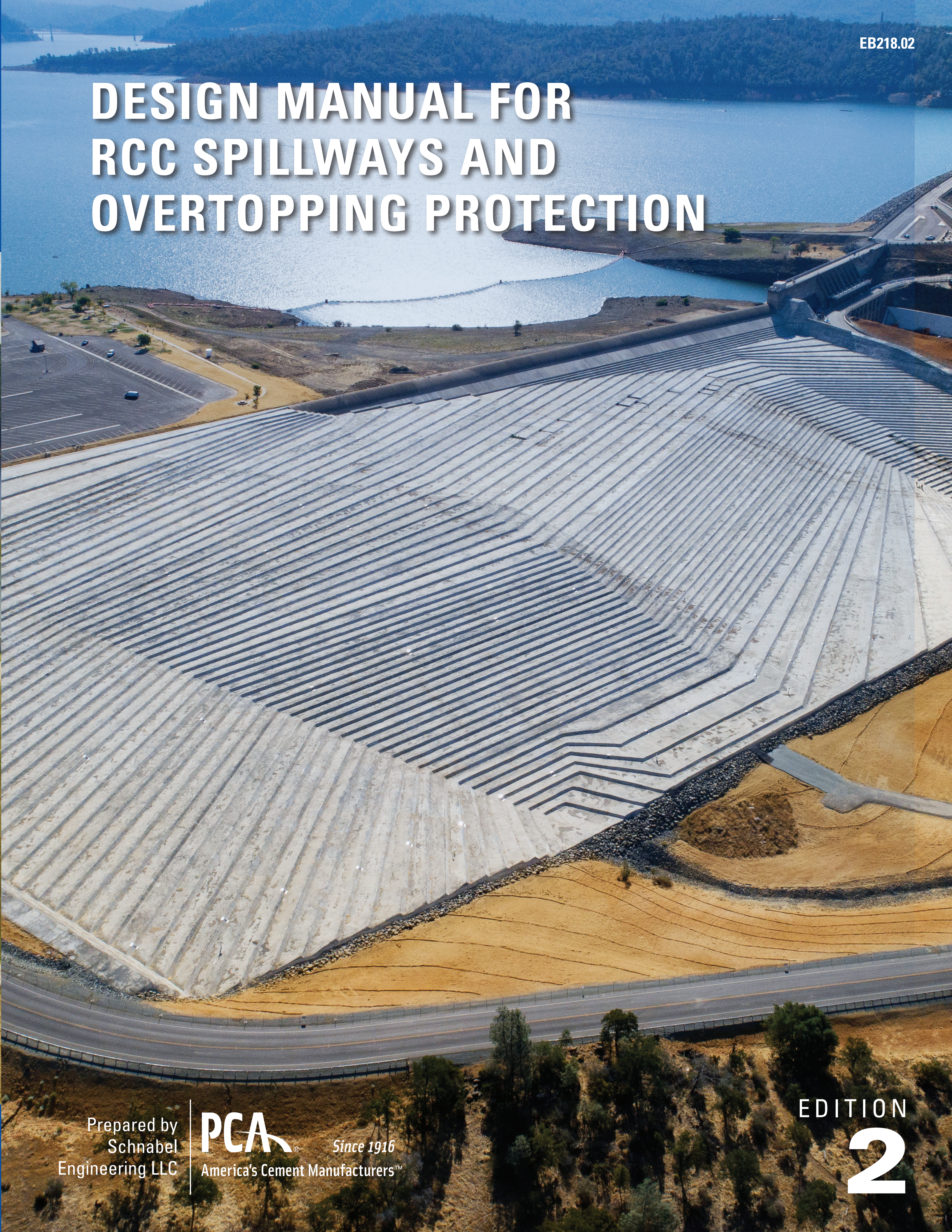
EB218.02

DESIGN MANUAL FOR RCC SPILLWAYS AND OVERTOPPING PROTECTION



Prepared by
Schnabel
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EDITION
2

DESIGN MANUAL FOR RCC SPILLWAYS AND OVERTOPPING PROTECTION

SECOND EDITION

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FRONT COVER PHOTOGRAPH: Oroville Dam Emergency Spillway, Oroville, California. Credit to California Department of Water Resources.

BACK COVER PHOTOGRAPH: Bear Creek Dam, Wise, Virginia. Credit to Schnabel Engineering.

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TABLE OF CONTENTS

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CHAPTER 1

RCC HYDRAULIC STRUCTURES

1.1 INTRODUCTION

With the rapid growth of the use of roller-compacted concrete (RCC) in hydraulic structures, the application of RCC has evolved along with the design methods and analyses that have similarly evolved over time. RCC for use in hydraulic structures has been used in 37 states with the work performed by over 90 different contractors. This document is intended to primarily be a technical design guide for the application of RCC for overtopping protection/spillway enhancement for new and existing embankment dams. However, many of the topics covered will be applicable to other hydraulic structures constructed with RCC.

FIGURE 1-1. Spillway flow in RCC repair area (Tarbela Dam, Pakistan).

1.2 BACKGROUND

Reinforced conventional vibrated concrete (CVC) has a long history of usage for spillways for dams and other hydraulic structures. RCC can have similar hardened properties as CVC but utilizes construction techniques that allow for rapid placement rates and the potential for reduced project costs. RCC takes advantage of both soil and concrete construction techniques. Consequently, RCC construction benefits from the simplicity of placing compacted fill and the strength and durability characteristics of CVC.

FIGURE 1-2A. Ocoee Dam No. 2, TN.

In general, applications using RCC were very limited prior to the beginning of the 1980s. Tarbela Dam in Pakistan is widely recognized as the advent of the modern application of concrete placed and compacted with earthmoving equipment which has come to be known as RCC. The need for rapid placement of rock and embankment material due to the collapse of material around the outlet tunnels, as well as for construction of stilling basins and channel walls for the auxiliary and service spillways, led to the application of RCC at Tarbela Dam (Figure 1-1). More than 420,000 cubic yards of RCC was placed. This new construction technique was quickly tested with a flow of 400,000 cubic feet per second for about six hours at Tarbela. No observable damage occurred from the test flow; subsequently, the structure continues to perform satisfactorily.

FIGURE 1-2B. Ocoee Dam No. 2, TN.

The application of RCC for water resource facilities in the U.S. began in 1980 at Ocoee Dam No. 2 in Tennessee. RCC at Ocoee Dam was used to stabilize a 30-foot-high rock and timber crib dam that was frequently damaged by flash floods. Since rehabilitation (see Figures 1-2A and 1-2B), the dam has been subjected to flash floods as well as frequent overtopping from operational releases for whitewater rafting. The RCC has shown no apparent damage due to water flows or weathering.

The development of the RCC construction technique was particularly timely for the rehabilitation of dams in the U.S. since it followed on the heels of the National Dam Safety Inventory and Inspection Program. The National Dam Safety Program was implemented in the late 1970s by the U.S Army Corps of Engineers (USACE). One of the most common deficiencies that was identified during the program was related to hydraulic structures. Namely, a hydraulic deficiency due to inadequate spillway capacity was noted in the inspections at a significant number of dams.

Hydraulic deficiencies can sometimes be repaired by rehabilitating existing spillways. However, during the National Dam Safety Inventory and Inspection Program, the spillway capacity that was required for many dams was found to be significantly higher than the capacity of the existing spillways. The higher required spillway capacity was due to present-day design criteria for inflow design floods (IDF), regulatory standards, and in many cases, changes in dam hazard classification due to downstream development. Typically, the required IDF for a spillway ranges from 25 percent of the probable maximum flood (PMF) up to the full PMF for certain high hazard dams. This ultimately resulted in very large peak inflows using present-day hydrometeorological standards. Climate change is further influencing these standards, and some regulatory agencies are requiring climate change be factored in to the determination of the IDF. As a result, a means to significantly increase the hydraulic capacity of the facility quickly and economically (and in particular for spillways) was needed to improve the safety of the dam.

The large spillway capacity required to safely pass the IDFs leads engineers to explore ways to provide an economical spillway capacity for large flows with a low frequency of occurrence. RCC has the advantage of rapid and potentially lower placement costs of large volumes of concrete than CVC. These advantages make RCC an ideal candidate for construction of enlarged spillway capacities by converting existing embankments into a spillway for infrequent flood events. The dam structure then serves both as a spillway and a water retaining embankment. This method of providing a spillway with a capacity for large flows, commonly referred to as overtopping protection, was first introduced for the Fairbury Dam Hydropower project in Nebraska. The design for Fairbury Dam proposed the use of soil-cement embankment as overtopping protection to create an emergency spillway over the dam embankment. Ultimately, this project was not constructed. The overtopping protection design concept was then applied in the early 1980s at projects such as Brownwood Country Club in Texas (Figure 1-3), North Fork Toutle River in Washington (Figure 1-4), Harris Park No. 1 in Colorado (Figure 1-5) and Spring Creek Dam in Colorado (Figure 1-6), where rapid construction and/or budget constraints were driving forces in identifying alternative designs. The cost-effectiveness of RCC overtopping protection was proven in these

FIGURE 1-3. Brownwood Country Club, TX.

FIGURE 1-4. North Fork of the Toutle River, WA.

FIGURE 1-5. Harris Park No. 1, CO.

FIGURE 1-6. Spring Creek Dam, CO.

early projects where the relatively high hauling, placement, and compaction production rates yielded lower unit costs than for CVC spillways. Overtopping protection subsequently saw sporadic application in the following years with a total of 11 projects constructed in the 1980s; more than 196 spillway and overtopping projects were completed by 2021. The highest structure constructed to date is the New Creek Dam at a height of 114 feet. A list of completed overtopping protection projects is shown in Table 1-1A.

1.3 APPLICATIONS

Spillways

The most common application for RCC in hydraulic structures is to construct an overtopping protection spillway. Other applications of RCC include armoring of auxiliary spillways, grade control structures, stilling basins, and foundation support for conventional concrete (CVC) spillways.

Overtopping spillway projects generally range in height from 15 to 65 feet (with 10 projects over 65 feet) with the volume of RCC ranging from 1,000 yd^3 to 58,000 yd^3. The largest RCC overtopping project constructed to date is the Alvin J. Wirtz Dam (Figure 1-7) with an RCC volume of 160,000 yd^3. The typical project averages 35 feet high, with an average RCC volume of 8,000 yd^3, an average spillway discharge of 80 cubic feet per second (cfs) per lineal foot width of spillway, and an average overflow depth of 5 feet. Refer to section 1-4 for an analysis of trends by decade for RCC hydraulic structure projects completed between 1980 and 2020.

FIGURE 1-7. Alvin J. Wirtz Dam, TX.

Several RCC overtopping projects have roadways on a portion of the RCC. At the South Prong Dam in Texas (Figure 1-8), the roadway traverses a section of the downstream slope. At Dulce Dam in New Mexico (Figure 1-9), a roadway is located on the crest of the RCC spillway.

FIGURE 1-8. South Prong Dam, TX.

FIGURE 1-9. Dulce Dam, NM.

There are some significant differences between CVC and RCC spillways. CVC spillways consist of reinforced, air-entrained concrete placed in sections with water-stopped joints, under-drains, and anchorage to resist uplift. By contrast, RCC spillways typically consist of non-air-entrained concrete, without reinforcement, water-stopped joints or anchorage. Most RCC structures rely on mass for stability and durability with no need for reinforcement due to less shrinkage potential due to lower water to cementitious materials ratio (w/cm). RCC spillways that serve as principal spillways might consider transverse joints underlaid by a geomembrane at the joint. RCC spillways do have under-drain systems similar to CVC spillways.

RCC overtopping spillways have been designed for flood frequencies of less than 100 years with several serving as the principal spillway across all or a portion of the RCC width as with Lake Royer Dam in Maryland (Figure 1-10). However, typically for earthen dams that impound water, designing spillways with RCC is generally limited to emergency spillways with flood frequencies of 100 years or higher. RCC spillways have been used extensively in Nevada and New Mexico in stormwater detention basins. A list of completed detention basin dam projects with RCC spillways is shown in Table 1-1B. In many of the projects noted, the aggregate was obtained from the on-site alluvial gravel deposits with little processing requirements. Some of the spillways operate to direct flood runoff into the basins while others serve as emergency spillways to pass the flows downstream. The Pioneer Detention Basin in Las Vegas (Figure 1-11) used two RCC spillways to pass water into the basin and then out of the basin. When identifying the design flood frequency for the operation of any spillway, the legal liability for the owner of the facility for changes in the flooding characteristics (both upstream and downstream) is an important consideration, as well as the technical requirements of the project.

FIGURE 1-10. Lake Royer Dam, MD.

FIGURE 1-11. Pioneer Detention Basin, NV.

Erosion Protection

RCC can provide erosion protection in existing vegetative earthen auxiliary spillways. Many older dams have earthen auxiliary spillways that, when analyzed using current engineering criteria, fail due to head-cut erosion during the design storm. Some of these spillways are taken out of service with the spillway capacity replaced by RCC overtopping protection while others have used RCC to provide head-cut erosion protection. The most recent example of this application is the RCC slope protection installed below the auxiliary spillway control section at Oroville Dam in California (Figure 1-12).

FIGURE 1-12. Auxiliary spillway control section (Oroville Dam, CA).

Foundation Support

RCC can provide support for CVC spillways. It is used both for rehabilitation of older spillways and in new dam construction. For example, RCC was used during the replacement of the service spillway at Oroville Dam in California to provide foundation support (Figure 1-13). The first application of this type was at the Dolet Hills Dam in Louisiana in 1985.

FIGURE 1-13. Foundation support (Oroville Dam, CA).

TABLE 1-1A. RCC Overtopping Protection Projects

	DAM (year completed)	CITY / STATE	OWNER / ENGINEER	MAX HEIGHT (ft)	RCC VOLUME (cu yd)	MSA (in)	CEMENT + FLY ASH (lb/cu yd)	MAX UNIT DISCHARGE (cfs/ft)	MAX OVERFLOW HEIGHT (ft)	CONTRACTOR
1	Ocoee #2 (1980)	Ocoee, TN	Tennessee Valley Authority	27	4,450	–	–	–	–	
2	North Fork Toutle River (1980) Replacement service spillway	Castle Dale, WA	US Army Corps of Engineers Portland District	38	18,000	1½	500 + 0	–	8	Mountain Eng. & Const. Co. Bozeman, MT
3	Brownwood Country Club (1984)	Brownwood, TX	Brownwood Country Club Freese & Nichols	19	1,400	1½	310 Type IP	24.7	5.5	Central Plains Const. Co. Shawnee Mission, KS
4	Dolet Hills Dam (1985) Foundation Support for Concrete Spillway	De Soto Parish, LA	Southwester Electric Power Co. Freese and Nichols	–	26,120	–	160 + 64	–	–	Central Plains, KS
5	Lake Brazos Dam (1985) Foundation Support for Spillway	Waco, TX	City of Waco, TX Harza Engineering (Now MWH)	–	17,000	–	300 + 0	–	–	Young Bros. Contracting Waco, TX
6	Spring Creek (1986)	Gunnison, CO	Colorado Div. of Wildlife Morrison-Knudsen Engineers (Now URS)	53	4,840	1½	225 + 0	44.4	4.5	GEARS, Inc. Crested Butte, CO
7	Harris Park #1 (1986)	Bailey, CO	Harris Park Water & San. Dist.	18	2,300	1½	285 + 0	91	10	Pridemore Const. Co. Montrose, CO
8	Comanche Trail (1988)	Big Spring, TX	City of Big Spring Freese & Nichols	20	6,500	1½	232 + 39	60	6	Versatile Const. Co. Logan, NM
9	Addicks & Barker (1988) (#1)	Houston, TX	US Army Corps of Engineers Galveston District	48.5	28,350	1½	292	7.1	2.2	Hassell Const. & Ernst Const. Co. Houston, TX
10	Addicks & Barker (1988) (#2)	Houston, TX	US Army Corps of Engineers Galveston District	36.5	28,350	1½	244	10.7	2.2	Hassell Const. & Ernst Const. Co. Houston, TX
11	Bishop Creek #2 (1989) New Emergency Spillway	Bishop, CA	Southern California Edison So. Cal. Edison / J.M Montgomery (Now MWH)	41	4,000	1½	195 + 195	24	3	El Camino Const. Fresno, CA
12	Tellico Saddle (1989) New Emergency Spillway	Lenior City, TN	TVA	11	19,500	¾	250 + 190	–	–	
13	Boney Falls (1989)	Escanaba, MI	Mead Paper Co. Harza Engineering (Now MWH)	25	4,850	¾	217 + 165	–	6	
14	Goose Lake (1989)	Nederland, CO	City of Boulder Harza Engineering (Now MWH)	35	4,200	3	360 + 0	9.1	2.4	Nicholas Const. Co. & SLM Const. Lakewood & Grand Jct., CO
15	Comanche (1990) New spillway	Estes Park, CO	City of Greeley Morrison-Knudsen Engineers (Now URS)	46	3,500	1½	300 + 0	101	10	ASI–RCC Buena Vista, CO
16	Kemmerer City (1990)	Kemmerer, WY	City of Kemmerer Woodward-Clyde Consultants (Now URS)	31	4,100	3	439 + 0	24	3.6	Nicholas Const. Co. Lakewood, CO

TABLE 1-1A. RCC Overtopping Protection Projects (continued)

	DAM (year completed)	CITY / STATE	OWNER / ENGINEER	MAX HEIGHT (ft)	RCC VOLUME (cu yd)	MSA (in)	CEMENT + FLY ASH (lb/cu yd)	MAX UNIT DISCHARGE (cfs/ft)	MAX OVERFLOW HEIGHT (ft)	CONTRACTOR
17	Thompson Park #3 (1990)	Amarillo, TX	City of Amarillo HDR Engineering	30	2,730	1½	330 + 0	30	4.3	Versatile Const. Logan, NM
18	White Cloud (1990)	White Cloud, MI	City of White Cloud OMM Engineering	15	1,000	¾	250 + 190	–	1.5	Smalley Const. Scottville, MI
19	Ringtown #5 (1991) Combined principal and emergency spillway	Ringtown, PA	Borough of Shenandoah Gannett-Fleming	60	6,300	1½	228 + 174	56	7	Mount-Joy Const. Co. Landisville, PA
20	Saltlick (1991) Two emergency spillways	Johnstown, PA	Johnstown Water Authority Gannett-Fleming	110	11,100	1¾	117 + 125	54	6.6	Charles J. Merlo, Inc. Mineral Point, PA
21	Ashton (1991)	Ashton, ID	Pacific Power-Utah Power Black & Veatch	60	7,700	¾	300 + 100	122	12	Gilbert Western (a Kiewit Co.) Murray, UT
22	Lake Lenape (1991)	Mays Landing, NJ	Atlantic County O'Brien & Gere	17	3,050	1	295 + 0	–	3	PHA Const. Cologne, NJ
23	Goose Pasture (1991)	Breckenridge, CO	Town of Breckenridge, etc. Tipton & Kalmbach (Now Stantec)	65	4,230	1½	330 + 0	95	10	GEARS, Inc. Crested Butte, CO
24	Holmes Lake (1991)	Marshall, TX	T & P Lake, Inc. East Texas Engineering	31	2,800	2½	300 + 0	–	5	Marshall Paving Co. Marshall, TX
25	White Meadow Lake (1991)	Rockaway, NJ	White Meadow Lake Assn. O'Brien & Gere	20	1,000	1	295 + 0	–	1.4	PHA Const. Cologne, NJ
26	Butler Reservoir (1992)	Camp Gordon, GA	Fort GordonUS Army Corps of Engineers Savannah District	43	9,150	1½	223 + 162	137	13.2	Curry Contracting Co. Atlanta, GA
27	Horsethief (1992)	Rapid City, SD	Black Hills National Forest / US Forest Service, Denver	65	6,250	2	325 + 0	17	4.24	GEARS, Inc. Crested Butte, CO
28	Meadowlark Lake (1992)	Ten Sleep, WY	Bighorn National Forest / US Forest Service, Denver	28	2,550	2	325 + 0	118	10.25	ASI-RCC Buena Vista, CO
29	Philipsburg #3 (1992)	Philipsburg, PA	PA - American Water Co. O'Brien & Gere	20	1,400	1	295 + 0	14	6.9	
30	North Potato Creek (1992)	Copperhill, TN	Federal Bankruptcy Court / Dames & Moore (Now URS)	35	4,500	1½	170 + 110	340	20	Dames & Moore (Now URS) Atlanta, GA
31	Lake Diversion (1993) New emergency spillway	Wichita Falls, TX	City of Wichita Falls, etc. Biggs & Mathews	85	43,230	1½	225 + 37	316	20.4	Central Plains Const. Shawnee Mission, KS
32	Lima (1993)	Dell, MT	Beaverhead Co. Red Rock River W&S District HKM Assoc. (Now DOWL HKM)	54	14,800	2	417 + 0	61	9.3	Pete's Excavating Torrington, WY
33	Rosebud (1993)	Rosebud, SD	Rosebud Sioux Tribe Harza Engineering (Now MWH)	33	4,700	1	131 + 151	55	7	Pete's Excavating Torrington, WY
34	Umbarger (1993)	Canyon, TX	US Fish & Wildlife Service GEI Consultants	40	28,500	1½	330 + 0	216	17.5	ASI-RCC Buena Vista, CO

TABLE 1-1A. RCC Overtopping Protection Projects (continued)

	DAM (year completed)	CITY / STATE	OWNER / ENGINEER	MAX HEIGHT (ft)	RCC VOLUME (cu yd)	MSA (in)	CEMENT + FLY ASH (lb/cu yd)	MAX UNIT DISCHARGE (cfs/ft)	MAX OVERFLOW HEIGHT (ft)	CONTRACTOR
35	Ponca (1993)	Herrick, SD	Rosebud Sioux Tribe Harza Engineering (Now MWH)	35	7,700	1	200 + 170	167	16	GEARS, Inc. Crested Butte, CO
36	Lighthouse Hill (1993)	Altmar, NY	Niagara Mohawk Power O'Brien & Gere	18	4,700	1½	295 + 0	50	6.5	Tuscarara Const. Co. Pulaski, NY
37	He Dog (1994) Combined principal & emergency spillway	Paramalee, SD	Rosebud Sioux Tribe Harza Engineering (Now MWH)	45	9,500	1	200 + 170	190	17	Pete's Excavating Torrington, WY
38	Coddle Creek Dam (1994)	Cabarrus County, NC	Cabarrus Co. NC HDR Engineering	65	10,300	–	–	–	10	
39	Long Run (1994)	Lehighton, PA	Borough of Lehighton Gannett Fleming	28.5	3,100	1	250 + 150	15.6	2.5	KC Construction Co. & VFL Huntington Valley, PA
40	Lake Dorothy (1994)	Barberton, OH	PPG Industries ICF Kaiser Engineers	35	6,000	1½	197 + 142	–	4	Kokosing Const. Co. Loudenville, OH
41	South Dam #1 (1994)	St. Clairsville, OH	City of St. Clairsville Burgess & Niple	40	2,200	1	250 + 0	16	3	Beaver Excavating Canton, OH
42	Cooks Slough (1994)	Uvalde, TX	Uvalde Co. TX NRCS, TX	–	7,300	–	–	52	8	W.H. Casder Gould, AR
43	Anawalt (1994)	Anawalt, WV	WV Dept. of Natural Resources Triad Engineering	34	3,000	2	361 + 0	61	7.8	Heeter Const. Co. & Gears Spencer, WV
44	North Poudre #6 (1994)	Wellington, CO	North Poudre Irrigation Co. Smith Geotechnical	40	2,400	1	350 + 0	30	5	National Const. & Gears Boulder, CO
45	South Prong (1994)	Waxahachie, TX	Ellis Co., WC&I Dist #1 Freese & Nichols	62	49,492	1½	210 + 105 & 270 + 0	48	6.25	Central Plains Shawnee Mission, KS
46	Salado Site 10 (1994) (New)	Long Horn, TX	NRCS	–	–	–	–	155.24	14.1	ASI Construction
47	Cold Springs Dam (1995) Spillway Outlet Channel	Hermiston, OR	Bureau of Reclamation	85	17,800	–	300 + 0	–	–	Wilder Construction
48	Lake Ilo (1995)	Kildeer, ND	US Fish & Wildlife Service GEI Consultants	38	3,850	1½	312 + 0	58	7	Park Const. Co. & Gears Denver, CO
49	Lower Lake Royer (1995) Widened Principal Spillway	Fort Ritchie, MD	US Army Corps of Engineers, Baltimore District	40	10,000	1½	200 + 100	44.4	6	Kiewit Const. Co. & Gears Baltimore, MD
50	Warden Lake (1995)	Wardensville, WV	WV Dept. of Natural Resources Triad Engineering	38	3,100	1½	350 + 0	127	12	Heeter Const. Co. Spencer, WV
51	North Stamford (1995)	Stamford, CT	Stamford Water Co. Roald Haestad, Inc.	25	2,100	1½	200 + 128	22	3.8	John J. Brennan Shelton, CT
52	Big Beaver (1995)	Meeker, CO	Colorado Div. of Wildlife Boyle Engineering (Now AECOM)	92	8,600	3	325 + 0	125	10	Park Const. Co. & Gears Denver, CO

TABLE 1-1A. RCC Overtopping Protection Projects (continued)

	DAM (year completed)	CITY / STATE	OWNER / ENGINEER	MAX HEIGHT (ft)	RCC VOLUME (cu yd)	MSA (in)	CEMENT + FLY ASH (lb/cu yd)	MAX UNIT DISCHARGE (cfs/ft)	MAX OVERFLOW HEIGHT (ft)	CONTRACTOR
53	Smith Lake (1996)	Garrisonville, VA	Stafford County, VA Woodward Clyde Consultants (Now URS)	60	25,300	2	308 + 0	58	5.6	Branch Hwys. Roanoke, VA
54	Lost River Site 4 (1996) Auxiliary Spillway	Hardy Co., WV	Potoma Valley Conservation District NRCS	–	13,144	1½	350+0	198.1	17.5	Conti Enterprises Inc NJ
55	Petrolia Dam (1996)	Winnett, MT	Montana DNR MSE-HKM Engineering	59	3,300	1	450 + 0	35.4	5.4	COP Construction Co Billing, MT
56	Ochoco Dam (1996) Spillway Outlet Channel	Prineville, OR	Ochoco Irrigation District Bureau of Reclamation	155	18,500	–	427 + 0	–	–	Stiemple-Wiebelhaus
57	Lake Throckmorton (1996)	Throckmorton, TX	City of Throckmorton Hibbs & Todd	21	3,000	1½	280 + 0	–	–	Nobles Road Const. Abilene, TX
58	Tongue River (1997) Phase II	Decker, MT	Montana Dept. of Natural Resources ESA Consultants (Now Strand)	91	58,600	2	171 + 0	167	12.5	Barnard Construction Bozeman, MT
59	Hungry Mother (1997)	Marion, VA	VA Dept. of Parks Dewberry & Davis / GEI Consultants	40	16,450	1½	350 + 50	50	6.6	W&L Paving & Contracting Madison, VA
60	White Oak Dam (1997)	Marison Co. VA	Marison Co, VA NRCS/Schnabel	80	6,700	–	303 + 149	120.9	11.8	Wilkins Construction Amherst, VA
61	Douthat (1997)	Clifton Forge, VA	VA Dept. of Parks Timmors Engineering / Schnabel Engineering	45	15,000	1½	292 + 0	–	–	Branch Hwys. Roanoke, VA
62	Alvin J. Wirtz (1997)	Marble Falls, TX	Lower Colorado River Authority Freese & Nichols	105	160,000	¼	230 + 230	–	14	Barnard Construction Bozeman, MT
63	Mona (1997)	Juab County, UT	Current Co. Woodward Clyde Consultants (Now URS)	43	3,400	–	350 + 0	–	–	ASI–RCC Buena Vista, CO
64	C&O Canal No. 5 (1998)	Williamsport, MD	Corps of Engineers Dewberry and Davis/ GEI Consultants	20	3,900	–	180 + 180	–	–	C.J Merlo Mineral Point, PA
65	Dulce Lake (1998)	Dulce, NM	Jicarilla Apache Tribe Benham Holway Power Group (Now Atkins)	27	5,726	1½	325 + 0	–	–	Barnard Construction Bozeman, MT
66	Left Hand Valley (1998)	Boulder, CO	St. Vrain and Left Hand Conservancy District / Rocky Mountain Consultants (Now Tetra Tech)	45	4,920	1½	325 + 0	63.9	7.9	GEARS, Inc. Crested Butte, CO
67	Hayes Dam (1998)	Pierre, SD	Office of Schools and Public Lands / Arron Swan and Associates	26	4,620	–	198 + 132	–	–	Anderson Constructors Ft. Pierre, SD

TABLE 1-1A. RCC Overtopping Protection Projects (continued)

	DAM (year completed)	CITY / STATE	OWNER / ENGINEER	MAX HEIGHT (ft)	RCC VOLUME (cu yd)	MSA (in)	CEMENT + FLY ASH (lb/cu yd)	MAX UNIT DISCHARGE (cfs/ft)	MAX OVERFLOW HEIGHT (ft)	CONTRACTOR
68	Bear Creek (1999)	Portsmouth, OH	Ohio Department of Natural Resources Fuller, Mossbarger, Scott and May (Now Stantec)	25	3,363	1½	300 + 0	20.4	4.1	Lo-Debar Const. Newark, OH
69	Wolfden Lake (1999)	Portsmouth, OH	Ohio Department of Natural Resources Fuller, Mossbarger, Scott and May (Now Stantec)	23	2,141	1½	300 + 0	32.1	3.6	Lo-Debar Const. Newark, OH
70	McBride (1999)	Portsmouth, OH	Ohio Department of Natural Resources Fuller, Mossbarger, Scott and May (Now Stantec)	22	1,944	1½	300 + 0	20.8	2.5	Lo-Debar Const. Newark, OH
71	Robinson's Branch (1999)	Clark Township, NJ	Clark Township Schnabel Engineering	20	4,500	1½	291 + 0	55	4.7	J.A. Alexander Inc. Belleville, NJ
72	Choctaw Site No. 8A (1999) (New)	Sherman, TX	NRCS	34.5	–	–	–	100	9.7	Beaver Construction Co.
73	Lake Tholocco (2000)	Fort Rucker, AL	U.S. Army Corps of Engineers - Mobile District Kellogg Brown & Root	36	26,000	1½	275 + 50	–	6.5	Thalle Construction Mebane, NC
74	Saddle Lake (Middle Fork Str. No.1) (2000)	Hooiser National Forest, IN	Hoosier National Forest NRCS, OH	49	9,102	1½	320 + 0	–	8.1	T-C Inc. Indianapolis, IN
75	Gunnison (2000)	Gunnison, UT	Gunnison Irrigation District / Jones & DeMille Engineering	35	3,700	1½	350 + 0	81	9	Nordic Ind. Salt Lake City, UT
76	Jackson Lake (2000)	Jackson County, OH	Ohio Dept. Natural Resources BBC&M Engineers	21	3,600	1½	309 + 0	72	4.63	Lo-Debar Const. Newark, OH
77	Coal Ridge Waste (2000)	Longmont, CO	Platte Valley Irrigation Co. / Rocky Mountain Consultants (Now Tetra Tech)	28	2,300	1½	325 + 0	–	5	DeFalco-Lee Longmont, CO
78	Teter Creek (2000)	Barbour County, WV	West Virginia Dept. of Natural Resources Civil Tech Engineering	33.5	5,800	–	361 + 0	182	11.7	West Virginia Paving Grafton, WV
79	Many Farms (2000)	Many Farms, AZ	US Bureau of Indian Affairs US Bureau of Reclamation	45	6,200	1½	280 + 70	–	7.1	Barnard Construction Bozeman, MT
80	Fawell (2000)	Naperville, IL	Dupage County URS Corp.	23	9,200	1½	375 + 0	–	3.5	James Cape & Sons Racine, WI
81	Thomas Dam (2000)	City of Thomas, WV	Civil Tech Engineering	–	2,000	–	400 + 0	–	–	Alwood Contracting Co. GEARS, Inc.
82	Bunnell Pond (2000)	Bridgeport, CT	State of Connecticut Milone & MacBroom	30	10,000	1	225 + 0	–	–	D V Morin Construction Meriden, CT
83	Black Rock (2001)	Zuni, NM	Pueblo of Zuni GEI Consultants	79	18,000	–	260 + 0	–	–	Laguna Construction Company Laguna, NM
84	Lake Blalock (2001)	Spartanburg, SC	Spartanburg Water System Black & Veatch	70	27,200	–	268 + 53	–	–	Thalle Construction Company Hillsborough, NC

TABLE 1-1A. RCC Overtopping Protection Projects (continued)

	DAM (year completed)	CITY / STATE	OWNER / ENGINEER	MAX HEIGHT (ft)	RCC VOLUME (cu yd)	MSA (in)	CEMENT + FLY ASH (lb/cu yd)	MAX UNIT DISCHARGE (cfs/ft)	MAX OVERFLOW HEIGHT (ft)	CONTRACTOR
85	Leyden (2001)	Arvada, CO	City of Arvada, CO URS Corp.	43	8,900	1½	425 + 0	92	8.4	ASI RCC Buena Vista, CO
86	McKinney (2001)	Hoffman, NC	N.C. Wildlife Resource Commission URS Corp & Schnabel Engineering	17	1,615	1½	450 + 0	47	5	Atlas Resource Management Fayetteville, NC
87	Vesuvius (2001)	Ironton, OH	U.S. Forest Service Bureau of Reclamation	45	10,000	1	360 + 0	35	5.7	T C Inc. Indianapolis, IN
88	Potato Creek No. 6 (2002)	Thomaston, GA	Upson Co. and Towaliga River Soil & Water Conservation Dist. Golder Associates	26	4,770	1½	375 + 0	74	7.3	DPS Ind. Marietta, GA
89	Misteguay No. 4 (2002)	Saginow, MI	Misteguay Creek Intercounty Drain Board Spicer Group, Inc.	39	6,575	1½	375 + 73	120.6	9.3	Champagne and Marx Excavating Saginaw, MI
90	Caldwell Lake (2002)	Chillicothe, OH	Ohio Dept. of Natural Resources / Bowser-Morner & Assoc.	35.5	5,675	1½	303 + 0	33.4	3.8	Maiden & Jenkins Construction Co. Nelsonville, OH
91	Standly Lake Dam (2002) Spillway Channel Protection	Westminster, CO	CH2M Hill (Now Jacobs)	90	26,000	–	–	–	–	ASI
92	Great Gorge (2002)	McAfee, NJ	Great Gorge Resort, Inc. / Schnabel Engineering for Schoor DePalma	35	1,400	¾	300 + 0	10	2.4	Van Peenen Contractors, Inc. Wayne, NJ
93	East Fork Above Lavon 1A (2003)	Mckinney, TX	Collin County SWCD NRCS	44.5	2,953	–	–	54	6.4	Beaver Contracting
94	Stonelick Lake (2003)	Newtonsville, OH	Ohio DNR / Bowser-Morner & Assoc.	29	5,360	2	330 + 0	167	7.7	Lo–Debar Const. Newark, OH
95	Yellow River Y–14 (2003)	Lawrenceville, GA	Gwinett County, GA Golder Associates	39.5	4,850	1½	250 + 250	82	7.4	Thalle Construction Hillsborough, NC
96	Willowdale Lake (2003)	Akron, OH	Willowdale Homeowners Assoc. Burgess Niple	27.3	2,500	1	300 + 0	–	7.5	Great Lakes Const. Co. Gears, Inc.
97	Sweet Arrow (2003)	Pine Grove, PA	Schuykill County WJP Engineers	33.5	8,257	1	–	–	7	K.C. Construction Co. Ivyland, PA
98	Lake Hauto (2003)	Nesquehoning, PA	Lake Hauto Homeowners Assoc. O'Brian & Gere	–	15,000	–	–	–	–	No. 1 Construction Co. Ashley, PA
99	Tanglewood Lake (2003)	Geauga, OH	Homeowners Assoc. BBC&M Engineers	37.4	4,000	1	300 + 0	104.4	7	C J Natale, Inc. Hudson, OH
100	Paulins Kill (2003)	Stillwater, NJ	Community of Stillwater / Malcolm Pirnie (Now ARCADIS)	13	2,500	–	–	–	–	Ritacco Construction Belleville, NJ
101	East Fork Above Lavon 3C (2003)	McKinney, TX	Collin County SWCD M & E Engineering	44.5	3,302	–	–	42	6	Jester Brothers Const. Whitewright, TX
102	Hackberry Draw 1 (2003) (auxiliary spillway)	Carlsbad, NM	Hackberry Draw Watershed Board NRCS NM	–	13,055	–	157 + 78	–	–	

TABLE 1-1A. RCC Overtopping Protection Projects (continued)

	DAM (year completed)	CITY / STATE	OWNER / ENGINEER	MAX HEIGHT (ft)	RCC VOLUME (cu yd)	MSA (in)	CEMENT + FLY ASH (lb/cu yd)	MAX UNIT DISCHARGE (cfs/ft)	MAX OVERFLOW HEIGHT (ft)	CONTRACTOR
103	Brunswick Lake (2004)	Brunswick, OH	City of Brunswick, OH MS Consultants	16.4	2,320	2	–	35.9	4.1	Lo-Debar Const. Newark, OH
104	Bear Creek # 11 (2004) (auxiliary spillway)	Goldsboro, NC	Wayne Co. Drainage District #1 / NRCS	23	2,538	1½	210	7.5	1.5	Thalle Construction Hillsborough, NC
105	Bear Creek #12 (2004) (auxiliary spillway)	Goldsboro, NC	Wayne Co. Drainage District #1 / NRCS	19	885	1½	210	7.5	1.5	Thalle Construction Hillsborough, NC
106	Salado Site 15R (2004) (New)	San Antonio, TX	NRCS	45	22,516	–	175 + 100	111.4	11	E.E. Hood and Sons
107	Lyman Run Dam (2005) Foundation Support for Labyrinth Spillway	West Branch Township, PA	Lyman Run State Park Gannett–Fleming	50	16,100	–	125 + 275	140	16.5	Allen Meyers / Peltz - Gears
108	Yellow River #17 (2005)	Gwinnett County, GA	Gwinnett County, GA USACE - Savannah District + Golder Assoc.	30	6,700	1½	250 + 80	34.5	7	ASI Constructors Pueblo West, CO
109	Marrowbone Dam Site 1 (2005)	Ridgeway, VA Henry Co.	NRCS, Virginia Schnabel Engineering	46	10,600	1½	330 + 80	143.4	12.9	ASI Constructors Pueblo West, CO
110	Locust Lake (2005)	Hope, NJ	John P. Neufville Schnabel Engineering for French & Perillo	25	1,600	¾	350 + 0	33	4.8	GEARS, Inc. Colorado Springs, CO
111	Bradford Dam No. 2 (Marilla) (2007)	Bradford, PA	Bradford City Water Authority GAI Consultants	41	8,500	1	400 + 0	26.7	2.9	Bob Cummins Const. Bradford, PA
112	Deegan Dam (2007)	Bridgeport, WV	City of Bridgeport, WV Civil Tech Engineering	32	2,230	1	360 + 0	50	4.9	Kanawba Stone, Inc. Poca, WV
113	Hinkle Dam (2007)	Bridgeport, WV	City of Bridgeport, WV Civil Tech Engineering	20	2,000	1	360 + 0	47	5.7	Kanawba Stone, Inc. Poca, WV
114	Yellow River Y15 (2007)	Gwinnett County, GA	Gwinnett County, GA Golder Assoc.	36	12,560	1½	250 + 35	80	8	ASI Constructors Pueblo West, CO
115	Yellow River Y16 (2007)	Gwinnett County, GA	Gwinnett County, GA Schnabel Engineering	34	3,000	1½	250 + 80	–	8.9	ASI Constructors Pueblo West, CO
116	Poe Valley (2008)	Centre County, PA	Pennsylvania Dept. of Cons. & Natural Resources Schnabel Engineering	30	15,600	1½	400 + 0	35	5	Jay Fulkroad & Sons, Inc. McAlisterville, PA
117	Lake Wanahoo Dam (2009)	Wahoo, NE	Lower Platte NRD Olsson Engineers	53	17,630	–	340 + 0	75	7.5	Commercial Contractors Lincoln, NE
118	Sugar L43 Dam (2009) Hard Point on Crest	Washita, OK	West Caddo Conservation District NRCS	60	2,744	1	400 + 0	81.2	8.7	Southern Rock Equipment Inc.
119	Sallisaw Creek Site 16 (2009)	Stilwell, OK	Adair County Conservation District NRCS Oklahoma	47	6,111	1½	206 + 69	55	6.8	C. Watts and Sons Construction Oklahoma City, OK
120	Cobb Site 1 (2009) (auxiliary spillway)	Colony, OK	Deer Creek Conservation District Schnabel Engineering	80	22,100	1	222 + 80	162.2	14.3	Total Investment Company/ASI Sub
121	North Fork Dam (2010) (auxiliary spillway)	Potter County, PA	Potter County NRCS, PA	55.9	3,949	–	250 + 0	109.7	11.4	KC Construction Ivyland, PA

TABLE 1-1A. RCC Overtopping Protection Projects (continued)

	DAM (year completed)	CITY / STATE	OWNER / ENGINEER	MAX HEIGHT (ft)	RCC VOLUME (cu yd)	MSA (in)	CEMENT + FLY ASH (lb/cu yd)	MAX UNIT DISCHARGE (cfs/ft)	MAX OVERFLOW HEIGHT (ft)	CONTRACTOR
122	Fawn Lake Dam (2010)	Wayne Township, PA	Wayne Township WJP Engineers	44	6,500	1½	–	–	5	KC Construction Ivyland, PA
123	Thorn Run (2010)	Butler, PA	PA American Water Co. / Gannett-Fleming	42	14,070	1½	200 + 200	54.3	7	Joseph B. Fay & ASI Tarentum, PA
124	Lake Solitude Dam (2011)	High Bridge, NJ	Borough of High Bridge Schnabel/French & Parello	55	2,300	1	350 + 0	–	–	Kyle Conti Construction Hillsbourgh, NJ
125	Roaming Woods Lake Dam (2011)	Lake Ariel, Wayne Co., PA	Hideout Community WJP Engineers and Hawk Engineering	26	6,000	1½	350	–	5	KC Construction Ivyland, PA
126	Bear Creek (2011)	Wise, VA	Town of Wise, VA Schnabel Engineering for Thompson & Litton	45	5,400	2	250 + 150	59	6.1	Estes Brothers Jonesville, VA
127	Big Haynes Brushy Fork #3 H3 (2011)	Grayson, GA	Gwinnett County, GA Golder Associates	30	4,000	1½	275 + 150	46	6	ASI Constructors Preblo West, CO
128	Cabresto Dam (2011)	Taoc Co., NM	Cabresto Lake and Llano Irrigation Community Ditch Associations / RJH	70	6,784	¾	–	195	10.4	ASI Constructors Pueblo, CO
129	Dutch Fork (2011)	Donegal Township, PA	Pennsylvania Dept. of Cons. & Natural Resources Michael Baker	42	7,500	1	400 + 0	164	10.4	Golden Triangle Imperial, PA
130	Fox Creek #4 (2011)	Flemingsburg, KY	Fox Creek Watershed Cons. District Schnabel Engineering	49	11,000	1½	200 + 200	128	9.8	Joseph B. Faye Russelton, PA
131	Stoney Creek Site 9 (2011)	Bedford, VA	City of Bedford, VA Schnabel Engineering for Thompson & Litton	56	10,000	2	250 + 150	76	9	Morgan Corporation Spartanburg, SC
132	T Nelson Elliott (2011)	Manassas, VA	City of Manassas, VA URS Corp.	74	8,580	1½	350 + 0	90	9.5	ASI Constructors Pueblo West, CO
133	Willow Crest Dam (2011)	Stillwater Township, NJ	Stillwater Township Civil Dynamics, Inc. (Now GZA GerEnvironmental, Inc.)	17	3,000	–	470 + 0	50	9	Concrete Construction Corp.
134	Wisecarver (2011)	Waynesburg, PA	Southwestern Pennsylvania Water Authority D'Appolonia	40	–	–	–	–	–	
135	Lake Oneida (2012)	Butler Co., PA	Pennsylvania American Water Schnabel Engineering	33	14,150	1	228 + 123	60	10	KC Construction Co. Ivyland, PA
136	Lower Owl Creek (2012)	Tamaqua, PA	Pennsylvania Fish & Boat Comm. Schnabel Engineering for Alfred Benesch & Co.	33	3,000	1½	450(1S) + 0	48	4.9	Performance Construction Services Harrisburg, PA
137	Berwind Dam (2012)	McDowell County, WV	Berwind Wildlife Management Area Civil Tech Engineering	36	2,530	–	405 + 0	122	6.1	Green Mountain Co. Charleston, WV
138	Nesbitt (2012)	Lackawana County, PA	Pensylvania American Water Gannett Fleming	101	38,000	1½	200 + 200	120.5	12.58	ASI Constructors Pueblo West, CO

TABLE 1-1A. RCC Overtopping Protection Projects (continued)

	DAM (year completed)	CITY / STATE	OWNER / ENGINEER	MAX HEIGHT (ft)	RCC VOLUME (cu yd)	MSA (in)	CEMENT + FLY ASH (lb/cu yd)	MAX UNIT DISCHARGE (cfs/ft)	MAX OVERFLOW HEIGHT (ft)	CONTRACTOR
139	West Reservoir (2012)	Akron, OH	Ohio Department of Natural Resources DLZ Ohio, Inc	21.5	2,400	2	250 + 120	9	5.2	Kenmore Construction Co. Akron, OH
140	Caney Coon Site 2 (2013)	Coalgate, OK	City of Coalgate and Coal County Conservation District URS	53.4	8,820	1½	235 + 79	86.69	10	Wynn Construction Oklahoma City, OK
141	Sallisaw Creek Site 26 (2013) (auxiliary spillway)	Creasy, OK	Creasy, OK NRCS	71.8	7,558	1½	195 + 63	31.8	10.8	WYNN Construction Gears, Inc.
142	Jefferson Lake Dam (2013)	Jefferson County, OH	Jefferson Lake State Park / BBC&M Engineers Now SME	–	1,500	–	–	–	–	Golden Triangle Construction Imperial, PA
143	New Creek #14 (2013)	Keyser, WV	NRCS - West Virginia Gannett Fleming	114	26,000	1½	200 + 200	140	13	Heeter Construction/ASI RCC Spencer, WV
144	Mountain Creek #10 (2014)	Midlothian, TX	Dalworth S & W Conservation District and Ellis County / NRCS - Texas	46	11,974	–	230 + 60	92.2	10	ASI Constructors Pueblo West, CO
145	Pike Lake (2014)	Benton Township, OH	Ohio Department of Natural Resources Michael Baker	22	6,720	1	300	26	3.2	Trucco Construction Delaware, OH
146	Pond Lick Lake (2014)	Nile Township, OH	Ohio Department of Natural Resources Michael Baker	25	5,355	1	310	33	6	Sunesis Construction Co. West Chester, OH
147	Renwick (2014) (RCC spillway and RCC road)	Akra, ND	Pembina Co Water Resource District NRCS North Dakota	49	19,718	1½	377 + 94	110	11.1	RSCI Group Meridian, ID (Prime) Meridian, ID
148	Roosevelt Lake (2014)	Nile Township, OH	Ohio Department of Natural Resources Michael Baker	19.5	4,765	1	310	85	8.9	Sunesis Construction Co. West Chester, OH
149	Tuscarawas River Diversion (2014)	South Akron, OH	Ohio Department of Natural Resources DLZ Ohio Inc.	29	12,500	1½	250 + 120	21	14.3	Kenmore Construction Co.
150	Speedwell Forge (2015)	Lititz, PA	Pennsylvania Fish & Boat Comm. Schnabel Engineering	36	10,800	1	315 + 0	80	9.5	KC Construction Co. Ivyland, PA
151	Cherokee Dam (2015)	Jefferson City, TN	Jefferson City TVA	–	22,815	–	205 + 135	–	–	Phillips and Jordan Gears, Inc.
152	Bradford Dam No. 2 (Gilbert) (2015)	City of Bradford, PA	City of Bradford GAI Consultants	40	13,500	–	300 + 0	31.1	4.7	Bob Cummings Construction Gears Inc.
153	Colyer Lake (2015)	Centre County, PA	Pennsylvania Fish & Boat Comm. Schnabel Engineering	42	10,300	1	325 + 0	43	6	Performance Construction Services Harrisburg, PA
154	Soque River #29 (2015)	Habersham County, GA	NRCS - Georgia Schnabel Engineering	54	3,000	1½	340 + 0	62.6	7.1	Phillips & Jordan Knoxville, TN
155	Soque River #34 (2015)	Habersham County, GA	NRCS - Georgia Golder Associates	58.6	7,950	1½	340 + 0	72	8	Phillips & Jordan Knoxville, TN

TABLE 1-1A. RCC Overtopping Protection Projects (continued)

	DAM (year completed)	CITY / STATE	OWNER / ENGINEER	MAX HEIGHT (ft)	RCC VOLUME (cu yd)	MSA (in)	CEMENT + FLY ASH (lb/cu yd)	MAX UNIT DISCHARGE (cfs/ft)	MAX OVERFLOW HEIGHT (ft)	CONTRACTOR
156	Soque River #36 (2015)	Habersham County, GA	NRCS - Georgia Schnabel Engineering	39.4	2,600	1½	340 + 0	65.8	6.6	Phillips & Jordan Knoxville, TN
157	Cacapon Resort State Park Dam (2016)	Berkely Springs, WV	Cacapon Resort State Park Dam Civil Tech Engineering	28.5	2,570	–	320 + 0	77	5.4	Heeter Construction Gears Inc.
158	Lunga (2016)	Quanfico, VA	US Marine Corps Schnabel Engineering	53	8,700	1	281 + 94	45	–	ASI Constructors Pueblo West, CO
159	Lake Laura (2016)	Bayse, VA	Virginia Dept of Conservation and Recreation Schnabel Engineering	80	15,500	1	210 + 90	80	9	ASI Constructors Pueblo West, CO
160	Lake White (2017)	Waverly, OH	Ohio Dept of Transportation, Ohio Dept. of Natural Resources Stantec	39	114,000	1	225 + 100	22.1	4.4	Sunesis Construction West Chester, OH
161	Honey Lake Dam (2017)	Hopewell Township, NJ	Honey Lake Association French & Parello	22	6,025	3/4	350 + 0	–	–	KC Construction Ivyland, PA
162	Olmitos – Garcia Site 7 (2017)	Rio Grande, TX	Starr County SWCD and Starr Co. M & E Consultants	45	9,209	1½	210 + 85	122	9	Heater Construction Mt. Morris, PA
163	Elm Fork 7A (2017)	Muenster, TX	Upper Elm Red SWCD and Cooke Co. M&E Consultants	51	6,902	2	258 + 160	47	6.4	Accelerated Critical Path Piano, TX
164	Kensington Mine Tailing Dam (2018)	Juneau, AK	Coeur Alsaka Inc Golder Associates USA		3,400	1/1/2002	294 + 126 (slag)	–	3.3	Alaska Aggregate Products
165	EFAL Site 4R (2018)	McKinny, TX	Collin CO SWCD NRCS	46	11,110	–	–	47.5	7.5	DDM Construction
166	Donegal Lake Dam (2019)	Stahlstown, PA	PA Fish and Boat Commission Michael Baker Inlt.		6,000		275			KC Construction Ivyland, PA
167	Chapman Lake Dam (2019)	Warren Co, PA	PA DCNR Michael Baker Inlte		12,000		275			KC Construction Ivyland, PA
168	Sylvan Lake Dam (2019)	Eagle, CO	Colorado Division of Parks & Wildlife Tetra Tech	26	8,160	1½	254 + 109	29	4.5	ASI Construction Colorado Springs, CO
169	Garden of the Gods Dam (2019)	Colorado Springs, CO	City of Colorado Springs, CO Wilson & Company	30	6,640	1	334 + 123	34	5	Mortenson Denver, CO
170	Upper Decker Creek Dam 1 (2020)	Preston Co., WV	Monongahela Conservation District Gannett Fleming	55	17,000	1½	200 + 200	95	10	Triton Construction
171	Oroville Dam (2020) Auxiliary Spillway + Spillway dental	Oroville, CA	CA Department of Water Resources Stantec	–	1,050,000	1½	175 + 175	242.8	–	Kiewit Construction
172	Beaverdam Creek Dam (2021)	PA	Chester County Water Authority/NRCS Gannett Fleming	35	4,500	1½	200 + 200	63	7.6	KC Construction Ivyland, PA

TABLE 1-1A. RCC Overtopping Protection Projects (continued)

	DAM (year completed)	CITY / STATE	OWNER / ENGINEER	MAX HEIGHT (ft)	RCC VOLUME (cu yd)	MSA (in)	CEMENT + FLY ASH (lb/cu yd)	MAX UNIT DISCHARGE (cfs/ft)	MAX OVERFLOW HEIGHT (ft)	CONTRACTOR
173	Leon Hurse Dam (New) (2021)	Ladonia, TX	Upper Trinity Regional Water District Freese & Nichols	108	136,000	1½	–	–	–	Granite Construction
174	Plum Creek Site 10 (2021)	Buda, Hays Co., TX	Plum CreekConservation District, Caldwell Travis SWCD, Hays Co M&E Consultants	36	4,774	–	280 + 0	36	5.4	Solid Bridge Construction New Waverly, TX
175	Rawson Hill Brook Dam (2022)	Shrewsbury, MA	MA DCR Gannett Fleming	16	2,800	1½	195 + 130 (slag)	20	3.5	KC Construction Ivyland, PA

TABLE 1-1B. Detention Basin Dams with RCC Spillways

	DAM (year completed)	CITY / STATE	OWNER / ENGINEER	MAX HEIGHT (ft)	RCC VOLUME (cu yd)	MSA (in)	CEMENT + FLY ASH (lb/cu yd)	MAX UNIT DISCHARGE (cfs/ft)	MAX OVERFLOW HEIGHT (ft)	CONTRACTOR
176	Upper Las Vegas Wash (New) (1993)	City of North Las Vegas, NV	Clark Co Black and Veatch	49	38,000	1½	–	242	18	Granite Construction
177	Kyle Canyon Inlet Structure 3 (New) (1995)	Clark Co., NV	North Las Vegas VTN	54	128,000	–	290 + 62	8.63	8	3 Inlet spillways and 1 1,390 ft outlet
178	Hiho Springs (New) (1996)	Clark Co., NV	Clark Co., NV Black & Veatch	82	52,700	–	228 + 114	–	7.5	550 ft wide spillway
179	Lauson Detention (New) (1998)	Anthony, NM	Dona Ana Flood Control Commission Leedshill - Herkenhoff (Now NV5)	22	–	–	–	22.8	3.8	C. S. McCrossan Construction
180	Breedlove Detention (New) (1998)	Anthony, NM	Dona Ana Flood Control Commission Leedshill - Herkenhoff (Now NV5)	24	–	–	–	16.9	2.9	C. S. McCrossan Construction
181	North Virginia Street Detention (New) (1998)	NV		–	–	–	–	–	–	
182	Anthem No. 2 (New) (1998)	Las Vegas, NV	Del Webb G.C. Wallace	–	–	–	–	38.96	5.5	
183	Windmill Wash Detention Dam (New) (1999)	Bunkerville, NV	Clark Co. NV CH2M Hill	50	38,925	–	298 + 149	86.5	8	
184	Blue Diamond Detention (New) (1999)	Las Vegas, NV	Clark Co. COE	61	75,864	1	270 + 68	80.8	7.5	American Asphalt & Grading Co. Las Vegas, NV
185	Red Rock Dam (New) (2000)	Las Vegas, NV	Clark Co. COE	63	–	–	–	–	–	
186	Oakhill Detention Dam (New) (2000)	Oakhill, TX Near Coverbride Dr		–	–	–	–	–	–	
187	Borrega Dam (New) (2001)	Albuquerque, NM	AMAFCA Wilson & Company	13	5,406	1	330 + 0	43	5	Chava Trucking Albuquerque, NM

TABLE 1-1B. Detention Basin Dams with RCC Spillways (continued)

	DAM (year completed)	CITY / STATE	OWNER / ENGINEER	MAX HEIGHT (ft)	RCC VOLUME (cu yd)	MSA (in)	CEMENT + FLY ASH (lb/cu yd)	MAX UNIT DISCHARGE (cfs/ft)	MAX OVERFLOW HEIGHT (ft)	CONTRACTOR
188	Pioneer Detention Dam (New) (2001)	City of Henderson, NV	City of Henderson PBS&J now Atkins	–	–	–	–	–	–	
189	Ann Road CAM 10 (New) (2002)	Las Vagas, NV	Las Vegas Louis Berger Group now WSP	–	35,510	2	395 + 107	17.92	3.45	Mix referred to high strength soil cement
190	McCuller Detention Dam (New) (2003)	NV		–	–	–	–	–	–	
191	Upper Duck Creek (New) (2004)	Las Vegas, NV	Clark Co Tetra Tech	54	65,140	–	230 + 110	–	10	Diamond Construction
192	Little Puerco Dam (2005)	Gallop, NM	Hale Co. SWCD and Hale Co.							
193	Indian Springs (New) (2004)	Clark Co. NV	Clark Co. VTN	30	11,400	1½	300 + 64	52.07	5.64	
194	R-4 Detention (New) (2006)	Las, Vegas, NV	Clark Co., NV COE LA District/ Montgomey Watson	44.3	27,415	–	269 + 68	37.12	5.75	
195	Boca Negra Dam (New) (2014)	Albuquerque, NM	AMAFCA Wilson & Company	26	6,290	1	330 + 0	110	9	Salls Brothers Const. Albuquerque, NM
196	Santa Ana Detention (New) (2014)	Santa Ana Pueblo, NM	Pueblo of Santa Ana Kleinfelder	22.7	5,500	1	140 + 160	–	3.2	

Note: where "(New)" appears next to the dam name, indicates that the RCC spillway was part of the design for a new structure (not as a feature added to an existing structure).

Some spillways have CVC control sections and have used RCC to protect the outlet channel. Standley Lake Dam in Colorado used RCC just below a CVC labyrinth control section and then protected the outlet channel with seven RCC drop structures (Figure 1-14), while at Cold Springs Dam in Oregon (Figure 1-15) RCC protects the outlet channel slopes and invert.

FIGURE 1-14. RCC drop structure in outlet channel (Standley Lake Dam, CO).

FIGURE 1-15. Erosion protection (Cold Springs Dam, OR).

1.4 ESTIMATING CONSTRUCTION COSTS FOR RCC HYDRAULIC STRUCTURES

The use of RCC continues to be one of the most widely accepted materials for new or rehabilitated hydraulic structures. Increasing spillway capacity with overtopping protection, providing foundation support for CVC spillways, erosion protection in auxiliary spillways, or in the protection of spillway outlet channels are some of the uses of RCC.

One of the main reasons RCC is selected for a project is its typically low unit cost relative to CVC. However, many variables can influence the RCC costs. Material costs include the aggregate, cement, possible fly ash, water, and sometimes additives for air entrainment and set retarders. The source of the aggregate can play a significant factor in the unit cost. Small projects can use an all-in-one aggregate such as a road base material. The gradation of this material may require additional cementitious materials due to lack of fines or gap grading but the additional cementitious costs may be offset by the less expensive aggregate costs. Larger projects may use a two aggregate blend where the gradation can be optimized, keeping the amount of cementitious material lower. Designers should investigate aggregate sources located near the project site to determine the best approach for the RCC mix design.

Many of the RCC projects in Table 1-1B from New Mexico and Nevada were new detention basin projects. The basins were typically located on outwash alluvial fans, and therefore the on-site gravel deposits were used as the RCC aggregate with minimal processing. The final gradations might not allow for the most efficient use of the cementitious material, but the total costs of the RCC can be kept low. Some designers use the term high strength soil-cement for these projects.

Other factors can play a significant role in the RCC costs. Some projects might require a certain type of cement such as low alkali or sulfate resistance or use of supplementary cementing materials such as fly ash, slag, or natural pozzolans. The necessity and availability of these special cements should be investigated early in the planning stage. The forming of spillway steps and training walls can be time consuming and impact RCC production if not planned for ahead of time.

For estimating purposes, it is suggested that engineers use their best judgment in using the cost curve considering any special conditions of the project or site constraints that would have a large impact on prices.

1.5 PROJECT DATA ANALYSIS AND TRENDS

An analysis was performed on the RCC project data presented in Table 1-1 to provide information on the overall trends of RCC applications for hydraulic structures through four decades from 1980 to 2020. The results of the analysis are presented below.

Completed Projects – 196 RCC spillway and overtopping protection projects were known to be completed in the U.S. between 1980 and 2020. Fourteen projects were completed in the 1980s. The 1990s were the most productive decade with 67 projects completed. The 2000s and 2010s saw 57 and 55 projects completed, respectively. The most productive year was 2000 with 13 projects completed, and there were three years (1981, 1982, and 1987) where no projects were completed. There has been at least one project completed in every year since 1987.

The average number of projects completed per year for all four decades is 4.8, which is the same as the average for the 2010s. One prediction for the future 2020s decade is that the average number of projects completed per year will be similar to the average of the 2010s. Another possibility for the 2020s decade, and perhaps more likely, is that growth over the previous decade will occur due to the persisting issue of inadequate spillway capacity on U.S. dams and the potential for increased funding opportunities.

Maximum Height – The average maximum height of projects completed between 1980 and 2020 is 43 feet, with a median of 39 feet. The New Creek Dam No. 14 in West Virginia completed in 2013 was the tallest at 114 feet, and the Telico Saddle Dam project was the shortest at 11 feet completed in 1989. The average height of projects in each decade ranged between 31 feet (1980s) to 47 feet (1990s). The 11-foot high Telico Saddle Dam project was built in the 1980s, and by the 2010s the smallest maximum height was 16 feet. The increase in the minimum height of a project may be related to the smallest total volume of RCC that makes a given project economically viable.

Total RCC Volume – There is a wide gap between the largest and second largest total RCC volume for projects; Oroville Dam used 1,050,000 cubic yards (CY) in 2018, and Alvin J. Wirtz dam used 160,000 CY in 1997. The median total volume of RCC used on projects between 1980 and 2020 is 6,600 CY. The average volume used per project by decade ranged between 10,150 CY (2000s) to 16,400 CY (1990s), and the minimum volume ranged from 880 CY (2000s) to 1,500 CY (2010s).

Maximum Size Aggregate (MSA) – The maximum size aggregate (MSA) used in RCC mixtures between 1980 and 2020 was an average of 2.0 inches and a median of 2.0 inches, with a maximum of 3 inches and a minimum of 0.25 inches. The average MSA decreased in each successive decade from a high of 3.0 inches in the 1980s to a low of 1.8 inches in the 2010s. The range of MSA was larger in the 1980s (3 inches to 0.75 inches) and 1990s (3 inches to 0.25 inches), and a consistent range emerged and persisted through the 2000s and 2010s with MSA between 2 inches and 0.75 inches.

Total Cementitious Material Content – The average cementitious materials content, which consists of the total amount of portland cement plus pozzolan such as fly ash, used for RCC mixtures has increased each decade since 1980. The average cementitious content was 325 pounds per cubic yard (LB/CY) in the 1980s and increased to 337, 344 and 352 LB/CY for each of the three successive decades through the 2010s. The maximum total cementitious content in each decade ranged from 460 LB/CY in the 1990s to 502 LB/CY in the 2000s. The minimum total cementitious content for the 1980s, 1990s and 2000s ranged from 171 to 224 LB/CY, and increased to 258 LB/CY in 2010. The increase in the total cementitious content in the 2010s may be representative of the better understanding of the benefits that higher total cementitious content provides to avoid the biggest challenges of RCC applications. The improved performance benefits include reducing segregation, improving freeze/thaw resistance, and enhancing surface abrasion resistance.

Maximum Unit Discharge – The average maximum unit discharge per foot-length of spillway for projects completed between 1980 and 2020 averaged 75 cubic feet per second per foot (cfs/ft) with a median of 58 cfs/ft. The largest maximum unit discharge of 340 cfs/ft is the North Potato Creek Dam completed in 1992, and the minimum of 7 cfs/ft is Addicks and Barker #1 completed in 1988. The average maximum unit discharge per decade ranged from 34 cfs/ft in the 1980s to 87 cfs/ft in the 1990s, and the minimum ranged from 7 cfs/ft in the 1980s to 9 cfs/ft in the 2010s.

Maximum Overflow Height – The average maximum overflow height is 7.5 feet for projects completed between 1980 and 2020, and the median was 7 feet. The largest maximum overflow height of 20 feet is the North Potato Creek Dam completed in 1992, which also has the largest maximum unit discharge. The smallest maximum overflow height of 1 foot is the White Meadow Lake dam completed in 1991. The average maximum overflow height per decade ranges from 5 feet in the 1980s to 7.8 feet in the 2010s.

CHAPTER 2

LOCATION AND OPERATIONAL REQUIREMENTS

2.1 GENERAL

The use of RCC in different types of hydraulic structures can have its own operational limitations and preferred locations. This chapter discusses several of the more common hydraulic structures where RCC is used, including overtopping protection, principal spillways, auxiliary spillways, and outlet channels.

Embankment overtopping protection has been found to be a practical and cost-effective method for providing additional spillway capacity to convey infrequent floods at existing dams with inadequate spillway capacity. Dam designers and dam safety officials have accepted overtopping spillways for embankment dams as an effective design method of adding auxiliary spillway capacity. When planning to use overtopping protection as an auxiliary spillway, the designer should consider the limitations and risks of conveying spillway flow over an earth embankment. Important engineering design considerations that should be evaluated include:

- The potential for an overtopping protection configuration to introduce significant quantities of concentrated flowing water over erodible materials such as an earthen embankment or foundation material at the dam toe or abutment contacts.
- Embankment overtopping protection has the inherent risk that uncontrolled leakage from the spillway could cause embankment erosion. Therefore, preference should be given to alternatives that will locate the spillway off the dam embankment and onto a rock foundation.
- Overtopping protection should not be considered as a low-cost substitute for a service spillway, especially where frequent use, high unit discharge, or high head is a design requirement or the structure impounds a substantial volume of water.
- Overtopping protection typically involves a significant change to the visual appearance of the structure. RCC overtopping protection can change a grass-covered embankment to a concrete-covered surface. In addition, RCC can have a rough, unfinished appearance when compared to CVC. Some consider the rough surface of RCC to be visually more appealing than CVC, although RCC mixes with higher cementitious quantities and non-plastic aggregate fines can be used to achieve the appearance of CVC. Some projects are covered with topsoil and vegetation to cover the RCC. A couple of RCC projects were stained to blend in with the colors of the local environment. Education of owners and the public regarding the esthetics of RCC is important.
- Numerous overtopping protection projects have been constructed, but few have seen significant use and have not been tested for full design flood conditions.
- There is the risk that debris carried in the flood flows such as a bed loading consisting of gravels and cobbles will impact or erode the overtopping protection if not accounted for in the design. Chapter 10 discusses the performance of RCC structures over the last 40 years.

RCC is less commonly used on principal spillways of dams. In certain cases where unit discharge and head are low, the use of RCC for principal spillways may be appropriate. RCC use in principal spillways has many of the same issues as overtopping protection. Because they will be operating on a more frequent basis, special attention should be paid to durability for good long-term performance. Higher compressive strengths should be considered, as well as using air entrainment where required by geographic conditions to guard against freeze/thaw damage. Installation of joints may be necessary for wide and/or long spillways so that appropriate drainage systems can be incorporated beneath the RCC.

RCC has been used to armor earthen auxiliary spillways and outlet channels on dams to prevent head cutting and channel erosion. Generally, the RCC plating method is used because of the mild slopes, except for the side slopes and any grade control structures where horizontal lifts are still used. At the terminus of the RCC, cutoff walls are used to prevent under cutting of the RCC section. Refer to section 6.4 for details about the RCC plating method.

2.2 OPERATION FREQUENCY AND SPILLWAY LOCATION

Overtopping Protection

RCC overtopping protection structures have been designed as principal spillways, such as Butler Dam in Georgia (Figure 2-1), and in-stream drop structures and auxiliary spillway armoring, such as Cooks Slough in Texas (Figure 2-2). However, most embankment overtopping protection projects function as auxiliary spillways and have principal spillways designed to pass the more frequent floods. It is particularly important that

FIGURE 2-1. Principal spillway (Butler Dam, GA).

FIGURE 2-2. In-stream drop structure (Cooks Slough, TX).

a CVC spillway be utilized for more frequent floods (commonly referred to as a service/principal spillway). When a structure requires a spillway capacity in excess of the service capacity of the principal spillway, an auxiliary spillway is constructed to convey the additional flow. Auxiliary spillways are commonly designed to operate at a return period exceeding the 100-year storm. When planning to increase the spillway capacity of an existing dam, the designer should try to maintain the hydraulic capacity of the existing principal spillway before operation of an embankment overtopping spillway. For example, if an existing principal spillway is capable of passing a 500-year flood without overtopping the dam, the planned overtopping protection would generally not be designed to begin operation more frequently than the 500-year flood event. However, a situation can occur where the embankment crest is lowered to physically accommodate the overtopping protection and the overtopping protection spillway begins operation before the original capacity of the principal spillway is achieved. The effects of changing the downstream flow regime can potentially change the risks to affected properties and change the potential liabilities due to flooding. At a minimum, the outflow conditions should usually not be increased for events more frequent than a one in 100-year event. The need to assess the risks associated with upstream and downstream flooding conditions should be evaluated for each project.

The conversion of an embankment to an overtopping structure can also lead to the introduction of a new potential failure mode for a more frequent event than the maximum capacity of the existing principal spillway. The new potential failure mode would be due to the potential for embankment erosion (that did not previously exist) when flow is allowed to pass over a dam embankment even though RCC embankment protection is provided. It should be recognized that no inflow design flood events have tested RCC overtopping protection, but historical performance records do exist, as a number of overtopping RCC spillways on embankment dams have occurred. Ocoee Dam No. 2 has experienced almost continuous overtopping for over 40 years with many large flow events. Chapter 10 discusses the performance of several of these projects.

Conventional auxiliary spillway designs locate the spillway away from the dam embankment whenever possible. If the RCC auxiliary spillway is to be located on the dam as embankment overtopping protection, then flow from the spillway should be directed to the downstream channel and away from the toe of the dam to reduce the risk of erosion of the dam embankment occurring from an overtopping event. The embankment overtopping protection should be designed so that the abutment groins and toe of the dam are protected from erosion caused by flow concentrations and high velocity flow. Areas of flow concentrations should be avoided since they can prematurely exceed the capacity of the energy dissipater and cause localized, accelerated erosion at the abutment groins and base of the dam.

Overtopping protection on embankment dams have been provided with different types of construction materials. Design considerations when selecting an overtopping protection material include flow velocity, discontinuities that can lead to irregular hydraulic flow patterns, effect of irregularities on the material, and the potential for debris to be carried over the dam. RCC has wide application for use as overtopping protection since the material is suitable for a wide range of velocities. It has an added advantage where debris lies within the drainage basin since RCC can generally resist captured debris impacts (such as trees, cobbles, boulders, etc.) without causing severe irregularities in the hydraulic flow and without snagging and displacing anchorage or linkage associated with other types of systems.

Spillways on embankments are usually designed to operate infrequently, and overtopping spillways have not been tested at their maximum design discharge. Conservative selection of loading conditions and design details is necessary due to the limited historical experience and the need to forestall problems that could lead to potential failure conditions.

RCC Principal Spillways

As mentioned earlier, in most cases principal spillways that have high unit discharges and large flow depths should be designed with reinforced CVC. There are a few RCC principal spillways that also serve as the auxiliary spillway. Lower Lake Royer (Figure 2-3) and Butler Dam (Figure 2-1) used a combined principal and auxiliary spillway in the form of overtopping protection. The principal spillway portion occupies a lower area towards one abutment.

FIGURE 2-3. RCC service spillway (Lower Lake Royer Dam, MD).

There are two dams in South Dakota where the RCC overtopping protection serves completely as the principal and auxiliary spillway. He Dog Dam (Figure 2-4) and Ponca Dam (Figure 2-5),

FIGURE 2-4. Downstream (top) and upstream (bottom) views of combined principal and auxiliary spillway (He Dog Dam, SD).

FIGURE 2-5. Combined principal and auxiliary spillway (Ponca Dam, SD).

both owned by the Bureau of Indian Affairs, used this combined RCC spillway concept. Designers should use caution and conservatism using RCC for principal spillways and only when low flow depths and low unit discharges are expected during the inflow design storm.

Armoring of Auxiliary Spillways and Outlet Channels

Many dams have been designed with vegetative earthen auxiliary spillway(s) located at one or both abutments. Extreme rain events over the last several decades have shown a propensity for severe head cut erosion to occur, sometimes leading to a failure of the spillway and loss of a portion of the reservoir. One method to armor the earthen spillway is to create an RCC covering over the residual soils. Oroville Dam in California is a good example of this application (Figure 2-6), where RCC was placed below the auxiliary spillway control section. When the Oroville auxiliary spillway operated, concerns arose when erosion of the residual soils started working upstream towards the control section.

FIGURE 2-6. Armored earthen spillway (Oroville Dam, CA).

In the application of armoring earthen spillways and outlet channels, designers have used both the plating and stair step methods. On mild sloping spillways and outlet channels where steps would be far apart the plating method has been used. The plating method is where the RCC is placed in lifts parallel along the same slope of the ground surface. The protection of the auxiliary spillway at Cooks Slough (Figure 2-2) uses a combination of horizontal stair steps and plating methods. The protection of earthen auxiliary spillways can have many of the same concerns as overtopping protection of embankments. If the RCC armoring is not adequately designed, erosion can occur of the residual soils, which can still result in the partial loss of the reservoir, though likely at a slower pace.

Most of the earthen spillways have been designed to operate at infrequent storm events, typically at the 100-year event. It is generally prudent to maintain that infrequent operation of the spillway even once it is armored. Similar to overtopping protection spillways, uplift must be accounted for and terminating the spillway with an end wall to prevent head cutting beneath the RCC. The geometry and location of the spillway must ensure that flows from the spillway do not encroach onto the downstream toe of the dam.

Some projects used RCC to protect the outlet channel below a CVC control section such as an ogee or a labyrinth weir. Standley Lake Dam outside Denver (Figure 2-7) used RCC below the labyrinth weir and then also to protect the outlet channel side slopes along with seven additional drop structures. Ochoco Dam (Figure 2-8) in Oregon in 1996 used RCC to armor the outlet channel. In addition to Ochoco Dam, the Bureau of Reclamation also used RCC at the Cold Springs Dam (Figure 2-9) in Oregon to armor the outlet channel.

FIGURE 2-7. Outlet channel protection (Standley Lake Dam, CO).

2.3 DAM STABILITY AND DOWNSTREAM EROSION

Construction of RCC overtopping protection can also impact the stability of an existing embankment. RCC on the downstream slope of the embankment can block existing seepage paths, increase the phreatic level, and decrease embankment stability. Changes to the embankment section can decrease the factor of safety for slope stability, in particular for excavation slopes during construction. These concerns should also be addressed for any type of concrete spillway over an existing embankment.

FIGURE 2-8. Outlet channel protection (Ochoco Dam, OR).

FIGURE 2-9. RCC side channel spillway (Cold Springs Dam, OR).

Erosion downstream of embankment overtopping protection can have a critical impact on the stability of the embankment and can also cause high seepage gradients to occur at the toe of the dam. Excavation at the toe of the embankment to construct the various features of the overtopping protection, in particular for the downstream apron or over steepening of the downstream slope, will change the stability of the overall embankment. If erosion at the toe of the dam is expected to occur during overtopping, then the eroded conditions should be evaluated in both the embankment stability and embankment seepage analyses. These critical stability and seepage conditions must be considered in the overtopping protection and embankment design.

CHAPTER 3

INVESTIGATION

3.1 GENERAL

Before designing a new dam or evaluating modifications to an existing dam, the site should be investigated to understand the geologic history of the foundation as well as the soils/rock used in any existing embankment and to develop appropriate geotechnical parameters for design. Geotechnical design parameters will generally be needed for analyzing embankment stability and seepage, evaluating the impact of the proposed modifications, estimating the bearing capacity of the foundation, providing analysis of filter compatibility, predicting heave or settlement, recognizing the erodibility of potential auxiliary spillway locations and outlet channels, and designing retaining walls, hydraulic structures, and other appurtenant structures as required. There may be special features or conditions associated with some projects that will also need to be included when planning the investigation. An experienced geotechnical engineer with local knowledge is strongly encouraged to be utilized for developing the investigation program.

For RCC spillways and overtopping design on a new dam, the objectives of investigation are generally the same as for existing dams. Evaluating the properties of the new embankment and developing information to predict the behavior and condition following construction need to be included. Many of the RCC spillways for new dams used the local gravel deposits for the aggregate source for the RCC. These deposits should be completely investigated during the planning phase.

3.2 PROJECT REVIEW AND SITE RECONNAISSANCE

The first phase of investigation involves a project review of available information prior to the site reconnaissance to develop an understanding of how the dam was constructed and how it has performed. The type of information can include design and construction drawings, construction records and photographs, records of inspections, and reviews by owners or jurisdictional agencies. In some cases, there may be substantial structure performance data from instrumentation programs. Instrumentation will usually include monitoring of the phreatic surface within the dam, seepage measurements, and surface and subsurface movement. Additional information can often be obtained from the owner's staff familiar with the operation and maintenance of the existing facility.

Whether or not instrumentation is in place or monitoring has been performed, visual observations can provide considerable information on the past performance of the dam. High phreatic levels, seepage, settlement, and shear displacement generally leave surface expressions such as soft soils, cracks, sloughs, and wet thriving vegetation that can be observed during a site reconnaissance. Guidelines have been prepared for performing inspection of dam embankments, including United States Bureau of Reclamation (USBR, 1983), Federal Emergency Management Agency (FEMA, 1987, 2004), and some state dam safety agencies. These guidelines include standard forms for evaluating the dam embankment and the foundation downstream of the dam. For new dams, the desk study should include a review of available geologic mapping of the area and any reports specific to the proposed dam site.

The planning phase should also include investigating the local availability of the various portland cement sources that might be considered as well as the various classes of fly ash. Bulk cementitious suppliers should be contacted to confirm they can meet the demand. Quarries near the project site should be contacted to determined what types of aggregate and stone classes are available. Most quarries can provide gradations, absorption, specific gravity, and sometimes aggregate-alkali reactivity and/or alkali-silica reactivity results. More details on RCC aggregates are provided in Section 3.4.

3.3 SUBSURFACE INVESTIGATION

Subsurface investigations are used to delineate subsurface strata and water levels in the embankment and foundation and to collect samples for laboratory testing. Of particular interest are the subsurface material and water levels (phreatic surface) in the downstream slope and in the foundation at the downstream toe. The scope of investigation usually includes drilling of test holes and/or excavation of test pits, with associated logging and sampling. The subsurface investigation scope should be planned and implemented under the direction of a qualified geotechnical engineer experienced in dam design.

Geophysical methods such as seismic refraction, ground penetrating radar, and electrical resistivity may be used for RCC overtopping protection investigations. Additional subsurface investigation methods include Cone Penetration Testing (ASTM D6951/D6951M) where the resistance and response of pushing a cone into the ground is monitored, but no sample is retrieved, and bucket auger drilling where a large diameter hole is drilled and a person is lowered into the hole to collect samples and record observations. These methods and other less common investigation methods may be required because of site-specific subsurface conditions or project constraints.

Test holes and test pits can be excavated to shallow depth by hand and to greater depths by drill rigs or excavators. Test hole and test pit locations and depths should be selected to sample embankment and foundation material where the hydraulic structure and appurtenant facilities are planned. Investigations must be completed without jeopardizing the safety of an existing dam. The Federal Emergency Regulatory Commission (FERC) Guideline for Drilling in and Near Embankment Dams and Their Foundations (version 3.1 June 2016) and the Corp of Engineers ER 1110-1-1807 Drilling in Earth Embankment Dams and Levees are two good sources of information to review. Drilling within the core of a dam should be approached with caution and only be done when necessary, since drilling – especially rotary drilling where water is introduced – can cause hydrofracturing of the soil.

Subsurface investigations may be needed to confirm the location, type, and condition of buried drainage systems within an existing dam. Drainage systems can include granular filters and drains and drain pipes. Utilities may also be present and must be properly identified and marked prior to any subsurface investigation. Subsurface investigations generally need to be performed in such a way that existing features are maintained, without significant impact. Granular drains and filters can be evaluated by test holes and test pits, with careful logging and sampling, as discussed later in this section. Geotextiles and geocomposites can be evaluated by partial excavation, if needed. Pipes can be evaluated by probing and visual inspection by remotely operated camera surveys inside the pipe.

The amount of investigation required can vary considerably depending on the size of the project, the subsurface conditions at the site, and the availability of information from previous investigation and construction records. Logging and sampling are needed to classify the soil and rock encountered, and samples are needed for laboratory testing. Logs of test holes and pits should be prepared in accordance with ASTM D5434 or locally accepted standard of practice.

Test pits should be backfilled and compacted following sampling and logging. Generally, the excavated material will provide suitable backfill. Test holes also need to be backfilled, usually with non-shrink grout, to fill the hole and to limit the potential for water and/or particle movement between strata. Alternatively, some exploratory holes are developed into observation and monitoring wells using instruments such as a standpipe piezometer or inclinometers.

Field sampling and testing are a function of the soil and rock types encountered, so some expectation of the soil types to be encountered and depth to rock is needed for planning the investigation, and flexibility is needed in the sampling and testing scope. Generally, soil without sufficient clay and silt content (Unified Soil Classification System (USCS) types ML, MH, SC, and SM) cannot be readily sampled without causing some disturbance, so laboratory testing of such materials is generally done on "disturbed" samples. Soil with significant clay and silt fractions, and without too much gravel (and coarser) fractions, can be sampled with limited disturbance using tube samplers, so laboratory testing on relatively "undisturbed" samples can be conducted.

Sampling is usually performed at 5-foot intervals and at changes in material type. A shallow depth sample (less than 5 feet) is often valuable because the earthwork involved in overtopping protection projects may be shallower than 5 feet. When drilling, disturbed samples can be collected with the split-spoon sampler as part of the Standard Penetration Test (SPT). The SPT is useful for evaluating the in situ properties of the soil, unless there is a large coarse gravel and cobble component. In fine grained soil, where tube samples are desired, penetration testing can be conducted by driving alternative tube samplers, most commonly the Modified California Sampler or the Dames & Moore Sampler. Blow counts recorded for penetration of these samplers can be converted to SPT N-values (ASTM D1586) and used in conjunction with the recovered samples to evaluate soil properties.

Permeability estimates may also be required for the embankment or foundation for seepage analysis to evaluate dewatering needs during construction and designing permanent seepage control measures. Permeability measurements can be made from test holes as well as on limited field samples prepared and tested in the laboratory.

Large bulk soil samples are generally needed for Proctor compaction testing (ASTM D698). In uniform materials, it may be acceptable to build a composite sample with cuttings over a large depth interval. Samples from discrete intervals cannot be obtained from drill cuttings. Large bulk samples are more typically obtained from test pits. Test pits can also be used to collect "undisturbed" tube samples of fine-grained materials. This is done by driving the tube by hand methods, then by excavating and trimming the soil around the tube. Soil properties are usually anisotropic and tube samples oriented vertically are generally preferred because the testing apparatus more closely replicates field conditions. Alternatively, a large block of soil can be cut out of a test pit and trimmed to the appropriate test specimen size in the laboratory.

The USCS designation should be recorded (based on visual classification) during drilling or test pits to provide information needed to finalize the sampling and testing program. Some typical sample types and testing based on the USCS classification are outlined in Table 3-1. USCS designations are fairly broad in the range of soil they describe; therefore, some

TABLE 3-1. Common Soil Testing for RCC Overtopping Protection Investigation

USCS DESIGNATION	SAMPLE TYPE	TEST TYPE										
		VC	WC	WC/ DUW	LL/PL	SA	HYD	WA (-200)	GS	COMP	UC	S/C
GW	M,S,B,BU	•	•			•		•		•		
GP	M,S,B,BU	•	•			•		•		•		
SW	M,S,B,BU	•	•			•		•		•		
SP	M,S,B,BU	•	•			•		•		•		
GM	M,S,U,B,BU	•	•	•	•	•		•		•		
GC	M,S,U,B,BU	•	•	•	•	•	•	•	•	•		
SM	M,S,U,T,B,BU	•	•	•	•	•		•		•		•
SC	M,S,U,T,B,BU	•	•	•	•	•	•	•	•	•		•
ML	M,S,U,T,B,BU	•	•	•	•	•		•		•	•	•
MH	M,S,U,T,B,BU	•	•	•	•	•		•		•	•	•
CL	M,S,U,T,B,BU	•	•	•	•	•	•	•	•	•	•	•
CH	M,S,U,T,B,BU	•	•	•	•	•	•	•	•	•	•	•

KEY TO TEST TYPES. (ASTM test designations based on Vol 04.08, 2000)

VC	Visual classification: D2488
WC	Water content: D2216
WC/DUW	Water content for the determination of dry unit weight: D2216
LL/PL	Liquid and plastic limits: D4318
SA	Sieve analysis: D422, or C136 and C117 (ASTM Vol. 04.02)
HYD	Hydrometer analysis: D422
WA	Wash analysis or percent fines determination: D1140 or C117 (ASTM Vol. 04.02)
GS	Specific gravity, specify sieves: D854 or C127, C128 (ASTM Vol. 04.02)
COMP	Compaction effort, procedures A, B, or C, moist or dry preparation: D698 or D1557 or C1435 (ASTM Vol. 04.02)
UC	Unconfined compression: D2166 or C39 (ASTM Vol. 04.02)
S/C	Swell/Collapse potential: D4546 (Swell & Settlement) or D5333 (Collapse)

KEY TO SAMPLE TYPES.

M	Modified California or other driven tube
S	Standard Penetration Test (SPT)
U	Osterberg, piston, pitcher
T	Shelby tube
B	Bag Sample (sealed plastic to obtain moisture content)
BU	Bucket/Bulk

exceptions to the sampling and testing shown in Table 3-1 should be expected. It may not be necessary to have all the indicated sampling and testing performed, because the properties and parameters required for each project vary. Consideration of the analyses required for design, and the necessary input parameters for those analyses, should be part of the investigation scoping process.

In addition to the tests listed in Table 3-1, other tests may be desirable for some projects such as consolidation testing including time-rate measurements, direct shear or triaxial shear testing for shear strength, chemical testing for aggressiveness of the ground on degradation of concrete and corrosion of steel, permeability tests, and dispersion tests to evaluate the potential for internal soil erosion. A project specific testing program should be developed by a qualified geotechnical engineer.

For projects such as armoring auxiliary spillways or outlet channels from service spillways, the field investigation requires sampling and testing of the residual soils and rock in the area subject to flows. The Natural Resources Conservation Service (NRCS) Sites program provides guidelines for determining the number of borings required as well as minimum depths. Specific testing requirements for the soil and rock are provided. At locations of possible cutoff walls, special attention should be paid to the erodibility of the residual soils and rock to identify the design depths of cutoff walls.

3.4 RCC AGGREGATE INVESTIGATIONS

Potential aggregate sources for RCC include near or on-site natural deposits, existing pits or quarries, and new or old inactive quarries. Borrow investigations are performed to identify and locate potential aggregate sources to be used for design, specifications, and cost estimates. The approach to the aggregate borrow investigation will be influenced by the project size, location, and site conditions.

The volume of material required generally has the most influence on identifying borrow sources for a project. Projects in urban settings can benefit from already established suppliers of sand and gravel that provide a ready and economic source of material, particularly when more than one supplier is available.

The volume of material required for most projects will generally not be limited by the availability and supply from existing suppliers; however, large volume projects can often require a peak demand that existing suppliers cannot meet. This can be partially offset by the Owner/Engineer working with local suppliers on the project need and schedule in advance of construction. In remote sites, the cost of aggregate hauling can make established, fixed location aggregate sources relatively expensive, and development of near or on-site pits or quarries should be considered.

At remote sites, the volume of RCC required can have a significant effect on the potential borrow sources that would be used for a project. On projects with small volumes required, such as 2,000 yd^3 or less, it will seldom be economical to open a new pit or quarry. Development of a new quarry/pit becomes more economical as the volume increases from 2,000 yd^3 to 20,000 yd^3 or more.

Planning the investigation for aggregate borrow sources should start with an assessment of the various conditions described previously. An investigation would typically include contacting local contractors and suppliers, county and/or city road departments, and state highway agencies to ascertain available information on existing or previously used quarries and pits. There is often sufficient available information that the initial investigation can be performed with only limited sampling and testing to determine the range of suitable material that is already available.

After canvassing the existing material sources, samples should be obtained to confirm the material properties of the most readily available and cost-effective sources that are identified as suitable for use in the RCC mix. Testing should include at a minimum the following tests: mechanical analyses (gradation), Atterberg limits, specific gravity, and absorption. Additional tests that may be required, if there is not a documented history of the aggregate properties at the source(s), should also include: soundness, mineralogy, Los Angeles Abrasion, freeze-thaw durability, and silica-alkali reactivity. The variability of the aggregate properties with existing sources, in particular the fines (material passing the #200 sieve) content, specific gravity, and absorption, should also be evaluated before a specific source is identified for a project, as variations in these properties have a significant effect on the mixture proportioning and workability of the RCC.

Based on the initial data gathering on the quality, quantity, availability, and cost of existing sources, a judgment can be made on whether additional sources of aggregate should be investigated. Investigations of aggregate sources should be performed to confirm a minimum of twice the quantity of material needed for the project to ensure that an adequate quantity is available so that shortfalls do not adversely impact the project schedule. Investigations of new sources can be conducted by a combination of test pits, drill holes, and core drilling.

Two other investigation methods should also be considered for large volume project sampling and test blasting. Large volume sampling is based on (1) obtaining the maximum size of the material (ASTM D75), field measurement, and splitting of samples from test pits, or (2) sampling in a dump truck (8 yd^3 or larger size sample) and processing at an aggregate plant or a laboratory to evaluate more fully borrow sources with large size aggregates. A test blast investigation would include excavation and processing and crushing of the material to assess the aggregate properties and to develop material for laboratory mix designs. Test blasts at potential quarry sites are typically performed primarily for larger projects (in excess of 50,000 yd^3).

CHAPTER 4

SLOPE STABILITY AND FOUNDATION ANALYSIS

4.1 GENERAL

An important aspect of constructing a spillway is the stability of the foundation. Slope stability analysis is required to evaluate whether an existing structure will have an acceptable factor of safety against slope failure both during construction and after construction is complete. Foundation analyses are also required to evaluate other potential modes of failure of the proposed modifications, such as bearing capacity, settlement or heave, overturning failure of retaining walls, adequate drainage, and sliding.

Certain projects may have special aspects that require analyses of specific conditions not described in this chapter. In some instances, more sophisticated models such as finite element or finite difference models of deformation may be warranted. However, the standard analyses described as follows are useful as a basis for determining whether more sophisticated analyses are needed, and for evaluating the results of other analyses or methods.

4.2 SLOPE STABILITY

Most dam modifications for RCC overtopping protection projects do not reduce the factor of safety against slope failure since they do not create significant changes to loading or water levels within the dam. In some cases, the need for computer-based slope stability analysis may be waived based on inspection by geotechnical engineers experienced in embankment stability and analysis based on closed form/chart solutions. However, because of the consequences of slope failure, slope stability analysis is generally appropriate and should be performed. The following paragraphs describe typical procedures and methods of analysis.

Slope stability analyses consist of five primary steps:

1. Characterizing the geometry of the slope and material boundaries.
2. Evaluating the material properties for each type of material in the embankment and foundation.
3. Evaluating internal and external water pressure and loading/seepage conditions.
4. Inputting geometry, material properties, and water pressure in a model for analysis of slope stability.
5. Solving for the minimum theoretical factor of safety.

Note that more than one cross-section may need to be analyzed if the phreatic surface varies laterally within the embankment.

Input parameters for slope stability analysis include material boundaries, water pressures or phreatic levels, material unit weights, and material strengths. Water pressure and strength parameters are most important because they can have a significant effect on the calculated factor of safety.

Unlike many construction materials, the strength of soil is highly dependent on loading conditions. Strength parameters c and Φ, which describe the cohesion intercept and friction angle of a material, respectively, are generally appropriate for analysis. Whether the parameters should be based on effective or total stresses depends on loading conditions being analyzed (e.g., end of construction, steady seepage, etc.). Strength parameters should be developed by an experienced geotechnical engineer.

Strength parameters to be used for end-of-construction analysis should be the lowest of: (1) current conditions prior to the proposed modifications, or (2) future conditions following the proposed modifications. Strength parameters to be used for steady seepage analysis should reflect the projected change in conditions (density, water content, seepage, etc.). The analysis should consider that the RCC may act as a barrier to evaporation, evapotranspiration, and seepage, and the phreatic surface may increase as discussed in Chapter 5, Seepage Analysis. If the RCC strength is included in stability analysis, it should be assumed that the RCC is cracked (transverse) and the strength on shear surfaces passing through the RCC is frictional.

Standard loading conditions for embankment dams include end-of-construction, steady-state seepage for normal pool conditions, steady-state seepage at flood pool, steady-state seepage earthquake loading conditions, and during rapid drawdown. These loading conditions are shown in Table 4-1 with published recommended criteria for the minimum factor of safety. Post-spillway operation is an additional loading condition that should also be considered for RCC embankment overtopping because operation of a spillway over an embankment can cause erosion at the toe of the embankment and perhaps elsewhere, creating significantly steeper slopes and lower factors of safety. The extent of possible erosion should be estimated for discharges with a probability of occurrence similar to that for rapid drawdown, and the calculated factor of safety, based on

TABLE 4-1. Slope Stability Criteria

LOADING CONDITION	RECOMMENDED MINIMUM REQUIRED FACTOR OF SAFETY		
	USBR[1]	USACE[2]	FERC[3]
End-of-Construction	1.3 - 1.4	1.3	1.3 - 1.4
Steady State Seepage, Normal Pool	1.5	1.5	1.5
Steady State Seepage, Flood Pool	1.2	1.4	1.4
Steady State Seepage, Earthquake	>1.0	1.0	>1.0
Rapid Drawdown	1.2 - 1.3	1.1 - 1.3	1.2

[1] U.S. Bureau of Reclamation (USBR) 2011
[2] U.S. Army Corps of Engineers (USACE) 2003
[3] Federal Energy Regulatory Commission (FERC) 2021

the eroded geometry, should be 1.5 or greater. Designers should check with their governing regulatory body for specific factor of safety requirements for differing loading conditions. However, in some cases a factor of safety less than 1.5 for the post-flood eroded condition may be appropriate if the event that causes erosion is remote, and depending on several factors including the regulatory environment, the expected time to make repairs or fill in the eroded feature, level of uncertainty in erosion depth, consequences of failure, and other factors.

Two-dimensional limit equilibrium analysis is the standard method of evaluating embankment dam stability. For simple geometry such as long, planar slopes or homogeneous embankments, closed form solutions or chart solutions may be applicable. Infinite slope analysis, wherein the slope is assumed to be infinitely long, is perhaps the simplest means of analysis. Infinite slope analysis is typically conservative (calculates a low factor of safety) because the end of slope conditions, which are ignored in this analysis, contribute more to the forces maintaining stability than to the forces driving a hypothetical failure. More realistic closed form/chart solutions have been published by Duncan (1996) and others. Closed form/chart solutions provide a factor of safety for slip surfaces specified by the input, but they do not search for the slip surface that has the minimum factor of safety. Analysis of different slip surfaces can be done manually or by use of available computer programs.

Most computer programs available today offer several methods of performing limit equilibrium analysis for circular slip surfaces. These methods generally analyze the slope as a series of vertical slices and differ primarily in how they treat the internal reaction forces between the imaginary slices. The Modified Bishop's Method is generally considered suitable for embankment slope stability analysis and is widely available. Search routines for a circular surface with the minimum theoretical factor of safety are located. Analyses by experienced geotechnical engineers using computerized search routines can successfully locate the circular surfaces with the theoretical minimum factor of safety based on the input parameters.

Non-circular surfaces generally do not have appreciably lower factors of safety unless there are material boundaries in the embankment that intercept potential failure surfaces. Material boundaries could be a bedrock or RCC contact, embankment zone interfaces, weak zones in the foundation or embankment, bedding planes or discontinuities in soil/geosynthetic surfaces and bedrock. If such features are present, non-circular analysis procedures may be necessary. Procedures such as those developed by Janbu, Morgenstern and Price, and Spencer (Duncan 1996), are capable of modeling non-circular slip surfaces. Considerable care must be used in application of searches for a minimum theoretical factor of safety for both circular and non-circular searches. However, because search routines for non-circular slip surfaces are not as effective as they are for analysis of circular slip surfaces, more understanding and analysis of field conditions that could influence potential slip surfaces is required.

4.3 FOUNDATION ANALYSIS

Embankment and structure modifications associated with RCC overtopping may require foundation analysis for design. Most RCC structures result in only nominal changes in loading. However, there may be changes in groundwater levels and phreatic levels that could have adverse impacts if they are not considered in the design. The following paragraphs describe analysis for evaluating settlement or heave, bearing capacity, dewatering, and designing retaining walls.

Settlement or Heave

Volume change in foundation soil can occur in response to changes in loading, water content, or weather. The degree of volume change is most significant in certain types of soils and conditions. Consolidation/settlement will generally be significant where soft, normally consolidated, or slightly over-consolidated clayey soil comprises the foundation. In such cases, even light loads can cause enough settlement to contribute to cracking and structural distress. Where possible, excavation and replacement of these soils should be considered. Where this is not possible or practical, it may be desirable to pre-consolidate the materials with surcharges or to include load compensation in the design.

In some cases, the potential for volume changes due to unloading of embankment soils resulting from temporary excavation for RCC structures and then subsequent reloading due to RCC placement and backfilling against RCC structures may need to be evaluated. The expected time for each construction stage, as well as the heave/consolidation properties of the embankment soils, must be well understood.

Settlement can also occur as a result of wetting of collapsible soils that undergo a radical rearrangement of particles and reduction in volume. This should be considered especially where silty and sandy soil are at relatively low density and are dry or unsaturated. Collapse can often be induced prior to constructing a structure by wetting and compacting the soil. Alternatively, the soil with collapse potential can be removed and replaced.

Heave or expansion resulting from increased moisture content in partially saturated clays and weathered claystone can represent a volume increase of 10% or more and it is not uncommon for uplift pressures to exceed 10,000 psi. The degree of heave or expansion can be reduced by compacting soil wet of optimum moisture content and maintaining a constant moisture content environment. Expansive clays (such as bentonite and montmorillonite) can swell to many times their original volume and should be avoided. In situations where these material types cannot be avoided, complex foundation systems are often required to resist uplift pressures.

Frost heave can occur where soil within the frost depth is moist or saturated. Frost heave is most significant in silty sand, where ice lens formation can cause heave of several inches. Uplift pressure from frost heave could be enough to crack or dislodge RCC and cause unsatisfactory performance. Free draining soil with significant coarse sand and gravel fractions are least susceptible to frost heave, even if they are moist or wet, because the soil is permeable enough to allow water to flow away from ice as it forms, thereby reducing the potential for volume change. Free draining bedding material is recommended where conditions for frost heave exist. Consideration should also be given to ensure RCC and underlying drainage materials extend below finished grades to a depth equal to or greater than the frost depth for the area where the project is located.

Bearing Capacity

Bearing capacity is generally not of significant concern because of the light loads typically applied and that these loads are often uniformly placed, resulting in very low to zero shear stresses being developed. For projects where CVC structures may bear upon RCC, such as an ogee weir over an RCC-lined entrance channel, bearing capacity of the foundation should be evaluated using standard equations relating soil strength and unit weight and the planned size and depth of foundations. If bearing capacity evaluations are required, the relative stiffness and potential for cracking of RCC should be considered when selecting the foundation width that is used in bearing capacity calculations.

Dewatering

Seepage and wet foundation conditions can have a significant effect on foundation strength as well as construction productivity. The nature of spillway construction means that seepage and water beneath the spillway slab or stilling basin foundation will be encountered. Dewatering of the construction area is needed to provide a firm foundation. Dewatering can often be accomplished by ditches and sumps at most sites since relatively pervious foundation materials are typically encountered at stilling basin locations. The depth of trenches and spacing of sumps varies based on the foundation material. In some foundations, well point dewatering systems or deep wells may be required.

Retaining Wall Design

Analyses for the design of retaining walls should be based on the active or at-rest coefficient of earth pressure and unbalanced water pressures. Water pressure has a large impact on design that can be reduced by installing drainage behind walls. Wall analyses should include evaluating sliding, overturning, and global stability, as well as the foundation bearing capacity. Standard methods of analysis should be used. Retaining wall design is covered in references such as Bowles (1996). When selecting coefficients of lateral earth pressures, the potential and/or desire to limit rotation of a wall should be considered. The long-term effectiveness and potential for clogging of drainage systems should also be considered if drain systems are relied upon to reduce lateral water pressures on walls.

CHAPTER 5

SEEPAGE ANALYSIS

5.1 SEEPAGE CONSIDERATIONS

Most RCC spillway and overtopping protection applications are for modifications and rehabilitation of existing dams. The design of the overtopping spillway protection must be compatible with existing seepage conditions and changes in existing seepage conditions caused by an upgrade/modification with RCC.

Seepage collection and control features may be required in the design of RCC spillways and overtopping protection for the following reasons:

- To collect and control seepage through the embankment, abutments, or foundation under normal reservoir conditions.
- To limit uplift pressures that could develop under the RCC as a result of the spillway operation.
- To collect and control infiltration of water through cracks and joints in the RCC.

Seepage and uplift design considerations, which also address infiltration through cracks and joints, are discussed in the following sections in greater detail.

Seepage – Under normal reservoir levels, seepage can develop through the embankment, abutments, and foundation and also through the foundation beneath a spillway. RCC, as with CVC, provides a very low permeability barrier to seepage. For all practical purposes, seepage only passes through the RCC layer by way of cracks or joints in the RCC, or a designed drainage system. If the RCC layer blocks locations where seepage would otherwise exit (see Figure 5-1), excess water pressures building up under the RCC could result. Excess pressures immediately beneath the RCC could lift and damage the RCC. Even if the pressures are not sufficient to damage the RCC, the construction of an RCC layer could redirect general embankment and foundation seepage to the locations of cracks or joints in the RCC. This could result in higher seepage gradients at the cracks or joints, which could allow piping (internal erosion) of the embankment and/or foundation soil to develop if not considered in the design. Blockage of seepage exit points by the RCC could also result in increased pore water pressures in the embankment, abutments, and foundation, which in turn would decrease the stability of the embankment.

If the existing embankment and foundation includes adequate seepage collection and control features such that the downstream slope is dry under all loading conditions, then it may not be necessary to include seepage collection and control features in the RCC design, except for seepage originating from inflow through cracks or joints in the RCC when the spillway operates. For example, if an embankment includes an upstream chimney and blanket drain, then it is not likely that uncontrolled seepage would reach the underside of an RCC overtopping protection layer. Similarly, if an embankment contains an effective low permeability core, seepage may not reach the downstream face where the RCC layer would be constructed. However, in this latter case, the designer should be cautious before concluding that seepage could not reach the downstream face. The lack of visible seepage on the downstream slope of an existing dam may not be sufficient to conclude that a drainage

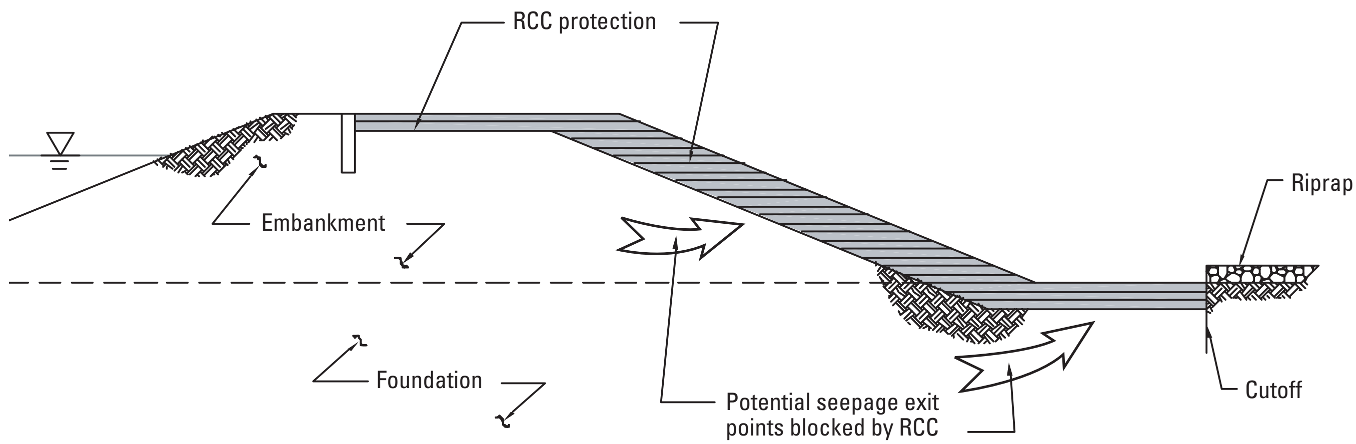

FIGURE 5-1. Seepage exit points blocked by RCC overtopping protection.

system may not be needed. The amount of seepage that reaches the face may be small enough that it evaporates in the open air or is consumed by vegetation through evapotranspiration. However, it could accumulate under an RCC spillway structure when not permitted to evaporate.

If the RCC protection is constructed downstream of existing seepage collection and control features, the design must include means for the discharge from those systems to safely pass through or around the RCC. Visual observations, design documents, and record drawings for an existing dam should not be used as the sole basis for excluding seepage collection and control features from an RCC spillway design. Rather, it is recommended that field investigations and instrumentation readings be used to confirm the actual seepage condition in the embankment, abutments, and foundation before that decision is made. The evaluation of data should also consider seasonal variations of phreatic levels in embankments, abutments, and foundations in dams as reservoir levels may fluctuate seasonally as part of normal operations, as well as water levels in abutments may vary seasonally due to precipitation and other factors.

Normally dry flood control dams can be a special case with respect to seepage considerations. If the embankment material is sufficiently low in permeability and/or the dam serves exclusively for stormwater detention, there may be insufficient time for seepage to develop through the dam. This case can be analyzed by a transient seepage analysis. However, in considering this case, the designer must also evaluate the effects of defects or design details (e.g., cracks in the embankment or pervious foundation layers) for the transient analysis.

If the designer decides that a complete seepage collection and control system is not required beneath the RCC overtopping spillway for seepage under normal pool conditions, it may be advisable to include a filter zone immediately beneath the RCC to control the potential for loss of fines through cracks or joints from surface water runoff or overtopping flows. In addition, a seepage collection and control system may be required because of uplift considerations, as discussed in the following text.

The most common method used to control seepage for an RCC spillway or overtopping design is a drainage layer placed underneath the RCC layer. The drainage layer must provide sufficient capacity to convey the anticipated seepage (plus a margin of safety), and it must meet filter criteria relative to the underlying soils so that internal erosion does not occur. The seepage control system must also include collection and outfall pipes or other means to discharge the seepage collected in the system.

Uplift Pressures – During flood events, when water flows over the RCC, there is a potential for hydrostatic pressures to increase beneath the RCC layer. These pressures can concentrate in a permeable drainage layer beneath the RCC or at the boundary between the RCC and less permeable underlying strata (if no drainage or an insufficient drainage layer is present). If the underlying hydrostatic pressure exceeds the combined weight of the overlying RCC and the water on top of the RCC, there is a potential for the RCC to be displaced and damaged. Movement of the RCC layer during flow over the RCC can lead to erosion, undermining, and failure of the RCC protection and the underlying embankment.

Hydrostatic pressures can concentrate beneath RCC overtopping spillways through two sources, as shown in Figure 5-2. First, the reservoir can come into direct or near-direct communication with the area under the RCC from erosion at the upstream end of the RCC. Second, water pressure can be transmitted through cracks and/or joints in the RCC during overtopping flows. Pressure from direct communication with

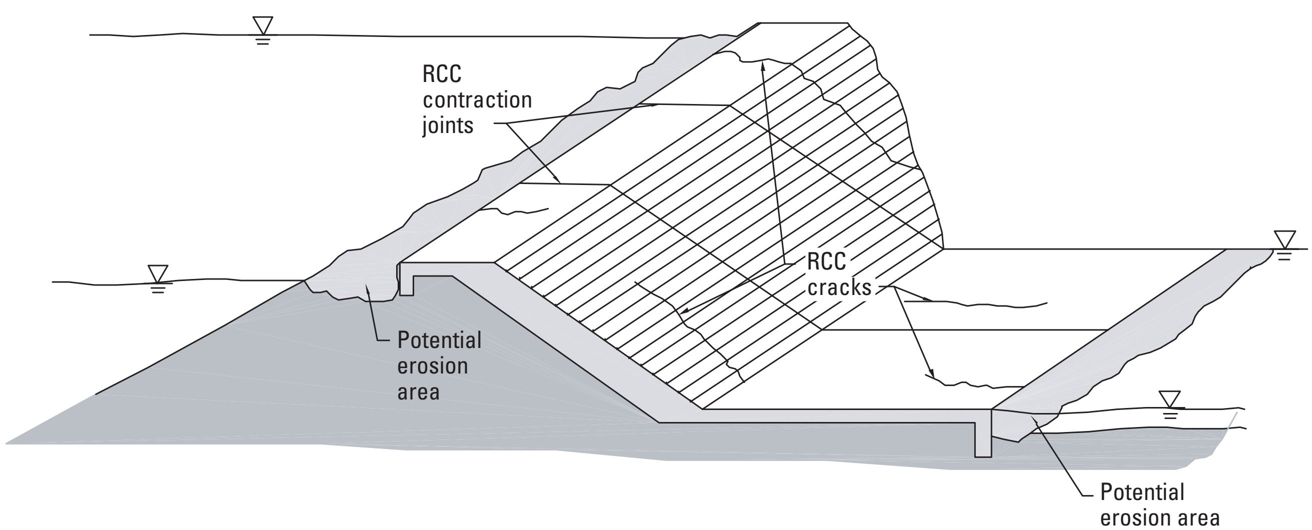

FIGURE 5-2. Cracks, joints, and potential erosion where water pressure can be transmitted to the area beneath the RCC.

the reservoir is of particular concern, because of the potential to transmit the full reservoir head to the area beneath the RCC.

For hydrostatic pressures beneath large areas of the RCC slab due to flow in the spillway, this pressure must be transmitted through the cracks by infiltration and spread laterally beneath the slab. The potential for pressure build up would increase as the spacing of the cracks decrease because the distance over which the pressure must be transmitted decreases. Consequently, an RCC design that results in more widely spaced cracks is less prone to development of this potential problem. The potential pressure beneath the RCC needs to be considered for normal conditions and during unusual conditions such as during an overtopping event and immediately after overtopping ceases. The most critical conditions can occur once overtopping ceases when there will no longer be water on top of the RCC, except for tailwater from other discharge sources (e.g., a service spillway or an outlet works). If the pressure cannot dissipate from beneath the RCC quickly enough, a condition could develop where pressure is trapped beneath the RCC without the gravity load from water on top of the RCC, and displacement of the RCC could result. In general, RCC displacement under these conditions would not be as serious as uplift that occurred during overtopping, since erosion and total failure of the RCC protection would not likely result. However, the RCC could be damaged, and the structure could be at risk to erosion damage during subsequent spillway flows before repair can be made. The designer should assess how serious this potential condition is and how it should be addressed in the design.

For low height (less than 10 feet) dams, the weight of the RCC layer may be sufficient to resist the full reservoir head, even at the toe of the dam. However, for taller structures, it may be necessary to include specific design features to address the uplift concerns. CVC spillway slab design includes reinforced concrete with waterstops at contraction joints, under-drains, and anchors. The primary design feature to reduce uplift is a pervious under-drain layer with pipe outfalls to limit the hydrostatic pressures. A typical cross-section for an under-drain beneath an RCC overtopping spillway is shown in Figure 5-3.

Since an under-drain design feature is very similar to the drainage system required for collection of embankment seepage, the two features can sometimes be combined into a single system that performs both functions. This design approach can also be used for RCC spillways. Like the seepage collection system, the under-drain system must meet filter criteria to prevent internal erosion of underlying soils.

Control of seepage and control of uplift pressures need to be considered not only for the sloping section of the RCC spillway or overtopping protection, but also for any RCC "runout" or energy dissipation apron that extends beyond the toe of the sloping portion of the RCC.

Since reinforcing steel, waterstops, and anchors used in CVC are not practical in RCC, the design should provide: (1) sufficient drainage to limit/prevent uplift pressures, (2) adequate mixture designs to develop sufficient strength to meet the loading conditions, and (3) widely spaced joints to limit cracks and allow for larger monolithic sections.

Based on this previous discussion, it follows that the analyses required for consideration of seepage in design of RCC spillways and overtopping protection consists of three stages:

1. Analysis of steady-state seepage under normal pool conditions.
2. Analysis of uplift pressures during and immediately after flow over the RCC.
3. Analysis of filter compatibility requirements.

The analyses for these three stages are discussed in the following sections.

5.2 STEADY-STATE SEEPAGE ANALYSIS

Methods for analysis of steady-state seepage through embankments and foundations are well-established in geotechnical engineering practice (Cedergren). For several decades, the most common method of seepage analysis was construction of a graphical flow net. An example of a graphical flow net is illustrated in Figure 5-4. Graphical flow nets are used to calculate the expected flow patterns and

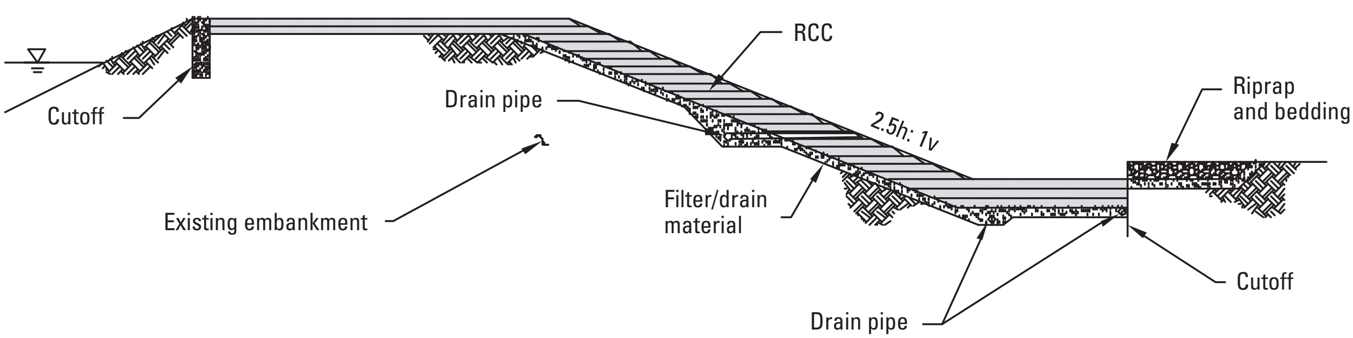

FIGURE 5-3. Typical RCC overtopping section with under-drain.

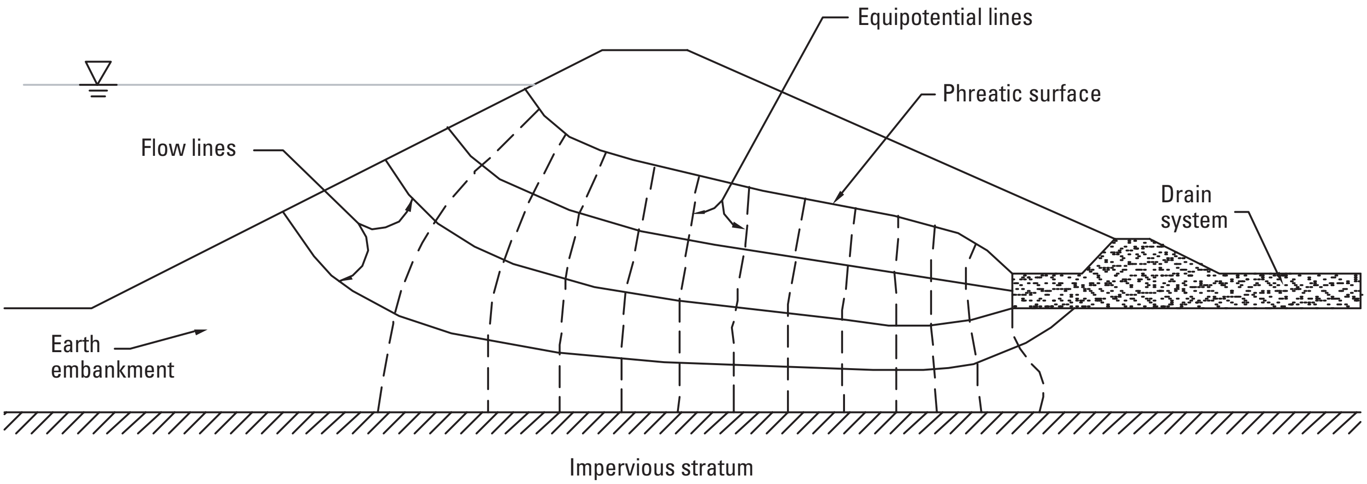

FIGURE 5-4. Example of a flow net solution for an embankment with a toe drain.

quantities for a wide variety of embankment and foundation conditions (Cedergren, Harr).

Today, computer programs have been developed for the analysis of steady-state seepage. However, for many years, these programs were not widely used for the analysis of steady-state seepage for dams. Personal computer (PC)-based numerical model software products for seepage analysis have been developed for general use in the geotechnical engineering profession. One example is the program SEEP/W (GEO-SLOPE International Ltd. 2021.4). With the advent of the PC-based programs, computer programs are being used more frequently for steady-state seepage analyses. An example solution using a finite element analysis program is shown in Figure 5-5.

Whether the steady-state seepage analyses are performed using a graphical flow net or a computer program, it must be recognized that the resulting computed flow quantities are

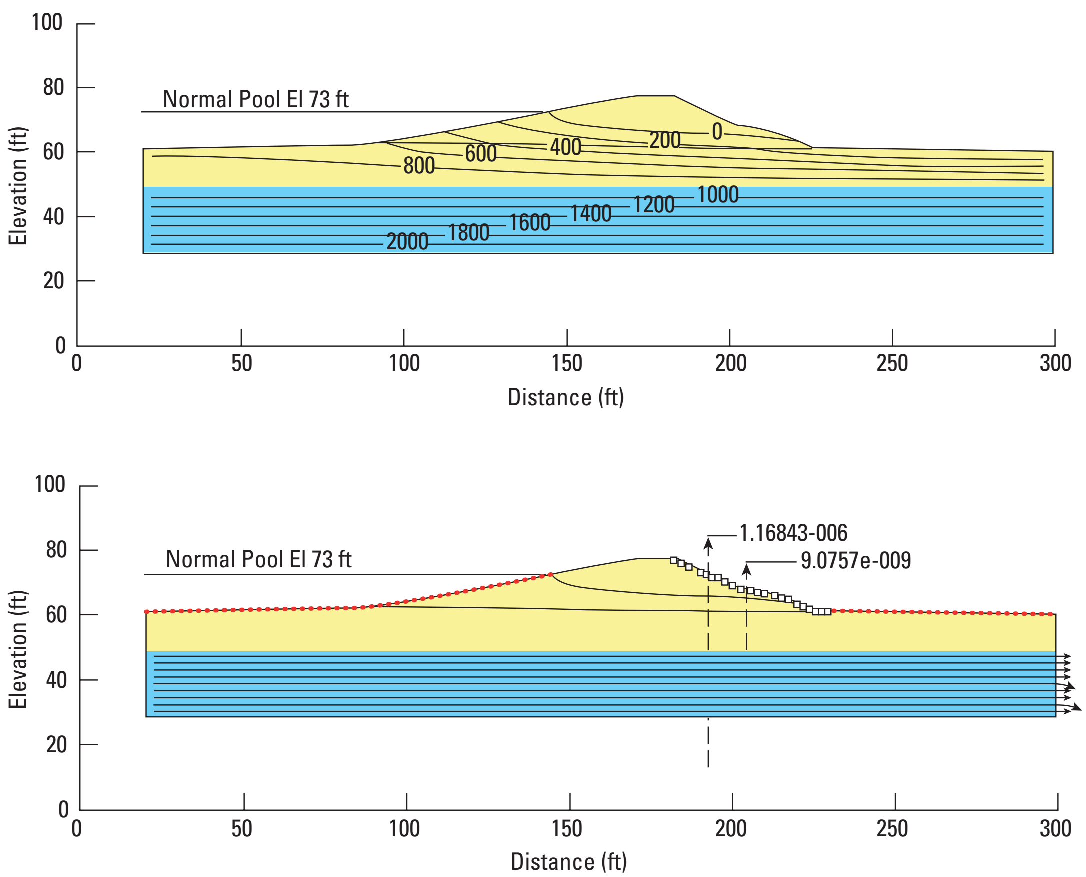

FIGURE 5-5. Example of results from SEEP/W finite element seepage analysis.

highly dependent on the soil and rock permeabilities used as input for the analysis. The permeability of a material is one of the more difficult properties to estimate in geotechnical engineering, and typically it is not known more precisely than one order of magnitude. Consequently, it is common practice in geotechnical engineering to increase the calculated seepage quantities by a factor of five to ten for sizing of drainage system components (e.g., sand filter layers, gravel drain layers, collector pipes, etc.).

RCC spillways and overtopping protection are often used for modifications to existing dams. In many cases for existing structures, seepage performance data is available in the form of piezometer readings and weir flow readings (or other measurements of seepage quantities). When existing data is available or instrumentation can be installed to collect the necessary data, the seepage analysis model should be calibrated to be consistent with this known data for existing conditions before the model is modified and used for design of dam modifications.

5.3 ANALYSIS OF UPLIFT PRESSURES

Unlike the case of steady-state seepage through an embankment, the method of analysis for uplift beneath RCC is not well established. The problem is simple, as illustrated in Figure 5-6. The combined weight of the RCC mass and the water on top of the RCC must be sufficient to resist the uplift pressure beneath the RCC. The weight of the RCC is relatively easy to calculate. However, there are significant uncertainties in the calculation of the depth of water on top of the RCC and especially in predicting the water pressure that could develop beneath the RCC.

The depth of water on top of the RCC would typically be calculated using water surface profile models, which is subject to the uncertainty inherent in those analyses. The depth of water during maximum overtopping would be calculated using a steady-state water surface profile analysis. The depth of water on top of the RCC as the flood recedes would need to be calculated using a transient water surface profile and would be affected by the degree of accuracy of knowledge of the inflow flood hydrograph and the downstream conditions that would control tailwater.

The water pressure beneath the RCC is the result of transient flow and seepage conditions. The conditions leading to water pressure beneath the RCC are: (1) water can pass through cracks or joints in the RCC as water flows over the layer; (2) water can seep through the embankment or foundation to a wide area beneath the RCC if the overtopping duration is long, and (3) if erosion occurs at the upstream end of the RCC, then seepage or infiltration can more easily occur beneath the RCC. As a flood abates, the reservoir level and the depth of water on top of the RCC will reduce. At the same time, the pressures

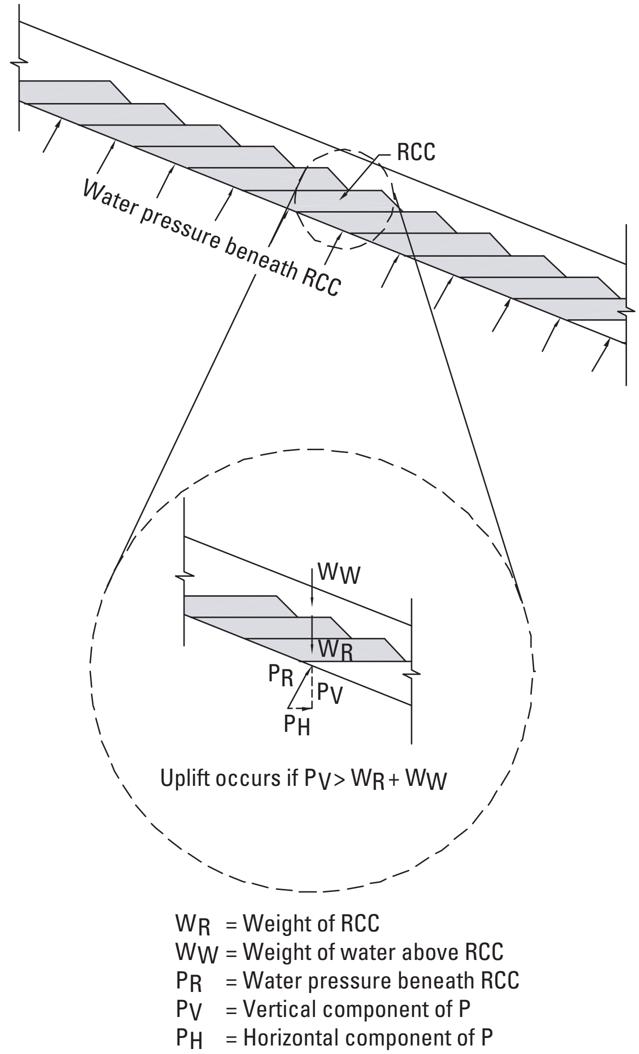

FIGURE 5-6. Physics of uplift for RCC overtopping protection.

beneath the RCC will begin to dissipate by flow out through cracks or joints, and flow through any under-drain system beneath the RCC. The dissipation of the pressures is a transient seepage problem, which can be analyzed by computer-based programs. However, in geotechnical engineering, the degree of accuracy of transient analyses of seepage is less than that of steady-state seepage analyses.

Because of the uncertainty in the analyses, the uplift condition is not often analyzed in detail. Instead, this challenge is often addressed by geotechnical engineers using simplified analysis models and judgment. One approach used within the profession is that the pressures beneath the RCC cannot be sustained significantly above the level of the water on top of the RCC because of the hydraulic communication provided by cracks and joints in the RCC. This approach is particularly valid if the cracks are narrow, which is typical for RCC and if a geomembrane strip is placed beneath joints to reduce infiltration. Following this school of thought, unbalanced uplift pressures would not build up significantly and no special design features other than joint details would be needed to address this concern. Another approach suggests that high pressures will build up under the RCC during overtopping and may not drain quickly, unless the RCC is underlain by a relatively high-capacity drainage system (e.g., a gravel drainage layer).

Few RCC spillways or overtopping spillway installations have been tested by significant flows, so at this time it is not known which of these approaches is most representative of the field conditions. However, those projects that have been tested have performed quite well. It is likely that more appropriate analysis methods will be developed as installed systems are tested by significant flood events. In the interim, it is left to each designer to evaluate this issue and analyze it as they see fit. Since cracks in the RCC provide a potential path for seepage that could lead to uplift pressures and possible slab jacking, the RCC should be designed to produce as few cracks and joints, as widely spaced, as practical.

Until better methods of analysis are developed, it is recommended that RCC spillways and overtopping spillway installations include as a minimum under-drains or pressure relief systems spaced approximately every 10 vertical feet. Typical details for an under-drain system are illustrated in Figure 5-7. Pipe drains that extend through sloping RCC sections should be designed to provide aspiration at the outlet end, so that they drain properly during flow over the RCC.

Pressure relief systems should also be included beneath horizontal runout or energy dissipation sections located at the downstream ends of the spillway sections, as is customarily done with CVC spillway stilling basins.

5.4 ANALYSIS OF FILTER COMPATIBILITY

Methods for analysis of filter compatibility are well established in the geotechnical engineering profession, and comprehensive criteria have been published by numerous organizations (U.S. Department of Agriculture, Soil Conservation Service, National Engineering Handbook, Chapter 26, August 2017; USACE EM 1110-2-2300, July 2004; U.S. Department of Interior, Bureau of Reclamation, Design Standard 13, Chapter 5, November 2011).

In principle, filter compatibility guidelines are developed to provide criteria to design a filter layer composed of soil with a grain size distribution which results in pore spaces that are small enough to prevent movement of soil particles from the "base" soil (embankment or foundation material) that is being filtered. When a filter is placed against a base embankment or foundation soil that varies in gradation, the filter must provide adequate protection to the finer grained soil. If a drainage system contains multiple layers (e.g., sand filters and gravel drains), then filter criteria must be met at each successive boundary. For example, the sand filter would need to provide adequate protection to base embankment or foundation soil, and the gravel would need to provide protection to the base sand filter. Similarly, if the drainage system includes slotted or perforated collection pipes, the slots or perforations would need to provide filter compatibility with the immediately surrounding filter medium. Published criteria for filter compatibility of slots and perforations are provided in the references noted previously in this section.

Filter compatibility must also be addressed for the material beneath cracks through the RCC. If the soil beneath the cracks is fine enough, relative to the width of the crack, then the soil

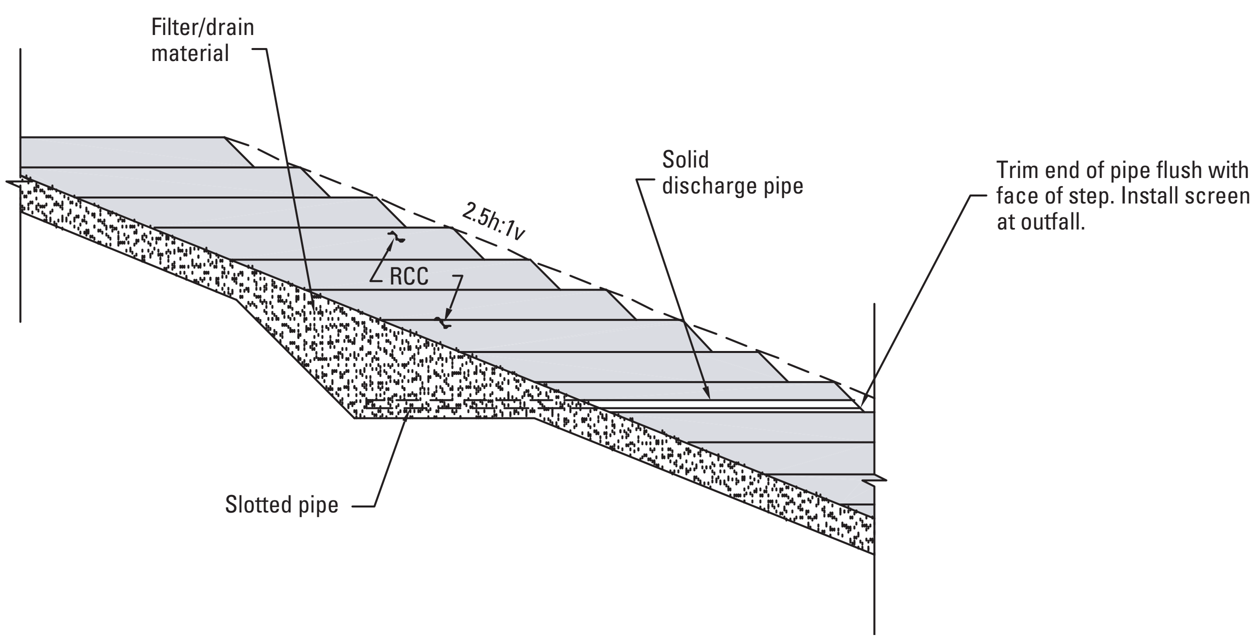

FIGURE 5-7. Typical details of an under-drain and outlet pipe.

can be washed out by flow through the crack. Flow through the crack could result from steady-state seepage, the release of water that infiltrated under the slab during overtopping, or precipitation events. If fine grain soils or fine sand filter material is in direct contact with the RCC and cracks are expected to occur in the RCC, then a filter compatibility analysis for the material immediately below the RCC and the expected crack width should be performed using the same methods used for slotted pipes, as described previously. The use of a geotextile immediately below the RCC as filter to prevent particle migration through cracks may be an option, but there are several long-term performance concerns related to geotextiles in dams as discussed in the following text.

Steady-state seepage is the most critical of the sources because the sustained duration seepage from this condition could lead to an internal failure of the embankment over the long term, if left uncontrolled. However, loss of soil from the other two sources could lead to formation of voids under the RCC, which could still be a significant problem that may be difficult to detect in advance of a serious dam safety incident or dam failure.

Geotextiles have been used to serve the filter function in some RCC spillway and overtopping protection applications. However, the designer should be cautious in the use of geotextiles in this application because they will be isolated underneath the RCC, and therefore, difficult to access for repair or replacement. Designers should check with their local regulatory agencies to see if geotextiles are allowed for their specific application. The history of the use of geotextiles for these types of applications is short, relative to the experience with soil filters. The potential for long-term deterioration or plugging of geotextiles is yet to be firmly established. Until the long-term performance of geotextiles in dam applications is better established, it is not recommended that geotextiles be used in an application where their function is critical to dam safety. Non-critical applications may be reasonable, subject to consideration of the potential for clogging, construction damage, sealing geotextiles around penetrations, and limited access to the geotextile for repair or replacement in the future.

CHAPTER 6

HYDRAULIC STRUCTURES DESIGN

6.1 INTRODUCTION

The design of hydraulic structures, such as spillways, requires a comprehensive knowledge of civil, structural, and hydraulic engineering, specifically with dams. This chapter provides an overview of RCC spillway design, including embankment overtopping protection projects. Experience with the design and performance of spillways, supplemented by specific technical references, is needed to understand the requirements for the spillway design. The design elements discussed in this chapter include:

- Spillway Location
- Hydraulics of Stepped Spillways
- Spillway Channel
- Width of Overtopping Protection
- Spillway Crests and Control Structures
- Approach Apron (Crest) Slab
- Downstream (Runout) Apron Slab
- Cut-off Walls
- Joints for RCC Spillway Slabs
- Drain Outlets
- Training Walls and Abutment Protection
- Soil Cover for RCC Spillways

6.2 SPILLWAY LOCATION

An RCC spillway can be located in three general configurations: (1) as a spillway separate from the dam embankment, (2) as spillway overtopping protection over the entire length of the dam embankment, and (3) as spillway overtopping protection conveying flow over only a portion of a dam embankment. The general design configurations are shown schematically in Figure 6-1.

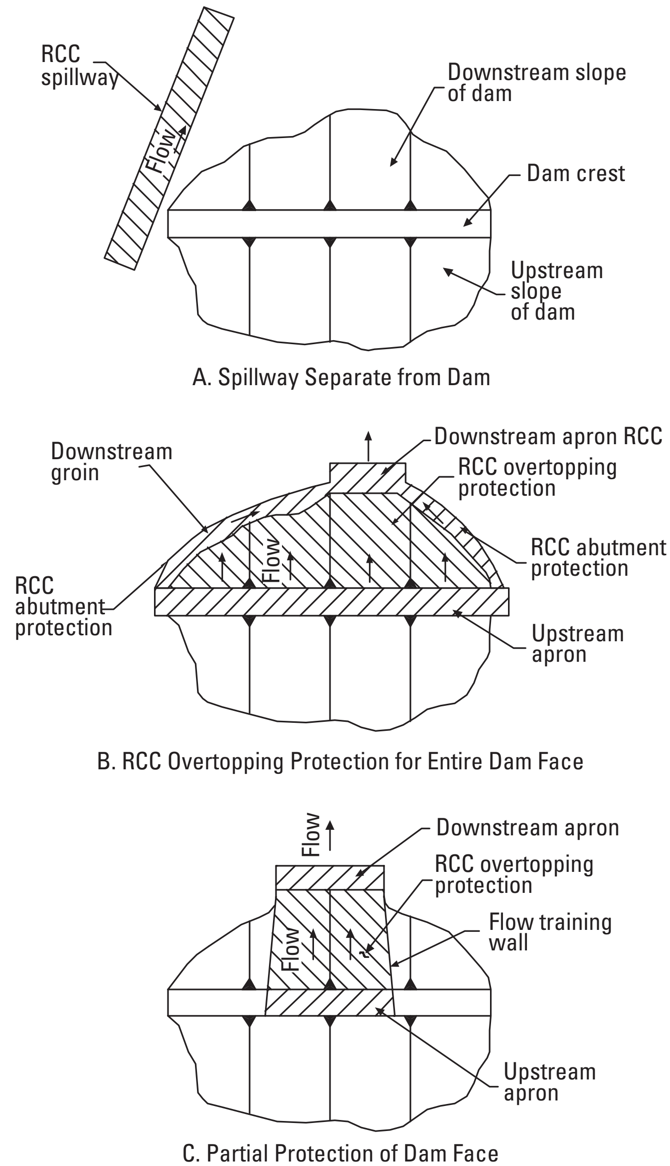

FIGURE 6-1. Alternative spillway locations.

The location of the spillway is one of the most important decisions of spillway design because the location has implications for dam safety and public safety, as well as hydraulics and energy dissipation, aesthetics, cost, and maintenance. When determining the location and configuration of the spillway, apron, and training walls, the designer should give preference to a location that::

- Is separate from the dam embankment when possible. Note that the armored length of a spillway (in an upstream/downstream direction) located on the abutment may be significantly greater than a spillway constructed over the embankment with an equivalent crest width. Therefore, it may be more economical to decommission the existing spillway located on the abutment and construct a new RCC armored spillway over the embankment.
- Would not cause excessive erosion at the downstream embankment groins or at the downstream toe of the dam.

- Is aligned with the downstream channel to minimize channel erosion and safely convey discharge away from the dam.

Overall concerns and some of the implications of the location selection and overall design requirements for an RCC spillway project are discussed in Chapter 2.

6.3 HYDRAULICS OF STEPPED SPILLWAYS

Stepped channels have been used for more than 3,000 years (Chanson 1995). In the early structures, overflow stepped spillways were used to enhance the stability of the dam for their simplicity of shape and then later to reduce flow velocities. Early irrigation systems in Yemen and Peru included drops and steps to increase energy dissipation. From the 16th to 18th centuries, large, stepped fountains were built in Europe and India, some larger than any existing stepped spillways. At the end of the 19th century, a significant number of dams were built with overflow stepped spillways (Schuyler 1909, Wegmann 1911, and Chanson 1995). Most were masonry or CVC structures with granite or concrete blocks protecting the downstream face. Since the beginning of the 20th century, stepped spillway chutes have been designed more specifically to dissipate flow energy. Stepped chutes significantly increase the rate of energy dissipation on the downstream face of the dam. The energy that is dissipated on the steps reduces the required size of the energy dissipation structure at the base of the dam and the potential for scouring of the stream channel and/or foundation material.

The development of the RCC construction technique renewed interest in stepped channels and spillways in the late 20th century. The construction of stepped channels is compatible with the placing and forming methods for RCC. It has been well established in hydraulic engineering that steps constructed with RCC provide energy dissipation/reduction. The unit discharge, width and height of the steps, overall slope of the spillway protection, and length of the chute should be considered in conjunction with design of the stilling basin. Model studies have been conducted for stepped spillways on the relatively mild downstream slope of embankment dams. Papers describing the model studies are summarized in a book entitled *Hydraulic Design of Stepped Cascades, Channels, Weirs and Spillways* (Chanson 1995). For more information see Houston 1987, Troupe and others 2001, Ohtsu 2004; USBR 2015; Hunt 2018; Hunt and Kadavy 2013, 2017, 2018, and 2021; Hunt and others 2012 and 2014; and Woolbright and others 2008.

6.4 SPILLWAY CHANNEL

The spillway channel can be analyzed and designed in three discrete components: (1) the approach apron slab, (2) the sloped chute, and (3) the downstream apron slab. A typical section through an RCC spillway is shown in Figure 6-2.

The chute is the portion of the spillway that conveys water down into the downstream channel. RCC for the chute is typically placed in horizontal lifts, as shown in Figure 6-3. RCC chute surfaces constructed in horizontal lifts can be constructed without formwork (Figure 6-3A) or by using vertical forms to create a more pronounced stepped chute surface (Figure 6-3B).

RCC for the chute can also be placed parallel to the sloped surface, as shown in Figure 6-4. Placement parallel to the slope (referred to as "plating") has been considered for projects where the depth of overtopping is less than 2 feet, the duration of overtopping is relatively short, and the spillway slope is 3H:1V or flatter. This design configuration can result in cost savings because of the thinner, nominal thickness of RCC. However, the reduced thickness may also reduce the resistance to uplift forces as compared to RCC placed in horizontal lifts. The plating configuration also dissipates less energy on the RCC surface than a stepped section, which would therefore require more energy dissipation at the base of the spillway. One plating application was at Toutle River (Figure 6-5) where one of the primary design considerations was to provide a structure that would allow debris from the Mount St. Helens volcano eruption to flow through the spillway. Some armoring of existing vegetative auxiliary spillways and spillway outlet channels have used the plating method of construction because of the mild exit slopes. Figure 6-6 shows the RCC plating method used to armor an auxiliary spillway.

The discussion of spillway chutes design in this chapter is intended for RCC placed in horizontal lifts, although much of this information could apply to RCC chutes placed parallel to the slope.

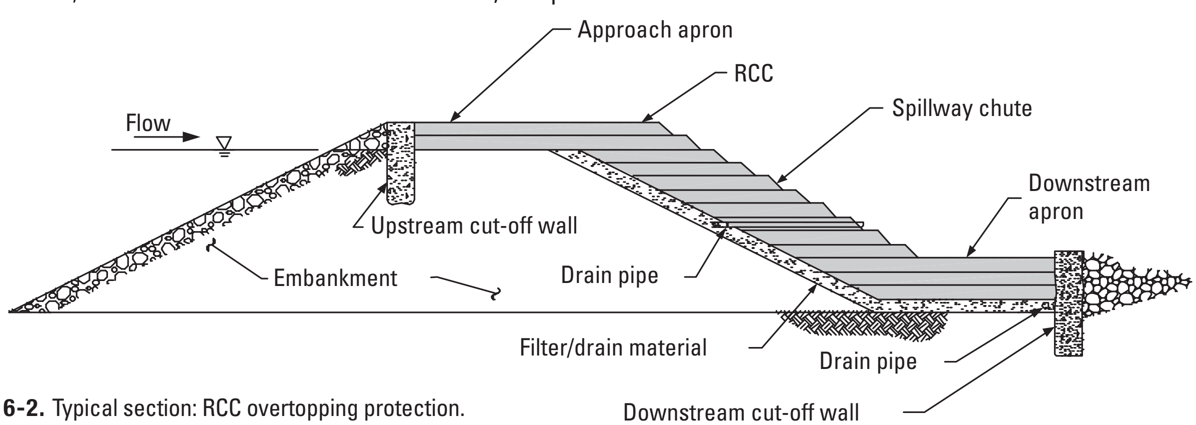

FIGURE 6-2. Typical section: RCC overtopping protection.

Unformed Chute

Unformed RCC chutes are usually less expensive and take less time to construct than formed RCC chutes. Unformed RCC is usually end dumped by trucks or placed by a loader and spread by a bulldozer. Compaction is performed by single drum or double drum vibratory rollers. During compaction, the unrestrained (unformed) face can result in RCC that is not fully compacted near the outside of the edge (the cross-hatched area shown in Figure 6-3A). The outside edge typically results in lower density RCC that can ravel and erode over time. Raveling would generally be limited to the depth of the more densely compacted RCC. In an unformed chute, this zone of lower RCC density should be considered as "sacrificial concrete" by the designer. A conservative design approach would be to not consider the sacrificial zone RCC as part of the wearing surface, and not include the material in the mass for the embankment stability analysis or uplift resistance computations.

An unformed RCC face can have the appearance of rough, irregular shaped CVC that has an exposed aggregate appearance and will often have exposed rock pockets. To some, an uncompacted RCC surface can have the appearance of poorly constructed or damaged CVC. Examples of the appearance of unformed, uncompacted downstream RCC faces are shown in Figure 6-7. To others, the rough, irregular appearance pleasingly blends into the natural surroundings. If a

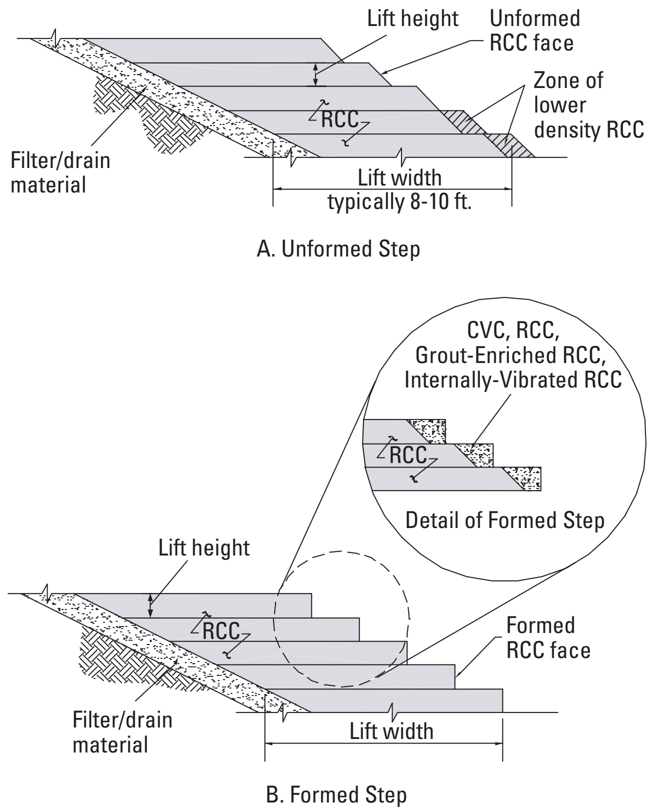

FIGURE 6-3. Downstream slope geometry of RCC overtopping section.

FIGURE 6-5. RCC plated spillway (Toutle River Dam, OR).

FIGURE 6-6. RCC plated armoring of auxiliary spillway (North Fork Dam, PA).

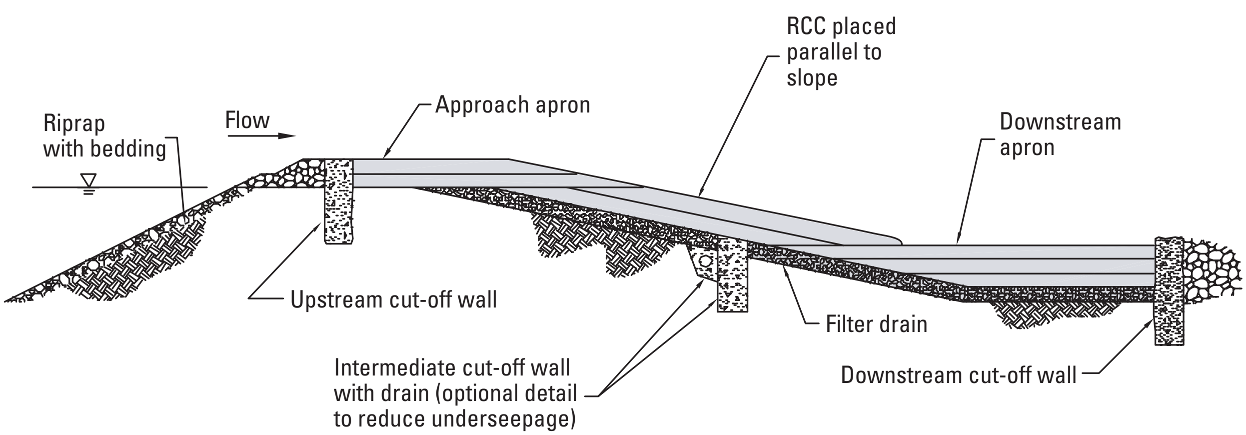

FIGURE 6-4. Overtopping protection with RCC placed parallel to chute (plating).

smoother finish surface is an important project requirement, the exposed RCC edge can be compacted or trimmed to give a more uniform CVC surface appearance. Compaction of the exposed RCC face will increase the RCC density and reduce raveling; however, scattered rock pockets will still be encountered. RCC mixes with a Vebe time less than 15 seconds are not well-suited for unformed steps. These mixes tend to spread out when compacted, making it difficult to maintain the proper thickness at the outer edge. Examples of unformed, compacted downstream RCC faces are shown in Figures 6-8, 6-9, and 6-10. Methods of trimming and compacting the outside edge are discussed in Chapter 9. The Natural Resources Conservation Service (NRCS) includes tolerances for unformed, exposed RCC faces (0.1-ft in 10-ft variation along the length of the face) in their standard Construction Specifications that can be specified to limit surface irregularities in unformed compacted steps.

Because unformed steps are not usually vertical but constructed on an angle, the amount of energy dissipation on the steps is assumed to be reduced from the traditional formed steps. To the authors' knowledge, there is no research on energy dissipation on unformed stepped spillways. It is assumed the angled face of the unformed steps will produce fewer recirculating vortices, and thus less energy dissipation than steps with a vertical face.

Formed (Stepped) Spillway Chute

When vertical forms are used to restrain the outside edge of the RCC lift during spreading and compaction, a stepped surface is created as shown in Figure 6-3b. The vertical form also provides confinement to the outside edge of the lift so that the RCC can be compacted against the form, resulting in higher RCC densities at the outside edge than can be achieved with unformed RCC.

The advantages of forming the outside edge of the RCC lift include: (1) a potential for increased energy dissipation on the chute surface; (2) higher RCC densities at the outside edge of a lift increases strength and durability of the outside edge, reduces raveling, and increases freeze-thaw resistance; and (3) the appearance of a formed surface (when well-constructed).

FIGURE 6-7. Uncompacted, unformed RCC downstream face (Bishop Creek Dam No.2, CA).

FIGURE 6-8. Compacted, unformed RCC downstream face (Mona Dam, UT).

Energy dissipation occurring on the steps is affected by the step height, depth of flow, slope of the spillway, and length of the spillway chute. Hydraulic model studies for sloped, stepped spillways associated with embankment overtopping projects is available in the papers summarized in Section 6.3. Technical information is available on the hydraulics of steeper sloped spillways associated with CVC gravity dams. However, the data will not directly apply to RCC spillways and embankment overtopping projects.

The step height can affect the cost, constructability, energy dissipation, and public access to the spillway area. Step heights

FIGURE 6-9. Unformed RCC chute with compacted RCC (Speedwell Forge Lake Dam, PA).

FIGURE 6-10. Unformed RCC chute with compacted RCC (Picati Lake Dam, NJ), courtesy of KC Construction.

for RCC spillways generally use 1- to 2-foot-high vertical forms; however, steps as high as 4 feet have been constructed. Contractors typically prefer higher steps because forms do not have to be jumped as frequently. Higher step heights have been used on gravity dams to provide increased energy dissipation for large spillway discharges and/or to inhibit public access on the downstream slope of a dam. As the step height increases, the form strength and the bracing requirements will become greater, as will the potential for public fall hazards, which should be considered during design. Greater step heights can also result in larger RCC volumes as shown in Figure 6-11.

Placement of RCC against a formed surface requires a more "workable" RCC mix than for a non-formed surface. Enhanced workability is required for consolidation of RCC against the formed surface to produce a smooth finished surface and to minimize rock pockets (the NRCS requires 80% of the formed RCC face to be free of honeycombs or other voids in their standard Construction Specifications). The workability of the RCC near the formed surface can be increased by: (1) providing an RCC mix with a higher cementitious (and/or non-plastic fines) content; (2) using pozzolans or additives; (3) increasing the w/cm ratio; and (4) lowering the Vebe time to under 15 seconds. The workability of the RCC adjacent to the forms has also been improved by enriching the RCC near the formed surfaces with a cement grout (grout enriched RCC, or GERCC). Examples of completed projects with a formed RCC chute are shown in Figures 6-12, 6-13, and 6-14A. Figure 6-14B illustrates one project where formliners were used to construct a small RCC dam to simulate the appearance of stone masonry.

The disadvantages of forming include: (1) decreased RCC placement rates; (2) increased requirements for laborers and carpenters to install, strip, and move forms; and (3) increased project costs. The Owner should be included when making decisions to use formed or unformed chute surfaces for the project and the importance of esthetics.

Sloped RCC Chute Thickness

The thickness of the RCC chute is commonly measured perpendicular to the spillway slope. The required thickness of the RCC chute is based upon the slope of the spillway, constructability requirements for efficient placement of RCC, and the structural requirements to resist uplift pressures and other loading conditions. The thickness of the RCC perpendicular to the slope is also dependent upon the lift width, which is typically at least 8 to 12 feet wide to accommodate common construction equipment used to place, spread, and compact RCC. These relationships are graphically shown in Figure 6-15.

The RCC chute must be designed to resist uplift pressures that may exist on the RCC slab, as discussed in Chapter 5. The location of the maximum uplift pressure under the slab is often found near the bottom of the slope, just above the base of the spillway. The RCC chute slab in this area may need to be designed for a loading condition similar to that used for the downstream apron slab. Most designers have adapted a minimum thickness of 2 ft. The thickness is generally increased as the overtopping depth increases. A graphical representation of this loading condition is shown in the design example in Appendix A. Design for uplift loadings on spillway slabs are

FIGURE 6-12. Formed RCC chute (Toby Creek Impounding Basin, PA), courtesy of KC Construction.

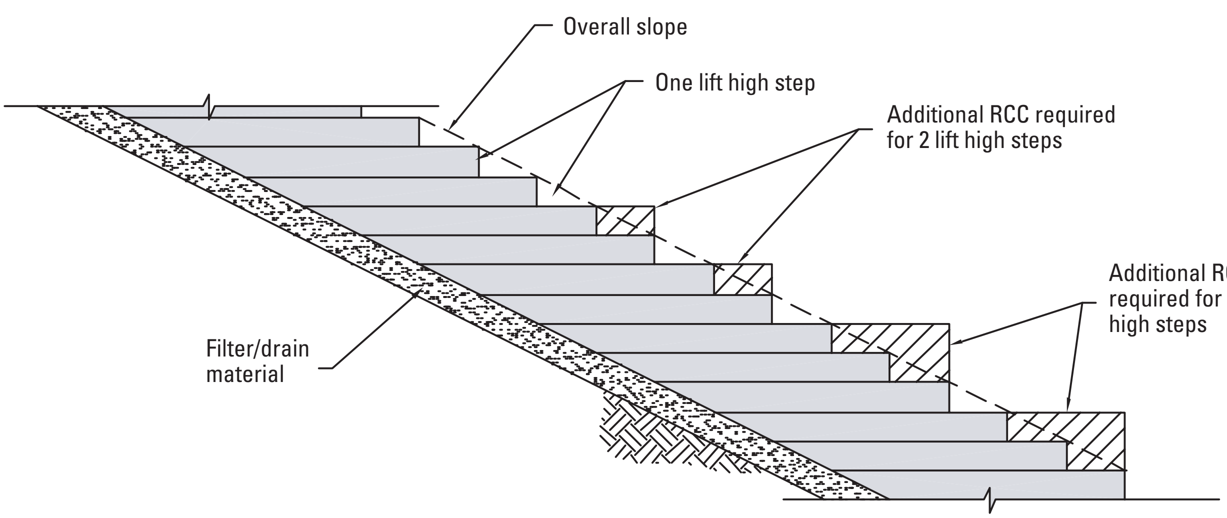

FIGURE 6-11. Step height comparison of RCC volume.

FIGURE 6-13. Formed grout enriched RCC steps.

FIGURE 6-14A. Formed RCC steps, 1-ft high (Stoney Creek Dam, VA).

FIGURE 6-14B. Formed liners used to simulate appearance of stone masonry (Gibson Pond Dam, SC).

published in design guides for spillways, such as "Design of Small Dams" (USBR 1987a) and "Hydraulic Design of Spillways" (USACE 1990). The loadings may require modification to account for less-than-ideal hydraulic conditions that can exist at the base of an overtopping spillway. However, the principles for this analysis are similar to the established design procedures for stilling basins and spillway slabs.

6.5 WIDTH OF THE OVERTOPPING SPILLWAYS

The width of an overtopping spillway is affected by both technical and economic considerations. Issues to consider when deciding on the length of dam crest to be used as width of the overtopping spillway protection include:

- Decreasing the required width by using a high capacity control section. (refer to section 6.6)
- Energy dissipation – A wider spillway will usually improve spillway performance by decreasing the depth of flow and the unit energy at the base of the spillway and increasing energy dissipation. Energy dissipation requirements become more important as the height of the dam and unit discharge increase. (High head/high unit discharge designs should be avoided or will need special design considerations. RCC overtopping protection is not a substitute for a high capacity, CVC service spillway or a spillway in bedrock.)
- Extending the RCC spillway across the entire crest of the dam and down the abutment groins will maximize the available spillway crest length and decrease the maximum water surface level. This configuration will also match existing site hydraulics and avoid an increase in downstream flooding compared to existing conditions.
- Conversely, the designer may want to reduce the crest width of the overtopping spillway to decrease the amount of flow at the abutment groins of the embankment, and to provide a better transition from the spillway channel to the natural channel. A narrower spillway will usually fit if the downstream channel is significantly narrower than the dam crest.
- Cost – Wider overtopping spillways will usually increase the size of the RCC chute, weir crest and stilling basin, and overall project costs.
- Converging spillway – A converging spillway can be used to transition a wider crest length to a narrow downstream channel to reduce construction costs. This design configuration requires consideration of the effects of wall convergence on spillway cross-waves and wall overtopping.

The transition from the spillway width to the downstream floodplain and channel is also important from both operation, maintenance, and land rights perspectives.

6.6 SPILLWAY CREST AND CONTROL STRUCTURES

RCC spillway crests can simply follow the shape of the embankment crest forming a broad crested weir as shown in Figures 6-16 and 6-17A, which has a relatively low hydraulic efficiency. By selecting a more efficient weir shape (i.e., a crest section with a higher discharge coefficient, Figure 6-17), the required spillway width can be reduced along with the peak reservoir level and/or flow depth through the spillway, which can reduce overall project costs. Therefore, it is important for the designer to be aware of the following alternative spillway crest designs that have been used with RCC spillways:

- Labyrinth weir (Figure 6-18). Very efficient and reduces the spillway outlet channel width.

- Sharp crested weir which can be constructed as an extension of an upstream cut-off wall (Figures 6-17B and 6-19). Simple design but only modest increase in efficiency.
- Ogee crest (Figures 6-17C, 6-20, 6-21, 6-22). Used frequently. Requires a lot of mass CVC.
- Flat curved crest, (Figure 6-17D). Requires less CVC than an Ogee but with less efficiency.
- Fuse plug (Figure 6-23). Only used once. May not be very reliable.
- Fuse Gates (Figures 6-24 and 6-25). Only used once. Has replacement costs.

CVC can be used to construct efficient crest control structures. While the spillway width may be reduced, efficient crest shapes

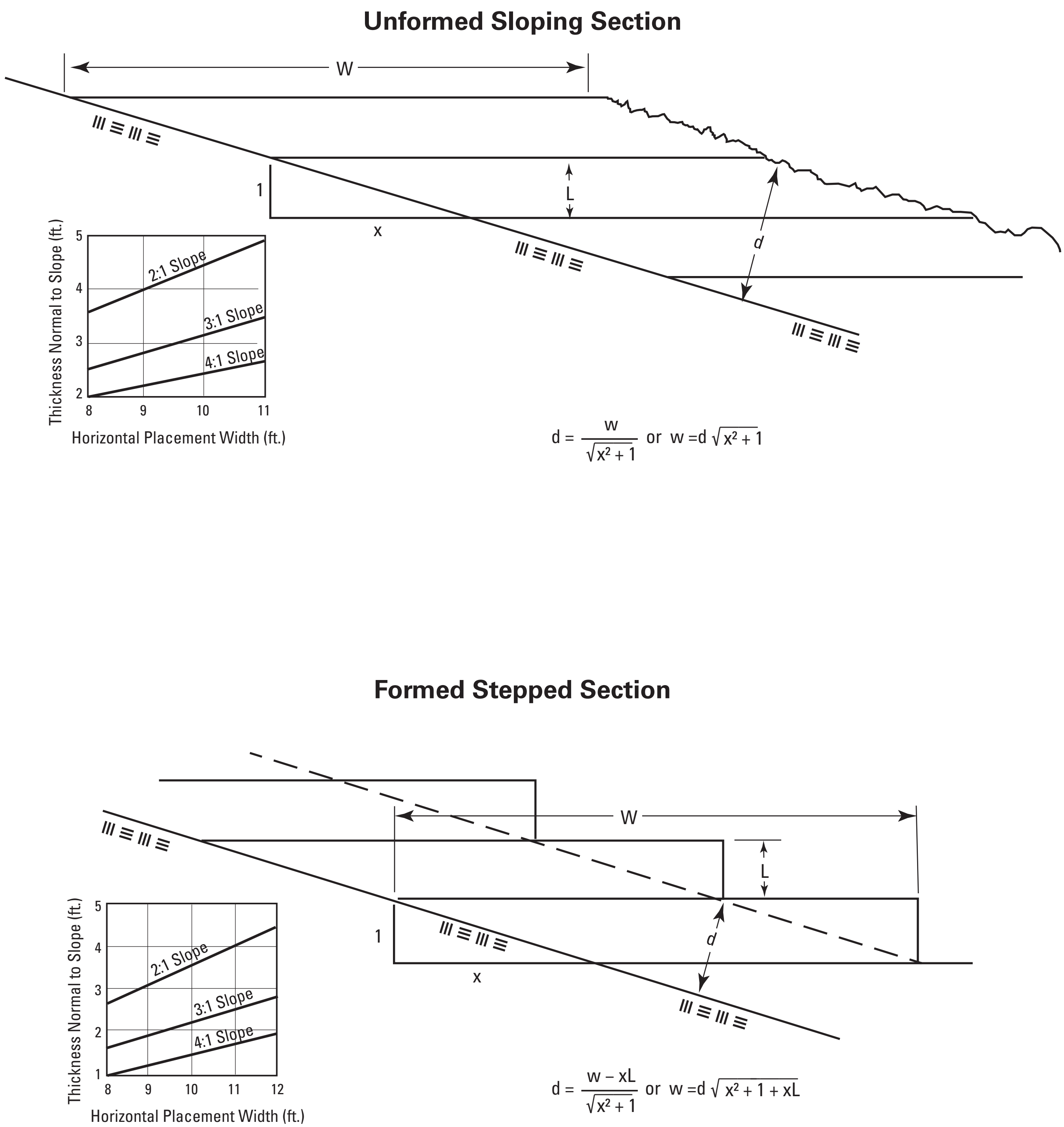

FIGURE 6-15. Thickness of RCC on the slope versus the width of the lift. Facing thickness (d) of RCC versus width (w) for a 1-ft thick (L) unformed RCC step.

FIGURE 6-16. Broad crested weir (Great Gorge Dam, NJ).

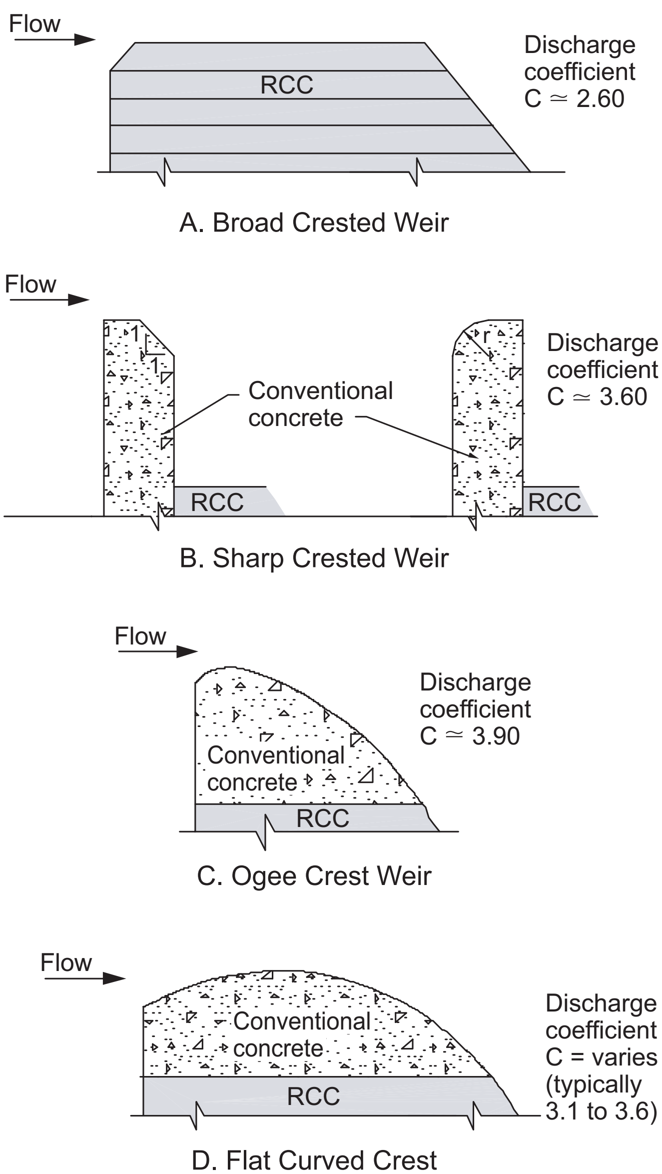

FIGURE 6-17. Alternative weir crest shapes.

FIGURE 6-18. Labyrinth crest weir (Standley Lake Dam, CO).

FIGURE 6-19. Sharp crested weir (Smith Lake Dam, VA).

FIGURE 6-20. Side view of installation of reinforcement for ogee crest weir (Fox Creek MPS No.4, KY).

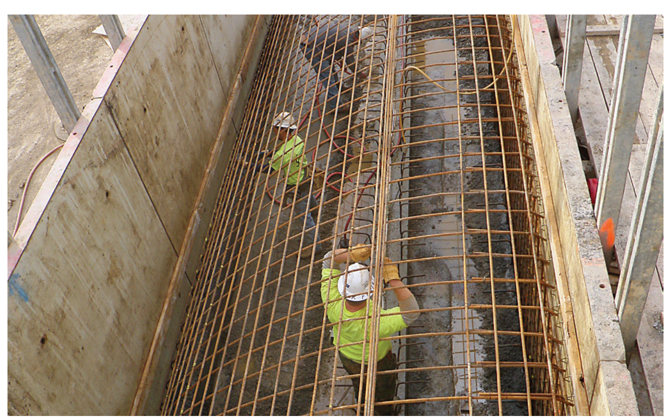

FIGURE 6-21. Top view of installation of reinforcement for ogee crest weir (Fox Creek MPS No. 4, KY).

FIGURE 6-22. Ogee crest weir (Fox Creek MPS No. 4, KY).

FIGURE 6-23. Fuse plug (Gunnison Dam, UT).

FIGURE 6-24. Fuse gates (Black Rock Dam, NM).

FIGURE 6-25. RCC spillway prior to installation of fuse gates shown in Figure 6-24 (Black Rock Dam, NM).

often require more CVC, reinforcement steel, and formwork, may complicate construction and limit future access compared to a broad crested weir.

Constructing highly efficient crest designs such as an ogee shape or labyrinth weir often requires significant steel reinforcement and highly skilled labor that may increase the unit cost of the RCC placement. Crest designs such as sharp crested weirs or modified ogee/flat curved shapes can be used to improve spillway hydraulics but with lower unit costs than an ogee section or labyrinth weir.

Note that discharge coefficients vary with the approach channel conditions, approach depth conditions, depth of flow over the weir, and tailwater conditions. Refer to general design references (USBR 1952 and 1987a, USACE 1996, and Brater and King 1976) for a discussion of these effects.

6.7 APPROACH APRON (CREST) SLAB

General

The approach apron slab is located upstream of the spillway crest control section and sloped chute. The approach apron functions to reduce erosion, to lengthen the under-seepage path, and to reduce seepage that could occur from the reservoir under the spillway chute (Figure 6-26).

Approach Apron Length

The design of the apron needs to be compatible with the internal geometry of the dam. The apron needs to extend far enough upstream so that the length is sufficient to reduce the potential for piping of embankment materials or excessive seepage from developing through the embankment, under the apron slab and crest section, and under the sloped RCC chute. An upstream cutoff wall is an important design feature for lengthening the seepage path under the approach apron and to prevent erosion and undermining of the upstream edge of the RCC apron. Design of the cutoff wall is discussed in Section 6.9. Seepage under the approach slab and spillway chute can cause excessive uplift pressure or saturation and instability of the embankment. Seepage analysis of the embankment may be required to design the required apron length upstream and the depth of the cutoff wall to control seepage and uplift pressures. Chapter 5 discusses spillway under-drainage requirements.

Approach Apron Thickness

The thickness of the approach apron is controlled by the requirement to provide adequate weight to resist uplift. When determining the minimum thickness of RCC, the designer should consider freeze-thaw and long-term weathering protection, and frost heave. Two 12-inch lifts of RCC should be considered as a minimum thickness for constructability and serviceability of an RCC approach apron. In regions where frost depth exceeds 3 feet, the designer should consider increasing the apron

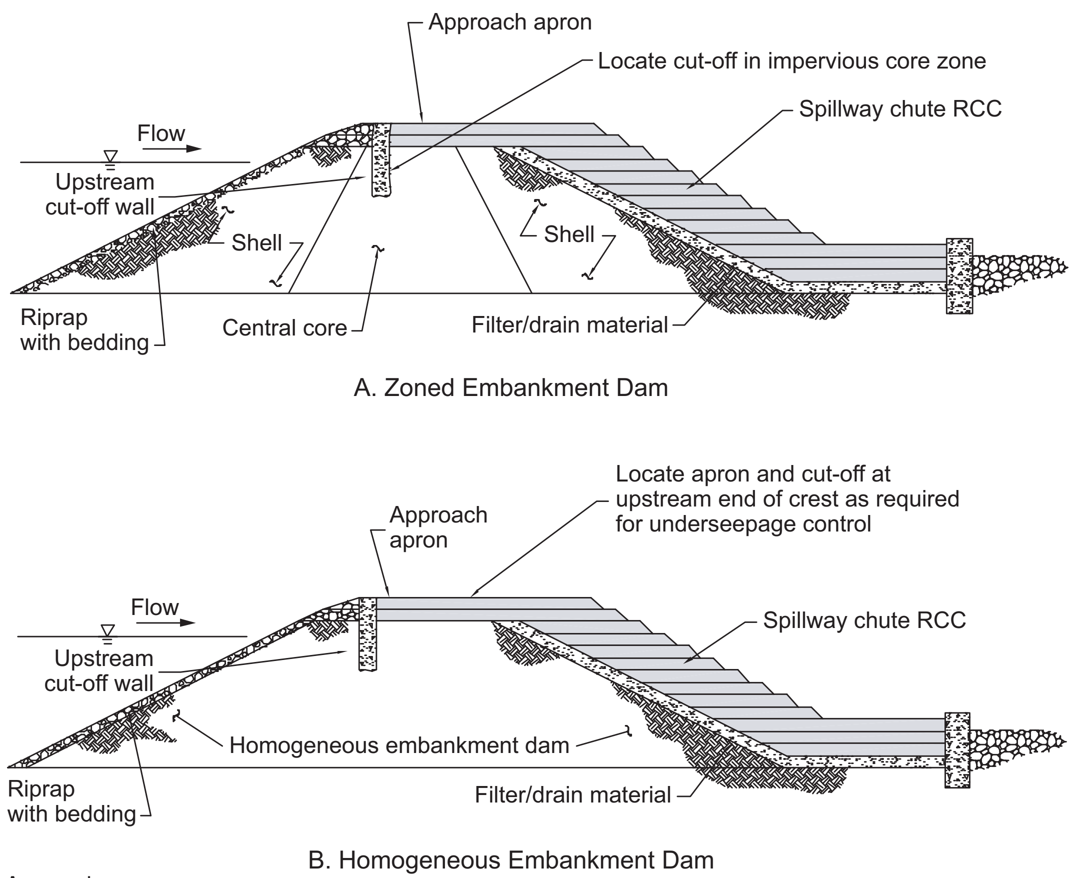

FIGURE 6-26. Approach aprons.

thickness. Bedding mortar should be placed between each lift of RCC in the approach apron regardless of the freshness of the lift surface to reduce the potential for delamination during overtopping flow. Refer to Section 9.11 for details about bedding mortar.

6.8 DOWNSTREAM (RUNOUT) APRON SLAB

General

The primary function of the downstream apron is to protect the RCC spillway and the dam embankment if overtopping protection is used from erosion caused by spillway flow. The length and thickness of the downstream apron depends upon energy dissipation and erosion control requirements. For embankment armoring, construction of complex stilling basins is not typically considered cost-effective, particularly where the armoring is applied to the entire embankment, resulting in a relatively wide chute and stilling basin. Therefore, a simple apron with or without an end sill are most applicable to these projects.

The downstream apron is one of the most critical features of the RCC spillway design, especially when the RCC spillway is located over the dam embankment. The designer must have a thorough understanding of the spillway and channel hydraulics, foundation conditions for the spillway, and how the apron design protects the spillway and dam from erosion. A conservative approach for designing the downstream apron is to utilize competent bedrock as the foundation, which may not be practical for some sites where rock is located excessively deep. The apron can also be located at an adequate depth below tailwater, and with adequate length, so that a hydraulic jump will form on the apron following methodologies described in *Hydraulic Design of Spillways* (USACE 1990).

The designer must also determine the erosion potential of the soil or rock downstream of the apron. As noted previously, most embankment armoring projects are designed for significant storm events, typically the 100-year flood or greater. The goal of embankment armoring is to protect against a dam failure and uncontrolled release of the reservoir. Some erosion of the area downstream of the armored embankment may be tolerable. Placement of riprap downstream of the apron is recommended to reduce scour potential.

The estimated depth of erosion and channel degradation can be determined for the full range of spillway operational flows. Estimates of degradation, scour, and erosion below a spillway should be developed by a hydraulic engineer experienced in channel hydraulics. Erosion and scour downstream of an RCC spillway are dependent on numerous variables and are difficult to model. *Design of Small Dams* (USBR 1987) provides an empirical equation for estimating ultimate scour depth; however, this method is likely overly conservative for embankment overtopping projects as described in Paxson

(2007). Scour downstream of a horizontal apron has been researched and methods for estimating scour are provided in several references, including Hassan and Naryanan (1985), Sarkar and Dey (2005), Dey and Sarkar (2006), and Oliveto (2009).

The hydraulic conditions that occur at the toe of the dam are usually less than ideal. Lateral flow can occur along the groins and tailwater may be insufficient. Therefore, design aids may not be able to be used for design and physical or numerical modeling may be required. Refer to Section 6.9 if scour estimates dictate the need for a downstream cut-off wall.

Examples of the downstream run-out apron, and an alternative stilling basin, are shown in Figures 6-27, and 6-28, respectively.

FIGURE 6-27. Run-out apron, end sill and riprap transition (Fox Creek MPS No. 4, KY).

FIGURE 6-28. Stilling basin constructed of RCC (Lake Tholocco Dam, AL).

Downstream Apron Thickness

The downstream apron must be designed for uplift pressures that are more severe than the upstream apron because of the high differential water pressures that may exist at the spillway base. The designer needs to estimate the tailwater depth at the downstream end of the apron slab and the depth of flow at the upstream end of the apron slab for the range of spillway discharges to evaluate the uplift loading conditions on the downstream apron. It is important to note that the critical uplift loading condition often occurs at flows less than the maximum spillway discharge. A graphic representation of this loading condition is shown in the design example in Appendix A. For further discussion concerning this loading condition, refer to spillway and stilling basin guidelines in *Design of Small Dams* (USBR 1987a) and *Hydraulic Design of Spillways* (USACE 1990). Based on typical construction conditions, a thickness of 3 feet should be considered as a minimum thickness for most projects. Similar to the approach apron slab, bedding mortar should be placed between each lift of RCC in the downstream apron to reduce the potential for delamination during overtopping flow.

6.9 CUT-OFF WALLS

Cut-off walls are typically located at the upstream and downstream ends of the RCC spillway. Figure 6-29 shows typical variations of downstream cut-off walls. The function of the upstream cut-off wall is to lengthen the potential seepage path, decrease seepage under the spillway, and minimize the potential for erosion upstream of the spillway due to scour. The primary function of the downstream cut-off wall is to prevent undermining of the spillway from channel erosion and

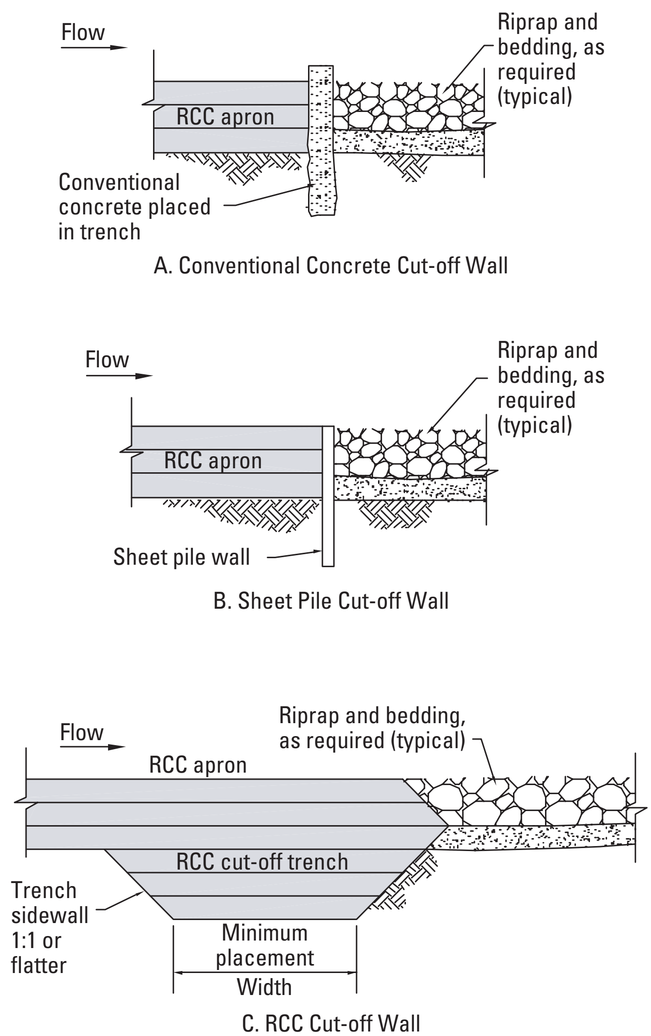

FIGURE 6-29. Cut-off wall details.

degradation. The depth of the downstream cut-off wall should extend to competent bedrock or to below the estimated depth of erosion that could occur from the spillway design flow. Refer to Section 6.8 for methods related to estimating scour depths downstream of an RCC spillway. Scour and/or channel degradation studies may be required to determine the depth of the cut-off wall, as well as post-scour stability analyses of the cut-off wall. Note that if the cut-off is not constructed directly on erosion-resistant bedrock or structurally attached to the apron, it should extend to a depth below the anticipated scour depth where it would be structurally stable after erosion has occurred downstream. Cut-off walls can be constructed of CVC, RCC, or steel sheet piling.

CVC Cut-off Walls

Cut-off walls can be designed as non-structural elements constructed by excavating a trench and backfilling the trench with CVC. A non-structural cut-off wall in a trench excavation can be designed with or without reinforcement. Cut-off walls can also be constructed as formed reinforced CVC walls. Formed wall construction requires a larger excavation than trenched wall construction because of the excavation required for the installation of the formwork. A formed wall design will require that the excavated slopes be laid back as required for trench safety and then backfilled and compacted to grade. Construction of typical upstream and downstream CVC walls are shown in Figures 6-30 and 6-31, respectively.

FIGURE 6-30. Upstream cutoff wall (Poe Valley Dam, PA).

FIGURE 6-31. Downstream cutoff wall (Poe Valley Dam, PA).

Sheet Pile Cut-off Walls

Sheet piling can be used to construct upstream or downstream cut-off walls (Refer to Figure 6-32). Some advantages of sheet piling are that trench excavation, dewatering, and placement of compacted fill in the trench are not required. However, use of sheet piling is usually suited to larger projects that can justify the mobilization expenses. Driven sheet piling also requires foundation conditions conducive to pile driving (limited cobbles, boulders, or interbedded hard layers). Sheet pile walls can be constructed by placing sheet pile in an excavated trench and then backfilling against the sheet piling. CVC and grout-enriched RCC has been used on some projects where compacting RCC with small equipment in and around the bellies of steel sheet piling has been difficult. Installing shear studs and anchors behind the sheet pile wall could also be considered to enhance structural connection with the RCC apron.

FIGURE 6-32. Backfilling steel sheet pile cutoff with conventional concrete (Lake Oneida Dam, PA).

RCC Cut-off Walls

Construction of RCC cut-off walls requires a larger trench excavation than CVC cut-off walls because of the minimum width requirements for placing and compacting the RCC (see Figure 6-29C.) In addition, the side slopes of the trench need to be excavated to a slope of 1:1 or flatter for worker safety. RCC cut-off walls may be preferred for projects where CVC would not otherwise be required.

RCC can also be placed over the entire crest of the dam and extended down the upstream face of the dam. This design serves as an upstream cut-off wall and minimizes the potential for contraction scour on the upstream face of the dam.

End Sills, Chute Blocks, and Impact Blocks

End sills, chute blocks, and impact blocks can be added to the downstream apron to improve the hydraulic performance of the energy dissipator and shorten the apron length; however, as mentioned in Section 6.8, construction of complex stilling basins are not typically used (Figure 6-33). The designer is cautioned that if a hydraulic jump-type energy dissipator

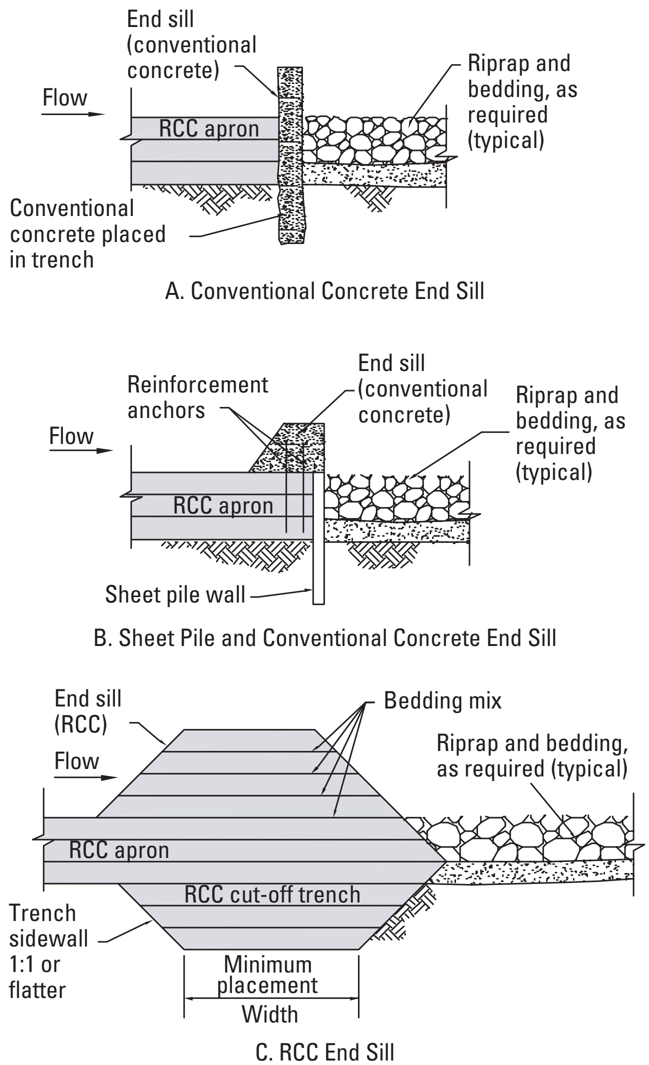

FIGURE 6-33. Alternative end sill details.

feature is used, adequate tailwater will be required for these features to function as designed. If chutes or impact blocks are used, "capping" the apron with a CVC slab to expedite construction of the blocks can also be considered. The end sill can easily be incorporated into the cutoff wall using CVC or RCC (Figure 6-33). Riprap is often placed downstream of the RCC apron to protect the downstream edge of the RCC and to transition to the stream channel.

6.10 JOINTS FOR RCC SPILLWAY SLAB

General

Three types of joints are discussed in this section: (1) joints between horizontal lifts of RCC, (2) construction joints, and (3) contraction joints.

Joints Between Horizontal Lifts of RCC

Joint surfaces naturally occur between successive horizontal lifts of RCC. The need to treat a joint depends upon the location of the joint and specific project requirements for joint bonding.

One approach (as described in Sections 6.7 and 6.8) is to require that bedding mortar be used between each lift of the approach apron slab and the downstream apron slab. Bond should also be developed between the sloped chute lift joints with proper curing and lift maintenance during construction. However, horizontal lift joint treatment in the spillway chute section is less standardized, and the extent of lift bond has not been thoroughly investigated in overtopping structures.

An important part of the design of the spillway chute is constructing a large monolithic mass. This serves the purpose of providing few paths for water to seep beneath the chute during spillway flows and a large mass to resist potential uplift forces. The performance of bonding between lifts for overtopping structures is largely empirical. Experience with the performance of horizontal joints from soil-cement slope protection projects has been good, and some research has been conducted on bonding of successive layers of RCC (Tayabji). Generally, delamination of RCC lifts in overtopping spillway applications has not occurred when proper preparation and care of horizontal lift joints is executed. As described in FEMA (2014), few RCC overtopping projects in the United States have experienced significant flows for relatively long durations. Based on limited experience, embankments with RCC overtopping protection have performed well during overtopping with no evidence of delaminated RCC lifts.

Chapter 9 describes procedures for joint treatment. An alternative/conservative approach would be to require bedding mortar concrete between each successive lift.

Factors favoring treatment of joint surfaces between successive lifts include:

- Bonded lifts provide a spillway slab that can act monolithically rather than laminated unbonded surfaces.
- Bond inhibits spillway flow from seeping through lift joints and under the slab.

Factors favoring not requiring treatment of joint surfaces between successive lift surfaces include:

- Monolithic action may not be structurally required, and RCC overtopping protection can be designed to resist uplift force based on its dead weight.
- Seepage through lifts can be designed to be safely handled by a properly constructed drainage blanket and drainage system beneath the RCC chute.

The decision to require bedding mortar on cold joint lift surfaces of RCC spillways is, at present, dependent upon project requirements and local engineering judgment. The minimum joint treatment recommended at this time would be:

- Placement of a bedding mortar on joint surfaces more than 24 hours old, and between each lift of the approach apron and downstream apron.

- Cleaning of the surface using compressed air prior to placement of a successive lift.
- Removal of contaminants, laitance, damaged RCC, or RCC that is not properly cured.
- Maintain the RCC surface in a moist condition prior to placement of the subsequent lift.
- Evaluate the need to provide a bedding mortar on joints that are more than 12 hours old.

Construction Joints

Construction joints for RCC spillways are typically located for the contractor's convenience for both planned and unexpected shutdowns in placement. The most common method to treat construction joints is to trim back on a 1 to 1 slope to fully compacted RCC, clean the joints to expose the coarse aggregate, and to place a bedding mortar on the joint surface prior to the placement of fresh RCC. Transverse construction joint locations should be documented, and joints should be staggered by at least 20 feet longitudinally when transverse joints are required in successive lifts. Methods for joint treatment are discussed in Chapter 9.

Contraction Joints

Contraction joints (and control joints) are placed in spillways to control the location of cracks caused by thermal contraction of the RCC. Contraction joints are intended to reduce random cracking, improve the appearance of the project, and reduce maintenance. Some RCC spillway projects have not been designed using contraction joints and have been allowed to crack freely (see Figure 6-34). Performance histories have not been compiled on the effectiveness of using contraction joints.

FIGURE 6-34. Naturally occurring shrinkage crack in RCC.

Spacing between contraction joints should be determined based upon the exposure conditions of the project and performance of other similar projects. In certain cases, a thermal analysis might be prudent to determine joint spacings. Where contraction joints have been constructed for RCC spillway projects, transverse (upstream/downstream) joints have been installed. Typical spacing of contraction joints has been from 100 to 300 feet. While the designer typically details the joint spacing, these locations may be adjusted in the field once the foundation is exposed. If on rock, the joints might be moved to where there are abrupt grade changes to account for likely stress concentrations.

Longitudinal (abutment to abutment) joints have not typically been installed and are generally not a good idea. Since RCC is not reinforced, longitudinal joints provide a mechanism for differential movement that could allow sections of RCC to "ride up" over a lower section. This is primarily of concern on the slope chute where movement of an upper section of RCC could ride up over the lower section, leading to erosion and structural and maintenance problems.

The objective of installing a contraction joint is to produce a linear joint that disbonds the RCC on either side of the joint, while not reducing the strength and density of the RCC near that joint. Geosynthetic membranes have been used under the joint to minimize infiltration of spillway flow through the joint, or geotextiles to control the potential for migration of fine particles through a joint to the foundation. Typical plan and section views of a contraction joint are shown in Figures 6-35 and 6-36. Methods used to construct contraction joints are discussed in Chapter 9.

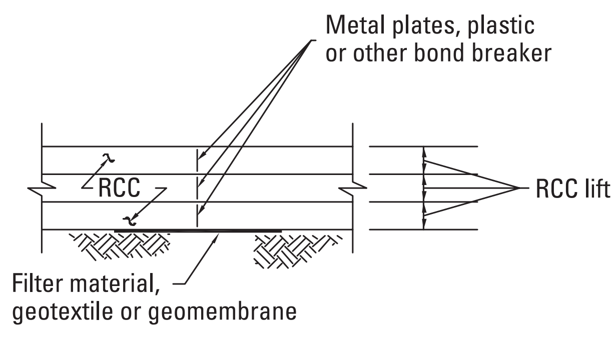

FIGURE 6-35. Typical section at contraction joint.

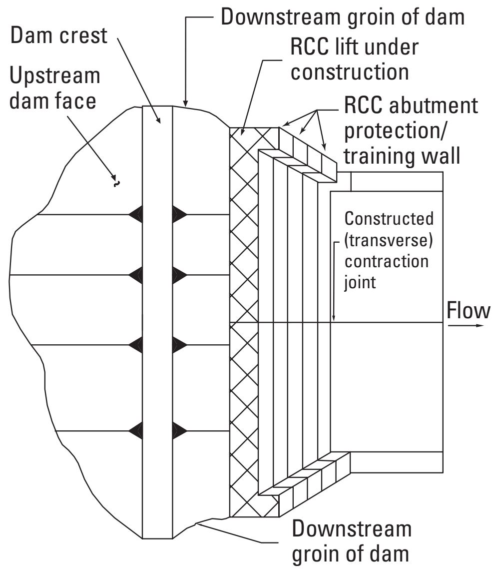

FIGURE 6-36. Plan of RCC overtopping and abutment protection partially constructed.

6.11 DRAIN OUTLETS

Drainage is typically installed beneath RCC chutes (see Figures 6-37 and 6-38) along with under-drainage piping as part of the spillway filter and drainage system. The purpose of the drainage system is to collect and filter embankment seepage, spillway under seepage, and to relieve high uplift pressures during overtopping flow events. Regarding embankment seepage, designers should carefully consider armoring existing embankments with a history of seepage issues, as the armoring will cover up the embankment and prevent the occurrence of potential seepage breakouts. If under-drains are included as part of the design, cleanouts and/or access pits should be provided for cleaning, inspecting, and maintaining the system. Drain outlets can range from pipes daylighting through the RCC steps (Figure 6-39) to substantial concrete channels (Figure 6-40). For narrow spillways, manholes and cleanouts can be located outside of the spillway walls. For wider spillways, spillway drain outlets can be provided through the RCC spillway chute. Pipe outlets should include animal guards.

FIGURE 6-37. Trench drain construction prior to RCC placement (McBride Dam, OH).

FIGURE 6-38. Blanket drain construction during RCC placement (Lake Oneida, PA).

FIGURE 6-39. Pipe outlet.

FIGURE 6-40. Concrete drain outlet structure (South Prong Dam, TX).

Spillway drain outlets and manholes must be designed to prevent spillway flow from entering the drainage system through the drain outlets. Improperly designed drain outlets or manholes can cause flow from the spillway to enter the drainage systems.

Check valves or flap gates should be installed as needed to prevent reverse flow, which could cause excessive uplift pressure on the RCC spillway slab.

Hydraulic model studies have been performed by the USBR to develop drain outlet details for creating negative pressure (aspiration) at the drain outlet. The negative pressure helps to prevent spillway flow from entering the drain and to encourage drainage of the filter/drainage system.

6.12 TRAINING WALLS AND ABUTMENT PROTECTION

General

Spillway training walls contain spillway flow within the RCC chute and protect the dam and abutments from potential erosion. Overtopping the spillway walls or the abutment protection can cause erosion of the dam embankment. This is of particular concern because high velocity concentrated flow can occur along the critical abutment areas of the dam where the walls are typically located. Examples of various types of training walls and abutment protection that have been used are shown in Figures 6-41, 6-42, 6-43, 6-44, and 6-45. Note, sloped stepped training walls should be avoided due to the risk of erosion at the transition to the embankment or abutment. Figure 6-42 shows sloped stepped training walls with deflection blocks that were added at the downstream end of each step to prevent erosion of the abutment.

FIGURE 6-41. Formed slope RCC training walls (Yellow River Dam No. 16, GA).

FIGURE 6-42. Formed sloped RCC training walls with deflection blocks to prevent erosion (Anthem Dam, NV).

FIGURE 6-43. Vertically formed RCC training walls (East Fork Above Lavon 3C, TX), courtesy of Dennis Clute.

FIGURE 6-44. Vertically formed RCC training walls (Red Rock Canyon Detention Dam, NV).

FIGURE 6-45. Conventional concrete training walls on completed RCC (Bear Creek Dam, VA).

Abutment Protection

Abutment protection is required for all overtopping designs. The abutment protection should be designed to safely contain the spillway flow within the embankment groins and transition to the stream channel. The abutment protection should be keyed into rock foundation when possible to prevent undermining the RCC slab if water overflows the abutment protection. Abutment protection should also be extended laterally to tie out at the design flood elevation. Designs which direct flow in a converging configuration (such as covering the entire downstream face as shown in Figure 6-1B), result in three-dimensional concentrated flow channels (which increases the velocity and flow concentration from top to bottom) at the abutment groins. Designers should be aware of complicated hydraulic conditions that could exist at the abutment groins and the erosion potential of the foundation. This erosion potential could lead to dam failure.

Abutment protection can be constructed by shaping the RCC to armor the abutments from erosion and to provide a "trough" to channel water from the downstream dam face to the natural channel below the dam.

The design of abutment groin protection warrants conservative design assumptions and can justify the use of physical or numerical hydraulic model studies to better understand the complex characteristics along the abutments and downstream of the spillway. The design of abutment groin protection for overtopping projects should be assessed by an engineer experienced with the design of spillways and the hydraulic phenomena that can be associated with overtopping flow and converging spillway.

Training Walls

Training walls can be constructed along the RCC chute to contain the spillway flow. Training walls are located on the downstream face of the dam and differ from abutment protection, which is located at the downstream embankment groins. The training walls can be designed with uniform chute width for the length of the spillway, parallel to the flow direction, or they can be designed with a converging chute transitioning the spillway crest to the base of the spillway. The geometries of both configurations are shown in Figure 6-46. An example of an overtopping section with CVC training walls is shown in Figures 6-47 and 6-48. Training walls constructed on the downstream face can mitigate the need for abutment groin protection.

The height of the flow training walls can be determined by the water surface profile for the design discharge. Determining the required height of the flow training wall should follow classic spillway design procedures. References for determining height are given in "Design of Small Dams" (USBR 1987a), "Hydraulic Design of Spillways" (USACE 1990), and "Estimated Splash and Training Wall Height Requirements for Stepped Chutes Applied to Embankment Dams" (Hunt and Kadavy, 2017). Research has shown that stepped training walls cause significant secodary flow that results in greater run-up height than is observed in a smooth wall condtion (Woolbright et al, 2008). The designer should be aware that RCC spillway surfaces are typically rougher than CVC chutes and bulking of flow due to greater air entrained in the flow must be considered in determining the depth of flow.

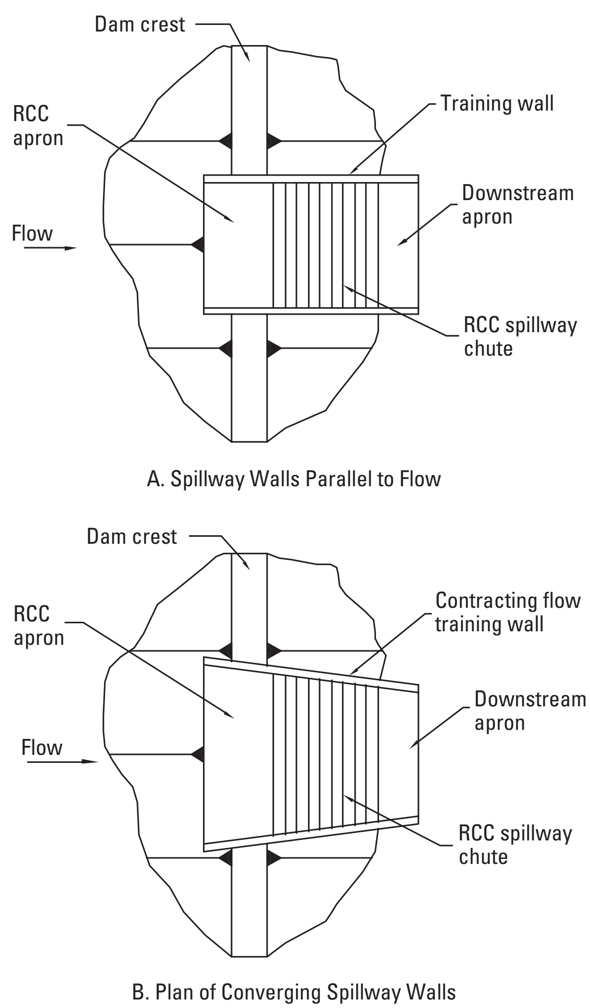

FIGURE 6-46. Spillway flow training walls.

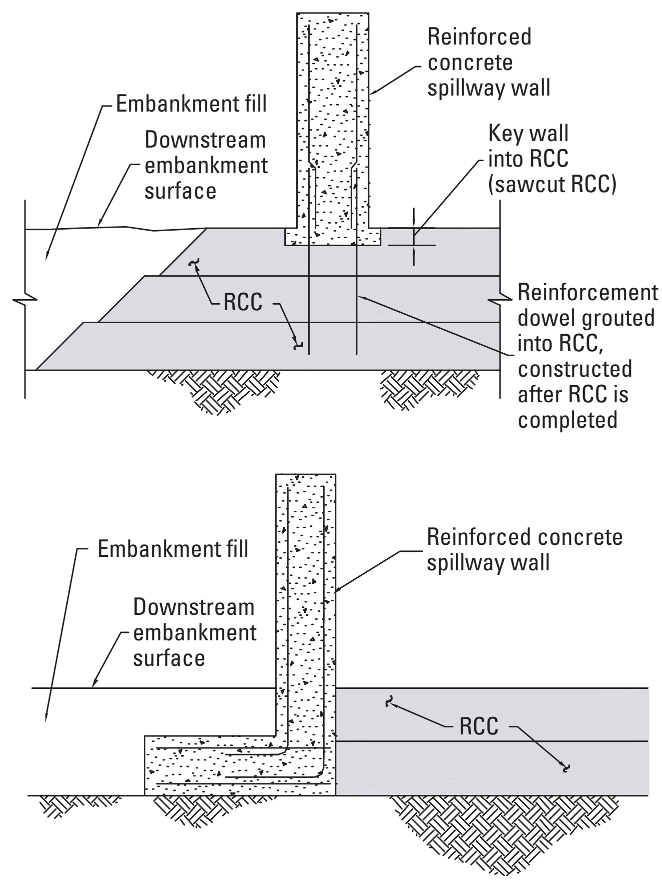

FIGURE 6-47. Reinforced concrete training walls (section looking downstream).

FIGURE 6-48. Conventional concrete training wall construction (Stoney Creek Dam, VA).

Determining the height of contracting spillway walls is more difficult to predict than straight walls. If the contraction angles of the walls are within the typical guidelines (USBR 1987a) to prevent cross waves on the chute, then standard design aids can be used to estimate wall height. To determine the maximum contraction angle that will not form cross waves, refer to the references described in the previous paragraph. Sharply contracting walls may require the use of a physical model to predict spillway performance and to determine the required wall height.

RCC flow training walls can be constructed by modifying the geometry of the RCC at each side of the spillway to contain the flow on the spillway surface. A benefit of CVC training walls is that they can be constructed after the RCC placement is completed. The construction does not complicate the lift geometry and will not interfere with the RCC placement operations. It is generally more economical to use CVC training walls if the spillway width is narrow.

6.13 SOIL COVER FOR RCC SPILLWAYS

Several RCC spillways have been covered with soil and grass (see Figures 6-49 and 6-50) when they are located in a park or residential setting where aesthetics is important. Covering RCC spillways with soil is usually considered for spillways that would operate infrequently, since operation of the spillway would cause the soil to wash downstream. This can create associated potential maintenance and environmental problems at the dam and in the downstream channel. The minimum thickness of soil cover is usually dependent upon the type of soil and the store and release requirements of the soil to support vegetation. Soil cover could also be provided as freeze-thaw protection of the RCC in climates subject to freeze-thaw conditions. The typical soil cover thickness has been about 2 feet.

FIGURE 6-49A. RCC spillway before topsoil cover is placed (Lake Laura Dam, VA).

FIGURE 6-49B. RCC spillway after topsoil cover is placed (Lake Laura Dam, VA).

FIGURE 6-50. RCC spillway with topsoil cover placement in progress (Sylvan Lake Dam, CO).

Several benefits that can be obtained by covering an RCC spillway with soil include:

- Covering the RCC with soil soon after placement aids in curing the RCC by keeping the surface moist and preventing surface drying caused by wind and thermal exposure.
- Soil cover helps maintain a uniform curing temperature for the RCC by limiting the daily thermal cycles of the RCC surface from solar radiation and nightly temperature drops.
- Covering the RCC with soil can bury the RCC below the frost level and limit potential freeze-thaw damage. Limiting freeze-thaw cycles can increase the useful life of the spillway and decrease long-term maintenance costs.
- Covering the RCC with soil and grass can also provide a more natural appearance to the finished construction.

Disadvantages of covering the RCC surface with soil include:

- The RCC surface is buried and not accessible for visual inspection.
- Operation of the spillway will likely cause erosion of the soil cover, which could result in maintenance costs and environmental problems downstream.
- Erosion in the soil without good grass cover may occur due to concentrated runoff from precipitation, developing erosion channels in the soil cover down to the RCC.
- Seepage outlet drains extending through the soil cover with large quantities of seepage can cause erosion of the soil cover. Alternative drain configurations should be considered.

The decision to cover the spillway should be based upon specific project requirements, including frequency of spillway use, aesthetics and operation and maintenance requirements. The Owner should be made aware of the advantages and disadvantages of soil cover so an informed decision can be made concerning the use of soil cover for an RCC spillway.

CHAPTER 7

RCC MIX DESIGN

7.1 GENERAL

The purpose of an RCC mix design is to develop project-specific properties to meet the structure design and performance requirements and to provide a basis for developing the project bidding documents. Perhaps the two most widely used properties in developing criteria for RCC mixes are the compressive strength of the RCC mixture and the workability (compatibility) of a mix. The compressive stresses in an RCC spillway structure are typically low, but durability requirements are high. Compressive strength is generally specified as an indirect indicator of the durability of the RCC mix, as there is currently no good measure for long-term durability of RCC mixes. For gravity dams, the tensile strength of the RCC and of lift joints is often a design requirement. For the hydraulic type structures covered in this manual, tensile strength is not a required design metric. The workability of a mix (such as slump in CVC) is also important in obtaining the desired in-place properties. A uniform measure of workability of RCC, which is referred to as a no-slump concrete, is the Vebe time which is obtained by the procedure detailed in ASTM C1170. A low Vebe time (10-15 seconds) is a good indicator of adequate paste to fill all the voids with little segregation of the coarse aggregate. For most hydraulic structures, a Vebe time between 10 and 30 seconds is typically specified. A third property that should be considered in developing RCC mixes is uniformity. Mix designs, and more importantly developing criteria for specifying mix designs, are discussed in this chapter.

RCC by its very nature has some characteristics of soil and some characteristics of concrete, and the nature of the RCC changes throughout the process. For example, an RCC mix is specified to provide the properties of CVC. Before curing, the RCC is placed and compacted like a granular soil with only frictional strength. After curing, the RCC is a cemented product that has both frictional and cohesive strength. This has a significant effect on all stages of design, construction, and quality control for RCC. Throughout this process, the long-established properties of soil mechanics and CVC technology intermingle. It is important that the user develop an understanding of both, so that the most appropriate discipline can be used to guide the application of RCC throughout the various stages of a project.

Perhaps one of the most diverse aspects of RCC is in the area of mix designs. Since the primary property that is used in RCC design is compressive strength, it is only natural that proportioning be performed using traditional CVC mix proportioning procedures. However, since RCC is spread and compacted with earthmoving equipment (bulldozers, surface compactors, etc.), CVC proportioning methods which were designed for material that is consolidated using internal immersion vibrators have not been correlated with the compaction equipment used in traditional earthwork. By contrast, fill control for earthwork placement is generally performed using a compaction test (such as ASTM D1557). A good correlation has been developed between compaction test properties and surface compactors.

The compaction test establishes the workability (compatibility) of the material over a range of moisture contents and is correlated to achieving the maximum dry density using surface vibratory compactors. However, granular fill placed to the maximum dry density at the optimum moisture content will contain more air voids than CVC and consequently can have a relatively higher permeability, and lower durability and compressive strength than CVC. The air voids content of a compacted granular fill can be reduced by placing the granular fill at a moisture content above optimum, during which air is further driven out of the mixture as compaction occurs.

The experience of practitioners from the geotechnical and CVC disciplines has given rise to two general approaches to mix designs for RCC. Mix designs can be performed using a modified Proctor compaction test (ASTM D1557) process or a CVC mix proportioning method. Both are suitable for developing RCC mix proportions but use different indicators for workability. With experience in RCC construction, a similar mixture can be developed by either method, and mixes have been found to be similar when the moisture content of the RCC is about 1% above optimum moisture content (ASTM D1557).

7.2 SOIL COMPACTION METHOD OF MIX DESIGN

In general, the ASTM D1557 test (modified Proctor compaction test) procedure is typically used to establish a moisture-density relationship of granular material. The standard test procedure involves compaction of the minus ¾-inch fraction of the fill material in a 6-inch diameter by 4.584-inch high steel mold. The material is placed in five lifts and compacted by 56 blows per lift, using a 10 pound hammer falling 18 inches, which results in a total compaction energy imparted to the sample of 56,250 foot-pounds per cubic foot. Samples are compacted at various moisture contents, and typically a range of placement moisture contents is specified based on the compaction characteristics. In granular fill such as crushed stone base, fill placement requirements are then specified. A typical specification for placement of crushed stone base would include the following requirements:

- Maximum loose lift thickness: 8 inches
- Placement moisture: optimum moisture plus or minus 2%
- Compaction: Minimum of 95% of the maximum dry density

Experience has shown that RCC compacted at optimum moisture, based on the modified Proctor compaction test, does not have adequate workability to yield a uniform, densely compacted material in a 12-inch thick compacted lift. This can primarily be attributed to the following factors:

- Lift thickness
- Compaction control based on achieving the maximum dry density

RCC is typically placed in 1-foot thick compacted (approximately 13- to 14-inch thick loose) lifts, which is significantly thicker than typical granular fill. In addition, the void content of RCC compacted at optimum moisture is higher than CVC due to entrapped air content. Historically, CVC has shown that with an air void content of 5% due to incomplete consolidation, loss of strength (as much as 30%) can occur. To compensate for the higher air voids content that can occur at optimum moisture content, the placement moisture content specified for RCC is usually about 0.5 to 1.5% above the optimum moisture content for the maximum dry density. In granular material, the wet density frequently continues to increase for a small range above the optimum moisture content. The higher placement moisture content increases the workability of the RCC mixture which allows placement and compaction of the thicker lift with a lower air voids content (more impermeable) and higher wet density. This is primarily due to the fact that air is being driven out during compaction of the more workable mix, which is desirable for increasing the impermeability of the mixture. A typical moisture density relationship of an RCC mix is shown in Figure 7-1. However, a balance in mixture water content must be achieved since a higher water content reduces strength (at a constant cement content), and roller efficiency can be reduced at very high water contents. The wet density of the mix should be used for RCC compaction quality control testing in contrast to soil testing which typically specifies a maximum dry density range as previously noted.

Consequently, experience has shown that RCC mix designs using the soil compaction method should be developed with a moisture content above the optimum moisture content (ASTM D1557). While there is no uniform amount that provides an agreeable workability, it is widely accepted that the moisture content should be about 0.5 to 1.5% above optimum.

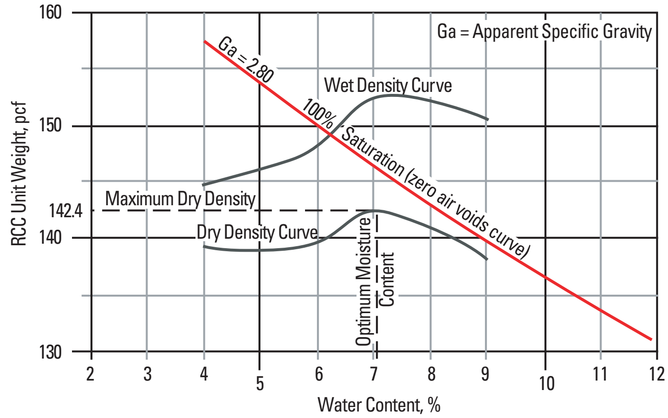

FIGURE 7-1. Typical RCC moisture-density relationship (ASTM D1557).

The primary design criteria for an RCC mixture is the compressive strength and placement moisture content that will allow uniform compaction (for the full lift thickness) to a high density (low air voids content), minimize aggregate segregation, and provide the required strength and durability. The following procedure for RCC mix designs has been used on multiple projects:

Step 1

Determine the properties for the RCC, including:

- Nominal maximum size of aggregate that can be used
- Expected exposure conditions
- Specified strength and test ages
- Workability
- Aggregate quality requirements
- Cement type and SCMs (if used) and properties
- Aggregate gradation

Step 2

Well-graded aggregates with a large nominal maximum size have less voids than smaller nominal maximum size aggregate and require less mortar per unit volume of RCC. The maximum size aggregate for RCC dams has generally been 3 inches or smaller. Considering the increased difficulty of controlling segregation with a large maximum size, combined with the narrow placement area for RCC overtopping spillways, a 1-inch maximum size is preferred for RCC hydraulic structures unless a lower Vebe, high-paste mix is being specified which would allow for a larger maximum size aggregate. Aggregate is selected that fits a design grading band (such as the examples shown in Figure 7-2) or using CVC mix design procedures for combining fine and coarse aggregate contents (as described in the following section). Aggregate gradations from several completed projects are shown in Table 7-1.

Step 3

Criteria for selecting water:cement ratios for RCC for various exposure conditions has not been developed at this time. RCC is relatively freeze-thaw resistant when it is not critically saturated, even in severe climates, but may be susceptible to freeze-thaw damage when critically saturated. The addition of an air-entraining admixture is a common method to improve the freeze-thaw durability of concrete, and RCC mixes with entrained air up to 6% have been used since the early 1990s. The keys to consistently producing air-entrained RCC are RCC mixes that have more paste than necessary to fill the voids and a moisture content that results in a Vebe time under 15 seconds. Mixing plants must provide mixing times to produce a uniform RCC mix. The current practice for design of RCC for overtopping protection spillways is to specify a minimum compressive strength of 3,000 psi and a maximum water to cementitious materials ratio (w/cm) of less than 0.9. The structures are also generally designed to be maintained in a dry condition (not critically saturated). If the RCC is to be exposed to water continuously or has the potential to become critically saturated, the designer should consider the use of traditional air-entraining admixtures or CVC in areas of severe exposure conditions. Some designs specify the use of a minimum compressive strength of 4,000 psi for severe service conditions.

An RCC mix for laboratory testing should be designed at a mid-range cement content. As an initial trial, a mid-range cement content would be about 1 pound of cement per yd^3 for each 7.5 psi of design compressive strength (i.e., for a design compressive strength of 3,000 psi at 28 days, 400 pounds per yd^3 of cement should be used). Type I or Type II ordinary portland cement (OPC) as defined in ASTM C150 are the most commonly used unless aggregate or site conditions dictate the need for a sulfate resistant or low alkali cements. Type IL blended cement as defined in ASTM C595 is an alternative to OPC.

All types of cement are acceptable for use in RCC. In the US, three separate standards may apply depending on the category of cement. For portland cement types, ASTM C150 describes:

Cement Type	Description
Type I	Normal
Type II	Moderate Sulfate Resistance
Type II (MH)	Moderate Heat of Hydration (and Moderate Sulfate Resistance)
Type III	High Early Strength
Type IV	Low Heat Hydration
Type V	High Sulfate Resistance

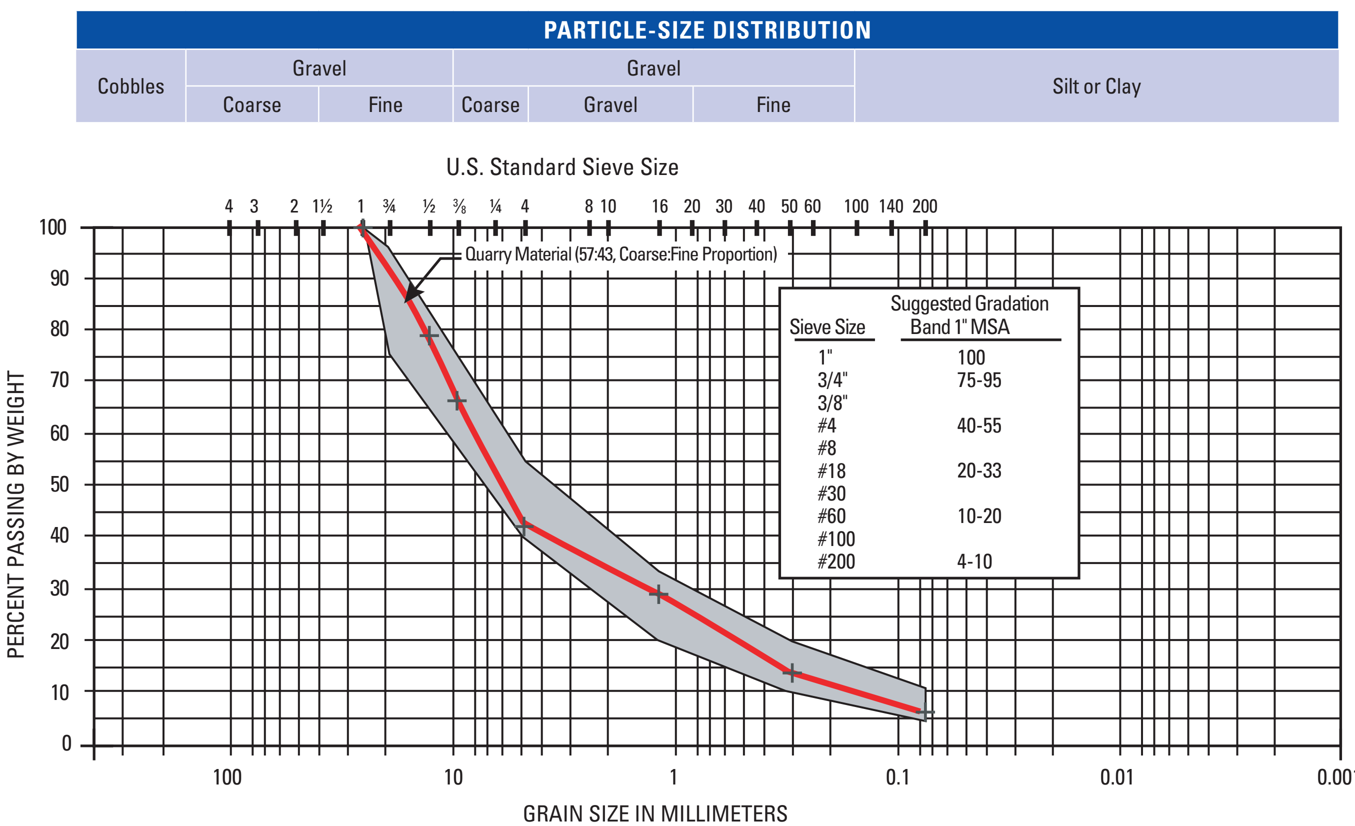

FIGURE 7-2. RCC aggregate design gradation bands.

TABLE 7-1. RCC Gradations.

SIEVE SIZE	FAWELL DAM, IL	MONA DAM, UT	SMITH LAKE DAM, VA	LEYDEN DAM, CO	LAKE THOLOCCO DAM, AL	SADDLE DAM, IN
1½″	100	100	100	100	99	80-100
1″	96	96			92	
¾″		84	70	75	84	70-90
½″	69				74	
⅜″		59	53	58	69	
#4	46	41	42	43	52	35-60
#8			35	28	38	26-50
#16	23	21	29	22	28	
#30			20	18		12-30
#50			11	13	18	
#100			9	9		
#200	7	4	7	6	10	5-10

For blended hydraulic cements – specified by ASTM C595 – the following nomenclature is used:

Cement Type	Description
Type IL	Portland-Limestone Cement
Type IS	Portland-Slag Cement
Type IP	Portland-Pozzolan Cement
Type IT	Ternary Blended Cement

With an interest in the industry for performance-based specifications, ASTM C1157 describes cements by their performance attributes:

Cement Type	Description
Type GU	General Use
Type HE	High Early-Strength
Type MS	Moderate Sulfate Resistance
Type HS	High Sulfate Resistance
Type MH	Moderate Heat of Hydration
Type LH	Low Heat of Hydration

Different types of portland cement are manufactured to meet various physical and chemical requirements for specific purposes (Wilson and Tennis, 2021), and it is recommended to check local availability and project compatibility.

Step 4

Develop the modified Proctor compaction curve (ASTM D1557) using the RCC mix at the mid-range cement content. If the aggregate contains material greater than ¾-inch, the ASTM D1557 procedures should be adjusted as described by Wong, Bischoff, and Johnson (1988) and Arnold (1992). Experience with RCC has shown that the oven dry water content of RCC samples from the compaction test can be erratic; therefore, the calculated water content should also be considered for constructing the compaction curve based on controlled tests of theoretical water content versus oven dry (ASTM D2216). One successful method of controlling the design water content of the compaction samples is to pre-measure material for each sample point, add the required water to each compaction sample at about 1% increments, and allow the samples to "season" for 24 hours in sealed containers without cement. The cement is then manually mixed into each sample immediately prior to compacting the sample. The moisture content can also be measured, for information purposes, using the entire compaction sample. Comparison of "hot plate" drying versus controlled theoretical moisture contents have also shown a more reasonable comparison with the theoretical moisture content than the conventional ASTM D2216 test.

A compaction curve is drawn of the dry unit weight versus calculated water content and wet unit weight versus calculated water content, as shown in Figure 7-1.

Step 5

A design water content of about 0.5% above optimum moisture content based on the maximum dry unit weight (or the water content at the maximum wet unit weight), is then selected for the RCC mix design.

Step 6

Prepare cylinders for compressive strength testing using ASTM C1435 and other testing deemed appropriate for the mid-range cement content for compression testing over the range of design age. Use a minimum of two, preferably three, cylinders for each age to be tested.

Step 7

Calculate mix proportions for RCC at the design water content from the compaction curve, or determine the unit weight of the RCC cylinders and calculate the entrapped air content or measure in accordance with ASTM C1849. The entrapped air content should range between 1 and 2%. If the entrapped air content is higher than about 3%, uniform compaction of the RCC full depth will likely be difficult. A higher air void content will result in a more pervious, lower durability, and potentially lower strength RCC. Adjust the water content, if needed, to provide a more workable RCC mix. Prepare a series of cylinders prepared in accordance with ASTM C1435 at different cement contents. Vary the cement content by increments of 10 to 15%, above and below the mid-range cement content.

Step 8

Prepare a plot of compressive strength versus age for each cement content (or water to cement ratio). The cement content to attain the design compressive strength can then be selected from the curves. When selecting a cement content, consideration should be given to the variability of the compressive strength of RCC, which is usually higher than CVC.

Step 9

Additional cylinders can also be prepared at different water contents to evaluate the effect of water content on the strength of each mix. The final determination of the water content will usually be decided in the field based on a test/control fill placement to evaluate the workability of the mix with the project specific materials and equipment.

Step 10

Mix proportions that can be used by the plant operator are prepared for the specifications. Mix proportions (by weight and volume) that result in a unit volume measured by absolute volume should be provided for each constituent. Mixing plants are set up in various methods to proportion the various constituents. These range from proportioning material based on: (1) the weight of each constituent, expressed as a percentage of the total dry weight of material (i.e., weight of dry aggregate plus cement plus fly ash); (2) the weight of each constituent in a saturated surface dry (SSD) condition (i.e., weight of aggregate in an SSD condition plus cement plus fly ash); and (3) the weight of each constituent in the dry condition and the total water content (i.e., weight of dry aggregate plus cement plus fly ash plus total water content [absorbed water plus excess water above the SSD condition]). It is critical that the engineer understands how the plant operates so that the design mix proportions can be correctly conveyed to the contractor. At a minimum, constituents to be provided are:

- Cement type, pozzolans (if used), content
- Aggregate content (SSD) (coarse and fine)
- Water content (above SSD)
- Total water content
- Specific gravity (SSD)
- Absorption
- Air content
- w/cm ratio

Step 11

Field adjustments will be required to account for the equipment type and environmental conditions for full scale production. Initial field adjustments are typically made in a test section separate from the structure. Test section construction should include evaluation of the in-place wet density throughout the lift thickness and the entrapped air content versus the mix design properties. Adjustments will also be required during production placement to account for variations in the aggregate moisture content and placement conditions.

Mix design calculations for an example project are shown in Appendix A.

7.3 CVC METHOD OF MIX DESIGN

CVC mix proportioning looks at achieving a gradation that can be densely consolidated with internal vibrators. Workability for CVC is usually measured by a slump test (ASTM C143). However, the slump test is not a suitable method for indicating workability (compatibility) of "no slump" RCC using surface compactors. A test procedure was previously developed for testing of no-slump concrete. The test for no-slump concrete uses a vibratory table to consolidate a sample with an external load placed on top. The test (ASTM C1170) is commonly referred to as the Vebe test.

Mixture proportioning using the CVC procedures for no-slump concrete and using the Vebe test for workability assessment is detailed in references such as USACE Engineering Manual EM 1110-2-2006 and American Concrete Institute (ACI) 211.1, *Selecting Proportions for Normal-Density and High Density-Concrete - Guide* (ACI 2022). Proportioning methods include both weight and absolute volume methods. The absolute volume method is more accurate and is the method summarized in the following.

RCC is designed with a consistency that is sufficiently stiff to support vibrating rollers. A key to the design of RCC mixtures using CVC proportioning procedures is providing sufficient paste to fill all the voids between aggregate particles and to allow consolidation under externally applied vibration. The following procedure for RCC mix designs has been used on several projects:

Step 1

Determine the properties for the RCC including:

- Nominal maximum size of aggregate that can be used
- Expected exposure conditions
- Specified strength and test ages
- Workability (Vebe time)
- Aggregate quality requirements
- Cement type, SCMs (if used), and properties
- Aggregate gradation

Step 2

Aggregate selection – There is a tendency for aggregate larger than 1½ inches to segregate when deposited in small areas such as the narrow "lanes" that are typical of overtopping spillway construction. Therefore, a maximum aggregate size of 1 inch is typically recommended for the RCC overtopping section design. Compare gradings of combined coarse and fine aggregate with other gradings such as Tables A-3 and A-4 in Appendix A. (USACE EM 1110-2-2006).

The maximum fines content will vary depending on the type of material. The use of crusher fines, rock flour, and non-plastic fines can serve as a mineral filler in an RCC mix. Fines with a plasticity index (PI) greater than 4 should only be considered after appropriate laboratory testing.

Step 3

Estimate the water requirements using Table A-2 in Appendix A (EM 1110- 2-2006), and select the required cement content from Figure A-8 in Appendix A (EM 1110-2-2006) for the design strength requirement.

Step 4

Calculate the absolute volume of cement and water, and assume an entrapped air content (typically 1 to 2%).

Step 5

Calculate the absolute volume of total aggregate by subtracting the absolute volume of each material from the unit volume.

Determine the sand content of the total volume of aggregate from Table A-2 in Appendix A.

Determine the absolute volume of the coarse aggregate by subtracting the volume of sand from the total volume of aggregate.

Step 6

Calculate the volume of paste and mortar and the ratio of the volume of paste to the volume of mortar from the absolute volumes computed above. The mortar volume includes the aggregate finer than the No. 4 sieve, cementitious material, water, and entrapped air. The paste volume includes the volume of aggregate finer than the No. 200 sieve, cementitious material, water, and entrapped and any entrained air (Refer to Table A-2 in Appendix A). The minimum volume of paste/volume of mortar ratio should be about 0.42 to ensure that all voids are filled. Adjust the fine aggregate content, if required, to increase or decrease the mortar volume. The quantity of cementitious material can be adjusted or the quantity of aggregate finer than the No. 200 sieve increased to change the paste-to-mortar ratio.

Step 7

Convert absolute volume to weight per unit volume for each constituent in the mixture using the specific gravity of the constituent.

Step 8

Measure out material weights and mix trial batch. Run Vebe test in accordance with ASTM C1170 to evaluate workability. Adjust water content to modify mixture workability to desired Vebe time. RCC mixes with a Vebe time of less than 20 seconds have a tendency to "pump" during compaction. This usually is due to a higher water content and paste content. At the higher water content, lower strengths will occur for the same cement content. Therefore, a higher cement content will be required to meet the design strength. For Vebe times in excess of 45 seconds, some mixes may be too dry for adequate compaction for the full depth of the RCC lift. This can result in higher air voids content with an increased permeability, decreased workability, and lower compressive strength RCC with potentially less durability. In general, a Vebe time between 20 and 30 seconds provides good workability, and the typical cement content to achieve this range provides acceptable strength and durability.

Step 9

Prepare cylinders for testing using the required cement content for compression testing over the range of design age. Prepare a series of cylinders using ASTM C1176 or C1435 (a minimum of two, preferably three, cylinders for each age are tested). Vary the cement content by increments of between 10 and 15% above and below the selected cement content.

Step 10

Determine the unit weight of the RCC cylinders and calculate the entrapped air content, or measure in accordance with ASTM C138 or C231, and compare with the theoretical air free (TAF) unit weight. Since RCC cannot be consolidated by rodding, the sample should be consolidated in the air meter container using the Vebe table (ASTM C1176) or by an electric hammer (ASTM C1435). The entrapped air content should range between 1 and 2%.

Step 11

Prepare a plot of compressive strength versus age for each water-cement ratio. The cement content to attain the design compressive strength can be interpolated from the curves. When selecting a cement content, consideration should be given to the variability of the compressive strength of RCC, which is usually higher than CVC.

Step 12

Additional cylinders can also be prepared at different water contents to evaluate the effect of water content on the strength of each mix. The final determination of the water content will usually be decided in the field based on a test fill placement to evaluate the workability of the mix with the project-specific materials and equipment.

Step 13

Mix proportions that can be used by the plant operator are prepared for the specifications. Mix proportions (by weight and volume) that result in a unit volume measured by absolute volume should be provided for each constituent. At a minimum, constituents to be provided are:

- Cement type, pozzolans (if used), and content
- Aggregate content (SSD) (coarse and fine)
- Water content (above SSD)
- Total water content
- Specific gravity (SSD)
- Absorption
- Air content
- w/cm ratio

Step 14

Field adjustments will be required to account for the equipment type and environmental conditions for full scale production. Initial field adjustments are typically made in a test section separate from the structure. Test section construction should include evaluation of the in-place wet density throughout the lift thickness, and the entrapped air content versus the mix design properties. Adjustments will be normally required during production placement.

Mix design calculations for an example project are shown in Appendix A.

7.4 GROUT-ENRICHED RCC

Grout-Enriched RCC (GERCC) is an RCC where grout is added to increase the paste content and makes the material act more like a low-slump concrete. This allows the material to be consolidated using immersion vibrators. In this design, traditional RCC is placed and then amended in-place with cementitious grout, giving it the workability of a low-slump concrete. Similar to a low slump concrete, GERCC can be consolidated using immersion vibrators, and when applied against formwork it provides a more aesthetically appealing exposed face than traditional RCC. GERCC has been used at the formed exposed surface of overtopping spillways, against rock abutments, and around CVC structures, but it requires extra labor for mixing and placement of the grout that adds complexity and cost to a project.

7.5 IMMERSION-VIBRATED RCC

Immersion-Vibrated RCC (IVRCC) is a high-paste mixture that is capable of being compacted using surface compaction equipment and immersion vibrators. This design came to North American use in 2020. FOSCE Consulting Engineers used IVRCC at De Hoop Dam in South Africa and later at the Enciso Dam in Spain and has been in the forefront of its development. These gravity dams are quite large and used several hundred thousand cubic yards of RCC. FOSCE found that the optimization of the RCC mix is highly dependent on the specifications for the fine aggregate. The quality of the fine aggregate as related to gradation and shape helps in reducing the water demand and thus allows for a lower w/cm ratio. Fine aggregate content as a percentage of total aggregate is typically around 35%, which is similar to CVC. Minimizing the percent of voids in the fine aggregate to under 0.3% is a typical target. The percent of fines passing the 200 sieve can be over 10% as long as they are non-plastic. IVRCC mixtures can be developed using the CVC method detailed in Section 7.2.

No hydraulic spillway-type structures have yet to use IVRCC. In the United States it was used in 2021 for a small gravity dam in South Carolina and for buttressing an existing concrete dam in Oklahoma. Because of the small quantity of RCC, the mix design used a graded crushed stone road base aggregate. The CVC mix design approach was used because of the high paste and moisture contents. In step three as mentioned previously, where the cementitious content is selected, with IVRCC, this content is increased due to the additional water demand needed to attain a Vebe time between 8 and 12 seconds and achieve the desired strength. It is recommended to increase the cementitious content by 10 to 15% as the initial starting point. Some of this additional cementitious content can be replaced by non-plastic 200 minus material. The mix design program should evaluate several different cementitious contents and moisture contents to arrive at the specified compressive strength and achieve a Vebe time less than 10 seconds. Entraining air in IVRCC mixes is possible due to the high paste and water contents. Figure 7-3 shows a typical fresh IVRCC surface and Figure 7-4 shows a laboratory test block of two lifts of IVRCC.

FIGURE 7-3. Typical fresh IVRCC surface.

FIGURE 7-4. Laboratory test block of two lifts of IVRCC.

CHAPTER 8

INSTRUMENTATION AND MONITORING

Proper instrumentation and monitoring of the performance of dams are important fundamental elements of a dam project's safety program. The selection of the instruments and their locations should be based on the potential failure modes of the project, at locations where design assumptions require verification, and in areas where long-term performance monitoring is required. Instrumentation is a means to evaluate risks associated with a failure mode and to assist in determining if a failure mode is developing that requires prompt attention. Following construction, dams and their ancillary facilities such as spillways should be monitored on a regular basis and immediately after any significant event such as an earthquake or large storm event.

The construction of RCC spillway overtopping projects essentially entails placing a concrete layer on the crest and downstream slope of an existing embankment dam. It is important to maintain the operation of any existing instrumentation in the embankment. Often, during the rehabilitation of an embankment dam, it is necessary to protect and/or modify the existing instrumentation systems to allow for the continuation of the monitoring program. Existing instrumentation systems such as piezometers, inclinometers, and borehole extensometers are often exposed on the downstream slope of the embankment. The designer must make provisions to protect, modify, or properly abandon and replace existing instrumentation systems.

Additional instrumentation and monitoring systems that may be required include:

- Blanket and/or toe drains monitoring. Drain outlets need to be readily accessible so flows can be measured and protected with animal guards.
- Survey monuments to monitor embankment and any wall movements. Areas to be considered are the overflow crest and training walls.
- Water level gauges or piezometers to monitor the internal phreatic level and reservoir level. In addition to the embankment, piezometers might be located beneath the spillway and overtopping protection.
- Stainless steel pins grouted into the RCC so that any loss in section can be measured. While this has not been used frequently, it should be considered when the RCC is unformed and the design includes a superficial RCC thickness.
- Crack/joint meters to measure openings and monitor trends. These should be considered if the monitoring program identifies cracks that exceed hairline width.

CHAPTER 9

CONSTRUCTION CONSIDERATIONS

Construction methods implemented for RCC hydraulic structures are a hybrid of earthfill and CVC construction techniques. The building of RCC hydraulic structures involves construction processes that are not typical for contractors that perform strictly earthwork or CVC construction techniques. The speed of earthfill placement can be realized in RCC construction, but the timing and the extra level of cleanliness of CVC construction is necessary to obtain an RCC product that meets design and construction requirements. RCC construction also involves significantly higher placement rates than typical CVC placement, as well as the transport method and compaction equipment typically used for earthworks projects. This chapter discusses construction issues and conditions that the designer should be aware of, as well as preparation of construction drawings and technical specifications for the successful completion of an RCC project.

9.1 CONSTRUCTION ACCESS/SITE LAYOUT

In design of an RCC hydraulic structure, site staging and access issues must be addressed. Site access, staging locations, and layout will greatly influence the contractor's ability to successfully complete construction of an RCC project. Some of the issues that should be considered in the planning and preparation of the construction documents are discussed below.

RCC Production Plant Location – The preferred location of the RCC production plant is adjacent to or as close as possible to the RCC placement area to minimize the time from mixing to placement. Often, because of a limited staging area or because of the generation of dust and noise in urban areas, an RCC production plant cannot be set up at the project site. In these instances, the production plant may be located some distance away from the site. The distance to a temporary plant site or an existing plant must be relatively close to the project site such that a sufficient quantity of material can be delivered, placed, and compacted within the required time constraints. Typically, the time allowed from the addition of water until final compaction of the RCC mix is in the range of 45 to 60 minutes unless a set retarder is used. The use of set retarders can extend the haul time, however the use of these admixtures and their impact on the design requirements of the RCC must be fully considered and evaluated before their use is approved. A travel time of more than 15 minutes for RCC delivery from off-site may be too long to allow adequate time for placement and compaction of the RCC. Traffic volumes in urban areas, especially during rush hour, can greatly impact the travel time to the placement area and may require adjustment in work hours to avoid traffic complications.

When the RCC plant is located on site, the owner must provide adequate space for the contractor to set up a plant and to deliver and store aggregates and cement (and any supplementary cementitious materials (SCMs) if used) at the site. This entails locating materials silos adjacent to the plant and providing sufficient room for transport trucks to maneuver and off-load the aggregates and cement (and SCMs).

On-site RCC production plants also require space to store a sufficient quantity of aggregate on site, in order to provide for a continuous supply of material for uninterrupted production of RCC. If sufficient room for aggregate storage is not available on site, the contractor will need to coordinate material deliveries with production demand and address varying traffic impacts in urban areas. The lack of on-site space for material storage will likely increase the construction cost of an RCC project because of delivery delays.

Test Section – In most cases, the test section is constructed in close proximity to the project, near the RCC production plant location if the plant is on site. If space does not allow for a test section to be constructed on site, the test section can possibly be built as part of the initial construction in the hydraulic structure in a non-critical area.

Waste Areas – Typically, an RCC hydraulic structure such as overtopping protection involves the removal of a portion of the existing embankment and the replacement of some embankment material with RCC. The designer must either incorporate the excess material on the project or identify a suitable on-site or off-site location to dispose of excess material. The use of off-site waste areas will obviously increase the overall project cost, and accommodating placement on site should be utilized where possible. On many overtopping protection projects, the waste soils are used to infill the existing earthen auxiliary spillway.

Diversion and Control of Surface Water – Generally, the contractor is responsible for the design of the water diversion system that is compatible with the contractor's sequencing and equipment. The designer's plans and specifications should provide the contractor with hydrologic data or reference data sources that the contractor could use to design the diversion for both upstream in-flows and tailwater conditions that could flood the construction area.

The designer should complete sufficient analysis of the impacts of flooding to the construction area, so that the design and construction schedule are flexible enough for a temporary diversion. Any milestone dates related to water storage and/or the coordination of required downstream releases should also be addressed by the designer. The selection of the storm event that will be used to design the temporary diversion works should receive careful consideration because in many cases the embankment is lowered to accept the RCC spillway exposing it to dangerous conditions if the diversion works are under-designed. Designers should consider developing a construction Emergency Action Plan to account for the temporary modifications to the dam. Design storm selection can vary significantly between projects depending on the criticality of the impacted area downstream of the construction site.

Construction Water – Sources and limitations on owner-provided construction water for the moisture conditioning, production of RCC, curing of in-place RCC, and control of dust on haul roads should be considered in the design phase so as not to become an unexpected restriction or a change order in the construction phase. Sources of water can include dewatering wells, local streams or reservoirs, and municipal supplies. The contractor can then identify the construction water source(s) and associated cost without discovering limitations and/or unidentified costs for owner-provided water that are sometimes encountered during construction.

Water used for the production and curing of RCC must be clean and free from injurious amounts of sediment, oil, acids, alkalis, salts, organic materials, or other substances that may be deleterious to the RCC and should meet the requirements of ASTM C94. Sediment contained in water used for curing can cause staining of RCC and affect the aesthetics of a completed project. Portable water treatment plants can be used to remove suspended solids.

Dams in Urban Areas – Constructing an RCC project in an urban area adds some challenges that are less of a concern at more remote sites. These issues can include the items below.

Limited Work Hours – Limitations are often required on the working hours on a project in an urban area. It is not uncommon for a city (or governing agency) to allow work only between 7 a.m. and 6 p.m. with no weekend or holiday work. Understanding limitations during the design stage is important, since planning for the cold joints in the design may be needed, and a suitable construction duration for the contract documents can be made. Also, because of a restricted schedule, the construction duration may be significantly increased, affecting both the cost and the length of time that the area will experience construction activities.

Safety – In urban areas, there can be a high volume of pedestrian traffic around a reservoir and dam. Construction typically attracts interested individuals. Measures must be taken to keep unauthorized people away from the project site during construction for their safety and the safety of the workers on-site.

Environmental Impacts – Many urban areas have more strict requirements related to the control of impacts to the local environment such as noise, light, air quality, road damage, traffic, and many similar issues. The specific environmental requirements of the urban areas where work is to be performed must be fully understood and planned for prior to construction.

9.2 DEWATERING AND FOUNDATION PREPARATION

The history of the site can significantly influence the groundwater conditions and, consequently, the extent of the dewatering system required for a project. Lakes and reservoirs generally increase the groundwater elevation near the dam. However, with flood control structures, the groundwater elevation can fluctuate more with water level changes in the stream or river, particularly in areas of gravelly sand and silt foundations. Flooding of low-lying construction areas can occur if surface water and groundwater fluctuations are not taken into account in the contractor's dewatering system. Seasonal fluctuations should also be expected.

When the structure foundation consists of soil, in particular sands and silts, it is necessary to lower the groundwater table to a depth such that a firm subgrade is obtained and the subgrade does not deteriorate under the actions of heavy construction equipment. In clay and weathered rock foundations, dewatering systems may be less extensive. On many projects, sumps, pumps, and ditches provide suitable groundwater control. A typical dewatering pump is shown in Figure 9-1.

FIGURE 9-1. Dewatering sump adjacent to downstream cut-off wall in stilling basin runout apron.

Prior to placement of the RCC and under-drain system on the foundation, soft and weathered materials are removed, all overhangs are chipped off, and sometimes a CVC mud slab is required and placed on the freshly excavated surface to prevent further subgrade deterioration during construction. Often, the first lift of RCC placed on soil cannot be compacted to the target compaction density due to yielding of the subgrade. The designer should account for this and either designate the first lift as a non-critical or "sacrificial" lift of RCC or incorporate a stabilized subgrade layer beneath the first RCC lift. Some examples of stabilizing the subgrade would be placing rock or gravel layers or stabilizing with a mud slab or "leveling slab" incorporated into the design.

On bedrock foundations, loose material is removed and the surfaces cleaned using commercial vacuum-type equipment such as a wet/dry vacuum truck and/or compressed air or compressed air combined with water as shown in Figure 9-2. Caution must be taken when using compressed air combined with water to not deteriorate partially weathered rock surfaces if the surface has been approved for placement. Bedding mortar is often applied to the rock surface to bond the RCC to the rock when seepage control is needed. RCC is compacted with hand operated compaction equipment if the geometry of the rock surface will not allow for the adequate compaction of the RCC with heavy vibratory rollers.

FIGURE 9-2. Foundation preparation and cleaning using compressed air with water.

9.3 RCC PRODUCTION

There are two main types of concrete plants used for the production of RCC: continuous mix plants and batch plants. Continuous mix plants (Figure 9-3) operate using calibrated belts and screws that proportion the RCC mix components continuously, based on the rate of production, with mixing in a pug mill. A batch plant weighs each component of a batch, with mixing in either a drum or compulsory mixer. Both types of plants are suitable for RCC production. Continuous mix plants have the capability of producing between approximately 150 to more than 600 tons per hour, depending on the plant. Batch plant capacities are typically lower, on the order of about 100 to 500 tons per hour. On most RCC overtopping projects, the plant capacity rarely governs the rate of RCC placement. Rather,

FIGURE 9-3. Continuous mix plant.

the placement rate is typically controlled by the capacity of the delivery system and the coordination of placement activities, such as RCC delivery, cleaning, joint preparation, cold joints forming, and placing methods.

On very small overtopping projects, RCC has been batched using a mobile plant and mixed in ready-mixed concrete transit trucks or in mobile volumetric concrete mixer trucks as shown in Figure 9-4. This method has a relatively low production capacity because to achieve thorough mixing of the RCC, the full capacity of transit mixers cannot be used. Controlling the segregation of the RCC during discharge can also be difficult.

FIGURE 9-4. Mobile volumetric concrete mixer trucks being used to produce RCC.

A critical element of production is the mixing time to produce uniformly mixed RCC. If mixing time is inadequate, the uniformity of the mix will be substandard with inconsistencies in the moisture and distribution of cement (and SCMs). What's more, the degree of segregation will vary throughout the batch resulting in loss of strength and durability. Most continuous-type plants have limitations on adjusting mixing times.

Sensitive environmental conditions or the proximity of the site to residences can prohibit the production of RCC on site. In these instances, off-site central mix plants have been used to supply RCC for overtopping projects. Established central mix plants have the advantage of an in-place quality control and quality assurance program, regular calibration, and a history of operation. However, there are some unique conditions that need to be considered. Conditions to be aware of include:

- Wear on paddles is greater than wear observed for batching CVC.
- Buildup of hardened RCC is more rapid and requires more labor to maintain than observed for CVC.
- The mixing time to obtain uniform RCC is typically greater than that for CVC using a drum mixer.
- With CVC, central mix plants provide initial mixing, and the remainder of the mixing is done in a transit mixer. Since RCC is a no-slump concrete and usually transported in a haul truck, all the mixing must be done in the central mix plant, resulting in longer mixing times.
- A ready mix producer may be resistant to dedicating a plant to an RCC project because of some of the non-standard issues or due to commitments to ongoing customers.
- Type II or Type I/II ordinary portland cement (OPC) as defined in ASTM C150 is typically specified for RCC hydraulic structure projects. Cements meeting the requirements of ASTM C150, C1157, or C595 may all be specified for RCC hydraulic structures type projects (see Section 7.2 regarding cement types). The availability and types of cement and SCMs (if specified) can vary regionally and can also vary over time as their production changes. Project costs can be affected if a separate silo is required to provide a cement or SCM that differs from the typical usage at an existing ready mix plant.

RCC Mix Temperature – The temperature of RCC for hydraulic structures is usually similar to CVC requirements. Unlike mass RCC gravity dam projects where temperature control is critical to mitigate cracking, hydraulic structures are typically relatively thin sections. The RCC cross section is typically 2 to 3 feet thick normal to the slope, where heat generation is not as critical an issue and can be quickly dissipated. However, in hot climates it may be necessary to: add chilled water to the mix to lower the mix temperatures; evaporative cooling of the coarse aggregate; shading of aggregate piles; or work at night to meet the typical temperature requirement for RCC. Typically, specifications limit the maximum temperature of the RCC during placement to 90 degrees Fahrenheit. If the hydraulic structure has thick components, the designer should consider temperature restrictions for the RCC in those areas.

9.4 RCC DELIVERY/TRANSPORT SYSTEMS

There are numerous methods to transport RCC from the production plant to the placement site. The main goal for RCC delivery is to provide a quality product free of segregation or contamination in a timely manner and by economical means. The designer needs to keep this goal in mind when design documents are prepared and provide specifications that do not unnecessarily restrict or dictate the contractor's means and methods of delivery. Unnecessarily restrictive or prescriptive specifications usually result in an inflated project cost with little or no project benefit.

Delivery systems that have been used to transport RCC can be placed into three general categories: (1) motorized haul vehicle systems, (2) conveyor systems, and (3) combinations of the two.

Motorized vehicle systems, including wheeled and track haul trucks (primarily end dumps), have been used successfully on many overtopping projects to efficiently and economically transport RCC from the production plant (both on site and off site) to the placement area as shown in Figures 9-5 and 9-6. They have a long history of satisfactory usage on many RCC projects. Some considerations for this transport method are:

- It is often necessary to construct temporary ramps to access the placement area.
- Cleaning and care of the lift surface is required to keep contaminants from the surface and to provide a suitable lift surface that is ready to receive a succeeding lift of RCC.

FIGURE 9-5. Haul truck being loaded with fresh RCC at mixing plant.

FIGURE 9-6. Haul truck delivering fresh RCC to the placement area.

- Cleaning of the vehicle tires or tracks and the occasional cleaning of old RCC from the bed of the vehicle is required.
- The driver cannot make sharp turns that would tear the RCC lift surface.

Care must be taken to limit segregation of the RCC mix when motorized vehicle systems are used. Modification of the truck bed is often required to reduce segregation. Typical modifications include welding of steel plates in the bed to eliminate square corners (significant segregation can occur during loading truck beds that are square and not beveled) and welding an extension to the end of the truck bed to limit the dump height to a maximum of 4 feet. Ejector type trucks where the RCC is mechanically pushed from the bed help reduce segregation. Spreader boxes can also be used to provide lateral confinement during spreading on the lift surface. Track hoe excavators have also been used as part of the RCC delivery process (Figure 9-7). Truck beds, spreader boxes, and skid boxes that are used to haul or temporarily hold RCC will usually experience buildup of hardened RCC. Equipment used for hauling, conveying, and spreading RCC will require periodic cleaning to keep the

FIGURE 9-7. RCC delivery with long stick hydraulic excavators.

hardened RCC from contaminating the placement area.

Conveyor Systems – Conveyor systems are often used when RCC is placed in steep valleys where access to the lift surface by trucks is limited. If the RCC is produced on site, the production plant can discharge the RCC directly onto the conveyor system or into a gob hopper, which will then discharge onto the conveyor system and then to the placement area. An example of one conveyor delivery system is shown in Figure 9-8. Long placement areas typically require long, multi-conveyor systems or frequent equipment moves. Conveyors can typically operate at slopes of 30 degrees, plus or minus a few degrees. Therefore, multiple conveyor segments with transfer points can be required for delivery over the full height of the structure. An alternate conveyor-type system that has been shown to be quite effective is a "super swinger" type system that uses a short conveyor delivery system and a mobile feed hopper (Figure 9-9).

FIGURE 9-8. RCC moved to placement area using conveyor delivery system.

FIGURE 9-9. "Super Swinger" conveyor delivery system.

Conveyor/Motorized Vehicle Systems – In many cases, the site geometry will dictate that a combination of a conveyor and wheeled/tracked vehicle delivery system be used for the transport of the RCC to the placement area. This is especially true when the structure is located in a narrow site. For this type of site configuration, the RCC will typically be conveyed to a central location near the structure and then be transported by haul vehicle or a super swinger-type equipment (Figure 9-9) to transport the RCC to the placement area. The use of conveyor systems tends to reduce segregation of the RCC.

Crane and Bucket – Though not used often, a crane with a large, modified bucket can be the best choice depending on site restrictions. The RCC is loaded into a truck from the plant, and taken to the crane location, and loaded into the large buckets (Figure 9-10). The RCC can then be placed directly on the lift surface (Figure 9-11).

FIGURE 9-10. RCC loaded into concrete bucket to be delivered to placement area via crane.

FIGURE 9-11. RCC placement using concrete bucket lifted by crane.

9.5 SPREADING OF RCC

Care must be taken when dumping, placing, and spreading RCC on the fill surface. To reduce segregation in the lift, the RCC should be dumped when possible on uncompacted rather than compacted RCC or placed in windrows by a rubberized conduit (elephant trunk) when using conveyors. By dumping and placing RCC on uncompacted RCC, the spreading equipment is able to provide some additional mixing of the RCC and reduce segregation that can result from hauling or transporting the RCC to the placement area. Also, the RCC should be spread in full lift thicknesses up to 12 inches and not multiple thin lifts (minimum thickness is 4 inches).

Pushing RCC over long distances can also cause segregation. Based on field observations, the allowable distance RCC can be pushed with spreading equipment without segregating is generally about 50 feet. The designer should be aware that this distance is a function of the design of the RCC mix. Drier mixes with larger, maximum aggregate sizes tend to segregate more than mixes with high moisture contents and smaller maximum aggregate sizes. A typical spreading and compaction sequence is shown in Figure 9-12.

When spreading RCC against forms, care must be taken to control segregation of coarse aggregate that can occur against the formwork. When segregation occurs in the RCC, rock pockets (areas of high voids) will result. Often thin lifts, 6-inches or less, and hand placement (Figure 9-13) are required to reduce segregation and the formation of rock pockets in formed RCC faces. Extra labor is required to address segregated RCC if the contractor does not take the necessary steps to minimize segregation. Use of a smaller, maximum-size aggregate (such as 1 inch) and/or a "wetter" higher paste (lower Vebe time) RCC mixture can also reduce the segregation potential.

FIGURE 9-12. Typical RCC spreading and compaction sequence at placement area.

FIGURE 9-13. Hand placement of RCC.

Equipment – Various types of equipment have been used to spread RCC for the construction of a lift, with the most common being a track-type bulldozer. An optimal size bulldozer for an RCC overtopping project is usually equivalent to a Caterpillar Model D4, John Deere Model 450, or Case Model 850 track-type dozer. Bulldozers larger than these sizes tend to be too large for the work area available for most hydraulic structures. The bulldozer blade can be modified to assist with minimizing segregation (Figure 9-14).

FIGURE 9-14. Modified bulldozer blade to minimize segregation of RCC.

Other types of machinery that have been used for the spreading of RCC include bulldozer-mounted spreader boxes and paving machines. Hydraulic excavators, backhoes, loaders, and skid-steers are also used with hand labor to spread RCC in tight areas not accessible to larger spreading equipment.

When using track-type bulldozers and track-mounted hydraulic excavators, it is preferable to use machines with street pads or worn cleats. When equipment with deep cleats is used, breakdown of the aggregate can occur. Cleats will also damage a lift surface when the equipment is driven on the surface of the compacted lift. Old conveyor belts or wooden planking have been used successfully to move equipment over compacted RCC surfaces to prevent or reduce damage to the lift surface.

Control of Lift Dimensions – The control of the lift thickness and geometry are important items that the designer of an RCC project should consider in preparing construction documents. The lift thickness can best be controlled by the use of laser leveling, whether hand operated or mounted to the bulldozer blade. The lift geometry is usually controlled with the formwork laid out by a licensed surveyor (see Figure 9.15), but if no formwork is being used, it can be controlled using a string line such as that used in common CVC and earthwork construction.

FIGURE 9-15. Formwork layout performed by licensed surveyor to control lift geometry.

9.6 COMPACTION OF RCC

Equipment – Several sizes of compactors should be specified for an RCC hydraulic structure project. Each size and type of equipment has advantages and disadvantages. One piece of equipment will not satisfy all the requirements on a project. Equipment to be considered should include a large drum vibratory roller for production compaction, a small drum vibratory roller for compaction in tight areas and adjacent to form work, and hand-operated compactors for areas that cannot be accessed with vibratory drum rollers. Application and limitations of various equipment types include:

- Large-diameter single smooth drum and double-drum vibratory compactors are ideal for production compaction of RCC. They are able to rapidly and efficiently compact large quantities of RCC to depths up to 15 inches in loose thickness. Limitations include:
  - Difficulty operating in tight areas where they are unable to maneuver
  - Cannot operate closer than about 2 feet from the face of forms because the weight and compaction force tends to cause deflections in the forms, causing alignment problems
  - Operation along the edge of unformed faces tends to cause “shear” failures in the lift surfaces near the edge
- Requirements for a large-diameter single or double-drum vibratory roller are:
  - Drum drives and transmits dynamic impact to the surface through a smooth, steel drum by revolving weights, eccentric shafts, or other equivalent methods
  - Minimum gross weight of 21,000 pounds
  - Average weight per unit width of drum of 150 pounds per linear inch and producing a minimum dynamic force of 450 pounds per linear inch of drum width
  - Adjustable frequency with a minimum frequency of 1,500 vibrations per minute
  - Amplitude of 0.03 to 0.07 inches
- Small dual-drum vibratory compactors are applicable for use in compacting RCC in tight areas, up against forms, and for smooth finishing of lift surfaces. Small rollers are generally not efficient for high production RCC placement. When preparing specifications, the minimum requirements for a small drum vibratory compactor should be as follows:
  - Drum drives and transmits dynamic impact to the surface through a smooth, steel drum by revolving weights, eccentric shafts, or other equivalent methods
  - Average weight per unit width of drum of 150 pounds per linear inch, producing a minimum dynamic force of 300 pounds per linear inch of drum width

- Hand-operated compactors can be effective in compacting RCC in areas that are not accessible to small drum vibratory rollers. A jumping jack-type compactor is typically used adjacent to forms and structures. The typical requirement for a hand-operated compactor is that it develops a minimum force per blow of 3,500 pounds per square foot. Hand-operated vibratory "plates" are effective for smoothing the RCC lift surface and for the compaction of unformed RCC steps (Figure 9-16). However, they are unable to effectively compact RCC deeper than about 4 inches for Vebe times greater than 25 seconds. The typical requirements for reversible vibratory plates are that they have a minimum gross weight of 500 pounds and a minimum compaction force of 4,000 pounds. The decision should be left to the contractor to determine the equipment to be used and to demonstrate its suitability during performance of the test section.
- Compaction with vibratory plates using an attachment to a hydraulic excavator has been used to obtain satisfactory compaction of the exposed downstream RCC face as shown in Figure 9-17.

Table 9-1 lists typical effective compaction depths in RCC for various types of compaction equipment.

FIGURE 9-16. Vibratory plate compactor used to compact unformed RCC steps.

FIGURE 9-17. Vibratory plate compactor attachment on hydraulic excavator used to compact exposed RCC face.

Time of Compaction – Often a designer will specify a time requirement by which RCC should be compacted, usually within 45 to 60 minutes of introducing water to the mix. Under most circumstances, this time for compaction is acceptable; however, there are times when it is not, and construction procedures in the field may need to be modified. During daylight hours in hot, windy, dry climates, the available time to achieve minimum compaction requirements can be effectively reduced, in some cases to as short as 30 minutes. In these circumstances, it is sometimes necessary to begin compaction immediately. Conversely, at night, when lower temperature and higher humidity conditions exist, RCC can often be effectively compacted up to 90 minutes after adding the mix water. It should be noted that each RCC mix is unique and the time to effectively compact the RCC will vary based on a variety of factors including the ambient air temperature and humidity conditions, the cement and SCM contents of the mix, the mix moisture content, the water-to-cement ratio of the mix and the initial set time. Set retarders can be used in the RCC mix to extend the time to compaction. The amount of the set retarder

TABLE 9-1. Typical effective depths of compaction.

EQUIPMENT TYPE	TYPICAL EQUIPMENT	EFFECTIVE DEPTH OF COMPACTION
Small vibratory plates	Wacker VPG - 165A Wacker BPS - 2550	2 to 3 inches
Rammer "jumping jack" type compactor	Wacker BS -92Y Bomag BPR 55/52D	6 to 12 inches
Large reversible vibrating plates (sleds)	Wacker BPU-4045 H	Up to 12 inches
Hand-guided vibratory drum compactors	Wacker W-74	6 to 8 inches
Small dual-drum vibratory compactors	Caterpillar CS-323C Ingersol Rand DD-24	6 to 12 inches
Large single-drum and double-drum vibratory rollers	Caterpillar CS-563 Caterpillar CB-634 C Bomag BW-213B Ingersol Rand DD-90HF	Up to 14 inches

Note: This list does not include all manufacturers that supply equipment suitable for RCC compaction.

admixture should be arrived at during the RCC mix design based on the desired time to initial set and the anticipated mix placement temperature.

When RCC Doesn't Achieve Compaction – With quality control of the aggregate, cement, SCMs, admixtures, and moisture contents of an RCC mix, it is rare that the desired target compaction density cannot be met, but this does occur occasionally during construction. Some of the causes for not meeting compaction requirements are:

- The time of effective compaction for the RCC has been exceeded
- Ineffective compactive effort was used
- A change in the physical properties of the mix has occurred, such as:
  - Aggregate moisture content in the stockpile has changed, and the mix is either too wet or too dry
  - Aggregate absorption and specific gravity has changed if the aggregate source has changed
  - Aggregate gradation has changed, either from a change in the material source or improper loading of segregated aggregates from the stockpiles
- A deficient quantity of cement or SCM in the RCC mix
- Segregation of the mix during transport and/or placement

The alternatives available when RCC does not make the desired compaction are to apply additional compaction efforts if the time restriction has not been exceeded, to remove it and replace with fresh RCC, or leave the lower density RCC in place. Depending on the degree of under-compaction (the likely cause of not meeting the compaction requirements), the extent of affected area, and the potential effects of RCC removal, the RCC may be removed or left in place. Because of the rapid nature of RCC construction, it is imperative that a timely decision be made to determine what needs to be done with the lower-density RCC. The design engineer or his representative must be available during these situations to provide sound recommendations and issue corrective instructions.

Appearance of Compacted RCC Surfaces – The appearance of the final compacted RCC surface is greatly influenced by several factors including grade control during the spreading of the RCC, uniformity of the RCC mix, moisture content of the RCC mix, and paste content of the mix. Grade control is an obvious influence on the uniformity of the lift surface. Other factors which may not be as apparent are discussed in the following text.

When it is desired to obtain a near smooth, uniform CVC-like finish on the RCC surface, it is usually necessary to have an RCC mix with smaller maximum size aggregate and higher moisture and paste contents. During compaction, the paste will tend to work to the lift surface. A problem with using a wet RCC mix is that the roller may have a tendency to sink into the RCC lift during compaction and leave the final compacted surface with a more undulating surface. To reduce the undulations in the finished surface, a drier mix can be used. However, there are drawbacks to using a drier mix. Dry mixes can be difficult to compact to a uniform density for the full depth of the lift, and dry mixes can have a tendency to segregate and develop rock pockets in the lift. Rock pockets are often considered a sign of poor construction since most people are used to seeing a smooth, uniform finish on concrete surfaces. Rock pockets can also serve as preferential pathways for seepage. It is critical for the design engineer to proportion an RCC mix that has the desired properties and for the RCC to appear and behave as intended. In many cases, the RCC mix will be modified for the final exposed surfaces. More cement/SCM may be added to increase the paste content and the moisture content adjusted so that compaction can be achieved without causing depressions in the lift surface. A typical good-quality RCC surface texture is shown in Figure 9-18.

FIGURE 9-18. Typical good-quality RCC surface texture.

The mixing time during the batching of RCC can be important in controlling the appearance of the final lift surface. If the RCC is not uniformly mixed, the moisture content could vary. For example, the first third of a batch is wet, and the remaining two-thirds of a batch are dry because of insufficient mixing. The compactors will have a tendency to sink in the wet area of the lift and bridge over the dry portions of the lift, leaving the lift surface with a widely varying surface and an undulating appearance.

9.7 CURING OF RCC AND EFFECTS OF CLIMATE

Like CVC, RCC must be properly cured and protected from adverse climatic conditions to ensure development of the durability and strength. American Concrete Institute (ACI) committee reports (ACI 207.5R, 306R, and 308.1) related to CVC curing procedures and protection from climatic conditions are also applicable to RCC hydraulic structure construction. One exception is that curing compound should not be applied to RCC surfaces on which successive lifts of RCC are to be placed. Curing compound serves as a de-bonding layer.

Curing of RCC – Generally, RCC surfaces must be kept continually moist for 7 to 14 days. A light mist of water should be applied to the compacted surface in a manner such that it does not erode the paste from the RCC. During construction, the RCC lift surface should be kept moist, but water should not be allowed to pond on the surface. Ponded water on the surface should be removed prior to the placing of the successive lifts of RCC. Applying water to uncompacted RCC must be avoided because the w/cm ratio of the RCC can be greatly increased and the strength reduced. Spray directed at the RCC surface will wash the surface paste from the mix and develop a "grit" layer of sand/cement that will no longer chemically react to bond the lifts together. In fact, the grit can form a de-bonding layer if it is not removed before the next RCC lift is placed.

Water for curing is typically applied to the exposed surfaces by using heavy-duty garden hoses with misting nozzles. Often, plastic sheeting used in conjunction with soaker hoses are used to promote the curing of RCC surfaces on which construction traffic will not be traversing and during periods of construction inactivity. Once the surface of the RCC has set up, wetted burlap may be placed on the lift surface to assist in curing in lieu of plastic sheeting.

It is important that clean water, free of sediments, be used for curing RCC. Concentrated runoff from curing water can also cause streaking on the exposed surface from calcium from the RCC or soil washed on to the surface. When sediment-laden water is used for curing, a thin film of fines can be deposited on the RCC lift surface, thus preventing or reducing the bonding between lifts. In addition, stained water can permanently discolor the exposed RCC surface.

Rain Events – During periods of light rain or mist, RCC construction activities can sometimes be continued. During periods of rain, the RCC placement should be observed closely for changes in the compaction characteristics, surface appearance, and roller action. Visual changes are a good indicator of when the rainfall is affecting the RCC placement and properties. During periods of moderate to heavy rains, RCC placement activities should be stopped. The performance of the compaction equipment on the RCC provides a good indication of whether RCC placement can proceed. When the rain intensity is high enough that the moisture content of the RCC surface is increased and/or the compacted RCC adheres to the surface of the smooth drum roller during compaction, RCC placement should be stopped.

When threatening weather develops, placement and compaction operations are usually "tightened up" with the compactor operating closely behind the spreading equipment. This is to limit the area that could be exposed to excessive rain that could require removal of RCC damaged by the rain. During rain events, the surface of the compacted RCC should be covered with plastic to prevent the erosion of the cement paste on the RCC surface. When the erosion of the cement paste occurs, a thin layer of uncemented fines can be left on the RCC surface, which can prevent the bonding of the successive lift of RCC and can also serve as a preferential sliding plane in the structure. When this condition occurs, the lift surface should be properly cleaned. The primary reasons that significant volumes of RCC have been removed during construction are: (1) rainfall on uncompacted RCC, (2) continued compaction of RCC during rain, and (3) the RCC surface has been allowed to dry out.

Cold-Weather Protection – The protection of RCC during cold weather is similar to the requirements for CVC construction, as described in ACI Committee Report 306, "Cold Weather Concreting." RCC must be protected from freezing for a minimum period of 7 days. The surface temperature of the RCC should not be allowed to drop below 35 degrees Fahrenheit. When cold weather can drop the RCC temperature, the RCC must be covered with plastic or insulated concrete blankets. In extreme weather conditions, heat must be required beneath the cover material to prevent the RCC from freezing. If the RCC does freeze, the design engineer must decide if any remedial measures are necessary prior to the placement of succeeding lifts of RCC. RCC can be placed for short durations when the ambient temperature is below freezing, but the RCC must be protected from freezing. This has been done by adding heated water to the mix and/or covering the lift surface immediately after compaction to protect it from freezing.

Upon completion of the RCC structure, an alternative to continual moist curing and/or to protect the exposed RCC surfaces from freeze-thaw effects for long periods is to place a layer of soil over the RCC surface to serve as insulation. This is not usually done for intermediate lift surfaces except for a winter shutdown due to cleanup and surface preparation required prior to the startup of RCC placement. Also, this method will likely leave staining on the RCC that will affect the aesthetics of the project unless the RCC will be completely covered with soil at the end of construction.

Hot, Arid Climates – Curing techniques for RCC in hot, arid climates are essentially the same as that for curing RCC under average climatic conditions, with the exception that greater care and effort is required to keep the exposed RCC surface continually moist. This may require the contractor to provide additional personnel and equipment. It is often beneficial for the designer to include provisions in the construction specifications, noting that additional resources will be needed when it is anticipated that hot, dry climatic conditions will exist during construction.

Flooding – A compacted RCC lift is essentially impervious to deleterious effects of water (with the exception of surface effects described above) and will not be damaged by submersion in floodwater. However, some repair of the lift surface may be required if erosive forces caused by the runoff occurs or by back cutting if the partially constructed RCC structure is undermined.

9.8 DOWNSTREAM RCC FACE

As described in Chapter 6, there are several types of downstream facing available for RCC projects, each of which has its own merits. The different types of downstream facing can be categorized as: (1) unformed, uncompacted RCC, (2) unformed, compacted RCC, (3) formed RCC, (4) precast CVC panels, (5) formed, grout-enriched RCC (GERCC), and (6) immersion-vibrated RCC (IVRCC). Construction considerations for these facing systems include the following:

Unformed, Uncompacted RCC Steps – This facing type is typically the easiest and most economical to construct, but the least aesthetic of the downstream face options. The RCC for this type of facing is compacted to the lift edge. The uncompacted RCC at the angle of repose (Figure 9-19) is left exposed. Constructability issues to be addressed for this design include:

- The minimum recommended lift width of a step is 9 feet. This allows for the operation of equipment and for overbuild of the lift because of the limitations of the construction methods. Hauling and compaction equipment range from about 6 to 8 feet wide.
- The RCC is more susceptible to freeze-thaw effects because the exposed face is uncompacted.
- Designer must include an ample amount of overbuilt particularly in very wet and freeze-thaw prone environments.

FIGURE 9-19. Unformed, uncompacted RCC steps.

Unformed, Compacted RCC Steps – This facing type has some of the same limitations as the unformed, uncompacted RCC steps. Construction procedures for this facing method are similar to those described previously, with the exception that the exposed RCC face is compacted (Figure 9-20) with a vibratory plate as one example. With this type of step, it is very important to control segregation of the RCC during spreading. Where segregation occurs, the contractor should remove rock pockets prior to the step face being compacted. This system is aesthetically more attractive and durable than the unformed, uncompacted step system, with some increase in cost.

FIGURE 9-20. Unformed, compacted RCC steps.

Formed RCC Steps – The construction procedures for this type of facing system involve the placing and compaction of RCC against a form. This gives a spillway chute a vertical stepped appearance. Design considerations for this type of facing include:

- A "wet," more workable RCC mix (such as GERCC or IVRCC) needs to be developed to minimize segregation and provide sufficient paste to fill voids of rock pockets that occur against the form.
- Large vibratory compactors should not operate closer than about 2 feet from the forms unless the forms are adequately braced. Large rollers operating adjacent to the forms tend to deflect the forms causing a misalignment in the steps.
- RCC should be compacted with small drum vibratory compactors and/or jumping jack-type tampers (thin lifts ± 6 inches are sometimes required) adjacent to formwork to reduce the deformation of the forms. Plate compactors should be operated at a slow pace to allow paste to move towards the formed face.
- For the best vertical surface finish, smooth metal forms should be used.
- The form system design must be rigid enough to allow for the adequate compaction of the RCC immediately adjacent to the forms without loss of compaction energy to deflect the forms (i.e., the compactor energy should be continued to consolidate the RCC, forcing the paste to fill voids without allowing the energy to transfer laterally to loose forms).

Examples of good, fair, and poor RCC faces placed against vertical forms are shown in Figures 9-21, 9-22, and 9-23, respectively.

FIGURE 9-21. Good RCC face.

FIGURE 9-22. Fair RCC face.

FIGURE 9-23. Poor RCC face.

Formed CVC Steps – Formed, CVC steps have been included in the design and construction of RCC gravity dams. Of the facing systems described previously, this system is the most durable and aesthetically pleasing, but it is also the most expensive. The construction of this facing system involves encasing the lifts of RCC in a shell of CVC. The most important detail the designer must address is the consolidation of the interface of the RCC and the facing CVC. Typically, the CVC should be placed first with the RCC placed against the CVC before the CVC reaches its initial set. An immersion vibrator should be used to consolidate the two mixes together.

Precast CVC Panels – The overtopping protection spillway at Tongue River Dam in Montana (Figure 9-24) used precast panels for the vertical step faces and CVC over the RCC for the step treads. The precast panels also served as the forms for the steps.

FIGURE 9-24. Precast CVC panels used for vertical step faces (Tongue River Dam, MT).

Formed, Grout-Enriched RCC (GERCC) Steps – GERCC is an RCC where grout is added to increase the paste content and make the material act more like a low-slump CVC, allowing the material to be consolidated using immersion vibrators. This concept for constructing facing for RCC was initially developed in China. Aesthetically, this construction method is similar to formed, CVC steps as shown in Figure 9-25. The overall material costs are lower, but labor effort and costs are therefore higher. One general procedure for formed grout-enriched RCC steps would be as follows:

1. A cement grout (cement and water) is placed on the previous lift surface out a distance of about 2 feet from the face of the form.
2. Loose, uncompacted RCC is placed up to the form.
3. The loose, uncompacted surface is perforated using a pitchfork or other surface puncturing tool.
4. The cement grout is then applied to the surface of the uncompacted RCC.
5. Immersion vibrators are then used to mobilize the grout and RCC into a uniform, fluid mixture. Gang vibrators working in tandem provide the best results.

FIGURE 9-25. Formed CVC steps.

Test trials are needed to refine the construction procedures and to determine the quantity of cement grout per linear foot of facing to add to the uncompacted RCC to produce the desired engineering properties. Typically, 0.5 to 1.0 gallons of grout per linear foot is used.

A couple of alternative "grout-enriched" placement methods that have shown some promise in test section trials are: (1) grout sprayed into the RCC mix as it is being spread adjacent to a form face, and (2) injection into the RCC after it has been spread and prior to compaction.

Immersion-Vibrated RCC (IVRCC) Steps – IVRCC is a relatively new RCC mix design philosophy that has many of the same benefits as RCC and CVC. The IVRCC mix can be consolidated by both external and internal methods. IVRCC was first used in the U.S. in 2020 to construct a small gravity dam and has also been used to construct a stability berm and auxiliary spillway chute for an existing CVC dam. Both projects demonstrated the successful application of IVRCC, and it could be a viable choice for hydraulic structures.

9.9 CONTROL JOINTS

The purpose of control joints is to control the location of the RCC cracks. Control joints or crack inducers have been constructed and installed using different materials and construction techniques. The types of materials used for control joints have generally included:

- Steel plates driven into or buried in the lift (as shown in Figure 9-26)
- Plastic sheeting or nonwoven geotextile buried into the lift as shown in Figure 9-27
- Saw cuts
- Monolithic construction with bond breakers

FIGURE 9-26. Steel plate inserted into fresh RCC to create control joint.

FIGURE 9-27. Plastic sheeting buried in RCC to create control joint.

The decision as to the type of material to use for construction of a control joint is generally based on the designer's preference. Steel sheets driven into the lift surface appear to be the most efficient method of installing a control joint from a construction production viewpoint. Saw cuts are not recommended because the crack formed in the RCC tends to be wider than the other methods which could increase the potential for fines to migrate through the RCC. If saw-cut joints are used, the design should include a filter fabric beneath the joint location and saw cutting must be of sufficient depth to control cracking at the design location.

Experience has shown that more than 50 percent of the lift thickness needs to be penetrated to form a crack. Therefore, the designer should consider placing the steel sheet metal plates as crack inducers in every lift using a plate height of between 8 and 10 inches. The joint material should be left 2 inches below the finished surface and from exposed RCC faces.

Construction control joints should line up vertically to ensure that a vertical crack forms where it is intended. Control of the crack inducer locations should be maintained both upstream and downstream of the placement area. Beneath any joint location, a strip of geomembrane should be placed on the subgrade material below the joint to impede loss of fines and water infiltration.

9.10 COLD JOINTS AND JOINT TREATMENT

Cold joints often result from delays in placement due to weather, plant breakdowns, and shutdowns due to weekends or holidays. Contractor plans for cold joints should be established prior to the start of construction. The age of the RCC lift and strength development of the RCC mix will determine the effort required to adequately treat a cold joint. Cold-joint treatment typically requires that the RCC lift surface be prepared to expose the fine aggregate without undercutting the coarse aggregate, prior to the placement of the successive lift of RCC. Joint treatment can include placing a thin layer of a bedding mortar (consisting of sand, cement, and water) or grout on the lift surface just prior to the placement of the RCC to aid in bonding the old RCC lift with the new.

Preparation of a cold joint can begin once the RCC has reached its initial set. The following types of equipment have been successfully used for the preparation of cold joints:

- High-pressure water blasters
- Compressed air/water jet
- Compressed air

The type of equipment to prepare a cold joint will depend on the strength of the RCC at the time of treatment. Relatively fresh RCC can be eroded if high pressures are used to treat the joint. Fresh RCC is typically cleaned only with compressed air. Additionally, if too much of the fine aggregate matrix is removed from the lift, the coarse aggregates can loosen and require additional treatment. The full range of methods should be available to treat the cold joint from relatively "fresh" to full strength condition, for every project. In some full-strength, cold-joint treatment conditions a skid-steer mounted grinding tooth can be used to roughen the surface prior to cleaning with compressed air/water to establish a positive bonding surface for the next lift. Bedding mortar should be added to all lift surfaces that have reached final set prior to installing the next lift of RCC. Examples of cold joint preparation and a well-prepared joint surface are shown in Figures 9-28 and 9-29.

FIGURE 9-28. Preparation of cold joint.

FIGURE 9-29. Roughened surface prepared using skid-steer mounted grinding tool.

9.11 BEDDING MORTAR

Bedding mortar or bedding concrete is often used in the construction of overtopping structures. Its function is to either:

- Bond lifts of RCC to RCC
- Bond the RCC to rock
- Bond the RCC to previously placed CVC structures (spillway walls, outlet works conduits, etc.)

Typically, bedding mortar is a mixture of sand, cement, water, and set retarder. Occasionally, SCMs and air entrainment admixtures are added to the mix to improve its workability and reduce segregation. Bedding mortar is spread in thin lifts of about ¼ to ½ inch in thickness. The maximum aggregate size is typically about ⅜ inch. The bedding mortar typically has a 28-day compressive strength of 2,000 to 3,000 psi and at a minimum, equals the compressive strength of the RCC. The bedding mortar is designed with a slump of about 8 to 10 inches. Bedding mortar placement is shown in Figure 9-30. When small quantities of bedding mortar are required, contractors often will utilize small ¼-yd^3 mixers.

FIGURE 9-30. Bedding mortar placement.

9.12 BEDDING GROUT

Cementitious grouts have been used successfully between RCC lifts. Grouts with Marsh times around 20 seconds work well between lifts. Some projects have used a method of producing a grout in-place. Dry cement is broadcast on the RCC lift surface and then sprayed with water to create a paste. This method is less precise and should be used with caution. If the RCC mix uses large aggregates and is subject to segregating, a thin grout layer may not be sufficient to bond the lift together. A thicker mortar might be more appropriate.

9.13 LIFT TREATMENT

RCC construction is comprised of successive horizontal lift surfaces. For each successive lift to bond to the previously placed lift, the surface of that lift needs to be clean and free of loose uncompacted RCC, laitance, contaminants, dust, and water. The designer should specify that each lift be cleaned with compressed air, at a minimum, prior to the placement of RCC for the next lift. Techniques used in the past include the use of blow pipes (with and without water) attached to air compressors, vacuum trucks, and hand tools including push brooms and shovels.

At a minimum, joint treatment with a bedding mortar or grout should be implemented: (1) for lift surfaces more than 24 hours old, and (2) between the top three lifts of a spillway, overtopping section, and in the stilling basin. Joint treatment for lift surfaces less than 24 hours old is still an area of designer preference, without an accepted standard at this time.

9.14 CONSTRUCTION JOINTS AT WORK STOPPAGES

It is rare that an RCC project is constructed continuously without interruptions in the placement schedule. Therefore, the designer must specify the type of construction joint treatment that will be required at work stoppages.

Typically, the designer will require the contractor to provide controlled construction joints at work stoppages as follows:

- Transverse joints at work stoppages are trimmed through compacted RCC to form a straight, beveled joint at an inclination of not more than 1 horizontal to1 vertical. The exposed surface of the joint is to be compacted after trimming.
- When lanes or areas of RCC are placed in adjacent areas, the longitudinal joint is typically required to be trimmed and compacted.
- Transverse joints of adjacent lifts are typically offset by a distance of 15 to 20 feet to prevent the establishment of unplanned joints with no geomembrane underneath, resulting in possible preferential seepage paths in the structure.

9.15 CONSTRUCTION OF TRANSITION AREAS

RCC hydraulic spillway projects typically consist of the modification of existing embankment structures. Within these structures there are zones where the new RCC material will transition with existing components, including the earth embankment and abutments, rock abutment or foundations, outlet works conduit, spillway training walls, and other miscellaneous CVC structures (see Figure 9-31). Some construction considerations the designer should consider in planning are described in the following text.

FIGURE 9-31. Placement of RCC at transition area.

RCC to Embankment Transitions – These transitions are best handled using two very different but effective construction techniques:

- Sculpting RCC at the interface zone, or
- Constructing a discrete interface zone with a CVC training wall.

When RCC is "sculpted" at the embankment interface, the designer must consider the methods used to place and compact the RCC, including bulldozers and vibratory rollers. Various types of production equipment have difficulty operating in tight areas because of their turning radii, and damage to the already compacted RCC can occur. Production in this zone is typically the slowest on the project. RCC is difficult to place in turning (curving) and tapering lifts and is slow. As part of the design, the interface between the embankment and the RCC needs to be protected from erosion, including sheet runoff and erosion during the flood event, including head-cutting and back-cutting. This is often done by either constructing RCC wing walls or dikes or placing riprap or similar slope protection.

The interface between the RCC and the embankment and earth abutments can also be constructed using CVC walls. These tend to be the easiest and quickest to construct, but their cost-effectiveness must be evaluated. RCC can easily be placed and compacted against CVC walls. CVC training walls have also been constructed on the completed RCC surface.

RCC to Rock Transition Areas – When RCC is placed against rock abutments or foundation contacts, the main consideration the designer must address is if a watertight bond needs to be developed between the RCC and rock. If not, RCC can be placed directly against the cleaned rock surface. Prior to RCC placement, all loose rock is typically barred off, and the surface to receive the RCC is blown off with compressed air blowpipes or cleaned with a vacuum device. If the interface is to be "watertight," such as at or near the crest or abutment interface, a greater degree of preparation should be performed. Typically, all loose rock is removed using pry bars and hand labor, and air and water is used to clean the rock surface of dust and fines. All residual water is removed from the undulations in the rock, and a layer of bedding mortar a minimum of ½-inch thick is applied to the rock. RCC is then placed and compacted up against the bedding/rock.

Transition between RCC and Existing Structures – At the interface between existing walls and conduits, the designer must evaluate if a bond between the existing structure and the RCC is required. If a bond is required, the existing structure should be sandblasted, cleaned with water, and a bedding mortar or GERCC placed at the RCC and structure interface. If a bond is not required, the designer can allow the RCC to be placed against the structure once all dirt and contaminants have been removed from the surface. The CVC surface should be at a saturated surface dry (SSD) condition prior to placement of the RCC.

9.16 RCC CONSTRUCTION IN CONFINED AREAS

When starting RCC placement, the first lift is often too small for standard production equipment to operate on. RCC can be placed and compacted in tight areas using small-scale compaction equipment, small backhoes, and hand-operated equipment. In general, this type of construction can be slow and not practical. An alternative for working in a tight area off the foundation is to place a CVC starter slab at an elevation to which production equipment can more readily operate. RCC production can then start once the slab has gained adequate strength on which to operate equipment.

When the first lift of RCC is placed on a soil foundation, it can be difficult to obtain the specified compaction as the result of the subgrade yielding. The designer can account for this condition in the specifications by:

- Assuming that the first RCC lift is a somewhat sacrificial lift meeting a lower density than specified for the successive lifts.
- A mud slab or starter slab of CVC can be built to begin RCC placement.

Both of these methods have been used successfully.

9.17 REPAIR OF RCC

Because of the general nature of RCC, when first produced, it behaves as a crushed stone base course material and over time takes on the properties of hardened concrete. The method and level of effort to repair RCC is therefore dependent on its age. If substandard RCC is to be removed, the decision should be made in a timely manner (as described in Section 9.6). The effort to remove fresh RCC is considerably less than the effort to remove RCC that has hardened and set with time. Judgment should be used in the determination of the necessity to remove compacted RCC. The engineer must consider that, at times, it may be desirable to leave slightly substandard RCC in-place and undisturbed, rather than damage the surrounding "good" RCC, if the area considered for removal is of limited extent or in a non-critical section of the structure. An inadequate patching effort can be worse than doing nothing (Figure 9-32).

FIGURE 9-32. Results of inadequate effort to patch substandard RCC surface.

Repair of Fresh RCC – The repair of fresh RCC can be necessitated for some of the following reasons:

- Too much or too little water
- Inability to obtain the target compaction
- Bearing failure of the lift edge caused by the vibratory roller operating too close to the edge of the lift during compaction
- The presence of pockets of segregated coarse aggregates in the lift of vertical face of steps
- Lack of the proper quantity of cementitious material, resulting in low strength

When it is necessary to remove and repair fresh RCC, considerable effort can be required to do it properly, depending on the quantity required to be removed. If it is required to replace a large volume of RCC, it is usually favorable to remove the bulk of the material with heavy equipment and then remove the remainder of the material by hand. If the removal of a small volume of RCC is required, it is usually preferable to remove the material with a jackhammer, pneumatic spade, or other hand tool. The following are steps that are typically done for the repair of fresh RCC:

1. Delineate the area of substandard RCC for removal and use marking paint to outline this area.

2. If the area of removal is large, use a piece of heavy equipment such as a loader or backhoe to remove the bulk of the substandard RCC. If the area is small, use hand tools, a jackhammer or power spade to remove the bulk of the RCC.
3. Trim the edges of the RCC adjacent to the removed RCC at a 90-degree angle such that the new RCC can be effectively compacted against the old RCC.
4. Remove any loose and uncompacted RCC from the trimmed edge and the lift surface with a vacuum, brooms, shovels, and/or compressed air.
5. Place and compact new RCC in the prepared area. The type of compaction equipment to use will depend on the available space, but it is preferable to use the largest piece available that can operate in the area. If it is necessary to have a bond between the old and the new RCC, the engineer will need to require the contractor to place bedding mortar against the trimmed edge just prior to the placement of the new RCC.

Repair of Old RCC – The repair of old RCC is necessitated by the same reasons as the repair of fresh RCC, except that the RCC has been allowed to set. Considerable effort will be required by the contractor to remove RCC that has been allowed to set. The procedure for the repair of old RCC will be the same as that for the repair of fresh RCC, with the exception of the following:

1. The RCC may have sufficient strength, such that a loader or a backhoe is unable to remove the RCC. In this case, it will be necessary to chip out the hardened RCC with a jackhammer.
2. The prepared surface to receive RCC should be washed with water to remove any dust and fines that will prevent the fresh RCC from bonding to the old RCC.
3. A bedding mortar should be used to provide better bonding between the prepared RCC surface and the new RCC lift.

Repair of RCC with CVC – Occasionally during construction the contractor will request to be allowed to repair RCC in tight areas by replacing it with CVC. This substitution is generally adequate, but the following must be considered when allowing this substitution:

- The compressive strength of the CVC must be equal to or greater than that of the RCC.
- Consideration must be given to the aesthetics of the substitution. A change in appearance between the RCC and CVC may occur if there is a significant difference in the cement content and the aggregate sources of the two materials.

The procedures for repairing RCC with CVC should be the same as those for repairing fresh RCC or old RCC, as stated previously.

Treatment of Rock Pockets – Occasionally during the placement of RCC, rock pockets will occur on the lift surface and on the exposed face of formed and unformed compacted faces as a result of segregation of the RCC mix. The following should be considered for these areas:

Rock Pockets on Lift Surfaces – Occasionally during placements, rock pockets will be observed on the surface of an RCC lift. Treatment of this condition can be handled in two ways. The preferred way is for the contractor to remove the segregated aggregate from the lift using hand tools prior to the compaction of the lift. Alternatively, if the lift has been compacted, the contractor should remove the zone of segregated aggregate immediately after compaction, to a minimum depth of 4 inches, and the segregated material should be replaced with fresh compacted RCC.

Rock Pockets in Compacted Unformed Faces – When rock pockets are discovered in compacted, unformed faces, the options for repair are as follows:

1. If the RCC is fresh, the rock pocket can be removed and replaced with new, unsegregated RCC. Because the area will be small, hand compaction of the RCC will be required. The contractor must take care and have patience in placing and compacting the RCC in thin 3- to 4-inch lifts because of the limitations of the compaction methods.
2. If the RCC has set, the contractor may chip out the segregated material and place a patch (sand and mortar or a polymer) over the area. The designer must be aware, though, that a patch will likely appear aesthetically unappealing, not matching the color or shading of the surrounding RCC. Also, there is a potential that the patch, if not properly placed, will pop off as a result of weathering and freeze-thaw effects. The placing of patching material over the surface of segregated aggregate (without its removal) is discouraged because of the likelihood that the patch will not adhere to the repair area due to weathering and freeze-thaw effects.
3. Leave the rock pocket in place as it is. The prevention of rock pockets is typically not critical to the structural integrity of the project, rather the existence of a rock pocket is typically an aesthetic issue.

Rock Pockets in Formed Faces – The repair options for rock pockets in formed faces are limited to patching and leaving the rock pocket in place (refer to the discussions previously). Because of the age of the RCC, when the formwork is typically stripped, it is usually not possible to remove the segregated RCC and replace it with fresh unsegregated material. Rather, the contractor must take extra precautions against the formation of rock pockets against the forms.

CHAPTER 10
PERFORMANCE

RCC has been used in hydraulic structures in the United States for over 40 years. The RCC overtopping protection built in 1980 at the Ocoee Dam in Tennessee has experienced flow several thousand times, including a major flood in 1990 that overtopped the structure by approximately 12 feet and is still in service and operating efficiently. Except for the few projects where a portion of the overtopping protection serves as a principal spillway or the RCC spillway serves as both the principal and auxiliary spillway, it is estimated that less than 15% of all the projects have seen flow over the RCC. No known cases of flow depths reaching the designed maximum depths have been documented for RCC overtopping structures, which is also the case for most auxiliary spillways whether vegetated earth, CVC, or other revetment systems. The U.S Army Corps of Engineers owns many dams with CVC and RCC spillways that have never operated.

FIGURE 10-1. Loose aggregate after 10 years in service (Bishop Creek Dam, CA).

FIGURE 10-2. Sloping step concept (Left Hand Valley Dam, CO).

10.1 DOWNSTREAM FACING METHODS

Many of the early RCC overtopping protection projects had unformed downstream faces because they were simple to construct and didn't require formwork. It is difficult to achieve compaction of the RCC along the outside surface; therefore, designers often assumed several inches of the RCC was sacrificial and could be expected to be lost during an overtopping event. To account for the expected loss of material, many designs included extra thickness of the RCC to allow for this occurrence. This unformed surface was also subject to damage from wet/dry and freeze-thaw conditions because the RCC was not very dense and therefore had a lower compressive strength and higher permeability relative to the well-compacted RCC several inches back into the mass of the lift. The Bishop Creek Dam project in Figure 10-1 shows the loose aggregate on the surface after 10 years in service. As a result of the challenges and poor aesthetics of unformed steps, designers evolved to specifying compacted unformed sloping steps. This was done to improve the density of the face of the RCC and still achieve some energy dissipation. Figures 9-15 and 9-16 show typical methods to achieve the sloping step concept. The Left Hand Valley Dam in Colorado used the sloping step concept (Figure 10-2). Further evolution in design methodology occurred such that most RCC spillways now incorporate formed steps on the downstream slope. Although it adds complexity to construction due to the requirement for formwork, it is easier to obtain thoroughly and consistently compacted RCC against the formwork, and no overbuild is necessary.

10.2 STRUCTURES THAT EXPERIENCED MAJOR FLOOD EVENTS

The RCC structures that have experienced flow events have performed very well. In Georgia, a large flood event in September 2009 produced flows ranging from the 500-year event up to approximately 50% of probable maximum flood (PMF). Several NRCS Small Watershed dams in Gwinnett County had just recently been protected with RCC overtopping protection. Several had a grass covering over the RCC for aesthetic reasons. Figure 10-3 shows the Yellow River Structure Y-16 during the flood event, and Figure 10-4 shows the structure after the event. As the design anticipated, the overtopping protection activated and portions of the grass covering eroded to expose the RCC. The RCC performed as designed, and no damage or breaching of the embankment the RCC was designed to protect occurred. Three other watershed structures also experienced overtopping. In all cases, the RCC performed exceptionally well with no damage, and the structures remained ready to protect the embankments if additional flooding were to occur. In Colorado, Left Hand Valley Dam (Figure 10-5) and Leyden Dam (Figure 10-6) overtopping spillways operated during a storm event, and aside from some loose aggregate observed in the stilling basins, no serious damage occurred as a result of the event.

FIGURE 10-5. Overtopping spillway activated during storm event (Left Hand Dam, CO).

FIGURE 10-3. Overtopping spillway during 2009 flood (Yellow River Structure Y-16, GA).

FIGURE 10-6. Overtopping spillway activated during storm event (Leyden Dam, CO).

FIGURE 10-4. Overtopping spillway immediately after 2009 flood (Yellow River Structure Y-16, GA).

Many stormwater detention basins were constructed around the periphery of Las Vegas, Nevada, to protect the urban area from rare but severe flash flooding events. Many of these structures used RCC for drop inlet spillways into the basins or as the overtopping auxiliary spillway over the detention basin embankment to pass the PMF. The structures are located in the channels or "washes" that have the potential for large, infrequent flash flooding. RCC was generally the material of choice because the washes provided a suitable aggregate source on-site for the RCC with minimal processing. Two of these structures, Red Rock and Hiko Springs, have experienced several flash flood events that resulted in several thousand yards of sands, gravels, and cobbles over the RCC spillways into the basins. Figure 10-7 shows the RCC steps at Red Rock after one event. Very little of the RCC material was removed from the heavy abrasive flow regime, and after the accumulated debris was removed from the basins, the structures were immediately ready to provide flash flood protection to Las Vegas again.

FIGURE 10-7. Debris on steps after flood event with very little loss of RCC material (Red Rock Basin, NV).

FIGURE 10-9. Damaged and poorly-patched RCC surface.

10.3 FREEZE-THAW RESISTANCE

RCC has shown very good resistance to abrasion in the field and in the laboratory. Even freeze-thaw resistance has been very good if the exposed RCC surfaces are thoroughly compacted and not subject to frequent saturation. The step treads and the tops of training walls and spillway flat surfaces should be sloped to drain so that the RCC can dry quickly. Freeze-thaw damage can occur if the RCC surfaces are consistently exposed to moisture. More recently, some projects have used an air-entraining additive. As a general rule, the drier the mix, the more difficult it is to entrain air. The higher paste mixes typically use enough water that the air additive can be effective in achieving approximately 4% air content. The worst-case performance RCC project is a result of freeze-thaw damage. Marrowbone Dam in Virginia had RCC overtopping protection added in 2005. This area of Virginia is subject to frequent freeze-thaw cycles. Figures 10-8 and 10-9 show the damaged surfaces of the RCC.

FIGURE 10-8. Damaged RCC surface.

The damage is superficial and does not affect the structural integrity or hydraulic performance of the spillway.

With nearly 200 RCC hydraulic structure projects completed, located in a wide variety of environmental conditions, the performance over the last 40 years has been extremely good. Today's designers are comfortable using RCC and have learned from the early RCC pioneers what mix designs and engineering details work best for each individual location.

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APPENDIX

EXAMPLE PROJECT – TYPICAL DAM

Typical dam was originally constructed for irrigation and to provide water for livestock. Due to growth in the surrounding communities, the project site is surrounded by residential development. Because of downstream development there is now a growing need for flood protection. Typical Dam has been identified to be upgraded to provide downstream flood protection, and for development as a park in the future.

Because of the downstream residential development, the hazard classification has increased from low to high. The State Dam Safety regulation for spillway capacity for a high hazard dam is the probable maximum flood (PMF). There are generally other dam safety concerns that may be reported in dam safety inspections or by the site reconnaissance conducted during the initial project planning phase. For purpose of this example, it will be assumed that there are no other significant dam safety issues to be addressed.

The purpose of this study is to investigate alternatives for modifying the existing dam to provide flood control storage and to meet the current requirements of the State Engineer's Office (SEO) Dam Safety Regulations.

PROJECT DESCRIPTION

Typical Reservoir is situated about 13 miles northwest of the city of Anywhere, USA. Typical Dam is an earthfill structure constructed in 1952, with a maximum height of approximately 48 feet at elevation 615 and a crest length of 1,500 feet. The reservoir has a maximum storage capacity of 1,200 acre-feet and a surface area of 69 acres at the existing spillway crest elevation 600. The original spillway was an earth channel located around the right abutment of the dam. The unlined channel was converted to a concrete-lined spillway in 1970 with a capacity of 13,100 cubic feet per second (cfs) and is still in satisfactory condition.

The outlet works is located near the left side of the embankment and consists of a 36-inch-diameter ductile iron pipe about 260-feet long with a gated concrete intake structure and a concrete terminal structure. The existing outlet pipe was constructed with a concrete cradle. The primary existing project features are summarized in Table A-1.

The original design drawings indicate that the embankment is homogeneous with a cut-off trench. An 8-inch drain tile exists at the downstream toe of the dam with a measured flow of 36 gallons per minute (gpm). No seepage or wet areas have been observed on the downstream slope or at the toe of the dam. A cross-section of the existing dam is shown in Figure A-1.

TABLE A-1. Typical Dam Project Features

GENERAL	
Name	Typical Dam
Stream	Typical Creek
Hazard Classification	I (High)
EMBANKMENT	
Type	Earthfill
Crest Length	1,500± feet
Crest Width	14± feet
Crest Elevation	615
Maximum Dam Height	48 feet
Upstream Slope	1 vertical to 3 horizontal
Downstream Slope	1 vertical to 2½ horizontal
SERVICE SPILLWAY	
Type	Concrete overflow crest with concrete chute
Location	Right abutment
Crest Elevation	600 feet
Crest Length	75 feet
Chute Length	450 feet
Discharge Capacity	13,100 cfs at water surface El 614
RESERVOIR	
Storage Volume	1,200 acre-feet at spillway crest El 600

The downstream slope is grass covered and is mowed once per year. No major seepage areas exist.

SUBSURFACE CONDITIONS

The subsurface conditions have been characterized by 6 test holes and 4 test pits on the crest of the dam, the downstream slope, and near the toe of the dam.

The results of the subsurface investigation indicate that the embankment is homogeneous and primarily comprised of sandy clay. The sandy clay is generally medium stiff to stiff, except near the downstream toe of the embankment where the consistency is generally soft (SPT N-values of 3 to 7) to a depth of about 4 feet. The foundation soil is medium stiff to stiff clay below 4 feet. Sandstone bedrock was encountered about 20 feet deep, at the downstream toe of the dam.

The embankment soil has moderate plasticity, with a liquid limit ranging from 29 to 60 and plasticity indices of 15 to 35. The natural water content varies from about 7 percent to 20 percent. Gradation test results show about 25 to 30 percent clay size material, 30 to 50 percent silt size material, and 20 to 40 percent sand size. The maximum dry density (ASTM D698) of the fill material is 106 pcf, and the optimum water content is 16 percent.

Swell/consolidation tests, performed on samples from the upper 10 feet of embankment, show 5 percent swell under 1 psi surcharge and a pre-consolidation pressure of 1,500 psf.

Water levels were measured in the test holes 24 hours after drilling. The water level was 20 feet below the embankment slope at the test hole locations. The water level in test holes at the downstream toe of the dam, was 1 foot below the ground surface.

HYDROLOGY

The drainage basin area upstream of the dam is approximately 8.9 square miles. The Inflow Design Flood (IDF) was evaluated using an incremental damage assessment based on analyzing and comparing floods from various ratios of the PMF event. The Probable Maximum Precipitation (PMP) was estimated based on the National Weather Service's (NWS) Hydrometeorlogical Report. Results of the analysis indicate that the peak inflow into Typical Reservoir as a result of the PMF is 81,300 cfs with a total runoff volume of 76,600 acre-feet.

Results indicate that incremental flooding resulting from a hypothetical failure of Typical Dam would result in additional structures in the downstream floodplain being in a high danger flood zone. Therefore, the recommended inflow design flood (IDF) for Typical Dam is the PMF. The total storage capacity of the reservoir is approximately 1,200 acre-feet and no significant flood attenuation occurs during routing of the PMF. Typically, flood routings should be performed as part of the spillway design, since attenuation of the flood that can occur in the reservoir can decrease the peak spillway discharge from the peak inflow. However, for the purpose of this example, we will assume that the entire peak inflow at 81,300 cfs must be passed by the existing service spillway and a new emergency spillway.

The estimated discharge capacity of the existing service spillway is about 13,100 cfs. The service spillway will not be modified to maintain the same discharge characteristics as the existing condition up to the 100-year frequency flood. The peak outflow for the 100-year flood through the existing spillway is 5,600 cfs at elevation 608.

BORROW MATERIAL

The area around the dam and lake is highly developed. There is a small 2-acre area that could be used to provide a limited amount of sandy clay (up to 5,000 yd^3) for fill material. Aggregate with a maximum size of about 1½ inches is needed for conventional concrete or RCC. Since on-site material is not suitable to produce concrete or RCC aggregate, it will have to be transported to the site from commercial sources.

EXAMPLE PROBLEM

The existing concrete side channel service spillway is in reasonably good condition, and with limited repairs, the spillway can be used as the service spillway for flows up to 5,600 cfs at elevation 608 and 13,100 cfs at the maximum water level elevation 614. Property and topographic limitations will not allow increasing the existing spillway capacity to pass the IDF at the service spillway location. The outlet works is a ductile iron pipe that is in satisfactory condition to drain the flood pool following storm events. Since the embankment was constructed of relatively impervious soil and the structure will be maintained with a low permanent pool, the drain blanket beneath the RCC overtopping protection will be extended up to the elevation of the permanent pool level, and filter material will be placed beneath the remainder of the overtopping section as shown in Figure A-4. Seepage analyses shows that a drain outlet will be needed at elevation 573. Because there is little chance of freeze-thaw conditions at the site and the RCC will not be subject to critically saturated conditions, low to moderate service conditions will be assumed. Therefore, the design compressive strength will be 3,000 psi at 28 days.

The soft sandy clay at the downstream toe of the dam will be removed beneath the RCC overtopping foundation and stilling basin and replaced by structural fill and a blanket of drain/filter material.

TASK 1 – HYDRAULIC SIZING OF EMERGENCY SPILLWAY

RCC embankment overtopping protection will be evaluated for use as an emergency spillway. The emergency spillway will operate for storms greater than the 100-year frequency event. Evaluate the spillway hydraulics for different spillway crest shapes (broad crest, ogee, and sharp crested weir) and determine the associated spillway crest length for each crest type. Then determine the type of spillway crest to be used and the required spillway crest length.

The maximum water surface is limited to elevation 614 due to land development upstream. This will provide 1 foot of freeboard to the dam crest during the PMF. In order to maintain no change in spillway flows for floods up to the 100-year event, the emergency crest elevation will be set at elevation 608. Therefore, the maximum head on the emergency spillway will be 6 feet (El 614 - El 608). Calculate the required spillway crest length (spillway width) for three alternative emergency spillway crest configurations, using the maximum design head of 6 feet.

1.1 Estimate the crest length for the required emergency spillway capacity using the weir equation:

$$Q = CLh^{3/2}$$

Where: Q = total discharge in cfs (and q_{h6} is the unit discharge in cfs per lineal foot)

C = variable discharge coefficient based on the shape of the weir and head above the weir

L = length of the weir in ft

h = head above the weir crest in ft

The required emergency spillway capacity for all three configurations will be equal to the peak outflow minus existing (service) spillway capacity:

= 81,300 cfs – 13,100 cfs

= 68,200 cfs

a) For Broad Crest Weir: C = 2.6 at 6 ft (h_6) flow depth. Using Figure A-2, estimate the unit discharge at 6 ft of head per lineal foot of crest length (q_{h6})

q_{h6} = 38.2 cfs/LF

Required Spillway Crest Length:

L = (68,200/38.2) = 1785 LF

b) For Ogee Crest Weir: C = 3.9 at 6 ft flow depth. Estimate the discharge from Figure A-2.

q_{h6} = 57.3 cfs/LF

Required Spillway Crest Length:

L = (68,200/57.3) = 1190 LF

c) For Sharp Crested Weir: C = 3.6 and 6 ft flow depth. Estimate the discharge from Figure A-2.

q_{h6} = 52.9 cfs/LF

Required Spillway Crest Length:

L = (68,200/52.9) = 1290 LF

1.2 Estimate the volume of RCC and conventional concrete for an overtopping spillway for the three spillway crest types being considered (i.e., broad crest, ogee crest, and sharp crested weir). Assume a downstream slope of 2½H:1V, a RCC lift width of 10 feet, a crest apron length of 20 ft, and a flow depth of 6 feet over the weir crest. Also, assume that the upstream and downstream cut-off walls and stilling basin length will be the same for all three alternatives.

1.2a **Broad Crest Weir Configuration:**

Calculate the height of overtopping protection. Height of Dam to Spillway Crest (hydraulic height) is equal to the height of dam minus freeboard minus hydraulic head over the spillway crest (see Figure A-2):

= 48 ft - 1 ft – 6 ft = 41 ft

RCC for Crest:

Crest = 3 ft thick x 20 ft crest apron ÷ 27 ft^3/yd^3

= 2.22 yd^3/LF

RCC Overtopping Slope Protection: Hydraulic height minus height of weir.

= 41 ft – 0 ft = 41 ft

= 15.2 yd^3/LF (from Figure A-3)

RCC for Stilling Basin: Assume 40 ft long for comparison purposes.

= 3 ft thick x 40 ft long ÷ 27 ft^3/yd^3

= 4.44 yd^3/LF

Total RCC Required for a Broad Crest Weir:

= (2.22 + 15.2 + 4.44) yd^3/LF

x 1,785 LF (from Step 1.1a)

= 39,030 yd^3

1.2b **Ogee Crest Weir:**

Calculate the height to the spillway crest as shown in Step 1.2a.

Hydraulic Height:

= 41 ft (from Step 1.2a)

RCC VOLUME:

RCC Overtopping Slope Protection: Assume the height of the weir equals ½ of the flow depth.

Height of RCC overtopping protection:

= 41 ft – 6 ft/2 = 38 ft

= 14.1 yd^3/LF (from Figure A-3)

RCC for Approach Apron: Crest apron width at base of ogee crest.

= 2 ft thick x 20 ft crest apron – 8.7 ft (from Fig. 248, USBR 1987a) ÷ 27 ft^3/yd^3

= 0.84 yd^3/LF

RCC for Stilling Basin: Assume 40 ft long for comparison purposes.

= 4.44 yd^3/LF (from Step 1.2a)

Total RCC Required for an Ogee Crest Weir Spillway:

= (14.1 + 0.84 + 4.44) yd^3/LF

x 1190 LF (from Step 1.1b)

= 23,060 yd^3

CONVENTIONAL CONCRETE VOLUME:

Conventional Concrete Crest: Ogee crest to be constructed of conventional concrete.

For a 3 ft high ogee weir (cross-sectional area = 21 ft^2)

= 21 ft^2 / LF ÷ 27 ft^3/yd^3

= 0.78 yd^3/LF

Total Conventional Concrete Required:

= 0.78 yd^3/LF x 1190 LF (from Step 1.1b)

= 928 yd^3

1.2c **Sharp Crest Weir:**

Calculate the height as shown in Step 1.2a.

Hydraulic Height = 41 ft (from Step 1.2a)

RCC VOLUME:

RCC Overtopping Slope Protection: Assume height of weir equals ½ of the flow depth.

Height RCC overtopping

= 41 ft – 6ft/2 = 38 ft

= 14.1 yd^3/LF (same as Step 1.2b)

RCC Apron Downstream of Weir: (see configuration on Figure A-4):

= 1.11 yd^3/LF

RCC for Stilling Basin: Assume 40 ft long for comparison purpose.

= 4.44 yd^3/LF (from Step 1.2a)

Total RCC Required for Sharp Crested Weir:

= 14.1 + 1.11 + 4.44

= 19.65 LF x 1290 yd^3 (from Step 1.1c)

= 25,350 yd^3

CONVENTIONAL CONCRETE VOLUME:

Conventional Concrete for Sharp Crest: Sharp crest to be constructed of conventional concrete.

Height of conventional concrete

= 3 ft

Thickness of concrete weir

= 1.5 ft ÷ 27 ft^3/ yd^3

= 0.17 yd^3/100 LF

Total Conventional Concrete Required:

= 0.17 yd^3/LF x 1290 LF

= 220 yd^3

At this point, the designer would obtain RCC unit cost information from recently completed projects similar in size to this example project to estimate the comparative cost for the three spillway types. For this example, assume that the 1,290-foot long sharp crested spillway will be required, in addition to the service spillway, to pass the Inflow Design Flood of 81,300 cfs.

TASK 2 – STILLING BASIN/HYDRAULIC DESIGN

Hydraulic jump type stilling basins are often used as energy dissipaters for RCC spillways. The hydraulic jump which occurs in a stilling basin will have distinctive characteristics based on the energy to be dissipated and the depth of the flow. The characteristics are expressed by the Froude number parameter:

$$F = \frac{V}{\sqrt{gd}} \quad \text{(USBR 1987a)}$$

Where: V = average velocity at the toe of dam (ft/sec)

g = acceleration due to gravity (ft/sec²)

d = depth of flow entering the stilling basin at toe of the dam (ft)

The USBR and others have performed a series of tests to determine the properties of the hydraulic jump and have recommended certain types of hydraulic jump stilling basins for a range of Froude numbers (USBR 1987a).

A tailwater rating curve is needed to design a hydraulic jump type basin. If the tailwater is not known or cannot be developed, an end sill or wall can be placed at the downstream end of the basin to develop the tailwater required to form the hydraulic jump. Energy dissipation or erosion protection will then need to be provided downstream of the sill.

Assuming that tailwater is available for the entire range of operation for this example, the depth (d2) of the hydraulic jump will be computed. The formula for calculating the conjugate (also referred to as the alternate or sequent depth) depth of the hydraulic jump in a horizontal channel of rectangular cross section is:

$$d_2 = \frac{-d_1}{2} + \sqrt{\frac{d_1^2}{4} + \frac{2V_1^2 d_1}{g}} \quad \text{(USBR 1987a)}$$

Where: d^1 and d^2 are the depths before and after the jump, respectively (see Figure A-5), and V1 is the mean velocity in the water before the jump.

An example of sizing the stilling basin is shown in the following steps.

Step 1

Determine unit discharge (q) for a sharp crested weir:

$$q = CLH^{3/2} = (3.6)\,(6)^{3/2} = 52.9 \text{ cfs/ft}$$

Step 2

Determine tailwater elevation from a tailwater rating curve determined using Mannings equation or computer programs such as HEC RAS (HEC-2001). (The tailwater rating curve is the relationship between the spillway discharge and the depth of flow at the downstream end of the energy dissipator.) For this example assume these computations have been provided by the hydraulic engineer.

Step 3

Determine the energy dissipation on the spillway chute. A stepped spillway surface can decrease the velocity of flow at the bottom of the spillway and therefore reduce energy dissipation requirements when compared to a smooth spillway chute.

The following computations show one method for estimating energy loss for a stepped spillway. There have been numerous hydraulic model studies papers and a hydraulic book that have been written for the design of stepped spillways. At the time of this writing, equations for predicting energy loss on stepped spillways is based upon the results of laboratory studies and theoretical models. The authors are not aware of energy loss data for prototype stepped spillways operating at full design capacity. Some experts believe that the model studies may not accurately predict energy loss for stepped spillways. The designer may find energy loss predictions to be quite large when compared to losses predicted for smooth spillway chutes; this can then result in lesser requirements for stilling basins, rip-rap sizing, plunge pool sizing, etc). The designer is encouraged to become knowledgeable with the full range of analysis of hydraulic structures before designing an energy dissipater for a stepped spillway. The method described herein for estimating energy dissipation for flow through an RCC stepped spillway was developed by Chanson (1995). Depending on the flow regime (nappe flow or skimming flow) developed on the stepped spillway, the energy dissipation can be expressed as a function of flow discharge, dam height, spillway slope, and geometry of spillway steps.

The first step to calculate energy dissipation is to determine flow regime in the spillway. There are typically two types of flow that could occur on a stepped spillway: 1) nappe flow when water bounces from one step onto the next to form a series of free-falling nappes, and 2) skimming flow when the water flows down the stepped slope as a coherent stream, skimming over the step edges. According to Chanson (1995), the limiting condition for skimming flow is:

$$\frac{d_c}{h} \geq 1.1 - 0.4 \left(\frac{h}{l}\right)$$

Where: d_c = critical flow depth (ft)
h = step height (ft)
l = step length (ft)

The following table summarizes the limiting step height for skimming flow under different flow discharges and spillway slopes. If the design step height is smaller or equal to the limiting height in the table, the flow is skimming flow. Otherwise, nappe flow will occur.

For the example Typical Dam, the unit discharge is 52.9 (cfs/ft). The table below shows that the flow over the stepped spillway will be skimming flow when the step height is less than about 4.54 ft, on a 2.5:1 spillway slope.

	SKIMMING FLOW LIMITING DEPTH (ft)		
UNIT FLOW DISCHARGE (cfs/ft)	SPILLWAY SLOPE 2:1	SPILLWAY SLOPE 2.5:1	SPILLWAY SLOPE 3:1
5	1.02	0.98	0.95
10	1.62	1.55	1.51
20	2.57	2.46	2.40
30	3.37	3.23	3.14
40	4.08	3.91	3.80
50	4.74	4.54	4.41
60	5.35	5.12	4.98
80	6.48	6.21	6.04

For skimming flow, the energy dissipation is determined by the following equation (Chanson 1995).

$$\frac{\Delta h}{H_{max}} = 1 - \left[\frac{\frac{d_0}{d_c}\cos\alpha + \frac{1}{2}\,Ec\left(\frac{d_0}{d_c}\right)^2}{\frac{3}{2} + \frac{h_{dam}}{d_c}}\right] \quad \text{(Chanson 1995)}$$

Where:

- H_{max} = maximum available head from downstream toe to waterlevel at top of the stepped spillway (ft)
- h_{dam} = head from downstream toe to crest of stepped spillway (ft)
- d_o = uniform flow depth (ft)
- d_c = critical flow depth (ft)
- E_c = kinetic energy correction/coefficient
- α = spillway slope

A detailed description of these terms, and the relationships described herein are presented in Chanson (1995).

$$d_c = \left(\frac{q^2}{g}\right)^{1/3}$$

$$E_c = \frac{(N+1)^3}{N^2\,(N+3)}$$

Typically, N varies from 6 to 10: Assume N=8.

With the given flow discharge, dam geometry, and geometry of spillway steps for "Typical Dam," the different components required to calculate the energy dissipation are calculated:

$$dc = \left[\frac{(52.9)^2}{32.2}\right]^{1/3} = 4.43 \text{ ft}$$

$$H_{max} = h_{dam} + 1.5\, d_c = (38 + 1.5\,(4.43)) = 44.65 \text{ ft}$$

$$\frac{d_o}{d_c} = \sqrt[3]{\frac{f_e}{8 \sin\alpha}} \quad \text{(From Chanson 1995)}$$

$$= 0.407$$

Where:

f_e = Darcy friction factor (for aerated flow assume 0.2)

$$\alpha = \tan^{-1}\left(\frac{1}{2.5}\right) = 21.8°$$

$$\Delta h = (44.65)\left[1 - \frac{(0.407 \cos 21.8°) + \frac{1}{2}\,(1.0355)\left(\frac{1}{0.407}\right)^2}{\frac{3}{2} + \frac{38}{4.43}}\right]$$

$$\Delta h = 29.13 \text{ ft}$$

Step 4

Determine the flow depth and flow velocity at the toe of the spillway.

After the calculation of energy dissipation over the stepped spillway, an energy equation (such as the Bernoulli equation, USBR 1987a) can be used to calculate the depth of flow at the base of a spillway:

$$\frac{V_0^2}{2g} + Z_0 + d_c = \frac{V_1^2}{2g} + Z_1 + \Delta h + d_1$$

where V_0 is the velocity of water at the top of the stepped spillway, and Z_0 is the elevation of the upstream apron (see Figure A-4); and V_1 is the flow velocity at the base of the spillway, Z_1 is the elevation of the stilling basin (see Figure A-4), and d_1 is the flow depth before the jump, at the toe of the spillway (see Figure A-5). It was assumed that the upstream apron is sufficiently long so that flow over the sharp-crested weir does not affect the development of critical flow at the top of the spillway. With the flow characteristics given in this example, the flow depth at the base of the spillway was determined as follows:

Since,

$$V_0 = \frac{q}{d_c} = \frac{52.9 \text{ cfs / ft}}{4.43 \text{ ft}} = 11.94 \text{ fps}$$

Then,

$$\frac{(11.94)^2}{2(32.2)} + 605.0 + 4.43 =$$

$$\frac{V_1^2}{2(32.2)} + 567 + 29.13 + \frac{52.9}{V_1}$$

$$\frac{V_1^2}{64.4} + \frac{52.9}{V_1} = 15.51$$

$$V_1 = 29.7 \text{ fps}$$

Note: There may be more than one solution to the above equation. Only professionals experienced in hydraulic analysis should determine which answer should be used for design purposes.

The flow depth at the toe of the spillway is:

$$d_1 = \frac{q}{V_1} = \frac{52.9}{29.7} \; 1.78 \text{ ft}$$

Step 5

Compute the conjugate (sequent) depth, d_2, and flow velocity:

$$d_2 = \frac{-1.78}{2} + \sqrt{\frac{(1.78)^2}{4} + \frac{2(29.7)^2 \times 1.78}{32.2}}$$

$$= \frac{-1.78}{2} + \sqrt{0.79 + 97.5}$$

$$d_2 = 9.0 \text{ ft}$$

$$V_2 = \frac{q}{d_2} = \frac{52.9}{9.0} = 5.88 \text{ fps}$$

The conjugate flow depth after the hydraulic jump is calculated to be 9.0 ft. Therefore, the bottom of the stilling basin should be set at an elevation 9.0 ft below the tailwater elevation in order to have a fully developed hydraulic jump in the basin. If the tailwater depth is greater that 9.0 ft, the hydraulic jump will become submerged. If the tailwater is less than 9.0 ft, an undulating hydraulic jump will develop in the basin. Assume that the tailwater elevation for the maximum spillway discharge of 68,200 cfs is elevation 576. Therefore, the elevation of the stilling basin will be set at elevation 567.

Step 6

Calculate required length of stilling basin:

The length of a basin is usually based on a multiple of depth d_2. The length of the basin will vary depending on the type of basin selected. A Type II basin, as defined in USBR 1987a, was assumed in determining the length of the basin for this example.

Determine Froude number parameter:

$$= \frac{V_1}{\sqrt{gd_1}} = \frac{29.7}{\sqrt{32.2 \times 1.78}} = 3.9$$

Based on design charts in Design of Small Dams (USBR 1987a) Figure 9-39, the length of stilling basin is equal to 5.75 d_2. For a depth (d_2) of 9.0 ft, the length of the stilling basin would be approximately 52 ft.

Note: The basin floor elevation with respect to tailwater must be within the proper range for the hydraulic jump basin to operate for the entire range of spillway discharges. If the tailwater is too low, a sweep out condition could occur. The undesirable result of a sweep out condition is that high velocity flow can cause significant erosion downstream of the basin. If the tailwater is too high, a drowned condition could occur and reduce the effectiveness of the basin. Site conditions or other restraints may exist which require placing a stilling basin floor at an elevation outside of the recommended range for the hydraulic jump. In such cases, the designer must accept that the basin will not operate properly. Refer to various publications (such as USBR 1987a) for further discussion.

TASK 3 – TRAINING WALL HEIGHT

The training wall height is calculated by computing the water surface profile along the training wall. The height of the wall will depend upon the computed depth of flow plus additional factors for: bulking due to air entrainment, wave action, and freeboard from the water surface to the top of the wall.

An empirical expression for estimating freeboard for straight spillway walls and has been developed by the USBR (1987a). Because of the greater surface roughness in RCC stepped spillways, the design should consider increasing freeboard height estimated for smooth spillway chutes. In a smooth channel conducting flow at supercritical stage the surface roughness, wave action, air bulking, and splash and spray can be approximated using the following empirical expression from USBR (1987a).

$$H_F = 2.0 + 0.025V\sqrt[3]{d}$$

Where:

H_F = freeboard height

V = velocity of unbulked flow

d = depth of flow

The required wall height should be computed at several locations along the spillway wall. For this example, assume that the water surface depth and velocity at several locations along the chute have been estimated and the wall height is being computed where the maximum flow depth is 3 ft.

Where:

$$V = \frac{q}{A}$$

q = unit discharge from Task 2, Step 1

A= unbulked area of flow per foot width (flow depth)

$V = \frac{q}{A} = \frac{52.9}{3(1)}$

V = 17.63

$H_F = 2.0 + 0.025(17.63)\sqrt[3]{3}$

$H_F = 2.0 + 0.64 = 2.64$

Wall Height (unbulked depth of flow + freeboard)
= 3 + 2.64 = 5.64 ft

Use Wall Height = 6 ft

TASK 4 – CHECK UPLIFT PRESSURES

Check the RCC chute and stilling basin slabs for uplift pressure loading.

Unbalanced uplift pressures can exist under the RCC chute and stilling basin slab. These unbalanced pressures can be caused by an ineffective drainage blanket under the slab or by differential water depth (outside and inside of the basin) caused by normal operation of the spillway and stilling basin, Figure A-5.

The RCC chute slab and the basin slab should be checked for uplift during spillway loading conditions. A spillway chute floor slab should be designed to withstand a minimum of 5 feet of differential hydrostatic uplift when constructed on earth foundations as recommended by the U.S. Army Corps of Engineers (USACE 1990). Hydraulic analysis for design of the spillway chute and stilling basin may indicate a larger uplift pressure could occur which would then be the basis for the design. Probable uplift forces should be estimated conservatively as their magnitude is difficult to accurately predict. Even with a good drainage system under a slab, the effectiveness at a drain should be reduced due to the possibility of plugging.

Basin and chute slabs are usually designed to withstand differential water pressures. During basin operation, water at the inside of the basin can be supercritical and shallow in depth, with high tailwater pressures existing outside of the walls and beneath the basin floor. The following computations check the required thickness of RCC at the base of the sloping chute and at the basin slab for these loading conditions. For example, if the water depth outside the basin is 9.0 ft and the depth of flow in the basin is 2.0 ft, 7.0 ft of net uplift pressure (see Figure A-5) could occur on the slab. The uplift pressure would be the differential pressure head times the unit weight of water (7.0 ft)(62.4 lbs/ft^3) = 437 lbs/ft^2. The required thickness of RCC would then be calculated as follows: Uplift pressure divided by the average unit weight of RCC.

$$\frac{437 \text{ lbs / ft}^2}{150 \text{ lbs / ft}^3} = 2.91 \text{ ft}$$

Therefore, use a 3-foot thick basin slab.

In general, the recommended minimum RCC slab thickness for a stilling basin is typically 3, 1-foot RCC lifts. This is generally adequate to resist uplift pressure for differential water pressure up to approximately:

$$\frac{3 \text{ ft } (150 \text{ lbs / ft}^2)}{62.4 \text{ lbs / ft}^3} = 7.2 \text{ ft of head}$$

TASK 5 – CUT-OFF WALL DESIGN

A cut-off wall is typically constructed at the downstream end of a stilling basin to control erosion at the downstream end of the basin slab. Conventional concrete walls are often constructed for this purpose. The depth of this type of wall will depend on the erosion and degradation potential of material of the downstream channel. The cut-off wall should extend to competent bedrock or below the depth of estimated channel degradation or erosion.

The final layout is of the RCC overtopping spillway section shown in Figure A-5.

TASK 6 – RCC MIX DESIGN

Soil Compaction Method of Mix Design

Step 1

Design Criteria

- RCC overtopping protection is required for an emergency spillway in a moderate climate. The RCC will be normally dry.
- The required compressive strength is 3,000 psi at 28 days.
- The RCC overtopping protection will be placed in 10-foot-wide lifts on the downstream slope of the dam.
- A 1-inch maximum size aggregate (MSA) will be used to limit segregation during placement.
- Air entraining admixture will not be used.
- The project site is in an urban area with existing gravel pits.

- Aggregate quality is to meet ASTM C39 requirements, except for gradation changes shown listed below.

Step 2

Aggregate is available from a nearby quarry that meets the suggested RCC aggregate gradation range (as shown on Figure A-7) and conventional concrete aggregate properties. Aggregate properties are:

PROPERTY	AGGREGATE PROPERTIES	
	COARSE	FINE
Absorption (%)	1.51	1.95
Specific Gravity (saturated surface dry)	2.63	2.85

Step 3

Design RCC mixes at 5 cement contents in about 10 percent increments

- 320 pcy
- 360 pcy
- 400 pcy (mid-range "target" cement content, assuming 7.5 psi per pound of cement)
- 440 pcy
- 480 pcy

Step 4

Conduct modified Proctor compaction test with 400 pcy of Type II, low alkali cement with a specific gravity of 3.15. Since the RCC contains 1-inch maximum size aggregate, ASTM D1557 procedures can be used. The full circular face should be used instead of the sector face. If a 2-inch or 1½-inch maximum size aggregate is used, the following change to ASTM D1557 should be made: each sample point should be placed in 3 lifts instead of 5 lifts and compacted with 93 blows per lift.

The modified Proctor compaction curve for the example RCC mix is shown in Figure 7-1.

Step 5

Select mix design water content:

Optimum moisture content for maximum density = 7.0%
+ 0.5% (Added to provide a more workable mix for full depth compaction)

Therefore:

RCC mix design water content = 7.5%

Step 6

Prepare cylinders following ASTM C1435.

Step 7

Calculate mix proportions and theoretical air content of the RCC at a moisture content of 7.5%, based on the compaction test completed in Step 4:

From Figure 7-1 the wet density equals 151.7 pcf.

Assuming a 1 cubic foot sample, calculate material quantities:

Dry Weight of Solids: 151.7 lbs / 1.075 = 141.1 lbs

Total Water Content: 151.7 – 141.1 = 10.6 lbs

Dry Weight of Aggregate:

$$141.1 \text{ lbs} - \left(\frac{400 \text{ pcy cement}}{27 \text{ ft}^3 / \text{yd}^3}\right)$$

$$= 141.1 - 14.8 + 126.3 \text{ lbs}$$

Dry Weight Coarse Aggregate (plus No. 4 sieve size) (57% of total aggregate) = 126.3 x 0.57 = 72.0 lbs

Dry Weight Fine Aggregate = 126.3 – 72.0 = 54.3 lbs

Absorbed Water in Aggregate:

SSD Weight of Coarse Aggregate
= 72.0 x (1 + 0.015) = 73.1 lbs

SSD Weight of Fine Aggregate
= 54.3 x (1 + 0.0195) = 55.4 lbs

Coarse Aggregate
= 73.1 – 72.0 = 1.1 lbs

Fine Aggregate
= 55.4 – 54.3 = 1.1 lbs

Entrapped Air Content (calculated):

$27.0 \text{ ft}^3 - 26.108 = 0.89 \text{ ft}^3$

$$\frac{0.892 \text{ ft}^3}{27 \text{ ft}^3} = 3.3\%$$

Step 8

Calculate proportions for different cement contents and then prepare additional cylinders for laboratory testing.

Prepare semi-log plot of compressive strengths versus age (See Figure A-7)

Step 9

Select cement content for mix design based on the compressive strength test results and the project design requirements, and then develop mix proportions for specifications and plant operation.

Based on the family of curves in Figure A-7 and a target/design strength of 3,000 psi at 28 days, specify a cement content of 412 pounds per yd³.

Recalculate RCC mix proportions with a cement content of 412 pcy and a total water content of 286 pcy. Assume 3.3% entrapped air content.

CONSTITUENT	WEIGHT OF MATERIAL (lbs/ft³)	WEIGHT OF MATERIAL (lbs/yd³)	SPECIFIC GRAVITY	ABSOLUTE VOLUME (ft³/yd³)
Cement	14.8	400	3.15	2.03
Coarse Aggregate (SSD)	73.1	1974	2.63	12.03
Fine Aggregate (SSD)	55.4	1496	2.85	8.41
Total Water	10.6	286	1.0	N/A
Absorbed Water	2.2	59	1.0	N/A
Free Water (total water – absorbed water)	8.4	227	1.0	3.63
Total Volume of Constituents	N/A	N/A	N/A	26.11

Calculate the weight of constituents assuming 1 cubic foot of RCC, as shown in Step 7.

Dry weight of solids = 141.1 lbs (from Step 7)

Total water content = 10.6 lbs (from Step 7)

Dry weight of aggregate

$$= 141.1 \text{ lbs} - \frac{400 \text{ pcy cement}}{27 \text{ ft}^3/\text{yd}^3} = 125.8 \text{ lbs}$$

Dry weight of coarse aggregate
= 125.8 lbs x 0.57 = 71.71 lbs

Dry weight of fine aggregate
= 125.8 lbs – 71.7 lbs = 54.09 lbs

Absorbed Water in Aggregate:

SSD weight of coarse aggregate
= 71.71 lbs x (1+0.015) = 72.79 lbs

SSD weight of fine aggregate
= 54.09 lbs x (1+0.0195) = 55.14 lbs

Coarse aggregate absorbed water
= 72.79 lbs – 71.71 = 1.08 lbs

Fine aggregate absorbed water
= 55.14 lbs – 54.09 = 1.05 lbs

Convert to weight of material per cubic yard:

Cement: = 412 pcy

Coarse Aggregate:

Dry = 71.71 pcf x 27 ft³/yd³ = 1936.2 pcy

SSD = 72.78 pcf x 27 ft³/yd³ = 1965.1 pcy

Fine Aggregate:

Dry = 54.09 pcf = 1460.4 pcy

SSD = 55.15 pcf = 1489.1 pcy

Absorbed Water = 2.13 pcf = 57.6 pcy

Free Water = 286.2 pcy – 57.6 pcy = 228.6 pcy

Step 10

Evaluation of the effect of water content on the strength of the selected mix will not be performed for this example.

Step 11

Design mix proportions by four different methods that can be used for specifications, are shown below for comparison.

	RCC DESIGN MIX PROPORTIONS			
CONSTITUENT	DRY WEIGHT (lbs/yd³)	SSD WEIGHT (lbs/yd³)	PERCENT OF DRY WEIGHT OF AGGREGATE	PERCENT OF DRY RCC MATERIAL
Cement	412	412	12.1%	10.8%
Coarse Aggregate	1936	1965	57%	51%
Fine Aggregate	1460	1489	43%	38%
Total Water Content	286 (7.5%)	N/A	8.4%	7.5%
Air Content (3.3%)	N/A	N/A	N/A	N/A
Free Water	N/A	229	N/A	N/A

Theoretical Unit Weight:

= (412 lbs + 1965 lbs + 1489 lbs + 229 lbs)

= 4095 lbs/yd³ ÷ 27.0 ft³/yd³

= 151.7 lbs/ft³

Step 12

Conduct test section prior to construction of the permanent structure. Evaluate workability, compaction of the RCC to a uniform density for the full depth and a low entrapped air content, equipment type, and number of passes. Adjust mix proportions as needed to meet compaction and mix design requirements.

CONVENTIONAL CONCRETE METHOD OF MIX DESIGN

Step 1

Design Criteria:

- RCC overtopping protection is required for an emergency spillway in a moderate climate. The RCC will be normally dry.
- The required compressive strength is 3000 psi at 28 days.
- The RCC overtopping protection will be placed in 10-foot wide lifts on the downstream slope of the dam.
- A 1-inch MSA will be used to limit segregation during placement.
- The project site is in an urban area with existing gravel pits.
- Aggregate quality is to meet ASTM C39 requirements, except for gradation changes listed below.

Step 2

Aggregate is available from a nearby quarry that meets conventional concrete aggregate properties. Aggregate properties are:

PROPERTY	AGGREGATE PROPERTIES	
	COARSE	FINE
Absorption (%)	1.51	1.95
Specific Gravity (saturated surface dry)	2.63	2.85

Step 3

Estimate the water requirements for a mix with a Vebe time less than 30 seconds and an MSA of 1 inch from Table A.2. Use a water content of 253 pcy. Using Figure A-8, a target design strength of 3000 psi and an aggregate source with a history as conventional concrete aggregate, use a cement content of 375 pcy for the initial trial.

Step 4

Calculate the absolute volume of cement, water and entrapped air content in 1 cubic yard of mix. Cement will be Type II, low alkali with a specific gravity of 3.15.

Cement:	375 lbs/3.15 x 62.4 pcf	= 1.908 ft^3
Water:	253 lbs/62.4 pcf	= 4.055 ft^3
Air (assume 1.5%):	27ft^3 x .015	= 0.405 ft^3

Step 5

Calculate Aggregate Volume:

Unit Volume:	27.0 ft^3
Cement	– 1.908 ft^3
Water	– 4.055 ft^3
Air	– 0.405 ft^3
Total Aggregate Volume	**= 20.632 ft^3**

From Table A-2,

Fine Aggregate (sand rounded) Content = 43%

Fine Aggregate Content:

20.632 ft^3 x 0.43 = 8.872 ft^3

Coarse Aggregate Content:

20.632 ft^3 – 8.610 ft^3 = 11.760 ft^3

Step 6

Calculate volume of paste and mortar and the ratio of the volume of paste to the volume of mortar.

Volume Mortar (V_m):

Cement:	= 1.908 ft^3
Water:	= 4.055 ft^3
Air (Entrapped)	= 0.405 ft^3
Fine Aggregate (minus No. 4)	= 8.872 ft^3
V_m	**= 15.240 ft^3**

Volume of Paste (V_p):

Cement:	= 1.908 ft^3
Water:	= 4.055 ft^3
Air (Entrapped)	= 0.405 ft^3
Fine Aggregate (minus No. 200)	= 20.632 x0.062 (from Figure A-6) = 1.279 ft^3 (approximate)
V_p	**= 7.647 ft^3**

Check paste/mortar volume ratio:

$$V_p / V_m = \frac{7.647\ ft^3}{15.240\ ft^3} = 0.50$$

The ratio is within the limits in Table A-2

Step 7

Convert absolute volume to SSD weight using specific gravity of material (absolute volume x specific gravity x unit weight of water).

CONSTITUENT	ABSOLUTE VOLUME (ft^3)	SPECIFIC GRAVITY (SSD)	WEIGHT (SSD) (lbs)
Cement	1.91	3.15	375
Water (Free)	4.05	1	253
Air (Entrapped)	0.41	N/A	0
Coarse Aggregate	11.76	2.63	1930
Fine Aggregate	8.87	2.85	1578
Total	27.0 ft^3	N/A	4136 lbs

water:cement ratio = 0.67

Theoretical Wet Density (with 1.5% entrapped air content)
= 4136 lbs/27 ft^3 = 153.2 pcf

Step 8

Prepare trial batch and run Vebe test in accordance with (ASTM C1170). Test results indicate a Vebe time equal to 10 seconds. The mixture is well proportioned but is too wet. Adjust mixing water by 3% for each 10 second change in Vebe consistency. Calculate new trial mixture with 6% (2 x 3%) less moisture = 253 lbs/(1 + 0.06) = 239 pcy.

Repeat Steps 4 through 8.

Convert absolute volume to weight using the specific gravity of the material.

CONSTITUENT	ABSOLUTE VOLUME (ft^3)	SPECIFIC GRAVITY (SSD)	WEIGHT (SSD) (lbs)
Cement	1.91	3.15	375
Water (Free)	3.82	1	239
Air (Entrapped)	0.41	N/A	0
Coarse Aggregate (SSD)	11.89	2.63	1951
Fine Aggregate (SSD)	8.97	2.85	1595
Total	27.0 ft^3	N/A	4161 lbs

water:cement ratio = 0.64

Theoretical Wet Density (with 1.5% air)

$$= \frac{4161 \text{ lbs}}{27.0 \text{ ft}^3} = 154.1 \text{ pcf}$$

Calculate volume of paste and mortar and the ratio of the volume of paste to the volume of mortar.

Volume Mortar (V_m):

Cement:	= 1.908 ft^3
Water:	= 3.825 ft^3
Air (Entrapped)	= 0.405 ft^3
Fine Aggregate (minus No. 4)	= 8.971 ft^3
V_m	**= 15.109 ft^3**

Volume of Paste (V_p):

Cement:	= 1.908 ft^3
Water:	= 3.825 ft^3
Air (Entrapped)	= 0.405 ft^3
Fine Aggregate (minus No. 200)	= 20.862 x 0.062 (from Figure A-6) = 1.293 ft^3 (Approximate)
V_p	**= 7,431 ft^3**

Check paste/mortar volume ratio:

$$V_p / V_m = \frac{7.431 \text{ ft}^3}{15.109 \text{ ft}^3} = 0.49$$

The ratio is within the limits given in Table A-2. Vebe time = 30 seconds. Mixture is well proportioned and workable. Use for trial mix proportions.

Step 9

Prepare cylinders in accordance with ASTM C1435. RCC mix with 375 pcy cement, resulted in an average lab density of 153.5 pcf.

Step 10

Re-calculate entrapped air content for molded cylinders. Unit weight of mix: theoretical air free density = 4160.6 lbs/(27-0.405) ft^3 = 156.4 pcf

$$\text{Entrapped Air Content} = \frac{156.4 - 153.5}{156.4} = 1.9\%$$

Step 11

Proportion additional RCC mixes at different cement contents (in 10 to 15 percent increments), using Vebe test to obtain similar workability. The cement contents should bracket the estimated cement content of 375 pcy. In addition, the mix design constituents, including fine and coarse aggregate, need to be adjusted for each cement content by repeating Steps 4 through 7, assuming an entrapped air content of 1.9%.

Step 12

Plot unconfined compressive strength results at the different cement contents. Based on the family of curves from Figure A-9, a cement content of 440 pcy should be specified to achieve a design target strength of 3000 psi at 28 days. The water content and mixture proportions can then be interpolated between the mix designs for the 425 and 475 pcy cement mixes.

Step 13

Re-proportion RCC mix constituents with a cement content of 440 pcy, a water content of 246 pcy, and the calculated air content of 1.9%.

Repeat Steps 4 through 7

Cement: 440 lbs/(3.15 x 62.4) = 2.239 ft³

Water: 246 lbs/62.4 pcf = 3.94 ft³

Air (1.9%): 27 ft³ x 0.019 = 0.513 ft³

Calculate Aggregate Volume:

27.0 ft³ – 2.239 ft³ – 3.94 ft³ – 0.513 ft³ = 20.30 ft³

Fine Aggregate Content:

20.308 ft³ x 0.43 = 8.732 ft³

Coarse Aggregate Content:

20.308 ft³ – 8.732 ft³ = 11.576 ft³

Calculate Aggregate Weight

Fine Aggregate (SSD):

8.732 ft³ x 2.85 x 62.4 pcf = 1552.9 lbs

Coarse Aggregate (SSD):

11.57 ft³ x 2.63 x 62.4 pcf = 1899.6 lbs

From Step 8, check that the paste/mortar volume ratio is within the limits of Table A-2:

$$V_p / V_m = \frac{7.952}{15.425} = 0.52$$

The ratio is within the limits of Table A-2.

Calculate the total water content = Free water content + absorbed water in aggregate, divided by the weight of dry aggregate plus cement:

Free Water	246 lbs
Coarse Aggregate absorbed water 1899.6 – [1899.6/(1+0.015)]	= 28.1 lbs
Fine Aggregate absorbed water 1552.9 – [1552.9/(1+0.0195)]	= 29.7 lbs
Total Weight of Water	**= 303.8 lbs**

Coarse Aggregate Dry Weight 1899.6 – 28.1 lbs	= 1871.5 lbs
Fine Aggregate Dry Weight 1552.9 – 29.9 lbs	= 1523.2 lbs
Cement Dry Weight	= 440 lbs
Total Dry Weight	**= 3394.7 lbs**
Total Water Content (303.7 lbs/3394.7 lbs)	= 8.9%

Theoretical Wet Density (with 1.9% air):

4138 lbs/27 ft³/yd³ = 153.3 lbs/ft³

Step 14

Conduct test section prior to construction of the permanent structure. Evaluate workability, compaction of the RCC to a uniform density for the full depth and a low entrapped air content, equipment type, and number of passes. Adjust mix proportions as needed to meet compaction and mix design requirements.

Comparing Results

In the design example, the RCC mix design was determined using the soil compaction method and conventional concrete method. Although the aggregate source and required compressive strengths (3,000 psi at 28 days) were the same for both methods, the resultant cement, water, and air contents were not identical. These variations are to be expected since the testing procedures for each method are different. These differences highlight the importance of using personnel experienced with RCC and the test section for evaluating the constructability of laboratory mix designs.

CONSTITUENT	ABSOLUTE VOLUME (ft³)	SPECIFIC GRAVITY (SSD)	WEIGHT (SSD) (lbs)
Cement	2.239	3.15	440
Water (free)	3.941	1	246
Air (entrapped)	0.513	N/A	0
Coarse Aggregate (SSD)	11.575	2.63	1900
Fine Aggregate (SSD)	8.173	2.85	1553
Total	27.0 ft³	N/A	4138

Water: cement ratio = 0.56

CONSTITUENT	RCC DESIGN MIX PROPORTIONS			
	DRY WEIGHT (lbs/yd³)	SSD WEIGHT (lbs/yd³)	PERCENT OF DRY WEIGHT OF AGGREGATE	PERCENT OF DRY RCC MATERIAL
Cement	440	440	13%	11.5%
Coarse Aggregate	1872	1900	55.1%	48.8%
Fine Aggregate	1523	1553	44.9%	39.7%
Free Water Content (above SSD)	N/A	246	N/A	N/A
Water Content	304	N/A	8.9%	7.7%
Air Content (1.9%)	N/A	N/A	N/A	N/A

TABLE A-2. Typical Values for Use in Estimating RCC Trial Mixture Proportions

Water Content, Sand Content, Mortar Content, Paste-Mortar Ratio, and Entrapped Air Content for Various Nominal Maximum Size Aggregates. (USACE Technical Memorandum EM 1110-2-2006, Table 3-3)

CONTENTS	NOMINAL MAXIMUM SIZE OF AGGREGATE[a]			
	¾″ (19 mm)		2″ (50 mm)	
	AVERAGE	RANGE	AVERAGE	RANGE
Water content[b], lbs/yd³ (kg/m³)				
a) Vebe < 30 sec	253	224 - 305	206	180 - 236
	(150)	(133 - 181)	(122)	(107 - 140)
b) Vebe > 30 sec	226	185 - 260	201	175 - 210
	(134)	(110 - 154)	(119)	(104 - 125)
Sand content, % of total aggregate volume				
a) crushed aggregate	55	49 - 59	43	32 - 49
b) rounded aggregate	43	38 - 45	41	35 - 45
Mortar content, % by volume				
a) crushed aggregate	70	63 - 73	55	43 - 67
b) rounded aggregate	55	53 - 57	51	47 - 59
Paste: mortar ratio, V_p /V_m, by volume	0.41	0.27 - 0.55	0.41	0.31 - 0.56
Entrapped air content on – 1½ in. (37.5-mm) fraction, %	1.5	0.1 - 4.2	1.1	0.2 - 4.1

[a] Quantities for use in estimating water, sand, mortar, and entrapped air content for trial RCC mixture proportioning studies.

[b] Lower range of values should be used for natural rounded aggregates and mixtures with low cementitious material or aggregate fines content.

TABLE A-3. Fine Aggregate Grading Limits

SIEVE SIZE	CUMULATIVE PERCENT PASSING
3/8 in. (9.5 mm)	100
No. 4 (4.75 mm)	95 – 100
No. 8 (2.36 mm)	75 – 95
No. 16 (1.18 mm)	55 – 80
No. 30 (600 µm)	35 – 60
No. 50 (300 µm)	24 – 40
No. 100 (150 µm)	12 – 28
No. 200 (75 µm)	6 – 18
Fineness modulus	2.10 – 2.75

(USACE, EM 1110-2-2006, Table 3-2.)

TABLE A-4. Ideal Coarse Aggregate Grading

SIEVE SIZE	CUMULATIVE PERCENT PASSING		
	3 in. TO NO. 4 (75 TO 4.75 mm)	2 in. TO NO. 4 (50 TO 4.75 mm)	¾ in. TO NO. 4 (19.0 TO 4.75 mm)
3 in. (75 mm)	100		
2½ in. (63 mm)	88		
2 in. (50 mm)	76	100	
1½ in. (37.5 mm)	61	81	
1 in. (25.0 mm)	44	58	
¾ in. (19.0 mm)	33	44	100
½ in. (12.5 mm)	21	28	63
⅜ in. (9.5 mm)	14	18	41
No. 4 (4.75 mm)	–	–	–

Reference: USACE, EM 1110-2-2006, Table 3-1.

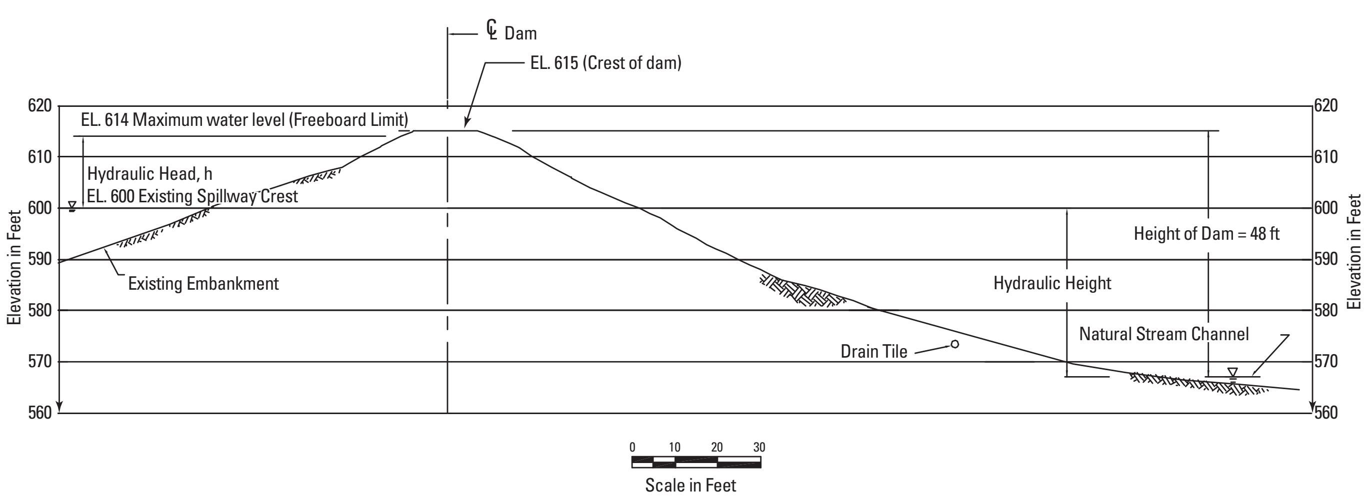

FIGURE A-1. Typical Dam – Existing cross section.

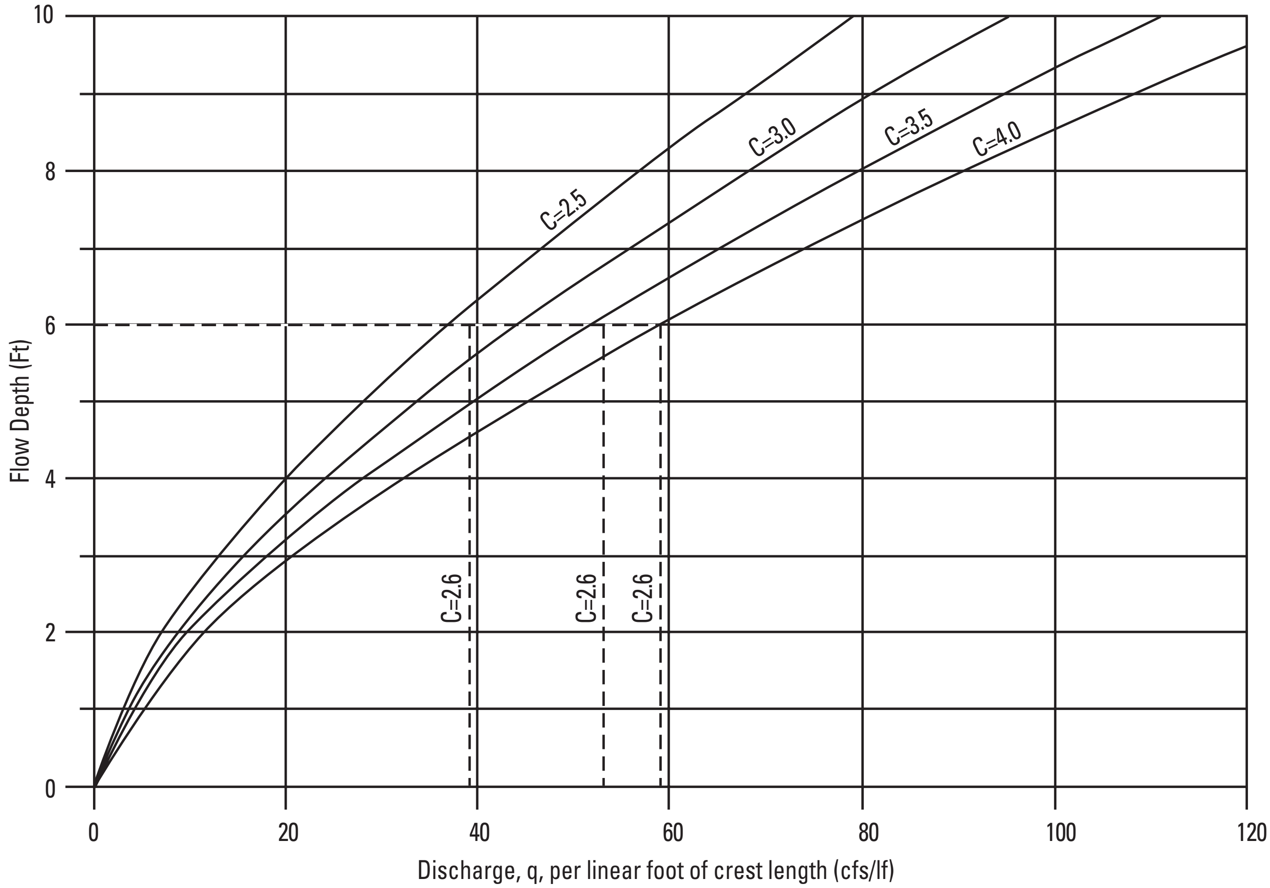

FIGURE A-2. Discharge versus crest coefficient and flow depth ($q = C*L*h^{3/2}$).

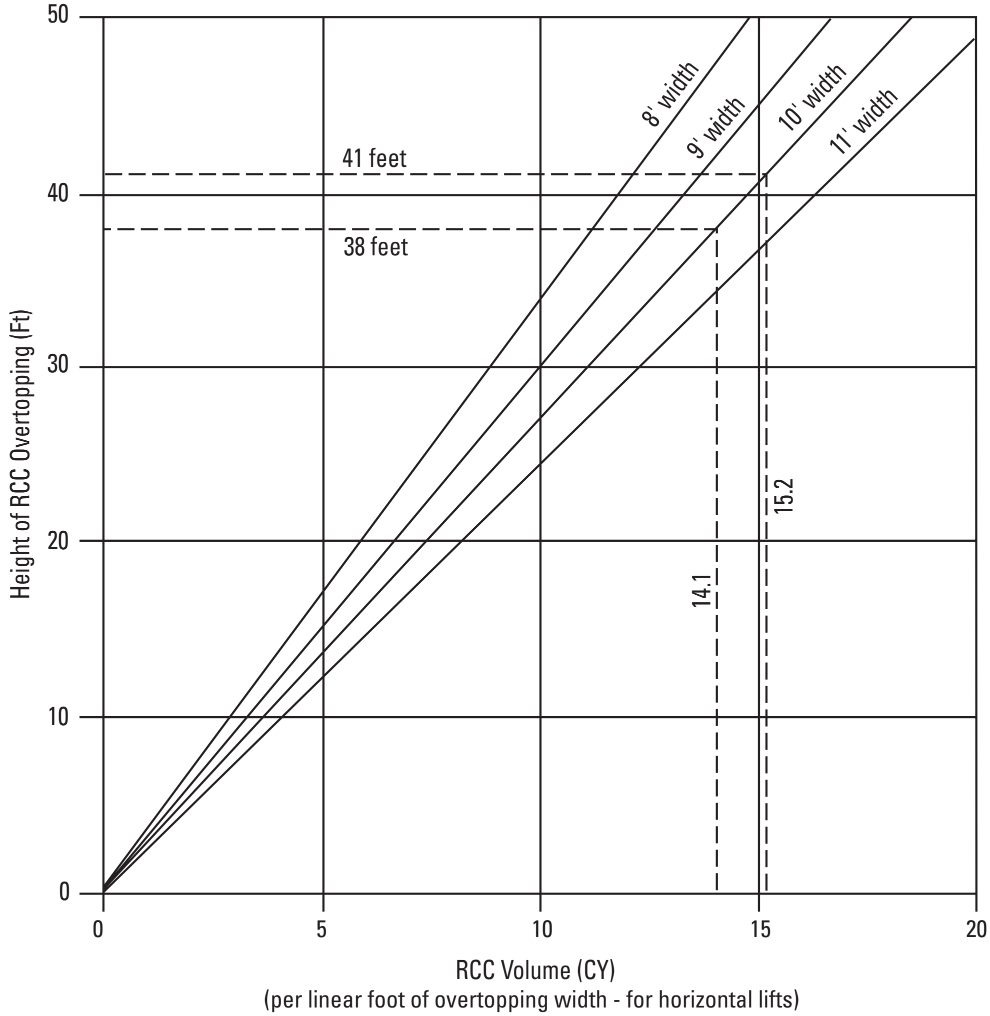

FIGURE A-3. Height of RCC overtopping versus RCC chute volume.

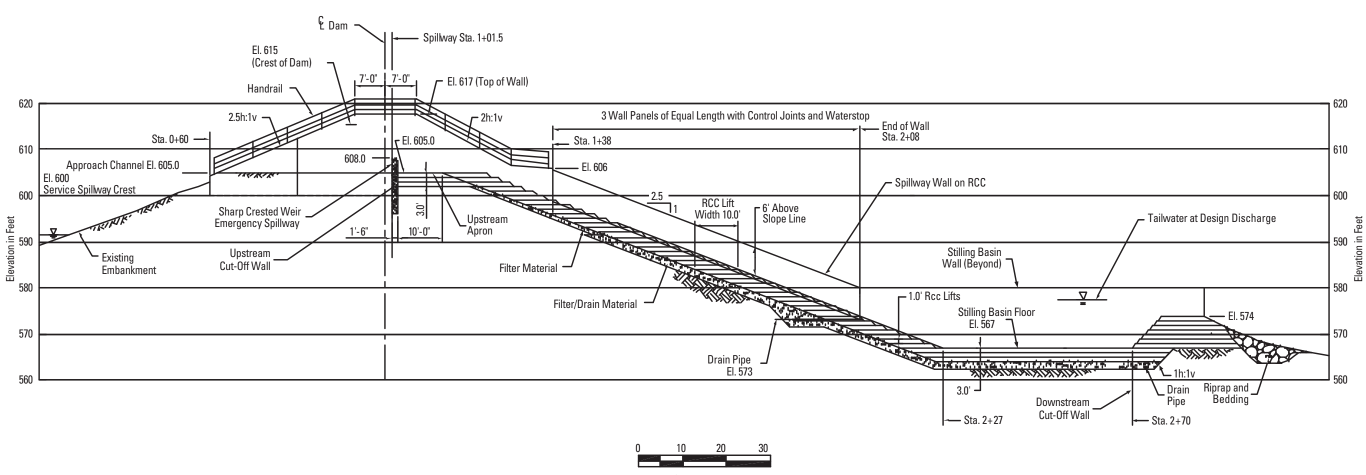

FIGURE A-4. RCC spillway overtopping section – Typical Dam.

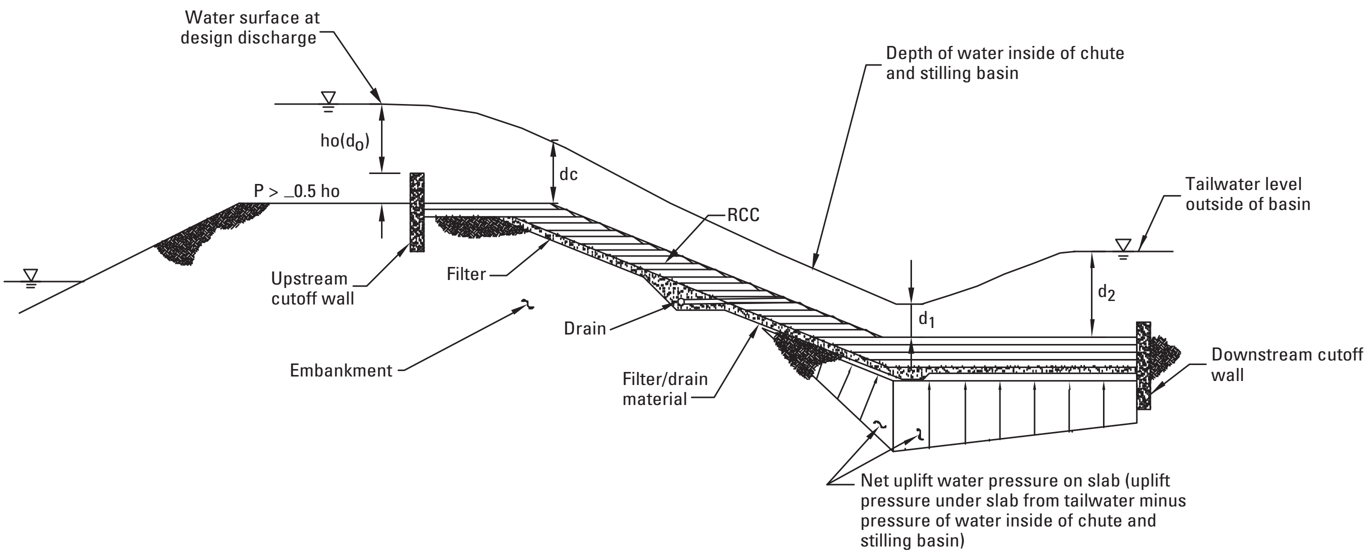

FIGURE A-5. Hydraulic design parameters.
NTS

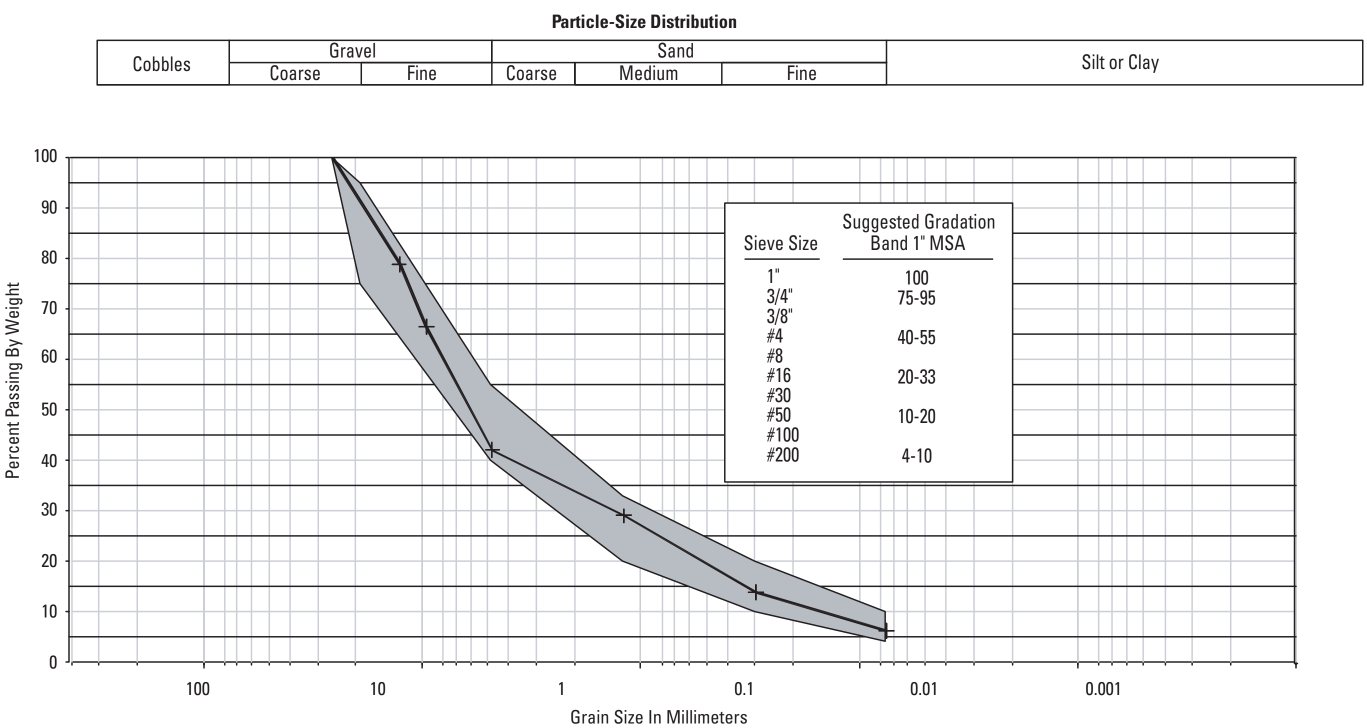

FIGURE A-6. RCC aggregate gradation example.

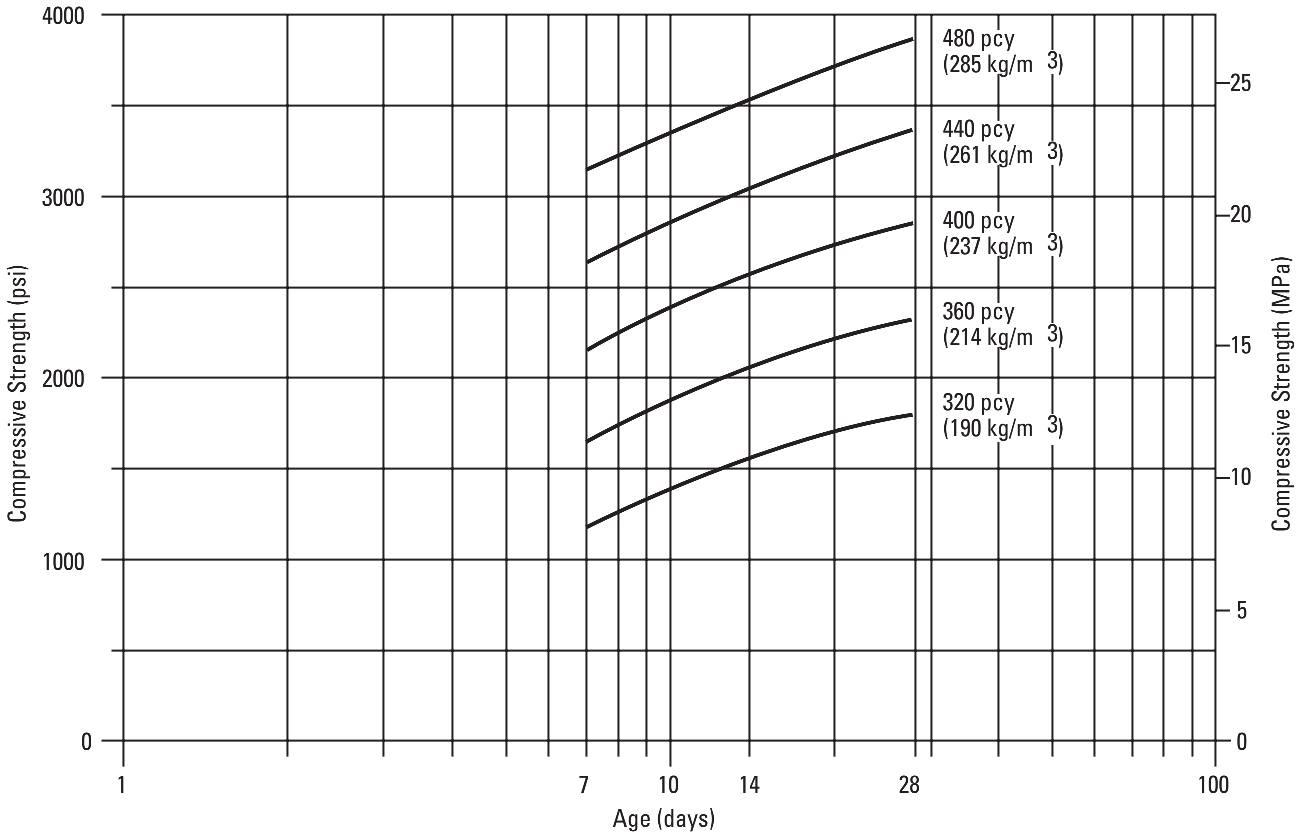

FIGURE A-7. RCC compressive strength versus age for various cement contents – soil compaction method.

(Important note: Information provided in this graph is based on specific project data. This graph is meant as a proportioning aid for trial batches and should not be relied upon to accurately estimate results for final mixture designs.)

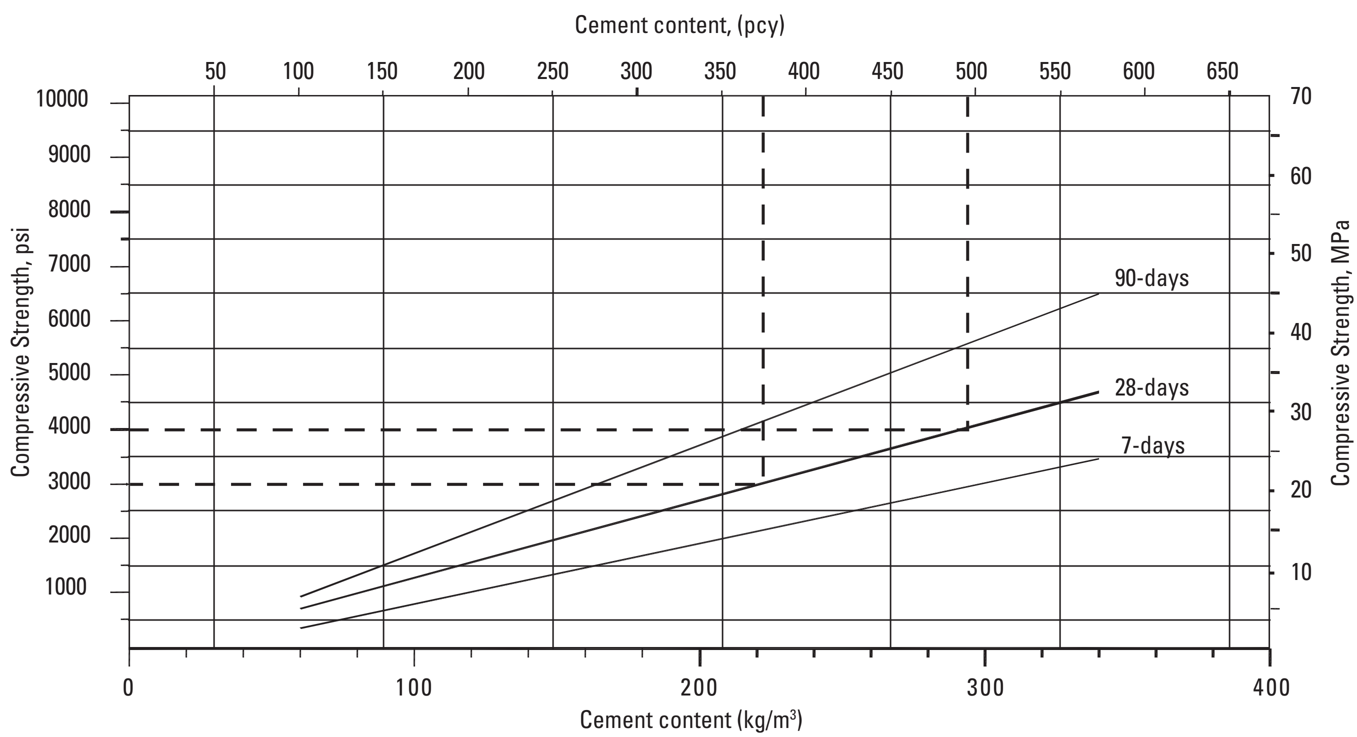

FIGURE A-8. Compressive strength versus cement content for various ages.

(Ref. USACE Technical Memorandum EM 1110 Figure 3-2.)

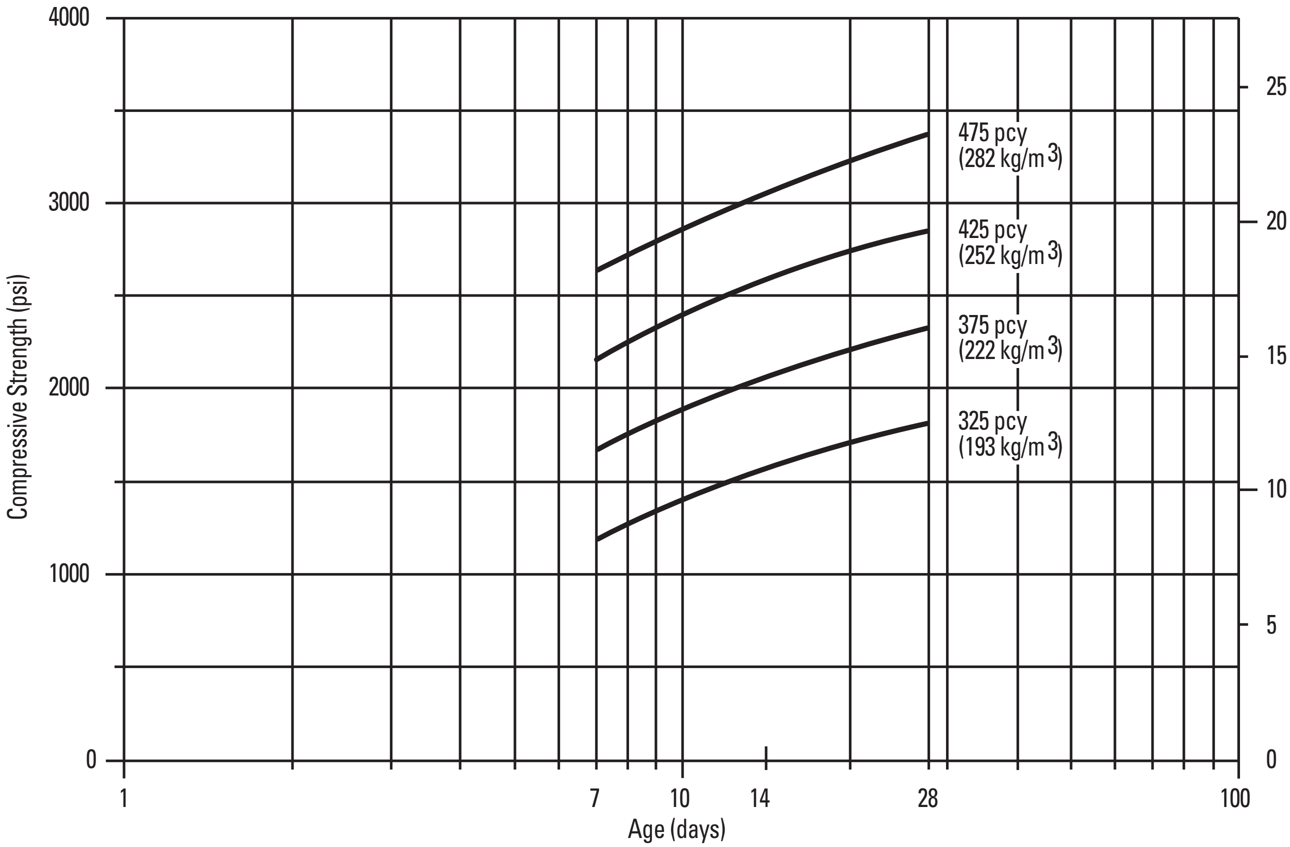

FIGURE A-9. RCC compressive strength versus age for various cement contents – conventional concrete method.

(Important note: Information provided in this graph is based on specific project data. This graph is meant as a proportioning aid for trial batches and should not be relied upon to accurately estimate results for final mixture designs.)

SELECTED CONVERSION FACTORS TO SI UNITS

TO CONVERT	INTO	MULTIPLY BY
Square yard (yd^2)	Square meter (m^2)	0.8361
Square foot (ft^2)	Square meter (m^2)	0.0929
Foot (ft)	Meter (m)	0.3048
Inch (in.)	Millimeter (mm)	25.4
Ton (2000 lb)	Kilogram (kg)	907.185
Pound (lb)	Kilogram (kg)	0.45359
Pounds per square inch (psi)	Kilopascals (kPa)	6.8948
Cubic yard (yd^3)	Cubic meter (m^3)	0.7646
Horsepower (HP)	Kilowatt (kW)	0.7457
Fahrenheit (°F)	Celsius (°C)	5⁄9 (°F - 32)
Cubic foot (ft^3)	Liter (L)	28.316
Gallon (U.S.)	Liter (L)	3.785
Fluid ounce per pound (fl. oz./lb)	Milliliter per kilogram (mL/kg)	65.2

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