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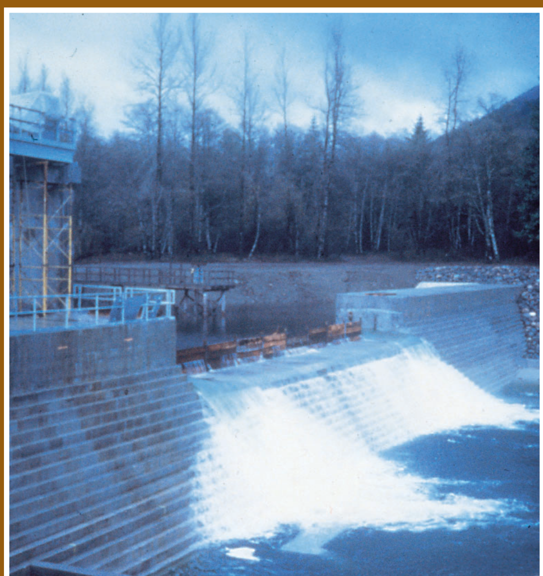
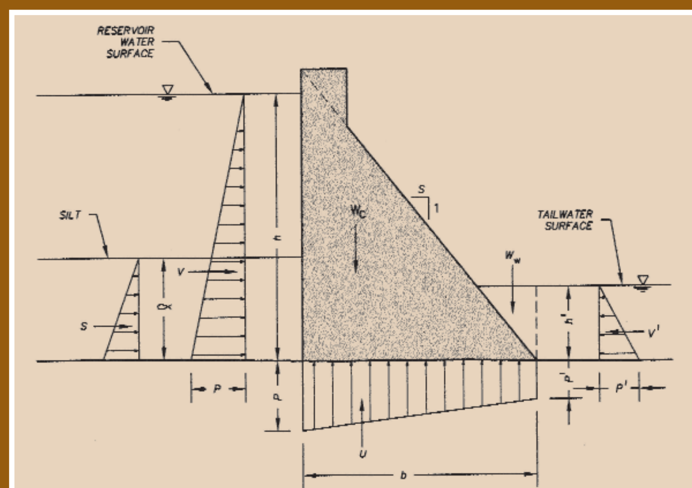
Portland Cement Association

Design Manual for Small RCC Dams

Prepared by Schnabel Engineering Associates, Inc.





Design Manual for
Small RCC Gravity Dams

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Design Manual for Small RCC Gravity Dams

PORTLAND CEMENT ASSOCIATION

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Chapter 1

Introduction

1.0 Background

A gravity dam is a solid structure so designed and shaped that its weight provides sufficient sliding and overturning resistance to ensure stability against the effects of all imposed force or load combinations. Gravity-type dams of uncemented masonry were reportedly constructed as long ago as 4000 B.C. The oldest documented masonry dam was completed in 1586 near Almanza in Spain. It was built using rubble masonry, founded on rock, and reached a maximum height of about 48 ft (14.6m). Early masonry dams used clay mortar as the binder between the individual stone blocks. Later, lime mortar was discovered and used to build masonry gravity dams.

Following its invention in 1824, Portland cement was used for the mortar in masonry dams. Concrete steadily replaced masonry for gravity dams starting in the 1880's. Mass concrete gravity dams were quite popular in the first half of the 1900's. However, by the 1950's, more and more embankment-type dams were being built on sites that could accommodate concrete dams. This was mainly because earth moving construction methods had advanced more rapidly than concrete dam construction methods. This situation led Engineering-News Record to editorialize in its March 6, 1969, issue:

The technology of mass concrete construction simply has not kept pace with the art and science of earthmoving. It is time for a study into ways of reducing the cost of concrete dams….Dams must be conservatively designed and carefully built. However, it does seem that in all the years since Hoover Dam, there should have been more change in the bucket-by-bucket method of moving mass concrete into place. What is needed is a lot more systems analysis and a bit less "grandpa-ism".

Although embankment dams cost less, they had problems. They were more prone to failure, mainly due to overtopping during a flood or to internal erosion, called piping. Concrete dams had, and continue to have, an excellent performance record, as only one concrete dam (Gillespie Dam, Arizona in 1993) in the United States is known to have failed during the past 70 years for any reason.

Figure 1.1. Great Hills Dam in Austin, TX forms an aesthetically pleasing lake for a business park development.

Both structural and geotechnical engineers were thus seeking a way to solve the problem of producing a concrete dam at less cost while maintaining its inherent safety. This is the situation that helped bring about the development of the roller compacted concrete (RCC) dam in the early 1980's. Fundamentally, it involves building a concrete gravity dam by methods usually associated with earth dam construction. Typically, 1 ft (300mm) thick layers of no-slump concrete are spread horizontally and compacted with construction quickly proceeding from abutment to abutment. The lower cost of RCC dams is primarily derived from rapid mechanized construction and reduced labor.

RCC is more of a new construction method than a new material. It is a true concrete that is usually mixed in a pugmill mixer, transported by trucks, large front end loaders or conveyor belts, spread by dozers, and compacted by smooth drum vibratory rollers.

In comparing RCC with conventional slump concrete, less water is used and consolidation is achieved externally with steel drum vibrating compactors. Because less water is used, less cement is required to produce an equivalent water/cement ratio. Less water in the mixture leads to less drying shrinkage and less cement results in less heat generation. The reduction in drying shrinkage and heat generation, in combination, reduces cracking potential. Additionally, reduced water content and vibratory roller compaction increases unit weight.

1.1 Preface and Scope

While RCC dams have been completed to heights greater than 500 ft (155m) worldwide, this manual will provide engineers with information and data to aid in the design of small RCC dams. For the purpose of this manual, small RCC dams are defined as structures up to 50 ft (15m) high except for RCC water storage dams on non-rock foundations. For the latter structures, the maximum height is further limited to gravity dams whose maximum net head (headwater minus tailwater) does not exceed 20 ft (6m).

In recognition of the limited engineering costs justified for small dams, emphasis is placed on efficiency and relatively inexpensive procedures to develop the data necessary for design of the dam. This manual is not intended for dams of large volumes where significant economies can be obtained by using more precise methods of analysis and high dams where more detailed design development is required.

Figure 1.2. North Bosque River Channel Dam diverts water for the Clifton, TX water supply system.

Figure 1.3. Reichs Ford Road Dam in Frederick, MD serves as a storm water detention dam at a local landfill.

1.2 Difference Between Conventional Concrete and RCC Gravity Dams

There is little difference between gravity dams built of conventional concrete and those constructed of RCC. RCC is a true concrete placed by a different construction method. Both types of concrete gravity dams require the same attention to foundation investigation and improvement. Also, the basic method of structural analysis is identical. However, there are a few differences introduced by the RCC construction method. These differences typically include:

(1) Potential for reduced shear and tensile strength properties at the lift joint between successive layers of RCC,
(2) A greater minimum width at the crest of the gravity section,
(3) Less ability to provide surface shaping for smaller scale features, and
(4) The possibility of exposed, unformed RCC faces (surface ravel and rough, unfinished, and uneven finished appearance).

While conventional concrete gravity dams are built in blocks that are usually 5 to 7.5 ft (1.5 to 2.3m) deep, RCC is typically placed in 1 ft (300mm) thick compacted lifts extending from one abutment to the other. In the design of a conventional concrete gravity dam, it is assumed that the strength at and across the lift joint is roughly equivalent to the intact mass of concrete. Depending on the adequacy of the lift joint preparation method, this may or may not be a valid assumption.

For RCC dams, it is fairly well accepted in most cases that the shear and tensile properties at the lift joint are less than those of the parent RCC. This reduced shear strength needs to be considered in determining the adequacy of resistance to sliding between any two layers of RCC, at the RCC/foundation rock contact, and/or RCC/dental concrete contact. Adequate shear or tensile strength at the lift line is required for RCC dams to resist all applied loads. This means that in the design of an RCC dam, there may be a greater emphasis placed on lift joint preparation including the need to increase strength properties at the lift joint using a bedding mortar (or concrete). Bedding mortars placed on lift surfaces near the upstream face of an RCC dam will also reduce seepage along lift joints. Slightly reduced RCC shear strength properties compared to conventional concrete have typically led design engineers to flatten the downstream slope of RCC dams, thereby lengthening the potential shearing surface and increasing shear resistance. In addition, some engineers prefer to incorporate flatter downstream slopes in lieu of specifying extensive lift joint preparation to ensure high shear values at the successive lift interface. Extensive lift joint preparation slows RCC production rates. Whereas, conventional concrete dams of moderate height may be adequate with a 0.7H:1.0V downstream slope, small RCC dams tend to have a 0.8H:1.0V or flatter slope.

Unless the crest width is controlled by a roadway across the dam, the width of the crest of an RCC dam tends to be greater than for a gravity section built of conventional mass concrete. The crest width is typically defined to accommodate passing construction equipment (dozer and roller) plus the thickness of the upstream and downstream facing systems. This generally translates to 16 to 20 ft (4.9 to 6.1m) as a minimum crest width.

For small RCC dams less than 20 ft (6m) high, this additional crest width will likely add excess volume and subsequent cost to the structure. The need for two equipment widths for the construction of low RCC dams may therefore have to be compromised even though it could increase the unit cost of the RCC placed near the top to the dam. Because the RCC construction method requires at least one equipment width at the crest (8 ft [2.4m]), the design engineer may want to consider a formed conventional concrete gravity section where the top width may be less than 8 ft (2.4m). The prime decision factors regarding the use of RCC versus conventional concrete for low dams are economics and aesthetics.

Where an unformed RCC downstream face is contemplated, the design engineer needs to consider both constructability and durability of the exposed slope. Without special equipment or forms, the steepest face that can reasonably be constructed using crushed aggregate in the RCC mixture is 0.8H:1.0V. From a durability standpoint, an exposed RCC slope has been shown to perform adequately where little or no seepage exits on the downstream face and in areas where there are few freeze-thaw and limited wet-dry cycles. The durability of exposed RCC can be improved with higher strength RCC mixtures and increased compaction, or by forming the exposed outside edge.

Exposed RCC, either formed or unformed, will not have the smooth finish found with conventional formed concrete. RCC surfaces, even when great care is taken by the contractor, will have some honeycombing and rock pockets; therefore, the appearance of the exposed RCC will not be as good as conventional formed concrete.

1.3 Applications for Small RCC Dams

The main purposes for which RCC has been used for low dams in the United States include water supply storage and diversion, flood control / storm-water management, recreation, irrigation, and in at least one case, to form an aesthetically pleasing lake in a business park development. This latter dam, Great Hills Dam at Austin, Texas is shown in Figure 1.1. The North Bosque River Channel Dam, used to divert water for the Clifton, Texas water supply system, is shown in Figure 1.2. Reichs Ford Road Dam in Maryland is an example of a storm water detention dam (Figure 1.3), located at a landfill.

Irrigation diversion dams are also prime candidates for a low RCC structure because of the ability of the con-

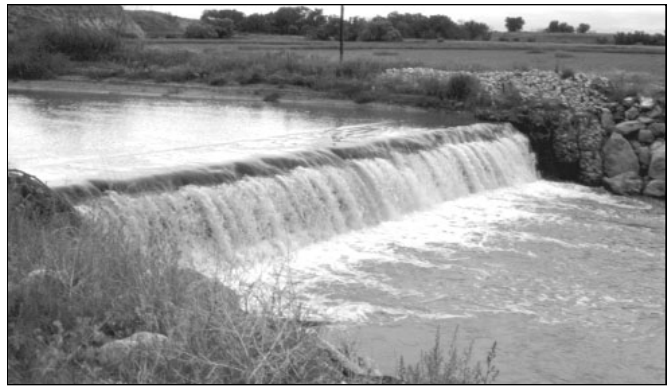


Figure 1.4. Sahara Diversion Ditch Dam near Kaycee, WY diverts water for irrigation.

Figure 1.5. Cache Creek Dam near Yolo, CA helps provide flood control for Sacramento.

crete gravity section to be continuously and safely overtopped. These structures generally have sloping upstream and downstream faces with no forming requirements. The Sahara Diversion Ditch Dam near Kaycee, WY (Figure 1.4), and Cache Creek Dam at Yolo, CA (Figure 1.5), are other examples of small RCC gravity dams.

In addition, RCC has been used to replace or remediate distressed or failed embankment dams, including structural, hydraulic, and seismic upgrades. With RCC, the owner obtains a more reliable structure that can be built quickly at a low cost. In some cases, the shorter base width of a concrete gravity section is a factor in the decision to use RCC. For cramped sites, this means that less room is needed for the dam and reservoir capacity is not diminished. This was the reason for the selection of RCC for the Atlanta Road Dam in Georgia. Due to property constraints, an earthfill embankment would take up too much space and reduce the available storage for flood control. The smaller footerprint for the RCC gravity dam fit within the available property and met Cobb County's flood detention requirements. (Figure 1.6)

Bear Creek Dam, PA was in a serious state of disrepair. This old timber crib dam needed to be replaced and the community wanted to retain the charm of the existing historic timber crib dam. The engineer developed a unique plan where both safety and aesthetics concerns could be satisfied. An RCC gravity section was designed to the basic same dimensions of the old timber crib structure, i.e., vertical downstream face and sloping upstream face. The RCC was then covered with a timber planking façade to replicate the appearance of a timber crib dam (Figures 1.7 and 1.8). Table 1.1 presents a list of RCC dams in the United States that are less than 50 ft high.

Figure 1.6. Atlanta Road Dam in Georgia stores storm water at a cramped site near Atlanta.

Figure 1.7. Bear Creek, PA, timber crib dam prior to reconstruction.

Figure 1.8. Rebuilt Bear Creek showing timber planks being installed along downstream vertical face of new RCC dam.

Table 1.1 - Roller Compacted Concrete Dams in USA (less than 50 ft high)

Dam (Completion Date)	Owner/Engineer	Max.Dimension Height ft (m)	Length ft (m)	RCC Volume cu yd (m³)	Cement & Fly Ash lb/cu yd (kg/m³)	Purpose Of Dam	RCC Unit Cost $/cu yd ($/m³)
North Loop Detention (1984) (2 dams) Austin, TX	Trammell Crow Co. Freese & Nichols	32 (10) 35 (11)	1,600 (490) 1,200 (370)	20,670 total (17,300)	200 + 80 (119 + 47)	Flood Control	$26.20 ($34.30)
Great Hills (Arboretum) (1985) Austin, TX	Trammell Crow Co. Camp Dresser & McKee	41 (12)	450 (140)	13,000 (10,700)	246 + 98 (146 + 58)	Esthetics for Business Park	$40.00 ($57.30)
Kerrville Ponding (1985) (replacement) Kerrville, TX	Upper Guadalupe River Authority Espey-Huston	22 (7)	598 (180)	22,670 (17,300)	236 + 0 (average) (140 + 0)	Water Supply and Recreation	$50.77 ($66.41)
Cedar Falls (1986) (replacement) North Bend, WA	Seattle City Light R.W. Beck & Assoc.	30 (9)	440 (130)	5,500 (4,200)	185 + 55 (110 + 33)	Water Supply – Municipal	$50.33 ($65.83)
Dryden (1986) (replacement) (2 dams) Dryden, WA	Chelan Pub. Util. District CH2M Hill	10 (3) 14 (4)	200 (61) 325 (99)	4,800 total (3,700)	400 + 100 (337 +59)	Water Supply for Apple Orchard	$60 (approx.) ($52.30)
Tellico Saddle (1989) Lenoir City, TN	Tennessee Valley Authority	11 (3)	2,000 (610)	19,500 (14,900)	250 + 190 (148 +113)	PMF Upgrade	$40 (approx.) ($57.30)
Ferris Ditch Diversion (1990) Torrington, WY	Ferris Irrig. Co. + Town of Torrington Western Water Cons.	6 (2)	260 (79)	870 (670)	350 + 0 (208 + 0)	Diversion for Irrigation	$58.13 ($76.30)
Oxide Mine #3A (1990) Leeds, AL	Cyprus-Miami Mining Woodward-Clyde (now URS)	46 (14)	278 (85)	9,000 (6,900)	320 + 0 (190 + 0)	Water Quality – Flood Control	$62.60 ($53.18)
Grace Lake Dam (1991) Leeds, AL	Birmingham Area Council Boy Scouts Southern Company	35 (11)	140 (43)	5,000 (3,800)	185 + 155 (110 + 92)	Recreation for Boy Scouts	$60(approx.) ($78.50)
Cache Creek Spillway (1992) Yolo, CA	U.S. Army Corps of Engrs. Sacramento Dist. + Tudor Engrs.	21 (6)	1,740 (530)	31,100 (23,800)	300 + 100 (178 + 59)	Flood Diversion Control	$62.60 ($81.90)
Echo Lake (1992) (replacement) South Lake Tahoe, CA	Pacific Gas & Electric (PGE) PGE + Berloger Geotech	10 (3)	340 (100)	5,000 (3,800)	250 + 0 (148 + 0)	Earthquake Upgrade	—
Faraday (1992) (part replacement) Estacada, OR	Portland General Electric Ebasco Services (Raytheon)	40 (12)	1,200 (370)	31,300 (23,900)	165 + 200 (98 + 119)	Earthquake Upgrade	—
Sahara Ditch Div. (1994) Kaycee, WY	Sussex Irrig. Co. Western Water Cons.	14 (4)	90 (27)	1,000 (760)	350 + 0 (208 + 0)	Diversion for Irrigation	$80.00 ($104.60)
Reichs Ford Rd. Div. (1994) Frederick, MD	Frederick County Chester Environmental	45 (14)	350 (110)	8,500 (6,500)	170 + 0 (110 + 0)	Storm Water Control	$60.00 ($78.50)
Prairie Creek Detention (1995) Richardson, TX	City of Richardson Carter & Burgess	20 (6)	215 (66)	4,940 (3,780)	291 + 0 (173 + 0)	Flood Detention	$70.66 ($92.42)
North Tyger River (1997) Spartenburg, SC	SJWD Water District Black & Veatch	38 (12)	500 (150)	20,000 (15,300)	165 + 165 (98 + 98)	Municipal Water Supply	$54.25 ($70.96)
Woody Branch Dam (1997)	City of Dallas, TX Powell and Powell	35 (11)	1,100 (340)	22,000 (16,800)	286 + 0 (170 + 0)	Flood Detention	$59.00 ($77.17)
Echo Lake (1997) (replacement) Dekalb County, GA	Dekalb County Jordan, Jones & Goulding	20 (6)	130 (40)	1,700 (1,300)	160 + 160 (95 + 95)	Recreation	$115.00 ($150.40)
N. Bosque River Channel Dam (1998)(replacement) Clifton, TX	City of Clifton HDR Engineering(30)	16 (5)	100	4,400 (3,400)	383 + 0 (227 + 0)	Municipal Water Supply	$96.06 ($125.65)
4th Street Low Water Dam (1999) Ft. Worth, TX	Tarrant Regional Water District Tarrant Regional Water District	24 (7)	150 (46)	11,000 (8,400)	325 + 0 (193 + 0)	Recreation	Lump Sum
Tobesofkee Dam (1999) Forsyth, GA	City of Forsyth McGill-Schnabel Engineering	25 (8)	500 (150)	6,500 (5,000)	375 + 0 (222 + 0)	Municipal Water Supply	$85.14 ($111.36)
Whipps Mill Dam (1999) Louisville, KY	Louisville and Jefferson Co. Metropolitan Water District Odgen Environmental	18 (5)	1,000 (300)	12,000 (9,200)	110 + 110 (65 + 65)	Flood Control	$91.60 ($119.60)
Atlanta Road Dam (1999) Cobb County, GA	Private Developer McGill - Schnabel	23 (7)	150 (46)	1,900 (1,450)	325 + 0 (193 + 0)	Flood Detention	$110.34 ($144.32)
Beach Street Dam (2001) Ft. Worth, TX	Tarrant Regional Water District Tarrant Regional Water District	19.5 (6)	275 (84)	11,000 (8,400)		Recreation	$64.50 ($84.40)
Stamford Dam (2001) Stamford, TX	City of Stamford Freese and Nichols	47 (14)	640 (200)	12,000 (9,200)		Water Diversion for Water Supply	$60.81 ($79.54)
Clear Lake Replacement (2002) Tulelake, CA	U.S. Bureau of Reclamation U.S. Bureau of Reclamation	42 (13)	840 (260)	18,000 (13,800)	280 + 0 (166 + 0)	Irrigation	$128.70 ($168.30)

*Unit Cost does not include mobilization cost. Prices have not been adjusted to present day costs.

Chapter 2

Preliminary Investigations

2.0 General

Preliminary site investigations are required to help determine the type of dam, provide layout and preliminary sizing of key features such as spillways and outlets, and to develop preliminary project cost estimates. It is important during the development of the preliminary investigation program to keep in mind the degree in which these issues need to be defined, and plan the investigation program accordingly.

The key issues to be defined during the preliminary phase of the project are the hydrologic and foundation conditions at the site, together with the dam's hazard category. The hazard classification will establish the framework for hydrologic studies, and the results of the hydrologic studies will establish the type and size of the spillway. The size of the spillway can weigh heavily when deciding what type of dam to select. A small capacity spillway can be economically combined with either a gravity dam or an embankment dam. A large spillway can more easily be incorporated into a gravity dam at less cost than an embankment dam and is commonly a key factor supporting the selection of an RCC dam. Typically, an embankment dam that requires a large capacity spillway requires locating a structural spillway on one abutment. This layout requires favorable topography at one of the abutments and adds significantly to cost. The elements of a hydrologic study are outside the scope and mission of this document. The remainder of this chapter will focus on site and foundation exploration.

2.1 Site Exploration

Site exploration starts with the acquisition of available information such as topographic, soil and geologic mapping, property boundary surveys, and previous engineering reports. Following review of the available information, a thorough site walk-over allows the design team an opportunity to become familiar with the site setting and to identify site development opportunities and constraints.

Important features to be identified during the site reconnaissance include access to the site, staging areas, river / stream alignment, topography, rock outcrops, river deposits, and borrow source availability. Ease of access to the site is important to the foundation and geologic investigation program. Not only will access impact both the cost and scheduling of the foundation investigation program, but also it may dictate the type of exploratory techniques used for the project. An example would be a difficult site characterized by a lack of existing roads, heavy overgrowth, and very steep abutment slopes. These circumstances might influence the engineer to increase use of geophysical methods in place of conventional drilling. The size of the river / stream may prohibit crossing and require access from each abutment.

Topographic features affecting project layout and design include steepness of abutments and valley shape. These topographic features do not preclude the construction of an RCC dam, but rather influence the method of RCC delivery from the production plant to the placement surface and layout of project features.

The alignment of the river / stream in combination with the shape of the valley will effect the layout of diversion works during construction and location of permanent outlet facilities. A broad flood plain adjacent to the stream course allows for placement of a diversion channel and the outlet works with an invert elevation similar to the stream with a minimum of excavation. A narrow flood plain may require significant excavation of the abutment to set the diversion / outlet at stream grade. The excavation may include both earth and rock. For an RCC dam, it may be advantageous to locate the conduit or conduits in a trench cut into the foundation rock or up against one of the rock abutments. In this way, RCC can be placed efficiently without interference.

Rock outcropping is a highly favorably indication that bedrock is to be found at shallow depth, and provides a direct opportunity to characterize a portion of the rock mass. At a minimum, the rock type and degree and orientation of jointing can be identified from outcropping. This type of preliminary information can strengthen correlation with geologic mapping of the area.

At this point in the investigation process, the preferred dam type is generally unknown; however, some effort should be made to identify the location, type, and approximate quantity of available earth materials and concrete ingredients that are located close to the site. Once the dam type has been established, a more detailed borrow source evaluation is warranted.

Depending on the level of information required for the site investigation, review of available mapping (geology, soils, and topography) and a site visit may be sufficient for preliminary purposes. For many preliminary investigations and screening level site assessments, this level of investigation may be sufficient. However, foundation explorations will be prudent or necessary if subsurface conditions are likely to be either difficult to interpret or are subject to significant local variations.

2.2 Foundation Investigation

The ideal foundation for an RCC dam is competent bedrock at shallow depth, but small RCC dams have been located directly on non-rock foundations. A competent foundation can be defined as one that is capable of supporting the structural loads imparted by the dam, without excessive deformation, sufficiently impermeable to preclude significant seepage under the dam, and resistant to dissolution and erosion caused by seepage. The degree of competence required for a foundation is also a function of the size of the dam. Adequate load transfer and structural support are rarely of concern for dams less than 50 ft(15m) high founded on bedrock. However, even for small gravity dams, most foundations merit some treatment to enhance performance.

The intent of a foundation exploration program is to assess the character of the foundation materials and define the type and extent of modifications required to improve the foundation to effectively provide the above noted conditions. Foundation investigation techniques for RCC dams do not materially differ from other dam types. Programs focused on concrete gravity dams, however, tend to focus more on rock characteristics because this is the common bearing strata for a gravity dam. Investigation tools commonly include soil borings, rock corings, test pits, and associated field testing. This may be supplemented by geophysical studies. Information of importance regarding the overburden soils includes gradation, statigraphy, depth, origin, and mechanical properties. Even if the dam is to bear on a rock foundation, overburden soils information is important to considerations relating to excavation, diversion, dewatering, backfilling and disposal. Important information regarding bedrock includes rock type, origin, stratigraphy, variability, jointing, and mechanical properties. Identification of groundwater depths is also very important.

If the dam is to bear on a non-rock foundation, extreme care must be taken in assessing the foundation conditions. Important information regarding a non-rock foundation includes soil type, origin, density, stratigraphy, uniformity, permeability, strength, and consolidation properties.

Often times, the foundation exploration program is conducted in phases. The initial phase is used to characterize the depth and type of foundation materials characteristic of the dam site, and to identify the location and quantity of borrow (if natural aggregates are available) or identify potential quarry sites. The scope of the initial phase can vary, but might typically include borings spaced at between 100 and 300 ft (30 and 90m) along the proposed axis. Standard penetration testing and sampling of the overburden soils should be performed. Bedrock should be cored to depths up to the proposed hydraulic head of the dam. Where rock is known to be of adequate quality, boring techniques capable of sample recovery can be partially or fully substituted with air-track borings and / or geophysical surveys.

Potential aggregate borrow areas can be investigated with both borings and geophysics, followed by conformation test pits. If both RCC and earth dam options are under consideration, additional borrow area investigation for earth fill materials should be included. However, for small volume RCC dams, concrete aggregates usually come from an established nearby quarry. Preliminary investigation findings, in conjunction with hydrologic and topographic information, are usually sufficient for a decision regarding the dam type.

Once an RCC dam is confirmed as the preferred dam type, and preliminary siting and project layouts prepared, a second phase investigation program can be developed and implemented. The second stage program will focus on characterizing both the physical and mechanical properties of the foundation materials both laterally and with depth, as well as identifying and quantifying the type and extent of improvements that may be incorporated in the design. For an RCC gravity dam founded on bedrock, the foundation investigation would include additional core borings into the bedrock at about a 50 to 200 ft (15 to 60m) spacing and in-situ rock pressure testing. Rock cores would be tested in the laboratory for physical and mechanical properties. Geophysical surveys and test pits would be performed to identify groundwater levels and rock surface contours.

For non-rock foundations, the boring spacing would be the same, between 50 and 200 ft (15-60m). Soil sampling should be continuous and sufficient quantities of samples obtained for laboratory testing. Also undisturbed samples should be recovered for testing, especially in areas where the type or consistency varies. Additional quantities of material can be obtained with test pits. Laboratory testing should include index testing as well as mechanical testing (strength, consolidation, permeability).

Unless processed aggregates are to be acquired from a commercial aggregate source, additional investigation of the intended aggregate borrow sources would also be required to verify that sufficient quantities of suitable material are available within reasonable haul distances to the dam site, and to characterize the extent of processing and/or mixing needed for these aggregate materials. Testing of proposed aggregate sources would usually include gradation, organic content, plasticity (fines fraction), specific gravity, absorption, resistance to abrasion, weathering, and chemical attack, as well as alkali-aggregate reaction potential.

As previously noted, dams requiring relatively small volumes of RCC (usually less than 5,000 to 8,000 cu yd [3,800 to 6,100 m^3]) are usually built using aggregates obtained from off-site commercial quarries. Most commercial quarries can provide gradations, specific gravity and absorption values as well as the results of durability testing. Further, information on site investigations can be found in references 1, 2, 3 and 4.

Chapter 3

Foundations

3.0 General

Because each potential dam site is unique, engineers experienced in the evaluation of foundations for dams should investigate the site and determine what, if any, foundation improvements or treatments are required to produce a properly functioning structure. The conditions from the dam/foundation contact to a depth equal to the height of the planned dam are considered to be of the greatest importance. The foundation conditions in this area have the greatest effect on the ability of the foundation to withstand short-term or long-term deformations or movements.

Foundation investigation and identification of appropriate treatments are just as important as the design of the gravity dam section itself. History has shown that the potential for failure of concrete dams is extremely remote. Nearly all failures related to concrete dams have occurred through the foundation material rather than through the concrete in the dam.

3.1 Rock Foundations

Sound rock foundations are considered most suitable for an RCC dam of any height. Rock foundations are preferred because these materials possess high bearing capacity, low settlement potential, and have a high degree of both erosion and seepage resistance.

The most desired properties obtained from a foundation rock investigation for a low RCC dam are 1) compressive strength, 2) shear strength, and 3) permeability. For higher dams or low dams with higher than usual stress conditions, values for deformation modulus and Poisson's ratio may also be desired. In addition to these critical material properties, the degree and orientation of jointing and fracturing is extremely important to the designer. Special attention should be given to identifying potential sliding planes in the foundation rock, especially those that "daylight" downstream of the dam.

Pressure grouting is commonly used to improve the performance of a rock foundation. Curtain grouting is used to reduce horizontal permeability, and therefore, seepage potential. Consolidation grouting is used to improve the bearing capacity and resistance of the foundation to the applied loads. The layout and depth of grout holes, grout proportions, and grouting procedures are a function of the type of bedrock and character of the jointing and fracturing. For additional information on grouting, see references 2, 3, 4 ,5 and 6.

The bedrock surface can be improved by the application of dental concrete. Dental concrete is used to fill voids, cracks, fissures, overhangs, and other discontinuities that would make RCC placement difficult and/or compromise the integrity of the foundation contact.

3.2 Non-Rock Foundations

Small RCC gravity dams have been successfully placed on non-rock foundations. However, design engineers need to carefully study several factors before placing a concrete dam on a low modulus rock or non-rock foundation material. The principal considerations are differential settlement of the structure, seepage, uplift pressure distribution, piping potential, and hydraulic erosion of the foundation due to spillway or conduit flows.

There are numerous solutions for non-rock foundations requiring improvement for seepage. The main potential problem can be caused by relatively high seepage gradients associated with the narrow cross-sectional geometry of a gravity dam and the inability of non-rock foundation materials to withstand this seepage gradient. Most of these solutions focus on lengthening the seepage path, causing a reduction in the seepage gradient. Design features such as upstream and downstream aprons, upstream earth berms (usually using fine grained material), cutoff walls, grout curtains, and filtered drainage systems have been used.

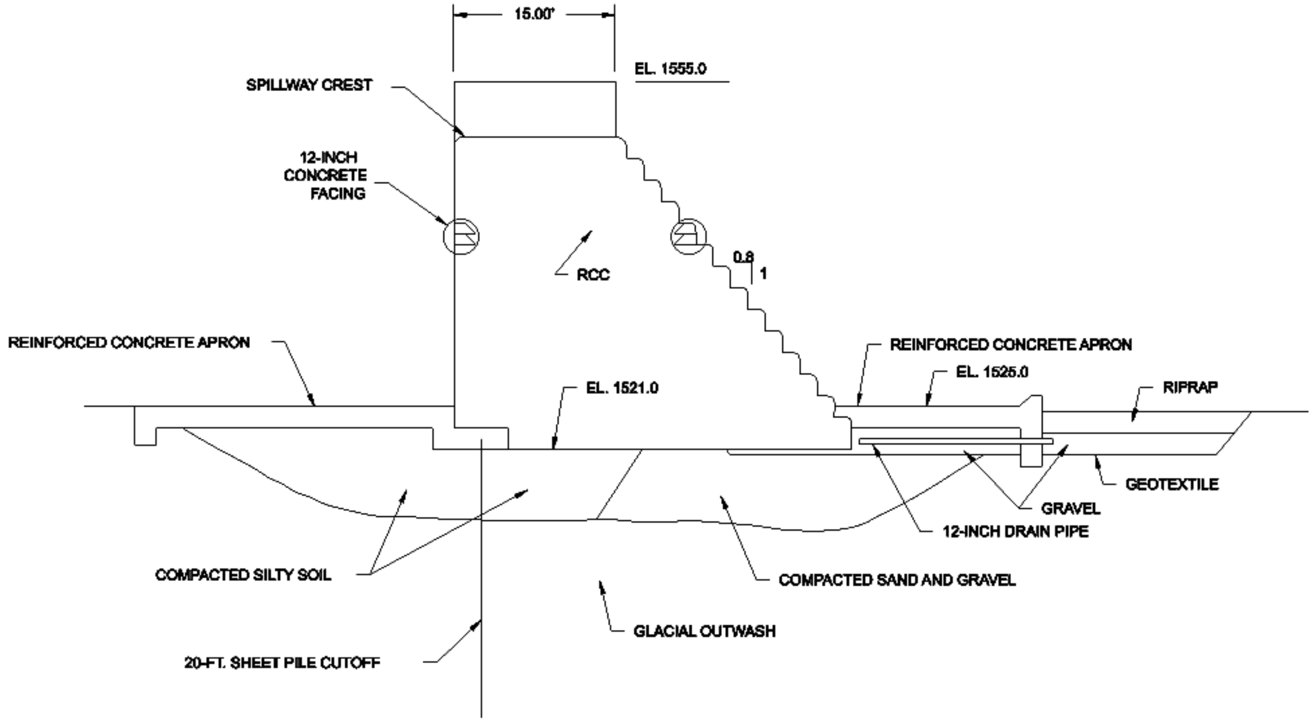


Figure 3.1. Section of Cedar Falls Dam, WA.

3.3 Non-Rock Foundation Design Examples

3.3.1 Cedar Falls Dam

The Cedar Falls project on the Cedar River southeast of Seattle, WA is a key water supply element for the City of Seattle. The project initially consisted of a 16 ft (4.9m) high rockfill timber crib dam constructed in 1902. In 1913, a 230 ft (70m) high masonry dam was constructed downstream, which caused the crib dam to be inundated at times.

The owner and its consultants determined that the old deteriorated crib dam would have to be replaced with a more durable structure, which could withstand frequent overtopping and complete submergence. A small RCC dam was determined to be the best design solution to meet operational, cost, and construction schedule requirements.

The Cedar Falls Dam is located in a valley where glacial deposits are up to 600 ft (180m) deep. It would not be practical to try to found the concrete dam on bedrock under these conditions. An earth dam could not be safely inundated and thus was not considered further. Seismic shaking also needed to be considered for the replacement structure.

Faced with constructing the replacement dam on this non-rock foundation, the design engineers incorporated a number of defensive measures to produce a safe, well-performing dam. The foundation design solution included replacing the upper most 16 ft (4.9m) of existing low-density sand with compacted fill. In this manner, they were able to reduce the potential for earthquake-induced liquefaction, decrease the seepage below the 30 ft (9.1m) high RCC dam, and reduce settlement of the structure.

Figure 3.2. Completed Cedar Falls Dam near North Bend, WA

A 20 ft (6.1m) deep vertical steel sheet-pile cutoff at the dam's upstream face and a horizontal concrete upstream blanket were provided to further reduce foundation seepage as well as lengthening the seepage path below the dam. A filter and drain system consisting of a uniformly graded gravel with a geotextile fabric was designed for placement below a downstream concrete apron to collect seepage, control uplift, and counter piping potential.

A downstream slope of 0.8H:1.0V starting at the edge of the 15 ft (4.6m) crest widened the base width and helped balance pressures at the heel and toe under the structure, reducing the potential for differential settlement. A typical design cross-section for this small RCC dam is shown in Figure 3.1. Figure 3.2 shows the completed concrete-faced RCC dam.

3.3.2 Big Haynes Dam

Big Haynes Dam, a 88 ft (29m) high RCC water supply dam is located at Rockdale County, GA. While higher than the 50 ft (15m) limit for small RCC dams, it is a good example of an RCC dam founded on a variable strength foundation. The geology at the site was such that along 500 ft (150m) of the left abutment, conditions varied from soil, with STP blow counts of 7 to 20 to hard rock. This portion of the dam was divided into three sections, called monoliths. One section was founded on hard rock, the middle section on over 30 ft (9m) of soils, and the other on partially weathered rock. Each section was analyzed as a stand-alone gravity section. The only connection between each monolith was an 80-mil thick geomembrane joint material that was heat welded to the membrane attached to the rear side of the precast concrete upstream facing panels. Figure 3.3 shows the completed Big Haynes Dam with a break in the dam's axis to accommodate differing foundation conditions.

Figure 3.3. Big Haynes Dam near Conyers, GA built in monoliths with a break in the axis.

For the sections supported on soil and partially weathered rock, a detail involving an upstream concrete cutoff wall, a concrete starter footer, the geomembrane, and a drainage trench accommodated differential settlement while still maintaining seepage control in this area.

Post construction surveys indicated that the 46 ft (14m) high monolith founded on soil settled about 2 in. (50mm) during construction. The drainage trench has been discharging 30 gpm (114 liters/min) on a continuous basis. The design called for the RCC test section to be constructed along the downstream toe of the monolith founded on soil. It was left in place to protect against ero-

Figure 3.4. Prepared soil foundation for Big Haynes Dam, GA. Note trench drain.

sion of the soil foundation due to rain runoff, or if this non-overflow section ever overtopped. Figure 3.4 shows the prepared soil foundation for the monolith section. The top of the drainage trench is visible along the downstream portion of the foundation.

Chapter 4

Design Loads

4.0 General

In the analysis and design of a concrete gravity dam, the following forces/loads are typically applied as part of various loading conditions.

- Dead load (weight of concrete, [Wc])*
- External water pressure (headwater, tailwater, and water weight, [H, H^2, and Ww])*
- Internal water pressure (uplift [U])*
- Silt and earth pressures ([S])*
- Seismic forces (earthquake [Pw_{eq} and P_{eq}])*
- Ice pressure (cold climates only)
- Wind pressure
- Wave pressure
- Temperature
- Subatmospheric negative pressure (overflow spillway sections only)
- Foundation reaction*

Considerations relating to stability of small dams are most commonly assessed using a two-dimensional analysis. For the purposes of this manual, a unit width slice of the dam will be assumed for analysis, and only the loads identified with an asterisk (*) will be discussed further and incorporated in the example problem. The remaining forces are discussed in detail in references 1, 2, 4, 7, 8, and 10. Figure 4.1 provides a typical cross section of an RCC gravity dam showing the applied loads, and approximately where they act.

4.1 Dead Load

The dead load is the weight of the structure plus any appurtenant structures, such as gates, piers, and bridges. For computational purposes, the dead load is the cross sectional area of the RCC dam section being analyzed times the unit weight. The unit weight of RCC is largely dependent on the specific gravity of the aggregates. Unit weights can range from 140 pounds per cubic foot (pcf) to 160 pcf (2240 to 2560 kg/m^3). A unit weight of 150 pcf (2400 kg/m^3) is usually assumed in preliminary analyses.

4.2 External Water Pressure

Reservoir water (headwater) and tailwater forces are the result of the hydrostatic pressures that the reservoir and the downstream tailwater apply to the dam. The magnitudes of these forces are a function of the water surface level upstream and flow conditions downstream at any given time. The maximum values correspond to flood discharges that are calculated as part of the hydrologic and hydraulic analysis.

For tailwater loads, both horizontal and vertical components acting on the sloped downstream face are considered. For the nonoverflow sections of the dam, these forces are typically calculated using the full depth of the water. The tailwater loads at the overflow (spillway) section are a function of the flow conditions over the spillway and the depth of backwater. When the depth of backwater is great compared to the depth of overflow, the tailwater forces can be computed using the full depth or nearly the full depth of tailwater. When there is significant discharge over the spillway, the impingement of high velocity spillway flows on the tailwater can produce complex pressure distributions that can reduce the effective back-pressure against the dam. To accommodate this, tailwater loads are adjusted to account for these reduced pressures. Several references including references 7 and 8 suggest using approximately 60% of the full tailwater depth for significant discharge conditions. Uplift (Section 4.3) is calculated using full depth of tailwater regardless of the cross-section and flow conditions being analyzed.

4.3 Internal Water Pressure

Uplift pressure (or the buoyancy pressure) is assumed

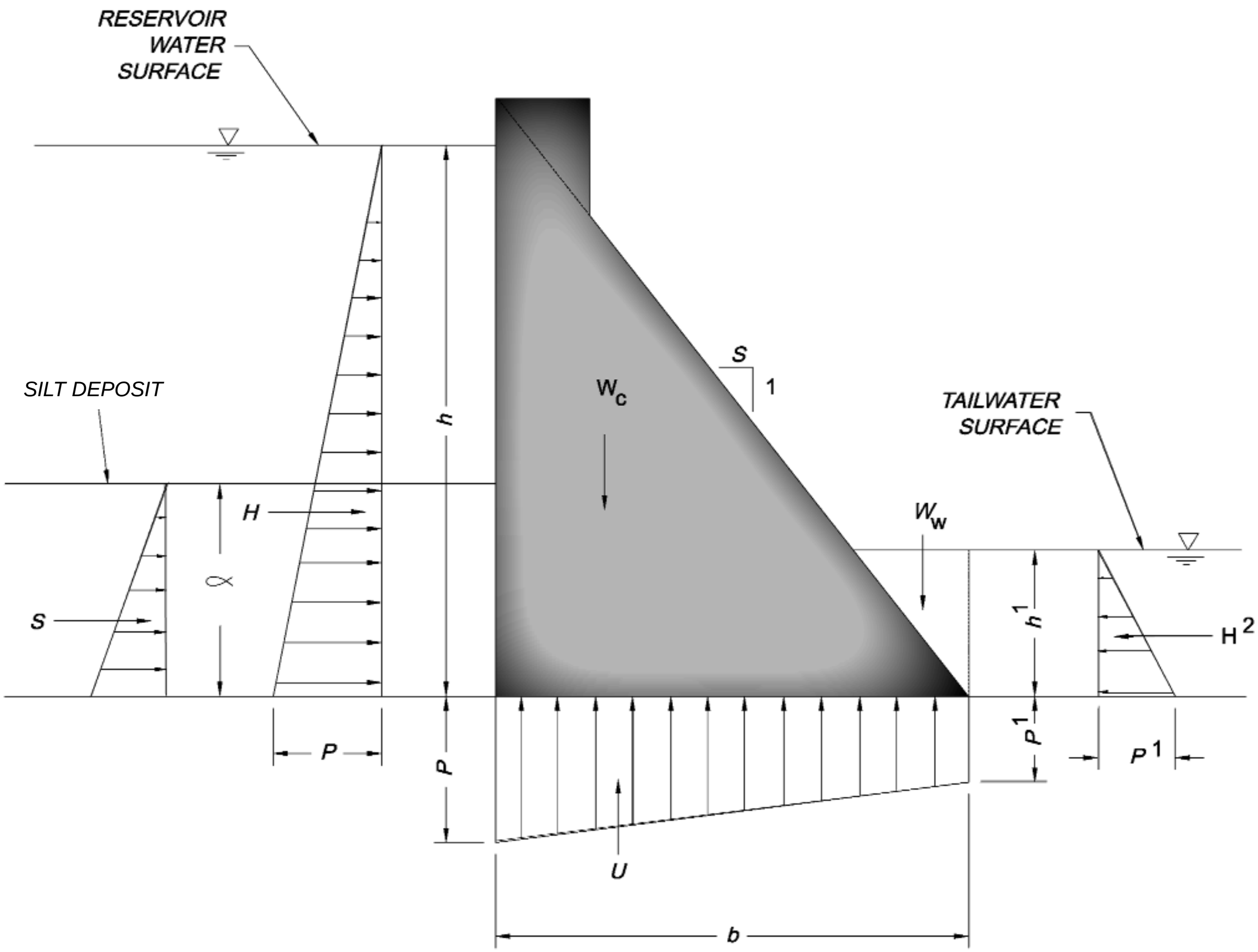


Figure 4.1. Loads on typical gravity section

to occur along any horizontal or near horizontal plane within the dam, at the dam / bedrock contact, and within the foundation bedrock. The method of RCC construction results in near-horizontal lift joints positioned usually every 12 in. (300mm) throughout the height of the structure. If there are no drains within the structure that can reasonably be depended upon to reduce internal pressures, the uplift pressure is generally taken as a linear distribution of pressure from full headwater at the heel (upstream end of the dam cross-section) to full tailwater at the toe (downstream end of the dam). If there are drains and their effectiveness can be verified, a reduction in pressure at the drain location can be provided to reduce the local internal water pressure. The amount of reduction is a function of the size and spacing of the drains and the permeability characteristics of the material being drained (references 8 and 9), and can only be quantified through instrumentation measurements.

4.4 Silt and Earth Pressures

Earth pressures will exist if sediment accumulates or soil backfill is placed against the face of the concrete dam. For locations where the soil backfill is submerged, buoyant unit weight should be used to calculate earth pressures. The at-rest earth pressure coefficient should be used. Typical values are shown in Table 4.1.

Silt pressures should be used for design if siltation is anticipated over the life of the structure. The depth of silt used in the design of a new dam should be developed based on anticipated sediment accumulation over time, tempered with experience and judgment. If appreciable, the sediment depth for an existing dam can be based on hydrographic surveys. Buoyant unit weight values for silt, deposited through natural forces, can range between 20 and 60 pcf (320 and 960 kg / m^3). Vertical silt pressure (including the effect of water) may be assumed equivalent to the pressure of a soil with a wet density of 120 pcf (1925 kg / m^3).

Table 4.1. Values of K_0—Coefficient of Earth Pressure at Rest (Reference 9)

Soil	K'_o, Effective Drained	K_{ou}, Total Undrained
Soft clay	0.6	1.0
Hard clay	0.5	0.8
Loose sand, gravel	0.6	
Dense sand, gravel	0.4	
Over consolidated clay	0.6 to > 1	
Compacted, partially saturated clay	0.4 to 0.7	

4.5 Seismic Force

For small structures, the pseudo-static earthquake analysis is appropriate and will be the only method covered in this Manual. This method, also referred to as the seismic coefficient method, is not used to assess stresses developed internally. A dynamic analysis is required to evaluate transient internal stresses caused by strong seismic loads.

The pseudo-static method is based on analyzing the earthquake load as the equivalent of a static inertial force on the dam. The load is converted into two forces: an inertial force of the dam due to its own mass, affected by the earthquake acceleration (many times limited to a horizontal inertial force only), and an inertial force of the reservoir water against the dam. Figure 4.2 presents the loads caused by an earthquake on a typical RCC gravity section. The inertial force due to the mass of the dam is assumed to act through the center of gravity of the section being analyzed. The seismic coefficient is the ratio of the peak earthquake acceleration to the acceleration of gravity. The seismic coefficient is dimensionless and is considered to be the same value for the foundation as well as the entire height of the dam section. These coefficients, which vary geographically, are summarized in Figure 4.3 from reference 8.

Using the psuedo-static method of analysis, the following equation is used to calculate the inertial force caused by the mass of the dam

$$P_{eq} = M \times a_{eq} = M \times g\alpha' = W \times \alpha' \qquad \text{(eq 4.1)}$$

where:

P_{eq} = horizontal earthquake force, $\left(\mathrm{lb/ft^2}\left[\mathrm{kN/m^2}\right]\right)$

M = mass of dam

a_{eq} = horizontal earthquake acceleration ($g \times \alpha'$), $\left(\mathrm{ft/s^2}\left[\mathrm{m/s^2}\right]\right)$

W = weight of dam, $\left(\mathrm{lb}\left[\mathrm{kN}\right]\right)$

g = acceleration due to gravity, $\left(\mathrm{ft/s^2}\left[\mathrm{m/s^2}\right]\right)$

α' = seimic coefficient (see Figure 4.3)

The inertial force on the dam due to the reservoir (in addition to the static water load) is given by the Westergaard formula using a parabolic approximation:

$$Pw_{eq} = 2/3C_{eq}\alpha' y\left(\sqrt{hy}\right) \qquad \text{(eq 4.2)}$$

where:

Pw_{eq} = additional total water load down to depth y

(when y = h, Pw_{eq} resultant force is located at 0.45h)

C_{eq} = factor depending generally on depth water and earthquake vibration period (usually taken to be 0.051 kip-sec-ft) for a vertical upstream force

h = total height of reservoir water

4.6 Foundation Reaction

To maintain static equilibrium, all of the applied forces must result in equal and opposite reactions in the foundation. The applied loads are converted to net horizontal and net vertical forces. These forces are opposed by a normal and tangential foundation reaction. The magnitude and location of the foundation reaction must result in the summation of all forces and moments being equal to zero (no unbalanced force or rotational tendency that could produce an instability).

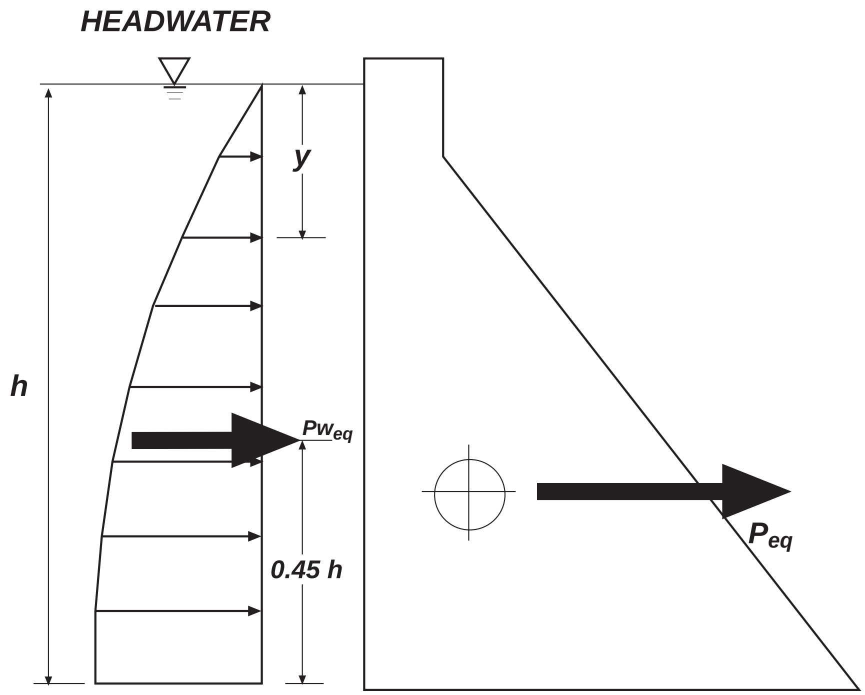


Figure 4.2. Seismically loaded gravity dam, non-overflow monolith

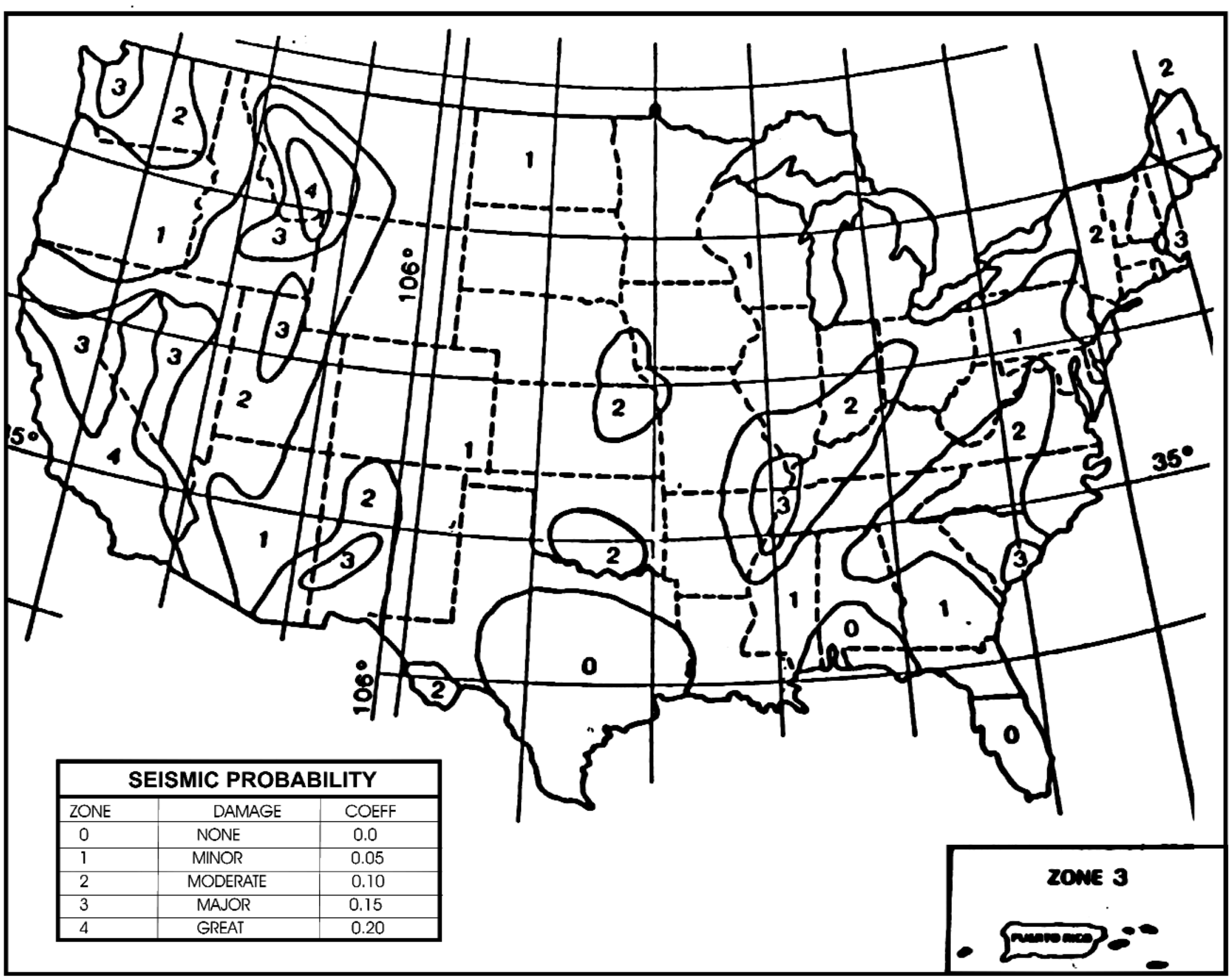


Figure 4.3. Seismic zone map of the contiguous United States and Puerto Rico

Chapter 5

Stability Analysis

5.0 General

Before a structural stability analysis can be performed on a new RCC gravity dam, a typical cross section needs to be established. The most common structural shape for a gravity dam is a truncated right triangle with the base and vertical upstream face forming the right angle. The crest is formed by truncating this triangular shape to accommodate the RCC construction process. For higher RCC dams the width at the crest is usually between 16 to 20 ft (4.9 to 6.1m), where there is no roadway across the dam crest. This dimension is needed to allow equipment to pass in two lanes. From the downstream edge of the crest, the section is taken vertically many times to intersect the downstream slope. In this manner, the volume of concrete is reduced. This crest detail is termed the "chimney" section as shown in Table 5.1 (a) and is usually only considered for dams 40 ft (12.2m) or higher.

A survey of designs actually used for small RCC dams shows that most of these structures have been designed with shapes shown as Table 5.1 (b) or (d). Table 5.1 (b) has a vertical face and the downstream slope that starts at the downstream edge of the crest. A variation of the gravity section shown in Table 5.1 (b) is Table 5.1 (c). Here, the base width is established from the upstream edge of the crest to satisfy structural stability requirements. The point where this slope intersects the foundation is then connected with the point at the downstream edge of the crest. This produces a stable section with less volume than Table 5.1 (b). However, the actual downstream slope thus produced is frequently steeper than the 0.8H:1.0V limit for an unformed RCC surface. Thus, the downstream face for this section requires forming.

For very low dams, (less than 15ft [4.5m] high) the section tends to be all RCC with both the upstream and downstream faces sloped as shown in Figure 5.1 (d). For these low dams the cost of additional RCC associated with the two sloping faces is more than offset by the savings from eliminating forming and facing materials.

Experience has shown that RCC can be built to slopes of 0.8H: 1.0V or flatter without forming or special equipment. This assumes the aggregate in the RCC is crushed. If rounded river gravel is used for the RCC coarse aggregates, the limit for building a slope without forms is about 0.9H: 1.0V. For low dams with two sloping faces 1.0H:1.0V might be a reasonable slope for construction purposes. This, then, means that for all slopes steeper than about 0.8 or 0.9H: 1.0V, forms are needed to produce the desired section. Formed downstream slopes of either RCC or conventional concrete are typically stepped. Stepped spillways are both hydraulically efficient and aesthetically pleasing. However, a formed sloping downstream face can be constructed using conventional concrete placed concurrently with the RCC.

The design of a gravity dam is performed through an interactive process involving a preliminary layout of the structure followed by a stability and stress analysis. If the structure fails to meet criteria then the layout is modified and reanalyzed. This process is repeated until an acceptable cross-section is attained.

Analysis of the stability and calculation of the stresses are generally conducted at the dam base and at selected planes (lift joints) within the structure. If weak seams or planes exist in the foundation, they should also be analyzed.

5.1 Basic Loading Conditions

The following basic loading conditions are generally used in concrete dam designs:

1. Usual loading condition—normal operating
   a. Headwater elevation at top of spillway crest
   b. Minimum tailwater
   c. Uplift
   d. Silt pressure, if applicable
   e. No ice pressure

2. Unusual loading condition—flood discharge
   a. Headwater at flood level
   b. Increased tailwater pressure
   c. Uplift
   d. Silt, if applicable
   e. No ice pressure

3. Unusual loading condition—ice load
   a. Headwater at top of spillway crest
   b. Minimum tailwater
   c. Uplift
   d. Silt, if applicable
   e. Ice pressure

4. Extreme loading condition—normal operating with earthquake
   a. Horizontal earthquake acceleration in downstream direction
   b. Usual pool elevation
   c. Minimum tailwater
   d. Uplift at pre-earthquake level
   e. Silt pressure, if applicable
   f. No ice pressure

5. Extreme loading condition—probable maximum flood
   a. Pool at probable maximum flood (PMF)
   b. Tailwater at flood elevation
   c. Uplift
   d. Silt pressure, if applicable
   e. No ice pressure

Often times the critical loading condition is Loading Condition No. 2, where the headwater level is elevated from flood discharge, but the tailwater is not equally elevated. This condition may not occur during the 100% PMF flood. Stability calculations should be performed for headwater and tailwater levels that correspond to increments of the spillway design flood, such as 20%, 40%, 60% and 80% of the PMF.

5.2 Stability Criteria

The basic stability requirements for a gravity dam for all conditions of loading are:

1. That it be safe against overturning at any horizontal plane within the structure, at its base, or at any lane within the foundation. When the resultant of all forces acting above any horizontal plane through a dam intersects that plane outside the middle third, of the upstream-downstream distance a tension zone will result. For the usual loading condition, it is generally required that the resultant force along the plane of study remain within the middle third to maintain compressive stresses in the concrete. For an unusual loading condition, the resultant must remain within the middle half of the base. For an extreme load condition, the resultant must remain sufficiently wit in the base to assure that base pressures are within the prescribed limits (see Table 5.2).

2. That it be safe against sliding on any horizontal or near-horizontal plane within the structure, at the base, or on any rock seam in the foundation.

3. That the allowable unit stresses in the concrete or in the foundation material shall not be exceeded.

Characteristic locations within the dam in which a stability criteria check should be considered include planes corresponding to dam section changes and high concentrated loads. Galleries and openings within the structure and upstream and downstream slope transitions are specific areas for consideration.

Table 5.2 and 5.3 provides criteria established by several major dam building or dam regulating agencies in the United States. While there are some differences in the criteria between the agencies, there are more similarities than differences. All the agencies agree on the three basic stability requirements noted above as well as the usual, unusual, and extreme loading conditions. The stability criteria against sliding and overstressing portions of the concrete dam or its foundation are expressed in terms of minimum factor of safety values or maximum allowable stresses. Although not exactly the same, the limiting values are generally consistent for the Corps of Engineers and Bureau of Reclamation.

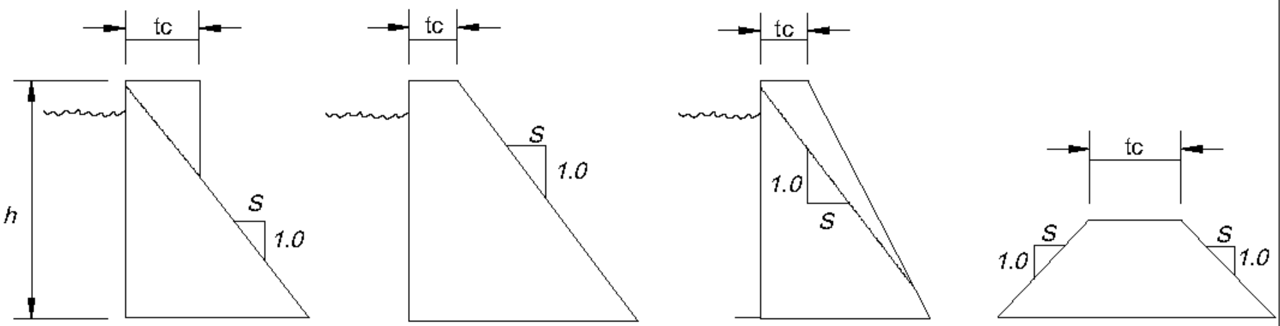
Stability criteria for both agencies include conventional equilibrium analyses and limit state theory. Both agencies evaluate stability of a concrete gravity dam using calculations for cracking potential and sliding stability. For further information on the similarities and differences between the federal agencies, see reference 11.

5.3 Stability Analyses

Gravity dam stability analyses are generally straight-forward from an engineering perspective and can readily be completed using hand computations or with the aid of a simple computer program or spreadsheet. Inaccuracies are typically related to incorrect assumptions regarding the manner in which loadings and applied moments (rotational tendencies) act.

For a two-dimensional analysis, it is convenient to analyze a 1 ft (0.3m) wide slice of the dam (unit width) and to use kips (1,000 pounds [kN]) as a unit of force, kips per square foot (ksf [kN / m^2]) for pressure, and foot-kips (kN-m) for moments. The weight of the dam and the forces acting on the dam for the unit width are first identified (see Figure 4.1), quantified, and separated into horizontal and vertical components. Horizontal and vertical forces are then separately summed to develop net horizontal (H_{NET}) and net vertical (V_{NET}) forces that act on the foundation interface.

Table 5.1. Possible Sections and Dimensions for Small RCC Dams



(a) (b) (c) (d)

Section	Usual Height Range (h)	Usual Crest Width (tc)	Usual Slope (S)	Downstream Facing Treatment
(a)	40'-50' (12-15m)	10' to 16' (3.0-4.9m)	0.8 to 0.9	Formed or Exposed
(b)	10'-50' (3-15m)	10' to 12' (3.0-3.7m)	0.8 to 0.9	Formed or Exposed
(c)	10'-50' (3-15m)	10' to 12' (3.0-3.7m)	0.7 to 0.9	Formed - less volume than (b)
(d)	5'-20' (1.5-4.6m)	8' to 10' (2.4-3.0m)	0.8 to 1.0	Usually Exposed

Table 5.2. Stability and Stress Criteria, Corps of Engineers (Reference 7)

Load Condition	Resultant Location At Base	Minimum Sliding FS	Foundation Bearing Pressure	Concrete Stress: Compressive	Concrete Stress: Tensile
Usual	Middle 1/3	2.0	allowable	$0.3f'_c$	0
Unusual	Middle 1/2	1.7	allowable	$0.5f'_c$	$0.6f'_c{}^{2/3}$
Extreme	Within base	1.3	1.33 allowable	$0.9f'_c$	$1.5f'_c{}^{2/3}$

Note: f'_c is 1-year unconfined compressive strength of concrete. The sliding factors of safety (FS) are based on a comprehensive field investigation and testing program. Concrete allowable stresses are for static loading conditions.

Table 5.3. Recommended Factors of Safety by Federal Energy Regulatory Commission (FERC) and U.S. Bureau of Reclamation (USBR) (References 8 and 10)

Recommended Factors of Safety	
Dams Having a High or Significant Hazard Potential	
Loading Condition	Sliding Factor of Safety
Usual	3.0
Unusual	2.0
Extreme	Greater than 1.0
Dams Having a Low Hazard Potential	
Loading Condition	Sliding Factor of Safety
Usual	2.0
Unusual	1.25
Extreme	Greater than 1.0

The distribution of each force is assessed and the distance from the toe of the dam (point of rotation) at which each force would act is computed. This distance is the moment arm. As an example, headwater pressure increases from zero at the water surface to a maximum at the base of the upstream face of the dam, creating a triangular distribution of pressures (Figure 4.1). For a triangular distribution, the headwater pressure acts as if the total load were applied at 1/3 of the total depth from the base.

Moments are computed for each force to arrive at the net moment (M_{NET}). This analysis is important in understanding the dam's ability to withstand overturning by the applied forces. Stabilizing moments, downward vertical forces, and upstream to downstream horizontal forces are considered to act in the positive direction (standard convention).

The factor of safety against sliding along the base is computed as follows:

$$\text{Sliding F.S.} = \frac{(V_{NET} \times f) + (B_w \times 1)(\tau)}{H_{NET}} \qquad \text{(eq 5.1)}$$

Where:

V_{NET} = Net vertical force, (lb / ft [kN / m])
f = tanØ =Shear friction factor at base (or any horizontal plane, such as a lift joint)
B_w = Base width, (ft [m])
τ = Cohesion (bond), (lb / ft^2 [kN / m^2])
H_{NET} = Net horizontal force, (lb / ft [kN / m])

The location of the resultant force (M_{NET}/ V_{NET}) measured from the mid-point of the base is called the eccentricity. Eccentricity is calculated as follows:

$$e = \frac{B_w}{2} - \frac{M_{NET}}{V_{NET}} \qquad \text{(eq 5.2)}$$

Where:

e = Eccentricity, (ft [m])
M_{NET} = Net Moment, (ft-lb / ft [N-m / m])
V_{NET} = Net Vertical Force, (lb / ft [kN / m])

The foundation pressure at the toe and heel is computed as follows:

$$P_{TOE} = \frac{V_{NET}}{B_w}\left(1 + 6\left(\frac{e}{B_w}\right)\right) \qquad \text{(eq 5.3)}$$

$$P_{HEEL} = \frac{V_{NET}}{B_w}\left(1 - 6\left(\frac{e}{B_w}\right)\right) \qquad \text{(eq 5.4)}$$

See the example problem in Appendix A for use of these equations.

An additional consideration is where foundation reaction pressures are negative (in tension). Concrete can develop high compressive strengths in relation to strengths required. However, concrete is notably weak in tension. Moreover, adhesion of concrete to the rock foundation is questionable, and fractured rock directly beneath many dams cannot support nor adequately transfer tensile stresses to deeper portions of the foundation. Therefore, unless site specific circumstances can be interpreted to support tensile stresses, it is assumed that the base of the dam will crack where tensile stresses exist. When analyzing a cracked base for a dam, it also is conventionally assumed that full headwater pressure will penetrate to the full depth of the crack. This requires an adjusted model to assess performance (a cracked section analysis). Cracked section analyses inevitably result in decreased factors of safety and an increase tendency towards overturning (rotation of the dam); because uplift will be increased (reducing both net vertical forces and net moment) and shear resistance will be available only from the not cracked portion of the base.

Generally, in the design of new small dams, the cross section is proportioned such that the foundation reactions remain positive (in compression) for all loading conditions. No tension is allowed. For information on analysis of a concrete gravity section with a cracked base refer to reference 10.

Chapter 6

Provision for Spillways

6.0 Introduction

When comparing the relative merits of alternative dam types, the presence of rock at shallow depth and the need for significant spillway capacity are the most commonly noted factors supporting the selection of an RCC dam. Large spillways for earth dams typically add a significant amount to the project budget in excess of the cost of the impounding structure. Where large spillway capacity is needed, an earth dam usually requires a separate structural spillway. These structural spillways are most commonly cut into one or both abutments, with reinforced concrete chutes used to safely convey flow to a point beyond the toe of the dam. Although some sites have been designed with reinforced concrete chute spillways over the embankment, this approach is technically demanding and does not often save on cost. In this latter case, the embankment is first built to full height and then, following settlement, excavated to the extent needed for spillway construction.

The provision for incorporating significant spillway capacity in the central portion of an RCC dam typically adds only nominally to the project's cost, in terms of:

- added forming requirements for the spillway sides and possibly for steps on the downstream face
- separation of the structure into two placement areas near the top of the dam
- some conventional concrete for the spillway control section and possibly the chute portion and stilling basin
- the cost of concrete or RCC for the spillway's side training walls

6.1 Basic Spillway Crest Control Considerations

Once the needed spillway capacity is developed, the size of spillway can be easily computed in terms of the spillway width and depth of flow, using the weir equation $Q = C\ 3\ L\ 3\ H^{3/2}$, where:

Q = Desired spillway capacity
C = Spillway discharge coefficient
L = Length of spillway perpendicular to the flow direction
H = Difference from the maximum reservoir water surface elevation to the spillway crest elevation (head)

For detailed, specific guidance on spillway design considerations, the reader is directed to references 2 and 12. The guidance presented here is sufficient for comparative assessments at the conceptual design level. Depending upon project specific cost and performance trade-offs, the spillway control section (with the spillway discharge coefficient noted) can be either :

- broad crested – a horizontal or near-horizontal crest where the width in the direction of flow is greater than the head (C = 2.6 to 2.8)
- sharp crested – a sloped or raised control section where the width in the direction of flow is less than one-half of the head (C = 3.1 to 3.4)
- ogee – a highly efficient, streamlined crest shape requiring special forming (C = 3.9)

The sharp-crested weir configuration may be a good solution because it does not cost significantly more than the broad-crested weir while having better hydraulic performance characteristics. If capacity and project appearance are not major issues, a final lift of RCC can be used as the spillway control section to produce a broad-crested weir. Only where aesthetic appearance or maximum hydraulic efficiency are highly desired does an ogee spillway provide value, due to the added cost of forming and mass concrete placement for this complex precise shape.

Figure 6.1 shows an ogee crest weir being constructed for Tellico Saddle Dam, Tennessee. Figure 6.2 shows the completed the North Tyger River Dam at

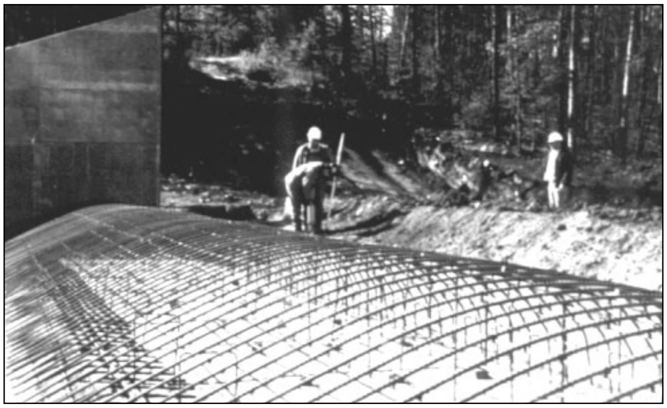


Figure 6.1. Ogee crest being constructed for Tellico Saddle Dam, TN

Figure 6.2. Cast-in-place concrete ogee crest and precast downstream steps for North Tyger River Dam at Spartanburg, SC

Spartanburg, South Carolina with its cast-in-place concrete ogee crest and precast concrete downstream steps. For Whipps Mill Dam at Louisville, Kentucky, shown in Figure 6.3, a sharped crest weir was used along with energy dissipation concrete blocks at the crest. A broad-crested weir was used for the 4th St. Low Water Dam at Ft. Worth, Texas (Figure 6.4) along with a series of corrugated metal conduits at the crest to accommodate low over flow conditions.

Additionally, where an RCC dam is constructed on a competent durable rock foundation, consideration can be given to providing spillway capacity sufficient for passage of less than the total required spillway design flood. Allowances for overtopping of the entire dam need to be based on acceptable durations, depths, and frequencies of overtopping during extreme flood conditions. It is imperative that the stability of the dam not be compromised during overtopping. Therefore, the potential for erosion of soil backfill, erosion undermining of rock, and other conditions that can be instigated by flow over non-spillway portions of the dam be appropriately analyzed if overtopping the entire dam is considered.

6.2 Spillway Chute and Training Walls

Forming of steps as a part of RCC placement is neither difficult nor costly for most applications. Steps also provide discrete roughness elements that tend to reduce flow acceleration and speed air entrainment due to the formation of pockets of turbulence at each step. Spillway steps have been constructed using exposed RCC, cast-in-place concrete, and pre-cast concrete elements (reference 13). Forming a smooth conventional concrete chute is more costly than forming steps; therefore, steps are a common and recommended approach for design of chutes for small RCC dams.

Figure 6.3. Sharped crested weir and concrete energy dissipation blocks at the crest of Whipps Mill Dam at Louisville , KY

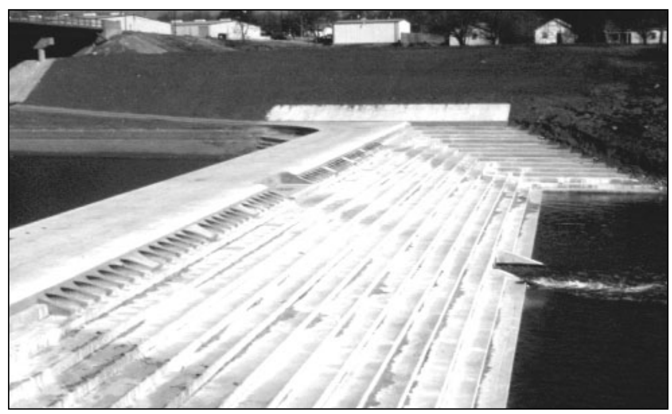


Figure 6.4. Broad crested weir and series of corrugated metal conduits at the crest of the 4th Street Low Water Dam at Ft. Worth, TX

The extent of energy loss is complex and is the subject of continuing research; however, it is clear that for a given project setting, the residual energy at the toe of a dam increases with increasing head, decreases with increasing step size, and decreases with decreased chute slope. For most small RCC dams, a 2 ft high (0.6m) step size is recommended, with 1 ft high (0.3m) steps generally satisfactory for dams less than 15 to 20 ft high (4.6 to 6.1m).

For small RCC gravity dams, the ratio H_{dam}/H_{max} can be used as shown in the following equations adapted from reference 14 to estimate residual energy at the toe of the dam for stilling basin design:

H_{dam}/H_{max} from 1 to 10,

$$\text{Residual energy} = (1 - 0.06 \times H_{dam}/H_{max}) \times \text{Total energy}$$

H_{dam}/H_{max} from 10 to 20,

$$\text{Residual energy} = (0.5 - 0.01 \times H_{dam}/H_{max}) \times \text{Total energy}$$

Where:

H_{dam} = Dam height from spillway crest to downstream channel

H_{max} = Maximum spillway head

These relations are approximate and are not recommended to be applied where energy dissipation considerations are critical to project performance. Terminal energy dissipation needs and considerations are beyond the scope of this document. See reference 2 for information on stilling basin design.

For smaller RCC dams, it is sometimes expedient to design both overflow and non-overflow portions of the dam to the same cross-sectional geometry, with the spillway RCC lift placement terminated early to form a depressed overflow area. Because steps dissipate energy, flow does not accelerate as much as occurs on a smooth chute. This results in greater depths of flow. Also, stepped spillways tend to bulk flow through air entrainment, leading to increased depths. Training walls, where needed, should be designed to sufficient height to contain and direct flood flows into the stilling basin, apron, or downstream channel (Figure 6.5). Guidance on training wall height is included in the paper by Boes (reference 14). Training walls can be eliminated in low head applications and circumstances where lateral spillage on to adjacent portions of the RCC dam would not pose an erosion or energy dissipation problem. Figure 6.6 shows the Hudson River Dam No. 11 where no training walls were used. The stilling basin was wider than the emergency spillway width.

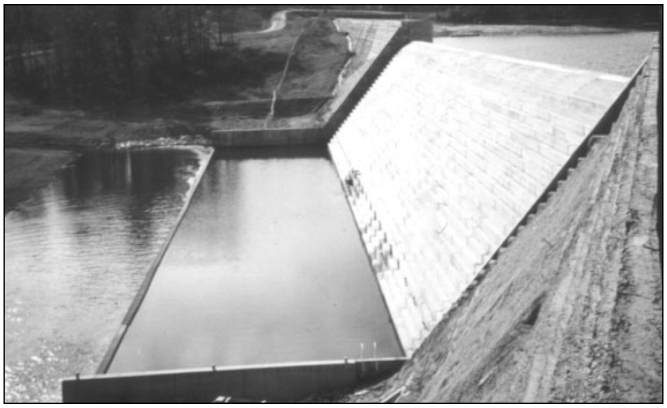


Figure 6.5. Big Haynes Dam, GA, showing training walls to contain and direct flows over the spillway and into the stilling basin.

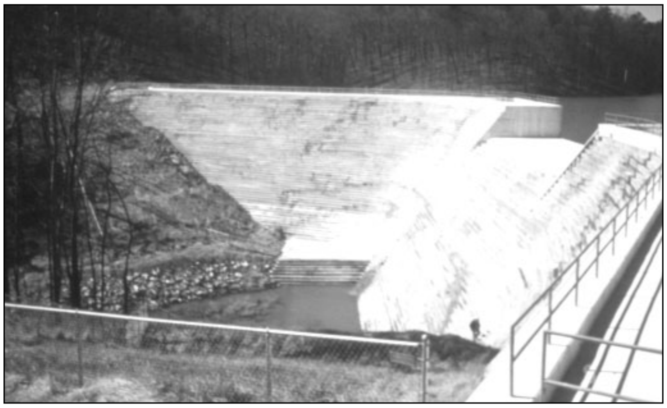


Figure 6.6. Hudson River Dam No. 11, GA, showing spillway with no training walls.

Chapter 7

Design Issues and Details

7.0 Foundation Improvement

The extent to which foundation improvement or control measures need to be incorporated into the overall dam design depends primarily on the stratification, strength, and permeability of the foundation material. A very tight rock without excessive shear zones or joints may require little or no foundation improvement.

Also, the function of the dam needs to be taken into account. For a usually dry flood detention dam, seepage control through the dam and its foundation is not as critical as for a water supply dam that operates with a full or near full reservoir over extended periods of time.

The usual methods of rock foundation improvement include consolidation grouting, curtain grouting, drainage, and replacement of weak zones and discontinuities with dental concrete in or, some cases, RCC (see Chapter 3).

For non-rock or low modulus rock foundations, additional improvement measures need to be considered. For example, loose foundation material may be replaced with compacted fill or possibly an RCC foundation mat. Also to reduce seepage and lengthen its path under the RCC structure an upstream blanket, and/or a downstream apron and a cutoff wall with a seepage collection system such as a trench drain have been used.

Any downstream apron needs to be of sufficient length to contain the hydraulic jump during overflow conditions, otherwise, a scour hole could develop downstream of the apron. In some circumstances apron undermining can lead to piping under the dam or undermining of the dam's foundation. Where critical hydraulic jump conditions are not fully contained within an apron or stilling basin for short duration extreme flood conditions, downstream scour protection is needed to protect the structure from undermining. Also, drainage should be provided below an apron to reduce uplift forces. For low RCC dams steel sheet piling or some type of concrete wall may be used as a cutoff in non-rock foundations. See Section 3.3 for the foundation improvements designed for Cedar Falls Dam in Washington, founded on a glacially deposited sandy-gravel material.

For rock foundations, any loose material that may reduce the bond between the concrete and rock should be removed. Low pressure grouting of the foundation can help fill naturally occurring voids, fracture zones, and cracks in the rock. Dental concrete or RCC are commonly used to fill large volume voids in the foundation rock.

The design engineer should determine if consolidation grouting on a regular pattern or a grout curtain is required for the dam depending on the quality of the rock and purpose of the dam. If a grout curtain is deemed advisable, drain holes downstream of a grout cutoff wall may also be needed.

Because low RCC dams will invariably be designed without a gallery within the dam, the location for drilling grout and drain holes needs further consideration. Grout holes may be drilled vertically or near vertical from a concrete toe block placed immediately upstream of the dam's upstream face. Drain holes, if required to collect seepage that may bypass the grout curtain and reduce uplift forces can be drilled from a location near or from the downstream toe and sloped towards the upstream face. Figure 7.1 shows this method of seepage drain holes at Big Haynes Dam in Georgia. The steel pipe relief wells on this project were grouted within the RCC section with drain holes open within the rock foundation.

Figure 7.1. Series of steel pipe relief wells along the downstream toe of Big Haynes Dam, GA

Table 7.1 Facing Systems for Small RCC Dams

Dam (Completion Date)	Upstream Face**	Downstream Face
North Loop Detention (1984) Austin, Texas	earthfill-RCC 1.2V:1H	precast panels to form planter boxes-stepped
Great Hills (Arboretum) (1985) Austin, Texas	precast panels w/planter	vertical-precast panels-for planter
Kerrville Ponding (1985) replacement Kerrvile, Texas	RCC against earthfill with reverse slope	exposed RCC 1.0H:1.0V
Cedar Falls (1989) (replacement) North Bend, Washington	conventional concrete	0.8H:1.0V conventional conc steps
Dryden (1989) (replacement) Dryden, Washington	exposed conc. against earthfill	1.0H:1.0V although designed 0.8H:1.0V exposed RCC
Telico Saddle (1989) Lenoir City, Tennessee	exposed RCC w/conv conc. ogee cap	exposed RCC 1.0H:1.0V
Ferris Ditch Diversion (1990) Torrington, Wyoming	exposed RCC w/1.0H:1.0V	exposed RCC 1.0H:1.0V
Oxhidemine #3A Claypool, Arizona	membrane over conv. conc.	exposed RCC 08H:10V
Grace Lake (1991) Leeds, Alabama	unformed RCC	unformed RCC 1H:1Y
Cache Creek Spillway (1992) Yolo, California	exposed RCC 0.8H:1.0V	exposed RCC-stepped 0.8H:1.0V
Echo Lake (1992) (replacement) So. Lake Tahoe, California		
Faraday (1992) (part replacement) Estacada, Oregon	conventional concrete	chimney-conc. conc+ exposed RCC 0.8H:1.0V
Sahara Ditch Div. (1994) Kaycee, Wyoming	exposed RCC 0.8H:1.0V	exposed RCC 0.8H:1.0V
Reichs Ford Rd. Div. (1994) Frederick, Maryland	precast conc. panels	exposed RCC
Prairie Creek Detention (1995) Richardson, Texas	exposed RCC steps 1.0H:1.0V	exposed RCC steps 1.75H:1.0V
th Tyger River (1997) Spartenburg, South Carolina	precast panels w/membrane	precast conv. conc. Steps 0.8H:1.0V
Woody Branch (1997) Dallas, Texas	exposed RCC against earthfill	exposed RCC
Echo Lake (1997) (replacement) Dekalb County, Georgia	membrane	top two steps conventional concrete formed RCC below
Bosque Channel (1998) (replacement) Clifton, Texas	exposed RCC 0.75H:1.0V	exposed RCC steps 1.75H:1.0V
4th Street Low Water Dam (1999) Ft. Worth, Texas	exposed RCC steps 1.54H:1.0V	exposed RCC steps 3.0H:1.0V
Tobesofkee (replacement) (1999) Forsyth, Georgia	RCC against earthfill	exposed RCC steps 0.75H:1.0V
Whipps Mill (1999) Louisville, Kentucky	conv conc. 0.8H:1.0V	RCC @ 0.8H:1.0V covered with grassed earth @ 3H:1V
Atlanta Road (1999) Cobb County, Georgia	exposed RCC w/ some GE-RCC	exposed RCC steps 0.8H:1.0V
Beach Street (2001) Ft. Worth, Texas	exposed RCC 1.5:1	exposed RCC 3.0H:1.0V
Stamford (2001) Stamford, Texas	vertical exposed RCC	1.0H:1.0H exposed RCC Steps
Clear Lake (replacement) (2001) Tulelake, California	Conventional Concrete	exposed RCC steps 0.67H:1.0V

** vertical face unless noted

7.1 Facing Systems

7.1.1 General

For many RCC dams the predominant factors that affect selection of a facing system include cost, seepage control, durability, constructability, and appearance. The evaluation of these factors leads to the use of facing systems for the RCC mass of conventional cast-in-place concrete, precast concrete panels, either plain or membrane faced, or formed RCC (Table 7.1 also reference 15).

Seepage control through the structure takes on a higher degree of importance for water storage dams than usually dry flood detention dams or low diversion dams that are overtopped on a continuous or nearly continuous basis. Durability also takes on a higher degree of importance at locations where many freeze-thaw or wet-dry cycles may be expected. Constructability is important to the degree that the facing system installation affects the project's construction schedule. The importance of appearance as a major factor in selecting a facing system depends on the dam's location and the owner's desires. Appearance becomes less of a factor for a dam not visible to the public or for a dam impounding a usually full reservoir where little of the upstream face is visible.

Cost is nearly always an important factor in determining whether a small RCC dam will have some type of concrete face or the RCC will be left exposed. As the dam height becomes less, the cost of an applied facing becomes a greater percentage of the total cost of the structure. Therefore, the desire to reduce the overall cost of the structure has led design engineers to use several variations of exposed RCC for the upstream face of low RCC dams.

7.1.2 Upstream Facing Systems

7.1.2.1 Exposed RCC Faces—When the RCC is to be exposed to water and weather, other factors need to be considered, including mixture proportions to increase durability and reduce permeability of the RCC itself, as well as measures to reduce seepage at the lift joints.

Figure 7.2. Unformed RCC face for Sahara Ditch Creek Diversion Dam, WY.

Many very low RCC dams have slopes on both faces. For these designs where the slopes are typically 0.8H: 1.0V or flatter (usually 1.0H:1.0V), the key decision for the design engineer is whether to do nothing at the upstream outer edge, compact it roughly parallel to the slope, or form the RCC lifts, usually in 1.0 ft (0.3m) high steps. As these treatments increase in cost, so does the RCC's durability and appearance.

Figure 7.3. Formed RCC steps for Prairie Creek Detention Dam, TX

If the RCC is to be uncompacted and therefore of low density at the outer edge, it may be reasonable to assume that some RCC will erode with time. The design engineer should verify that the loss of some surface RCC (considered sacrificial) on the outside of the mass will not adversely affect stability. Figure 7.2 shows an uncompacted outer edge for the Sahara Ditch Diversion Dam in Wyoming. Formed 1 ft (0.3m) high RCC steps were used for both the downstream (shown in Figure 7.3) and the upstream slope of the Prairie Creek Flood Detention Dam located in a park at Richardson, Texas.

Figure 7.4. Compacted clay berm used to help form and decrease seepage for Tobesofkee Creek Replacement Dam, GA (lake in drawdown condition)

For RCC sections designed with a vertical or near vertical face the RCC can be formed in the conventional manner with forms or an earth berm can serve for both forming and seepage reduction. Where an earth berm is used, the earth and RCC are placed concurrently in the usual 1 ft (0.3m) thick lifts and compacted. It is typical to

Figure 7.5 Formed RCC vertical face with some grout-enriched RCC for a portion of Atlanta Road Dam, GA

place the RCC first followed by the earth fill to prevent contamination of the RCC lift surface. If clay or other low permeability soil is available at the site, the compacted earth berm also improves seepage control for the small RCC dam. A clay berm was used to help form and provide increased water tightness for the Tobesofkee Creek Replacement Dam (see Figure 7.4).

An exposed RCC face that included the first test area of grout-enriched RCC in the U.S. was used for the

Figure 7.6. Grassed earth berm on upstream face of Woody Branch Detention Dam, TX

Atlanta Road Dam, as shown in Figure 7.5. The dam is an on-site stormwater detention structure for a mixed-use land development. A grassed berm was used for forming the upstream face and seepage control aid at Woody Branch Flood Detention Dam at Dallas, Texas as shown in Figure 7.6.

To improve durability and permeability of exposed RCC, modifications to mixture proportions are required from mixes used for higher RCC dams capped with more watertight concrete or upstream membrane faces. These modifications include increased cement content, smaller maximum size aggregate, and higher sand contents. See Chapter 8 for a further discussion on RCC mixture proportions.

Also, for exposed RCC sections subjected to overtopping, a full surface mortar bedding is generally used

Figure 7.7. Membrane-faced precast concrete panels form upstream vertical face of North Tyger River, SC water storage dam

between the uppermost two or three lift joints. This improves bond (cohesion) in this area where there is little weight above the lift-line to provide adequate shear-friction resistance.

7.1.2.2 Concrete Facing Systems—Facing systems of either cast-in-place concrete or precast concrete tend to increase in cost-effectiveness as height increases. Conventional concrete facings are beneficial for those projects that must retain a full or nearly full water storage reservoir. Precast concrete panels may be either plain or membrane faced (downstream side of the panel). Figure 7.7 shows membrane faced precast concrete panels being used for the North Tyger River Water Storage Reservoir at Spartanburg, South Carolina. Where plain precast panels are used without a membrane, the panels act as forms and also provide a durable, attractive upstream face. In this case, the RCC behind the panels acts as the main water barrier, as the joints between panels are generally not sealed.

Figure 7.8. Monksville Dam, NJ, shows where RCC was placed first followed by conventional concrete to form upstream vertical face

Where a cast-in-place concrete face is used, it is most often placed concurrently with the RCC in 1 ft (0.3m) thick layers. The interface between the slump concrete and the RCC needs to be immersion vibrated to enhance bond. Usually, the conventional concrete is placed first, followed immediately by the RCC; however, this placement order can be reversed if considerable care and effort are taken to ensure that a well-consolidated union is produced between the two similar materials with differing consistencies (see Figure 7.8). A set retarding, water reducing admixture is generally added to the cast-in-place concrete to help promote a well-bonded interface between the conventional concrete face and the RCC.

If the cast-in-place concrete is placed first, its width at the top of the lift can be as little as 1 ft (0.3m). The section then slopes at an angle approximating the angle of repose of the material, usually 45 degrees (1.0H: 1.0V), producing an average thickness for the concrete facing of 1.5 ft (0.46m).

A more detailed discussion on upstream facing systems, mainly for higher dams, is contained in reference 15.

7.1.2.3 Grout-Enriched RCC (GE-RCC) Faces— GE-RCC is a relatively new development that appears to have considerable promise when used as a facing for small RCC dams. It is more of a process than a product and consists of adding grout to uncompacted RCC adjacent to a formed face, and then vibrating the grout and RCC together. The grout mix is proportioned such that when it is blended into the RCC mix with immersion vibrators, an insitu conventional concrete face is produced.

The GE-RCC method produces the equivalent of a concrete face for an RCC dam at considerably less cost than the traditional method of mixing and placing each material separately and then vibrating them together. A thorough understanding of RCC and grout mixture proportions and placement is necessary to produce a homogenous concrete face.

GE-RCC faces are still in the development stage in the U.S.; however, based on the experience in China and other countries, the keys to producing an acceptable GE-RCC face are:

1. The RCC needs to have a wet consistency as indcated by a low Vebe time. This is usually 20 seconds or less, but it has reportedly worked with a 25 to 30 second Vebe time.
2. The grout needs to be relatively lean. This W/C is usually 1.0 (1 part water, 1 part cement by weight). With a super plasticizer admixture this can be reduced to 0.8. The grout should be allowed to permeate the RCC for a short time before applying vibration to consolidate the two materials together.
3. The volume of grout applied atop the loose RCC needs to be metered. Care must be taken not to overdose the RCC with grout. About 1/2 gal/yd^2 of grout is suggested for a 16 in.width. (For a 0.4m width, this equates to 8L/m.)

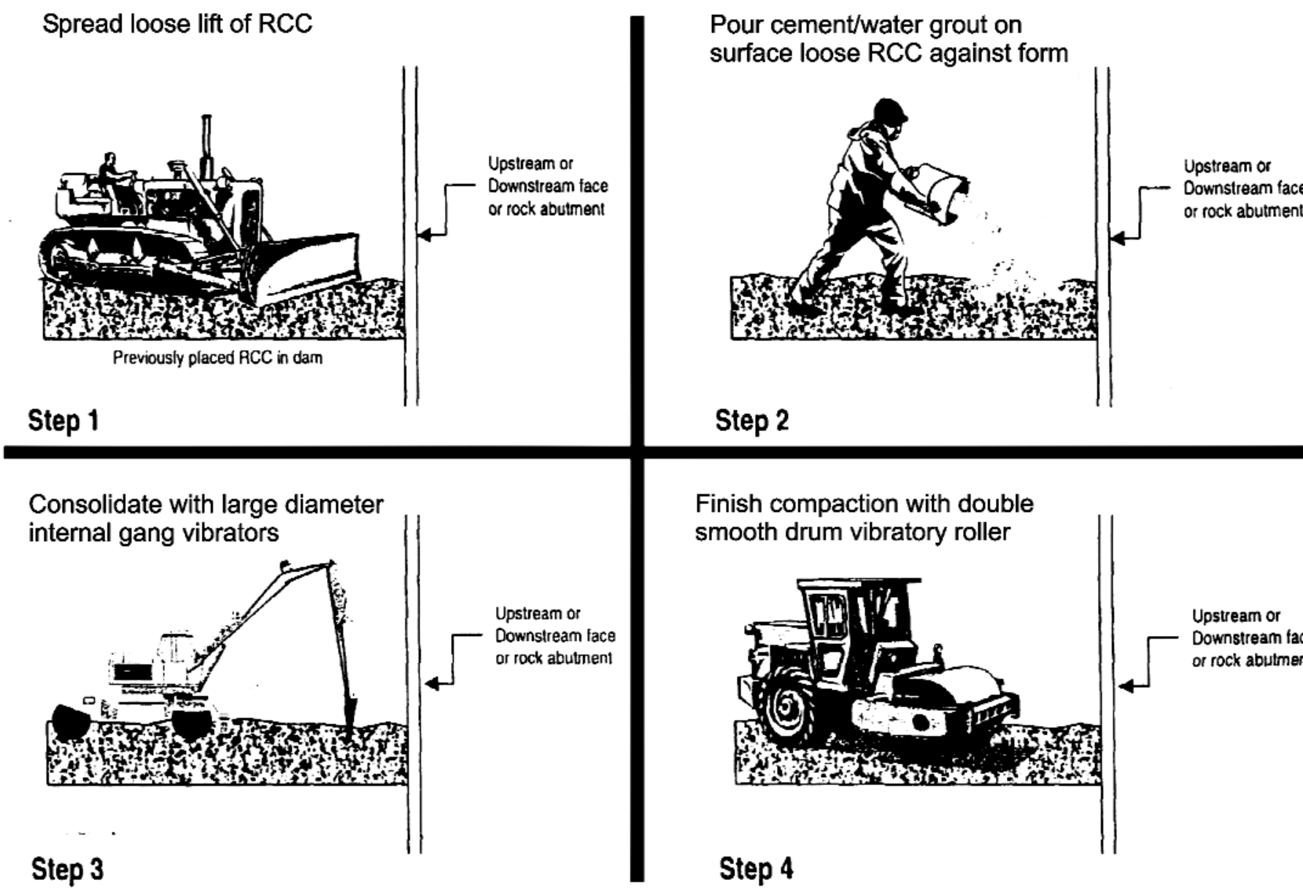


Figure 7.9 Grout Enriched RCC Method (Courtesy of William Moler)

Figure 7.10 . Two-foot (0.6m) high exposed RCC steps for downstream face of Tobesofkee Creek Replacement Dam,

Figure 7.11. Slip-form attached to tamping paver used to form 1-ft (0.3m) high steps for Woody Branch Detention Dam, TX

Figure 7.12. Completed stepped downstream face for Woody Branch Detention Dam, TX

Before the contractor is allowed to place GE-RCC for the upstream face of a dam, a test section should be built separate from or at a non-critical surface on the downstream face of the dam. In this manner, the engineer can evaluate the contractor's construction technique and the appearance of the GE-RCC face, and make final adjustments to the materials and processes to enhance production and performance levels. GE-RCC can be used for either or both faces of an RCC dam. The steps involved in producing a GE-RCC face are shown in Figure 7.9.

Figure 7.13. Uncompacted RCC downstream face for Kerrville Ponding Dam, TX following overtopping

Figure 7.14. Kerrville Ponding Dam, TX being overtopped by 16.2 ft (4.9m) in 1987

Figure 7.15. Shows operation of RCC spillway for Lake Grace Dam, AL

7.1.3 Downstream Faces

Downstream faces for small RCC dams can be either exposed RCC or conventional concrete. Because of cost considerations, the downstream faces for many small RCC dams have been designed using exposed RCC. The exposed RCC has been stepped for hydraulic and aesthetic reasons on quite a few projects. Two-foot (0.6m) high steps were used for the Tobesofkee Creek Replacement Dam shown in Figure 7.10. For the Woody Branch Dam, an

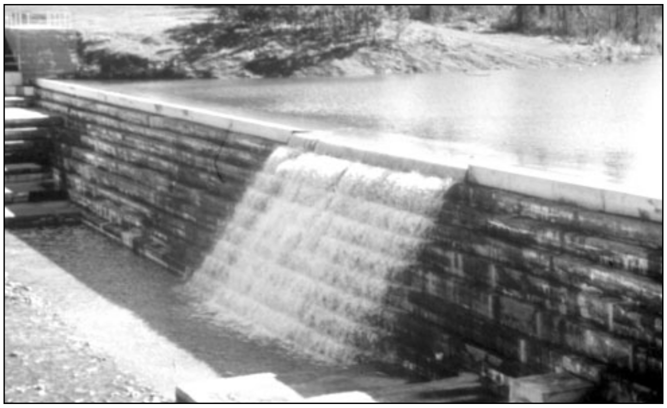


Figure 7.16. Two-foot (0.6m) high steps of formed RCC for Tobesofkee Creek Replacement Dam, GA provide hydraulically efficient and visually attractive spillway

Figure 7.17. Precast panels to form downstream planter boxes for North Loop Detention Dam No. 1, TX

edge-form attached to a tamping paver laydown machine produced a formed 9 in (230mm) high outer edge as shown in Figure 7.11 during construction and in Figure 7.12 upon completion of this flood detention dam. By using this RCC placement technique, the contractor was able to minimize any overbuild to the dam section. An RCC downstream face with no special outer edge treatment was used for the Kerrville, Texas Ponding Dam. Figure 7.13 shows the exposed face a year after the RCC structure was overtopped by 13.4 ft (4.1m) during a flood. The overtopping only removed any loose or poorly compacted RCC at the outer edge.

Figure 7.14 shows the Kerrville, Texas Ponding Dam being overtopped by 16.2 ft (4.9m) two years after it was overtopped by 13.4 ft (4.1m). Figures 7.15 and 7.16 show operating spillways for the exposed RCC downstream faces of Lake Grace Dam, Alabama and Tobesofkee Creek Dam, Georgia.

Similar to the upstream face, concrete either conventionally placed or in the form of precast panels can be applied as a downstream face. Precast concrete panels were used to form planter boxes on the downstream side of the North Loop Detention Dam No.1 at Austin, Texas, as shown in Figure 7.17; however, after a number of years, vandals had spray painted graffiti on the panels and the

Figure 7.18. Partial replacement for Faraday Dam, OR has vertical formed chimney section plus unformed RCC downstream

planting had apparently not been maintained, thus reducing the planned positive visual effect. For the partial replacement of Faraday Dam near Estacada, Oregon, a chimney section was used at the crest of the dam. In this case, the formed vertical portion was conventional concrete while the sloped downstream portion was exposed RCC with no special edge treatment as shown in Figure 7.18.

Precast concrete sections were used at North Tyger River Dam (Figure 6.2) for both the overflow and non-overflow sections. The precast sections were 2 ft (0.6m) square and typically 24 ft (7.3m) long. These sections served as the formwork during RCC placement and provide a desirable attractive finished appearance.

For small RCC dams, step heights have ranged from 1 to 2 ft (0.3 to 0.6m) to coincide with a height of one to two RCC lifts two-foot (0.6m) high steps tend to require slightly more RCC material, but the unit forming cost could be less if the higher forms are readily available to the contractor. In the overflow section, two-foot (0.6m) high steps are also more hydraulically efficient than shorter steps.

7.2 Shrinkage and Cracking

Invariably, an RCC dam will develop transverse cracks through the structure. The cracks are caused by a volume reduction due to a drop in temperature of the structure coupled with restraint, usually provided at the base of the structure. These cracks tend to start at the restrained base of the structure and proceed upward to the crest thus forming a full section crack. Some surface cracking may also occur, due to loss of moisture from the concrete to the atmosphere.

The transverse cracks do not pose any threat to the stability or safety of the dam. Even if there is no shear transfer from one side of the crack to the other, a properly designed gravity section is structurally stable.

Cracks can, however, pose a problem if they are wide enough to pass water. In addition to the value of the lost water, seepage can accelerate weathering through increased wet-dry and freeze-thaw cycles, and encourage vegetation growth. Heavy seepage can also pose a public perception problem. This could be in the form of loss of confidence in the safety of the structure or a reduced visual appeal of the dam.

7.3 Crack Control

A temperature related volume reduction can be controlled through the use of crack inducers to form joints that will place the crack at a desired location rather than let the structure crack in an uncontrolled manner. However, most small RCC dams built to-date in the United States have not included joints for several reasons, including: (a) cracking was not determined to be detrimental to the performance and therefore the cost of joints was not warranted; and (b) water-stops usually associated with transverse joints cannot be placed in the RCC itself and very few small RCC dams are designed with conventional concrete faces.

Several examples of projects where joints were used include the 1740 ft (530m) long Cache Creek Spillway in California, (Figure 1.5), where cracks were induced at 100 ft (30m) spacing with no water-stops in the RCC. The crack inducers, which were placed in every other 1 ft (0.3m) thick lift, worked well in controlling crack widths. Because the structure retained sediment, the design engineers did not want soil materials passing through wide cracks during a flood. The Sacramento District Corps of Engineers performed a level one Corps of Engineers thermal study in order to determine a crack width and a transverse joint spacing. A complete example of the thermal study for the Cache Creek project is contained in reference 16. A crack width needs to be assumed in order to calculate crack or joint spacing. In this case, a width of 0.15 in. (4mm) was assumed. All the induced contraction joints opened properly during the first few months after completion of construction. Actual crack widths varied from 0.06 to 0.25 in. (1.5 to 6mm). Similar to transverse joints in concrete pavements, all cracks in low, long RCC structures do not occur at the same time, nor are they of the same width.

The Bullard Creek Floodwater Retarding Structure (Figure 7.19), a 53 ft (16m) high all-RCC structure at Lakeview, Oregon, utilized two types of transverse seepage control joints. They were 3 in. (76mm) deep triangular crack control notches formed into both the upstream and downstream RCC faces and three contraction joints placed at strategic locations. The notches were spaced at 20 ft (6.1m) across the 320 ft (98m) crest length. See Figure 7.18 for the completed dam.

The contraction joints consisting of galvanized metal sheets (Figure 7.20) installed into each lift were located at the change in slope at the base of both abutments and at a sharp step in the left abutment rock surface. The latter joints which extended across the entire structure in the upstream-downstream direction terminated in a crack

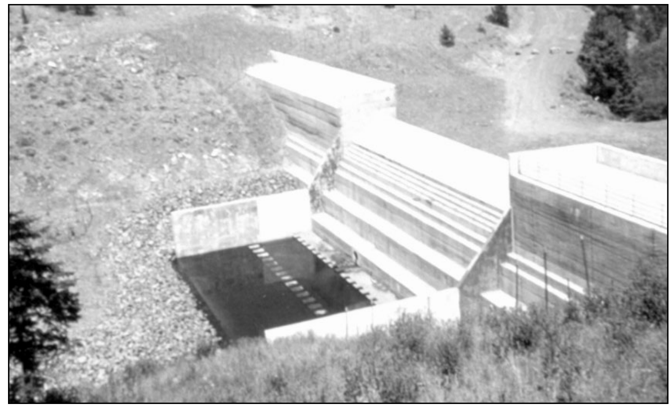


Figure 7.19. Completed Bullard Creek Dam, OR which includes exposed RCC steps

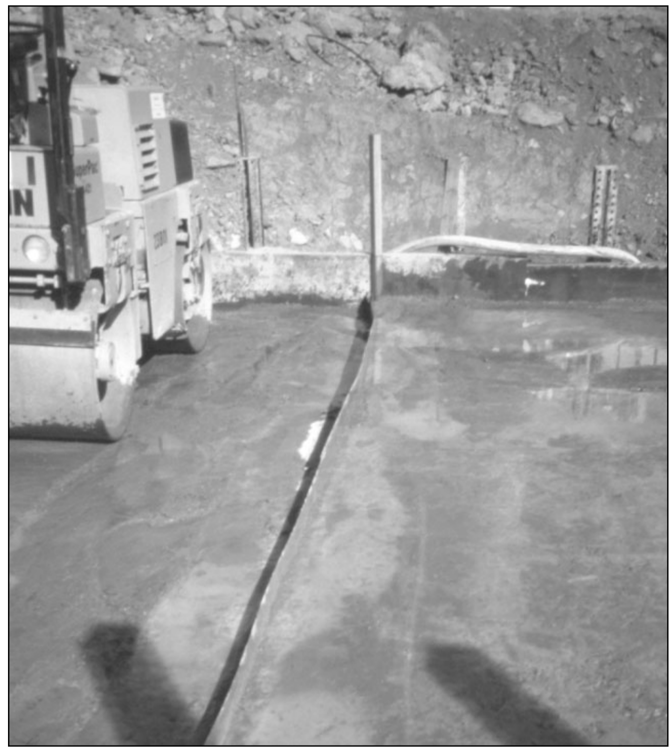


Figure 7.20. Contraction joints consisting of galvanized metal sheets

control notch on both faces. No water stops wereinstalled in the RCC. Thus, the only form of seepage control was to apply a bead of sealant in the upstream notches. The purpose of the crack control notches and full depth contraction joint was to force any crack to form at these points of reduced transverse section.

7.4 Lift Treatment

While small RCC dams may be built quickly, the average placement rates are lower than for larger, more massive RCC structures. This is due to the fact that large surface areas are not available for high placement rates.

As noted in Chapter 5, shear resistance at the lift lines is rarely a controlling factor in the stability of low

RCC dams. However, there are conditions where mortar bedding between successive lifts improves performance of the structure. Bedding adjacent to the upstream face not only improves shear resistance in this area, but is effective in reducing seepage along lift lines. The mortar bedding improves cohesion (bond) in this area, and also fills any voids due to segregation at the bottom of the lift surface.

Other areas that benefit from mortar bedding are several surface lifts near the dam's crest in a spillway area or entire RCC surface lifts that will experience overtopping. In these areas, there is little weight above to mobilize a high degree of shear friction (V_{net} 3 tanØ) and there is a greater possibility of a "cold-joint" (little to no lift surface cohesion), due to slower placement.

As noted above, cold-joints can occur due to a delay between placement of successive lifts. A significant reduction in shear resistance "cold-joint" can be caused by a number of factors, including: (a) the exposure time of a lift prior to coverage by the next lift; (b) the RCC surface temperature during the time of exposure; and (c) the placement temperature. Greater time delays between lift placement and increased temperatures result in lower cohesion values at the lift-line.

The realization that time and temperature effect shear resistance at the lift line has produced a concept termed joint-maturity. Joint-maturity is expressed in Fahrenheit degree-hours and is the product of the surface temperature and time of exposure until the next lift is placed. There is no direct metric conversion to Fahrenheit degree-hours except at a single temperature value. During the first 300 degree-hours, RCC mixtures which have not been retarded with a chemical admixture and which do not contain a high proportion of pozzolan in the mix, lose about one-half of the shear strength compared to the unjointed parent material. The shear strength loss is due to loss of cohesion. There is generally no effect on the friction angle. Ultimately, this value could average one-third of the parent material as evidenced by the results of shear-tests of cores extracted from actual RCC dams in the United States (reference 17).

If the design engineer finds that a reasonable level of shear resistance is needed at lift joints, a joint maturity in the 250 to 300 degree-hour range is likely to be appropriate. However, for small volume RCC dams, measuring both time and temperature can be cumbersome. Specifications generally require some type of surface treatment, usually application of a mortar bedding, when a certain joint-maturity value is exceeded, but this value is of little guidance to the contractor on how to bid or schedule the work.

Small RCC dams can usually be built in short periods of time, have lesser joint shear strength requirements, and incorporate higher cement content mix designs than larger RCC dams. Therefore, the design engineer can in most cases simplify joint maturity requirements to exposure hours. Historical weather records can be referenced to assess the range of temperatures expected at the scheduled time of construction. Acceptable exposure time can be estimated by dividing an acceptable joint maturity value by the expected temperature for given time frames. If the expected temperature is in the 65 to 75° F (18-24° C) range, a four hour maximum exposure can be specified before bedding mortar needs to be applied.

This exposure time can be extended for admixture retarded RCC mixtures and also those which contain a high percentage of pozzolan (at least 40% of the cementitous content). High pozzolan content RCC mixtures retard early strength gain.

Compacted lifts should be kept both moist and clean of all loose material before placing the next lift. If excess cure water is allowed to pond, it will reduce cohesion at the lift joint due to an increase in the water-cement ratio of the RCC mixture in this area.

7.5 Mortar Bedding

The thickness of mortar bedding is preferably between 1/4 to 1/2 in. (6 to 13mm). If the mortar layer is too thick, it can pump moisture to the surface upon compaction of the layer above. For small RCC dam projects, the mortar is usually mixed in transit mixed concrete trucks, allowed to flow onto the RCC surface and then spread by brooms, rakes or lutes. (Figure 7.21). For very small projects where bedding mix use is limited, a bagged mortar mix has been used and mixed on-site in a mortar mixer. In some cases, a shovel full of cement has been added per bag to increase strength of the mortar.

In order to be effective as a bonding agent between successive lifts, the mortar bedding needs to be stronger than the RCC itself. In simple terms, the "glue" needs to be stronger than the materials being glued or bonded together. A bedding mortar consisting of portland cement, sand water, and usually a retarding admixture, should generally be proportioned to meet the following guidelines:

Figure 7.21. Spreading bedding mortar from a transit-mixed concrete truck with brooms

Slump	6 to 9 in. (150 to 230mm)
Maximum size aggregate	1/4 to 1/2 in. (6 to 9.5mm)
Minimum Cement Content	500 lbs/cu yd (296 Kg/.m^3)
Minus #200 sieve material	3% maximum
Admixture – ASTMC494, Type D – (water reducing and retarding)	retard initial set to greater than 3 hrs. at 95° F (35°C)
Design strength	2000 psi minimum (13.8MPa) at 7 days or 2500 psi minimum (17.2MPa) at 28 days

7.6 Instrumentation

Instrumentation for small RCC dams is usually minimal, unless an unusual design or foundation condition exists. Because small RCC dams invariably do not include a gallery within the section, seepage through the dam can not be collected and measured within the dam. Therefore, any seepage measuring devices would have to be located downstream of the structure. This could take the form of a weir or flume at the abutment groin or other location. Visual observation is a simplified alternative method of determining whether the dam is performing satisfactorily with respect to seepage.

Other instruments typically considered for small RCC dams are staff gauges to measure water level at the upstream face and possibly also in the stilling basin, as well as survey monuments. The latter can take the form of a target or brass cap monument on the dam crest.

Instruments such as piezometers, inclinometers, thermocouples, borehole extensometers, strain gauges, or seismographs are usually reserved for use in higher dams. They should be considered for small RCC dams if site specific conditions exist that merit on-going monitoring.

Chapter 8

RCC Mixture Proportions

8.0 Introduction

The basic objective in proportioning RCC mixtures is to produce a concrete that satisfies the performance requirements using the most economical combination of readily available materials that can be placed by roller compaction methods (reference 18). For small RCC dams the desired physical properties of the mix depend largely on the design selected for the structure and its geographical location.

From a design standpoint, the main item to be considered is whether the RCC will be exposed to the weather or will be covered with conventional air-entrained concrete. Many small RCC dams have been designed to be built of only RCC. Thus, the RCC is exposed and adequate durability is the primary design objective rather than any strength requirements. In the absence of an air-entraining agent in the RCC mixture, higher compressive strengths with good compaction will enhance durability.

The level of durability required depends on the number of freeze-thaw and wet-dry cycles yearly and if the RCC will be critically saturated. Small exposed RCC dams located in areas subject to many freeze-thaw cycles will require a higher degree of durability, which equates to a higher required compressive strength and a reduced porosity. The porosity can be minimized with a well graded aggregate and sufficient paste volume.

The size of the structure and therefore the volume of RCC required also plays an important role in the selection of materials and mixture proportions. For small volume RCC dams, more consideration needs to be given to reducing the cost of aggregate and minimizing equipment requirements than reducing the amount of portland cement in the mix. Small RCC dams are not very massive. Therefore, concern for thermal related issues of the RCC is not as critical as for higher, larger volume concrete dams.

8.1 Materials for RCC

Materials used for RCC include cementitious materials (portland cement and a pozzolan, such as fly ash), aggregate, water, and an admixture at times. A wide range of materials has been used successfully to produce RCC mixtures for dams.

8.1.1 Cementitious Materials

A review of the first 22 small RCC dam projects completed in the United States finds the cementitious content in the RCC mix has ranged widely from 170 to 500 lbs/cu yd (101 to 297 kg/m^3). See Table 1.1. The average of nearly 325 lbs/cu yd (193 kg/m^3) of cementitious material is slightly more than 1/3 greater than the average for larger RCC dams. The higher average cementitious content is due in large part to the fact that many of the smaller RCC dams needed to be designed for a more critical durability requirement than strictly to meet a minimum strength level. In addition, designers for small dams usually do not include sufficient funds for extensive mix design studies aimed at optimizing proportions through the reduction of cementitious contents.

Eleven of the first 22 small RCC dams contained both cement and fly ash in the mixture while the other half specified cement only. Contractors for small volume RCC dams prefer cement only mixes rather than handling and having to furnish an additional storage silo and metering device for the fly ash. Also, many designers would prefer to control only one cementitious material for these projects where the RCC placement time is measured in weeks, not months or years.

8.1.1.1 Portland Cement—For small RCC dams, the type of cement specified should be readily available from more than one supplier. ASTM specification C150 Type I/II (moderate heat) cement has been generally used for small volume RCC dams. A Type V (sulfate-resisting) cement has been used on some projects subjected to acidic water or soils. Type IV (low-heat) cement has been difficult, if not impossible, to obtain in the United States for many years and should not be considered for small volume projects. Any modifiers to a typical cement specification such as a limit on calories per gram, need to be carefully evaluated. A specification modifier may result in a special cement that is not readily available or at a higher cost, if available.

An ASTM C595, Type IP Portland-pozzolan cement or ASTM C595, Type IS, a portland-blast furnace slag cement, may be available in certain areas at a competitive price. These cements combine portland cement at the manufacturing plant with a pozzolan such as fly ash or with a ground blast furnace slag.

8.1.1.2 Pozzolans—The selection of a pozzolan suitable for RCC should be based on its conformance with an applicable standard (ASTM C618), the specific pozzolan's past performance in concrete, as well as its cost and availability. Most RCC mixtures that have included a pozzolan have used a Class F (low-lime) fly ash. There have been a few cases where a Class C (high lime) fly ash has been used successfully in RCC where the class F ash was not readily available.

8.1.2 Aggregates

For small volume RCC dams, aggregates can be the most costly material in the RCC mix.

This is especially true if the aggregate needs to be hauled a long distance and grading or other specification requirements are overly restrictive. Because of the quantity involved, aggregates for small RCC dams are rarely produced on site. Therefore, they must be purchased from a quarry, hauled to the site, and stockpiled for use in the RCC mixture.

Purchasing aggregate from an established quarry usually eliminates the need for an extensive amount of new testing to determine that the aggregate is of suitable quality for use in concrete. Basically, the aggregate should conform to ASTM C33-Concrete Aggregates. The supplier should have conducted aggregate testing, and can furnish recent documentation to certify that the aggregate has adequate resistance to abrasion (ASTM C131 and C535) and sulfates (ASTM C88) and does not contain organic impurities in the fine aggregate (ASTM C40) that may affect strength gain.

Specific gravity, absorption (ASTM C127 and C128), and bulk unit weight or density (ASTM C29) are other material characteristics that should be available from the supplier. The design engineer should also check if there have been any reported cases of alkali-aggregate reaction (AAR) using the proposed aggregate. If there is any chance that the aggregate is potentially reactive, the supplier should run the 14-day mortar bar test (ASTM C1260) as a minimum. If there is any potential for AAR, the aggregate should not be used or possibly measures taken to mitigate alkali-silica reactions (ASR). See reference 19 concerning materials and methods to inhibit ASR.

Once it has been established that the proposed aggregate is of sufficient quality and quantity to meet the project needs, the design engineer needs to determine how he can obtain the desired properties for the RCC without adversely affecting aggregate cost. Considering the relatively small quantities of aggregate required, some designers have opted for a single stockpile ranging from the maximum size aggregate (MSA) to fines rather than requiring separate piles of coarse and fine aggregate (sand). In addition, the grading band should not be so tight as to require excessive processing to meet the project specifications. Figure 8.1 illustrates a typical aggregate gradation band.

In order to obtain aggregate for RCC at a reasonable cost, many engineers have specified a base course material that meets the local State Highway Depart-ment specification. The aggregate is generally of adequate quality and its grading is generally consistent with typical RCC mixture requirements. However, the grading band for road base course can be quite open and should be tightened up to produce the desired consistent results for RCC. This is especially true for material passing the No. 200 sieve (0.075mm) where a range from 2 to 7% is suggested. Also, for a 1-1/2-in. MSA base course, the amount of material passing the No. 4 sieve (4.75mm) should be at least 40% in order to keep the mix from being too "bony," leading to excessive voids and segregation. The objective in specifying a certain aggregate grading band is to minimize voids that need to be occupied by paste. A well-graded aggregate will produce a high density RCC. See Table 8.1 for some aggregate gradations that have been specified as well as actual gradations supplied for small RCC dams. The cementitious contents for these projects are also listed in Table 8.1.

8.1.3 Water

The basic requirements for water in RCC mixes are that it be free from excessive amounts of alkalis, acids, or organic matter that might inhibit proper strength gain. Water obtained from a municipal water supply system is generally acceptable for use in RCC. An acceptability criterion for water to be used in concrete is given in ASTM C-94.

If water planned for use in RCC is of questionable suitability, it can be evaluated by making mortar cubes (ASTM C109) using the water in question and comparing results with companion specimens made with potable or distilled water. The water being evaluated is considered acceptable for use in concrete if its 7-day compressive strength is at least 90% that of the known good quality water.

8.1.4 Admixtures

The introduction of chemical admixtures such as an air-entraining agent (ASTM C260) or a water reducing and

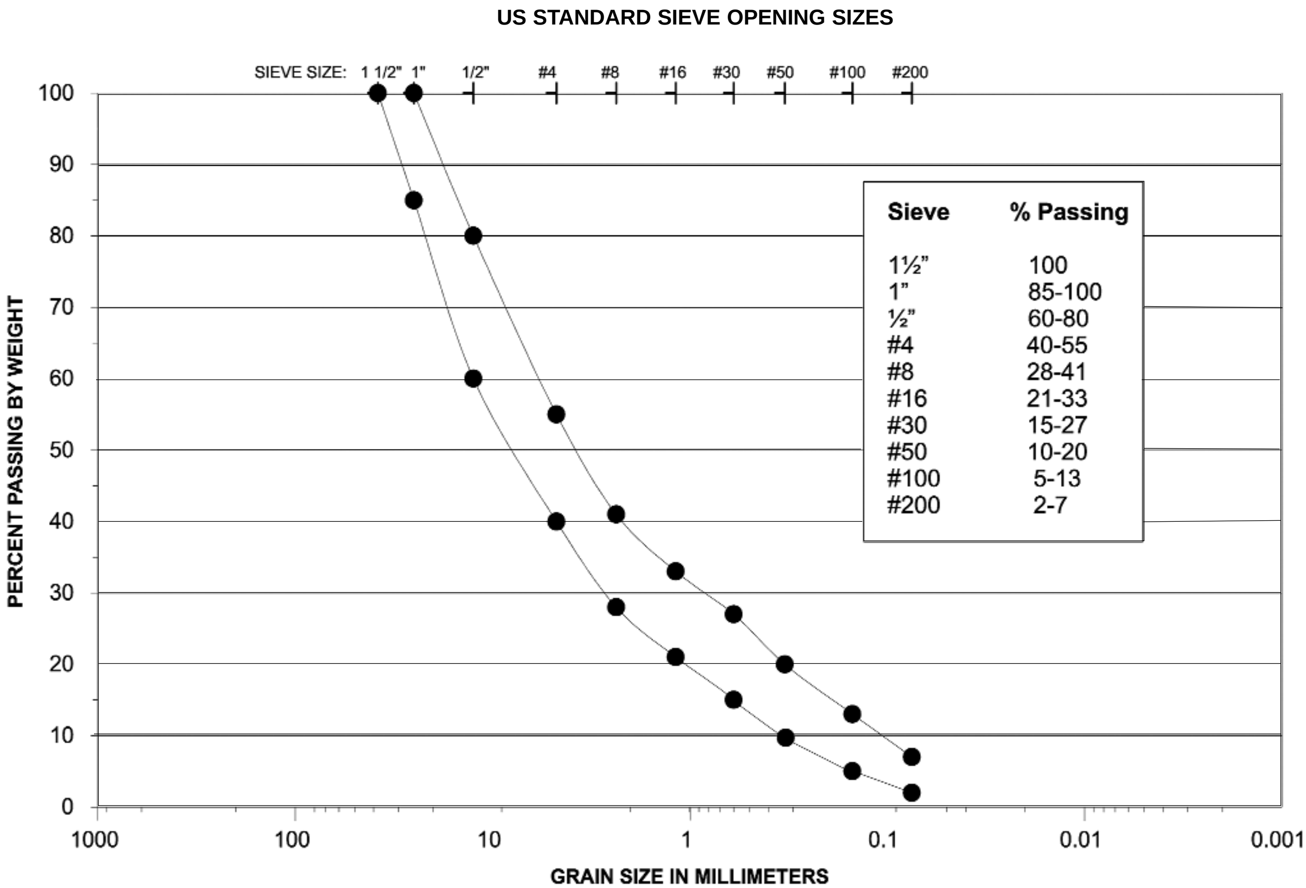


Figure 8.1. Typical aggregate gradation band for a small RCC dam

retarding admixture (ASTM C494, Type D) have been tried in RCC mixtures with limited success in the field. Admixtures produce better results when incorporated in wetter consistency RCC mixes (low Vebe times) and at dosages several times greater than that recommended for conventional slump concrete.

In general, admixtures have rarely been specified for use in RCC for small volume dams. If an admixture is to be used, its effectiveness needs to be evaluated in the field using the RCC mixture of specified consistency and actual mixing equipment.

The usual method of mixing RCC in twin-shaft continuous pugmills also has an impact on the effectiveness of chemical admixtures. It is the opinion of some experienced engineers that mixing in pugmills, with their relatively short retention times, tends to produce poorer admixture results than mixing in a drum mixer with a longer retention time.

8.2 Laboratory Tests and Equipment

Tests conducted to determine RCC mixture proportions should involve equipment that is readily available to most materials testing laboratories. The results of any testing need to be sufficient so that the selected RCC mixture proportions will produce the desired properties on a consistent basis.

8.2.1 Specimen Preparation

For strength properties the traditional 6 x 12 in. concrete cylinder (152 by 304mm) is used. These cylinders can accommodate RCC mixtures with up to 2 in. (50mm) maximum size aggregate (MSA). The mold must be sufficiently rigid to withstand the various methods of compaction without distortion. In many cases, steel cylinder molds have been modified to facilitate specimen removal. The modifications include split cylinder molds or plastic molds inserted in an oversized steel cylinder.

Instead of rodding the specimen as accomplished for conventional concrete, various methods for compacting or consolidating the no-slump RCC in the mold have been used. They include impact compaction, vibration, and two forms of tamping compaction, namely using an electric vibrating tamper or a pneumatic pole tamper.

There are a number of factors that will influence which method should be used in the preparation of RCC specimens for strength testing. They include the consistency of the RCC mixture, availability of compaction equipment and the time necessary to produce acceptable

Table 8.1. Mixture Proportions and Aggregate Gradings for Selected Small RCC Dams

	Atlanta Road, GA	Whipps Mill Road, KY	Prairie Creek TX (1)	North Bosque River, TX (1)
Mix Design	lb/cu yd (kg/m³)	lb/cu yd (kg/m³)	lb/cu yd (kg/m³)	lb/cu yd (kg/m³)
Portland Cement	375 (222)	110 (65)	291 (173)	383 (227)
Fly Ash	0	110 (65)	0	0
Fine Aggregate	-		1638 (972)	1391 (825)
Coarse Aggregate			2001 (1187)	2087 (1238)
Combined Aggregate	3515 (2085)	3600 (2136)		
Water	220 (131)	230 (136)	255 (151)	250 (148)
Aggregate Gradation	Specified (Actual) % passing	Actual % passing	Specified % passing	Actual % passing
2 in. (50mm)	100 (100)	100		
1-1/2 in. (37.5mm)	97 - 100 (100)		100	100
1 in. (25mm)		100	75 - 100	96.8
3/4 in. (19mm)	60 - 90 (84)	97	65 - 85	84.4
1/2 in. (12.5mm)		77		
3/8 in. (9.5mm)		65	45 - 60	46.7
No. 4 (4.75mm)	>40 (45)	43	30 - 45	40.5
No. 8 (2.36mm)		30		
No. 10 (2.00mm)	25 - 45 (25)		20 - 35	34.0
No. 16 (1.18mm)		23		
No. 40 (0.425mm)			8 - 20	17.0
No. 50 (0.30mm)		16		
No. 60 (0.25mm)	5 - 30 (9.8)			
No. 100 (0.15mm)		14		
No. 200 (0.75mm)	3 - 8 (6.5)	1	2 - 8	0.3
Compressive Strength	psi (MPa)	psi (MPa)	psi (MPa)	psi (MPa)
Actual 7 Day	1840 (12.7)	960 (6.6)	2955 (20.4)	4300 (29.7)
Specified 28 Day	2500 (17.2)	1000 (6.9)	3600 (24.8)	3500 (24.1)
Actual 28 Day	2900 (20)	1290 (8.9)		5700 (39.3)

(1) Prairie Creek and North Bosque River were designed by different consulting engineers with the same aggregate gradation specification.

cylinders. The goal of any cylinder preparation method is to achieve a consistent density (unit weight) in the laboratory specimen that simulates that being achieved in the field using full-scale equipment.

8.2.1.1 Impact Compaction—Early in the development of RCC dams, impact compaction methods were used to prepare cylinders for compressive strength testing. This was especially true for the drier-consistency RCC mixes that were being used mostly by engineers with a geotechnical engineering background.

Most of the procedures for preparing impact compaction specimens have been developed using the same type of equipment that is used for the modified Proctor test procedure for soils (ASTM D 1557). This was because modified Proctor compactive effort produced dry densities in the specimens that correlated reasonably well with densities measured on actual early projects. The modified compaction test employs a 10-lb (4.5kg) hammer that drops 18 in. (450mm) before striking the surface of RCC material, producing a compactive effort of 56,000ft-lbf / ft³ (2700 KN-m / m³).

Some researchers later felt that this compactive effort was too high for some RCC mixes, while others found that the typical small sector-shaped hammer caused breakage of aggregate for dry-consistency mixes. However, the biggest problem encountered in using impact compaction for RCC cylinder preparation was that it was too time consuming. In order to produce a cylinder with modified Proctor compactive effort, the RCC had to receive 122 blows per layer if placed in six 2-in. (50mm) layers (Figure 8.2). If the RCC were to be compacted in 3-in. (75mm) or 4-in. (100mm) layers the number of blows of the hammer had to be correspondingly higher. Therefore, because of this time factor, most laboratories do not favor the impact compaction for preparing RCC specimens recently.

Figure 8.2. Modified Proctor compaction equipment being used to produce an RCC cylinder

8.2.1.2 Vibration—Vibrated test specimens are used for RCC mixtures that have a high paste volume as indicated by a Vebe time of 20 seconds or less. The 6 x 12 in. (152 x 304mm) steel cylinder is rigidly clamped to the same vibrating table used in the Vebe test and filled in three equal layers. A 20-lb (9.1kg) weight is placed on top of each layer to approximate the pressure exerted by a vibratory roller in the field. The cylinder is vibrated until mortar is noticeable completely around the edge of the surcharge weight (Figure 8.8). After the third repetition, the excess RCC is struck off and the cylinder capped similar to that for conventional concrete cylinders. This test procedure is designated as ASTM C 1176.

Vibrated specimens have seen limited use in the preparation of cylinders for RCC mixtures used in the construction of small dams. This is because most non-governmental testing laboratories do not have a Vebe apparatus and most RCC mixes used for these small volume projects do not have sufficient paste to provide consistent and repeatable Vebe times (20 seconds or less).

8.2.1.3 Electric Vibrating Tamper—The apparatus that has been used to the greatest extent in the preparation of RCC cylinders recently is the vibrating tamper or rammer (ASTM C 1435). It consists of a vibrating breaker tool manufactured by Hilti, Bosch, Kango, and others to which a flat round plate has been securely attached to the end of the shaft. The electric-powered hammer must be capable of providing at least 2000 impacts per minute. The diameter of the tamping plate is usually 5-3/4 in. (146mm), but some of these tools have been modified with tamping plates of 5-1/4 to 5-1/2 in. diameter (133 to 140mm) to allow for a greater annular space between the plate and the cylinder mold wall. A Hilti hammer being used in the field to prepare RCC cylinders is shown in Figure 8.3 while a mounted Kango hammer for use in the laboratory is shown in Figure 8.4.

In this procedure, the cylinder is filled in three lifts. The vibrating hammer is applied to consolidate the RCC for 20 seconds or until a ring of mortar completely fills the space between the outer edge of the tamping plate and the inside mold wall. When the mortar ring forms, the vibrating hammer should be stopped and the next layer of RCC added. After the third lift, the concrete at the top is struck off and the cylinder capped,

Figure 8.3. Modified Hilti hammer used to prepare RCC cylinder in the field

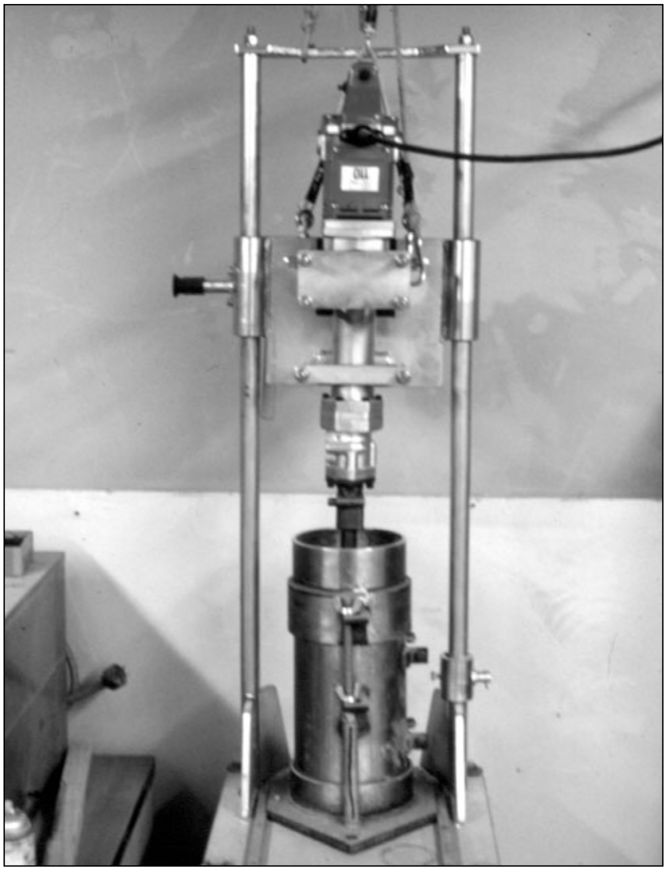


Figure 8.4. A mounted Kango hammer for use in preparing laboratory RCC cylinders

cured, and tested similar to a conventional concrete cylinder. Prior to testing, it is good practice to visually inspect the sides of the stripped cylinder and note any voids. If large or excessive voids are noticed, the cylinder preparation method or RCC mixture should be modified to produce a uniform side surface with a minimum of voids. Also, prior to breaking, all specimens should be weighed to determine their density (unit weight).

The vibrating hammer has gained considerable acceptance as a method for producing RCC cylinders because the equipment is readily available at a reasonable cost, its vibration frequency is close to that of a vibratory roller, the tool can be handled and transported by a single worker, it can be used with a wide range of RCC mixtures with variable consistencies, and the specimens can be produced quickly.

8.2.1.4 Pneumatic Pole Tamper—Another method that has been used to produce RCC cylinders is the use of a pneumatic pole tamper. Pole tampers, also referred to as "pogo sticks" or "jumping jacks", have much greater amplitude than a vibrating hammer. The pole tampers can have a stroke of as much as 6 in. (150mm) compared to the almost imperceptibly low amplitude of the vibrating hammers. The frequency of strokes for the pole tamper is less than 600 impacts per minute and can be dependent upon the efficiency of the unit's air compressor. The pole tamper may be better described as an impact compaction method than a tamping method for RCC cylinder preparation (Figure 8.5).

Pole tampers have traditionally been used to compact lean dry mixes. For wetter mixes with a Vebe time, say less than 20 seconds, use of a pole tamper can result in paste coming to the surface quickly, which can adhere to the tamping face. For the drier consistency mixes for which pole tampers are most applicable, the energy imparted to the sample may be greater than that produced using a vibrating hammer or rammer. The greater energy will produce a slightly higher density in the cylinders.

Figure 8.5. A pneumatic pole tamper used to make RCC cylinders

8.2.2 Moisture Determination

In order to prepare concrete cylinders for strength testing, one needs to determine the moisture or water content for an RCC mixture. The desired water content can be determined by applying geotechnical (soils) or concrete principles. The water content determined by geotechnical moisture-density relationships is usually expressed as percent water content by oven dry weight

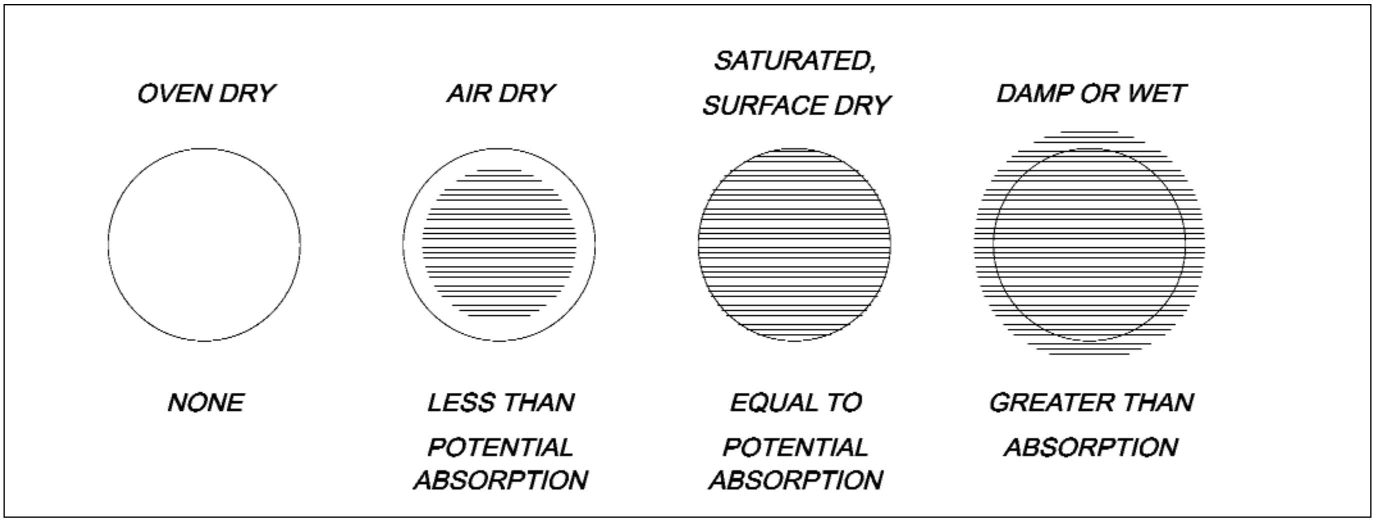


Figure 8.6 . Various moisture conditions of aggregate

of solids. Similarly, portland cement and fly ash can also be specified in the same manner. Mixes specified as a percentage by dry weight are helpful in the calibration of volumetric proportioning RCC mixing plants, called pugmills.

In traditional concrete mixture design, water and all other ingredients are specified as pounds per cubic yard (kg/m^3). If the design engineer knows the specific gravity of each component of the RCC mixture as well as a measured or assumed percent air voids remaining after compaction, mixes expressed in percent by dry weight of solids can be calculated to mixture proportions per unit volume, or vice versa. From a payment and yield standpoint, mixes expressed in pounds per cubic yard (kg/m^3) are useful.

In determining and specifying a moisture (water) content for RCC mixes, it is important to understand the typical variability in moisture conditions for aggregate. Figure 8.6 illustrates graphically the following moisture conditions (reference 20):

1. Oven dry—fully absorbent
2. Air dry—dry at the particle surface but containing some interior moisture, thus still somewhat absorbent
3. Saturated surface dry (SSD)—neither absorbing water from nor contributing water to the concrete mixture
4. Damp or wet—containing an excess of moisture on the surface (free water)

The water content mixes expressed in pounds per cubic yard (kg/m^3) is based on the aggregate being in a saturated surface dry (SSD) condition, while those expressed as a percent are based on an oven dry aggregate. The difference is the absorption of the aggregate, or the amount of water necessary to fill pores in the aggregate to bring it to an SSD condition.

In the field, aggregates are rarely completely oven dry or at an SSD condition. In most cases, there is excess moisture in the aggregate. This excess moisture must be taken into account in determining the amount of water to be added at the mixing plant to produce the total desired water content in the mixture.

8.2.2.1 Moisture-Density Test—Although this method is not used very much to produce RCC cylinders as noted in Section 8.2.1.1, the moisture-density test is an effective method for determining the optimum moisture content and maximum density for RCC laboratory compacted test specimens. The compactive effort generally used is that of modified Proctor (ASTM D 1557). A 6-in. (152mm) diameter mold is used with a height of about 4-5/8 in., (117mm). The ASTM specification allows for up to 30% material retained on a 3/4-in. (19mm) sieve. However, some laboratories have run the test using the full-proposed RCC mixture with an MSA up to 1-1/2 in. (38mm).

Mixtures are molded at varying moisture contents and compacted. The results are plotted as moisture content (as a percent) vs. oven dry density (in lbs/cu ft or kg/m^3). The maximum density on the curve so plotted determines the optimum moisture content. Because cement hydration starts as soon as water is added to the mixture, the tests in the laboratory should be accomplished as soon as possible. A delay in compaction will tend to produce a decrease in maximum density and an increase in the optimum moisture content.

The optimum moisture content so derived can be used for the preparation of laboratory cylinders or for proportioning RCC mixtures in the field. However, experience has shown that a moisture content 1/2 to 1 percent wet of optimum will produce more desirable results in actual construction. These slightly wetter mixes produce a more visually acceptable compacted RCC surface. Also, this additional water may be needed to account for surface evaporation, cement hydration, and delay in compaction in the field compared to laboratory prepared specimens.

8.2.2.2 Vibration Consistency—The purpose of vibration tests is to establish a water content that corresponds to a desired consistency or workability. This test is accomplished using a modified Vebe apparatus. Modified means that a surcharge is used in the test. The surcharge or weight has varied considerably worldwide from 22 to 50 lb (10 to 22.7 kg).

In the United States, the test is performed in accordance with ASTM C 1170, Test Method A where a 50 lb (22.7kg.) surcharge weight is used. The Vebe apparatus is shown in Figure 8.7. The vibration consistency or Vebe test follows three basic steps:

1. The open container is filled loosely with uncompacted concrete, leveled off, and the surcharge applied to the RCC.
2. The cylindrical container is attached to the vibrating table, which has a specified constant frequency and amplitude. The specimen is then vibrated with the surcharge on the surface.
3. The time in seconds is noted when a ring of mortar has formed completely around the inside edge of the cylindrical container. (Figure 8.8.)

Figure 8.7. Modified Vebe apparatus

Figure 8.8. Ring of mortar formed between surcharge and inside of Vebe container

The mortar ring indicates that full consolidation has occurred and there is more paste in the mixture than voids in the aggregate. In order to more accurately determine when full consolidation has occurred and thus the Vebe time, a see-through plastic container developed in Brazil may be used. (Figure 8.9.)

The Vebe time is a measure of the consistency or workability of the mixture. Therefore, the water content to be used in the RCC corresponds to a certain desired Vebe time. A short Vebe time indicates the mix consolidates or liquifies quickly when vibration is applied. The concrete may have a measurable slump with a Vebe time of 7 seconds or less. Most RCC dams in the United States where RCC mixtures have been proportioned using vibration consistency have specified a Vebe time in the 15 to 20 second range. Dams with an unformed face must have a mix with a Vebe time of not less than 25 seconds.

In the field, RCC mixtures with Vebe times in the 10 to 15 second range can produce noticeable rutting during construction, less segregation, lower compressive strengths, but higher tensile and shear strengths at lift joints when compared to drier consistency mixes. Increased strength properties at the lift joint are aided by use of a retarding admixture in the RCC mix.

Figure 8.9. Clear plastic container being used to determine Vebe time

8.3 Mixture Proportioning Criteria

RCC mixture designs for small gravity dams are seldom based on satisfying a certain level of stress in the concrete structure. Also, because a minimum crest width is needed for constructability and in many cases the downstream slope starts from the downstream edge of the crest, the RCC section thus produced is greater than needed for basic stability. This section designed for constructability thus lowers the maximum compressive or shear stress in the structure.

Designing the mixture for a certain level of durability and/or permeability is therefore the design criteria most commonly applied to the design of small RCC dams. This is especially true of dams constructed of only RCC which are exposed to the weather. Although strength may not be directly related to durability or permeability, most engineers will specify a minimum compressive strength that will produce adequate durability for the intended purpose.

Because small RCC dams are built rapidly and put into service quicker than larger, more massive concrete dams, the age at which a certain minimum compressive strength is desired is less. At times, this age can be up to 90 days, but is usually 28 days. This differs from larger RCC dams where the time specified for attaining a certain strength level can be 90, 180, or even 365 days.

Currently, there is no universally accepted minimum strength criteria for the design of small RCC gravity dams. Based on their observation of some exposed RCC rehabilitation projects in service, McLean and Hansen, (reference 21) suggested a minimum compressive strength of 3000 psi (20.7 MPa) at 28 days for exposed RCC projects located in a freeze-thaw climate. In areas where the RCC is exposed to few, if any, freeze-thaw cycles per year, a minimum strength of 2100 psi (14.5 MPa) was suggested. The authors further noted that using a well-graded, high quality aggregate, a minimum cementitious content of 325lb/cu yd for the 3000 psi mix and 250 lbs/cu yd (193 and 148 kg/m^3) for the 2100 psi mix, would generally produce the desired durability.

Several other sources of information provide support that a minimum cementitious content of about 325 lbs/cu yd (193kg/m^3) is a good starting point for producing reasonable mix proportions for RCC subjected to freeze-thaw cycles. Based on the first 22 small RCC dams built in the United States the average cementitious content for was nearly 325 lb/cu yd (193 kg/m^3). Additionally, it has been found that about 337 lb/cu yd (200 kg/m^3) of cementitious material is needed to achieve a permeability in the RCC at the lower level of that achieved in concrete dams built of conventionally placed concrete. Referring to the US Army Corps of Engineers Manual on RCC (reference 22, Fig. 3.2), achieving a 3000 psi (20.7 MPa) compressive strength requires that 297 lb/cu yd (160 kg/m^3) of cement is needed on average to attain this strength level in 90 days and 371 lb/cu yd (220 kg/m^3) in 28 days.

If the RCC is capped with conventional concrete and not exposed to the weather, a strength level near the 2100 psi (14.5 MPa) at 28 days suggested by McLean and Hansen (reference 21) for exposed RCC overtopping protection projects in non freeze-thaw areas appears reasonable.

8.4 Mixture Proportioning Methods

Because small RCC dams contain relatively small volumes of RCC, the amount of laboratory testing that can be accomplished is usually limited by budget constraints. In other words, there are generally insufficient funds available for any extensive testing aimed at optimizing mixture proportions or reducing cementitious contents. Therefore, quicker, more conservative approaches to determining an adequate RCC mix design are taken. These mixes may contain more cement or cement plus fly ash than is absolutely necessary, but the design engineer has little basis for using a lesser quantity of these materials. In addition, the heat generated by the additional cementitious material in the mix usually does not cause any detrimental effect in small, not very massive dams.

8.4.1 Mix Design Approaches

RCC by its very nature has some characteristics of soil and some characteristics of concrete. Because of this, two methods of developing an RCC mix design have been developed. The soil compactions method uses ASTM D 1557 procedure typically used for granular fill control. The concrete approach uses the conventional concrete mix proportioning method. Both methods are suitable for developing RCC mix design. A detailed description of each method is contained in reference 24 which is the PCA "Design Manual for RCC Spillways and Overtopping Protection".

8.4.2 Empirical

For small volume projects usually requiring less than 5,000 cu yd (3820 m^3), an empirical approach may be used to determine RCC mixture proportions. It involves no laboratory testing. If the design engineers have knowledge of adequate performance of an RCC mix used on a similar project subjected to similar climatic conditions, he may select this mix for his next small dam. Such was the case in selecting a mix for the Atlanta Road Dam, which required only 1900 cu yd (1450 m^3) of RCC and had to be built quickly. Therefore, the engineer specified basically the same RCC mixture that he had used with success on the Tobesofkee Creek Replacement Dam that had been built earlier that year (Table 8.1). Both dams are located within 80 miles (130km) of each other and both used a modified Georgia State Highway Department base course for the RCC aggregate. Use of this method should only be applied by engineers well versed in RCC mixtures and resulting properties.

8.4.3 Field Adjustment to Mixture Proportions

While the RCC mixture proportions derived using any of the laboratory methods have proven to be placeable using typical construction equipment, some minor field adjustments should be anticipated. Most field adjustments involve changes in water content.

Advantage should be taken during construction of RCC test sections or test strips to make desired field adjustments. The adjustments should be made based on visual observation, as well as the results of nuclear density or modified Vebe tests. Some engineers like to see a smooth compacted RCC surface without excessive voids or deformation (ruts). Once a determination is made that a mixture is too dry or wet, an adjustment is made only by adding or deleting water until the desired surface texture and density is achieved with about four passes of the roller in the vibratory mode.

Minor adjustments in water content should also be anticipated during actual construction depending mainly on air temperature, surface evaporation potential, and the varying moisture condition of the aggregate. The latter condition should be expected immediately after a rain when the outside of the stockpile is wetter than the interior. Most moisture adjustments in the field can be based on visual observation by either an experienced mixing plant operator or RCC placing superintendent.

Appendix

Design Example

This example will illustrate the steps in analyzing the stability of a non-overflow section on a rock foundation. The elevation at the base of the section is established at the top of competent rock. This elevation is determined through the subsurface investigation. The elevation of the top of the non-overflow section is based on the spillway crest elevation plus flood storage plus an allowance for freeboard. This design example is presented in in.-lb units only. Assume the project is located in northern Georgia.

Base elevation (top of rock) = 100 ft
Top of non-overflow section = 140 ft
Spillway elevation = 134 ft

The upstream slope will be formed and vertical. See Figure. A1. The downstream slope will be 2 ft high formed steps. Because the downstream slope will be formed, it will not be a function of the RCC's angle of repose. We will assume the downstream slope to be 0.75H:1.0V for the initial stability calculations. The 0.75H:1.0V slope results in a base width of 30 ft. The height of the chimney is dependent on the crest width. We will assume one-way placement traffic, with a crest width of 14 ft.

The following RCC material properties may be assumed for preliminary design; however, laboratory tests should be conducted at a later time to establish actual properties for final design.

RCC unit weight = 150 pcf
Cohesion = 10 psi
Friction angle = 45 degrees

Loading Condition 1—Normal Pool Steady State (see Figure A.1):

For the first loading condition assume headwater will be at the spillway crest elevation and there is no tailwater. Assume 5 ft of silt in reservoir.

Headwater elevation = 134 ft
Base elevation = 100 ft
Silt elevation = 105 ft, submerged unit weight = 60 pcf, earth pressure coefficient (Ko) = 0.33

Table A.1. Summation of Forces for Loading Condition 1

	Horizontal	Forces	Vertical	Forces	Stabilizing	Overturning
Loading	Force (kips)	Arm (ft)	Force (kips)	Arm (ft)	Moments (ft-k)	Moments (ft-k)
Weight of Dam (W)			109.6	20.1	2203.0	
Uplift (U)			-31.8	20.0		636.0
Headwater (H)	36.1	11.3				407.9
Silt (S)	0.2	1.7				0.3
Sum	**36.3**		**77.8**		**2203.0**	**1044.2**
					Net Moment 1158.8	

Determine factor of safety for sliding:

$$\text{Sliding F.S.} = \frac{(V_{NET} \times \text{Friction}) + (\text{Area Base})(\text{Cohesion})}{H_{NET}}$$

$$= \frac{(77.8\text{k})(\text{Tan } 45°) + (30 \times 1 \times 144)\left(\frac{10 \text{ psi}}{1000}\right)}{36.3\text{k}}$$

Sliding F.S. = 3.33

Required $FS_{SL} > 3.0$ Okay

Determine eccentricity for overturning:

$$\text{Eccentricity } [e] = \frac{B_W}{2} - \frac{M_{NET}}{V_{NET}}$$

$$= \frac{30 \text{ ft}}{2} - \frac{1158.8 \text{ ft-k}}{77.8\text{k}} = 0.11 \text{ ft}$$

Required e:

$$e \leq \pm \frac{B_W}{6} \leq \pm 5 \text{ ft}$$

$$0.11 \leq \pm \frac{30}{6} \leq \pm 5 \text{ ft}$$ Okay (For overturning as resultant within middle 1/3 of the base)

Determine foundation pressure:

$$\text{Toe} = \frac{V_{NET}}{B_W}\left(1 + 6\left(\frac{e}{B_W}\right)\right)$$

$$\text{Heel} = \frac{V_{NET}}{B_W}\left(1 - 6\left(\frac{e}{B_W}\right)\right)$$

$$\text{Toe} = \frac{77.8\text{k}}{30 \text{ ft}}\left(1 + 6\left(\frac{0.11 \text{ ft}}{30 \text{ ft}}\right)\right) = 2.65\text{ksf}$$

$$\text{Heel} = \frac{77.8\text{k}}{30 \text{ ft}}\left(1 - 6\left(\frac{0.11 \text{ ft}}{30 \text{ ft}}\right)\right) = 2.54\text{ksf}$$

Allowable foundation pressure for competent bedrock = 50 to 100ksf Okay

Loading Condition 2—Normal Pool with Seismic Load (see Figure A.2):

Since the project is located in northern Georgia, it borders on Seismic Zones 1 and 2—use Zone 2

$$\text{Seismic Coefficient } (\alpha') = 0.10$$

$$\text{Earhtquake Inertia Force } \left(P_{eq}\right) = (W) \times (\alpha')$$

$$= (109.6k)(0.1) = 11.0k$$

$$\text{Earthquake Hydrodynamic Force } \left(Pw_{eq}\right) = \frac{2}{3}\left(C_{eq}\right) \times (\alpha') \times \left(y^2\right)$$

$$= 0.67\ (0.051)(0.1)(34\ ft)^2 = 3.9k$$

Table A.2. Summation of Forces for Loading Condition 2

Loading	Horizontal Forces Force (kips)	Horizontal Forces Arm (ft)	Vertical Forces Force (kips)	Vertical Forces Arm (ft)	Stabilizing Moments (ft-k)	Overturning Moments (ft-k)
Weight of Dam			109.6	20.1	2203.0	
Uplift			-31.8	20.0		636.0
Headwater	36.1	11.3				407.9
Silt	0.2	1.7				0.3
Earthquake Inertia Force	11.0	17.0				187
Earthquake Hydrodynamic Force	3.9	13.3				51.9
Sum	**51.2**		**77.8**		**2203.0**	**1283.1**
					Net Moment 919.9	

Determine factor of safety for sliding:

$$\text{Sliding F.S.} = \frac{(77.8)(\text{Tan } 45°) + (30 \times 144)\left(\frac{10\text{ psi}}{1000}\right)}{51.2k} = 2.36$$

$$\text{Required } FS_{SL} > 1.0 \qquad \text{Okay}$$

Determine eccentricity for overturning:

$$\text{Eccentricity}\,[e] = \frac{30\text{ ft}}{2} - \frac{919.9\text{ ft}-\text{k}}{77.8\text{k}}$$

$$= 3.2\text{ ft}$$

Required e:

$$e \le \pm\frac{B_W}{6} \le \pm 5\text{ ft} \quad \text{Okay (For overturning as resultant within middle } 1/3 \text{ of the base)}$$

Determine foundation pressure:

$$\text{Toe} = \frac{77.8\text{k}}{30\text{ ft}}\left(1 + 6\left(\frac{3.2\text{ ft}}{30\text{ ft}}\right)\right)$$

$$= 4.25\text{ksf}$$

$$\text{Heel} = \frac{77.8\text{k}}{30\text{ ft}}\left(1 - 6\left(\frac{3.2\text{ ft}}{30\text{ ft}}\right)\right)$$

$$= 0.93\text{ksf}$$

Allowable foundation pressure for competent bedrock = 50 to 100 ksf Okay (reference 20)

Loading Condition 3—Partial Section at the Chimney Section During Maximum Pool (see Figure A.3):

Table A.3. Summation of Forces for Loading Condition 3

	Horizontal	Forces	Vertical	Forces	Stabilizing	Overturning
Loading	Force (kips)	Arm (ft)	Force (kips)	Arm (ft)	Moments (ft-k)	Moments (ft-k)
Weight of Dam			39.2	7.0	274.4	
Uplift			-8.2	9.3		76.3
Headwater	10.9	6.2				67.6
Sum	**10.9**		**31.0**		**274.4**	**143.9**
					Net Moment 130.5	

Determine factor of safety for sliding at chimney base:

$$\text{Sliding F.S.} = \frac{(31.0)(\text{Tan } 45°) + (14 \times 144)\left(\frac{10\text{psi}}{1000}\right)}{10.9\text{k}}$$

$$= 4.70$$

Required $FS_{SL} > 2.0$ Okay

Determine eccentricity for overturning at chimney base:

$$\text{Eccentricity } [e] = \frac{14 \text{ ft}}{2} - \frac{130.5 \text{ ft} - \text{k}}{31.0\text{k}}$$

$$= 2.79 \text{ ft}$$

Required e:

$$e \leq \pm\frac{B}{6} = 2.33 < 2.79 \quad \text{Does Not Satisfy Requirement}$$

Determine foundation pressure at chimney base:

$$\text{Toe} = \frac{31.0\text{k}}{14 \text{ ft}}\left(1 + 6\left(\frac{2.79 \text{ ft}}{14 \text{ ft}}\right)\right)$$

$$= 4.86\text{ksf}$$

$$\text{Heel} = \frac{31.0\text{k}}{14 \text{ ft}}\left(1 - 6\left(\frac{2.79 \text{ ft}}{14 \text{ ft}}\right)\right)$$

$$= -0.043\text{ksf} \quad \text{Tension in Heel}$$

Tension in the heel may be unacceptable. A cracked section analysis, which is beyond the scope of this document, may be performed. Alternatively, the section geometry and/or lift tensile strength parameters need to be modified.

Figure A1. Loading Condition 1 Normal Pool—Steady State

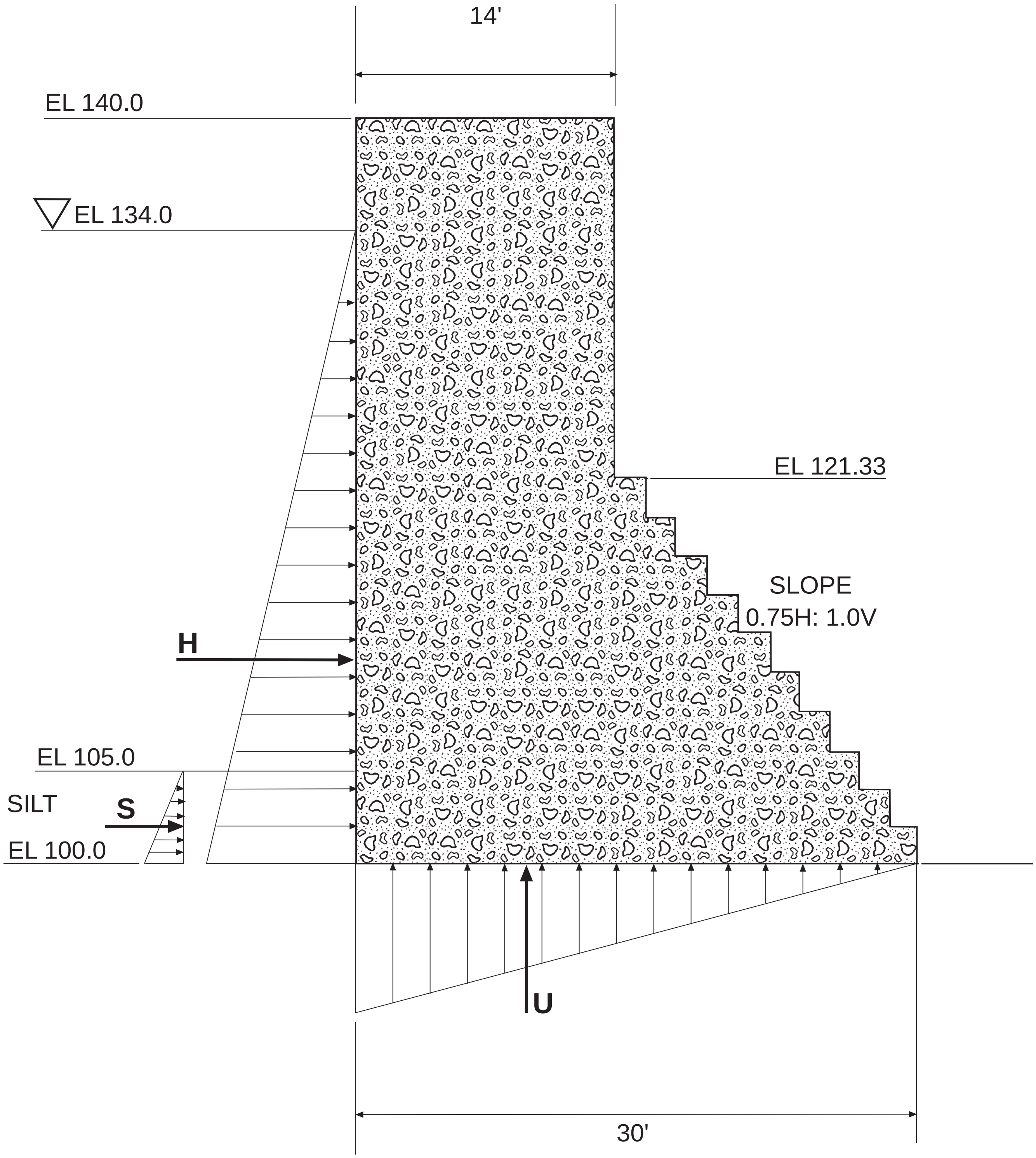


Figure A2. Loading Condition 2 Normal Pool with Seismic Load

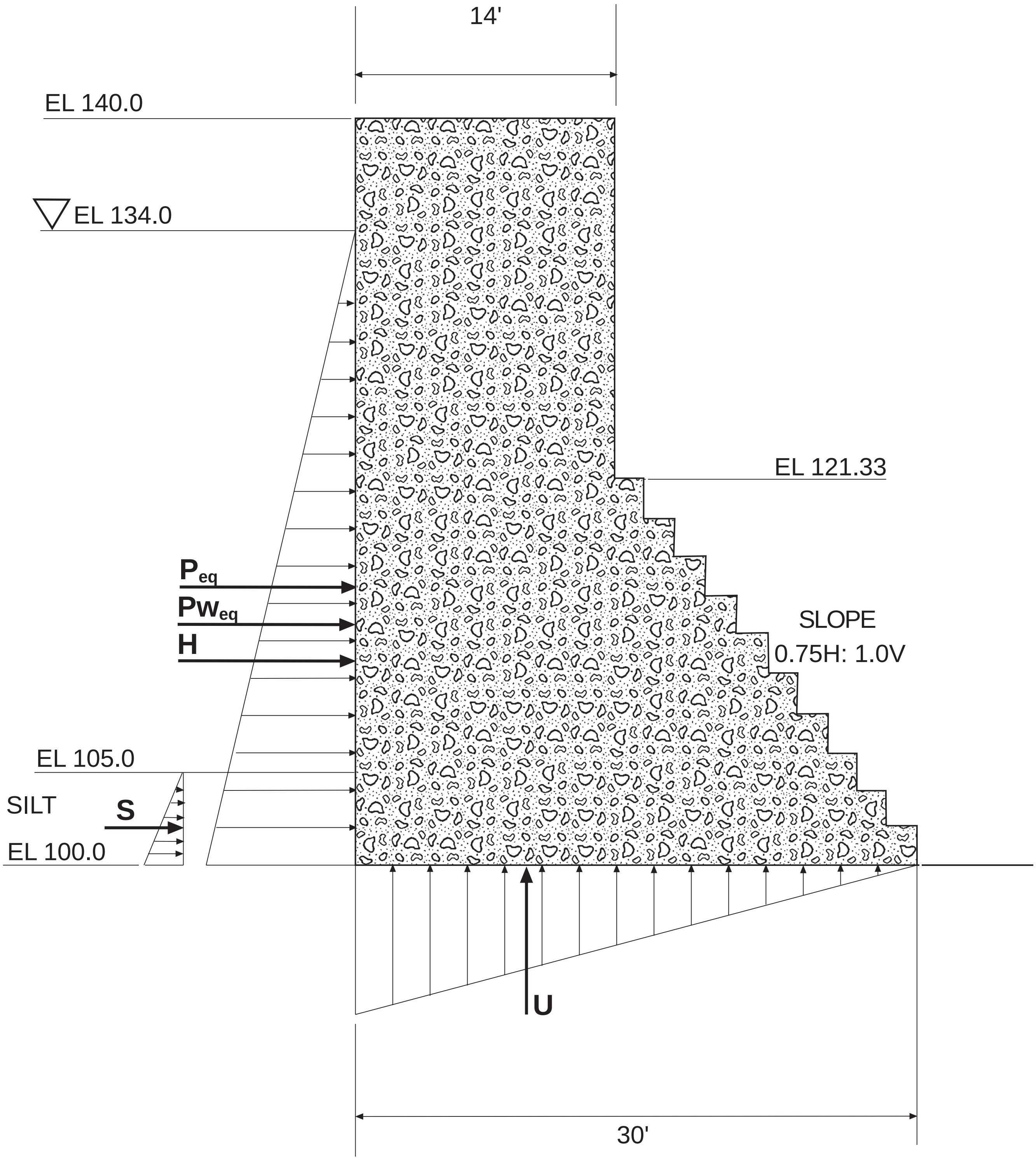
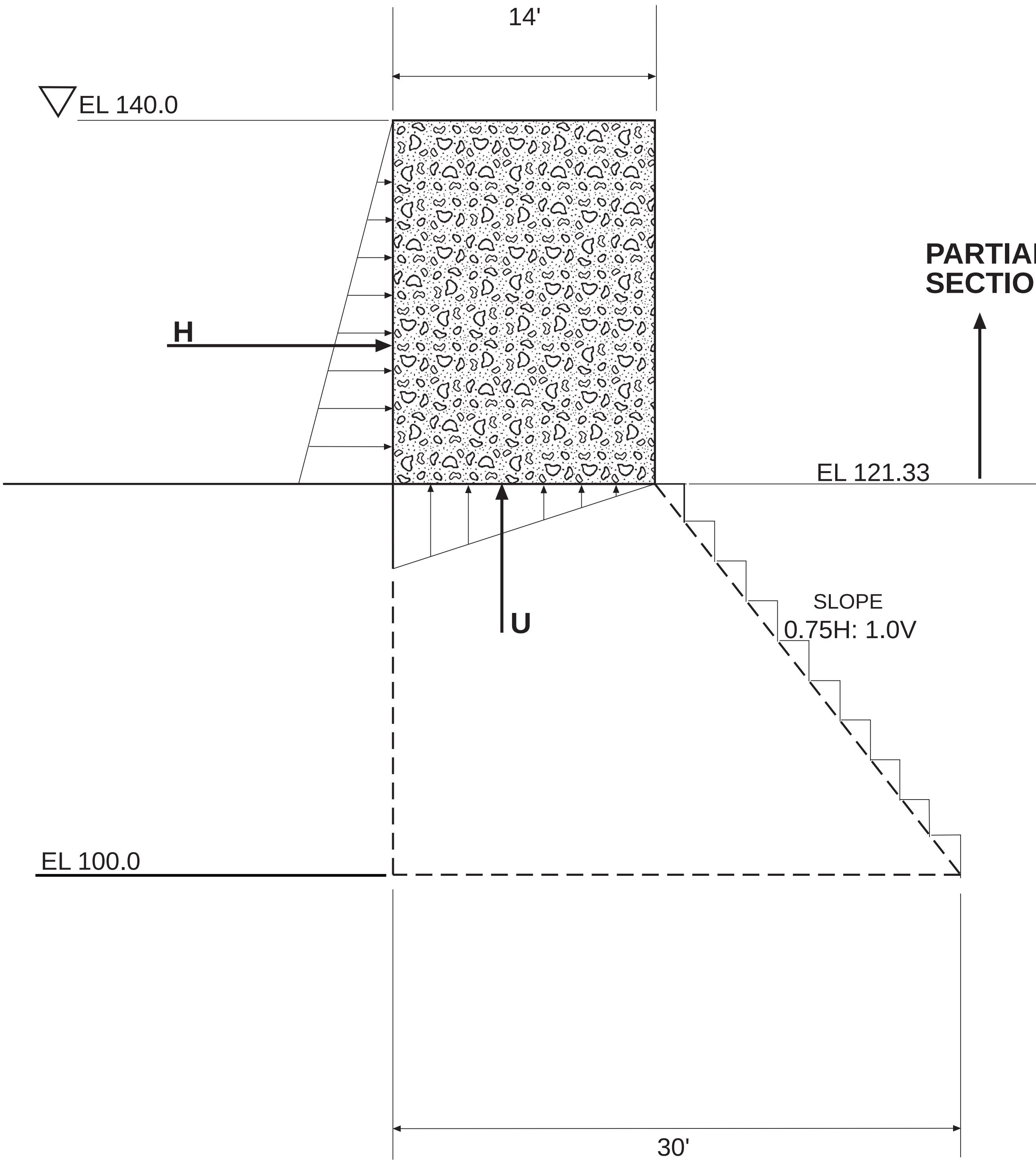


Figure A3. Loading Condition 3 Maximum Pool—Partial Section



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Metric equivalents are provided throughout the text. For the design example, only in.-lb units were used. For conversion to SI (metric) units, the following conversions should be used.

Selected Conversion Factors to SI Units

To Convert	Into	Multiply by
Feet (ft)	= meter (m)	0.3048
Pounds per square foot (psf)	= kilopascals (kPa)	0.04788
Pounds per square inch (psi)	= kilopascals (kPa)	6.8948
Pounds per cubic foot (pcf)	= kilograms/cubic meter (kg/m^3)	16.018
Kips (1000 lbs)	= kilograms (kg)	453.6
Kips per square foot (ksp)	= kilopascals (kPa)	47.88
Foot - Kips (ft -k)	= meter-kilograms (m-kg)	138.2

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