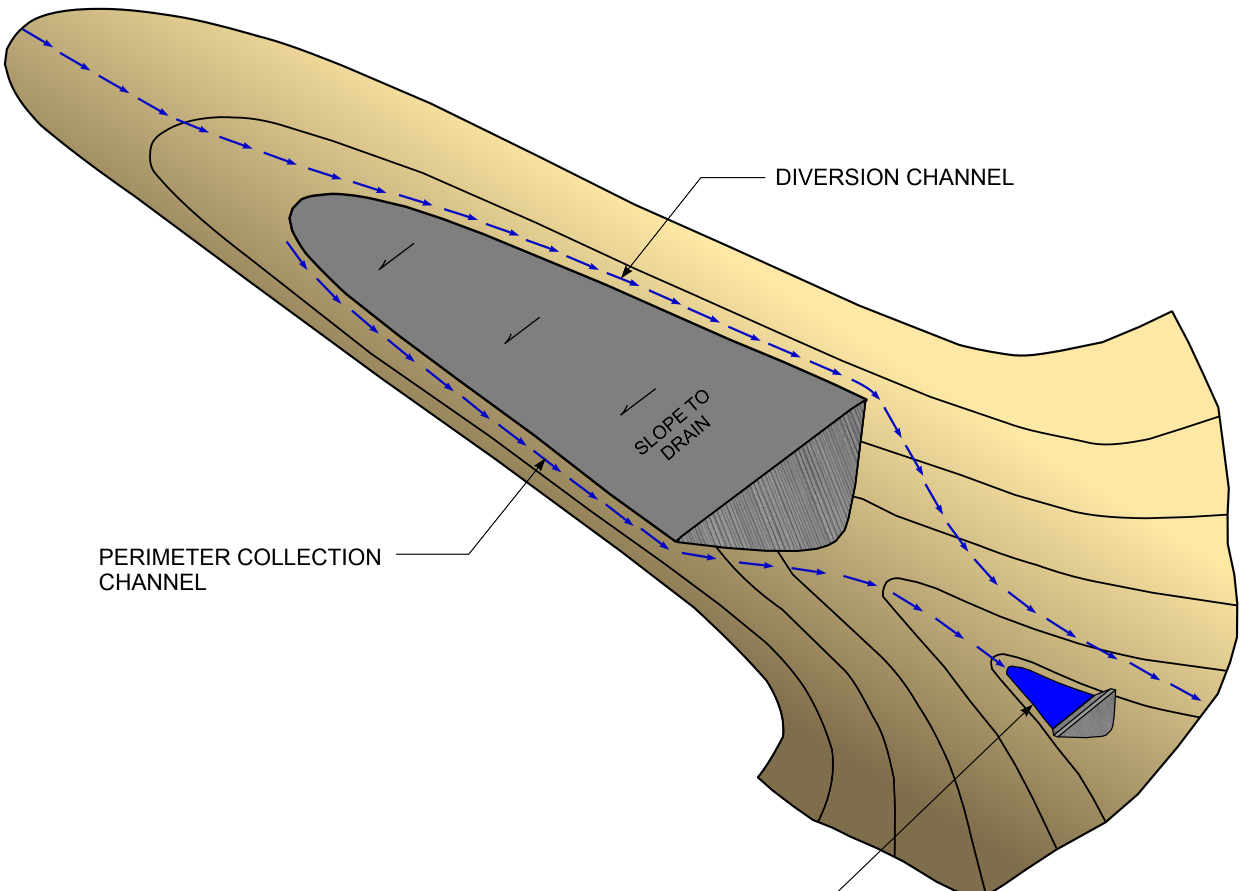


FIGURE 9.9 DRAINAGE CONTROL FOR VALLEY-FILL, NON-IMPOUNDING EMBANKMENT

The most difficult portion of the channel design is along the embankment face at the interface of the coal refuse and the natural ground surface where the steep slope typically results in high velocities. If practical, the channels should be extended along the valley wall, within natural soil and rock, beyond the limits of the coal refuse embankment to discharge beyond the embankment toe. If such an extension is not practical, it is normally necessary to construct a lined or otherwise protected channel at the interface of the refuse embankment and valley wall to carry the runoff safely to the valley floor. The long diversion/collection ditches along the crest of the disposal facility should be designed with a base width and slope that allows use of grass-lined channel sections, if possible.

The mixture of runoff, leachate and seepage may require treatment prior to discharge to the receiving waterway. Such treatment could entail construction of sedimentation ponds and also ponds for chemical treatment. Sufficient area for construction of the sedimentation/treatment ponds should be allocated. However, these facilities should be located above the level of the 100-year floodplain associated with the receiving stream and not in a position where they could be affected by normal stream flows.

9.4.1.2 Side-Hill, Ridge and Heaped Embankments

Side-hill, ridge and heaped non-impounding embankments have design configuration considerations similar to those for valley-fill embankments. The upstream and perimeter watersheds are generally smaller than for a valley-fill embankment, but the steepness of the final embankment slopes and the water quality of the runoff and seepage result in similar hydrologic and hydraulic design considerations as for a non-impounding, valley-fill embankment. Figure 9.10 shows drainage control for side-hill and heaped embankments.

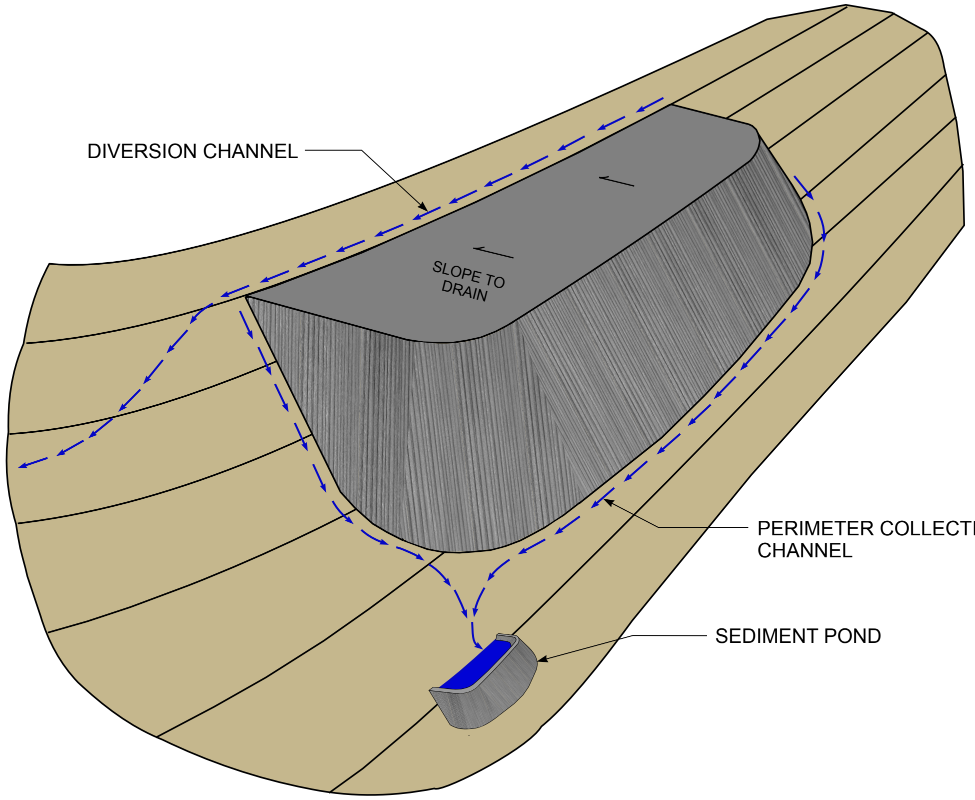
Side-hill embankments are usually constructed in stages that extend progressively higher on a natural slope. Therefore temporary diversion ditches are needed for collecting and diverting runoff at intermediate stages when the embankment is not at full elevation. The channel dimensions, slope and required erosion protection should be designed to meet the final conveyance requirements and to provide economical erosion protection. The location of the toe of the embankment should lie outside the 100-year floodplain limits of nearby streams to minimize any potential encroachments, and sufficient area should be available for sediment/treatment pond construction.

Ridge embankments are generally in the upper reaches of a watershed and may resemble a side-hill embankment extending above and over a ridge line. The collection and conveyance of precipitation falling directly onto the embankment is the primary issue since there is typically little if any upstream watershed. This type of facility generally has a limited downstream area available for sedimentation control and chemical treatment, and the natural ground surface may slope away from the disposal facility in several directions and potentially into other watersheds. Therefore, multiple sedimentation ponds and pumping to a common point for treatment may be required.

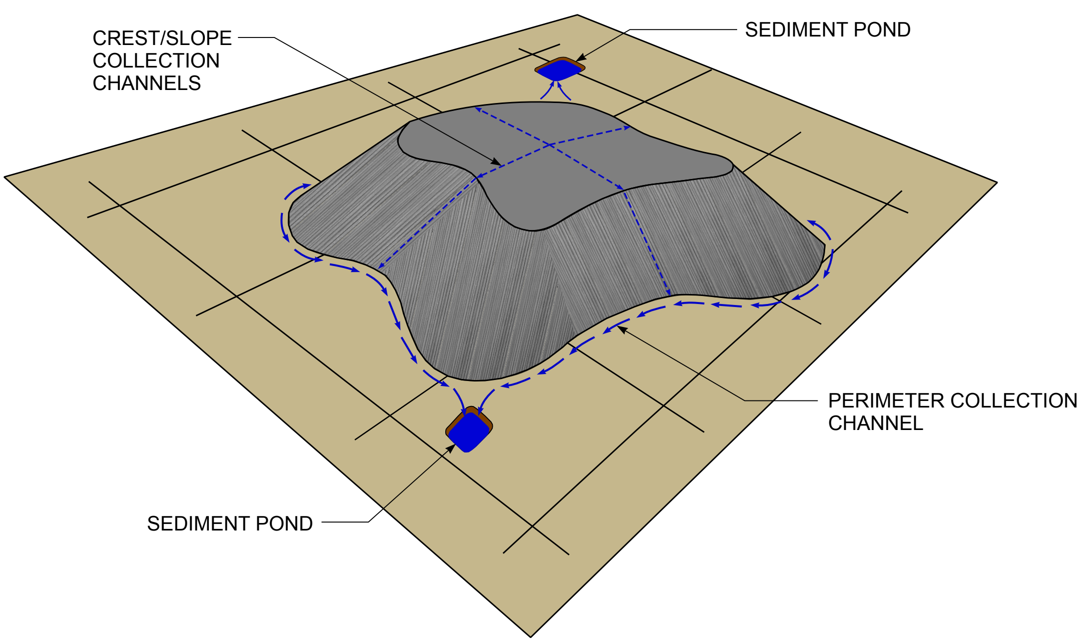
Heaped embankments are generally located on flat terrain. The collection and conveyance of runoff is primarily related to conveying precipitation that falls directly onto the facility and diverting adjacent area runoff away from the facility. Collection ditches on benches and at the crest are typically gently sloping, and grass-lining can normally be used as the channel erosion control. Ditches conveying runoff from the crest or benches to the toe of the embankment are steeper than the collection channels and typically require a more durable lining material such as riprap, concrete or manufactured erosion protection material. Also, the outlet structure must be sufficiently oriented and properly designed to prevent erosion of the embankment toe. For high embankments, special consideration is required at the discharge points so that the energy of the high velocity flow is dissipated and/or the flow is directed away from the embankment in a manner that prevents erosion of the toe.

9.4.2 Slurry Cell Embankments

The hydrologic and hydraulic aspects of slurry cell embankment design must accommodate the volumetric sequencing of the slurry cells as well as the collection and conveyance of both runoff around the cells and direct runoff that accumulates within the slurry cells. Individual slurry cell design must meet structural and hydraulic design requirements, and construction must be controlled in such a manner that the slurry cells do not become a large interconnected impoundment. The slurry cell concept is based on limiting the total capacity of all open cells (and flowable material if present in closed cells) to a level that is consistent with a low-hazard-potential classification for the facility or does not meet the criteria for a regulated impoundment provided in 30 CFR § 77.216.



9.10a SIDE-HILL EMBANKMENT DRAINAGE CONTROL



9.10b HEAPED EMBANKMENT DRAINAGE CONTROL

FIGURE 9.10 DRAINAGE CONTROL FOR SIDE-HILL AND HEAPED EMBANKMENTS

In order for a slurry cell embankment with multiple cells to not require an approved impoundment plan in accordance with the criteria in 30 CFR § 77.216, each individual cell must not exceed the 20-acre-feet size criterion. Furthermore, where the failure of one cell can result in the failure of another, or where slope failure can result in the release of water or slurry from multiple cells, the cumulative storage capacity of the affected cells must not exceed 20 acre-feet. In situations where multiple cells are operated or arranged such that they may interact and exceed the 20-acre-feet limit, the embankment should be classified as impounding and should be designed for the appropriate design storm based on its hazard classification. A critical consideration in determining the hazard classification for an impounding embankment is the flowability of the fine coal refuse. Generally, slurry cells work most effectively when the depth of fines in the cells is kept relatively shallow, preferably to five feet or less, such that after dewatering and capping the material is unlikely to be flowable. In instances where there is concern for draining of the fine coal refuse, the following guidance for assessing the flowability of fine coal refuse is suggested:

- Fine refuse is generally considered flowable for: (1) operating cells with active fine refuse disposal, (2) non-operating cells containing predominantly saturated, fine refuse deposits that have not been covered, and (3) covered cells with predominantly saturated fine refuse deposits characterized as very loose sand or very soft silt or clay.
- Fine refuse is generally considered non-flowable for: (1) non-operating cells with predominantly unsaturated, fine refuse deposits that have been covered and (2) covered cells with predominantly saturated fine refuse deposits characterized as medium dense sand or medium stiff silt or clay.
- Fine refuse should generally be considered flowable, unless additional testing and analysis demonstrates that it is non-flowable, for non-operating cells with predominantly saturated fine refuse deposits characterized as loose sand or soft silt or clay.

Michael et al. (2005) in an OSM report prepared a review of the flowability of impounded fine coal refuse that discusses recent work and ideas in the engineering profession.

The major hydrologic and hydraulic considerations for slurry cells are the collection, conveyance and discharge of runoff within the main diversion and perimeter ditches plus the discharge of direct runoff from individual slurry cells. As ditches are relocated and new cells are constructed at higher elevations, care should be taken so that the embankment is not advanced vertically to the extent that its impounding capacity exceeds the disposal plan criteria and affects hazard classification. Special consideration is required at the discharge points to control flow and prevent erosion of the embankment. The location of the toe of the embankment should lie above the 100-year floodplain limits of nearby streams in order to minimize the potential for encroachments, and sufficient area should be available for sediment/treatment pond construction. Figure 9.11 shows drainage control measures for a typical slurry cell facility.

9.4.3 Slurry Impoundments

The primary hydrologic/hydraulic issue associated with slurry impoundment design is the continuous balancing of coarse coal refuse disposal, fine coal refuse slurry disposal and maintenance of storm water runoff storage/routing capacity. Direct runoff at a slurry impoundment is typically controlled by a decant system or principal spillway, although some disposal facilities also employ an auxiliary (or emergency) spillway. The operation and performance of these outlet works is integral to fine and coarse coal refuse disposal and the safe operation of the impoundment. To protect the impounding embankment from erosion, perimeter runoff control structures must also be incorporated into the design. The location of the toe of the embankment should lie outside the 100-year floodplain limits of nearby streams to minimize any potential encroachments, and sufficient area should be available for sediment/treatment pond construction.

The type of coal refuse disposal facility configuration (e.g., cross-valley, diked or incised impoundment) is typically a function of topographic conditions in the vicinity of the coal mine. Frequently, a decant system and storage are used to control runoff and thus minimize costs associated with other types of outlet structures. However, this requires sufficient embankment materials to achieve the required storage and may not be feasible for large watersheds. Therefore, some impoundments with large watersheds have auxiliary (or emergency) spillways in combination with planned storage capacity and a decant system to control runoff from the design storm.

Regardless of the outlet structures chosen for various impoundment development stages, special consideration must also be given to the conditions that will exist when the site is no longer maintained as an impoundment. At that point, the impounding capacity must be eliminated by: (1) backfilling the impoundment (typically with coarse coal refuse), (2) excavating a channel through the embankment to the level of the backfilled stabilized fines, or (3) a combination of these methods, which is typically the most effective approach. The approach taken must include measures to prevent significant erosion.

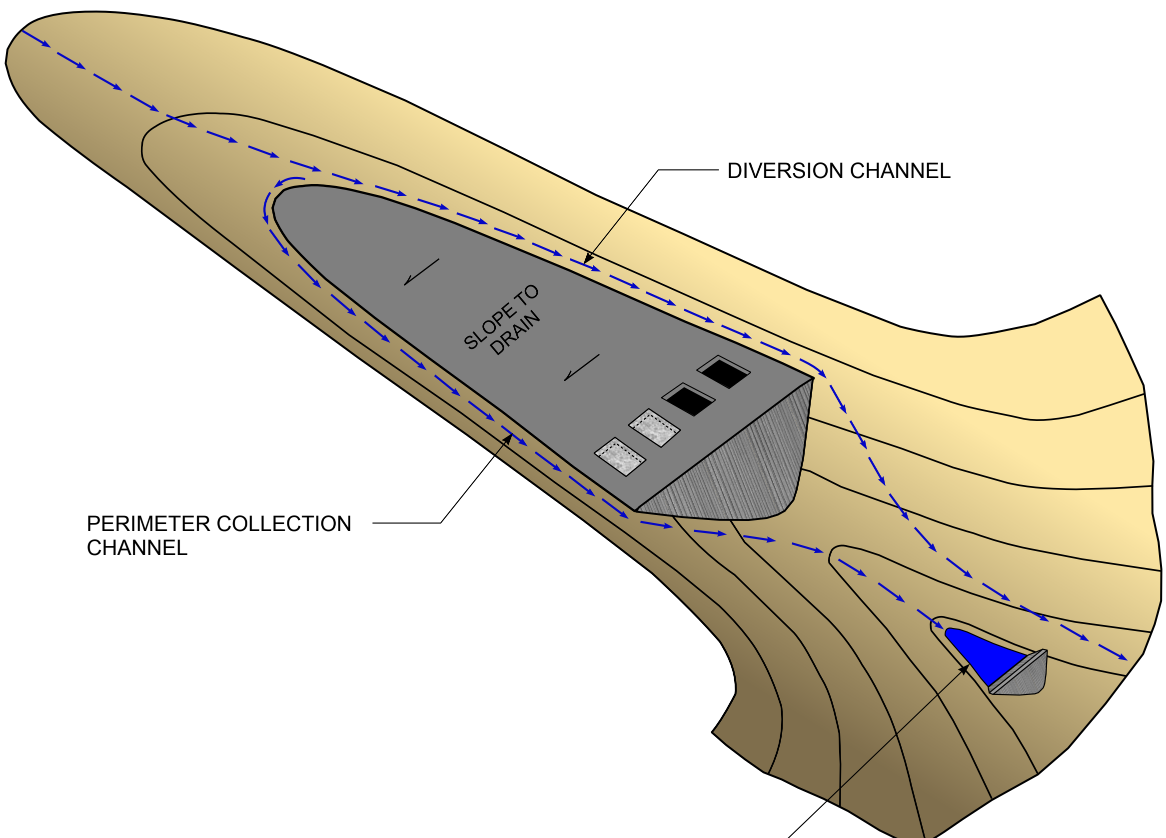


FIGURE 9.11 DRAINAGE CONTROL FOR SLURRY-CELL FACILITY

9.4.3.1 Cross-Valley Impoundment

A cross-valley impoundment typically consists of an embankment constructed primarily of coarse coal refuse that functions as a dam to impound a mixture of settled fine coal refuse, slurry, clarified water and runoff. The impoundment storage and outflow capacity determine the hydraulic structures needed for controlling runoff.

The most appropriate method for minimizing the spillway construction effort is to provide a very large surcharge capacity between the initial pond elevation and the initial embankment crest. A spillway can then be constructed at a significant height above the initial pond level, providing adequate surcharge capacity for a long operational period before the hydraulic system must be expanded. Coarse coal refuse typically provides the material for economically constructing this surcharge capacity. An extension of this approach would be to initially provide for total storage with no requirement for a spillway (although with this approach there must be provisions for drawing down the reservoir in response to consecutive or repeated storms).

Regardless of the percentage of runoff to be handled through reservoir storage, the design configuration must always accommodate the continual rise in the normal pool level due to the disposal of fine coal refuse slurry. Reduction in reservoir storage capacity due to upstream construction pushouts and stages must also be taken into account. A decant system allows the controlled discharge of surcharge runoff. It may also be used to evacuate clarified slurry water. Depending upon the configuration of the impoundment, an open-channel spillway may be needed to discharge runoff from larger storm events.

To protect the downstream face of the coal refuse embankment from erosion, perimeter runoff that is intercepted by embankment bench gutters, road gutters and collection and diversion ditches must be controlled and routed to a sediment/treatment pond. The conveyance structure configuration and erosion protection should be designed to be appropriate for all stages of development, including reclamation. Some typical drainage control measures for a cross-valley impoundment are illustrated in Figure 9.12.

9.4.3.2 Diked Impoundment

Diked impoundments have design constraints similar to those for cross-valley impoundments. If a facility is completely diked such that there is no upstream watershed, the required impoundment surcharge capacity is minimized, and the primary factor affecting the impoundment storage capacity is the production of fine coal refuse and clarification of slurry. Typically, a decant system and/or principal spillway are adequate for control of runoff. If an auxiliary spillway is employed, the channel section through the embankment requires erosion-resistant linings.

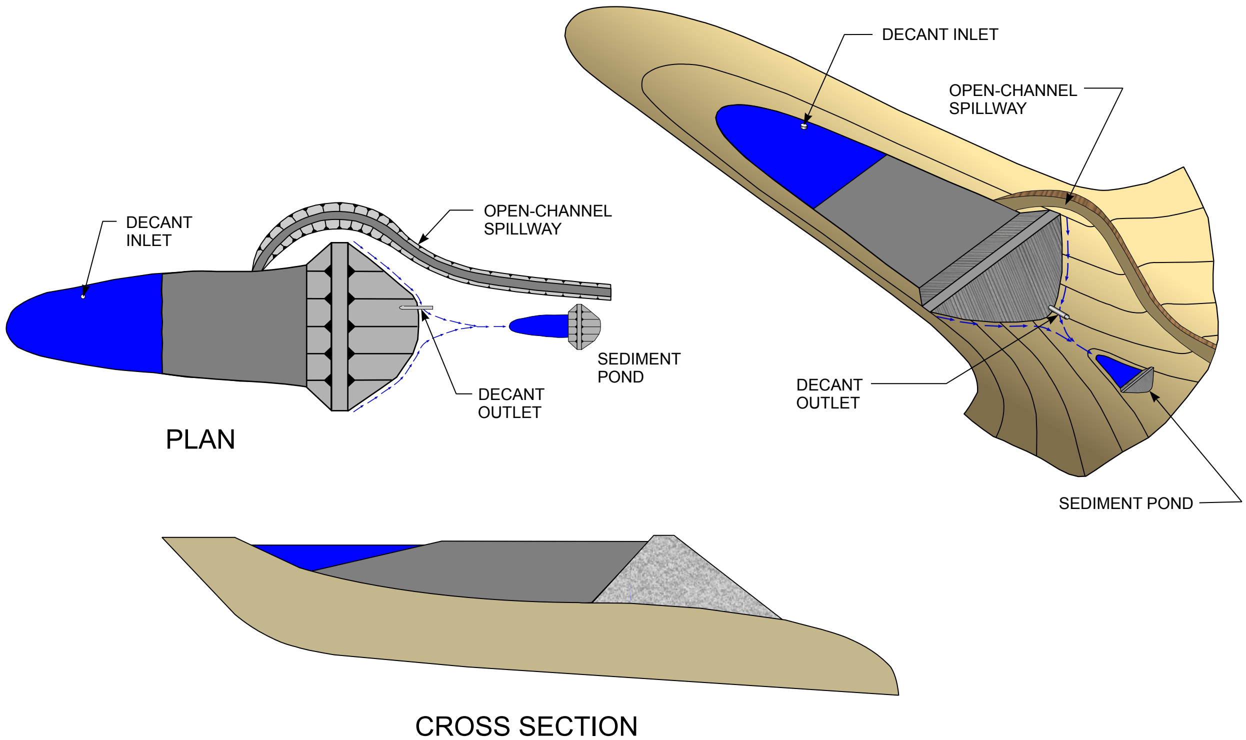
Perimeter ditches and bench gutters tend to be of substantial length and should be designed with sufficient slope to adequately convey runoff to sedimentation ponds and to drain effectively without low areas. Where ditches traverse embankment slopes, they should be provided with erosion-resistant linings. Figure 9.13 shows drainage control measures implemented for a typical diked impoundment.

9.4.3.3 Incised Impoundment

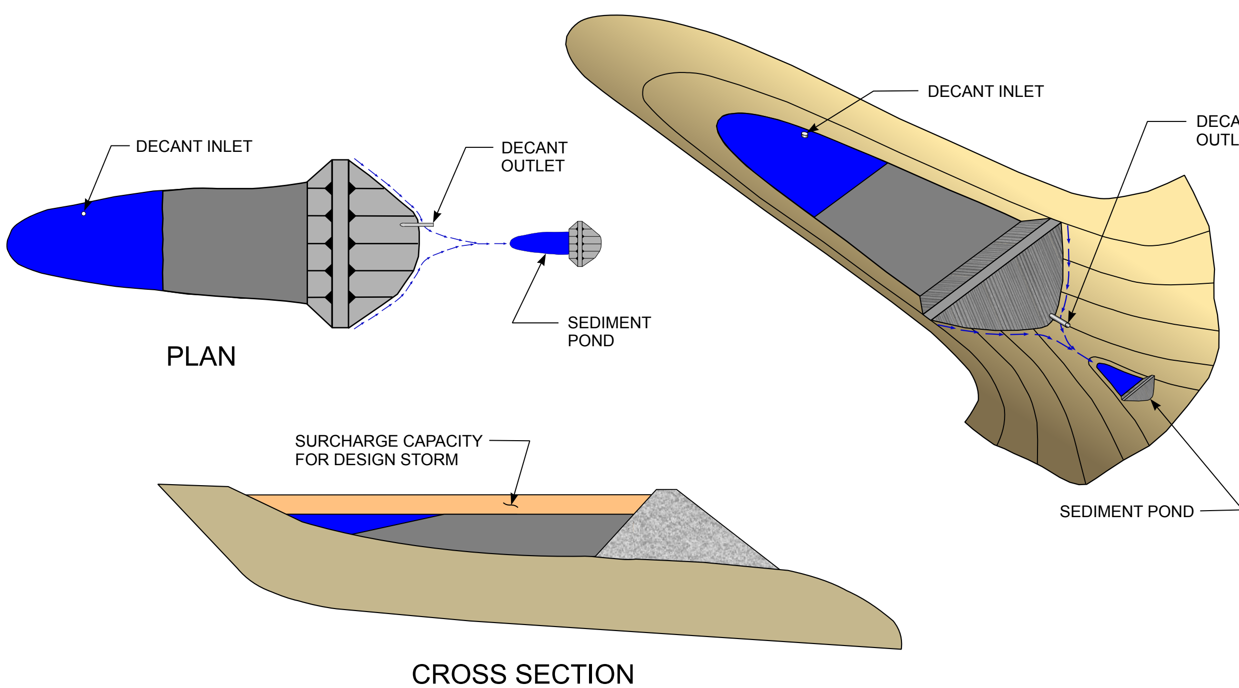
Incised impoundments, or ponds, are used for the disposal of fine coal refuse. They are typically small and often used for temporary or emergency disposal. The hydrologic and hydraulic considerations associated with cross-valley impoundments and diked impoundments are generally not major issues for incised ponds because of the reduced risk of catastrophic failure. There are three principal design considerations: (1) an outlet structure to decant or control the release of clarified process water, (2) diversion to convey adjacent area runoff around the incised pond, and (3) flooding potential, if the incised pond is located close to or within floodplain limits.

9.4.4 Other Impounding Structures

Coal mining operations generally include sedimentation, treatment and fresh water ponds. The capacity of each of these structures is a function of the intended use. Sedimentation or treatment



9.12a CROSS-VALLEY IMPOUNDMENT WITH OPEN-CHANNEL SPILLWAY



9.12b CROSS-VALLEY IMPOUNDMENT WITH DESIGN STORM STORAGE AND WITHOUT OPEN-CHANNEL SPILLWAY

FIGURE 9.12 DRAINAGE CONTROL FOR CROSS-VALLEY IMPOUNDMENT pond capacity is related to the ability of the structure to remove constituents such as suspended solids or metals that exceed effluent limitations. Fresh water ponds must have the reservoir capacity to meet the coal processing and other mining requirements. The size (height and reservoir storage capacity) and downstream impacts of failure of these structures determines the hazard potential and, as a consequence, the design criteria.

9.4.4.1 Sedimentation and Treatment Ponds

Sedimentation ponds and treatment ponds are typically located beyond the toe of coal refuse disposal facilities or below mining-disturbed land, so that they can receive gravity inflow. The sediment and settling capacity of these structures is typically specified in state erosion and sedimentation control guidelines and effluent limitations. Similarly, treatment pond size is dependent on the pond's ability to treat/remove and discharge acceptable water quality. Pond principal and auxiliary spillway structures should be designed to discharge water at a rate consistent with design storm criteria and state regulatory requirements. A primary consideration is the maximum anticipated runoff associated with the embankment staging based on watershed size, hydrologic considerations, and the surcharge storage capacity, which is significantly less than the gross impoundment capacity. For sediment ponds, as storage capacity drops, the principal and auxiliary spillways must be able to handle increased discharges. For ponds located below coal refuse disposal facilities, pond size is a function of the size of and outflow from the upstream structure. The inflow may be only surface runoff from the face of a coal refuse embankment, but it more typically includes decant water discharges, internal drain system discharges from the coal refuse disposal facility, and other adjacent area runoff.

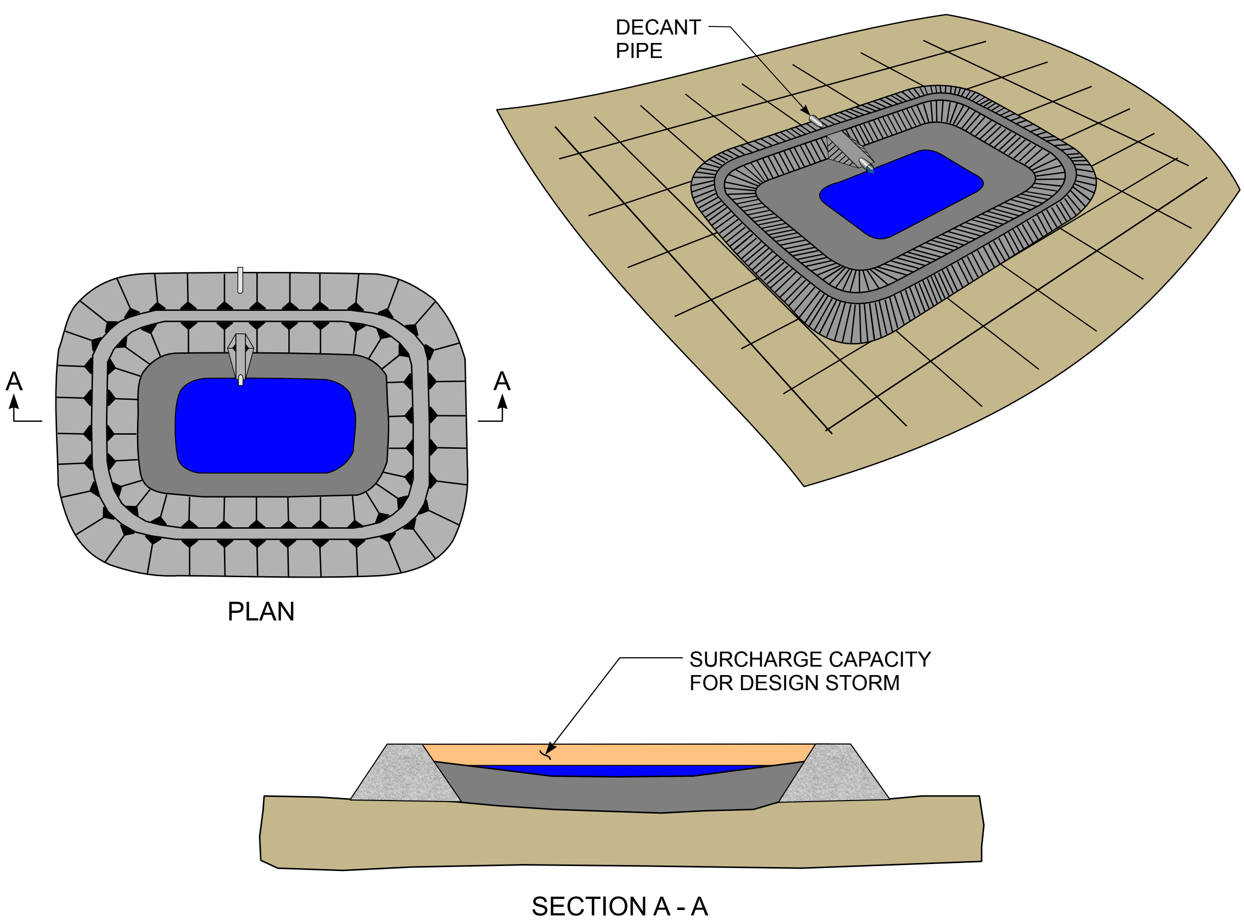


FIGURE 9.13 DRAINAGE CONTROL FOR DIKED IMPOUNDMENT

9.4.4.2 Fresh Water Impoundments

Fresh water impoundment capacity is determined by the mine and mine processing plant requirements. Fresh water impoundment capacities are generally large, and these impoundments are often regulated as high-hazard-potential structures. Fresh water impoundments should be designed and constructed according to accepted criteria for conventional dams. Outlet structures for these impoundments generally include both principal and auxiliary spillways.

9.5 DESIGN STORM CRITERIA

The quantity and distribution of runoff during a design storm for a coal refuse disposal facility site largely controls the design of hydraulic appurtenant structures. This section discusses design storm criteria in terms of the recurrence interval of the precipitation and the magnitude of precipitation measured in inches of rainfall. Section 9.6 discusses methods for converting design precipitation to design runoff volume and peak flow rates.

The appropriate design storm for a coal refuse disposal facility depends primarily on the consequences of the uncontrolled release of impounded material due to failure or faulty operation of the facility. Other factors that may affect the design storm include the facility configuration and size, type of hydraulic systems and operational period. Portions of the total hydraulic system, such as drainage culverts, ditches and some diversion channels will not generally create potentially hazardous conditions, so other design criteria can be selected for these structures. This situation is most likely to occur at non-impounding disposal facilities and at the perimeter of and appurtenant structures associated with impounding facilities.

Criteria for selecting a design storm for the operational period of an impounding facility are presented in Section 9.5.1. Design storms that are applicable for short-term conditions are discussed in Section 9.5.2. Design storm criteria for minor site drainage conveyance structures are presented in Section 9.5.3.

9.5.1 Design Storms for Impoundments

9.5.1.1 General Considerations

Numerous design storm criteria are employed in hydrologic analyses for water retention and flood control dams. The common factor associated with practically all of these criteria is that differentiations are made based on the projected maximum size of the impoundment and the magnitude of potential downstream hazard in the event of failure. MSHA has developed guidelines for design storms for the impoundments and embankments that they regulate; however, state and local criteria must also be considered. For any impoundment, the most conservative of applicable criteria should be used.

As part of the identification of the design storm, the size of the dam and reservoir and the associated hazard potential is typically determined either by inspection or analysis. Table 9.4 indicates appropriate design storms as related to impoundment size and hazard potential. Coal refuse impoundments should be designed for the Probable Maximum Flood (PMF) event, unless a lesser criterion can be justified consistent with Table 9.4. For determining the impoundment size, the impoundment volume and depth should include all water, sediment, and slurry that can be impounded. For determining the hazard potential, both the water and flowable materials retained in the impoundment should be considered.

The PMF is defined as the maximum runoff condition resulting from the most severe combination of hydrologic and meteorological conditions that is considered reasonably possible for the watershed. A PMF consists of an antecedent storm, a principal storm and a subsequent storm. The current assumed conditions for a PMF design storm in the MSHA guidelines are the following (MSHA, 2007):

1. <u>Antecedent storm</u> – 100-year precipitation event, with antecedent moisture condition II (AMC II) occurring 5 days prior to principal storm.

TABLE 9.4 RECOMMENDED MINIMUM DESIGN STORM CRITERIA FOR COAL REFUSE DISPOSAL IMPOUNDMENTS

A. Impoundment Size Classification

Category	Impoundment Size		
	Maximum Volume of Stored Water During Design Storm (acre-ft)		Maximum Depth of Water During Design Storm (ft)
Small to Intermediate	< 1,000	or	< 40
Large	≥ 1,000	or	≥ 40

B. Hazard Potential Classification

Category	Description
Low Hazard Potential	Facilities where failure results in no probable loss of human life and low economic and/or environmental losses. Such facilities would be located in rural or agricultural areas where losses would be limited principally to the owner's property, or failure would cause only slight damage, such as to farm buildings, forest, and agricultural land, or minor roads.
Significant Hazard Potential	Facilities where failure results in no probable loss of human life but can cause economic loss, environmental damage, or disruption of lifeline facilities. Such facilities would often be located in predominantly rural areas, but could be located in areas with population and significant infrastructures, and where failure may damage isolated homes, main highways, minor railroads or disrupt the use of service of public utilities.
High Hazard Potential	Facilities where failure will probably cause loss of life. Such facilities would be located where failure could be reasonably expected to cause loss of life, serious damage to homes, industrial and commercial buildings, important utilities, highways and railroads.

C. Recommended Design Storm for Long-Term and Short-Term Conditions(1)

Impoundment Size	Hazard Potential	Minimum Design Storm for Long Term	Minimum Design Storm for Short Term(1)	Additional Criterion
Small to Intermediate	Low	100-Year	100-Year	The indicated storm is appropriate only if the combination of spillways and decants for the facility can evacuate 90 percent of the incremental volume of stored storm water within 10 days.
	Significant	½-PMF	100-Year	
	High	PMF	½-PMF	
Large	Low	½-PMF	100 Year	
	Significant	PMF	½-PMF	
	High	PMF	½-PMF	

Note: 1. Situations where short-term criteria may apply include:

a. Initial construction. A new impoundment should be capable of accommodating the runoff from the short-term storm within one year and the long-term storm within two years.

b. Changing from an open-channel spillway to handle the design storm by storage. The time period when the long-term design storm cannot be accommodated should be kept as short as possible with detailed planning of the process.

c. Abandonment by elimination of impounding capacity. The impounding capability should be eliminated within two years after the impoundment can no longer accommodate the long-term design storm, and the work should be phased so that the facility is capable of accommodating less than the short-term storm for no more than one year.

2. Principal storm – Probable Maximum Precipitation (PMP) with AMC III. The principal storm rainfall must be distributed spatially and temporally to produce the most severe conditions with respect to impoundment freeboard and spillway discharge.
3. Subsequent storm – The subsequent storm criterion can be considered to be met if, within 10 days of the peak impoundment level associated with the principal storm, at least 90 percent of the volume of water stored above the normal operating level can be discharged from the impoundment. Alternatively, for facilities designed with sufficient storage but limited discharge capabilities that do not meet this criterion, the subsequent storm may be a second PMP storm with the same hydrologic and meteorological parameters as the principal storm, provided that the storage from both storms is drawn down at a rate sufficient to evacuate 90 percent of one storm from the impoundment within 30 days.

The antecedent storm precipitation can be obtained from National Weather Service publications. The most current definition of PMP (NWS, 1988) is "theoretically, the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location at a certain time of the year." The PMP can be determined from the National Weather Service publications discussed in the following paragraphs.

In the Western U.S., determination of the PMF may be based upon either: (1) the PMP and (2) the Probable Maximum Thunderstorm (PMTS). The PMTS is a very high-intensity, short-duration storm with intense precipitation occurring during a one-hour period. When designing a coal refuse disposal facility in this region, the more critical of these two criteria should be used. In this Manual, the term PMP represents the more severe of the PMP and PMTS for areas of the U.S. west of the 105^{th} meridian.

Dams or impoundments used for fine coal refuse disposal, fresh water retention, erosion and sediment control or other mine-related operations may need to have PMF storage/routing capacity. Less critical impoundments may have reduced design storm criteria based on embankment size and the potential downstream hazard. For such structures, both the 6-hour and 24-hour precipitation intensity (unless criteria are specified by state regulations) should be evaluated and the more conservative used for design.

As with water-impounding dams, basic design storm criteria apply to the long-term operation of coal refuse disposal facilities. However, short-term criteria, as summarized in Table 9.4, may be used for construction periods that typically extend from several months to two years for impounding structures subject to PMF design storm criteria. The designer of coal refuse disposal facilities must take into account that the configuration of the impounding embankment will be continually changing as additional refuse is placed and that the time associated with any one phase or the time between phases may be quite short. This can be accounted for by additional or modified design storm criteria presented in Section 9.6. These modified criteria should only be used for "unavoidable" situations that occur: (1) during short-term operations associated with initial construction of a disposal facility, (2) when a major modification is being made to an existing disposal facility, and (3) when a refuse disposal facility is being prepared for abandonment.

For water-retaining impoundments, different design storms are sometimes used for individual portions of the total hydraulic system such as the principal spillway and auxiliary spillway (NRCS, 2005b). This practice is generally not followed in the design of coal refuse disposal facilities provided the overall hazard criteria are satisfied because of the operational characteristics of a disposal facility, the dynamic nature of facility growth and the limited operational period. This practice may be applicable to other impounding facilities that support the mining operations (e.g., fresh water impoundment, sediment ponds, etc.).

In the design of coal refuse disposal facilities, it is important to differentiate between the functions of spillways and decants. The main function of a decant system is to discharge clarified water from the impoundment after the fine refuse has settled. Under normal precipitation conditions, the elevation of the decant inlet controls the normal operational water level in the impoundment. The capacity of a decant is limited and is typically too small to significantly affect the peak outflow during a large storm. Therefore, the storm runoff is almost totally controlled by impoundment storage or a combination of impoundment storage and auxiliary spillway capacity.

Even though an impoundment decant system does not have a significant impact on the outflow during the design storm, its capacity must be considered in other analyses related to storms. If the auxiliary spillway level is above the normal impoundment level (the typical condition) or if the hydraulic system design relies entirely on storage (no auxiliary spillway), the excess storm runoff must either be discharged totally through the decant system, or the decant system must serve as the primary outlet until the spillway level is reached. As indicated in Table 9.4, within ten days, the combined capacity of the spillway and decant systems must be capable of removing 90 percent of the maximum volume of water stored above the allowable normal operating water level during the design storm. The 10-day drawdown criterion begins at the time the water surface reaches the maximum elevation associated with the design storm. Alternatively, if there is sufficient impoundment capacity to store the runoff from two design storms (specifically, the antecedent storm and two principal storms), an extension of the 10-day criterion is reasonable, provided that an effective means for discharging the storage from both storms is available. Generally, an evacuation rate that will remove 90 percent of the stored runoff from one design storm within 30 days is considered to be reasonable.

9.5.1.2 Recommended Design Storm Criteria

Table 9.4 provides recommended minimum design storm criteria for coal-mining-related impounding facilities for both long-term and short-term conditions. Selection of the appropriate storm for a specific impounding structure is based on the impoundment size and hazard-potential classification. The selected criteria for the storage and routing of the design storm and hydraulic structure design should also reflect any other applicable regulatory reviewing agency criteria.

Dams and impoundments that are small to intermediate in size (less than 40 feet in height or 1,000 acre feet in storage volume) with low hazard potential should be designed for a long-term storm event with no less than a 100-year recurrence interval. For coal refuse impoundments equal to or greater than 40 feet in height or 1,000 acre feet in storage volume with low or significant hazard potential, the minimum long-term design storm should be either the ½ PMF or full PMF, respectively. The ½ PMF design storm should have one-half of the inflow rate and runoff volume of the full PMF. For coal refuse impoundments with high hazard potential, the minimum long-term design storm should be the full PMF. In cases where the design storm for long-term conditions is less than the full PMF, it may be prudent to adopt minimum design storm criteria greater than those provided in Table 9.4 and thus achieve greater protection from flood events and related damage.

The following paragraphs discuss the basis and/or justifications for criteria and information presented in Table 9.4. Procedures for quantitatively determining the magnitude of precipitation to be used in the calculation of runoff are discussed in Section 9.6.

9.5.1.3 Size and Hazard-Potential Classification

The rationale for relating the design storm to the size and hazard potential of the disposal facility impoundment is evident. Impoundment size is defined by the maximum depth and total volume of retained water, sediment and slurry; however, determining the hazard-potential classification requires judgment and, unless otherwise obvious, should be based upon hydraulic analyses. The bases for the criteria listed in Table 9.4 are discussed in the following subsections.

9.5.1.3.1 Impoundment-Size Classification

The size classification presented in Table 9.4 is based on the total volume and depth of all water, sediment and slurry impounded during the design storm. As indicated in the table, the recommended design storms for small and intermediate size impoundments are the same.

9.5.1.3.2 Hazard-Potential Classification

The hazard-potential classification presented in Table 9.4 is the same as that presented in Chapter 3 and used in the overall classification system for coal refuse disposal facilities. Dams that are located where loss of life is probable in the event of failure are classified as having high hazard potential. In applying these criteria, it is important to recognize the difficulty of determining whether minor or major damage or the loss of life will result from the failure of a refuse disposal facility. For most coal refuse disposal facilities, this determination is based upon: (1) the configuration and location of the facility and (2) the downstream conditions (both existing and planned) including population, topography and the size of streams that would receive flood flow resulting from an embankment failure or a breakthrough-type release from the impoundment. Downstream conditions are typically evaluated by reviewing USGS topographic quadrangle maps and by field verification. The manner that MSHA addresses the hazard associated with a breakthrough-type release is discussed in Section 3.1.

Generally, unless it is otherwise evident, the determination of hazard potential is based upon a dam or impoundment breach analysis and inundation mapping. Section 9.9" presents dam-breach-analysis methods. A dam-breach analysis should provide inundation levels for two conditions: (1) postulated failure of the dam under design-storm conditions and (2) postulated failure of the dam during normal operations (sunny day or fair weather breach failure). If doubt exists as to the possible effects of an impoundment failure on downstream areas, the more conservative hazard classification should be selected. However, it may also be useful to evaluate the downstream inundation and damage that could result from a major storm in the refuse disposal facility watershed, but without failure of the impoundment. This inundation level and related damage can then be compared to the incremental inundation and damage that would be caused by failure of the disposal facility under design-storm conditions. If the additional damage can be reasonably predicted as small, then a less conservative design storm may be appropriate (FEMA, 2004a), or the hazard-potential classification may be governed by the fair weather breach.

For most large dams and impoundments where downstream residential, commercial or industrial development is present adjacent to streams, a high-hazard-potential classification is selected based on probable loss of human life. Other situations can arise where the threat is less evident or where the distinction between significant and low hazard potential is important. FEMA (2004a) provides guidance for interpreting the probable loss of life by clarifying that "postulating every conceivable circumstance that might remotely place a person in the inundation zone should not be the basis for determining the appropriate classification level." In the definition of high hazard potential, the probable loss of human life is clarified to exclude consideration of the casual user of downstream or upstream areas. However, personnel who routinely or frequently work or occupy locations or structures in the downstream area should be considered in the assessment of hazard-potential classification.

USBR (1988) provides guidance based upon the number of lives in jeopardy (all individuals within the inundation boundaries who, if they took no action to evacuate, would be subject to danger) to aid in assessing the potential for probable loss of life. In cases where a dam-breach analysis indicates limited inundation at occupied structures in relatively undeveloped areas, such guidance in assigning hazard potential may be useful. USBR (1988) provides guidelines for interpreting the significance of predicted inundation depth and velocity at downstream residences, roadways, and pedestrian routes.

There have been a limited number of mining situations, primarily in the western United States, where high-hazard-potential dams have been designed using hydrologic design criteria associated with a lower hazard-potential-classification and incorporating a warning system. An example is a dam constructed across a watercourse for prevention or mitigation of flooding damage to a surface mine pit. To design a flood-control structure to totally accommodate the design event would necessitate the construction of a very large dam that would function only temporarily. Failure of this dam could possibly result in a higher hazard potential due to the additional storage. In such cases, some designers have proposed dams using low- or significant-hazard-potential criteria and incorporating warning systems. The warning systems are designed to notify the minimg operation when the water behind the dam reaches a specified level. At that time, all potentially affected personnel are withdrawn from the downstream area. Allowance for warning time must not be a substitute for appropriate dam design and construction. MSHA (Fredland, 2008) has indicated that this approach may be acceptable on a case-by-case basis for temporary mining operations. Conditions associated with warning systems for this approach are discussed in Section 3.7.

Hazard-potential classification is also dependent on the potential for economic, environmental or lifeline losses. If a dam or impoundment is not classified as having high hazard potential because there is no probable loss of human life, generally it reflects a situation where there are few downstream structures and thus limited potential for associated economic damages. FEMA (2004a) clarifies that for classification of a dam as having low hazard potential (as opposed to significant hazard potential), the economic, environmental or lifeline losses must be low and generally limited to the owner of the structure. While economic damages to downstream development may be determined to be low and thus could support classification of a dam as having low hazard potential, the possibility of environmental damages may warrant consideration of higher hazard classification levels.

9.5.1.4 Determination of Design Storm Precipitation

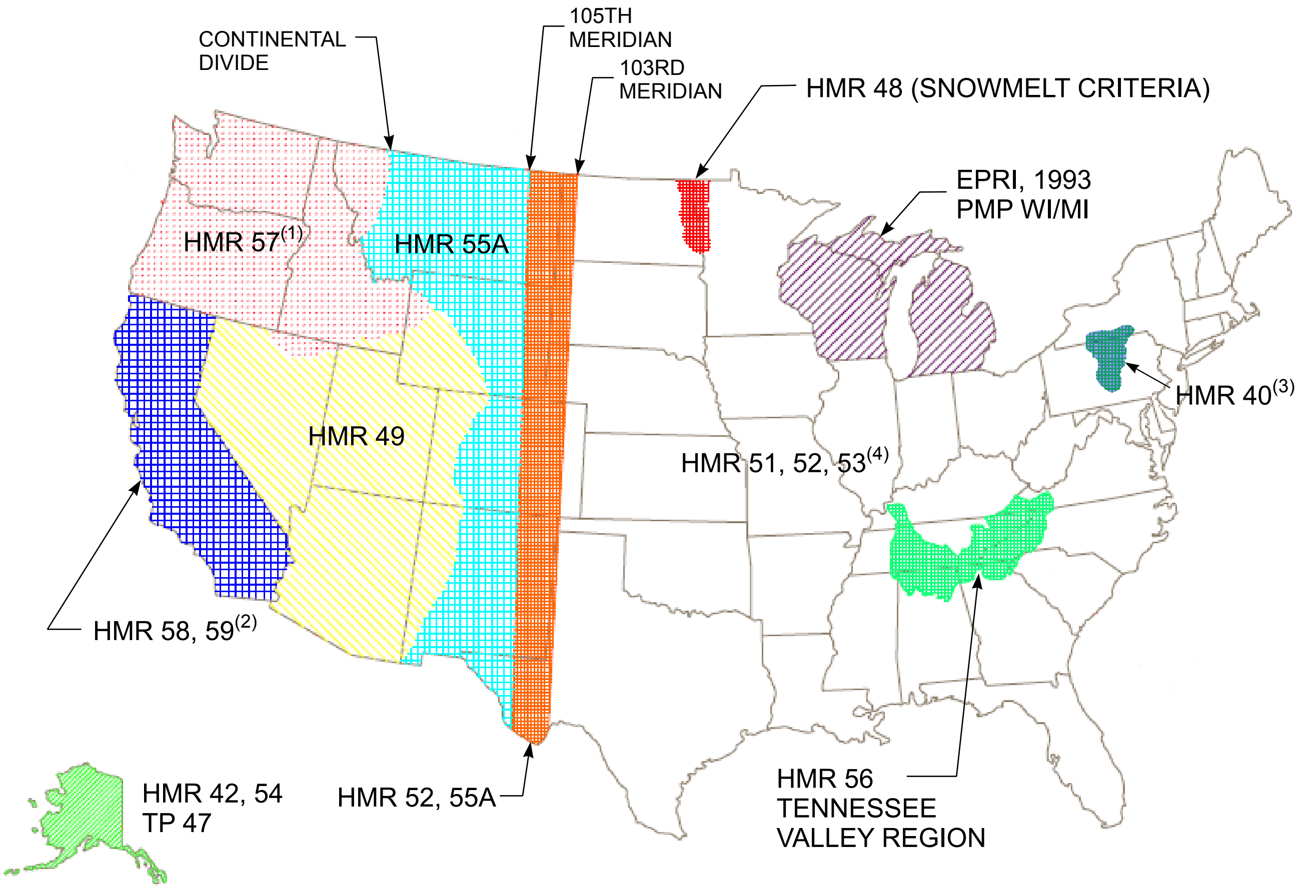
Once the size and hazard-potential classification of a disposal facility impoundment are established, the recommended design storm can be determined from Table 9.4. The procedure for determining the magnitude of the precipitation for the design storm is discussed in the following paragraphs, while the procedure for computing the resulting runoff is presented in Section 9.6.

9.5.1.4.1 Prediction of the PMP and PMTS

Predictions of the PMP (inches of rainfall) for a watershed of 10 square miles and durations of 6 to 72 hours are presented in reports prepared by the National Weather Service's Hydrometeorological Design Studies Center. Figure 9.14 identifies applicable Hydrometeorological Reports (HMRs) for various regions of the U.S. For areas east of the 105th meridian, HMR 51 (NWS, 1978) should be used for determining the PMP magnitude and for extending the PMP to longer durations. The only exception is an area in the Tennessee River Valley that is addressed in HMR 56 (NWS, 1986). Procedures for determining critical rainfall spatial and temporal distribution for areas east of the 105th meridian are provided in HMR 52 (NWS, 1982); however, the document may not be applicable to all watersheds, particularly watersheds with areas less than 10 square miles. Seasonal variation of PMP for areas east of the 105th meridian is addressed in HMR 53 (NWS, 1980). For the region between the 103rd and 105th meridian, HMR 55A (NWS, 1988) and HMR 52 should be used. For the area between the 103rd meridian and the continental divide, HMR 55A is applicable. For areas west of the continental divide, HMR 49 (NWS, 1977), HMR 57 (NWS, 1994), HMR 58 (NWS, 1998), or HMR 59 (NWS, 1999) should be used, as indicated by the shaded areas in Figure 9.14.

As indicated above, HMR 56 (NWS, 1986) was developed for the Tennessee Valley. While HMR 56 is recommended for projects in that region, the study indicates that in non-orographic areas numerous comparisons were made between the results from HMR 56 and the results from HMR 51 and HMR 52, indicating that minor differences in results can be expected depending upon the size of the study region. In more mountainous orographic areas, HMR 56 provides guidance for determining the areal distribution of storm-averaged depths with reference to HMR 52.

The extension of the PMP for watersheds exceeding 10 square miles and other durations are discussed in relation to analyses for determining runoff in Section 9.6. For coal refuse disposal impoundments, the applicable watershed is typically much smaller than 10 square miles, resulting in no or only limited adjustments for spatial distribution using HMR 52 (NWS, 1982). Because impoundments are designed with considerable storage capacity, and in many cases the ability to store the runoff from the entire design storm, determination of adjustments that affect the peak inflow rate may not be nec-



NOTE: 1. HMR 57 REPLACED HMR 43 IN 1994.
2. HMR 58 AND 59 REPLACED HMR 36 AND 49 FOR CALIFORNIA IN 1999.
3. HMR 51, 52, 53 MAY APPLY FOR BASINS UNDER 24,000 SQUARE MILES.
4. HMR 33 (SEASONAL PMP VARIATIONS FOR AREAS NOT SUPERCEDED BY SITE-SPECIFIC SEASONAL PMP REPORTS)

(ADAPTED FROM FERC, 2001)

FIGURE 9.14 U.S. REGIONS COVERED BY GENERALIZED PMP STUDIES

essary. However, in such cases the PMP must generally be extended to 72 hours. For impoundments with watershed areas as small as one square mile that rely on open-channel spillways for routing the PMF, HMR 52 (NWS, 1982) provides a means for estimating the adjusted PMP distribution using depth-duration ratios and 1-hour PMP values.

9.5.1.4.2 Prediction of the 100-year and Lesser Design Storms

The 100-year-recurrence-interval design storm and lesser design storm precipitation data can be obtained from several sources. NOAA Atlas 14 Volume 1 (NWS, 2006a) and Volume 2 (NWS, 2006b) provide rainfall frequency values for much of the U.S., and NOAA Atlas 2 (NWS, 1973) provides data for some areas of the western U.S not covered in Atlas 14 Volume 2. Other areas of the country are addressed in various technical publications available from NOAA or in other sources listed in Table 9.5. Access to precipitation, frequency, and intensity data for specific locations is available from the NOAA web site.

The current practice of precipitation frequency analysis is based upon the implicit assumption that past experience can be used to predict future events and that the climate will not change. In its current studies, the NWS is assuming that the full period of the available historical record is suitable for use, as current climate change forecasts do not reliably define future changes in precipitation frequency distribution.

9.5.2 Special Considerations for Short-Term Conditions

Although coal refuse disposal facilities are typically dynamic or constantly changing entities, careful consideration of growth characteristics and proper planning of modifications will result in compliance with long-term design storm criteria over the facility's entire service life. Occasionally, however, it may be impossible to meet long-term requirements during brief periods when significant physical changes to the facility are occurring.

Appropriate design storms for short-term conditions are provided in Table 9.4. Short-term conditions for periods of significant physical change are more related to general construction practices, and therefore the criteria for temporary (stream) diversions are generally dictated by state guidelines. The upper limit for a short-term condition is two years. The short-term design storms provided in Table 9.4 are more conservative (higher) than those normally used for dam construction (USBR 1987a). The more stringent criteria are recommended because planning and implementation of modifications at coal refuse disposal facilities are dependent upon day-to-day coal and refuse generation unlike other types of embankments.

It is stressed that these short-term criteria are not intended as less costly design alternatives based on the rationale that a short-term condition is always appropriate for a given site because it is continually changing in configuration. If such an approach is followed, it should be expected that regulatory acceptance of a lesser storm will not be granted. The temporary use of design storms of lesser magnitude than those required for long-term facility operation will likely be accepted only if the following conditions are met:

- The facility will be designed to satisfactorily meet the requirements for such interim use, including, but not limited, to safe control of the short-term design storm.
- As part of the overall design and planning process, interim periods of short-term use are unavoidable and are identified and their duration realistically scheduled. As these periods are approached during construction, the scheduling of these transitional periods should be adjusted as required and thereafter strictly followed. Such preplanning and scheduling should be done in a manner that minimizes the duration of the short-term condition and facilitates the speedy transition to either a long-term operating status or abandonment.

Periods during the service life of a refuse disposal facility when even careful planning may occasionally be insufficient to achieve compliance with long-term design criteria include:

- Initial construction of a new impounding structure. The impoundment should be capable of accommodating the runoff from the short-term storm within one year and the long-term storm within two years.
- Transitioning from a lower open-channel spillway to a higher open-channel spill-way as part of raising the embankment stage crest or changing from an open-channel spill-

way to handling the design storm by storage. The time period when the long-term design storm cannot be accommodated should be kept as short as possible, and a comprehensive plan and schedule for the sequence of the change should be provided.

- Abandonment by elimination of impounding capability. The impounding capability of the facility should be eliminated within 2 years after the time that the impoundment can no longer accommodate the long-term design storm. Additionally, abandonment should be phased such that the time period when the facility is capable of handling less than the short-term storm is no more than one year.

TABLE 9.5 NWS PRECIPITATION FREQUENCY PUBLICATIONS

Location	Design Storm Duration		
	5 to 60 min	1 to 24 hrs	2 to 10 days
DE, IL, IN, KY, MD, NJ, NC, OH, PA, SC, TN, VA, DC	NOAA Atlas 14 Volume 2 (NWS, 2006b)	NOAA Atlas 14 Volume 2 (NWS, 2006b)	NOAA Atlas 14 Volume 2 (NWS, 2006b)
Remainder of Eastern United States	Technical Memorandum NWS HYDRO-35 (Frederick et al.,1977)	Technical Paper 40 (Hershfield, 1961)	Technical Paper 49 (NWS, 1964)
AZ, NV, NM, UT, Southeast CA	NOAA Atlas 14 Volume 1 (NWS, 2006a)	NOAA Atlas 14 Volume 1 (NWS, 2006a)	NOAA Atlas 14 Volume 1 (NWS, 2006a)
Remainder of Western United States	Arkell and Richards (1986); Frederick and Miller (1979)	NOAA Atlas 2 (NWS, 1973)	Technical Paper 49 (NWS, 1964)
Alaska	Technical Paper 47 (NWS, 1963)	Technical Paper 47 (NWS, 1963)	Technical Paper 52 (NWS, 1965)

(ADAPTED FROM NRCS, 1986)

9.5.3 Hydraulic Design Criteria for Drainage Conveyance Installations

Hydraulic structures for both non-impounding and impounding coal refuse disposal facilities fall into three general categories:

1. Those structures that by failure, overtopping and/or blockage could threaten the overall stability of the disposal facility.
2. Those structures that by failure, overtopping and/or blockage would not threaten the overall stability of the disposal facility, but could lead to localized instability.
3. Those structures that, even if non-functional, would not endanger the overall stability of the facility and would not greatly affect day-to-day operation of the facility.

Hydraulic structures that are critical to the overall safety of coal refuse disposal facilities must be designed to adequately control the facility design storm. Although this most commonly applies to the hydraulic structures associated with impounding facilities, the requirement applies equally to hydraulic structures at non-impounding facilities. Permanent hydraulic structures (other than impoundment spillways) at coal refuse disposal embankments should be designed to handle the 100-year storm.

The purpose of many refuse disposal facility permanent hydraulic structures is to limit erosion or other types of localized instability rather than to provide total hydraulic control during major storms. Whether or not a facility is impounding, non-impounding, active or abandoned, these structures are important to the development and operation of a disposal facility and should generally be designed for the 100-year storm. Design criteria for these structures may also be governed by state or local regulations. Table 9.6 provides a summary of typical design criteria for minor hydraulic structures at locations that are not part of the coal refuse disposal facility. These structures typically include storm sewers, culverts, drainage ditches and gutters.

9.6 DETERMINATION OF RUNOFF QUANTITIES

The most important aspects of hydrologic analyses related to refuse disposal facility performance during and after storm rainfalls are the determination of the peak runoff rate and the total runoff volume at the point of interest. Four methods for determining these parameters that are available to the designer are presented in Table 9.7. The first three methods presented in the table are discussed in this section following a general discussion of basic hydrology parameters.

9.6.1 Basic Hydrology Parameters

There are three basic factors that must be considered when predicting runoff rates and quantities. These are: (1) precipitation (intensity and duration), (2) watershed (size and time of concentration), and (3) soil types and land use conditions. These factors are further explained in the following subsections.

9.6.1.1 Precipitation Intensity-Duration and Distribution

Storms are defined by their precipitation intensity-duration relationships. Storms can range from high-intensity, short-duration thunderstorms to low-intensity, long-duration rainfalls lasting several days. The intensity-duration relationship that should be used for hydrologic analyses and channel design is that which produces the maximum peak runoff rate. This is particularly true for the small watersheds common to coal refuse facilities where the time of concentration (time required for rainfall to travel from the most hydrologically distant point in the watershed to the point of interest) is

TABLE 9.6 TYPICAL DESIGN CRITERIA FOR MINOR HYDRAULIC STRUCTURES(1)

Structure Type and Condition	Design Criteria
Storm Sewers	10-year rainfall
Diversion Systems	
Temporary (1-year life or less and watershed > 5 acres)	
Construction areas, roads, pipelines	2-year rainfall
Permanent	
Sediment Retention Structures (watershed <100 acres and height < 15 feet):	10-year rainfall
Emergency spillway capacity	25-year, 24-hour rainfall
Principal spillway capacity	10-year, 24-hour rainfall
Culverts:	
Access Roads and Drainage Swales	10-year, 24-hour rainfall
Local and Urban Roads	25-year, 24-hour rainfall
Highways and Streams	100-year, 24-hour rainfall
Drainage Ditches and Gutters	10-year rainfall

Note: 1. These criteria do not apply to minor structures on coal refuse disposal facilities. Permanent perimeter ditches and bench gutters on coal refuse disposal facilities should be designed for the 100-year storm.

TABLE 9.7 METHODS FOR DETERMINING RUNOFF RATE AND VOLUME

Method	Applicable Conditions
Hydrograph Method (Section 9.6.2)	Applicable to all runoff analyses, but normally used when a time-related runoff distribution is required or when less exact methods for estimating runoff are not sufficiently accurate for design of an economical drainage system.
Peak Runoff Determination (Section 9.6.3)	For (1) determining estimates of peak runoff rate and runoff volume for system sizing when time-related runoff distribution is not required for final design or (2) preliminary system sizing prior to generating a runoff hydrograph for flood routing.
Rational Method (Section 9.6.4)	For designing drainage conveyance structures such as diversion and collection ditches and road culverts for small watersheds.
Stream Gage Data Analysis (USDA, 1972; Chow, 1964)	For predicting runoff by statistical analysis of measured stream flow records when a long history of data is available for the stream or for a nearby similar stream and watershed. Since these data are not generally available for the types of streams passing through or adjacent to coal refuse disposal facilities, methods using stream flow records are not presented herein.

small. In larger watersheds, the averaging effects of short and long times of concentration tend to compensate for small errors in the predicted intensity-duration relationship.

Figure 9.15 presents the dimensionless design storm distribution frequently used to evaluate the intensity-duration relationship (NRCS, 2005b) based upon a 6-hour-duration storm. The 24-hour storm can be constructed by critically stacking incremental rainfall amounts for successive 6-, 12-, and 24-hour durations, as discussed in HMR 52. Runoff determinations for mining facilities are typically based upon either 6-hour- or 24-hour-duration precipitation events (with extension to periods up to 72 hours for impoundments that rely on storage for flood routing) except for states west of the 105th Meridian where PMTS runoff must also be considered.

As indicated in Table 9.4, development of specific-frequency flood hydrographs may be required for the design of structures with low- or significant-hazard-potential classification. High-hazard-potential structures require design for the PMF, which for coal refuse disposal facilities is typically derived from the PMP for the watershed. The PMP is a 6- to 72-hour duration precipitation distribution that results in a peak intensity occurring during the third quadrant of precipitation. This distribution curve is recommended for most coal refuse impoundment hydrologic design and analysis applications. For smaller impounding structures, it is recommended that both short-duration (6-hour) and long-duration (24-hour) storms be utilized to determine the peak runoff for sizing of the outflow structures.

In addition to knowing how the intensity of precipitation may be distributed within a six-hour storm, it is also important to recognize that storms may continue for longer periods of time at decreasing intensities. Such storms may be critical for disposal facilities that rely primarily on reservoir storage to control runoff, since the amount of runoff occurring after the first six hours may represent a significant portion of the runoff volume of the total storm. HMR 51 and HMR 52 can be used to extend the predicted six-hour PMP.

Figure 9.14 shows applicable HMRs for determining magnitudes and temporal distributions for probable maximum storms based upon regionalized criteria. Charts for PMP values are presented in HMR 51 for most areas east of the 105th Meridian, and procedures are provided in HMR 52

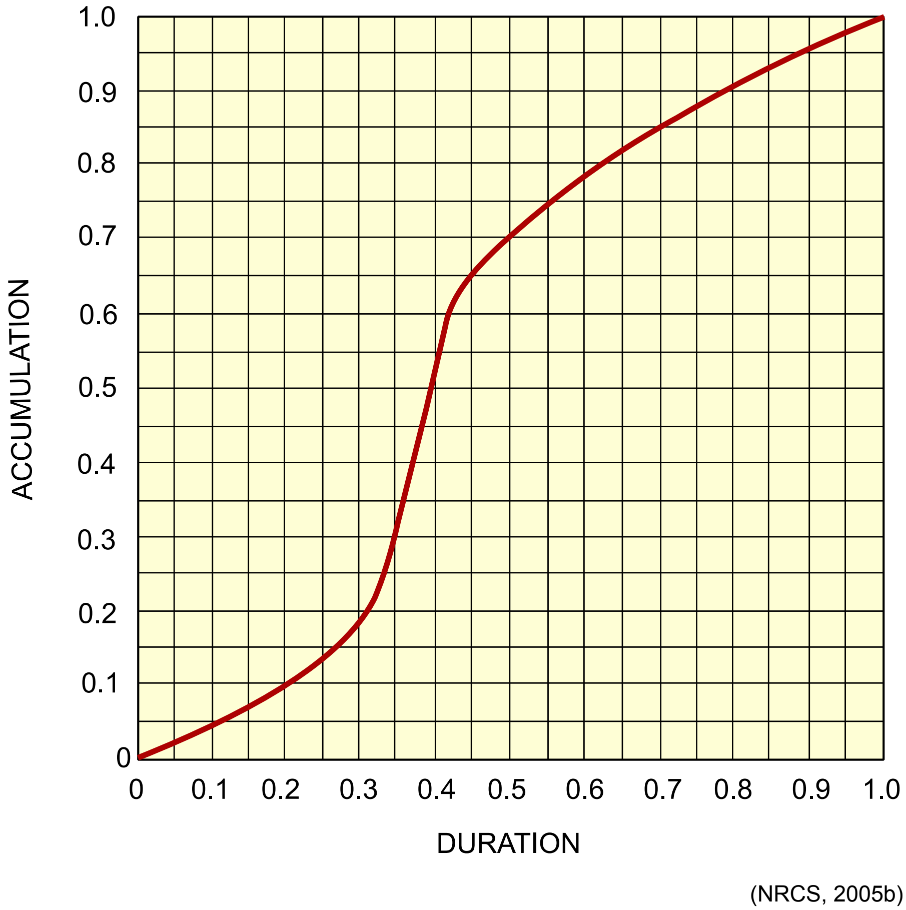


FIGURE 9.15 DIMENSIONLESS DESIGN STORM DISTRIBUTION

that translate these values to a spatially and temporally distributed estimate of the site PMP. The computer program HMR 52 developed by the U.S. Army Corps of Engineers (USACE, 1984b) determines the most severe storm conditions considering basin characteristics and regional conditions that are critical for watersheds with areas greater than 10 square miles. For many coal refuse impoundments, watersheds are small (typically less than 1 square mile) and the procedures in the computer program HMR 52 may need to be adjusted for these smaller watersheds using methods presented in NWS (1982).

HMR 56 is applicable in the Tennessee Valley region. For the region between the 105^{th} Meridian and the Continental Divide, HMR 55A should be used. Probable maximum storm estimates for areas west of the Continental Divide may be developed using HMR 49, HMR 57, and HMR 59, which account for orographic effects and include procedures for evaluating local (thunderstorm) PMP storms.

Short-term design storm criteria and low-hazard-potential dam design criteria require precipitation frequency information that is available from NOAA, as indicated in Table 9.5.

9.6.1.2 Unit Hydrographs and Time of Concentration

Unit hydrograph theory is the basis for computing inflow hydrographs for design storms. A unit hydrograph can be derived from observed hydrographs recorded on gauged streams, although for most coal refuse disposal facilities located in small watersheds, they are synthesized using relationships between rainfall and runoff that are dependent on watershed conditions. Empirical equations are typically employed to estimate parameters for synthetic unit hydrographs, although some government agencies can provide parameters for ungaged stream basins, including:

- USACE has developed coefficients for use in computing Snyder and Clark unit hydrographs for some areas of the U.S. USACE district offices can provide information on the results of studies in their districts.
- The USBR has developed a set of lag-time equations, dimensionless unit hydrographs, and S-graphs for different parts of the western U.S. (Cudworth, 1989).
- The USGS has performed regional studies for development of unit hydrographs in cooperation with state departments of transportation. These are published as USGS water resources investigation reports. Some of these are applicable to the states of Illinois, Tennessee, and Alabama (Graf et al., 1982; Robbins, 1986; Olin and Akins, 1988).

Before applying published parameters for watersheds in a region, the possible effects of differences in drainage area, cover, soil type, orientation, or geology should be evaluated. Additionally, the terminology used to define the various hydrographs and basin parameters in a regional study should be carefully reviewed so that application to ungaged watersheds is consistent (e.g., lag time and channel slope may be defined differently in the various methodologies).

If published parameters for watersheds in the region are not available, and the drainage basin is larger than about 100 square miles, a regional analysis may be prepared by analyzing rainfall and streamflow records at gauged watersheds to relate the peak flow rate and lag time to the drainage area. Procedures are described in FERC (2001).

The most common method available to designers for the small watersheds typically associated with coal refuse disposal facilities is based on empirically derived coefficients for synthetic unit hydrographs. The common methods for developing parameters from empirical equations include the Clark, Snyder and SCS unit hydrograph procedures. These methods are incorporated into the widely used computer programs for development of inflow design floods (e.g., HEC-1 and HEC-HMS developed by the U.S. Army Corps of Engineers Hydraulic Engineering Center (HEC) and similar privately marketed programs).

9.6.1.2.1 Snyder Unit Hydrograph

The equations used for the Snyder unit hydrograph are (USACE, 1990b):

$$t_p = C_t (L L_{ca})^{0.3} \tag{9-1}$$

$$C_p = (Q_p t_p) / (640 A) \tag{9-2}$$

t_p = time lag measured from the centroid of precipitation excess to the time of peak flow at the point of interest (hr)

L_{ca} = length along the main watercourse measured from the outlet upstream to a point nearest the basin centroid (mi)

L = length of the main watercourse (mi)

Q_p = peak flow rate of the unit hydrograph (cfs)

A = drainage area (mi^2)

The coefficients C_t and C_p are empirical values applicable to specific regions that account for watershed storage and slope and flood-wave velocity and channel storage, respectively. These parameters are obtained from regional studies and, if they are representative of conditions of the watershed being analyzed, are entered into HEC hydrologic software for a Snyder unit-hydrograph analysis.

9.6.1.2.2 Clark Unit Hydrograph

The Clark unit hydrograph uses a time-area curve to represent the watershed and uses a computed time of concentration (T_c) that can be calculated based on SCS procedures unless more reliable regional data are available. Additionally, the Clark unit hydrograph also uses a coefficient that reflects the effect of storage within the watershed. HEC hydrologic software (e.g., HEC-1) can be used to calculate the value of this coefficient through its optimization routine, but the result obtained should be evaluated and compared to published or available regional data. The Clark method is usually not employed for the small watersheds that are typically associated with coal refuse disposal facilities.

9.6.1.2.3 SCS Dimensionless Unit Hydrograph

The SCS method is the most commonly used approach for small watersheds and is frequently used for coal refuse disposal facility design. The primary analytical requirement for this method to be applied in a HEC-1 analysis is the estimation of the lag time for the basin, which is generally assumed to be equal to 0.6 T_c.

Time of Concentration and Lag Time

The time of concentration T_c is the time required for runoff to travel from the most hydrologically remote point in the watershed to the point of interest (Figure 9.2). The hydrologically most distant path within a watershed may not necessarily be along the longest water course; therefore, various watershed length and slope combinations should be evaluated.

Empirical equations have been developed by the SCS, the USBR, and others for estimation of T_c as a function of the length, surface texture and vegetation, and watershed slopes. Additionally, T_c may be computed by analysis of the overland and channel flow travel time using surface drainage software. A common method is to use the computer program TR-55 to determine flow velocity and associated time of concentration for subbasins within a watershed and thus estimate T_c. Empirical equations for determination of T_c are presented below.

USBR Method

The USBR (1973) determined T_c from the following equation that has historically been applied to small watersheds for design of small dams and coal refuse disposal facilities:

$$T_c = \left[(11.9L^3)/H\right]^{0.385} \tag{9-3}$$

where:

T_c = time of concentration (hr)

L = length of longest watercourse in watershed (mi)

H = elevation difference between the highest and lowest points in the watershed (ft)

For watersheds west of the 105th meridian and forested mountain watersheds east of the 105th meridian, Table 9.8 lists correction factors that should be applied to $T_{c,}$ as predicted by Equation 9-3. The Modified Snyder Method developed by USBR (1987a), which is discussed in subsequent paragraphs, is now more commonly used.

TABLE 9.8 RECOMMENDED CORRECTION FACTORS FOR T_C FOR WATERSHEDS WEST OF THE 105TH MERIDIAN

CN	T'_c / T_c
80	1.0
70	1.4
60	1.8
50	2.2

(USBR, 1973)

SCS Methods

The lag method developed by the Natural Resources Conservation Service (NRCS, 1986), also referred to as the curve-number method, applies to areas less than 2,000 acres:

$$T_c = 5/3\left[(L^{0.8}(S+1)^{0.7})/1900\,Y^{0.5}\right] \tag{9-4}$$

where:

S = $(1000/CN) - 10$
L = hydraulic length of watershed (ft)
Y = average watershed land slope (percent)
CN = curve number (dim)

The NRCS (1986) also developed an approach to determining time of concentration by computing the travel time (T_t) for runoff to traverse the watershed, considering three components of flow: (1) sheet flow in upland areas (generally applied to distances of 300 feet or less), (2) shallow concentrated flow as runoff concentrates beyond the sheet flow areas, and (3) open-channel flow as runoff is conveyed downstream. By summing the travel times, an estimate of T_c can be determined from the following relationship:

$$T_c = T_{t1} + T_{t2} + T_{t3} \tag{9-5}$$

Sheet Flow

$$T_{t1} = \left[0.007\,(nL_1)^{0.8}\right] / (P_2^{\,0.5}\, s^{0.4}) \tag{9-6}$$

where:

T_{t1} = sheet flow travel time (hr)
n = Manning's roughness coefficient for sheet flow (Table 9.9)
L_1 = flow length (ft)
P_2 = 2-year, 24-hour rainfall (in)
s = average watershed land slope (ft/ft)

TABLE 9.9 ROUGHNESS COEFFICIENTS (MANNING'S N) FOR SHEET FLOW

Surface Description	n
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils	
Residue cover ≤ 20%	0.06
Residue cover ≥ 20%	0.17
Grass	
Short grass prairie	0.15
Dense grasses(1)	0.24
Bermuda grass	0.41
Range (natural)	0.13
Woods(2)	
Light underbrush	0.40
Dense underbrush	0.80

Note: 1. Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass and native grass mixtures.
2. When selecting n, cover should be assumed to have a height of 0.1 foot. This is the only portion of the plant cover that will obstruct flow.

(NRCS, 1986)

Shallow Concentrated Flow

$$T_{t2} = L_2/(3600 V_2) \qquad (9\text{-}7)$$

where:

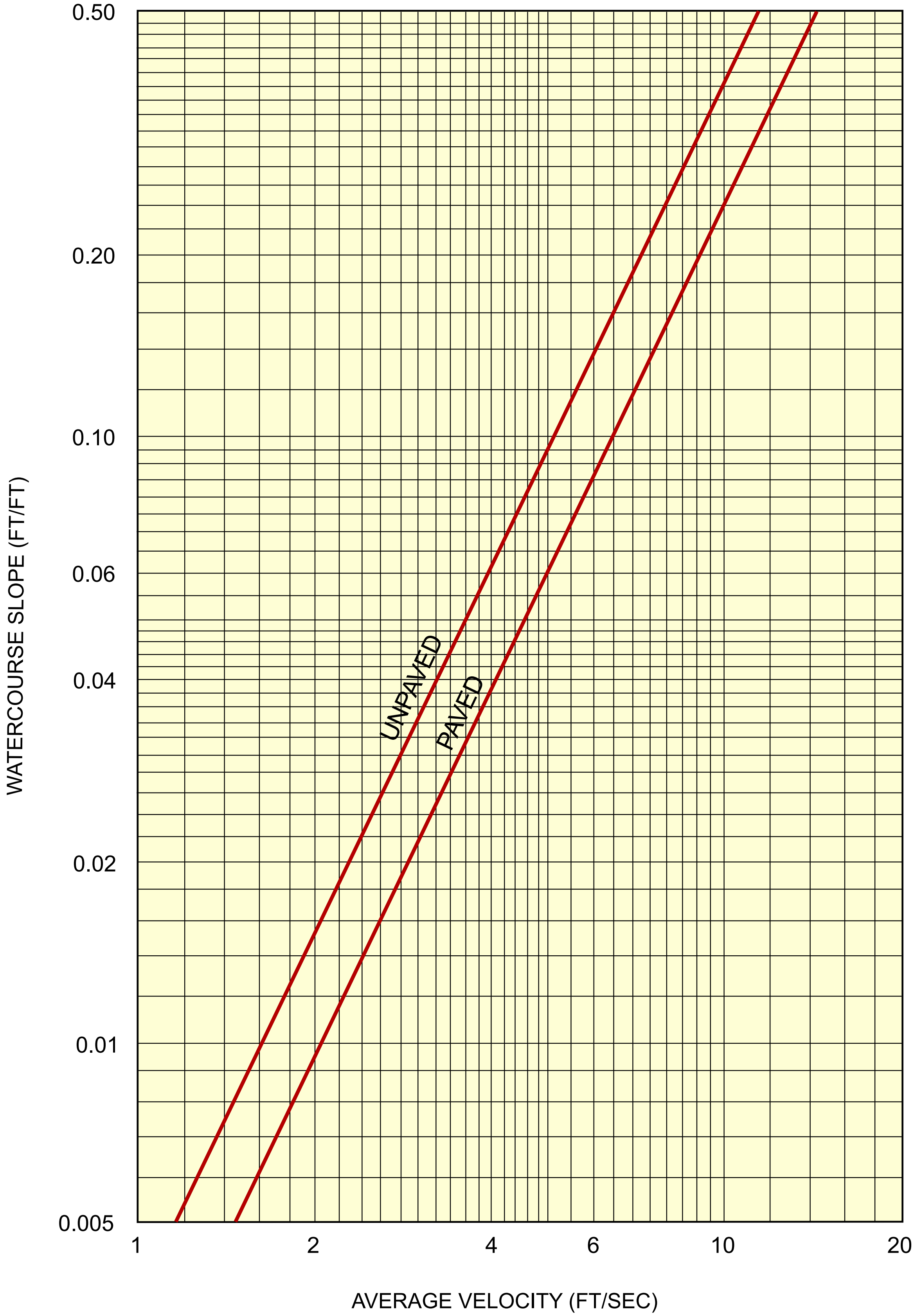
T_{t2} = shallow concentrated flow travel time (hr)
L_2 = flow length (ft)
V_2 = average velocity (ft/sec) from Figure 9.16

Open-Channel Flow

$$T_{t3} = L_3/(3600 V_3) \qquad (9\text{-}8)$$
$$V_3 = (1.49\, R^{0.67} s^{0.5})/n$$

where:

T_{t3} = open-channel-flow travel time (hr)
L_3 = channel flow length (ft)
R = hydraulic radius = cross-sectional-flow area/wetted perimeter (ft)
s = slope of the channel (ft/ft)
n = Manning's roughness coefficient (Section 9.7.2.2)
V_3 = average channel flow velocity (ft/sec)



(NRCS, 1986)

FIGURE 9.16 AVERAGE VELOCITIES FOR ESTIMATING TRAVEL TIME FOR SHALLOW, CONCENTRATED FLOW

USBR (Modified Snyder) Method

The USBR (1987a) provides guidance for unit hydrograph development based on lag time using the Modified Snyder Equation, including charts for regions of the country to assist in estimating the lag time:

$$L_g = C\left[(L\,L_{ca})/S^{0.5})\right]^{0.33} \quad (9\text{-}9)$$

where:

L_g = unit hydrograph lag time (hr)
C = constant (estimated as 26 times the average Manning's n value for in-channel flows with higher values for overbank flow conditions)
L = length of main watercourse (mi)
L_{ca} = length along the main water course measured from the outlet upstream to a point nearest the basin centroid (mi)
S = overall slope for longest water course (ft/mi)

9.6.1.3 Precipitation-Runoff Relationship

Generally, not all of the precipitation that falls on a watershed during a design storm becomes runoff; a portion is retained in the soil and on vegetation. Chow (1964), the USDA (1972), the NRCS (2004a), and the USBR (1973, 1987a) and other references on hydrology discuss the watershed characteristics that determine the amount of precipitation that becomes runoff. These characteristics include: (1) the types of soil and their effect on the amount of water seeping into the ground; (2) the conditions (wet, dry, snow covered or frozen) of the ground surface immediately prior to the precipitation (known as the antecedent moisture condition or AMC); (3) the type and density of vegetation; (4) the types of development, such as paved surfaces, channeling and storm sewers; and (5) dams, lakes, ponds, or swamps upstream of the site that could store water on a permanent basis, release water at a slow rate, or fail, thereby suddenly releasing large volumes of water.

9.6.1.3.1 General Rainfall Conditions

The NRCS (updating previous work when it was known as the SCS) has quantified precipitation runoff conditions for a wide range of soil, moisture and soil-cover conditions. The procedure utilizes a runoff curve number (CN) and the following equations:

$$Q = (P - I_a)^2/(P + 0.8S) \quad (9\text{-}10)$$

$$S = (1000/CN) - 10 \quad (9\text{-}11)$$

where:

Q = direct runoff (in)
P = rainfall (in)
I_a = initial abstraction = 0.2 S
S = maximum potential difference between P and Q at beginning of storm

Figures 9.17 and 9.18 present charts for estimating runoff from precipitation based on the above equations. Discussion of the derivation and use of these charts is presented in the NRCS (2004b) National Engineering Handbook (NEH) updating previous work by the USDA (1972). To determine the characteristic *CN*, it is necessary to know or to estimate four watershed conditions: (1) the hydrologic soils classification, (2) the land use and surface status or treatment, (3) the hydrologic effect of the land use and status, and (4) the antecedent moisture condition (AMC).

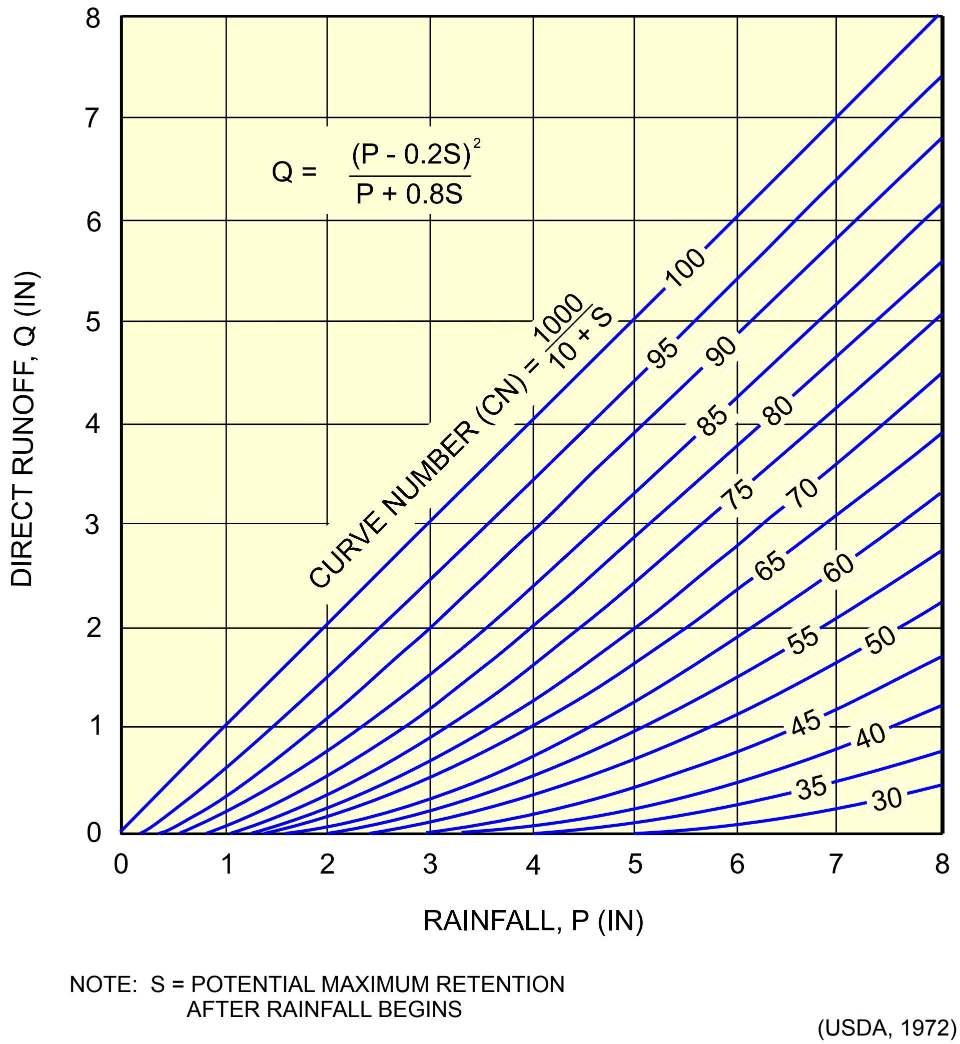


FIGURE 9.17 DIRECT RUNOFF FOR RAINFALL LESS THAN OR EQUAL TO 8 INCHES

Traditionally, the hydrologic soil classification has been determined from county soil surveys. The soils identified in the soil surveys are categorized into four hydrologic soil groups (HSGs), as described in NEH Chapter 7 (NRCS, 2007a). The HSGs for soils of the United States are presented in Appendix A of TR55 (NRCS, 1986; Appendix A updated 1999). Groups A through D for natural soils are described below. The disturbed area soil profile for each group for watershed areas that have been affected by mining related impacts and/or urbanization is also provided.

A. Low runoff potential – Soils having high infiltration rates even when thoroughly wetted – consisting chiefly of well to excessively drained deep sands or gravels, typically with less than 10 percent clay. These soils have a high rate of water transmission.

   Soil description: Sand, loamy sand, or sandy loam.

B. Moderately low runoff potential – Soils having moderately low infiltration rates when thoroughly wetted, consisting chiefly of moderately deep to deep soils with moderately fine to moderately coarse textures, moderately well to well drained, typically with 10 to 20 percent clay. These soils have a moderate rate of water transmission.

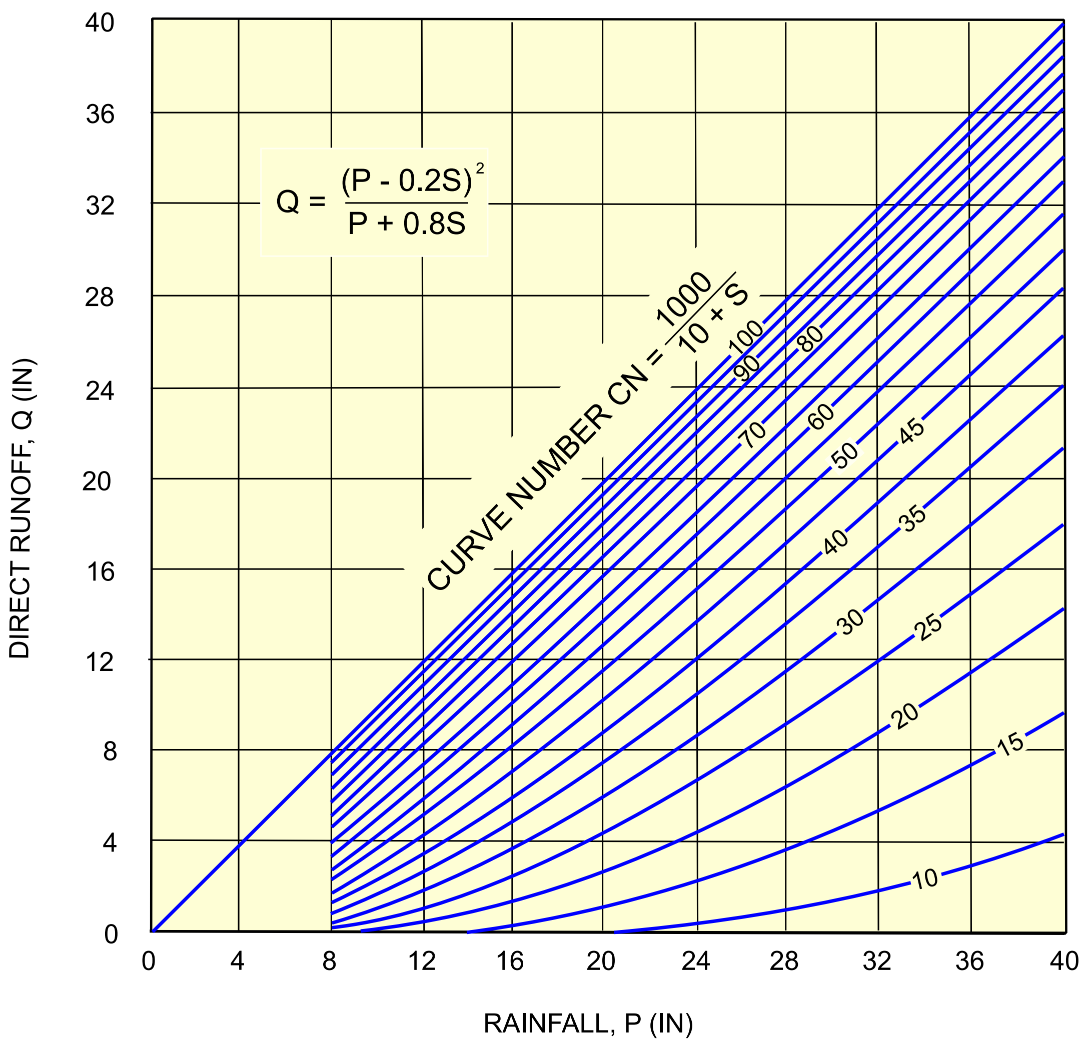
Soil Description: Silt loam or loam.

C. Moderately high runoff potential – Soils having slow infiltration rates when thoroughly wetted, consisting chiefly of soils with fine texture, or of soils with a layer that impedes downward movement of water, typically with 20 to 40 percent clay. These soils have a slow rate of water transmission.

Soil Description: Sandy clay loam.

D. High runoff potential – Soils having very slow infiltration rates when thoroughly wetted. These are typically clay soils with high swelling potential, soils with a claypan or clay layer at or near the surface, shallow soils over nearly impervious material, and soils with a permanent high water table. These soils typically have greater than 40 percent clay and a very slow rate of water transmission.

Soil Description: Clay loam, silty clay loam, sandy clay, silty clay, or clay



NOTE: S = POTENTIAL MAXIMUM RETENTION AFTER RAINFALL BEGINS

(USDA,1972)

FIGURE 9.18 DIRECT RUNOFF FOR RAINFALL GREATER THAN OR EQUAL TO 8 INCHES

The second and third watershed conditions are simply descriptions of watershed land use and surface status or treatment, such as woods or straight row, small grain crops, and the hydrologic density or impact of the land use. A general description of the hydrologic grading for soil-cover conditions typically encountered is described in conjunction with selection of the curve number (Table 9.10).

The fourth watershed condition is an index of watershed wetness referred to as the antecedent moisture condition (AMC). Antecedent moisture conditions are categorized into three groups:

AMC-I Optimum soil conditions – soils are dry but not to the point of wilting vegetation (not recommended for design).

AMC-II Average conditions and average value for annual floods.

AMC-III Wet or saturated conditions associated with heavy rainfall or light rainfall and low temperatures within 5 days prior to the given storm.

The antecedent moisture condition that should be used for design of impoundment structures is either AMC-II or AMC-III. The NRCS provides procedures for estimating the antecedent moisture condition based on the 5 days of antecedent rainfall. Typically AMC-II is used for the design of drainage channels, and AMC-III is used when determining the PMF for impoundment design.

The curve number can be estimated based on the hydrologic soils group, land use, and soil-cover conditions, as presented in Table 9.10 for AMC-II. Adjustments in the estimated *CN* value can be made for AMC-I and AMC-III using Table 9.11. The NRCS (2004b) provides methods for developing composite *CN* values when there are multiple antecedent moisture conditions.

Typically, basin or subbasin averaging is performed (as in the HEC-1 model) to represent watershed areas and compute runoff hydrographs using SCS or Snyder parameters. However, distributed calculations based on mapped soil and land use conditions yield more representative runoff estimates than basin-averaged parameters, and the availability of programs such as HEC-GeoHMS that utilize GIS terrain and spatial information will efficiently facilitate this approach when digitized soils surveys are available. An overview of procedures that can be employed is presented in FERC (2001).

When calculating runoff, it is important to consider actual conditions that may be present in the watershed, including the potential effects of any impoundments. The following guidelines are applicable to refuse disposal facilities:

- For watersheds having varying runoff characteristics, subbasins should be developed or a weighted average *CN* should be computed.
- The actual impoundment is generally considered impervious and contributes 100 percent to the runoff.
- Upstream impoundments require special consideration when determining runoff. Runoff hydrographs should be routed through such impoundments to confirm their operation during the design storm and the potential impact should the structures overtop and fail.

9.6.1.3.2 Thunderstorm Rainfall Conditions

Hydrometeorological reports for the western regions of the U.S. provide guidance for analysis of thunderstorm rainfall. For the Rocky Mountain region, the USBR (1987a) identifies a local, high-intensity thunderstorm event that should be considered along with the general storm event. Charts provide guidance for selection of lag time and development of unit hydrographs.

TABLE 9.10 RUNOFF CURVE NUMBERS FOR WATERSHED COMPLEXES AND AMC-II

Cover Description			Curve Numbers for Hydrologic Soil Group			
Cover Type		Hydrologic Condition	A	B	C	D
Cultivated Agricultural Lands(1, 2, 3, 4)						
Fallow	Bare soil	–	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T + CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T + CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C&T	Poor	63	73	80	83
		Good	51	67	76	80
Pasture, grassland, or range – continuous forage for grazing(5)		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
Other Agricultural Lands(1)						
Meadow – continuous grass, protected from grazing and generally mowed for hay		–	30	58	71	78
Brush – brush-weed-grass mixture with brush the major element(6)		Poor	48	67	77	83
		Fair	35	56	70	77
		Good	30(7)	48	65	73
Woods – grass combination (orchard or tree farm)(8)		Poor	57	73	82	86
		Fair	43	65	76	82
		Good	32	58	72	79

TABLE 9.10 RUNOFF CURVE NUMBERS FOR WATERSHED COMPLEXES AND AMC-II
(Continued)

Cover Description		Curve Numbers for Hydrologic Soil Group			
Cover Type	Hydrologic Condition	A	B	C	D
Woods[9]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30[7]	55	70	77
Farmsteads – buildings, lanes, driveways, and surrounding lots	–	59	74	82	86
Arid and Semiarid Range Lands[1, 10]					
Herbaceous – mixture of grass, weeds, and low-growing brush, with brush the minor element	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak-aspen – mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon-juniper – pinyon, juniper, or both; grass understory	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sagebrush with grass understory	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrub – major plants include saltbush, greasewood, creosotebush, blackbrush, bursage, palo verde, mesquite, and cactus	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

Note:
1. Average runoff conditions and $I_a = 0.2S$.
2. Crop residue cover applies only if residue is on at least 5 percent of the surface throughout the year.
3. Hydrologic condition is based on a combination of factors that affect infiltration and runoff, including: (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes, (d) percent of residue cover on the land surface (good ≥ 20 percent), and (e) degree of surface roughness.
4. Poor: Factors impair infiltration and tend to increase runoff.
   Good: Factors encourage average and better than average infiltration and tend to decrease runoff.
5. Poor: < 50 percent ground cover or heavily grazed with no mulch.
   Fair: 50 to 75 percent ground cover and not heavily grazed.
   Good: > 75 percent ground cover and lightly or only occasionally grazed
6. Poor: < 50 percent ground cover
   Fair: 50 to 75 percent ground cover
   Good: > 75 percent ground cover
7. Actual curve number is less than 30; use CN = 30 for runoff computations.
8. CN's shown were computed for areas with 50 percent woods and 50 percent grass (pasture) cover. Other combinations may be computed from the CN's for woods and pasture.
9. Poor: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
   Fair: Woods are grazed, but not burned, and some forest litter covers the soil.
   Good: Woods are protected from grazing, and litter and brush adequately cover the soil.
10. Poor: < 30 percent ground cover (litter, grass, and brush overstory)
    Fair: 30 to 70 percent ground cover
    Good: > 70 percent ground cover

(ADAPTED FROM NRCS, 1986)

TABLE 9.11 RUNOFF CURVE NUMBERS (*CN*) FOR ANTECEDENT MOISTURE CONDITIONS

AMC II	AMC I (Not Recommended for Hydrologic Design)	AMC III
100	100	100
95	87	98
90	78	96
85	70	94
80	63	91
75	57	88
70	51	85
65	45	82
60	40	78
55	35	74
50	31	70
40	22	60
30	15	50
20	9	37
10	4	22
0	0	0

(USDA, 1972)

9.6.1.4 Channel and Impoundment Storage Characteristics

For non-impounding coal refuse disposal facilities, storage normally does not enter into the hydrologic and hydraulic analyses. Channels are generally designed for the peak runoff rate, and storage beyond the channel capacity is not a consideration. Channel and flood plain storage can be a consideration when evaluating some natural drainage systems, and hydraulic analysis software can incorporate the associated storage into the model through cross sections determined from topographic maps. For impounding disposal facilities, reservoir storage provisions are an important component of the hydraulic design and can vary between:

- The condition where the impoundment does not have reservoir storage capacity to handle any significant portion of the runoff during a storm, requiring the outflow to essentially equal the inflow rate. This condition usually requires a relatively large spillway to pass the storm inflow.
- The condition where the impoundment does have storage capacity to temporarily handle all runoff during a storm, allowing the immediate outflow discharge to be essentially zero. This condition requires an embankment high enough to safely store the storm inflow.

Most impounding disposal facilities during some period of their life fall between these extremes, and part of the runoff becomes reservoir storage and the remainder is passed as outflow. An important factor that differentiates coal refuse disposal facility impoundments from other types of impoundments is that the embankment and impoundment configurations continually change with both the disposal of coarse refuse on the embankment and the disposal of fine refuse slurry into the impoundment. This variation must be taken into account in the hydraulic design of a disposal facility.

Several terms commonly used to describe conditions associated with an impoundment are defined below:

- Normal pool elevation – The surface elevation of water or slurry impounded by an embankment during normal disposal operations. The normal pool elevation is usually established by the decant inlet level or pump control level. With slurry impoundments, the normal pool elevation changes with time as fine refuse slurry is disposed and the decant level or pump control level is raised to successively higher elevations.
- Minimum pool elevation – The lowest surface elevation that can normally be attained. This is often the same as the normal pool elevation, but can be lower if drainage is provided by siphoning, pumping, or modifying the outlet system.
- Useful storage – The storage capacity between the normal and minimum pool elevations.
- Dead storage – The storage capacity below the minimum pool elevation that is usually filled primarily by settled slurry.
- Surcharge storage – The storage capacity between the normal pool elevation and the maximum permissible pool elevation. This capacity is intended primarily for temporary storage of runoff during storms.
- Freeboard – The difference in elevation between the dike or embankment crest (i.e., lowest portion of impoundment perimeter except for an open-channel spillway) and the impoundment water surface. Normal freeboard is the distance between the minimum embankment crest elevation and the normal pool level, as established by the lowest outlet structure used for flood routing purposes. Design-storm freeboard is the distance between the embankment crest and the maximum pool level during the design storm. The design-storm freeboard should be such that the embankment is not overtopped.

Reservoir storage volumes can be determined in several ways, all based on the topographic configuration of the impoundment area. Topographic data, generally obtained by aerial photography with 2- to 5-foot contour intervals, are suitable for most analyses. USGS topographic quadrangle maps may be used for preliminary calculations for evaluation of initial feasibility.

The normally preferred procedure for calculating the elevation-volume relationship for an impoundment is described in Section 9.2.3 and is illustrated in Figure 9.4. The impoundment surface area for each successive elevation contour is first determined, from which an area-elevation curve can be plotted. The storage is then computed as the area beneath the area-elevation curve at any elevation, from which the volume-elevation curve can be plotted. CADD software can be used to generate area- and volume-elevation data. Typically, both area and volume curves are presented as part of an impoundment design report or plan.

Design of impoundments for upstream or centerline construction involves placement of subsequent embankment stages within the impoundment area, which impacts reservoir storage associated with that stage of construction. Accordingly, the impoundment area-elevation and volume-elevation data for each stage of construction must allow for the effect of upstream or centerline construction.

Surcharge storage and freeboard to accommodate the design-storm runoff is required for most impoundments. In determining the surcharge storage, the volume of the settled fines above the pool level (delta deposits) must be taken into account, particularly for diked configurations where slurry is discharged over a substantial perimeter of the reservoir. The volume of the delta deposits is generally estimated based upon the position and elevation of the slurry discharge and an assumed slope of the deposit (typically between 1 and 3 percent).

Freeboard at an impoundment is provided in order to account for such factors as uncertainties in the hydrologic analyses, settlement of the embankment crest, and extreme wind effects such as wave runup. The design-storm freeboard for any impounding embankment is a function of the wave height and the wave runup conditions at the upstream face of the embankment. Guidance for the evaluation of freeboard for reservoirs is provided in USBR (1987a). Other factors that should be considered in determining freeboard requirements include: (1) frequency of the design storm, (2) duration of high water level, (3) ability to resist erosion, (4) and potential for settlement or mine subsidence. A reference for wave runup analysis is the *Coastal Engineering Manual* developed by the USACE (2002). Coal refuse impounding embankments are typically required to have a design-storm freeboard of 3 feet, which is consistent with a wind fetch of generally less than one mile.

9.6.2 Runoff Determination: Hydrograph Method

When a time history of runoff is required for final reservoir routing (Section 9.8), a runoff hydrograph must be developed. As illustrated in Figure 9.3, a hydrograph is a plot of flow rate at the point of interest versus time following storm initiation. Development of a runoff hydrograph requires the watershed runoff data discussed in the preceding section. The data are used to create unit value runoff hydrographs for small time increments within the total storm duration. A unit hydrograph models the time history of runoff flowing from the watershed at the point of interest for one time increment of precipitation. A composite runoff hydrograph can then be constructed by superimposing unit hydrographs for all increments of precipitation.

Computer programs developed by the USACE Hydrologic Engineering Center (HEC) have often been used for the hydrologic and hydraulic design of coal refuse disposal facilities. The HEC-1 Flood Hydrograph package calculates runoff hydrographs and has several optional capabilities. Use of HEC-1 for precipitation-runoff modeling requires subbasin boundary delineation, precipitation data, and runoff and routing parameters. Typically, synthetic unit hydrographs based on the SCS, Snyder or Clark methods are employed. The HEC-Hydrologic Modeling System (HEC-HMS) computer program provides advancements in several areas over the HEC-1 program, including the optional use of distributed analysis of runoff through interfacing with GIS terrain models.

The following capabilities of HEC-1 and HEC-HMS are frequently utilized during the design of coal refuse disposal facilities:

- Distributed runoff analysis using kinematic wave and Muskingum-Cunge routing, which can provide a more refined analysis of peak flows for impoundment designs incorporating open-channel spillways.
- Modeling of base flow, which may be important for large watersheds and inundation analysis.
- Channel routing using a variety of methods (e.g., Muskingum, Modified Puls, etc.) to perform inundation analysis and evaluation of impoundments in series.
- Reservoir routing (level pool routing) based on storage, elevation, and outflow rating parameters, which is a basic requirement for impounding facility design.
- Dam breach and inundation analysis for determination of the hazard-potential classification for an impounding facility and for preparation of an Emergency Action Plan.

User's Manuals for HEC-1 (USACE, 1998b) and HEC-HMS (USACE, 2000) provide documentation of their capabilities and applications. Several private companies market these programs with enhanced input and output features and also have developed similar programs with enhanced capabilities.

Regardless of the method of hydrograph development and analysis, the following computation and data validation issues should be considered:

- The computational time increment should be selected based on the lag time and precipitation data such that a smooth hydrograph is obtained. The maximum time increment of rainfall to be used in the hydrograph analysis should be the lag time divided by 5 ($L_g/5$) rounded to the next lower even number (FERC, 2001). The USACE (1998b) indicates that the time increment should be no larger than 0.29 L_g. It can be problematic to apply this limitation to small watersheds with small lag times because some software programs limit the time increment; however HEC-HMS does not have this limitation. For impoundments that are designed to store the design storm with release through a decant pipe, the volume of runoff and peak impoundment pool level is relatively insensitive to the computation time increment.
- Determination of the lag time and time of concentration using multiple methods is a useful check of the parameter validity prior to use in hydrograph simulation. If a regional analysis is performed to estimate these parameters, performing a check on the time of concentration using the TR-55 computer program should be considered.
- The applicability of any hydrograph development method is subject to uncertainty, and verification is sometimes accomplished by investigating multiple methods (and sources of watershed parameters such as the USACE) or by reproducing a large historical flood of record from a watershed with similar characteristics. This consideration is most important for impoundments where the flood hydrograph is routed through an open-channel spillway.

9.6.3 Peak Runoff Determination – Key Parameters Method

When a complete time history of runoff is not required, estimates of the key runoff parameters can be used to determine the total runoff volume and peak runoff rate developed during a design storm. The key parameters method is useful for final design when: (1) the entire volume of runoff will be temporarily stored so that the rate of runoff does not need to be calculated or (2) the storage capacity is so small that the peak outflow rate is essentially the same as the peak runoff rate and reservoir routing is not necessary. The key parameters method is also useful for estimating approximate storage and outflow requirements during feasibility planning for various facility configurations and preliminary sizing for various hydraulic structures prior to undertaking more detailed procedures using hydrographs and reservoir routing analyses.

The USDA (1973b) published charts for the determination of total runoff volume and peak runoff rates for small volumes of runoff that can be used to estimate key parameters for design purposes. These charts provide runoff values based on watershed size, slope, and runoff curve number (CN).

Figures 9.19 and 9.20 present total runoff volume as a function of total precipitation and watershed size for runoff $CN = 70$ and $CN = 80$, respectively. The curves were derived directly from Figures 9.17 and 9.18 and do not include the minimum retentions due to soil infiltration discussed in Section 9.6.1.3. Through interpolation and extrapolation, these curves may be used to estimate total runoff volume for other CN values.

Figures 9.21 and 9.22 may be used for estimating the peak runoff rate for a range of six-hour design storms for $CN = 70$ and $CN = 80$. These figures were prepared from hydrograph analyses utilizing the recommended SCS rainfall distribution presented in Figure 9.15. Through interpolation or extrapolation, these curves can be used to estimate the peak runoff rate for other CN values between 70 and 80.

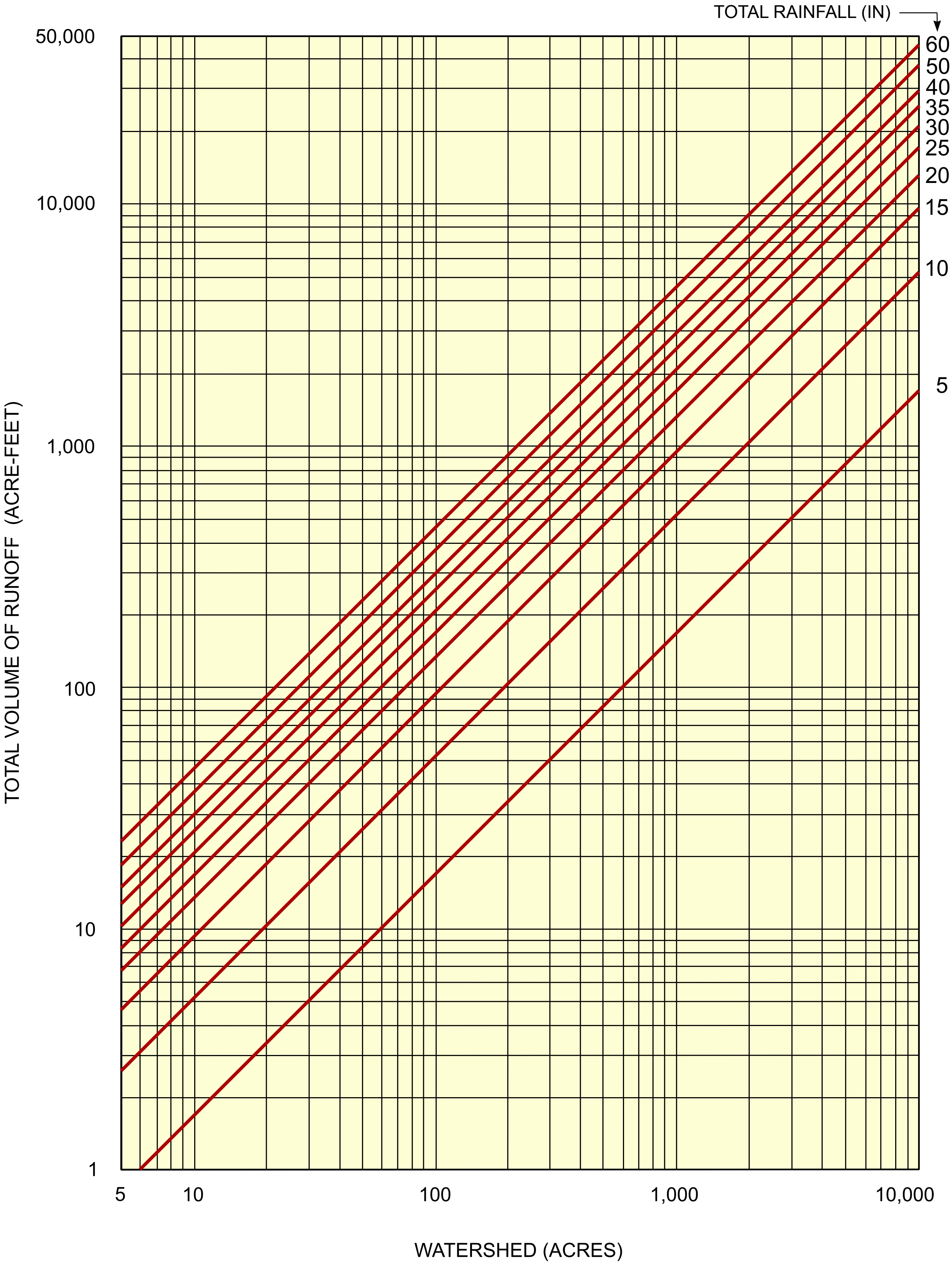


FIGURE 9.19 ESTIMATED TOTAL RUNOFF VOLUME FOR *CN* = 70

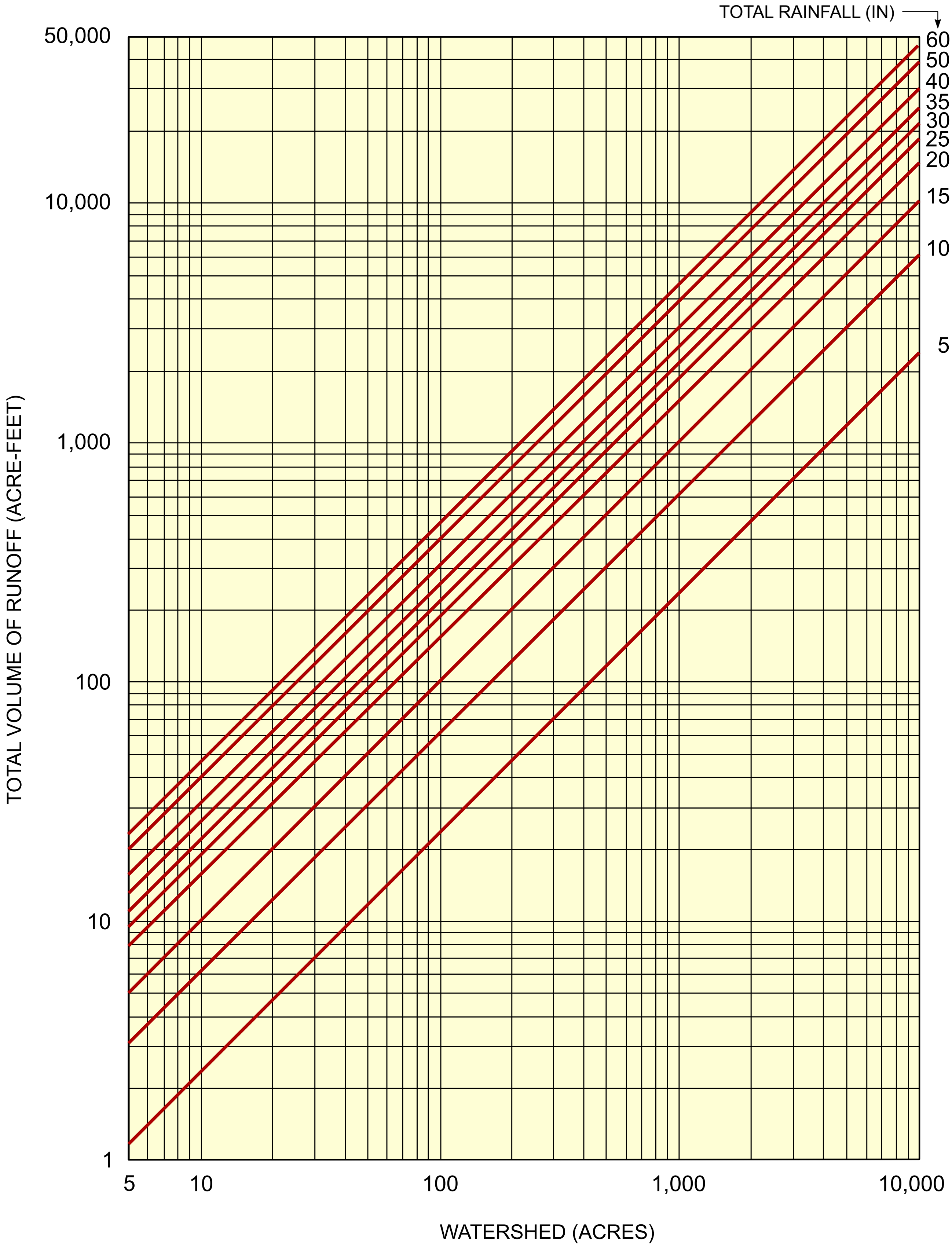


FIGURE 9.20 ESTIMATED TOTAL RUNOFF VOLUME FOR *CN* = 80

9.6.4 Runoff Determination – Rational Method

The rational method is the simplest procedure for estimating peak runoff rates and is typically used for developing design flows for minor drainage features. Originally developed by the U.S. Bureau of Public Roads and subsequently updated by the FHWA (2001), the method is usually restricted to watersheds of less than 200 acres and to storm recurrence intervals of less than 100 years. The USGS (2005) presents a review and comparison of the method with observed runoff events. Using the rational method, the peak rate of runoff is determined from the following relationship:

$$Q = C\,i\,A \tag{9-12}$$

Q = peak rate of runoff (cfs)

C = weighted average runoff coefficient

i = average precipitation intensity for a duration equal to the watershed time of concentration and the selected storm recurrence interval (in/hr)

A = watershed area tributary to the point of interest (acres)

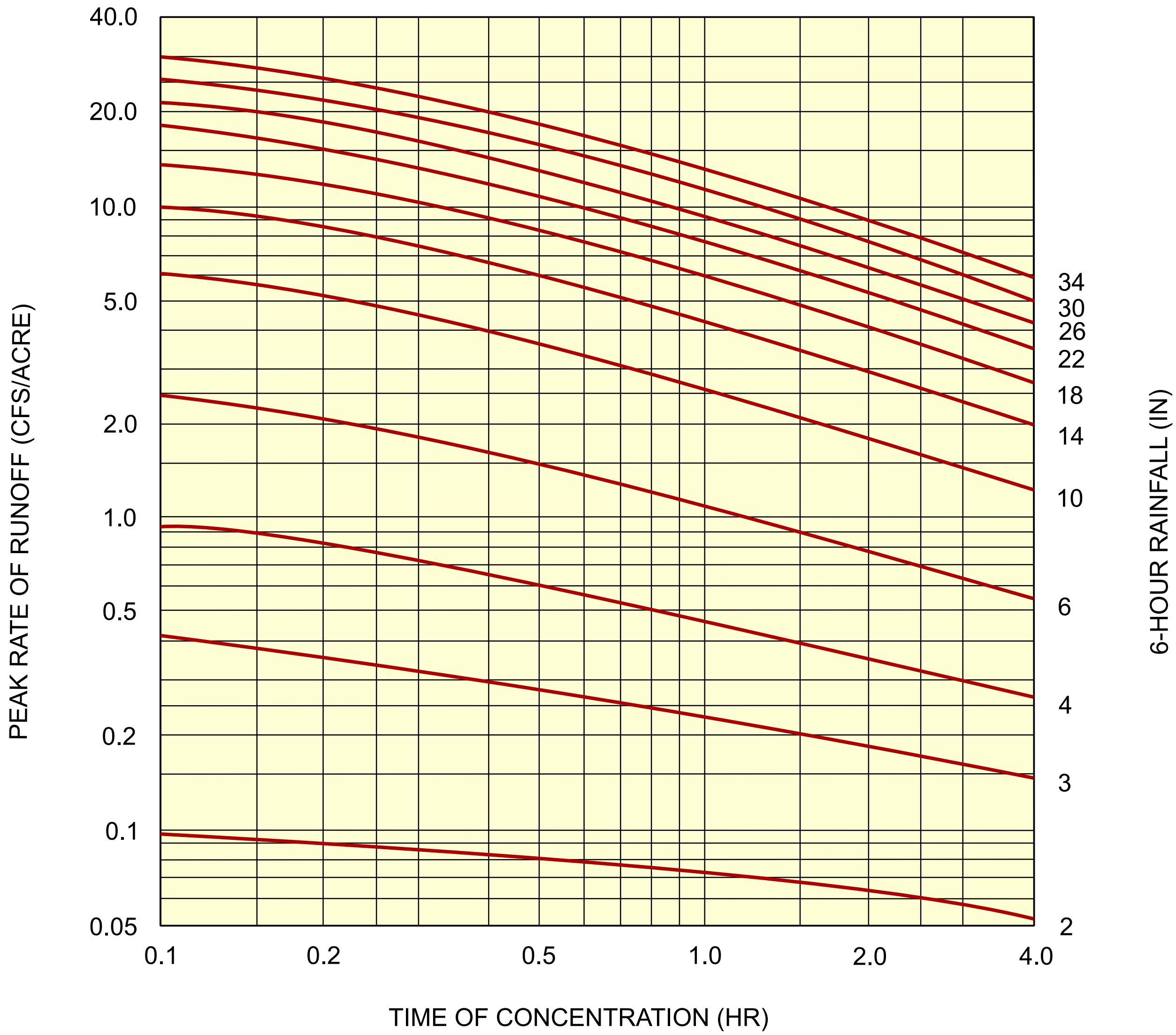


FIGURE 9.21 ESTIMATED PEAK RUNOFF RATE FOR 6-HOUR DESIGN STORM AND *CN* = 70

The runoff coefficient *C* can be further defined as the ratio of the rate of runoff to the rate of precipitation during the storm when all the drainage area is contributing to the runoff. The value of C can be estimated from the data provided in Table 9.12. The range of the coefficients for rural areas permits some allowance for differences in slope and ground cover conditions. The lower values in the table should only be used when the watershed is flat and the surface is permeable. When the watershed exhibits multiple slope and ground cover conditions, the runoff coefficient should be determined as a weighted average based on the relative area of each of the conditions present.

The time period is equal to the time of concentration. This is the time required for the entire drainage area to be contributing to the runoff. The average precipitation intensity (*i*) can be determined from precipitation frequency tables presented by the National Weather Service Precipitation Frequency Data Server (web site) as an update for several states covered by Technical Paper 40. Table 9.5 presents publications for precipitation frequency data.

It is emphasized that use of the rational method for determining runoff should be limited to the design of minor drainage appurtenances at a coal refuse disposal facility. An example of application of the rational method is presented in Figure 9.23.

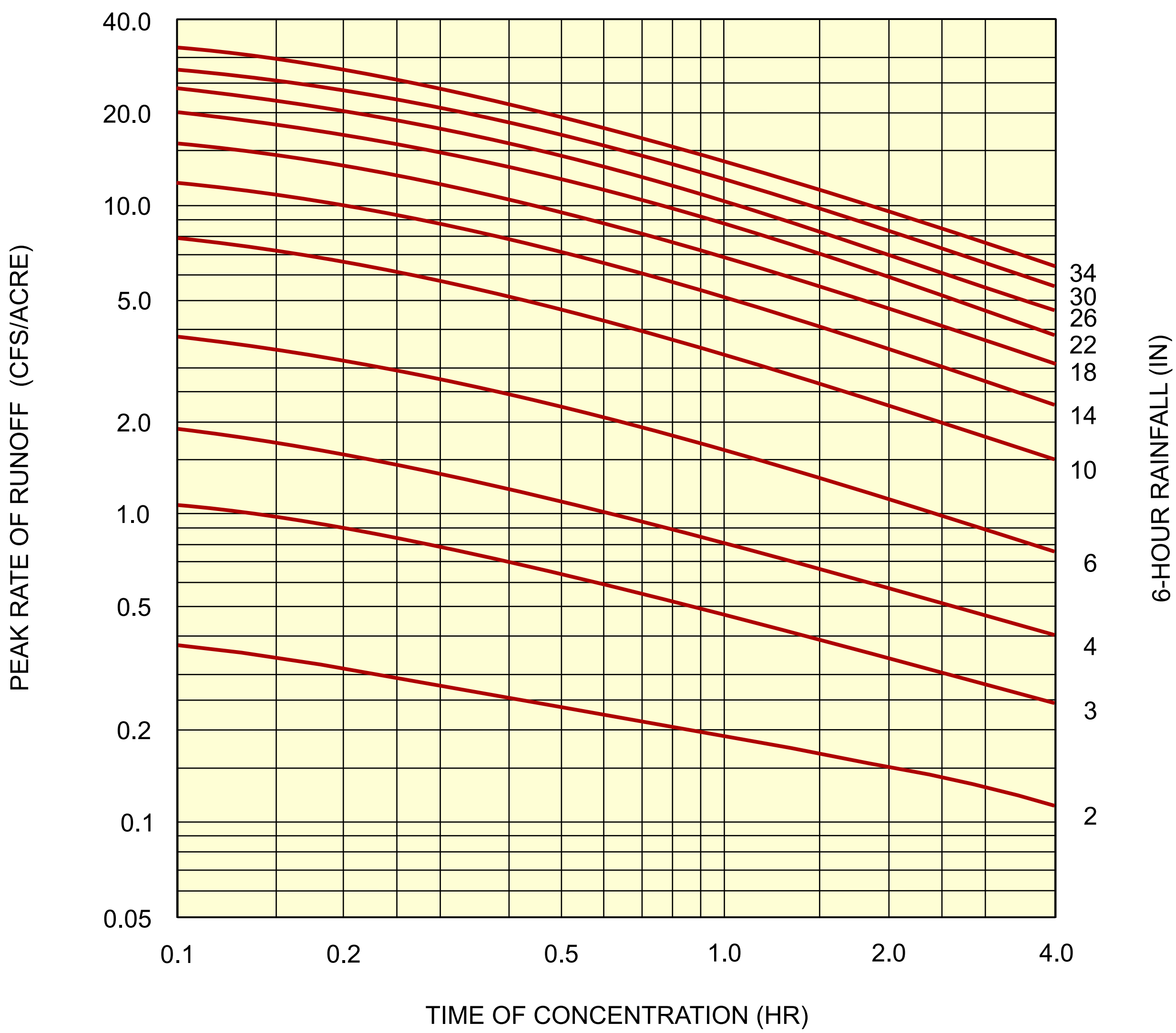


FIGURE 9.22 ESTIMATED PEAK RUNOFF RATE FOR 6-HOUR DESIGN STORM AND *CN* = 80

TABLE 9.12 RUNOFF COEFFICIENTS FOR USE IN THE RATIONAL METHOD

Rural Areas (Haan and Barfield, 1978)

Cover Type	Terrain		Soil Texture Runoff Coefficient		
			Open Sandy Loam	Clay and Silt Loam	Tight Clay
Woodland	Flat	(0-5% slope)	0.10	0.30	0.40
	Rolling	(5-10% slope)	0.25	0.35	0.50
	Hilly	(10-30% slope)	0.30	0.50	0.60
Pasture	Flat		0.10	0.30	0.40
	Rolling		0.16	0.36	0.55
	Hilly		0.22	0.42	0.60
Cultivated	Flat		0.30	0.50	0.60
	Rolling		0.40	0.60	0.70
	Hilly		0.52	0.72	0.82

Rural Areas (FHWA, 2001)

Cover Description	Runoff Coefficient$^{(1)}$
Concrete or asphalt pavement	0.8 to 0.9
Asphalt macadam pavement	0.6 to 0.8
Gravel roadways or shoulders	0.4 to 0.6
Bare earth	0.2 to 0.9
Steep, grassed areas (2:1 slope)	0.5 to 0.7
Turf meadows	0.1 to 0.4
Forested Areas	0.1 to 0.3
Cultivated fields	0.2 to 0.4

Note: 1. For flat slopes or soils with high hydraulic conductivity, the lower values should be used; for steep slopes or low-hydraulic-conductivity soils, the higher values should be used.

9.7 DESIGN OF OUTFLOW SYSTEMS

This section presents the planning and design requirements for hydraulic systems to safely transport watershed runoff through, around and beyond a coal refuse disposal facility. Because the discussion is general in scope, frequent references for specific applications are made to texts and publications on hydraulic design and engineering, including: Chow (1959), USBR (1987a), Brater et al. (1996), Henderson (1966), and the USACE (1990b).

The analytical steps necessary for hydraulic design are discussed with emphasis on the important relationship of each system component to the overall requirements of the disposal facility. The discussion is presented in the following order:

- The basic considerations that determine the types of analyses to be performed.
- Introduction to basic hydraulic system components including the inlet, transport and outlet components for both open-channel and closed-conduit flow.

- Identification of special design considerations for hydraulic structures including erosion protection, effects of direction changes, cavitation and materials selection.
- Discussion of the types of principal hydraulic systems commonly encountered at coal refuse disposal facilities including spillways, decants, diversion ditches, culverts and natural streams.

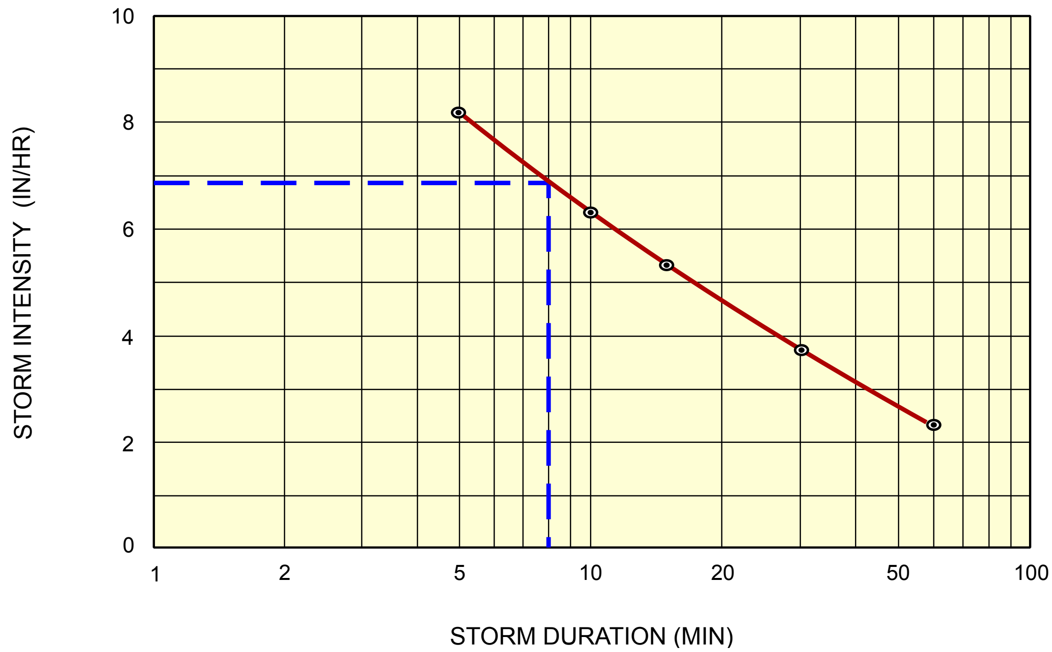
9.7.1 Basic Considerations

Geometric design constraints, the coarse and fine coal production rates and the estimated runoff rate and volume influence the selection of the outflow structure(s) that can be utilized for a coal refuse disposal facility embankment or impoundment. Basic considerations in the design of hydraulic systems are noted for the following four situations:

1. No runoff storage – In this case, the hydraulic system must be designed to handle the maximum rate of runoff (i.e., outflow capacity is equal to the peak runoff rate).
2. Maximum reservoir storage – Reservoir storage is maximized with limited spillway outflow during the design storm event (Section 9.8). In this case, the designer must provide outflow capacity for all runoff in excess of the available storage. Under some conditions the optimal solution may involve total storage of the design storm runoff with delayed release through a decant system.
3. Defined surcharge storage capacity – Surcharge reservoir storage is available, but the available volume is generally limited. In this case, an iterative procedure must be employed to establish the optimum balance between reservoir storage and outflow, after which the calculated maximum outflow is used to design the hydraulic system.
4. Fixed hydraulic structure capacity – For this case, the hydraulic system capacities are fixed and the runoff that will cause failure must be determined. This situation is most often encountered when evaluating an existing facility or the flow in a natural stream.

The types of impounding and non-impounding disposal facilities and their unique characteristics have been discussed in Sections 9.3 and 9.4. The major points of these previous discussions are summarized below:

- To accommodate the progressive development and operation of coal refuse disposal facilities, intermediate-stage hydraulic drainage systems may be needed.
- All operational periods of the impoundment hydraulic systems will need to be evaluated if the progressive development of a coal refuse disposal facility impacts available reservoir storage and/or outflow capacity.
- Coal operators may be performing site development construction and, as a consequence, have certain types of equipment always available on site; the design of hydraulic system components should reflect the equipment available and operator capabilities.
- The reclaimed configuration of a coal refuse disposal facility may impact hydraulic structure design at intermediate facility stages. The final facility abandonment configuration must be taken into account in the design of intermediate stage hydraulic structures.
- The design of hydraulic structures should take into account corrosion, abrasion, erosion, weathering and maintenance requirements.



PROBLEM DETERMINE THE RUNOFF FROM A 100-YEAR RECURRENCE INTERVAL STORM FOR A STEEPLY-SLOPED, 85-ACRE WATERSHED IN ALLEGHENY COUNTY, PENNSYLVANIA. THE WATERSHED HAS THE FOLLOWING CHARACTERISTICS:

LONGEST WATERCOURSE = 1900 FEET
ELEVATION DIFFERENCE = 100 FEET
TOPOGRAPHY = 60% WOODED, 30% PASTURE, 10% BARE EARTH

SOLUTION 1. DETERMINE A WEIGHTED-AVERAGE RUNOFF COEFFICIENT FROM TABLE 9.12

TOPOGRAPHY	RUNOFF COEFFICIENT
WOODED-STEEP	0.30
PASTURE-STEEP	0.70
BARE EARTH-STEEP	0.90

THEN C = 0.60 (0.30) + 0.30 (0.70) + 0.10 (0.90) = 0.48

2. DETERMINE 100-YEAR INTENSITY-DURATION RELATIONSHIP FROM TP-40 (HERSHFIELD, 1961)

DURATION		PORTION OF 30-MIN RAINFALL	RAINFALL	INTENSITY
HR	MIN		IN	IN/HR
1	60	—	2.35	2.35
½	30	1.00	1.85	3.70
¼	15(1)	0.72	1.33	5.32
—	10(1)	0.57	1.05	6.30
—	5(1)	0.37	0.68	8.16

NOTE: 1. FROM PAGE 5 OF TP-40.

3. DETERMINE 100-YEAR RAINFALL INTENSITY (i) FOR SITE

FROM FIGURE 30 OF USBR (1973), FOR L = 1,900 FEET AND H = 100 FEET, T_C = 8 MINUTES

FROM INTENSITY-DURATION PLOT ABOVE, i = 6.9 INCHES/HOUR

4. CALCULATE 100-YEAR PEAK RUNOFF FOR SITE:

$Q = C\,i\,A$ = 0.48 (6.9) (85) = <u>281.5 CFS</u>

FIGURE 9.23 EXAMPLE OF RATIONAL METHOD OF INFLOW CALCULATION

9.7.2 Hydraulic System Components

Hydraulic systems that transport runoff through, around and beyond coal refuse disposal facilities have three basic components:

1. The inlet, where flow enters the system.
2. The transport or conveyance section that carries flow between the inlet and outlet.
3. The outlet, where water is discharged in an acceptable manner.

The rate of flow can be controlled by any portion of the hydraulic system by varying the size, location, elevation, slope, shape or configuration of these three components. Interrelationships between the inlet, the transport section and the outlet must be evaluated as part of hydraulic system design. An important design consideration when optimizing the balance between storage and outflow (Section 9.8) is that the maximum head of water needed to develop the desired rate of outflow does not exceed the maximum permissible impoundment pool elevation at any time during the life of the structure.

The following sections detail important aspects of each hydraulic system component, identifying appropriate design procedures and discussing the role of each component in various types of hydraulic systems.

9.7.2.1 Inlets

The inlet to a hydraulic system can: (1) simply direct flow to the transport section or (2) regulate the rate of flow (volume and/or velocity) to the transport section.

In the first case, the inlet must have a larger flow capacity than the transport section so that the inlet does not restrict the flow, and the inlet must be arranged such that water cannot bypass the transport section. Often the design requirement for regulating the amount of flow passing into the transport section is associated with the entrance to a decant system (Section 9.7.4.1) or to a spillway (Section 9.7.4.2). In these cases, a primary design criterion is the relationship between the height of water above the controlling elevation of the inlet, referred to as head H and the discharge Q. Typical head-discharge curves for decant-conduit and spillway systems are presented in Figure 9.24.

As shown in Figures 9.25 and 9.26, most inlets have a form of weir control, and the basic equation for the associated flow is:

$$Q = CLH^{3/2} \tag{9-13}$$

where:

Q = discharge or rate of flow (cfs)

C = a coefficient that depends on the shape of the channel entrance and the head on the structure

L = the effective length of the entrance crest (ft)

H = the total head on the entrance crest (ft)

Equation 9-13 can be used to determine flow rates over both broad- and sharp-crested weirs. Broad-crested weirs are characterized by small H/B ratios, where H and B are as defined in Figure 9.25a. An unrestricted channel entrance, such as shown in Figure 9.25c, can sometimes be described as a broad-crested weir. The discharge coefficient C for broad-crested weirs is a function of the weir geometry and

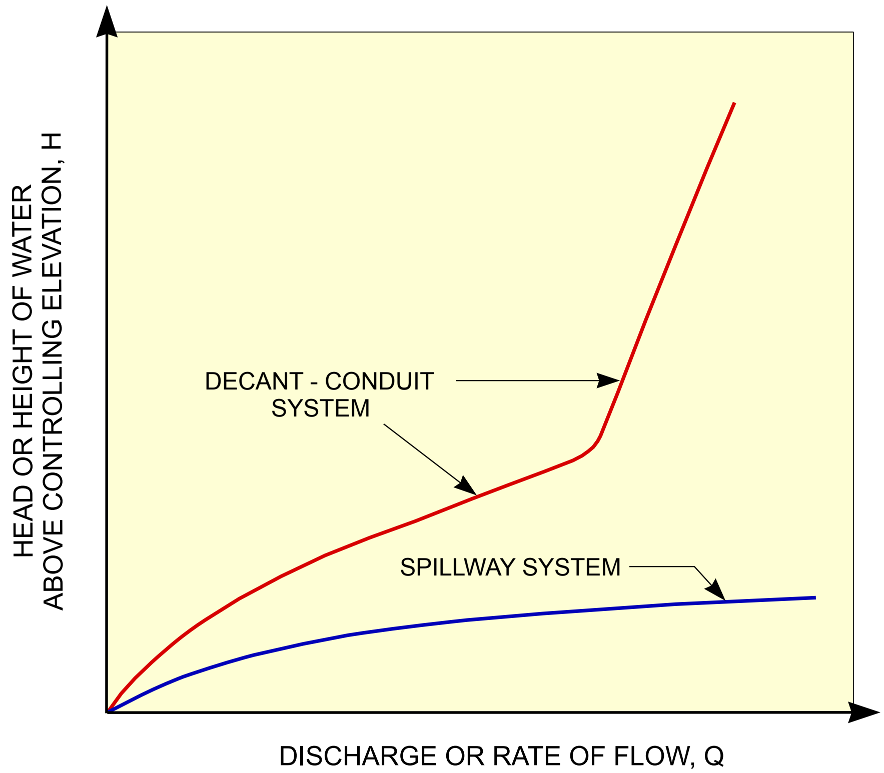


FIGURE 9.24 TYPICAL HEAD-DISCHARGE CURVES FOR HYDRAULIC SYSTEMS

approach conditions, as discussed in texts on hydraulics such as Brater et al. (1996), Chow (1959), and Henderson (1969). The value of the weir coefficient is generally about 3.0, but varies based on the weir configuration. Hydraulic analysis of the discharge and depth of flow through a broad-crested weir, including charts for determining flow profiles, is presented in USDA Technical Release 39 (1968).

Discharge coefficients for sharp-crested weirs, such as the decant inlets shown in Figure 9.26, are related to the weir height and the hydraulic head. The value of C can be determined from the revised Rehbock formula (Brater et al., 1996), which has been verified in tests performed by the USBR:

$$C = 3.22 + 0.44\,(H/P) \qquad (9\text{-}14)$$

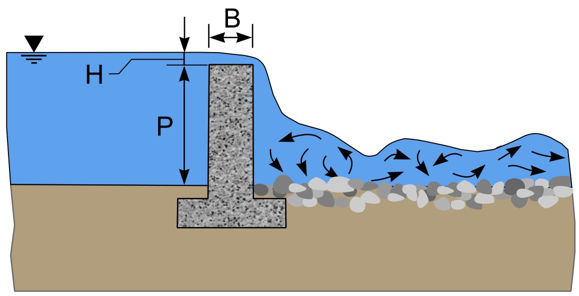
where:

H = hydraulic head (ft)

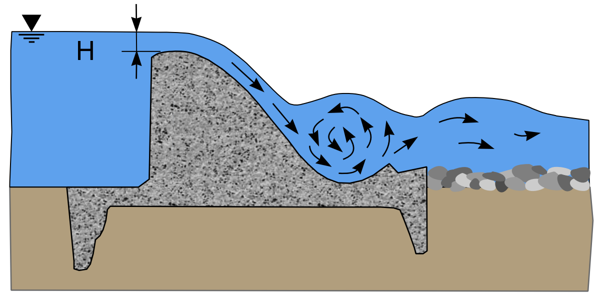
P = height of weir (ft)

A range of weir coefficients is presented in Figure 9.25b for an ogee crest. The designer is referred to the USBR (1987a) for a more detailed presentation of discharge coefficients for ogee crests. The USACE (1990b) presents design methods and discharge coefficients for elliptical (ogee) spillways and spillways with free outfall.

With increasing head on a decant or spillway conduit, the weir will become submerged and orifice flow conditions may govern the head-discharge relationship, or pressure flow may prevail depending on the transport section. Under orifice flow, the control section is located within the conduit or throat of the transition between the inlet and transport section, below the crest of the vertical intake shown in Figure 9.26a. USBR (1987a) presents procedures for determining the head-discharge rela-



9.25a RECTANGULAR WEIR



9.25b OGEE CREST DAM

$Q = C\ L\ H^{1.5}$

C = WEIR COEFFICIENT

L = LENGTH OF CREST

H = HEAD ON CREST

B = BREADTH OF CREST

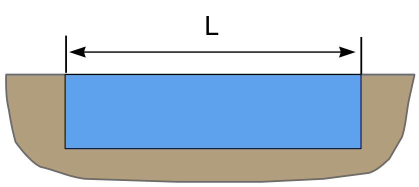
P = HEIGHT OF WEIR

$Q = C\ L\ H^{1.5}$

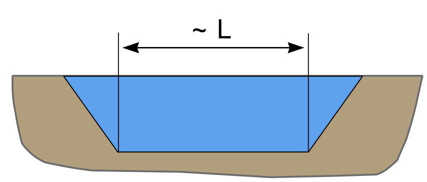
C = 3.80 TO 3.95

L = LENGTH OF CREST

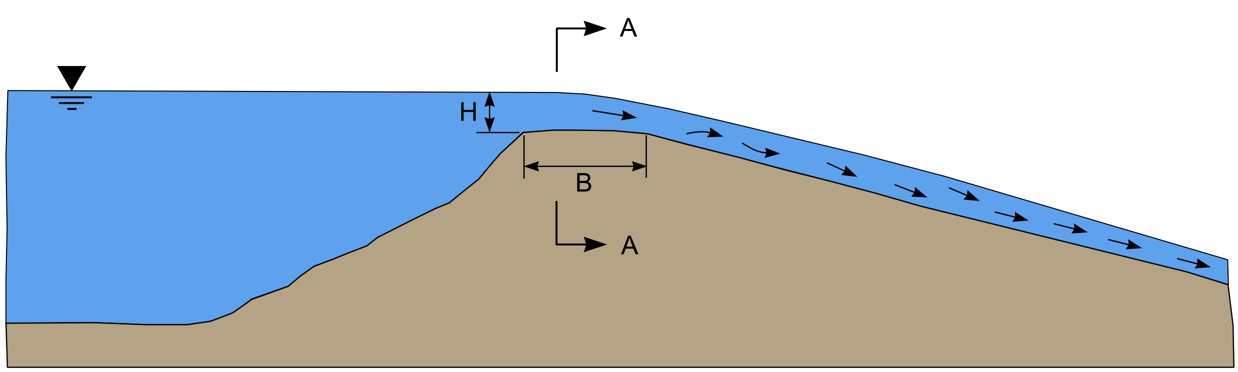
H = HEAD ON CREST



SECTION A - A
RECTANGULAR FLOW SECTION



SECTION A - A
TRAPEZOIDAL FLOW SECTION

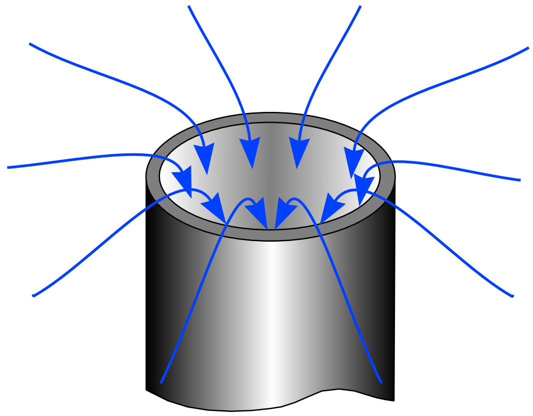


9.25c UNRESTRICTED CHANNEL ENTRANCE

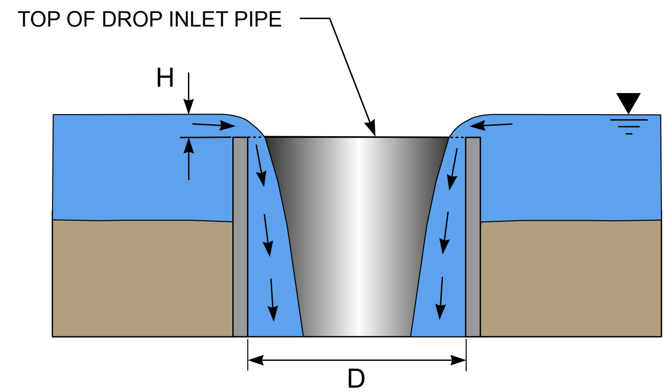
FOR STEEP CHANNEL SLOPE $Q = C\ L\ H^{1.5}$ WHERE C = 3.0

FOR MILD CHANNEL SLOPE Q DEPENDS ON CHANNEL GEOMETRY AND MAY BE CONTROLLED BY DOWNSTREAM CONDITIONS

FIGURE 9.25 SPILLWAY APPROACH CHANNELS



9.26a CIRCULAR DROP INLET PIPE



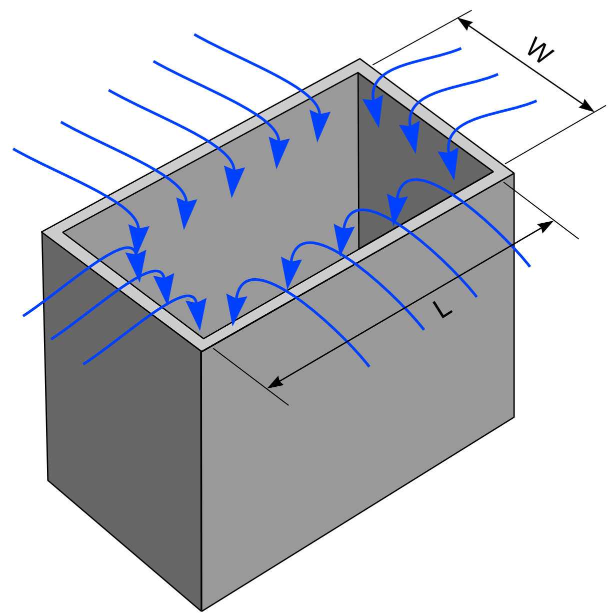
TWO-DIMENSIONAL CROSS SECTION
OF CIRCULAR DROP-INLET PIPE

$Q = C\pi DH^{1.5}$
C = WEIR COEFFICIENT
D = DIAMETER OF DROP-INLET PIPE
H = HEAD AT TOP OF DROP-INLET PIPE

$Q = 2C(W+L)H^{1.5}$
C = WEIR COEFFICIENT
W = WIDTH OF DROP INLET
L = LENGTH OF DROP INLET
H = HEAD AT TOP OF DROP INLET

NOTE: 1. FLOW IS SHOWN ON FOUR SIDES OF DROP INLET. IF PARTIALLY OBSTRUCTED, THE ACTUAL FLOW LENGTH SHOULD BE USED.

2. THE INLET CONTROL FLOWS ARE ONLY APPLICABLE PRIOR TO PRESSURIZED PIPE FLOW.



9.26b RECTANGULAR DROP INLET

FIGURE 9.26 DROP-INLET DECANT SYSTEMS

tionship for conduit spillways. For small-diameter conduits, as typically found with many decant structures, the head-discharge relationship under orifice flow control can be estimated as follows:

$$Q = CA(2gH)^{0.5} \tag{9-15}$$

where:

C = coefficient of discharge (dependent on configuration of conduit opening)
A = cross-sectional area of conduit (ft^2)
g = gravitational acceleration (ft/sec^2)
H = total head on the conduit entrance (ft)

For orifice flow, the coefficient of discharge C for vertical, circular decant systems is typically about 0.6 for square-edged conditions. Brater et al. (1996) and Haan and Barfield (1978) present the results of studies of varying orifice conditions and associated coefficient of discharges for various inlet shapes.

9.7.2.2 Transport Sections

The transport or conveyance section of a hydraulic system conveys flow from the inlet to the outlet. The two basic types of flow in transport sections are:

1. Open-channel flow – Gravity flow of water through an open channel where the flow depth and velocity depend upon the cross section, surface material and channel slope, and possibly also upon interrelationships with the inlet and outlet. Open-channel flow is typical for spillway channels, diversion ditches and natural streams, but may also apply to a less-than-full culvert or a decant conduit.
2. Pressure flow – Pressure flow through a closed conduit or pipe where the flow capacity depends upon the inlet and outlet conditions, the pressure head on the conduit and the conduit size and material. This type of transport section is common to decant systems.

Flow within a spillway system, as illustrated in Figure 9.25a and 9.25b, where the transport section is simply the free-fall flow over a weir or high-velocity flow over an ogee crest, are not generally encountered at coal refuse disposal facilities. The designer is referred to the USBR (1987a) and the USACE (1990b) for this type of hydraulic design

9.7.2.2.1 Open-Channel Flow

Figure 9.27 illustrates several types of open-channel cross sections. The flow rate Q for each channel is the product of the average flow velocity V and flow area A:

$$Q = VA \tag{9-16}$$

where:

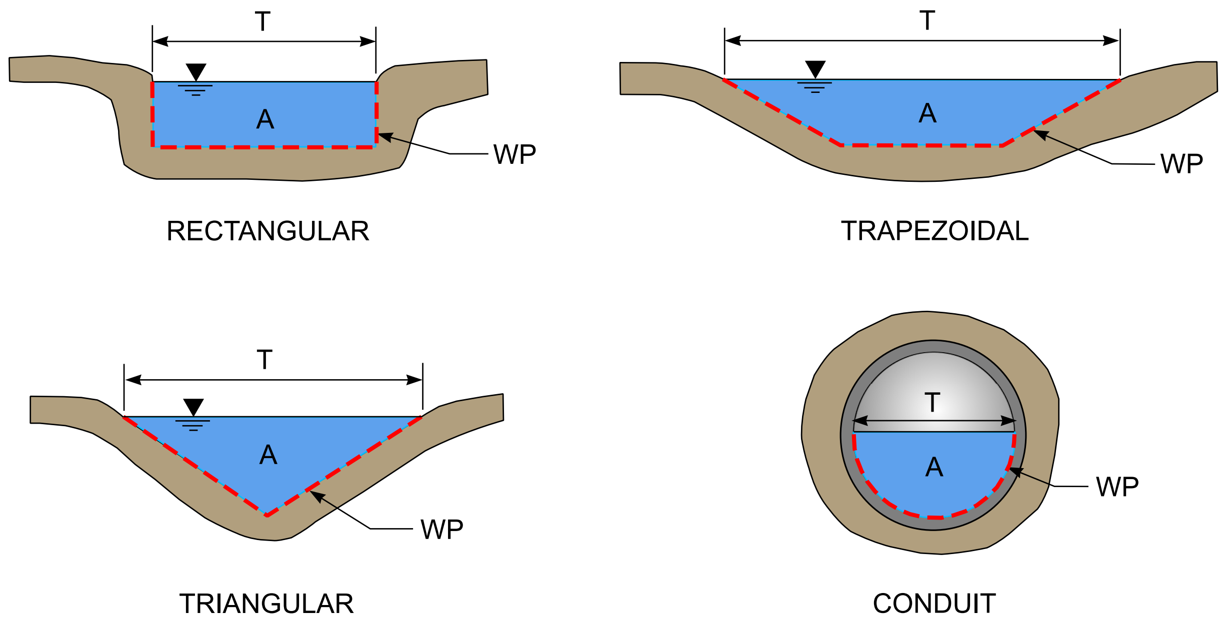
Q = flow rate (cfs)

V = average velocity of flow (ft/sec)

A = cross-sectional area of flow (ft^2)

The flow area is a function of the flow depth which, along with the required freeboard, establishes the required height of the channel. Velocity is important because it is directly related to the potential for erosion, cavitation and energy dissipation. Normally, the required flow area is determined first, because the channel geometry is often controlled by the refuse disposal facility and site configurations and the desire to have uniform channel cross sections to simplify construction.

Methods for the analysis of open-channel flow and procedures for calculating depth of flow at any point along a channel are available in many texts, including Brater et al. (1996), Henderson (1969), and Chow (1959). The FHWA (1961) presents tables and charts for determination of open-channel flow parameters in prismatic channels. Computer software for determining steady-state and transient flow conditions in open channels is also readily available. Accordingly, the following discussion of design and analysis considerations relates to flow regulation and the identification of specific conditions or features encountered at coal refuse disposal facilities.



$$\text{HYDRAULIC RADIUS (R)} = \frac{\text{AREA (A)}}{\text{WETTED PERIMETER (WP)}}$$

$$\text{HYDRAULIC DEPTH (D)} = \frac{\text{AREA (A)}}{\text{TOPWIDTH (T)}}$$

FIGURE 9.27 OPEN-CHANNEL CROSS SECTIONS

Four possible flow (and depth) conditions must be considered in the design of an open channel. These are: (1) critical depth, (2) depth at subcritical flow, (3) depth at supercritical flow, and (4) normal depth. The point at which open-channel flow passes through the critical depth is a regulating condition of flow. Hereafter, the point where this occurs is referred to as a control point or location regulating the flow rate. The depth of flow at a control point may also be referred to as the control depth. Several examples illustrating these basic flow conditions are presented in Figure 9.28. Additional explanation of these conditions is provided in the following:

1. Critical depth – The critical depth of flow in an open channel is the depth at which flow occurs with the minimum specific energy, defined as the minimum depth y plus the velocity head ($V^2/2g$) for the channel. At the critical state of flow, the velocity head is equal to half the hydraulic depth D, as defined in Figure 9.27. Chow (1959) and FHWA (1961) provide detailed discussion of the critical depth and minimum specific energy. For design, the critical depth is normally calculated for comparison to the actual depth to determine if the actual flow condition is subcritical or supercritical. Figure 9.29 presents curves for determining the critical depth for trapezoidal and circular channels. Except at flow control points, design at or close to critical depth should be avoided in order to prevent the occurrence of unstable flow regimes.

2. Depth at subcritical flow – For subcritical flow the channel slope is milder than that associated with critical flow, and the flow velocity is relatively low. Channel slopes with subcritical flow are called subcritical slopes. For subcritical flow, the control point will be downstream.

3. Depth at supercritical flow – For supercritical flow the channel slope is steeper than that associated with critical flow and the flow velocity is relatively high. Channel slopes with supercritical flow are called supercritical slopes. For supercritical flow, the control point will be upstream. If it is important that water be carried away from the inlet section without a possible back up, it may be desirable to design for a supercritical flow condition for a portion or all of the length of the channel. However, this may result in high flow velocities, and special attention will have to be given to factors such as erosion, flow at direction changes and energy dissipation at the discharge point. Most coal refuse disposal facilities have a large elevation difference between the embankment crest and downstream toe, and this generally results in some section of the channel between these two points having a supercritical flow condition.
4. Normal depth – Normal flow depth occurs when the energy increase during elevation drop is exactly balanced by friction losses along a channel. When this occurs, the flow depth and velocity in a channel of constant cross section and slope will remain constant. Open-channel flow naturally tends toward the normal depth condition, but for short channel lengths it often does not reach the normal condition. Because normal depth is easily determined, it is the flow condition most often used during preliminary hydraulic analyses to establish whether or not the selected channel slope is feasible (as limited by geotechnical or structural conditions).

The Manning equations are used to estimate the velocity and depth of flow for a given flow rate and channel configuration. These equations are:

$$Q = \frac{1.49}{n} A R^{0.67} S^{0.5} \tag{9-17}$$

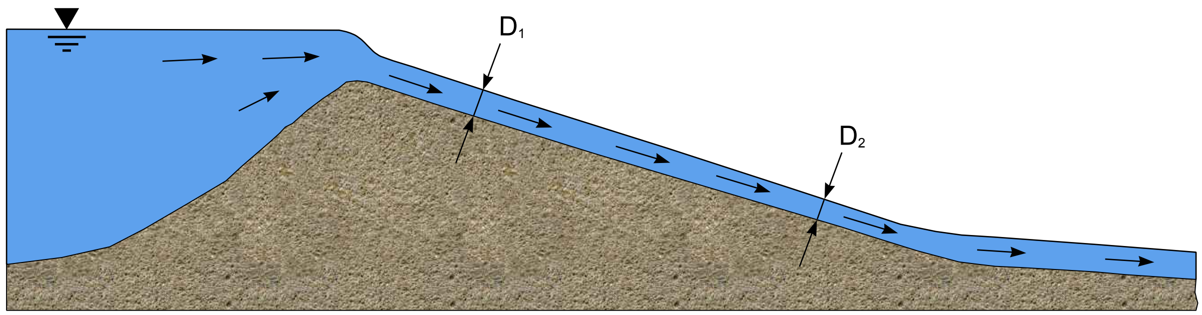
$$V = \frac{1.49}{n} R^{0.67} S^{0.5} \tag{9-18}$$

where:

Q	=	flow rate (cfs)
V	=	average velocity of flow (ft/sec)
A	=	cross-sectional area of flow (ft^2)
R	=	hydraulic radius (ft)
S	=	slope of the channel (ft/ft)
n	=	Manning's coefficient of channel roughness

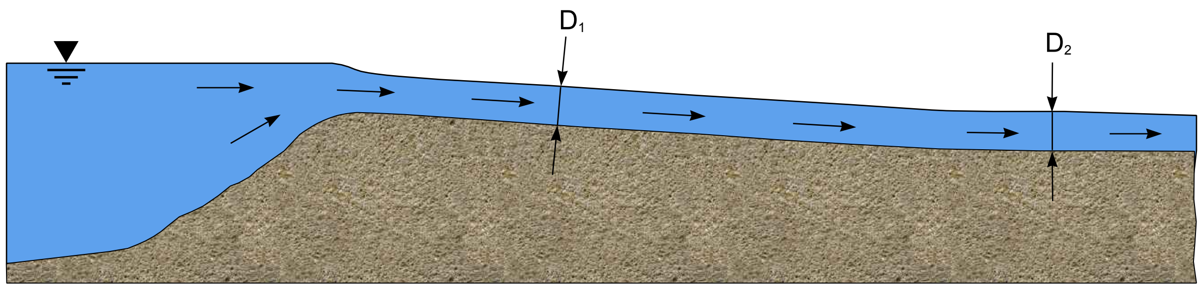
The depth determined from the Manning formula is normal depth. The term R is the hydraulic radius of the flow and is equal to the area of flow A divided by the wetted perimeter WP. The wetted perimeter for several types of cross sections is shown in Figure 9.27. Values of Manning's coefficient of channel roughness n are listed in Table 9.13. An extensive presentation of n values for a wide variety of channels is provided in Chow (1959) and FHWA (1961). Depending upon the effective roughness of the surface, n can range from as high as 0.2 for channels with dense brush growth and many obstructions to as low as 0.01 for smooth-finish, concrete-lined channels.

Since the cross-sectional area A and hydraulic radius R are both functions of flow depth, an iterative procedure is required to determine a normal depth that satisfies the Manning equations. Commercially available software can be used to determine flow depth, velocity, critical depth and slope, and various open-channel design parameters (e.g., HEC-RAS and privately marketed similar programs).



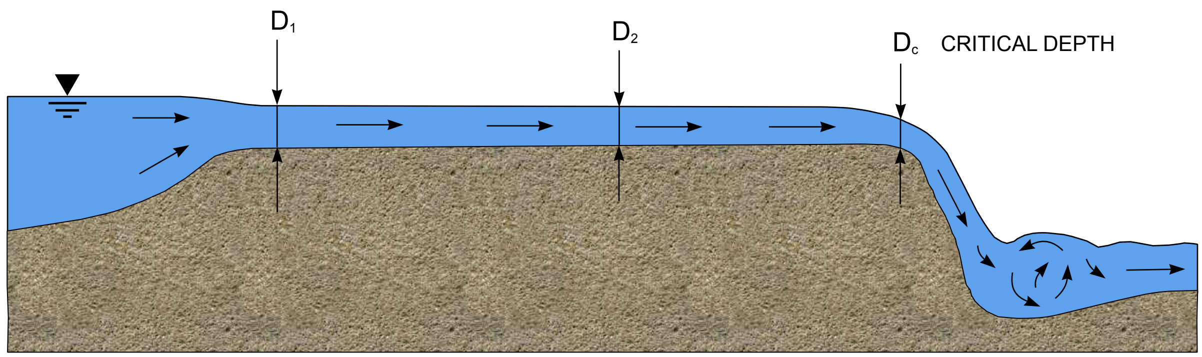
9.28a STEEP SLOPE (SUPERCRITICAL FLOW)

1. D_1 IS CONTROLLED BY INLET CONDITIONS
2. D_2 IS CONTROLLED BY D_1
3. D_2 APPROACHES NORMAL DEPTH



9.28b MILD SLOPE (SUBCRITICAL FLOW)

1. D_2 IS CONTROLLED BY DOWNSTREAM CONDITIONS
2. D_1 IS CONTROLLED BY D_2
3. D_1 APPROACHES NORMAL DEPTH



9.28c MILD SLOPE WITH STEEP DROP

1. CONTROL IS AT THE POINT WHERE CRITICAL DEPTH (D_c) OCCURS
2. D_c IS A FUNCTION OF CHANNEL SHAPE AND FLOW (Q)
3. D_2 AND D_1 ARE CONTROLLED BY D_c

FIGURE 9.28 CHANNEL FLOW CONDITIONS

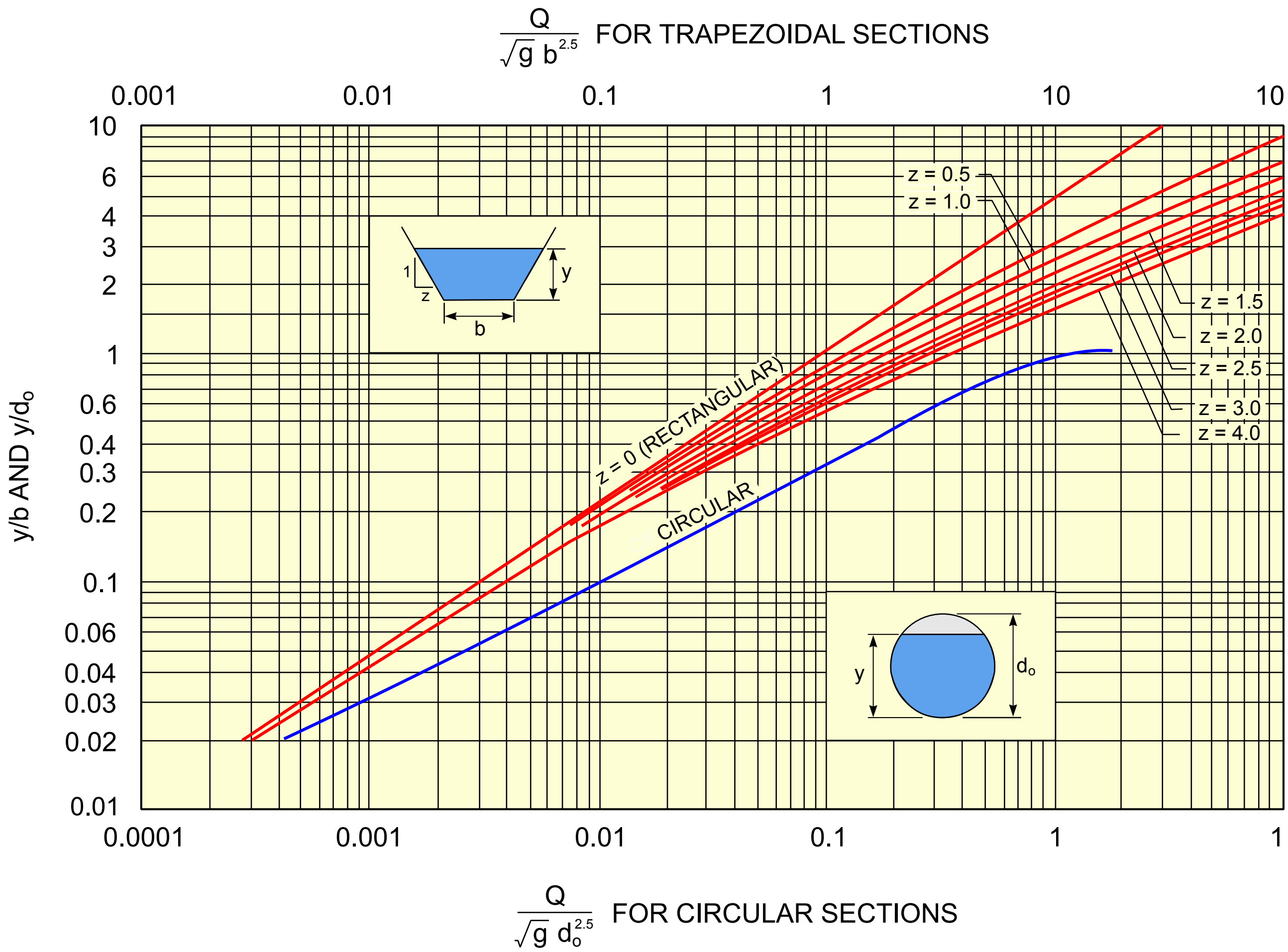


FIGURE 9.29 CRITICAL DEPTH FOR TRAPEZOIDAL AND CIRCULAR SECTIONS

Many commercially manufactured erosion control products have been developed to resist potential erosion. These products include vegetation control mats, interlocking concrete blocks, concrete mat systems, cellular confinement mats, etc. The Manning's *n* value for these erosion control products is typically provided by the manufacturer.

An experienced designer can determine if the normal depth of flow from the Manning equations is sufficient for determining flow conditions for final design. Detailed analyses of water surface profiles may be necessary, particularly when depths associated with changes in channel configuration, erosion protection and slope may be required. In these situations, the change in water surface elevation (water surface profile) due to velocity head and channel losses should be determined. The normal depth does not adequately describe conditions at the ends or transitional zones in the transport section. For example, where a channel with supercritical slope changes to a short length of channel with subcritical slope (e.g., benches on an embankment face), the designer must determine if the depth in the subcritical section will create a "hydraulic jump," and calculation of the flow profile along the length of the open channel often must be made. Another instance where flow profiles must be determined is when establishing a spillway rating curve (stage-discharge curve) for an approach channel between the impoundment and an open-channel spillway weir (control section) or when subcritical flow is present downstream of or within the spillway channel. Procedures for analyzing such channel sections are presented in many references on open-channel flow, including Brater et al. (1996), Henderson (1969) and Chow (1959). The FHWA (2006), USBR (1987a) and the USDA (1956) present methods and charts for determining the depths and other parameters associated with hydraulic jumps in open channels. If a hydraulic jump occurs, the channel depth must be sufficient to contain the sequent flow depth that will develop.

TABLE 9.13 MANNING'S COEFFICIENTS OF CHANNEL ROUGHNESS

Constructed Channel Condition	Values of n		
	Minimum	Maximum	Average
Earth channels, straight and uniform	0.017	0.025	0.022
Dredged earth channels	0.025	0.033	0.028
Rock channels, straight and uniform	0.025	0.035	0.033
Rock channels, jagged and irregular	0.035	0.045	0.045
Concrete lined, regular finish	0.012	0.018	0.014
Neat cement lined	0.010	0.013	0.011
Grouted rubble paving	0.017	0.030	0.025
Corrugated metal	0.023	0.025	0.024

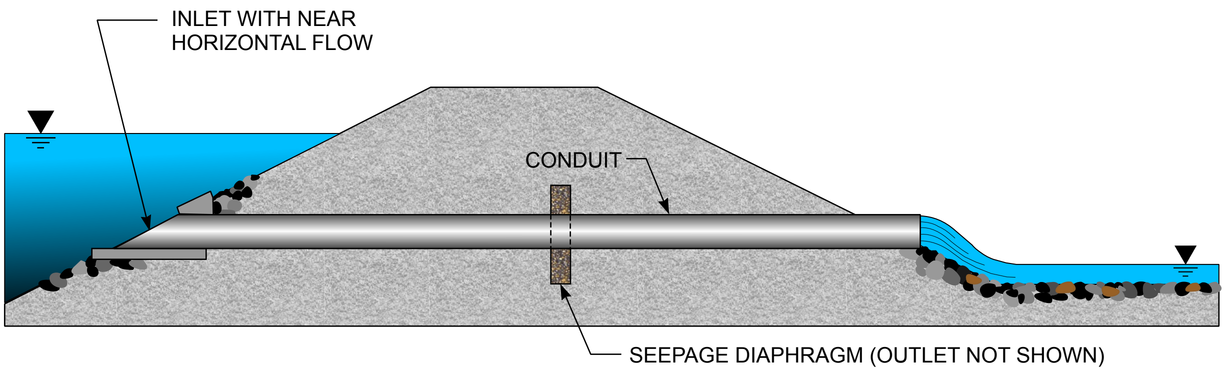
Natural Channel Condition	Value of n
Smoothest natural earth channels, free from growth with straight alignment	0.017
Smooth natural earth channels, free from growth, little curvature	0.020
Average, well-constructed, moderate-sized earth channels	0.0225
Small earth channels in good condition or large earth channels with some growth on banks or scattered cobbles in bed	0.025
Earth channels with considerable growth, natural streams with good alignment and fairly constant section, or large floodway channels well maintained	0.030
Earth channels considerably covered with small growth or cleared but not continuously maintained floodways	0.035
Mountain streams in clean loose cobbles, rivers with variable cross section and some vegetation growing in banks, or earth channels with thick aquatic growths	0.050
Rivers with fairly straight alignment and cross section, badly obstructed by small trees, very little underbrush or aquatic growth	0.075
Rivers with irregular alignment and cross section, moderately obstructed by small trees and underbrush	0.100
Rivers with fairly regular alignment and cross section, heavily obstructed by small trees and underbrush	0.100
Rivers with irregular alignment and cross section, covered with growth of virgin timber and occasional dense patches of bushes and small trees, some logs and dead fallen trees	0.125
Rivers with very irregular alignment and cross section, many roots, trees, large logs, and other drift on bottom, trees continually falling into channel due to bank caving	0.200

(USBR, 1987a)

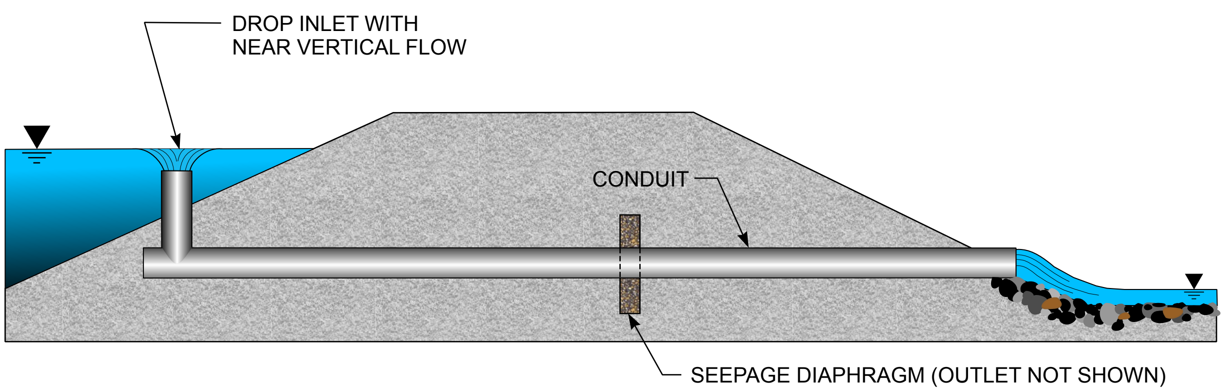
9.7.2.2.2 Pressure Flow

Closed conduits are used as the transport section for two types of hydraulic systems. When the flow enters the conduit with a nearly horizontal approach, the system is called a culvert. When the flow enters the conduit through a steeply inclined or vertical drop inlet, the system is called a decant. These two types of hydraulic systems are illustrated in Figure 9.30.

When flowing full, both types of closed-conduit systems behave in essentially the same manner, and their capacity can be determined from pipe flow analyses. However, the systems differ significantly in behavior during conditions leading up to the full-flow condition. The following paragraphs discuss the hydraulics of culvert systems in detail. The hydraulic behavior of decant systems is then presented with particular emphasis on the differences between decant and culvert systems.



9.30a CULVERT SYSTEM



9.30b DECANT SYSTEM

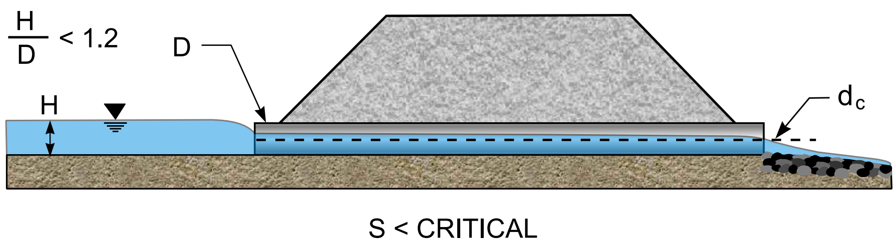
FIGURE 9.30 CLOSED-CONDUIT HYDRAULIC SYSTEMS

Culvert Conduits

A culvert is a pipe placed beneath an embankment with an entrance that is horizontal or slightly inclined to the direction of flow. A culvert can be prefabricated or cast-in-place and the cross section can be round, square, rectangular, oval or arched. Factors that control the flow in a culvert are slope, size, shape, length and roughness (material) and inlet and outlet configuration. The combined effects of these factors determine the hydraulic control location, which in turn determines whether the culvert flows partly or completely full and establishes the head-discharge relationship. As with open-channel flow, the slope of the culvert may be mild or steep, resulting in subcritical or supercritical flow conditions. For either slope condition, the control point may be either at the inlet or the outlet, depending on the system geometry and the upstream and downstream head conditions. Figures 9.31 and 9.32 illustrate the various factors that affect flow in culverts.

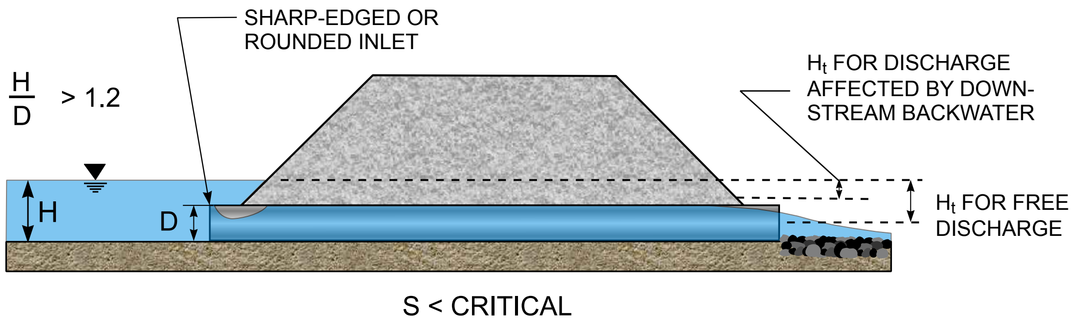
The control point for a culvert on a mild slope flowing partly full will usually be at the outlet if the inlet is not submerged. The flow ordinarily will be subcritical, and the discharge may be predicted according to the open-channel flow procedures discussed above. If the outlet discharges freely, the flow at that point will pass through critical depth (Figure 9.31a). As the inlet becomes submerged, the control point moves downstream within the culvert. The flow could be supercritical just inside the culvert if limited submergence is sustained ($H/D \sim 1.2$). As submergence of the entrance increases ($H/D > 1.2$, Figure 9.31b), or the culvert is sufficiently long, or the elevation of the downstream backwater is sufficiently high, full pipe flow occurs with control at the downstream end of the conduit.

PART FULL FLOW - INLET NOT SUBMERGED



9.31a MILD SLOPE - SUBCRITICAL FLOW: CONTROL AT CRITICAL DEPTH AT OUTLET

FULL FLOW - INLET SUBMERGED



9.31b MILD SLOPE - CONTROL AT OUTLET: EFFECTIVE HEAD = H_t - Σ LOSSES

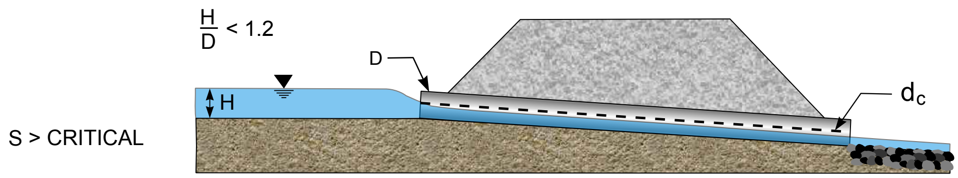
FIGURE 9.31 TYPICAL FLOW CONDITIONS – CULVERT CONDUITS ON MILD SLOPES

Most coal refuse disposal facility decant systems are less that 36 inches in diameter with a length of a few hundred to more than 1,000 feet and are thus sufficiently long (Chow, 1959) that there is full pipe flow in the conduit and free discharge from the outlet. If the elevation of the downstream backwater is above the critical depth elevation, the depth associated with the backwater may control the flow in the culvert. One of these conditions typically occurs, such that culverts flow full for their entire length (Figure 9.31b).

When a culvert is on a steep slope and the inlet is not submerged, the flow is controlled by critical depth at the inlet (Figure 9.32a), and open-channel flow at supercritical velocity will occur throughout the culvert. After the inlet has been submerged (*H* exceeds about 1.2*D*), it is still possible to have open-channel flow at supercritical velocities in the culvert if the control point remains at the inlet (Figure 9.32b). In this case, discharge is governed by orifice flow at the inlet, leading to formation of a flow contraction at the top of the culvert entrance and aeration over the remaining culvert length.

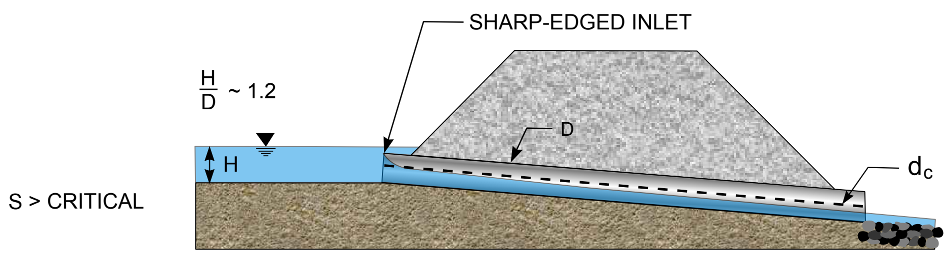
As the head at a submerged inlet increases, friction or local disturbances may reduce the flow velocity, causing the culvert to flow full near the outlet. This may seal the downstream end of the pipe, even though an orifice flow contraction tends to occur at the inlet. The associated high-velocity flow will tend to carry away the air trapped at the top of the culvert, thus reducing the internal pressure to less than atmospheric pressure. This can lead to damage to the culvert from cavitation (Section 9.7.3.3). However, if the entrance is shaped to eliminate the inlet flow contraction, the culvert will start to flow full near the inlet, after which the full flow zone will extend rapidly toward the outlet. The effect of the full flow condition will be a draft tube action (similar to siphonic action) that will

PART FULL FLOW-INLET NOT SUBMERGED



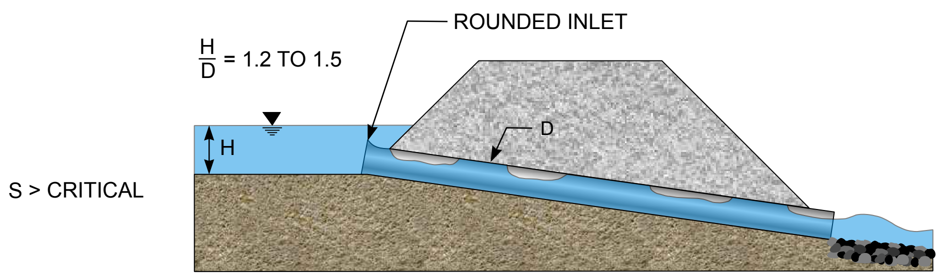
9.32a STEEP SLOPE - SUPERCRITICAL FLOW: CONTROL AT CRITICAL DEPTH AT INLET

PART FULL FLOW-INLET SUBMERGED

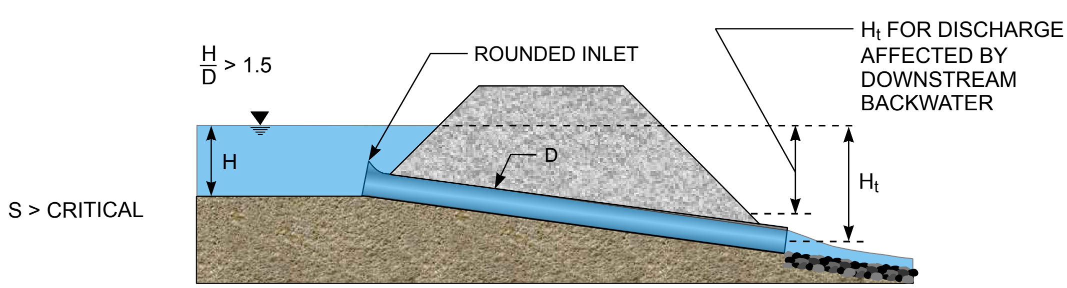


9.32b STEEP SLOPE - SUPERCRITICAL FLOW: ORIFICE FLOW CONTROL AT INLET

FULL FLOW-INLET SUBMERGED



9.32c STEEP SLOPE - SUPERCRITICAL PULSATING SLUG FLOW: CONTROL SWITCHING BETWEEN INLET AND SECTION WITHIN THE CONDUIT



9.32d STEEP SLOPE CONTROL AT OUTLET EFFECTIVE HEAD = $H_t - \Sigma$ LOSSES

FIGURE 9.32 TYPICAL FLOW CONDITIONS – CULVERT CONDUITS ON STEEP SLOPES increase the discharge. The increased discharge will cause a deep drawdown just upstream from the inlet, and a vortex will form, allowing air to enter the culvert, breaking the draft tube action. This immediately reduces the discharge and causes a return to orifice control at the inlet. The full flow action will begin again and the cycle will be repeated. This alternating action will cause a pulsating slug flow phenomenon (Figure 9.32c). When the storage elevation and culvert dimensions are such that the H/D ratio exceeds about 1.5, the inlet drawdown will be insufficient to allow air to enter the culvert and steady full flow will prevail (Figure 9.32d).

The flow velocity during the full-flow conditions illustrated by Figures 9.31b and 9.32d can be determined from the Bernoulli equation:

$$\frac{V^2}{2g} = H_t - \sum \text{losses} \qquad \text{(9-19)}$$

where:

V = flow velocity (ft/sec)

g = acceleration of gravity (ft/sec^2)

H_t = head (elevation difference) between the impoundment surface and the point of discharge (ft)

The term "$\sum$ losses" encompasses all flow-reducing conditions associated with the inlet, culvert geometry and culvert friction. These losses can be related to the velocity by:

$$\sum \text{losses} = \frac{V^2}{2g}\left(f\frac{L}{D} + \sum K_L\right) \qquad \text{(9-20)}$$

where:

f = friction factor for the culvert material and flow condition in the culvert

L = length of the culvert (ft)

D = diameter of the culvert (ft)

$\sum K_L$ = sum of head losses associated with the inlet, valves, constrictions and directional changes (ft)

For most flow conditions and circular culvert materials, the friction factor (f) can be determined from the following equation:

$$f = 185\, n^2/d^{\,1/3} \qquad \text{(9-21)}$$

where:

n = Manning's roughness coefficient for culvert

d = pipe diameter (ft)

By substituting terms from Equation 9-20 into Equation 9-19, the relationship between H_t and V becomes:

$$H_t = \frac{V^2}{2g}\left(f\frac{L}{D} + \sum K_L + 1\right) \qquad \text{(9-22)}$$

Typical head loss coefficients K_L are tabulated in Table 9.14. After all of the loss parameters have been determined, Equation 9-22 can easily be solved.

However, Equation 9-22 is only appropriate when dealing with water at normal temperatures. The friction factor approach is superior when working with smooth pipes and/or large values of Reynolds number. For materials and pipe sizes commonly used, the above equation will give acceptable results. When unusual surfaces or very large pipe sizes are involved or when very long conduits with large energy losses are used, an experienced hydraulic engineer should be consulted.

The geometry of an inlet is important to achieving discharge efficiency. Until the headwater surface is well above the culvert inlet, a square-edged inlet produces an inlet flow contraction without greatly reducing the discharge capacity. Flow contractions can also be formed (but at reduced discharge capacity) by a projecting inlet, a mitered inlet, an inlet orifice ring, or a top curtain wall closure. These inlet configurations are shown in Figure 9.33.

TABLE 9.14 HEAD LOSS COEFFICIENTS FOR CONDUITS

K_L for Inlets (USBR, 1987a)

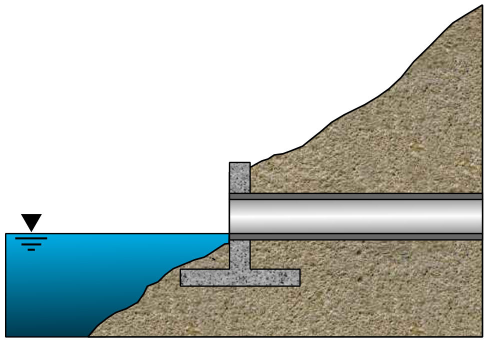
Inlet Type	K_L
Fully rounded entrances ($r / D \geq 0.15$) flush with vertical walls	0.10
Square edge entrances flush with vertical walls	0.50
Socket-ended concrete pipe flush with vertical walls	0.15
Projecting concrete pipe with socket ends	0.20
Projecting smooth-wall or corrugated pipe	0.85
Gate in thin wall – unsuppressed contraction	1.50
Gate in thin wall – corners rounded	0.50

K_L for Directional Changes (Linsley and Franzini, 1972)

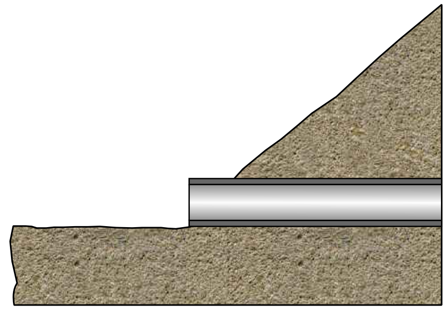
Radius of Bend/Pipe Diameter		Angle of Bend		
		90°	45°	22.5°
1	K_L	0.50	0.37	0.25
2	K_L	0.30	0.22	0.15
4	K_L	0.25	0.19	0.12
6	K_L	0.15	0.11	0.08
8	K_L	0.15	0.11	0.08

K_L for Valves and Fittings (Linsley and Franzini, 1972)

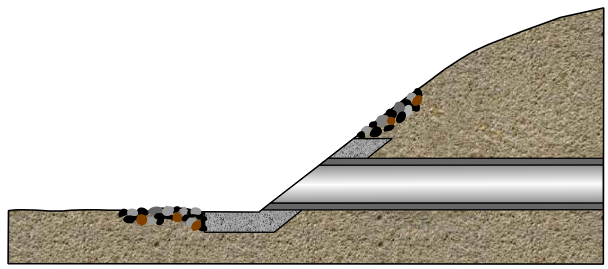
Type of Valve or Fitting	K_L
Butterfly valve (wide open)	0.2
Gate valve (wide open)	0.2
Gate valve (half open)	5.6
Return bend	2.2
Standard tee	1.8
Standard 90° elbow	0.9



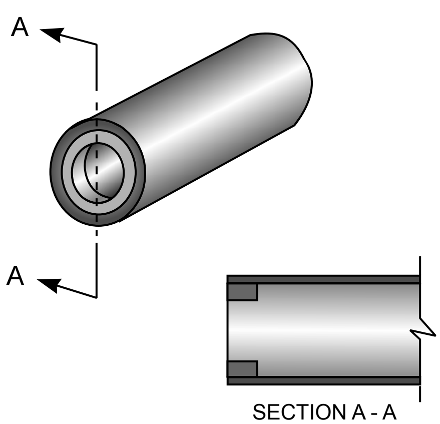
9.33a SQUARE-EDGED INLET



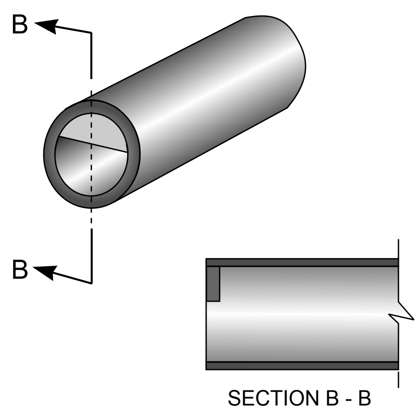
9.33b PROJECTING INLET



9.33c MITERED INLET



9.33d INLET ORIFICE RING



9.33e TOP CURTAIN WALL INLET

FIGURE 9.33 CULVERT INLET CONFIGURATIONS

If it is desired that a culvert flow full at increased headwater elevations, the control point during maximum flow will be at the outlet and the geometry of the inlet will be less significant. For this case, the inlet should be shaped so as to minimize the inlet flow contraction, thereby increasing the tendency for full flow for all conditions except when the inlet is not submerged. Streamlining the shape of the inlet will streamline the flow and reduce inlet head losses. The tendency to develop cavitation pressures will also be reduced. Rounded entrances or a gradually tapering transition in size to the basic culvert dimension will generally provide the desired streamlining.

It is obvious from the previous discussions and related tables and figures that culvert inlets may vary with respect to approach conditions, entrance arrangements, and cross-sectional shapes. For example: (1) the approach to the inlet may or may not be a well defined channel with or without constructed wing walls, (2) the culvert may be flush with or protrude past the upstream embankment surface or constructed headwall, (3) the face may be square, beveled or rounded, and (4) the cross section may be round, square, rectangular or arched. All such variations have a marked influence on culvert performance as they affect weir or orifice discharge, inlet flow contractions and head losses during flow entrance to the culvert.

For coal refuse disposal facilities, circular culverts with vertical square-edged headwalls and flared wingwalls are sometimes encountered. The hydraulic design for these installations is discussed in detail by the USBR (1987a). Also, procedures and charts for design of these types of culverts are presented in FHWA (2005b).

The purpose of hydraulic analysis of a closed-conduit system is similar to that for an open-channel spillway, which is to determine the relationship between the elevation of the impounded storage (headwater) and the rate of outflow or discharge from the system. The head-discharge relationship for culvert-type conduit systems may be controlled by weir, orifice and pressure flow conditions depending on the upstream head, as illustrated in Figures 9.31 and 9.32.

Drop Inlet Conditions

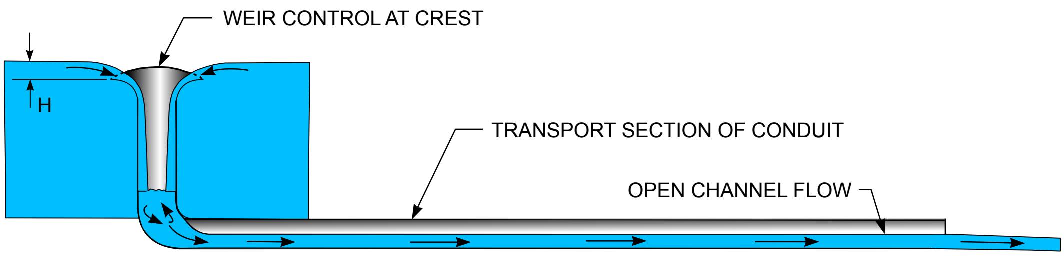
Closed-conduit flow in drop-inlet decant systems is similar to that for culvert systems, as discussed above. The relationship between the elevation of the impounded storage and the rate of discharge during these types of flow is indicated by the curves shown in Figure 9.34. For the drop-inlet systems illustrated in Figure 9.34, curve A results from crest-controlled (also referred to as weir-controlled) flow, as affected by the inlet geometry and water level conditions. The inlet geometry can range from a sharp-edged, vertically placed circular pipe to an elaborately formed concrete ogee called a morning glory spillway.

Curve B results from orifice-controlled flow. The flow control point for curve A is at the crest, as illustrated in Figure 9.34a. The flow control point for curve B is at the entrance to the transport section of the conduit, as illustrated in Figure 9.34b. The curve B flow condition occurs when the open-channel capacity of the transport section and the discharge capacity of the outlet are both greater than the capacity of the inlet orifice. This results in a backup of water in the drop tube and drowning of the inlet weir.

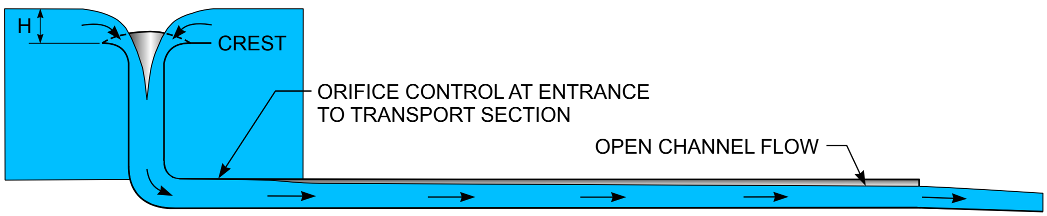
The curve C flow condition is associated with full flow, generally controlled by the transport section. Analysis for the curve C condition is the same as for a culvert system under full flow (Equation 9-22) except that an additional head loss term related to the drop inlet should be included. The loss coefficient K_L for the transition from the drop inlet to the transport section can range from 0.2 for an unobstructed, specially-formed structure to 2.0 for a debris-clogged, square-edged turn. A value of 1.0 is normally satisfactory for design.

To maximize the discharge capacity of decant systems under low heads, conical-shaped inlets that extend the crest control flow (curve A) have been employed. Other inlet shapes that incorporate an orifice ring have been used to maintain orifice flow (curve B) and to prevent the transport pipe section from flowing full or under pressure.

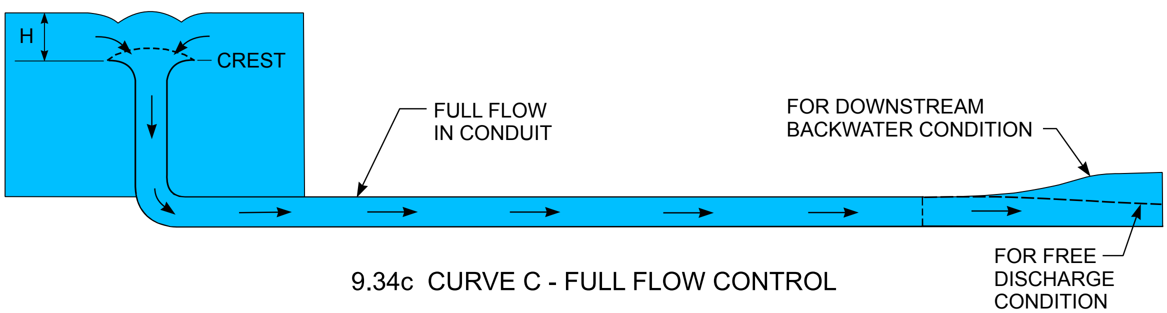
As a guide for assisting the designer in the evaluation of important secondary characteristics affecting closed-conduit flow, a list of Manual sections and supplemental references for secondary hydraulic design issues is provided in Table 9.15.



9.34a CURVE A - CREST CONTROL



9.34b CURVE B - ORIFICE CONTROL



9.34c CURVE C - FULL FLOW CONTROL

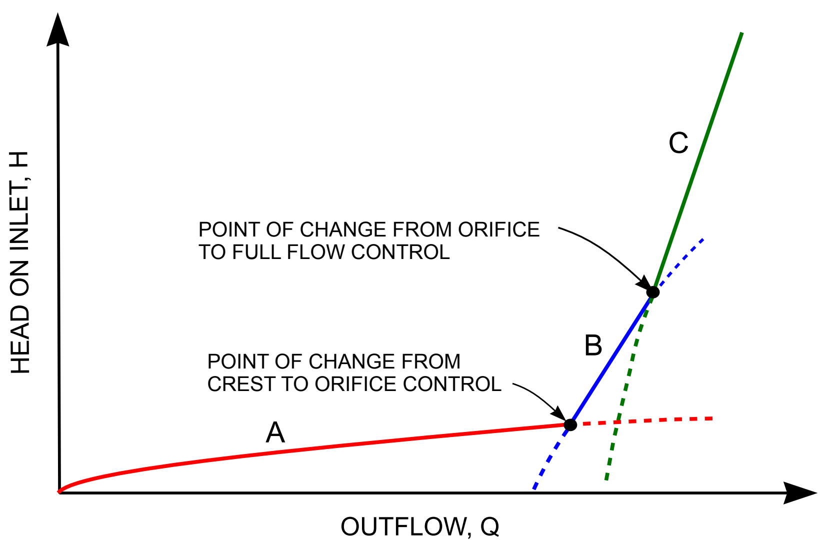


FIGURE 9.34 FLOW AND DISCHARGE CHARACTERISTICS OF DECANT-INLET, CLOSED-CONDUIT SYSTEM

TABLE 9.15 REFERENCES FOR SECONDARY HYDRAULIC DESIGN ISSUES

Item	Manual Section	Supplemental References
External Embankment Pressure	6.6.6.2	FEMA (2005a)
Internal Water Pressure	9.7.2.2	Brater et al. (1996)
Thrust Forces	9.7.3.5	Brater et al. (1996)
Trash Racks	9.7.4.1	USBR (1987a)
Vortex Control	9.7.4.1	USBR (1987a)
Flow at Bends	9.7.3.4	Brater et al. (1996), USACE (1980)
Cavitation	9.7.3.3	USACE (1980), USBR (1990)
Materials Selection	6.6.6.1	FEMA (2005a)

9.7.2.3 Outlets

There are two basic requirements that typically dictate the size, configuration and location of the outlet section of the hydraulic system:

1. If the outlet is intended to control the relationship between headwater storage and outflow for the entire hydraulic system, the outlet size and configuration must be designed specifically for this purpose.
2. The configuration and location of the outlet must be such that discharged water will not create a hazardous condition (e.g., erosion of the channel or embankment toe) and the energy of the outflow will be dissipated. The potential for localized damage must be controlled to a level acceptable to the owner and to regulatory agencies.

Whether the first requirement applies depends upon the design choices for the inlet and transport sections of the system, as previously discussed. If applicable, the outlet size and configuration must be designed to control the required discharge, generally by weir or orifice control. Evaluation of this discharge was discussed in Section 9.7.2.1 and is not discussed further herein.

The second requirement has more general significance and necessarily requires considerable planning at the earliest stages of design so that the outlet of the hydraulic system will not be incompatible with any other portion of the coal refuse disposal facility. Criteria to be considered in determining the degree of effort required for construction of an outlet system normally include:

- Under no circumstance should the outlet location or configuration allow flow to discharge beyond the outlet in a manner that could contribute to embankment failure, endangering lives or major downstream property.
- Situations where significant localized damage could occur during moderate storms should be avoided. Exceptions are when the cost of constructing a damage-safe outlet far exceeds the cost of occasional maintenance. Often the 100-year storm (Section 9.5) is considered as the basis for design when it is unlikely that lives or major downstream property could be endangered.
- For very large storms, such as those approaching the PMF (Section 9.5), it is generally not required that the outlet totally prevent downstream damage due to discharge, if the first criterion above is satisfied. Under such severe storms, significant downstream damage (such as erosion and roadway overtopping) can be expected to occur even if the hydraulic system of the disposal facility satisfies all requirements of good design.

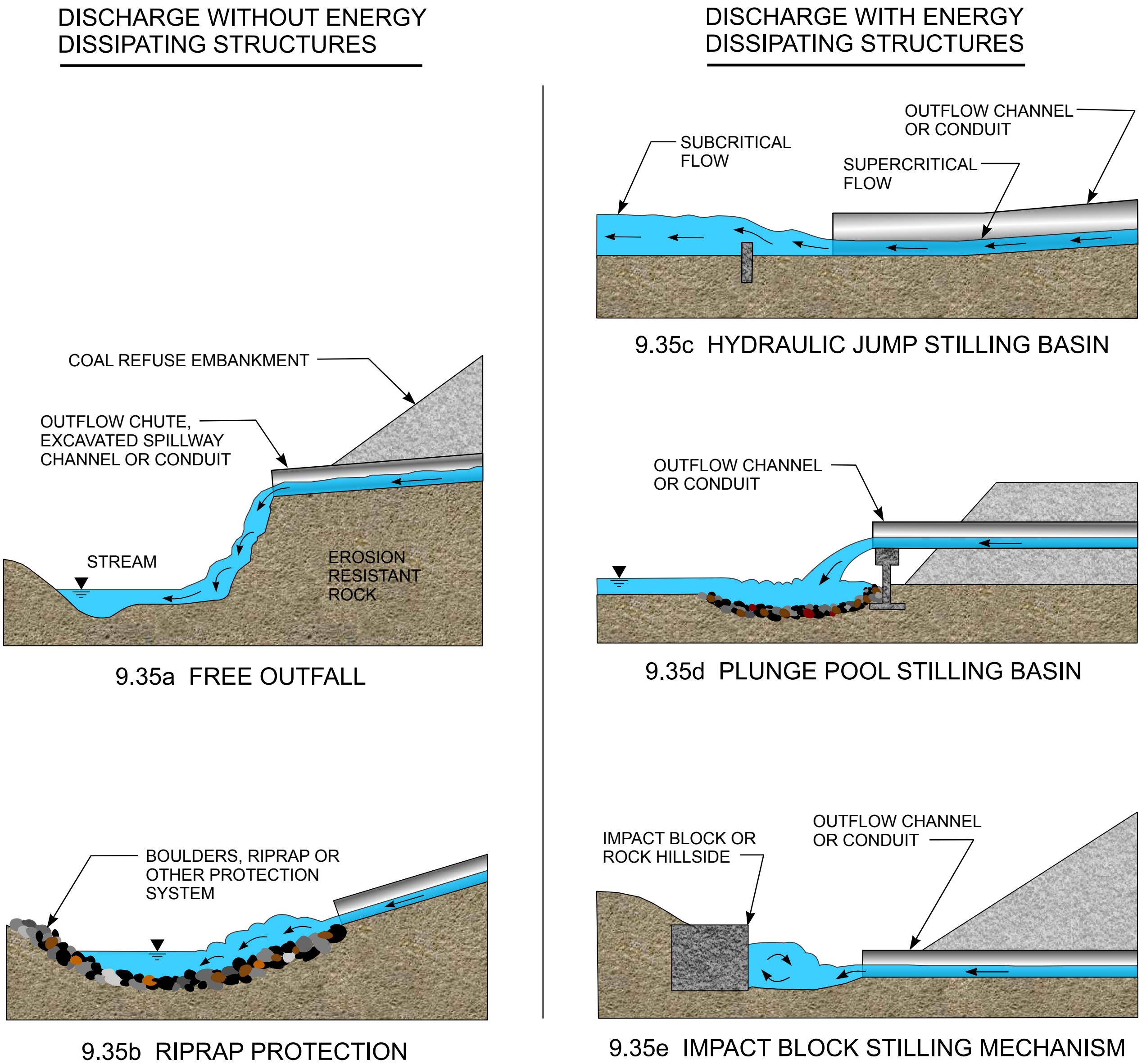


FIGURE 9.35 EXAMPLES OF CHANNEL AND CONDUIT OUTLETS

Several types of outlet conditions are illustrated in Figure 9.35. If the discharge point can be located an adequate distance away from the overall facility, the most reliable and least expensive outlets are those that allow the flow to discharge over erosion resistant rock or large riprap. This type of outlet condition is most often used when the channel or spillway will function only infrequently because of the limited recurrence of very large storms.

The energy dissipating structures shown in Figure 9.35 are commonly used for water-impounding earth dams, but are normally not necessary for coal refuse disposal facilities if sufficient access is available that periodic maintenance and repair can be performed. However, such outlets should be considered for moderate but regular discharges or when future access could be difficult. Also, if extending the transport section a significant distance beyond the downstream limits of the facility will be costly, coupling a short transport section with an energy-dissipating structure may be a more cost-effective solution.

State and local agencies may prefer specific methods or structures for energy dissipation at outlet structures, and applicable regulatory guidance should be reviewed as part of design. Hydraulic design requirements for structures similar to those shown in Figure 9.35 are discussed by FHWA (2006), USACE (1990b), USACE (1980), and USBR (1987a).

9.7.3 Special Design Considerations

The following discussion briefly emphasizes some important special considerations of hydraulic system design and illustrates situations where special conditions must be evaluated.

9.7.3.1 Channel Freeboard

Freeboard is needed in open channels to account for minor channel irregularities, air entrainment and wave action. The required freeboard is a function of the design flow velocity and flow depth and typically varies from less than 5 percent to greater than 30 percent of the depth of flow (Chow, 1959). Low freeboards may be appropriate for smooth, uniform channels with flow velocities less than 8 feet per second. The following is an empirical relationship that provides the desirable spillway channel freeboard based upon surface roughness, wave action, air bulking, splash, and spray under supercritical conditions (USBR, 1987a):

$$\text{Freeboard (ft)} = C_f + 0.025\, V\, D^{1/3} \qquad (9\text{-}23)$$

where:

C_f = freeboard coefficient

V = design flow velocity (ft/sec)

D = flow depth (ft)

The USBR (1987a) has indicated that a value of C_f equal to 2 is desirable for spillway channels for dams. While Equation 9-23 has been applied to spillways with the cited flow conditions, this formula has been used by some state agencies for the design of uniform channels ranging from perimeter and diversion ditches to small spillway channels by adopting values for C_f ranging from 0 (with not less than 0.3 feet of freeboard) to 1. MSHA (2007) has indicated that a value of C_f equal to 1 is prudent for design of perimeter ditches. Channel bends, convergence, and other geometric changes may require greater freeboard to address variations in the flow profile. Additional discussion is presented in Section 9.7.3.4.

9.7.3.2 Erosion Protection

Two methods are typically employed in the design of erosion protection linings for channels: (1) the permissible velocity method or (2) the tractive (shear stress) force method. Under the permissible velocity approach, the channel is assumed to be stable if the mean velocity is lower than the maximum permissible velocity for the channel materials. The tractive force approach focuses on stresses at the channel boundary. The factors that normally cause erosion are velocity of flow (e.g., in steep channels and at intakes) and water impact at points where direction changes occur (e.g., at sharp bends in channels, at the intersection of channels, and where free-falling water discharges from a channel or conduit). If unchecked, erosion can result in degradation of water quality and frequent maintenance. Excessive or uncontrolled erosion on or near an impounding embankment can result in catastrophic failure during large storms.

The shear resistance of the soil/rock in an unlined channel or the shear resistance provided by the channel lining will determine the stability of the channel relative to erosion. The shear force generated by water flowing through a channel section is given by:

$$\tau = \gamma\, R\, S \qquad (9\text{-}24)$$

where:

τ = shear stress along the wetted perimeter of flow (lb/ft^2)
γ = unit weight of water (lb/ft^3)
R = hydraulic radius = A/WP (ft)
S = energy grade line slope (equal to the channel slope for uniform flow) (ft/ft)

The maximum shear stress on the channel bottom and sides in a straight channel depends on the channel shape. For trapezoidal channels with a ratio of bottom width to depth greater than 4, the maximum shear stress on the channel bottom frequently governs the lining selection and can be estimated using the relationship (FHWA, 2005a):

$$\tau_d = \gamma \, d \, S \tag{9-25}$$

where:

τ_d = shear stress in channel at maximum depth (lb/ft^2)
d = maximum depth of flow in channel (ft)

For bottom width to depth ratios less than 4, the above equation conservatively overestimates the shear stress for straight uniform channels. If a more refined analysis is desired, Equation 9-24 should be applied.

The shear stress on the wetted perimeter (τ) is then compared with the permissible shear stress (τ_p) for the channel bottom or lining. If the permissible shear stress is greater than or equal to the computed shear stress, including a safety factor (*SF*), the channel and lining are considered stable. If this condition is not met, a different channel configuration or lining with a higher permissible shear stress is selected. The concept is expressed as:

$$\tau_p \geq SF \, \tau \tag{9-26}$$

The safety factor provides for a measure of uncertainty and conservatism for the designer. A safety factor of 1.5 is generally recommended for the following conditions:

- Critical or supercritical flows
- Climatic regions where vegetation may be uneven or slow to establish
- There is significant uncertainty regarding the design discharge
- The consequences of failure are high

Permissible velocity and tractive shear force values for channel linings are presented in Table 9.16. Determination of allowable tractive shear forces for commercially available erosion control products should be obtained from the manufacturers. Determination of tractive shear forces for grass- and riprap-lined channel sections is a function of the vegetation retardance class and the median riprap size for grass- and riprap-lined channels, respectively. More detailed explanations of the design procedures for grass- and riprap-lined channels are presented below. It should be noted that the use of grass and riprap erosion protection will generally be limited by the channel slope. For steep embankments with slopes greater than 10 percent, grass or riprap channel lining has limited application. Grouted riprap and concrete should be considered when tractive forces exceed the allowable limits for grass or riprap channel linings. Also, commercially manufactured products have been developed

< PREVIOUS VIEW

TABLE 9.16 PERMISSIBLE VELOCITY AND TRACTIVE FORCE FOR CHANNEL LININGS

Channel Lining	Permissible Velocity[1] (ft/sec)	Permissible Tractive Force[2] (lb/ft^2)
Temporary Lining		
Jute netting		0.45
Straw with net		1.45
Coir-double net		2.25
Coconut fiber-double net		2.25
Curled wood mat		1.55
Curled wood-double net		1.75
Curled wood-high velocity		2.00
Synthetic net		2.00
Vegetative Lining		
Class A		3.70
Class B		2.10
Class C		1.00
Class D		0.60
Class E		0.35
Kentucky Bluegrass, Tall Fescue[3]	5-7	
Grass Mixture, Red Canarygrass[3]	4-5	
Lespedeza Sericea, Weeping Lovegrass Redtop, Red Fescue, Annuals[3]	2.5-3.5	
Riprap[4]		
R-1	2.5	0.25
R-2	4.5	0.50
R-3	6.5	1.00
R-4	9	2.00
R-5	11.5	3.00
R-6	13	4.00
R-7	14.5	5.00
R-8	17	8.00
Gabions	22	8.35
Reno Mattress		
6-10 inches thick	12	8.35
10-12 inches thick	15	8.35
12-18 inches thick	18	8.35

Note: 1. USACE (1994), PADEP (2000); permissible velocity should only be applied in cases of straight, uniform steady flow (e.g., bank protection)
2. FHWA (2005a), PADEP (2000)
3. Permissible velocity range for easily eroded to erosion resistant soils (clayey fine-grained soils and coarse-grained soils) at slopes less than 5 percent. Use velocity exceeding 5 ft/sec only where good cover and proper maintenance can be provided.
4. Based on rock with unit weight of 165 lb/ft^3.

to provide greater tractive shear resistance for steeper slopes, but product limitations and the importance of quality construction practices must be clearly understood, if these products are used.

9.7.3.2.1 Grass-Lined Channels

Design of stable grass-lined channels is related to the type of vegetation selected, as reflected by the vegetation retardance class and the tractive stress associated with peak flow. The FHWA (2005a) presents design procedures and data for the design of vegetative linings. Grass linings control erosion by dissipating shear force within the grass stems before it reaches the soil surface, and the root and stem stabilize the soil against turbulent water forces. As indicated in Table 9.17, vegetation retardance for grasses is divided into five classes – A through E. In general, taller and denser grass species have a higher resistance to flow (Class A), while short flexible grass has a low resistance to flow (Class E). The design procedure presented in FHWA (2005a) includes initially estimating the flow depth and effective shear stress on the grass lining based on the grass retardance class, roughness, stiffness, and cover conditions. Manning's n is determined from the effective shear stress, and the discharge is computed and compared to the original estimate. Through a series of iterations, a solution is obtained that balances the flow depth, shear stress, and discharge. The permissible shear stress for the soil type is then estimated from soil properties such as grain size, plasticity, and void ratio for comparison to the effective shear stress.

Haan and Barfield (1978) present a simplified procedure in which permissible velocities for vegetated channels are used with the relationship between channel conveyance and Manning's n for the retardance classes shown in Figure 9.36. Table 9.17 presents the vegetative cover and condition for various retardance classes. The design procedure is to: (1) select the vegetation and an initial estimate of n, (2) determine retardance class and permissible velocity based on Table 9.17 and an initial estimate of VR (velocity V times hydraulic radius R) from the curves in Figure 9.26, (3) obtain the permissible velocity for the vegetation based on Table 9.16 (or state regulatory guidance manual values) and (4) compute the hydraulic radius R from Equation 9-18 using the permissible velocity. The product of the permissile velocity and R from Step 4 is compared with the initial estimate of VR to determine a new value of VR until convergence.

Commercially available erosion control products can be used to reinforce natural vegetation, including non-degradable synthetic fibers, filaments, netting and/or wire mesh. These materials can be integrated into the vegetation and soil lining or applied over the surface, affecting performance and altering the design procedure. The FHWA (2005a) presents design methods for incorporating such products into the design of channel linings, and manufacturers have developed software for analysis of unreinforced and reinforced vegetated channel sections (e.g., North American Green's Erosion Control Materials Design software). When such software is used, knowledge of the grass retardance class, vegetation type, soil type and duration of peak flow are essential requirements in the determination of the tractive shear resistance of the channel linings.

9.7.3.2.2 Riprap-Lined Channels

Riprap linings consist of a layer of rock or stone with a characteristic size, generally designated by D_{50}, the median grain size of the lining material. As with grass linings, the flow conditions and channel shear stress are a function of the Manning's n, and an iterative procedure is required for evaluating lining stability. Values for permissible shear stress for riprap linings are based on laboratory and field research. More turbulent flow conditions are more likely to cause lining failure, and a higher safety factor is recommended for such conditions. Typically, for situations where riprap is needed, a safety factor of 1.5 should be used, although a lower value may be justifiable with mild slopes and low velocities. The FHWA (2005a) presents a design procedure for riprap-lined channels.

Barfield et al. (1981) present a simplified approach for riprap lining design based on USACE procedures and the critical tractive shear stress. The force on median-size riprap at the threshold of

TABLE 9.17 VEGETAL RETARDANCE CLASSES

Retardance Class	Cover	Condition
A	Reed canarygrass	Excellent stand, tall (average 36 inches)
	Yellow bluestem ischaemum	Excellent stand, tall (average 36 inches)
B	Smooth bromegrass	Good stand, mowed (average 12 to 15 inches)
	Bermudagrass	Good stand, tall (average 12 inches)
	Native grass mixture (little bluestem, blue grama, and other long and short mid-west grasses)	Good stand, unmowed
	Tall fescue	Good stand, unmowed (average 18 inches)
	Sericea lespedeza	Good stand, not woody, tall (average 19 inches)
	Grass-legume mixture – Timothy, smooth bromegrass, or orchardgrass	Good stand, uncut (average 20 inches)
	Reed canarygrass	Good stand, mowed (average 12 to 15 inches)
	Tall fescue, with bird's foot trefoil or iodino	Good stand, uncut (average 18 inches)
	Blue grama	Good stand, uncut (average 13 inches)
C	Bahia	Good stand, uncut (6 to 8 inches)
	Bermudagrass	Good stand, mowed (average 6 inches)
	Redtop	Good stand, headed (15 to 20 inches)
	Grass-legume mixture – summer (orchardgrass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (6 to 8 inches)
	Centipedegrass	Very dense cover (average 6 inches)
	Kentucky bluegrass	Good stand, headed (6 to 12 inches)
D	Bermudagrass	Good stand, cut to 2.5-inch height
	Red fescue	Good stand, headed (12 to 18 inches)
	Buffalograss	Good stand, uncut (3 to 6 inches)
	Grass-legume mixture – fall, spring (orchardgrass, red-top, Italian ryegrass, and common lespedeza)	Good stand, uncut (4 to 5 inches)
	Sericea lespedeza	After cutting to 2-inch height, very good stand before cutting
E	Bermudagrass	Good stand, cut to 1.5-inch height
	Bermudagrass	Burned stubble

(HAAN AND BARFIELD, 1978)

motion is considered to be the critical tractive resisting force per unit area (τ_c) for riprap-lined channel sections. Since riprap is a layer of discrete, individual rocks, the movement of an individual rock can initiate further erosion, weakening the overall long-term erosion resistance of the riprap layer. For most situations at coal refuse disposal facilities, the general practice is to consider application of a safety factor for the channel base (SF_{base}) of 1.5 for the design of riprap channel sections. The critical tractive resisting force at the channel base can be estimated from the following equation:

$$SF_{base} = (\cos\theta \tan\phi)/(\sin\theta + \eta b \tan\phi) \qquad (9\text{-}27)$$

where:

ηb = τ / τ_c

τ = shear force per unit area on channel bed = $\gamma R S$ (lb/ft^2)

τ_c = critical tractive resisting force per unit area = $0.047 \gamma (SG - 1) D_{50}$ (lb/ft^2)

γ = unit weight of water (lb/ft^3)

R = hydraulic radius of channel (ft)

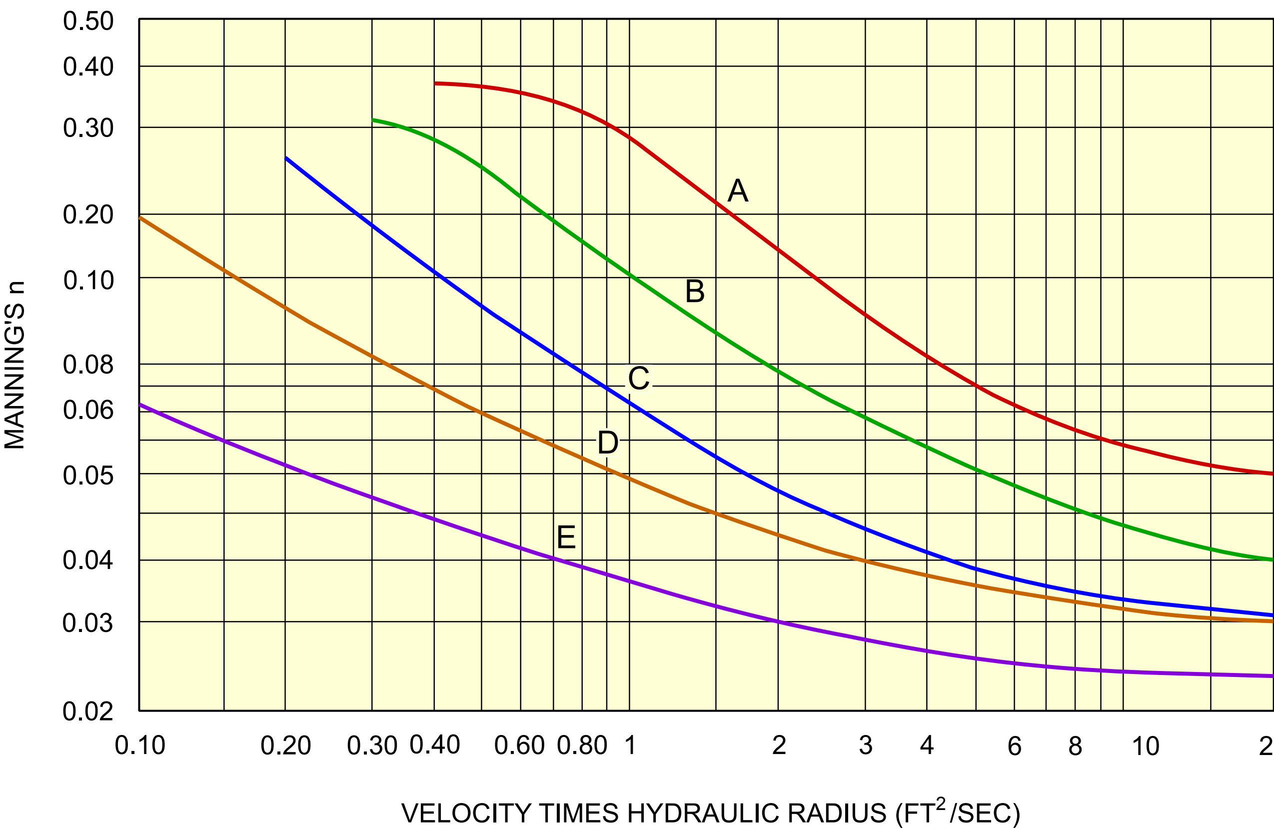
SG = specific gravity = 2.65 for durable limestone and sandstone

D_{50} = median riprap diameter (ft) (Table 9.18)

S = energy grade line slope (equal to the channel slope for uniform flow) (ft/ft)

θ = channel bed slope (deg)

ϕ = angle of repose of riprap (deg)



(HAAN AND BARFIELD, 1978)

FIGURE 9.36 MANNING'S *n* VERSUS *VR* FOR VARIOUS RETARDANCE CLASSES

Table 9.18 presents guidance for estimating D_{50} for various classes of riprap based on data from the National Stone, Sand and Gravel Association (NSSGA, 1989). The FHWA (1989) presents guidance for estimating the angle of repose of riprap based on size and angularity; common values are typically in the range of 35 to 40 degrees. The tractive force on the channel bank is less than that applied to the channel bottom and can be estimated separately (Barfield, et al., 1981), although due to constructability issues, the riprap used for the channel base is also generally used for the channel banks. The FHWA (2005a) also provides a method for checking bank riprap stability.

TABLE 9.18 RIPRAP SIZE DESIGNATION

Graded Riprap Stone

NSSGA No.[(1)]	Sieve or Square-Opening Size (in)			Recommended Filter Stone
	Maximum	D_{50}	Minimum[(2)]	NSSGA Size No.[(1)]
R-1	1½	¾	No. 8	FS-1
R-2	3	1½	1	FS-1
R-3	6	3	2	FS-2
R-4	12	6	3	FS-2
R-5	18	9	5	FS-2
R-6	24	12	7	FS-3
R-7	30	15	12	FS-3
R-8	48	24	15	FS-3

Filter Stone

NSSGA No.[(1)]	Sieve or Square-Opening Size (in)			Recommended Placement Thickness (in)
	Maximum	D_{50}	Minimum[(2)]	
FS-1	⅜	No. 30	No. 100	3
FS-2	2	No.4	No. 100	4 to 6
FS-3	6½	2½	No. 16	8 to 10

Note: 1. National Stone, Sand & Gravel Association (formerly National Stone Association) designation. The table is based on a stone dry density of 165 lb/ft^3.

2. Pieces smaller than the minimum size shown should not exceed 15 percent of the tonnage shipped.

(ADAPTED FROM NSSGA, 1989)

At channel bends, a correction factor K_3 is normally applied for estimating the shear force:

$$\tau = K_3 \gamma R S \tag{9-28}$$

where:

K_3 = $4V^2/R_d$ (dimensionless)

V = straight channel velocity (ft/sec)

R_d = channel radius of curvature measured at the outside channel bank (ft)

The FHWA (2005a) presents details and discussion of design procedures at bends, including the length over which increased shear stresses are experienced and the superelevation of the water surface at and downstream of bends.

Geotextiles have been used increasingly as the filter medium between the riprap and underlying material. However, riprap placed on a geotextile has a tendency to creep over time. Gravitational and water forces have a tendency to re-orient riprap toward the base of the channel resulting in exposed geotextile at the top of the channel slope. The use of an aggregate filter layer between the subbase soil and the riprap is preferred to geotextile due to the increased frictional resistance between the aggregate filter material and the riprap. If geotextile is used, extra riprap should be placed on the channel slopes.

The recommended thickness and gradation of the filter stone in relation to the riprap size is presented in Table 9.18. The recommended riprap layer thickness is greater than or equal to 1.5 times the D_{50} stone size or greater than or equal to the D_{100} size, whichever is greater (FHWA, 1989).

The publication Practical Riprap Design (Maynord, 1978) based on model testing provides riprap size as a function of channel flow depth and Froude number (function of velocity and depth). Additionally, the USACE (1994) provides a generalized procedure for determining riprap size and distribution, including guidance for steep slope conditions (up to 20 percent). The model studies and guidance have proved beneficial for determining riprap requirements in steep slope conditions where the tractive force determined from Equation 9-27 may result in large riprap size relative to flow depth such that uniform flow conditions may no longer be valid. In such steep slope conditions, grouted riprap is often employed to reduce the riprap size. Software is available for riprap design that utilizes the USACE and other procedures.

9.7.3.2.3 Grouted-Riprap Channels

Grouted-riprap channel lining consists of riprap with a cement grout filling the voids to create a monolithic erosion protection mat. The use of grouted riprap can reduce the size and quantity of rock required in a channel lining by creating a greater material mass to resist hydraulic forces. Because grouted riprap linings are rigid, they are susceptible to cracking and damage due to subbase movement or freeze-thaw action. The tractive force resistance of cracked, unconfined grout sections is dependent on intimate surface contact between adjacent sections, section mass, and flow characteristics. In situations of severe cracking with displacement, the primary resistance to tractive forces is the grout and rock fragment pieces, which may be only slightly greater in size and mass than the riprap alone. Thus, to limit the potential for displacement, subbase design is very important.

Design procedures for grouted-rock linings installed on channel banks are provided in FHWA (1989) and address aspects such as rock size and lining thickness, rock grading, filter design, and pressure relief. Rock size and lining thickness are a function of the flow velocity; the median rock size should not exceed two-thirds of the lining thickness, and the largest rock size should not exceed the lining thickness. Table 9.19 and Figure 9.37 can be used to estimate the riprap thickness based on velocity, riprap size and the recommended depth of the grout penetration as follows:

- Based on the average flow velocity, Figure 9.37 can be used to estimate the riprap thickness. For the case where the ratio of bottom width to depth is greater than 2, the flow velocity should be increased by 25 percent when determining the riprap thickness.
- The gradation should be selected based on available riprap class (e.g., cobbles), considering that the median rock size should not exceed two-thirds of the lining thickness and the largest rock size should not exceed the lining thickness. Table 9.19 presents AASHTO class designations based on weight, which is approximately equivalent to the effective diameter shown.
- Grout penetration recommendations for each class or size of riprap are also presented in Table 9.19.

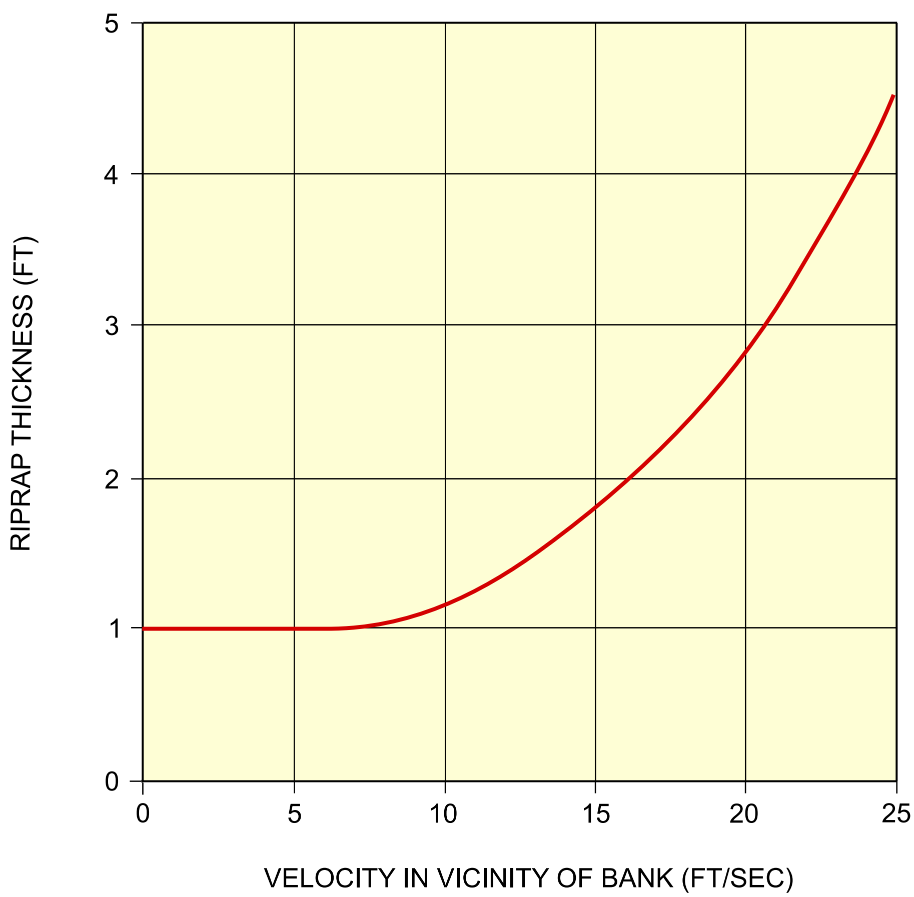
This procedure is applicable to trapezoidal channels where the ratio of bottom width to depth is less than 2. For wider channels, the velocity and shear stress on the channel bottom may be greater, and a corresponding adjustment in velocity or shear stress may be necessary. Typically, the design velocity will increase by 25 percent because the tractive shear for channel side slopes is approximately 75 percent of the maximum shear at the base of the channel.

TABLE 9.19 RECOMMENDED GRADING OF GROUTED ROCK RIPRAP LINING

Rock Sizes	Classes (Percent Larger Than Given Rock Size)					
Equivalent Diameter (ft)	2.75	2.25	1.75	1.25	1.0	0.5
3.5	0-5					
2.75	50-100	0-5				
2.25		50-100	0-5			
1.75	95-100		50-100	0-5		
1.25		95-100		50-100	0-5	
1.0			95-100	95-100	50-100	0-5
0.5					95-100	95-100
Minimum Penetration of Grout (ft)	2	1.5	1.3	0.8	0.67	0.5

(FHWA, 1989)

Similar to riprap channel lining, the use of an aggregate filter layer between the natural ground surface and the riprap is recommended, but geotextiles can also be used in this application provided the riprap does not shift on channel slopes during construction or under subsequent flow conditions.



(FHWA, 1989)

FIGURE 9.37 GROUTED RIPRAP LINING THICKNESS AS A FUNCTION OF FLOW VELOCITY

Foundation preparation is a critical factor in the performance of a grouted riprap lining. Grouted riprap is not flexible and foundation conditions, the potential for development of hydrostatic pressure beneath the rigid mat, and end treatment conditions need to be evaluated as part of the design and construction process. Damage to any section of grouted riprap lining can result in complete failure of the system.

The foundation for grouted-riprap channel lining should be a contoured, firm stable surface. Small surface irregularities can be accommodated within the filter stone layer. The slopes should be graded at no steeper than 1.5H:1V.

Weep holes for hydrostatic pressure relief should be installed in a grouted riprap channel lining. The weep holes should extend below the grouted surface to the gravel zone between the grout and the filter stone. It is recommended that 3-inch-diameter weep holes, constructed from PVC pipe and with protective end screening, be installed at maximum 10-foot vertical spacing (FHWA, 1989).

The head, toe and terminal ends of grouted riprap linings require special treatment such as extension of the lining to rock or to a depth below potential scour to prevent undermining. Additional riprap can provide extra protection against undercutting at a bank toe. Figure 56 of *Hydraulic Engineering Circular 11* (FHWA, 1989) presents recommended construction details for these treatments. Guidance is also provided for rock grading and quality, grout strength and penetration, and filter design and pressure relief measures. The grout mix should typically be designed for a strength of at least 3,000 psi, with maximum aggregate size of ¾ inch and a slump between 3 and 4 inches. Sand mixes may be used where roughness of the grout surface is undesirable. The finished grout should leave the face rock exposed for approximately one-quarter of their depth, and the surface of the grout should expose the matrix of coarse aggregate. The following construction details should be addressed in the specifications for effective grouted riprap lining:

- The prescribed method of riprap placement should prevent segregation of rock sizes, and the riprap should be wet immediately prior to commencing grouting operations.
- The prescribed method of grout placement should control segregation and uniformity.
- The prescribed method of grout placement should facilitate grout penetration by use of vibrating, spading and/or rodding.
- Quality control requirements during construction should be followed, including all material placement methods, grout mix design and strength testing, and recording of quantities of all materials. The volume of grout used should be compared to that required to meet penetration requirements.

9.7.3.2.4 Concrete-Lined Channels

Concrete can provide a continuous, rigid channel lining. Similar to grouted riprap, foundation conditions, hydrostatic pressure development and end treatments are design considerations that must be addressed. Offsets at joints may create additional hydrodynamic uplift forces. Reinforcing steel (No. 6 gage wire mesh or No. 4 reinforcing bars at 6-inch spacing are typical minimum recommendations for 4-inch and 6-inch lining thicknesses, respectively) to reduce the development of thermal stresses, shrinkage and flexural stresses within the Class A (AASHTO classification) concrete is normally provided. The FHWA (1989) provides guidance for design of concrete linings.

Filter layers should be placed below the concrete pavement, and weep holes should be employed to prevent the development of hydrostatic uplift pressures. The weep-hole configuration described above for grouted-riprap revetments should be the maximum spacing considered. Hydrodynamic uplift pressure may also be a consideration where vertical offsets or changes in channel slope occur, particularly at transitions from a steep slope to a flatter slope. The USBR (2007b) presents results of testing for a range of flow velocity, joint widths, and offset dimensions and provides empirical estimates of hydrodynamic uplift as a function of velocity head. The USACE (1990b) presents guidance for analysis of vertical curve transitions and recommends that construction joints be excluded from sections transitioning from steep to flatter slopes. Other measures that may be considered include anchor systems grouted into rock or secured to deadmen.

Edge treatment at the toe, head, and terminal ends should be utilized to prevent undermining. Stub walls or cutoff walls are recommended at expansion joints.

9.7.3.2.5 Commercially Available Composite Erosion Control Products

Commercially available composite erosion control products include permanent reinforced vegetation mats; interlocking concrete blocks; grout-filled nylon mats (unanchored or anchored); soil, rock or concrete filled cellular confinement mats; gabion mats; cabled concrete or other similar products. The allowable tractive shear for these products will generally be specified by the manufacturer or can be determined by computer software developed by the manufacturer. The effectiveness of a particular type of channel lining relative to the intended use should be verified with the manufacturer prior to design. The FHWA (1989, 2005a) provides guidance for some commercially available products.

Applications of interlocking concrete blocks, grout-filled nylon mats, cellular confinement mats, and cabled concrete at coal refuse disposal facilities have included emergency spillway linings, groin ditches, principal spillway outlet channels, and diversion ditches where high velocities are present and excavation into rock is not possible. Design procedures are generally available from the manufacturers and these procedures should include methods for assessing tractive forces and uplift. For grout and concrete systems, additives such as steel or polyester fibers are sometimes employed to increase the strength of the concrete, decrease cracking, and to improve resistance to hydraulic wear.

Some design and construction issues for concrete and grouted riprap linings may also apply to composite systems, including:

- The foundation should be graded smooth and compacted to maintain support and prevent detrimental movement. Some composite systems require a filter and/or drainage layer beneath the lining.
- Side and end anchorages should follow manufacturers' recommendations, and when unusual site conditions are encountered, the manufacturer should be contacted for input.
- Weep holes must be provided where necessary, consistent with manufacturers' recommendations.
- Access to allow inspection and maintenance must be planned. If vehicles and equipment have to cross the channel lining, a reinforced section should be designed.

9.7.3.3 Cavitation

Cavitation of a hydraulic system occurs when flow separates from a containment surface. The result is the formation of a region of subatmospheric pressure that can lead to deterioration of the confining surface. Cavitation can occur in either open-channel or closed-conduit flow. As a rule of thumb, cavitation should be investigated whenever flow velocities exceed 35 feet per second (USACE, 1990b). The potential for damage is a function of flow duration and geometry and the abrasion resistance of the material.

In open-channel flow, cavitation can occur at steepening grade changes. At constructed ogees and paved channels, formation of a cavity of subatmospheric pressure can lead to damaging vibrations; under extreme conditions, these vibrations may actually displace the structure. In unpaved channels, cavitation can produce an upstream propagating erosion of the channel bottom that will worsen and possibly cause failure of the entire hydraulic system if uncorrected.

In closed conduits, cavitation most frequently occurs at sharp turns. For hydraulic systems common to coal refuse disposal facilities, this is particularly likely to occur at the transition from a drop inlet to the entrance of the transport conduit. Should the frequency and duration of high-velocity flow and conduit susceptibility to cavitation damage warrant, methods to prevent cavitation in closed conduits (USBR, 1987a) and USACE (1980, 1990b) may be needed.

9.7.3.4 Directional Changes in Open-Channel Flow

Methods for predicting the superelevation of the surface of curving flow are available, but the superelevation is usually small for subcritical flow velocities. When the flow at a curve is supercritical, a directional change is an exceptionally complicated problem because of the formation, propagation and combination of channel cross waves. These factors must be considered in design if channel overtopping is to be prevented. A simplified determination of the additional flow depth associated with superelevated flow at a rectangular channel bend is provided by the following equation (Chow, 1959):

$$\Delta h = \left\{V_{max}^2/g\right\}\left\{(20/3)(r_c/b) - 16(r_c^3/b^3) + ((4r_c^2/b^2) - 1)^2 \ln\left[(2r_c + b)/(2r_c - b)\right]\right\} \tag{9-29}$$

where:

Δh	=	change in depth associated with the superelevation (ft)
V_{max}	=	velocity from Manning's equation in straight channel section approaching bend (ft/sec)
r_c	=	radius of curvature measured to the centerline of the channel (ft)
b	=	width of the channel (ft)
g	=	gravitational acceleration = 32.2 ft/sec^2

The FHWA (2005b) presents a method for estimating the superelevation of flow at bends in trapezoidal channels based on the velocity head. Additional guidance is provided in USBR (1987a) and USACE (1980, 1990b).

9.7.3.5 Directional Changes in Conduit Flow

Sharp directional changes and intersections in closed conduits cause static and dynamic thrusts that must be considered in design. Depending on the conduit size, the internal pressure and the flow velocity, thrust forces can cause the backfill surrounding the conduit to compress, and joints in the conduit can open or the deflection of the conduit can exceed the allowable pipe deflection due to the resulting movement. This is particularly a problem when the conduit location is shallow (or the conduit is supported above ground) and soil resistance to movement is low. Where thrust movements are identified as a potential problem, a concrete thrust block is often poured around the conduit to add mass and to distribute pressure over a larger area of soil. Discussions for evaluating the necessity of special construction measures at directional changes or intersections and procedures for analyzing requirements of thrust blocks are provided by Brater et al. (1996) and in engineering manuals from conduit manufacturers. The need for thrust blocks should be evaluated for fittings such as tees and prefabricated bends and at directional changes where conduit joints could separate.

9.7.3.6 Materials Selection

The construction of any type of hydraulic system, and particularly a closed-conduit system, should not be undertaken without investigating the hydraulic and structural suitability of the proposed construction materials. This topic is discussed in Section 9.3.3 and Section 6.6.6.1 of this Manual. The discussion herein is for open-channel systems and culvert-conduit systems with emphasis on the importance of selecting materials that will function as intended and avoid costly repair or replacement. A detailed discussion of decant system material selection is presented in Section 9.7.4.1 with specific attention to the structural integrity of the decant system due to embankment loading conditions.

9.7.3.6.1 Open-Channel Lining Systems

As part of the hydraulic design process, the suitability of the selected channel lining material should be determined. Material selection should be based on factors such as resistance to abrasion, the type of flow in the channel (continuous or intermittent), the acidic nature of the coal refuse, the impact that water quality characteristics may have on lining system integrity, the availability of the lining material, foundation conditions (particularly for more rigid lining systems), the constructability of the system (e.g., site topography and site access conditions), cost, and maintenance.

9.7.3.6.2 Conduit Materials

Culvert pipes can be corrugated metal (CMP); concrete; corrugated plastic (CPP), both smooth-wall or corrugated interior; high density polyethylene (HDPE); polyvinyl chloride (PVC); aluminum; steel or other materials. The type of material recommended is generally a function of intended use of the culvert (temporary or permanent), loading conditions, foundation conditions, construction limitations, and cost. Uncoated corrugated metal and steel are generally not recommended for long-term use in a mining environment because of the corrosion potential of mine water. Limited usage or protective coatings can make these material alternatives more acceptable.

Culverts constructed of concrete, CMP and aluminum are manufactured in various shapes (box, oval, arch, etc.) for installation in areas with limited height and clearance. Minimum and maximum cover height limitations are associated with all culvert installations. CPP and HDPE pipe are generally more flexible and structurally stable in conditions where minor settlements may occur.

In all applications, installation is critical to the structural integrity and hydraulic conveyance capabilities of conduits. The thickness of and installation procedures for bedding and backfill materials are critical to successful culvert construction. Joint connections in most culvert installations should be minimal. If joints are present, they should be soil-tight and in most applications watertight. Fusion-welded HDPE, gasketed joints for some concrete and CPP, glued PVC, and welded steel pipe provide the most watertight applications. Pressure testing can be performed to verify that joints are watertight.

9.7.4 Types of Hydraulic Systems

Sections 9.7.1, 9.7.2 and 9.7.3 have identified: (1) basic considerations for planning hydraulic systems at coal refuse disposal facilities, (2) the primary components of all hydraulic systems and techniques appropriate for their analyses, and (3) special design considerations associated with hydraulic systems. This section integrates this basic information into the following discussions of specific types of hydraulic systems most common to coal refuse disposal facilities.

9.7.4.1 Decant Systems

Decant systems at impounding disposal facilities serve one of the following three purposes or a combination thereof:

1. To remove clarified water during normal disposal of fine refuse.
2. To provide outflow during low-precipitation storms so that storage volume will be available if a large storm occurs.
3. To drain (possibly in conjunction with a spillway) the stored volume of inflow due to a large storm within a reasonable period after occurrence of the storm.

A decant system typically consists of: (1) an inlet section located in the impoundment at the elevation required for controlling or limiting the normal water surface level, (2) a transport section consisting of a closed conduit beneath or through the embankment, and (3) a discharge section located downstream from the embankment so as to minimize erosion of the embankment toe. It is the responsibility of the designer to select the optimum location for the transport section conduit and the discharge point, based on foundation conditions, conduit size and material, and discharge rates.

The optimal selection of the decant inlet type, configuration and location is primarily a function of the overall facility configuration, the required discharge capacity, the method for disposing of the fine refuse, the rate at which the impoundment level will rise during operations, and eventual abandonment or post-mining land use requirements. Design considerations include the height of the decant inlet invert above the settled fine coal refuse, the method of evacuation of clarified water between the decant inlet invert and the settled fine coal refuse level, the trash rack system for preventing debris from entering the decant pipeline, and the potential buoyancy of inlet and conduit.

Often the most challenging design consideration is selecting a conduit that will withstand the weight of the overlying refuse embankment and that can deform as the foundation and embankment materials settle vertically or displace laterally. Generally, only specially designed concrete, high density polyethylene (HDPE), or steel pipes are capable of withstanding the high pressures beneath a refuse embankment. Section 6.6.6 provides guidance for the selection of decant materials and designing for embankment loading, including design of the decant pipe bedding and backfill. The transition from the riser to the transport section of the decant must be designed to handle the impact loads associated with directional change in flow. This is particularly important for rigid pipe systems, which are sensitive to movement. Control of seepage along and adjacent to the transport section of the decant, where it extends through the embankment, must also be addressed (Section 6.6.2.3).

To prevent erosion, the decant outlet must be able to accommodate the design flow rate and velocity. The rate and velocity of outflow must also be considered in the design of downstream conveyance and/or storage structures.

In the following sections, guidelines for the design of the hydraulic conveyance components of a decant system are presented. The inlet, transport and outlet sections of a decant system will be discussed separately. The discussion is based on the use of concrete, welded steel, and HDPE, since these are the most commonly used materials for impoundment decant systems.

9.7.4.1.1 Inlet

Location within the Impoundment

Location of the decant inlet within the impoundment is a function of the following factors: (1) site terrain and foundation conditions, (2) limitations of the transport section of the decant such as length and foundation conditions, (3) type of embankment construction such as upstream or downstream construction, and (4) limitations in positioning the slurry discharge. Often, the inlet is positioned in the upstream portion of the impoundment, so that fine refuse can be deposited to form a delta at the upstream slope of the embankment and clarified water accumulates in the farthest upstream portion of the impoundment.

Trash Rack

The entrance to decant inlets should be protected by a trash rack. Even a partially obstructed pipe will have a substantially reduced capacity, thus increasing the potential for dam overtopping during a large storm event. Trash racks can become plugged if the openings are too small, and openings that are too large can result in the obstruction of the pipe due to the intake of large debris. The connection of the trash rack to the decant structure must be strong enough to withstand the hydrostatic and dynamic forces exerted on the trash rack during periods of high flow.

It is recommended that trash rack openings be sized so that they are a maximum of one-half the nominal dimension of the outlet conduit. The minimum opening size should be 6 inches by 6 inches or greater. The USBR (1987a) recommends that the area of trash rack openings be established based on the flow velocity through the rack. Where trash racks are inaccessible for cleaning, this velocity should not exceed 2 feet per second. A velocity of up to approximately 5 feet per second can be tolerated for racks that are accessible for cleaning. An anti-vortex device should be incorporated into the trash rack design to prevent the formation of a flow-inhibiting vortex during periods of high flow. The USBR (1987a) recommends that the anti-vortex device extend at least two diameters in front of and to each side of the inlet. In practice, these devices are usually part of the trash rack assembly and consist of a steel plate with width equal to the width (or diameter) of the rack.

Evacuation of Water below Riser Invert

As part of the design process, a sufficient minimum depth of water and associated height of riser above the settled fine coal refuse should be provided in order to prevent short circuiting of the impoundment and release of fine coal refuse slurry. This depth is frequently estimated based on experience and judgment, usually varying between 5 and 10 feet to accommodate settling of fine refuse in the slurry. Settling tests can be performed in the laboratory to aid in determination of the rate of settlement and the required impoundment retention time. The settling velocity can be determined from Stoke's Law based on the particle size and specific gravity of the fine coal refuse:

$$V_s = \left[g\,(S\text{-}1)\,D^2\right]/18\,\nu \qquad (9\text{-}30)$$

where:

V_s	=	settling velocity (cm/sec)
g	=	acceleration of gravity (981 cm/sec^2)
D	=	diameter of particle (cm)
ν	=	kinematic viscosity of fluid (cm^2/sec)
S	=	specific gravity of particle

The minimum height of the riser above the average settled fine coal refuse level can be calculated by determining the approximate detention time based on the outflow rate (slurry discharge rate plus watershed base flow) and impoundment geometry. To establish a balance between the detention time and period for settling, an iterative process is required. Because of the broad range in refuse particle sizes and varying impoundment geometry, experience and engineering judgment are generally used for determining the depth of water to be retained over the fine refuse.

Pumps are generally used to remove the clarified water below the riser invert. Such pumps should be capable of meeting the discharge requirement without increasing the flow rate through the settling zone sufficiently to cause removal of fine coal refuse.

Inlet Riser Pipe Buoyancy

Inlet riser pipe sections may be susceptible to the buoyancy forces sufficient to cause uplift, if the pipe weight and backfill height is not adequate. This is particularly a concern for steel and HDPE decant pipe inlets. Inlets to an HDPE decant pipe are generally installed in a trench extending up a natural hillside with the inlets located at specified elevation intervals. Sufficient fill or anchorage must be provided to overcome buoyant forces, with a recommended factor of safety of 1.5.

Extension of Inlet

The decant riser inlet elevation is generally established based on storm routing such that adequate impoundment freeboard and surcharge storage is provided. As a refuse embankment dam is raised to increase the capacity of the impoundment, the riser inlet is correspondingly raised to provide additional slurry disposal capacity. If the decant system has multiple risers, the lower riser is sealed and abandoned and the next upper riser is put into service. Extension of an inlet will require that the connection, extension, and new inlet section be designed to accommodate the flows and related forces associated with continuing operation. If the decant system is designed with multiple risers, sealing (such as by employing a bolt-on plate) and abandonment (such as by embedment in concrete) of the riser must be part of the design. To assess the potential for additional stress and deflection at the base of the sealed riser, the loads associated with abandonment (along with future embankment construction) must be evaluated.

In some instances where high embankments are required and there are concerns about pipe loading, designers have limited the service life of a decant inlet and transport pipe section by abandoning the entire system and installing a second system at a higher elevation. In such situations, the original inlet and transport section should be sealed and abandoned upon completion of the replacement system.

9.7.4.1.2 Transport Section

The transport section of the decant pipe must typically be watertight and must include seepage control structures to intercept the flow of seepage along and adjacent to the conduit where it extends through the refuse embankment. For these reasons, structural considerations based on the external loading generally govern the design of this portion of the decant once the diameter has been established. Determination of external loading conditions and structural design of a decant pipe are addressed in Section 6.6.6.

Rigid pipe used within a refuse embankment is typically concrete pressure pipe because of the large external loads and the requirement that the pipe be watertight. Concrete pipe is rigid and sensitive to differential settlements, particularly at the inlet-riser transition, resulting from directional flow impacts, foundation conditions, or imposed embankment loading conditions.

Infiltration and exfiltration leakage problems can develop within the transport section of the decant pipe. Irreparable damage of the pipe and possibly failure of the refuse embankment can occur if such leakage is undetected. As part of the installation process, project specifications should require that pipes installed within the limits of the impounding refuse embankment be pressure tested prior to backfilling so that detected leaks can be immediately repaired.

Joint tightness is also a concern in non-pressure pipe installations, such as in the upstream inlet section of the decant pipe or in a concrete riser extension. Infiltration or exfiltration at joints could impact the pipe backfill if the material is erodible. Testing of non-pressure pipe joints is recommended.

Flexibility, watertightness and relatively easy installation procedures have led to the use of HDPE for decant pipes. Structural evaluation procedures associated with the flexible conduit under large embankment loads are presented in Section 6.6.6. For flexible pipe, both structural and hydraulic design considerations may control the decant pipe size. Thus, evaluation of the embankment loading and required wall thickness of the pipe should be performed in parallel with the hydraulic design.

Schematic examples of decant inlets that are most adaptable to coal refuse disposal facilities are illustrated in Figure 9.38, while illustrations of actual decant systems that have been constructed are shown in Figures 9.39 and 9.40. The primary advantage of the inlet types shown in Figure 9.38a through d is their access for expansion as the level of settled fine refuse increases. The primary disadvantage is that the length of conduit required upstream of the embankment, beneath the settled slurry, is relatively great. The length of conduit in the Figure 9.38a and c decant systems is particularly significant because it must be as long as the entire impoundment. However, an advantage of these systems is that the inlet is located at the point where the water depth is normally the greatest (when the slurry is discharged at the embankment end of the impoundment), allowing the decant system to drain the water without pumping.

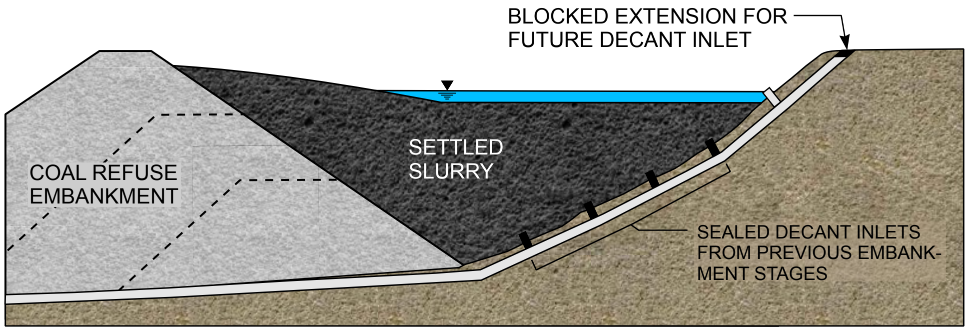
The inlet shown in Figure 9.38d appears to be the simplest arrangement because it offers the shortest conduit length and the inlet can be extended in height by adding subsequent sections as the settled fine refuse level rises. Major difficulties with this type of decant system are: (1) its location within the impoundment, which makes access very difficult, and (2) large impact forces at the point of directional change from the inlet to the transport section. With regard to the latter, the effect of forces due to falling water must be accounted for by providing a curved section at the base of the vertical riser and/or constructing a concrete pad to distribute impact loads to the underlying soils.

Figure 9.39 presents an example of a decant system installed at a diked-impoundment facility. In this example, a flexible pipe serves as the decant with the riser extending vertically up into the impoundment. A water return line to the coal preparation plant is installed parallel to the decant line, and the impoundment water level is controlled by a pumping system. The decant inlet is positioned so as to provide for storage of the design storm runoff, which for a diked impoundment is predominantly the precipitation falling upon the impoundment surface. Note that the water return line, as would the case for any conduit passing through the embankment, must be designed with seepage control measures.

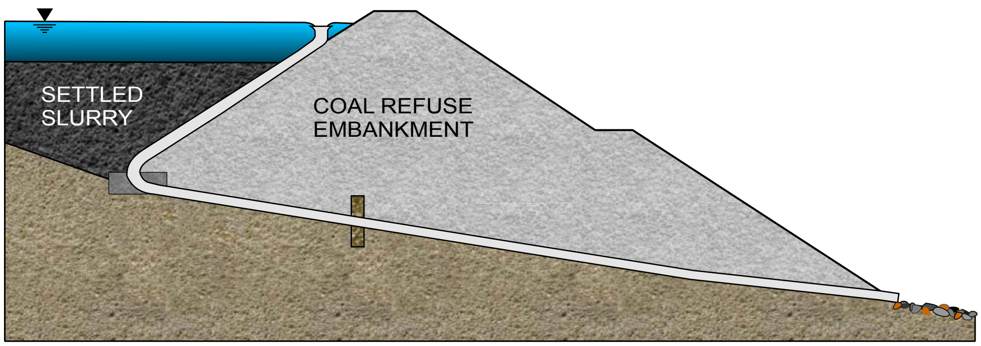
Figure 9.40 shows an example of a decant system installed at a cross-valley impoundment, where multiple risers, or inlets, extend from the transport section of the flexible pipe. In this example, a pumping system is used to maintain the impoundment water level, and the decant serves to rout significant storm runoff through the facility. As the settled slurry accumulates within the impoundment, the lower riser inlet is sealed and the next upper riser inlet is fitted with a trash rack for operation. In the example system shown in Figure 9.40, a seepage interception zone has been installed within the backfill for the decant system.

9.7.4.2 Overflow Spillway Systems

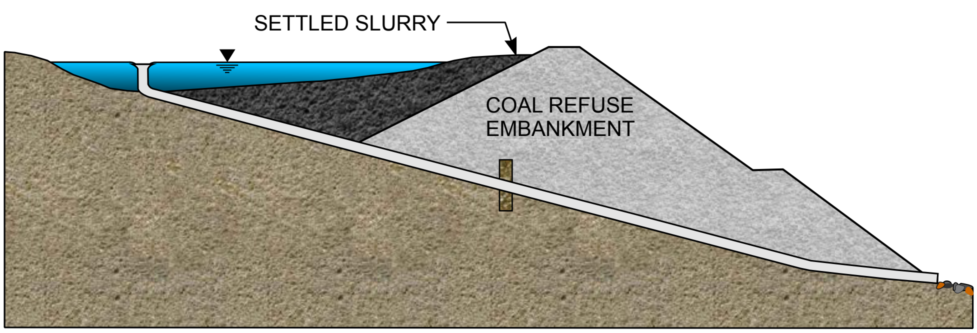
When the design storm is not stored in the impoundment at coal refuse disposal facilities, overflow spillways that protect downstream life and property by safely discharging the accumulated runoff from large storms are the most crucial hydraulic systems. They frequently must be designed to convey substantial flows over steep terrain. The vital importance and design complexity of these structures combined with the complications of a constantly changing embankment configuration, dictates that spillways for coal refuse disposal facilities be viewed differently than spillways for other types of impoundments.



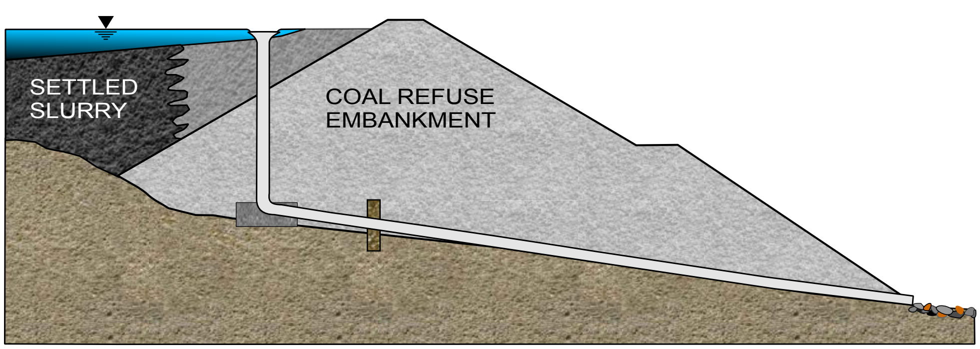
9.38a DECANT INLET ON SLOPE AT SIDE OF IMPOUNDMENT



9.38b DECANT INLET ON EMBANKMENT SLOPE

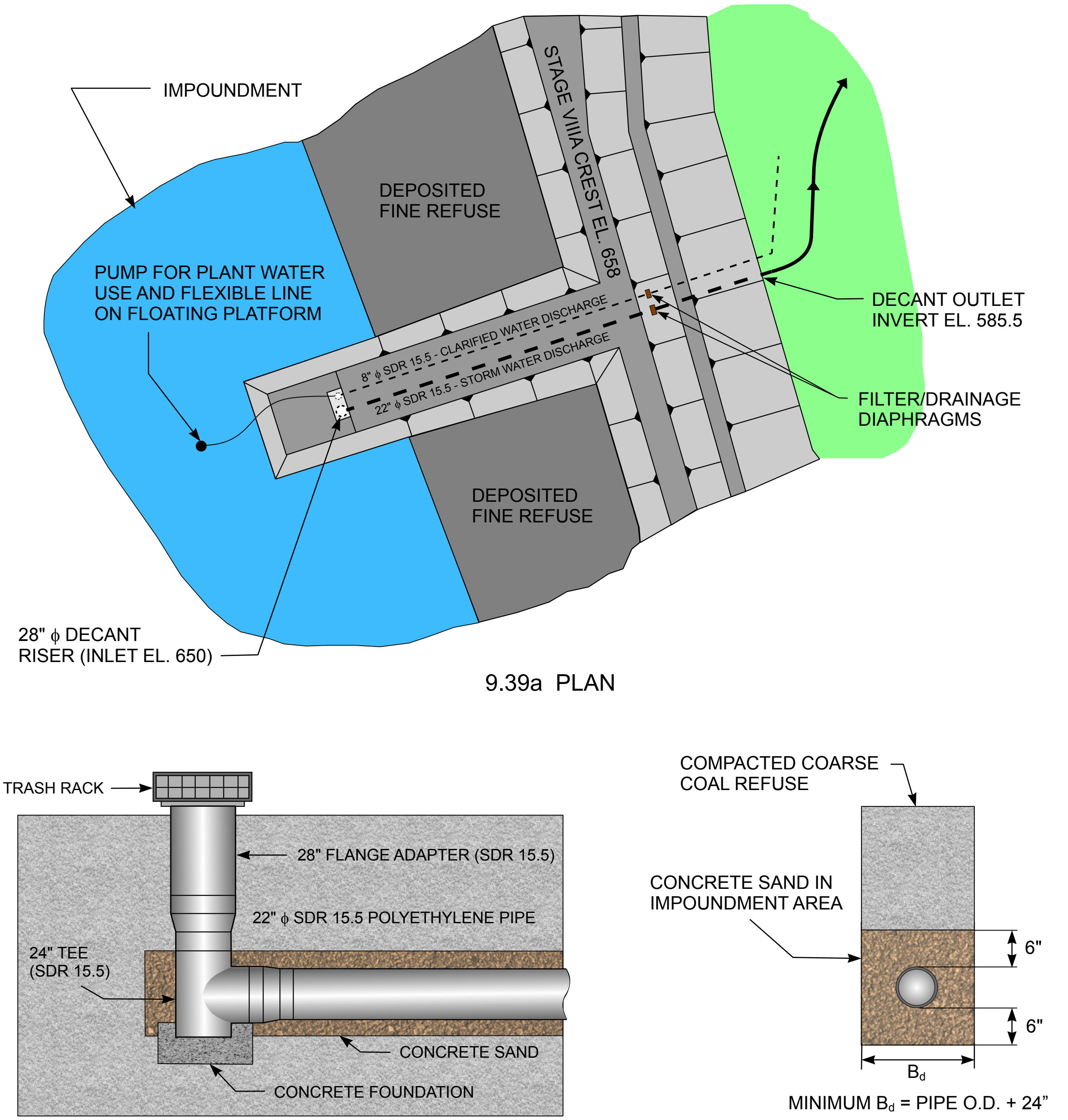


9.38c DECANT INLET AT UPSTREAM END OF IMPOUNDMENT



9.38d VERTICAL DECANT INLET

FIGURE 9.38 EXAMPLES OF DECANTS



9.39a PLAN

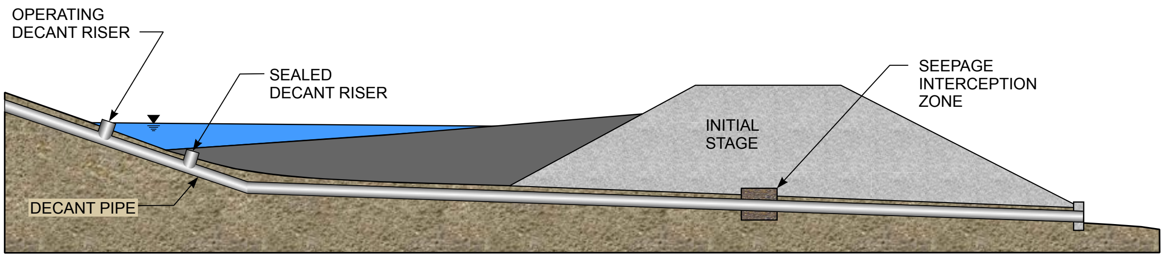
9.39b DECANT RISER DETAIL

9.39c TRENCH DETAIL

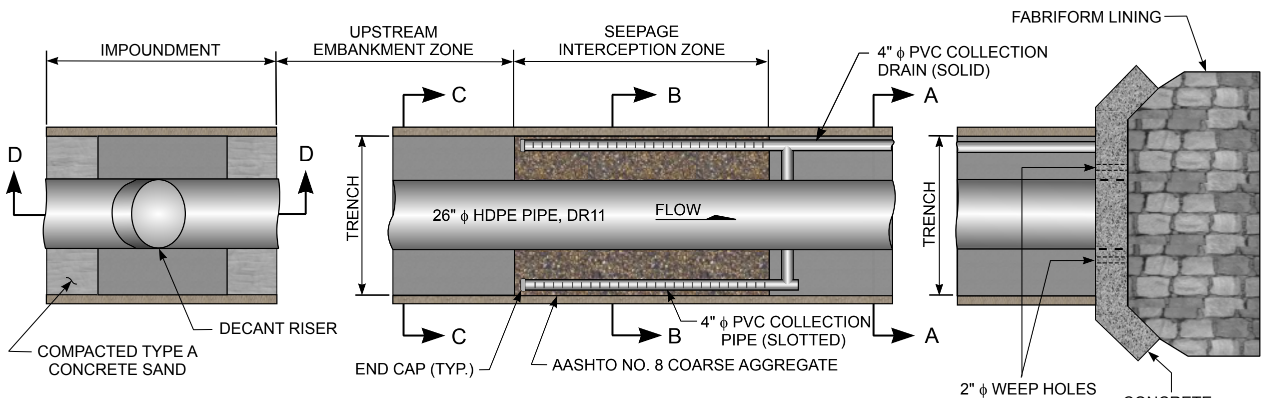
FIGURE 9.39 EXAMPLE DECANT SYSTEM AT DIKED IMPOUNDMENT

Most transport sections for open-channel spillway systems at impounding disposal facilities are channels excavated into an embankment abutment. Erosion protection should be provided if the abutment materials in the excavated channel are not durable and could erode, creating stability concerns. The spillway channel should extend to a point downstream of the embankment and be appropriately lined and have sufficient freeboard to protect the embankment. Examples of spillways that have been constructed at impounding disposal facilities are illustrated in Figures 9.41 through 9.43.

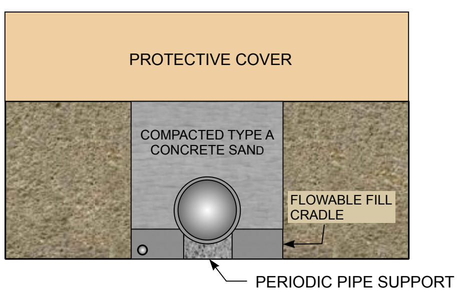
Figure 9.41 illustrates a condition where a spillway channel was constructed in an existing valley fill and side hill embankment, necessitating a variety of channel configurations and linings. Concrete and grouted riprap materials were used in steep channel segments and at changes in flow direction.



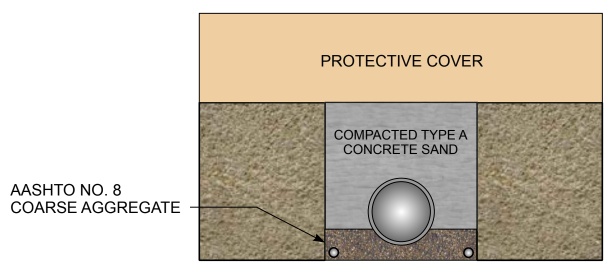
9.40a PROFILE



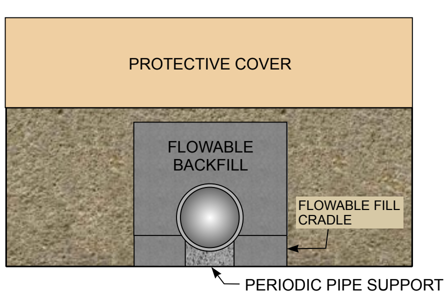
9.40b PLAN



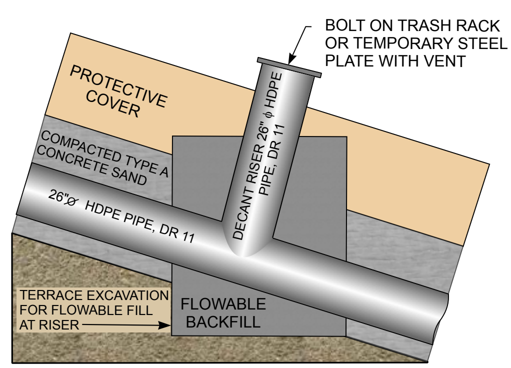
9.40c SECTION A - A



9.40c SECTION B - B



9.40e SECTION C - C



9.40f SECTION D - D

FIGURE 9.40 EXAMPLE DECANT SYSTEM AT CROSS-VALLEY IMPOUNDMENT

< PREVIOUS VIEW

Figure 9.42 illustrates a condition where the height of the natural abutment for a new refuse embankment was only slightly higher than the planned embankment crest. A spillway was constructed by making an excavation through the abutment so that storm flows would discharge into an adjacent valley and the refuse embankment would not be endangered.

Figure 9.43 illustrates a cascade spillway system. A new spillway at the higher elevation was excavated into a hillside to direct flow to a point above the beginning of the lower spillway. The plunge pool in the lower spillway dissipates the energy of the cascading water and turns the direction of flow downhill and away from the embankment.

The design capacity for spillway systems is usually determined by the reservoir routing procedures presented in Section 9.8. Spillway design involves selecting the control location and method to achieve the required flow capacity while fully utilizing the available storage capacity.

Figures 9.44 and 9.45 show typical inlet and outlet controls for spillway systems. To determine the most appropriate control for a particular situation, the following factors should be considered:

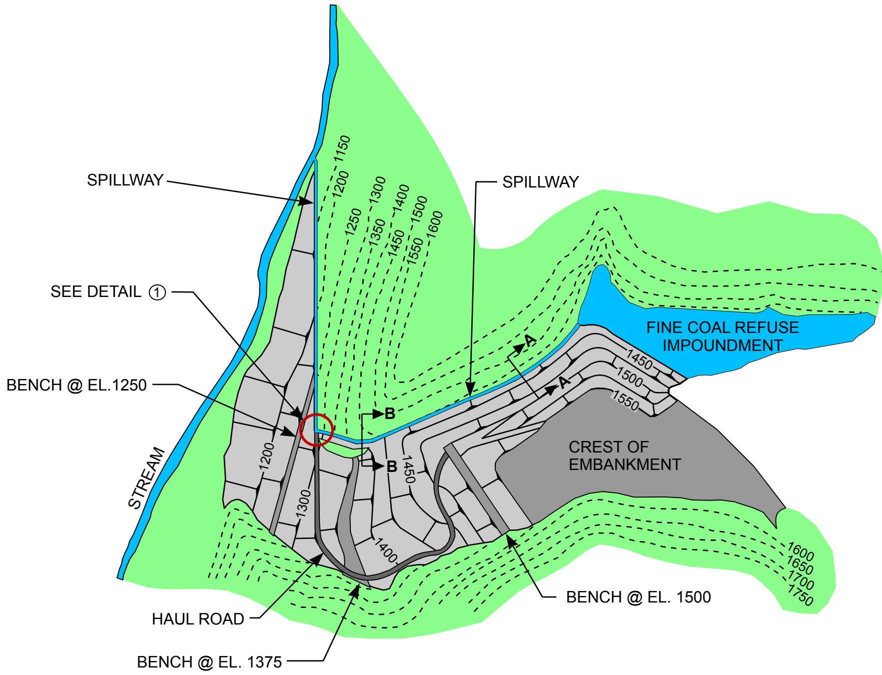
Inlet Control – Inlet control for spillway systems is desirable when it is important to minimize the size of the transport channel, including the following situations:

- The length of the channel downstream from the inlet that can be economically constructed is limited (flow will be supercritical downstream from the inlet).
- The area for construction of a downstream transport section is limited due to terrain instability (hilly and steep terrain).
- Substantial storage capacity is available in the impoundment and economies can be realized by reducing the size of the transport section or channel (encountered at some large facilities sited in long valleys).
- Competent, erosion-resistant material is available at the inlet of the system but not at the outlet (inlet control is more easily accomplished).

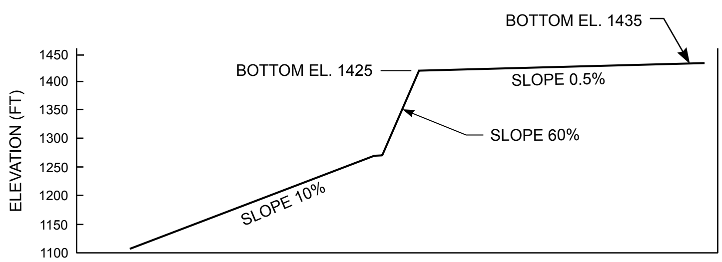
Outlet Control – Outlet control for spillway systems is desirable when it is important to minimize the velocity of flow or the grade of the transport section, including:

- The upstream flow in the transport section must be maintained at subcritical velocities to minimize erosion of soft channel materials. An overflow weir located at the downstream end of the transport section or channel can be used to create this condition.
- Competent materials that provide erosion resistance are present at the outlet.
- A natural overfall occurs at the outlet, where the water discharges by free fall without causing significant damaging erosion.

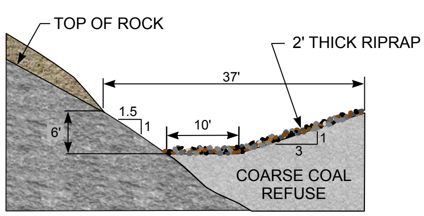
Regardless of the point of control, the primary requirement of spillway system design is that the discharged flow not adversely affect the safety of the overall coal refuse disposal facility. Section 9.7.2.3 discusses the types of outlets available for either safely discharging the flow away from the facility or for dissipating the flow energy with a stilling basin. Generally, the spillway discharge point is located a sufficient distance away from the embankment so that a special hydraulic structure is not required. The examples illustrated in Figures 9.41 and 9.43 show excavated channels discharging onto steep slopes. The major design issue for this approach concerns the frequency and severity of damage that might result from such discharge. Discharges should be rare occurrences, and damages should be limited to surface erosion of the steep slope without adverse impact to the disposal facility embankment. Also, the discharge point and slope should be within mine property ownership and not be



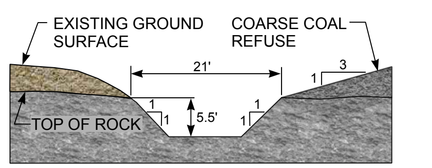
9.41a PLAN



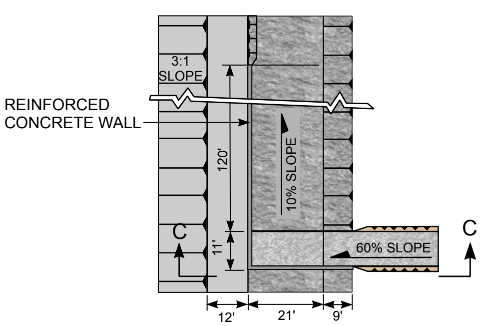
9.41b SPILLWAY PROFILE (NTS)



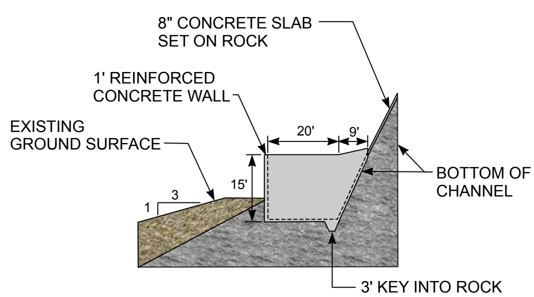
9.41c SECTION A - A



9.41d SECTION B - B

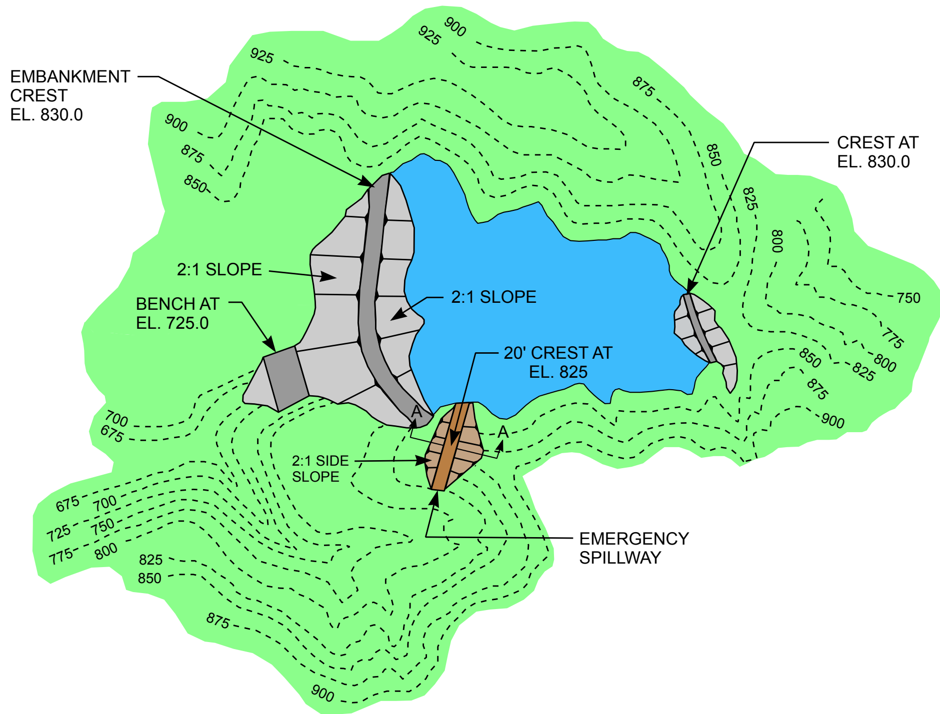


9.41e DETAIL 1

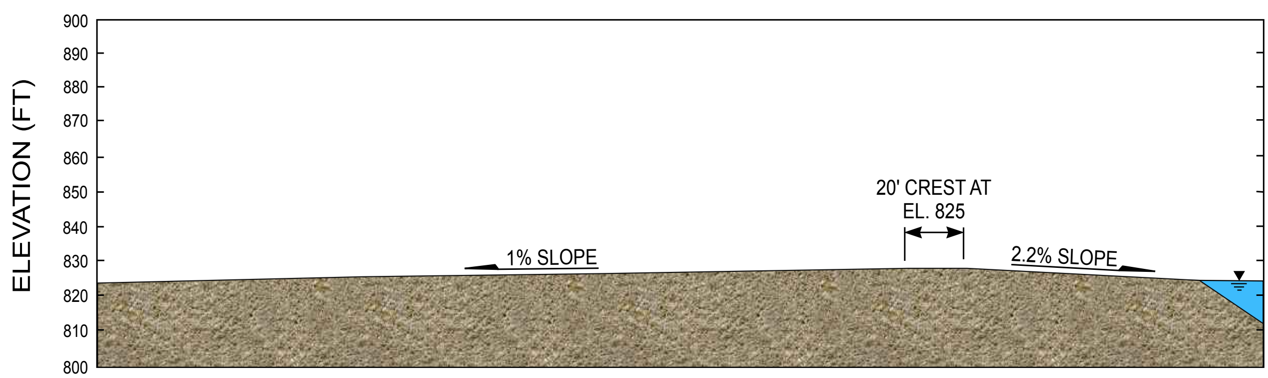


9.41f SECTION C - C

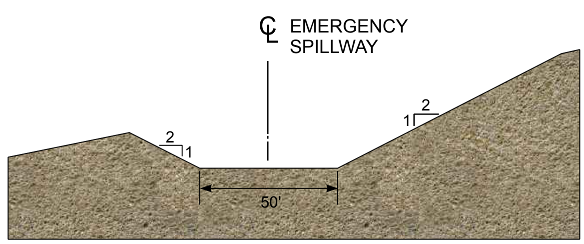
FIGURE 9.41 EXAMPLE SPILLWAY SYSTEM – A



9.42a PLAN

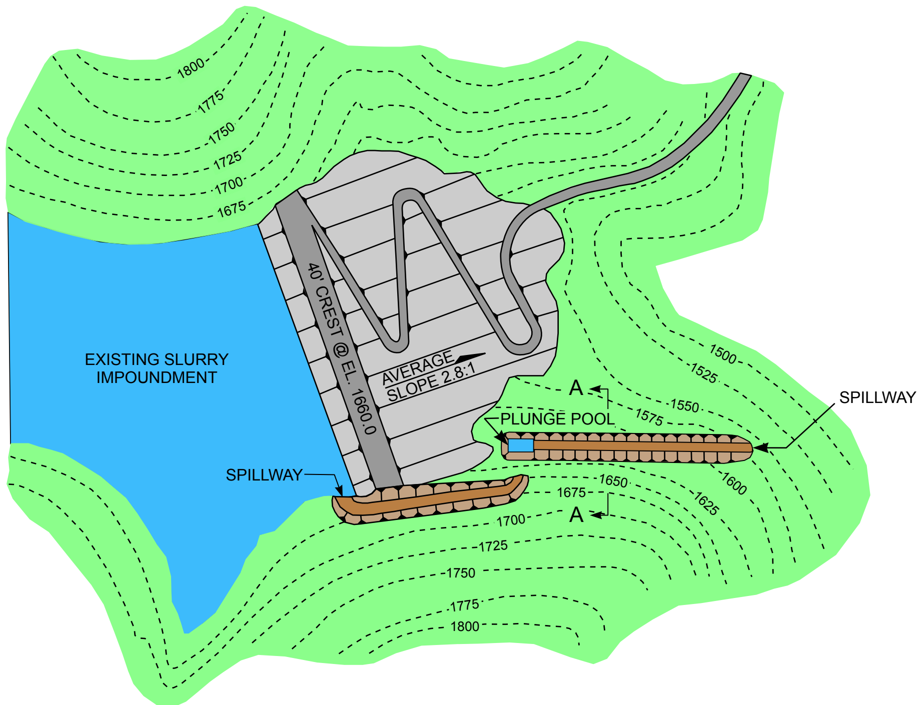


9.42b SPILLWAY PROFILE

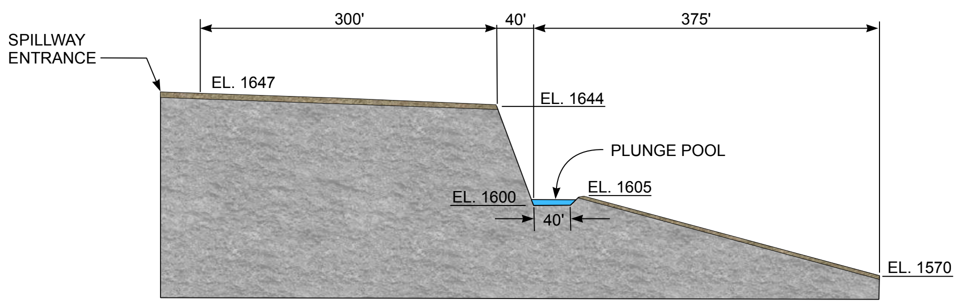


9.42c SECTION A - A

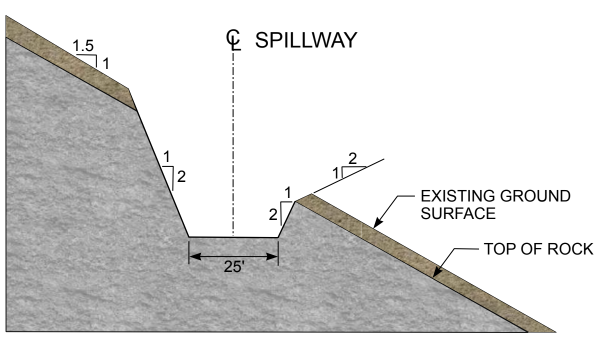
FIGURE 9.42 EXAMPLE SPILLWAY SYSTEM – B



9.43a PLAN

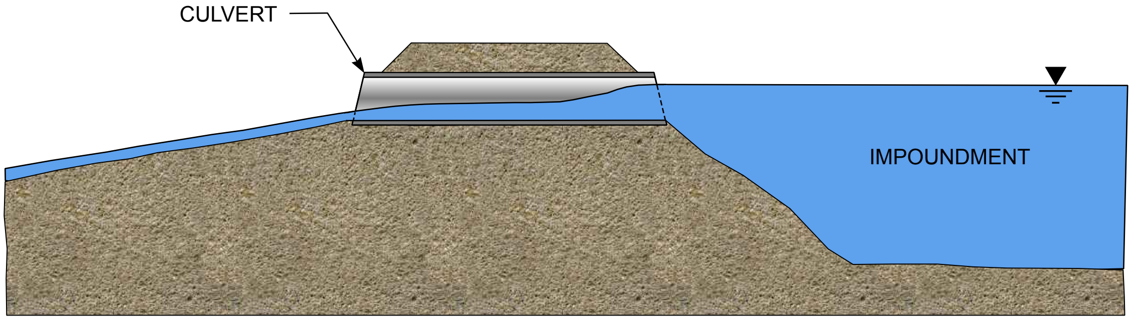


9.43b SPILLWAY PROFILE

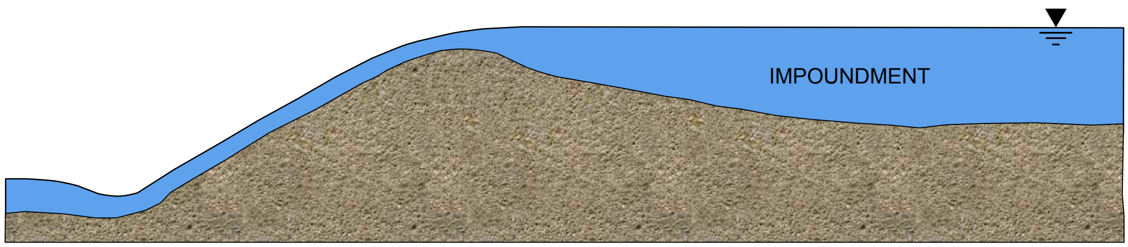


9.43c SECTION A - A

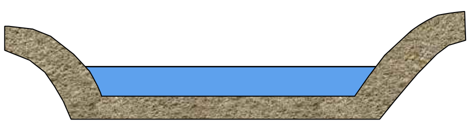
FIGURE 9.43 EXAMPLE SPILLWAY SYSTEM – C



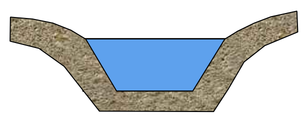
9.44a CULVERT INLET CONTROL



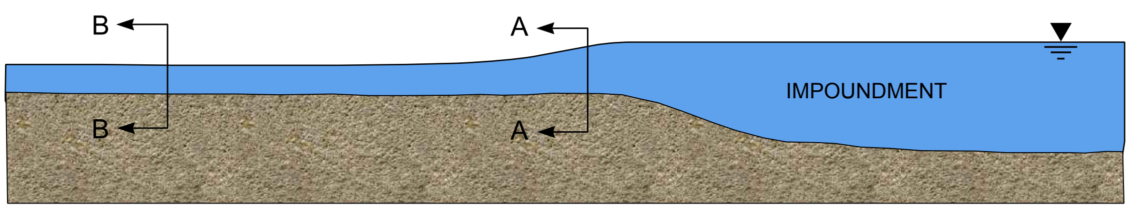
9.44b INLET TO STEEP CHANNEL CONTROL



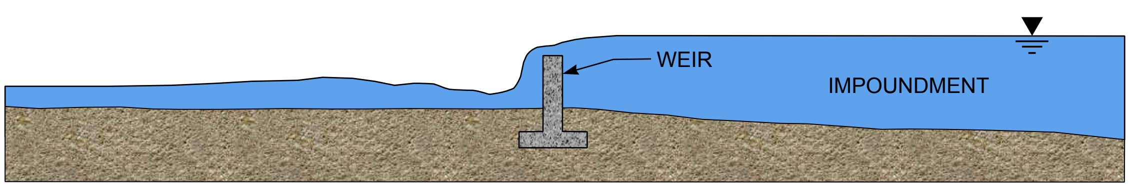
SECTION B - B



SECTION A - A

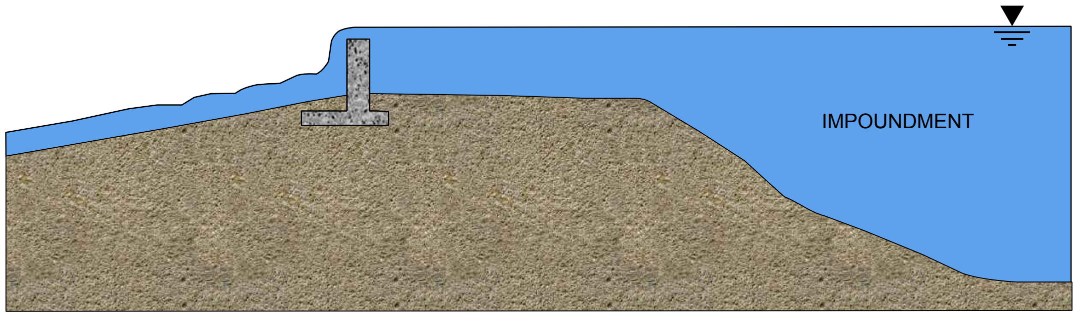


9.44c NARROW INLET CONTROL

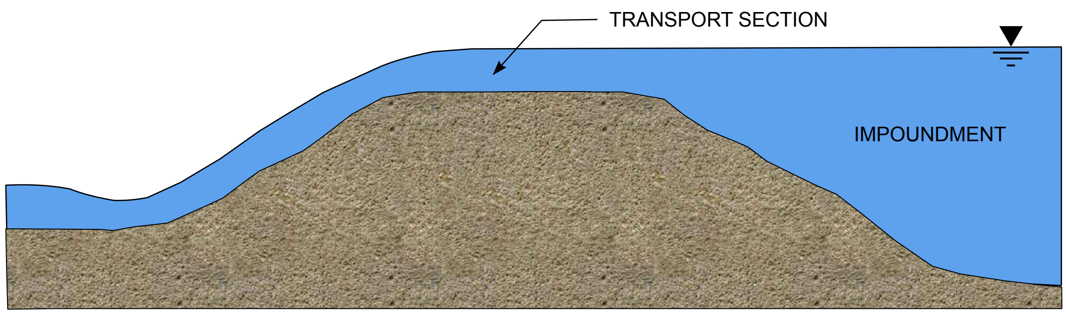


9.44d WEIR INLET CONTROL

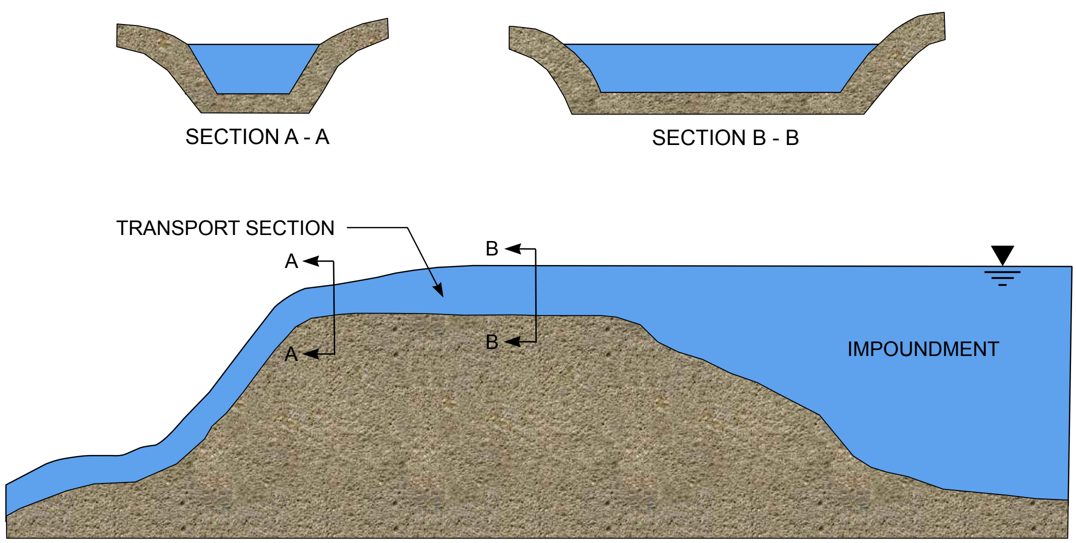
FIGURE 9.44 SPILLWAY INLET CONTROL



9.45a WEIR OUTLET CONTROL



9.45b OUTLET TO STEEP CHANNEL CONTROL



9.45c NARROW OUTLET CONTROL

FIGURE 9.45 SPILLWAY OUTLET CONTROL

upgradient from structures or public transportation facilities. More difficult to evaluate are property and/or environmental damages that might occur below the discharge point due to vegetation loss, surface erosion and even local landslides. The following criteria are suggested as design guides for preventing impacts to a disposal facility embankment when the discharge of open-channel spillways onto slopes is contemplated:

- If the spillway will only be activated during very large storms (i.e., on the order of the PMF), special provisions are seldom required at the outflow point because the resulting erosion damage typically will not be significantly greater than the erosion damage that would have occurred anyway, and the probability of the spillway functioning many times during the operational period of the facility is low.

- If the spillway is expected to be activated as often as every 25 years or more frequently, special provisions should be made to transport the flow without excessive erosion, or the flow should be discharged through a separate system.

High velocity flow in open channels represents a significant source of energy that must be controlled for safe water conveyance. Special considerations for the design of excavated channel spillway systems include:

- Initial planning and design should account for the manner in which the spillway system may need to be extended or modified as the disposal facility increases in size.
- The channel materials (natural or constructed) must be resistant to erosion (Section 9.7.3.1).
- The stability of the excavated slopes forming the channel must be evaluated (Section 6.6.5).
- Potential erosion at directional changes, particularly during supercritical flow, must be evaluated and accounted for in the channel design (Section 9.7.3.3).
- Rigid spillway linings must be designed to resist the development of hydrostatic and flow-induced uplift pressures.
- Sufficient freeboard to contain the discharge within the spillway channel must be provided.

An important consideration in the design of rigid spillway linings is uplift pressures. For spillways with rigid lining, uplift is typically estimated based on the hydrostatic head associated with the normal pool level applied at the upstream end, varying linearly to the hydrostatic head associated with the downstream flow depth (tailwater) level. Where there is a potential for open, offset joints in steep spillway chutes, the velocity may be converted to dynamic pressure. While the theoretical maximum dynamic pressure should be calculated (e.g., the stagnation condition where all velocity is converted to pressure), surface effects will limit the dynamic pressure. The USBR (2007b) presents the results of testing for a range of flow velocities, joint widths, and offset dimensions, which is intended to be a refinement of the estimation of associated potential uplift forces where open joints are a concern. However, the uplift associated with the impoundment level must be estimated separately based upon site-specific conditions. For example, the uplift pressure for dynamic forces is typically 50 to 75 percent of the stagnation pressure for joint offsets ranging between ⅛ and ¾ inch (e.g., at an average flow velocity of 35 feet per second, measured pressure was approximately 10 feet of head as compared to a 14-foot stagnation value for a ½-inch offset and ¼-inch joint). Such uplift forces should be considered in situations where lining failure would trigger impoundment failure. Measures for reducing or controlling the potential development of uplift pressures include:

- Grouting to control foundation seepage, where applicable.
- Installation of an underdrain system consisting of perforated pipe within a graded sand and gravel filter to control seepage.
- Use of rigid foam insulation between the concrete spillway and underdrain system to prevent freezing in cold climates.
- Installation of embedded waterstops in floor joints to prevent the flow of water through the joints to the foundation.
- Use of longitudinal reinforcement and transverse cutoffs at joints to prevent relative displacement.
- Increasing the weight of the channel lining.

For channels on rock foundations where the above measures may not be effective, structural design of the channel lining with anchors to resist uplift pressure should be employed.

9.7.4.3 Diversion and Collection Ditches

Diversion and collection ditches are important because the collection and control of runoff from a refuse embankment surface, from the slopes of an embankment, and from hillsides draining to and away from the embankment should minimize environmental damage to downstream waters, prevent damage to the embankment, and reduce maintenance and repair efforts related to site erosion. This is particularly true when a refuse disposal facility is reclaimed and the drainage facilities must function with limited repair for a long period of time. Basic procedures for designing diversion and collection ditches are the same as those for other types of channels, as discussed in Sections 9.7.1 and 9.7.3.

The following are the most important considerations in the design of drainage channels:

- Diversion ditches should be designed to reduce the amount of water reaching the disposal facility during moderate storms when it can be shown that they will appropriately reduce operating or environmental concerns. It is seldom practical to design diversion ditches that will not fail during very large storms such as the PMF.
- The constantly changing size and configuration of refuse disposal facilities often makes it necessary to provide short-term runoff diversion and collection ditches at intermediate development stages. Erosion protection criteria for these structures must be established on a case-by-case basis and may differ from the erosion protection procedures for more permanent ditches and channels.
- When a diversion or collection ditch will later serve as a permanent channel, the capacity and type of erosion protection material should meet the requirements for the permanent installation.
- When a collection ditch will also serve to route the design storm discharge from an impoundment outlet (by receiving the discharge from a decant or open-channel spillway), its capacity and durability should be based on the impoundment design storm. In this instance, the concern is that the collection ditch must be able to function without damage to the integrity of the embankment for storms up to the impoundment design storm.

9.7.4.4 Culverts

Culverts are typically used beneath access roads that permit runoff from minor storms to pass without loss of road use. The appropriate design storm for such culverts depends on the importance of the roadway to the overall operation and the effort and cost of repairing the road if it is overtopped or washed out. An exception might be a roadway to an impounding embankment that must remain open during or immediately following a large storm. However, the design criteria established for impoundments in Section 9.5 are based on the assumption that it is not practical to maintain access to the impoundment during that time period.

The basic design requirements for culverts were previously discussed in Sections 9.7.1 and 9.7.2.

9.7.4.5 Natural Streams

A natural stream flowing adjacent to a coal refuse embankment can cause significant damage if the water at flood elevation can reach the refuse embankment toe and cause erosion. Flow in natural streams can be determined based on the principles of open-channel flow (Section 9.7.2.2), although the analysis to determine maximum flow depth may be more involved due to the irregular cross sections of natural streams. Computer software for estimating the flood levels along natural streams is readily available.

As with constructed channels, the flow in a stream can be controlled by upstream or downstream conditions, depending on the stream slope, configuration and discharge. Normally, the flow may be assumed to be uniform and Manning's equation (Equation 9-17 or 9-18) can be used to approximate the flow depth and velocity, except immediately upstream of bridges, road embankment crossings, natural channel constrictions, etc. that cause backwater effects requiring the use of open-channel profile analysis for determination of flow depth and velocity. The roughness coefficient *n* for natural streams may be in the higher ranges shown in Table 9.13 due to vegetation including trees, variations in alignment, and the irregularity of cross sections. The hydraulic radius can be calculated in a manner similar to that for a constructed channel, except that the areas and wetted perimeter must be determined from topographic maps or cross-section drawings.

Based on the peak runoff to a natural stream, the approximate water depth can be calculated. If the computed flow depth indicates that the water in the stream could encroach upon the refuse embankment toe, provisions to prevent embankment erosion or disruption of the facility hydraulic system should be employed.

9.8 RESERVOIR ROUTING

As part of the design of the hydraulic system for an impounding refuse disposal facility, reservoir routing analyses are typically performed to determine the outlet spillway discharge and impoundment storage requirements. This is critical for open-channel spillway systems, but is also important for sites that rely on storage of the design storm and discharge of the runoff through the decant, because these facilities have 10 days to discharge the storm inflow in accordance with the criteria presented in Section 9.5. The methodology and key parameters for routing analyses are described herein, and references to the computer software typically employed are provided. For specific applications, frequent reference is made to texts and publications on hydraulic design and engineering such as Chow (1959), USBR (1987a), and the USDA *National Engineering Handbook* (1956).

9.8.1 Basic Routing Methodology

Reservoir routing is performed by analyzing the inflow hydrograph (Section 9.6), the storage capacity of the impoundment (Section 9.7), and the discharge-head relationship for the spillway outlet (Section 9.7) to determine the reservoir level and spillway outflow hydrograph. The spillway outlet may consist of a conduit system (decant or principal spillway), emergency open-channel spillway, or a combination of conduit and open-channel spillway. Flood routing analyses should be based upon an initial impoundment water level no lower than the lowest functional decant inlet. For impoundments that rely solely on a conduit system, the majority of the runoff from the design storm must be stored within the impoundment because of the limited discharge capacity of the conduit. However, the conduit must be capable of discharging 90 percent of the runoff within a 10-day period following the design storm.

Impoundments with an emergency open-channel spillway generally have significant discharge capacity, and thus less of the design storm runoff must be stored within the reservoir. The open-channel spillway inlet will be above the normal pool level, such that some initial storage or accumulation of runoff from the inflow hydrograph occurs before outflow through the open-channel spillway is initiated. Subsequently, the spillway discharges at a rate dependent on the reservoir level, which in turn is a function of the inflow hydrograph and storage capacity. After the peak of the inflow hydrograph passes, the reservoir level will continue to rise until the inflow rate and spillway outflow capacity are equal. Thereafter, the reservoir level will decline as spillway discharge becomes predominant. This relationship is shown in the hydrographs presented in Figure 9.3. The development of the reservoir-storage relationship is discussed in Section 9.6.1.4, and the spillway-discharge relationship is discussed in Section 9.7.2.

USBR (1987a) and the USDA *National Engineering Handbook* (1956) present mathematical procedures for computation of flood routing, using an iterative process to arrive at the outflow hydrograph. Computer programs such as HEC-1 and HEC-HMS are frequently employed to perform the routing analysis USACE (1998b, 2000).

9.8.2 Basic Routing Parameters

An important factor that differentiates coal refuse disposal facility impoundments from other types of impoundments is that the embankment and impoundment configurations continually change with both the disposal of coarse refuse on the embankment and the disposal of fine refuse slurry into the impoundment. These effects must be accounted for in the hydraulic design and reservoir routing for a refuse disposal facility.

Figure 9.46 presents a sectional view of an impoundment facility illustrating features that impact the routing of floods and the development of design parameters, including normal pool elevation, minimum pool level, surcharge storage, normal freeboard and design-storm freeboard.

The design-storm freeboard for an impounding embankment is a function of the wave height and the wave run-up conditions at the upstream face of the embankment. Guidance for the evaluation of design storm freeboard for reservoirs is presented in USBR (1987a). Coal refuse impounding embankments are typically required to have a minimum design storm freeboard of 3 feet above the maximum reservoir pool level associated with the design storm, consistent with a fetch of less than 1 mile.

Since the crest elevation of a slurry impoundment can change frequently, the facility plans and specifications should include a graph or table that shows the maximum allowable normal pool level and allowable spillway and decant levels for each stage of operation. To ensure that adequate freeboard is available to handle the design storm, the normal pool level and spillway inlets must not be raised until the appropriate crest elevation has been reached. The disposal of fine coal refuse within an impoundment affects the reservoir storage capacity. While most of the accumulation occurs below the impoundment operational water level, the slurry discharge results in the build-up of deposits forming "deltas" or "beaches" above the pool level. While this impact is frequently insignificant for many valley-fill type impoundments, it can have an impact on routing and freeboard particularly at diked-type facilities. This loss of storage capacity is generally estimated based

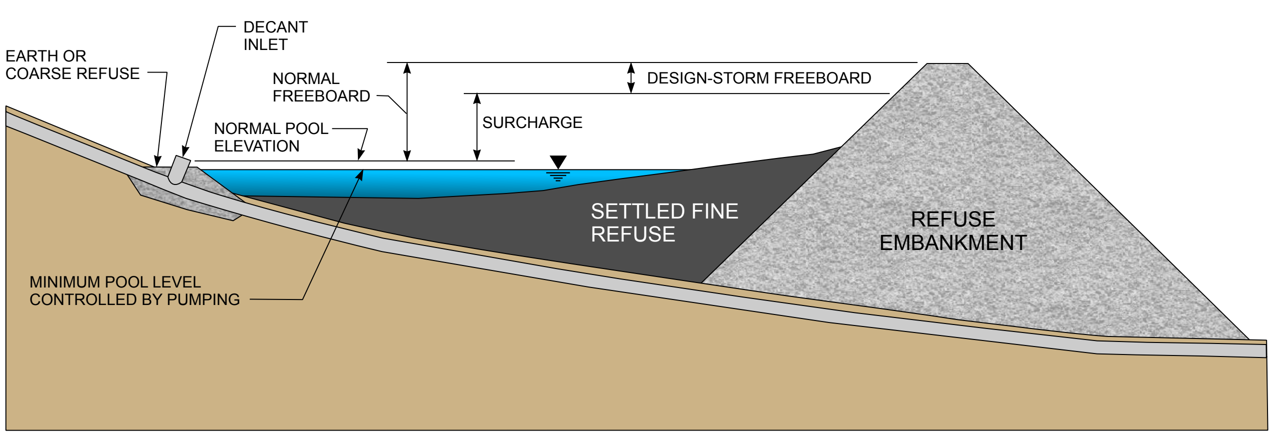


FIGURE 9.46 SECTIONAL VIEW THROUGH IMPOUNDING FACILITY

upon the position and elevation of the planned slurry discharge and an assumed slope of the deposit (typically between 1 and 3 percent).

9.9 DAM BREACH ANALYSIS AND INUNDATION MAPPING

9.9.1 Background

Impoundments are assigned a hazard-potential classification based on the consequences to downstream workers, residents and property in the event the dams were to fail. FEMA (2004a) classifies dams as "high hazard potential," "significant hazard potential," or "low hazard potential," and the states typically have comparable classification systems. A high-hazard-potential dam or embankment is one whose catastrophic failure would likely result in loss of human life. A significant-hazard-potential dam or embankment is one whose failure would not be expected to result in the loss of human life, but could cause substantial property damage. Minimal property damage would be expected from the failure of a low-hazard-potential dam or embankment. As discussed in Section 3.1, MSHA requires evaluation of the hazard potential for other impoundment breach pathways, such as breakthrough into underground mines.

In practice the hazard-potential classification for a dam may be apparent from site conditions; for example, a large impoundment located upstream from a populated area will likely be classified as having a high hazard potential. To aid in determining the hazard-potential classification and to assist in preparation of an Emergency Action Plan, a dam-breach analysis is performed to determine the downstream consequences of a dam failure.

An EAP should be prepared for high- and significant-hazard-potential dams and embankments, so that procedures are in place for responding to an emergency at the dam and to conditions in the potentially-inundated area downstream (Chapter 14.0). This is a requirement in several states, and MSHA encourages EAP preparation for high- and significant-hazard-potential dams in order to protect the public that would potentially be affected by a dam failure (MSHA, 2007). An important step in the preparation of an EAP is to perform a dam-breach analysis so that the potentially-inundated area downstream from the dam or refuse embankment can be defined. Scenarios for dam failure, methods of analysis, software used for analysis, data requirements, and other aspects of the dam-failure flow release and determination of the resulting inundated area are discussed in the following paragraphs.

9.9.2 Failure Scenarios

Dam failures can occur in a number of ways, but most result from: (1) overtopping of the dam due to inadequate spillway capacity, (2) failure of the dam structure as the result of an earthquake, or (3) the flow of water through the embankment leading to development of a breach in the embankment (i.e., "piping").

The first scenario is the most common, as there are many existing dams in the U.S. that do not have a spillway capacity adequate to handle the most extreme rainfall events. Consequently, states are requiring upgrades at these facilities to meet current requirements. These upgrades typically include such remedial measures as raising the dam crest and increasing spillway capacity. In some cases, measures such as armoring the dam with roller-compacted concrete are employed to allow overtopping to occur while the structure remains intact.

Piping failures result from pathways through a dam embankment where seepage gradually increases transport of fine materials until a point is reached where pore pressures are high over a relatively large area and a breach initiates. Spillway/decant pipes extending through a dam embankment are vulnerable locations for this phenomenon to occur, and care must be taken to minimize the potential for seepage flow along these structures.

A major earthquake can result in an increase in soil pore pressure and sliding failure, usually in the upstream portion of the embankment. The crest of the dam drops during the embankment failure, resulting in a breach. Upgrades to prevent this type of failure typically require major and very costly repairs.

Catastrophic dam failures can also result from causes such as landslides, foundation failure, sabotage or damage to operational equipment.

While some technical studies of breach formation have been carried out (Wahl, 1998), the composition of earth embankment dams is highly variable and there is little actual data available for calibrating model results. Thus, breach development has not been accurately related to a specific dam failure scenario.

At the present time, modeling of potential dam failures usually involves two basic scenarios. The first is overtopping of the dam during high flow (Inflow Design Flood) conditions and the second is catastrophic failure of the dam on a day with normal (sunny day or fair weather) flow conditions. These two scenarios provide a reasonable representation of the range of conditions resulting from the possible failure modes.

Coal refuse disposal facility embankments are developed in stages over several years, ultimately resulting in a massive structure sometimes 1,000 feet or more in width and several hundred feet high. Dam breach analyses should consider intermediate stages (which may have a narrower embankment cross section) as well as the final facility configuration, because the failure of an intermediate stage could occur more rapidly and could result in a greater breach flow and thus more significant downstream inundation than breach of the facility in its final configuration.

An important consideration for coal refuse slurry impoundments is the volume of fine coal refuse that could be released during a hypothetical dam failure. Based upon data from a wide range of tailings dam failures, Vick (2000) estimates that, while on average about 25 percent of the impounded contents are released, the release of impounded contents can approach 100 percent. Rico et al. (2008) evaluated historical records of tailings dam failures and releases to identify factors affecting the runout distance and peak discharge, finding that, on average, one-third of the tailings and water (to the post-failure level) was released. Overtopping from floods appears to mobilize and release more tailings.

While considerable uncertainty exists, the above estimate appears to be consistent with observations reported by Owens (2008) for incidents at coal refuse impoundments. The settled fine coal refuse frequently remains in a soft or loose condition until sufficiently consolidated and thus may be in a flowable state. Michael et al. (2005) performed a literature review to evaluate the ability of fine coal refuse to flow. While no specific test data are available for fine coal refuse, evaluation of other tailings materials led them to conclude that saturated refuse may be susceptible to high pore pressure and static liquefaction when containment is breached. In order to conservatively estimate the amount of fine coal refuse that could be released from a dam breach, some states prescribe consideration of all water, slurry and settled fine coal refuse contained within the impoundment from the breach invert to the crest, when computing downstream flows. Adoption of a reduced volume of settled fine coal refuse will generally require site-specific information concerning the consistency and resistance to flow of the material.

In addition to failure of the dam, breakthrough of the impoundment into an underground mine may represent another type of release pathway. This pathway could lead to significant discharges in streams in other watersheds, depending on the alignment and extent of extraction of the underground mine.

TABLE 9.20 INUNDATION ANALYSIS SOFTWARE

Program	Method of Analysis
HEC-1	Muskingum-Cunge Modified Puls
HEC-RAS	One-Dimensional Unsteady State Flow
SMPDBK	Approximate Method
DAMBRK	One-Dimensional Unsteady State Flow
FLDWAV	One-Dimensional Unsteady State Flow

9.9.3 Analytical Methods

The failure of a dam results in a condition referred to as rapidly varied unsteady flow. This is a very complex flow condition that can be modeled with computer software. However, programs that provide the most sophisticated modeling of rapidly varied unsteady flow can be difficult to use. Thus, the popular programs used for dam breach analysis represent simplifications to some degree of rapidly varied unsteady flow analysis. A listing of software frequently used for dam breach analysis and the analytical approach employed is provided in Table 9.20.

HEC-1 is frequently used for dam breach analysis for mine impoundments, and the other software listed in the table are less commonly employed. In terms of sophistication, HEC-RAS, DAMBRK and FLDWAV are the most technically advanced and should provide more accurate results than HEC-1. HEC-1 and HEC-RAS were developed by the USACE Hydrologic Engineering Center, while the latter three programs in the preceding table are National Weather Service programs.

HEC-RAS (River Analysis System) is a second generation program from the USACE Hydrologic Engineering Center and is the successor program to HEC-2. It was first released in 1995 and gained unsteady flow analysis capabilities in 2000. The unsteady flow portion of the program was adapted from UNET. The program has the capability of modeling mixed flow regimes and can account for channel constrictions and off-channel storage.

DAMBRK was developed by Fread (1988) for modeling unsteady flow associated with dam breaches. FLDWAV, which was introduced in 1998, is the successor to DAMBRK and DWOPER and provides advanced capabilities over both programs. SMPDBK is based upon an approximate methodology, and under some circumstances can provide results within 10 percent of the results provided by DAMBRK.

In terms of modeling accuracy, both HEC-1 and SMPDBK have clear limitations. The accuracy of these programs diminishes in situations involving channel constrictions and resulting backwater. The final version of HEC-1 was released in 1998, and the program has been replaced by HEC-HMS, which offers one-dimensional kinematic wave routing for dam breach analyses. This methodology does not account for inertial and pressure forces, and the energy slope is assumed to be equal to the channel slope. Thus, HEC-HMS is best suited to relatively steep channels and urban areas where natural channels have been modified to regular shapes and constant slopes.

FLDWAV is the most sophisticated program currently (2009) available. It utilizes finite-difference approximations to solve the Saint-Venant equations for one-dimensional unsteady flow and can account for natural features such as off-channel storage and channel constrictions. The program is capable of handling a wide range of channel configurations and data input. However, FLDWAV requires some calibration for optimum accuracy. Other programs such as HEC-RAS and predecessors to both HEC-RAS and FLDWAV are capable of providing adequate results depending upon the nature of the breach, outflow hydrograph and downstream channel configuration.

GIS-based software is gaining in popularity in hydrology and hydraulics applications and has been used in combination with unsteady flow analysis software. WMS (a GIS-based hydrologic model) can be used in conjunction with software such as HEC-RAS and SMPDBK. BREACH and FLDWAV have reportedly been used in combination with GIS-based software to assess dam breaches and inundation mapping. Also ESRI, the developer of ArcGIS, and the USACE Hydrologic Engineering Center have worked together to create HEC-GeoRAS, which allows the results of flood routing analyses to be displayed in a GIS environment. Eventually, GIS-based models, either packaged with or used in combination with sophisticated unsteady flow analysis software, will be the accepted approach for dam breach modeling and presentation of results.

Another issue related to software selection is the available topographic data for the analyses, the level of accuracy required, and the user's familiarity with the software. It is common to obtain topographic and cross-section information from USGS quadrangle maps with some field observation and verification. Generally, a high degree of accuracy is not required for defining inundation limits and for identifying potential evacuation requirements, particularly in remote areas. Thus selection of less sophisticated software for EAP development is quite often adequate. However, use of a breach analysis to support a hazard-potential classification other than high hazard potential may require careful evaluation of the assumptions incorporated into the software. As discussed previously, communication should be maintained with dam safety regulators (both state and federal) relative to software usage.

9.9.4 Input Data

9.9.4.1 Breach Parameters

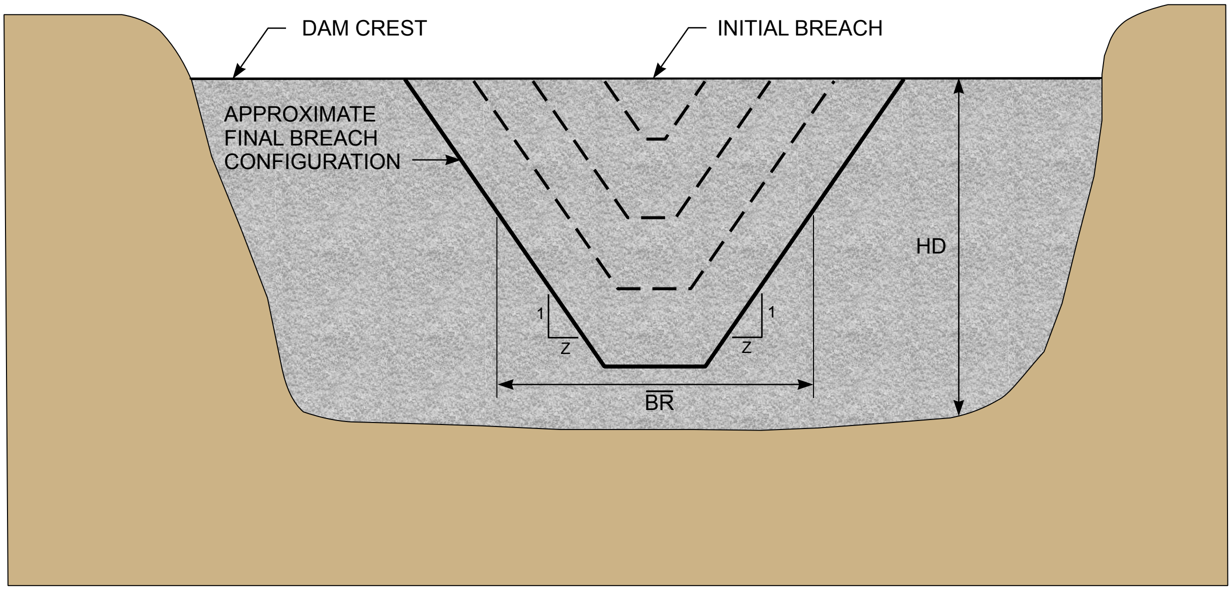
A breach in an earth embankment dam is generally assumed to be trapezoidal in shape and can be defined by depth, width, side slopes and development time. A comprehensive study of breach development was carried out by Wahl (1998) for earth dams. While an accurate depiction of breach development is desirable, the process is highly variable and difficult to predict, and conservative assumptions based on published guidelines are normally accepted. Discussions with agency personnel should prove valuable in this regard. In situations where the downstream channel is relatively broad in comparison to the volume of the release, the use of conservative breach development geometry and time may not matter, as discussed in Section 9.9.6. Some state agencies require that the release include the entire volume of tailings. When a site-specific evaluation is performed, it should be compared with the following guidance for estimating the minimum volume of release for fine refuse:

- An equal volume of fine refuse (as stored) and flood water
- One-third of the fine refuse stored

The volume of fine refuse released can then be used with other site-specific factors to estimate the breach depth and configuration and duration of outflow. Some guidelines for determination of breach parameters (FERC, 1993) are presented in Figure 9.47. Physical limitations such as the width and depth of the valley should be considered when applying the guidelines.

Programs used for routing dam breach flows (HEC-1 or HEC-RAS, DAMBRK, FLDWAV, etc.) typically have input parameters for defining the breach geometry and development time. These programs expand the size of the breach from zero to the full dimensions in the specified development time using an internal algorithm.

One software program for breach development is the National Weather Service (NWS) program BREACH, which was developed by Fread (1988). This program is a physically-based breach simula-



FOR ENGINEERED EARTHEN DAMS:

$HD \leq \overline{BR} \leq 5HD$

$0.25 \leq Z \leq 1$

$0.1 \leq TFH \leq 1.0$ HOUR

FOR NON-ENGINEERED DAMS OF SLAG, REFUSE MATERIALS:

$\overline{BR} \geq 0.8 \times$ CREST LENGTH

$1 \leq Z \leq 2$

$0.1 \leq TFH \leq 0.3$ HOUR

WHERE:

$\overline{BR}$ = AVERAGE BREACH WIDTH

HD = HEIGHT OF DAM

Z = HORIZONTAL COMPONENT OF BREACH SIDE SLOPE

TFH = TIME TO FULLY FORM BREACH

(ADAPTED FROM FERC, 1993)

FIGURE 9.47 BREACH PARAMETERS

tion model, but concerns regarding the model have been raised (Wahl, 1998). The hydrograph calculated by BREACH can be used as the dam breach hydrograph by unsteady flow modeling software. Coal refuse impoundments can include massive embankment stages, such that an intermediate embankment stage configuration may represent a more critical breach geometry and time of failure than the final development configuration. Modeling a partial failure of a coal refuse dam using programs such as SMPDBK may result in a peak outflow discharge that is higher than that from breaching the full height of the dam all the way down to the natural valley bottom. Since slurry impoundments, especially upstream-construction dams, often contain a considerable amount of consolidated slurry and have a relatively small water storage volume as compared to conventional water supply, flood control, or multipurpose dams, this may be more representative of the actual consequences of a slurry dam failure. However, some state regulatory agencies require that all of the saturated fine coal refuse be treated as flowable and that a breach analysis be based upon a breach extending the full height of the embankment (from the final crest to the foundation).

9.9.4.2 Initial Conditions

Generally, the flow into an impoundment and in the channel downstream from the dam is assumed to be constant and equal to the design storm condition immediately prior to dam breach. Frequently the design storm is the PMF, which represents an extreme upper-bound inflow to the impoundment that will only rarely be approached (Section 9.5). Sometimes the design storm will be less than the PMF, but as long as it exceeds the spillway capacity, the dam could be overtopped and catastrophic failure could occur. The impoundment water surface elevation used in the dam breach model should be the minimum required for breach initiation.

For a sunny day breach, normal steady-state stream flows into the impoundment and in the downstream channel should be assumed. These can usually be obtained from published stream gage data or on-site records. Estimates of flow can be developed based upon channel dimensions and slope, estimated roughness and calculated normal flow velocity in the absence of recorded flow data. The water surface in the impoundment is typically assumed at normal pool elevation for a sunny day breach.

9.9.4.3 Flow Channel Geometry and Roughness

All dam-breach flow models require a description of the downstream channel and floodplain geometry (i.e., cross sections) and roughness. Roughness is usually defined in terms of Manning's n, and values for Manning's n are available in the literature for a wide range of conditions (Chow, 1959). Typically, out-of-channel flow encounters substantial resistance from brush, trees, debris and even dwellings, so that the Manning's *n* for the floodplain is much higher than for normal channel (USGS, 1989).

A key factor that can cause elevated flood levels is the presence of constrictions such as bridge and railroad embankment crossings or severe natural channel narrowings in the reach downstream from the dam. These can cause temporary backwater elevation and localized increased flooding. Additional cross section data (i.e., closer cross section spacing) may be needed in these constricted areas.

The presence of an existing downstream impoundment or impoundments may result in the need to extend the flow model farther downstream.

Typically for a dam breach analysis under design storm conditions, the area of interest along the channel terminates when the flow reaches a certain increment above the flow elevation without the dam breach (typically 1 to 2 feet). For a sunny day breach, the area of interest along the channel normally terminates when the flow returns from the floodplain to the natural channel or to a level associated with a specified recurrence interval flood. Thus, downstream channel data should extend past the points where these control points are anticipated to occur.

In addition to natural channels, another pathway for an impoundment breach is via breakthrough to an underground mine, with discharge through the mine and out of associated mine openings. This can lead to potential inundation in watersheds other than that in which the impoundment is located, depending on the size and extent of the mine.

9.9.5 Results of Analysis/Inundation Mapping

The output from flow model software will be water surface elevations and flow velocity at each channel cross section in the model. Additionally, the model software can provide the time of arrival of the flood wave from the breach, which is useful for the EAP development. The maximum water surface elevation at each cross section following the dam breach will define the extent of inundation. It is important to note that the maximum inundation elevation for an unsteady flow analysis does not occur at the same time at each cross section. Therefore care must be taken to record the highest level at each cross section and to use these values in determining the limits of inundation.

The flow velocity is also computed for each channel cross section, which may be helpful in assessing the potential threat to occupied structures or roadway travelers, as discussed in Section 9.5.1.3.2. It is important to note that while the average velocity in the floodplain may be provided as part of the model output, it is advisable to perform an independent computation of the flood velocity for the structure location, considering the maximum water surface elevation and energy grade line.

As discussed in the previous section, the inundation map should extend downstream from the dam to the farther of the termination points associated with a breach occurring under design storm and sunny day conditions.

Plotting the extent of inundation can be tedious if done manually because the water surface elevation is falling relative to a fixed datum and thus the extent of inundation will not match or be parallel to any ground surface elevation contours. GIS-based software can provide plots of the inundation limits as part of the normal output and thus eliminate the need for manual plotting.

9.9.6 Sensitivity Analyses

It may be useful to perform multiple dam breach analyses with variations in selected input data to evaluate the effect of the variation on the analysis results. For dam-breach analyses, the parameters associated with the breach development (i.e., dimensions and development time) are likely to be the most controversial. Since a breach analysis can directly affect the safety of downstream residents, it is prudent that conservative dam-breach analyses be performed. If, for example, substantially reducing the breach development time does not significantly alter the results in terms of inundation levels and extent of inundation, then conservative breach assumptions can be used in the analysis and may expedite regulatory review of the EAP.

9.9.7 Hazard Classification

Most commonly, a dam-breach analysis is performed as part of preparing an EAP for a dam that has already been classified as having high or significant hazard potential. Another purpose for a dam-breach analysis could be to establish the hazard classification in the first place. In this event, the FEMA (2004a) or applicable state criteria should be followed. The FEMA criteria are listed in Table 9.21.

TABLE 9.21 HAZARD POTENTIAL CLASSIFICATIONS

Hazard-Potential Classification	Loss of Human Life	Economic, Environmental, Lifeline Losses
Low	None expected	Low and generally limited to owner
Significant	None expected	Yes
High	Probable – One or more	Yes

(FEMA, 2004a)

If populated areas are impacted, particularly areas located close to the dam, a high-hazard-potential classification should normally be assigned. As discussed in Section 9.5.1.3.2, other criteria, such as depth or velocity of flow, may also be considered by designers and accepted by regulatory agencies for determining the potential significance of the inundation level and to assign the associated hazard classification. Other classifications should be consistent with Table 9.21 and the limits of inundation.