DESIGN MANUAL FOR RCC SPILLWAYS AND OVERTOPPING PROTECTION

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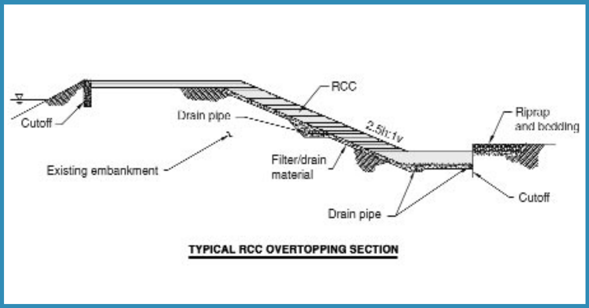

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Cover photos
Upper left: Kerrville Dam, Texas
Bottom: Blue Diamond Dam, Nevada.

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PORTLAND CEMENT ASSOCIATION

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CHAPTER 1

RCC Spillways and Overtopping Protection

1.0 BACKGROUND

Structural concrete has a long history of usage for spillways for dams and other hydraulic structures. Roller Compacted Concrete (RCC) is a relatively new construction technique for concrete placement that is being applied for hydraulic structures. 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 concrete.

In general, applications using RCC were very limited prior to the beginning of the 1980s. Tarbela Dam 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, Pakistan. More than 420,000 cubic yards (yd^3) of RCC was placed. This new construction technique was quickly tested with a flow of 400,000 cubic feet per second (cfs) for about 6 hours at Tarbela Dam (Figure 1.1). No observable damage occurred from the test flow and subsequently, the structure continues to perform satisfactorily.

Figure 1.1. Spillway flow in RCC repair area at Tarbela Dam, Pakistan.

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 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, b), the dam has been subjected to flash floods as well as frequent over-

Figure 1.2a. Ocoee Dam No. 2, Tennessee.

Figure 1.2b. Ocoee Dam No. 2, Tennessee.

topping from operational releases for whitewater rafting. The RCC has shown no apparent damage due to water flows or weathering. Other RCC water resource applications include:

- New gravity dams such as Willow Creek Dam in Oregon, Middle Fork and Stagecoach Dams in Colorado, Stillwater Dam in Utah, Winchester Dam in Kentucky, and Monksville Dam in New Jersey.
- Dam stability improvements at Gibraltar Dam and Littlerock Dam in California, and Santa Cruz Dam in New Mexico.
- Drop structures for channels and riverbeds in Arizona, New Mexico, and Nevada.

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 facilities.

Hydraulic deficiencies can often 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 spillway classifications due to downstream development. Typically, the required IDF for a spillway ranges from 50 percent of the probable maximum flood (PMF) up to the full PMF for high hazard dams. This ultimately resulted in very large peak flows using present day hydrometeorological standards. As a result, a means to significantly increase (economically) the hydraulic capacity of the facility (and in particular for spillways), was needed to restore 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 lower cost placement of large volumes of concrete than conventional concrete. These advantages made 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 (Figure 1.5) and Spring Creek Dam in Colorado (Figure 1.6), where

Figure 1.3. Brownwood Country Club, Texas.

Figure 1.4. North Fork of the Toutle River, Washington.

Figure 1.5. Harris Park No. 1, Colorado.

Figure 1.6. Spring Creek Dam, Colorado.

rapid construction and/or budget constraints were driving forces in identifying alternative designs. The cost effectiveness of RCC overtopping protection was proven in these early projects where the relatively high hauling, placement, and compaction production rates yielded lower unit costs than for conventional concrete spillways. Overtopping protection subsequently saw sporadic application in the following years with a total of 11 projects constructed in the 1980s, and then continued to grow to almost 50 projects in the 1990s. The highest structure constructed to date is 110 feet. A list of completed overtopping protection projects is shown in Table 1.1.

Overtopping spillway projects generally range in height from 15 to 65 feet (with five projects over 65 feet high) with the volume of RCC typically ranging from 1,000 to 56,000 yd^3. The largest RCC overtopping project constructed to date had 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 cfs per lineal foot width of spillway, and an average overflow depth of 5 feet.

There are some significant differences between conventional concrete and RCC spillways. Conventional concrete spillways consist of reinforced, air-entrained concrete placed in sections with water-stopped joints, under-drains and anchorage to resist uplift. In contrast, RCC spillways consist of non-air-entrained concrete, without reinforcement, water-stopped joints or anchorage. RCC spillways have under-drain systems similar to conventional concrete spillways. Also, reinforced concrete spillways are rarely constructed over an earth embankment.

RCC spillways/stream control structures have been designed for flood frequencies of less than 100 years with little or no impoundment storage. For structures that impound water such as earth embankments, designing spillways with RCC overtopping protection is generally limited to emergency spillways with flood frequencies of 100 years or higher based on discussions with engineering practitioners. When identifying the design flood frequency, 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 considerations.

With the rapid growth of overtopping protection projects, the application of RCC has evolved empirically, whereas design methods and analyses have been slower to develop. This document is intended to primarily be a guide for the technical 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.

Table 1-1. 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, Tennessee	Tennessee Valley Authority	27	4,450	—	—	—	—	—
2. North Fork Toutle River (1980) Replacement service spillway	Castle Dale, Washington	U.S. Army Corps of Engineers Portland District	38	18,000	1½	500 + 0	—	8	Mountain Eng. & Const. Co. Bozeman, Montana
3. Brownwood Country Club (1984)	Brownwood, Texas	Brownwood Country Club Freese & Nichols	19	1,400	1½	310 Type IP	24.7	5.5	Central Plains Const. Co. Shawnee Mission, Kansas
4. Spring Creek (1986)	Gunnison, Colorado	Colorado Div. of Wildlife Morrison-Knudsen Engineers	53	4,840	1½	225 + 0	44.4	4.5	Gears, Inc. Crested Butte, Colorado
5. Harris Park #1 (1986)	Bailey, Colorado	Harris Park Water & San. Dist. Edward Shaw	18	2,300	1½	285 + 0	91	10	Pridemore Const. Co. Montrose, Colorado
6. Comanche Trail (1988)	Big Spring, Texas	City of Big Spring Freese & Nichols	20	6,500	1½	232 + 39	60	6	Versatile Const. Co. Logan, New Mexico
7, 8. Addicks & Barker (1988)	Houston, Texas	U.S. Army Corps of Engineers Galveston District	48.5 & 36.5	56,700	1½	292 + 244	7.1 & 10.7	2.2	Hassel Const. & Ernst Const. Co., Houston, Texas
9. Bishop Creek #2 (1989) New emergency spillway	Bishop, California	Southern California Edisony So. Cal. Edison/J.M Montgomery	41	4,000	1½	195 + 195	24	3	El Camino Const. Fresno, California
10. Goose Lake (1989)	Nederland, Colorado	City of Boulder Harza Engineering Const.	35	4,200	3	360 + 0	9.1	2.4	Nicholas Const. Co. & SLM Lakewood & Grand Jct., Colorado
11. Boney Falls (1989)	Escanaba, Michigan	Mead Paper Company Harza Engineering	25	4,850	3/4	149 + 184	100	10.0 RCC 6.0 Dam	—
12. Comanche (1990) New spillway	Estes Park, Colorado	City of Greeley Morrison-Knudsen Engineers	46	3,500	1½	300 + 0	101	10	ASI-RCC Buena Vista, Colorado
13. Kemmerer City (1990)	Kemmerer, Wyoming	City of Kemmerer Woodward-Clyde Consultants	31	4,100	3	439 + 0	24	3.6	Nicholas Const. Co. Lakewood, Colorado
14. Thompson Park #3 (1990)	Amarillo, Texas	City of Amarillo HDR Engineering	30	2,730	1½	330 + 0	30	4.3	Versatile Const. Logan, New Mexico
15. White Cloud (1990)	White Cloud, Michigan	City of White Cloud OMM Engineering	15	1,000	3/4	250 + 190	—	1.5	Smalley Const. Scottville, Michigan
16. Ringtown #5 (1991) Combined principal and emergency spillway	Ringtown, Pennsylvania	Borough of Shenandoah Gannett-Fleming	60	6,300	1½	228 + 174	56	7	Mount-Joy Const. Co. Landisville, Pennsylvania
17. Saltlick (1991) Two emergency spillways	Johnstown, Pennsylvania	Johnstown Water Authority Gannett-Fleming	110	11,100	1¾	117 + 125	54	6.6	Charles J. Merlo, Inc. Mineral Point, Pennsylvania
18. Ashton (1991)	Ashton, Idaho	Pacific Power-Utah Power Black & Veatch	60	7,700	3/4	300 + 100	122	12	Gilbert Westein (a Kiewit Co.)
19. Lake Lenape (1991)	Mays Landing, New Jersey	Atlantic County O'Brien & Gere	17	3,050	1	295 + 0	—	3	PHA Const. Cologne, New Jersey
20. Goose Pasture (1991)	Breckenridge, Colorado	Town of Breckenridge Tipton & Kalmbach	65	4,230	1½	330 + 0	95	10	Gears, Inc. Crested Butte, Colorado

Table 1-1. 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
21. Holmes Lake Dam (1991)	Marshall, Texas	T & P Lake, Inc. East Texas Engineering	31	2,800	2½	300 + 0	—	5	Marshall Paving Co. Marshall, Texas
22. White Meadow Lake (1991)	Rockaway, New Jersey	White Meadow Lake Assn. O'Brien & Gere	20	1,000	1	295 + 0	—	1.4	PHA Const. Cologne, New Jersey
23. Butler Reservoir (1992)	Camp Gordon, Georgia	U.S. Army Corps of Engineers Savannah District	43	9,150	1½	223 + 162	137	13.2	Curry Contracting Co. Atlanta, Georgia
24. Horsethief (1992)	Rapid City, South Dakota	Black Hills National Forest U.S. Forest Service, Denver	65	6,250	2	325 + 0	17	4.24	Gears, Inc. Crested Butte, Colorado
25. Meadowlark Lake (1992)	Ten Sleep, Wyoming	Bighorn National Forest U.S. Forest Service, Denver	28	2,550	2	325 + 0	118	10.25	ASI-RCC Buena Vista, Colorado
26. Philipsburg Dam #3 (1992)	Philipsburg, Pennsylvania	PA - American Water Co. O'Brien & Gere	20	1,400	1	295 + 0	14	6.9	—
27. North Potato Creek Dam (1992)	Copperhill, Tennessee	Federal Bankruptcy Court Dames & Moore	35	4,500	1½	170 + 110	340	20	Dames & Moore Atlanta, Georgia
28. Lake Diversion (1993) New emergency spillway	Wichita Falls, Texas	City of Wichita Falls Biggs & Mathews	85	43,230	1½	225 + 37	316	20.4	Central Plains Const. Shawnee Mission, Kansas
29. Lima (1993)	Dell, Montana	Beaverhead Co. Red Rock River W&S District, HKM Assoc.	54	14,800	2	417 + 0	61	9.3	Pete's Excavating Torrington, Wyoming
30. Rosebud (1993)	Rosebud, South Dakota	Rosebud Sioux Tribe Harza Engineering	33	4,700	1	131 + 151	55	7	Pete's Excavating Torrington, Wyoming
31. Umbarger (1993)	Canyon, Texas	U.S. Fish & Wildlife Service GEI Consultants	40	28,500	1½	330 + 0	216	17.5	ASI-RCC Buena Vista, Colorado
32. Ponca (1993)	Herrick, South Dakota	Rosebud Sioux Tribe Harza Engineering	35	7,700	1	200 + 170	167	16	Gears, Inc. Crested Butte, Colorado
33. Lighthouse Hill (1993)	Altmar, New York	Niagara Mohawk Power O'Brien & Gere	18	4,700	1½	295 + 0	50	6.5	Tuscarara Const. Co. Pulaski, New York
34. He Dog (1994) Combined principal and emergency spillway	Paramalee, South Dakota	Rosebud Sioux Tribe Harza Engineering	45	9,500	1	200 + 170	190	17	Pete's Excavating Torrington, Wyoming
35. Long Run (1994)	Lehighton, Pennsylvania	Borough of Lehighton Gannett Fleming	28.5	3,100	1	250 + 150	15.6	2.5	KC Const. & VFL Huntington Valley, Pennsylvania
36. Lake Dorothy (1994)	Barberton, Ohio	PPG Industries ICF Kaiser Engineers	35	6,000	1½	197 + 142	—	4	Kokosing Const. Co. Loudenville, Ohio
37. South Dam #1 (1994)	St. Clairsville, Ohio	City of St. Clairsville Burgess & Niple	40	2,200	1	250 + 0	16.0	3	Beaver Excavating Canton, Ohio
38. Anawalt (1994)	Anawalt, West Virginia	W.Va. Dept. of Natural Resources Triad Engineering	34	3,000	2	361 + 0	61	7.83	Heeter Const. Co. & Gears Spencer, West Virginia
39. North Poudre #6 (1994)	Wellington, Colorado	North Poudre Irrigation Co. Smith Geotechnical	40	2,400	1	350 + 0	30	5	National Const. & Gears Boulder, Colorado
40. South Prong (1994)	Waxahachie, Texas	Ellis Co., WC&I Dist #1 Freese & Nichols	62	52,000	1½	210 + 105 & 270 + 0	48	6.25	Central Plains Shawnee Mission, Kansas

Table 1-1. 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
41. Lake Ilo (1995)	Kildeer, North Dakota	U.S. Fish & Wildlife Service GEI Consultants	38	3,850	1½	312 + 0	58	7	Park Const. Co. & Gears Denver, Colorado
42. Lower Lake Royer (1995)	Fort Ritchie, Maryland	U.S. Army Corps of Engineers, Baltimore District	40	10,000	1½	200 + 100	44.4	6	Kiewit Const. Co. & Gears Baltimore, Maryland
43. Warden Lake (1995)	Wardensville, West Virginia	W.Va. Dept. of Natural Resources Triad Engineering	38	3,100	1½	350 + 0	127	12	Heeter Const. Co. Spencer, West Virginia
44. North Stamford (1995)	Stamford, Connecticut	Stamford Water Co. Roald Haestad, Inc.	25	2,100	1½	200 + 128	22	3.8	John J. Brennan Shelton, Connecticut
45. Big Beaver (1995)	Meeker, Colorado	Colorado Div. of Wildlife Boyle Engineering	92	8,600	3	325 + 0	125	10	Park Const. Co. & Gears Denver, Colorado
46. Smith Lake (1996)	Garrisonville, Virginia	Stafford County, Virginia Woodward Clyde Consultants	60	25,300	2	308 + 0	58	5.6	Branch Hwys. Roanoke, Virginia
47. Lake Throckmorton (1996)	Throckmorton, Texas	City of Throckmorton Hibbs & Todd	21	3,000	1½	280 + 0	—	—	Nobler Road Const. Abilene, Texas
48. Tongue River (1997) Phase II	Decker, Montana	Montana Dept. of Natural Resources ESA Consultants	91	58,600	2	171 + 0	167	12.5	Barnard Const. Bozeman, Montana
49. Hungry Mother (1997)	Marion, Virginia	Va. Dept. of Parks Dewberry & Davis/ GEI Consultants	40	16,450	1½	350 + 50	50	6.6	W&L Paving & Contracting Madison, Virginia
50. Douthat (1997)	Clifton Forge, Virginia	Va. Dept. of Parks Timmors Eng./Schnabel Engr.	45	15,000	1½	292 + 0	—	—	Branch Hwys. Roanoke, Virginia
51. Alvin J. Wirtz Dam (1997)	Marble Fall, Texas	Lower Colorado River Authority Freese & Nichols	105	160,000	1/4	230 + 230	—	14	Barnard Const. Bozeman, Montana
52. Mona Dam (1997)	Juab County, Utah	Current Co. Woodward Clyde Consultants	43	3,400	—	350 + 0	—	—	ASI-RCC Buena Vista, Colorado
53. C&O Canal Dam No. 5 (1998)	Williamsport, Maryland	Corps of Engineers Dewberry and Davis/GEI	20	3,900	-—	180 + 180	—	—	C.J. Merlo Mineral Point, Pennsylvania
54. Left Hand Valley Dam (1998)	Boulder, Colorado	St. Vrain and Left Hand Conservancy District Rocky Mountain Consultants	45	4,920	1½	325+0	63.9	7.9	Gears, Inc. Crested Butte, Colorado
55. Bear Creek Dam (1999)	Portsmouth, Ohio	Ohio Dept. of Natural Resources Fuller, Mossbarger Scott and May	25	3,360	1½	300 + 0	—	4.1	Lo-Debar Const. Newark, Ohio
56. Wolfden Lake Dam (1999)	Portsmouth, Ohio	Ohio Dept. of Natural Resources Fuller, Mossbarger Scott and May	23	2,140	1½	300 + 0	—	3.6	Lo-Debar Const. Newark, Ohio
57. McBride Dam (1999)	Portsmouth, Ohio	Ohio Dept. of Natural Resources Fuller, Mossbarger Scott and May	22	1,940	1½	300 + 0	—	2.5	Lo-Debar Const. Newark, Ohio
58. Robinson's Branch Dam (1999)	Clark Township, New Jersey	Clark Township Schnabel Engineering	20	4,500	1½	291 + 0	55	4.7	J.A. Alexander Inc. Belleville, New Jersey

Table 1-1. 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
59. Lake Tholocco Dam (2000)	Fort Rucker, Alabama	U.S. Army Corps of Engineers	36	26,000	1½	275 + 50	—	6.5	Thalle Construction Mebane, North Carolina
60. Saddle Lake Dam (2000)	Hooiser National Forest, Indiana	Hoosier National Forest NRCS, Ohio	49	9,100	—	320+0	—	8.1	T-C Inc. Indianapolis, Indiana
61. Gunnison Dam (2000)	Gunnison, Utah	Gunnison Irrigation District Jones & DeMille Engineering	35	3,700	1½	350 + 0	81	9	Nordic Ind. Salt Lake City, Utah
62. Coal Ridge Waste Dam (2000)	Longmont, Colorado	Platte Valley Irrigation Co. Rocky Mountain Consultants	28	2,300	1½	325+0	—	5.0	DeFalco-Lee Longmont, Colorado
63. Teter Creek (2000)	West Virginia Barbour County	Civil Tech Engineering	28	5,700	—	361+0	—	12.0	West Virginia Paving Grafton, West Virginia
64. Many Farms (2000)	Many Farms, Arizona	Bureau of Reclamation	45	6,200	1½	280+70	—	7.1	Barnard Construction Bozeman, Montana
65. Fawell Dam (2000)	Naperville, Illinois	Dupage County URS Corp.	23	9,200	1½	375 + 0	—	3.5	James Cape & Sons Racine, Wisconsin
66. Leyden Dam (2001)	Arvada, Colorado	City of Arvada URS Corp.		8,900	1½	425	92	8.4	ASI RCC Buena Vista, Colorado
67. McKinney (2001)	Hoffman, North Carolina	N.C. Wildlife Resource Comm. URS Corp/Schnabel Engr.	17	1,570	1½	450+0	47	5	Atlas Resource Management Fayetteville, North Carolina
68. Jackson Lake Dam (2001)	Jackson, Ohio	Ohio Dept of Natural Resources BBC&M	23	3,600	—	309+0	30	4.6	LoDeBar, Inc. Newark, Ohio
69. Vesuvius (2001)	Ironton, Ohio	U.S. Forest Service Bureau of Reclamation	45	10,000	—	360+0	35	5.7	TC, Inc. Indianapolis, Indiana
70. Potato Ck No. 6 (2002)	Thomaston, GA	Upson Co. and Towaliga River Soil & Water Consrv. District Golder Associates	28	4,730	11/2	—	34.5	4.5	DPS Marrietta, Georgia

CHAPTER 2

Operational Requirements and Spillway Location

2.1 GENERAL

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. Many dam designers and dam safety officials have accepted overtopping spillways for embankment dams as an effective design method of adding emergency spillway capacity. When planning to use embankment overtopping protection as an emergency 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 use of an overtopping protection configuration introduces significant quantities of concentrated flowing water over erodible materials such as an earthen embankment or foundation material at the abutment contacts; which is not a typical engineering design.
- 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 of the dam embankment, and on a rock foundation, where possible.
- 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 changes a grass covered embankment to a concrete covered surface. In addition, RCC usually has a rough, unfinished appearance when compared with conventional concrete. Some consider the rough surface of RCC to be visually more appealing than conventional concrete. The aesthetics of RCC definitely depends on the eye of the beholder, the project setting, and the materials used in construction. Education of owners and the public regarding the aesthetics 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 will impact or erode the overtopping protection.

2.2 OPERATION FREQUENCY AND SPILLWAY LOCATION

RCC overtopping protection structures have been designed as service spillways, such as Lower Lake Royer (see Figure 2.1) and in-stream drop structures such as Cooks Slough (see Figure 2.2). However, most embankment overtopping protection projects function as emergency spillways and have service spillways designed to pass the more frequent floods. It is particularly important that a conventional spillway be utilized for more frequent floods (commonly referred to as a service spillway) for structures that have significant permanent storage. When

Figure 2.1. RCC service spillway (Lower Lake Royer, Maryland).

Figure 2.2. In-stream drop structure (Cooks Slough, Texas).

a structure requires a spillway capacity in excess of the service capacity of the service spillway, an emergency spillway is constructed to convey the additional flow. Emergency spillways are commonly designed to operate at a frequency not exceeding the 100-year storm. When planning to increase the spillway capacity at an existing dam, the designer should try to maintain the hydraulic capacity of the existing service spillway before operation of an embankment overtopping spillway. For example, if an existing service 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 service 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 1 in 100-year event. The need to assess upstream and downstream flooding conditions should be evaluated for each project.

The conversion of an embankment to an overtopping structure can also lead to introducing a new potential failure mode for a more frequent event than the maximum capacity of the existing service 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 RCC technology is relatively new and no significant historical performance records exist for RCC spillways on embankment dams.

Conventional emergency spillway designs locate the spillway away from the dam embankment whenever possible. An RCC side channel spillway is shown in Figure 2.3. If the RCC emergency 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.

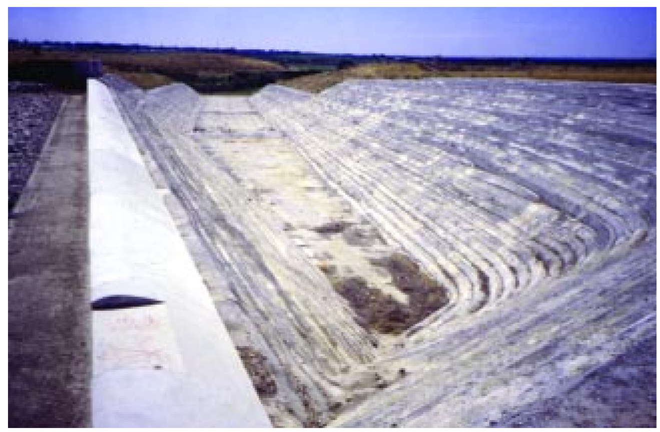


Figure 2.3. RCC side channel spillway (Cold Springs Spillway, Oregon).

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 (trees, cobbles, boulders etc.) without causing severe irregularities in the hydraulic flow, and without snagging anchorage or linkage 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 this lack of historical experience and the need to forestall problems that could lead to potential failure conditions.

2.3 DAM STABILITY AND DOWNSTREAM EROSION

Construction of RCC overtopping protection will also impact the stability of the embankment. RCC on the downstream slope of the embankment can block existing seepage paths and 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.

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 modifications to an existing dam, the embankment should be investigated to understand the current condition of the embankment, foundation, and downstream area, and to develop appropriate geotechnical parameters for design of the modifications. 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, and designing retaining wall and other structures. 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 should be utilized for developing the investigation program.

This section describes guidelines for investigation of modifications to existing dams. For RCC overtopping design on a new dam, the objectives of investigation are generally the same as for existing dams. However, evaluating the properties of the embankment and developing information to predict the behavior and condition following construction need to be included.

3.2 DESK STUDY AND SITE RECONNAISSANCE

The first phase of investigation involves a desk study prior to the site reconnaissance and investigation. Available information on existing dams should be reviewed 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 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 has been monitored, 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 that can be observed during a site reconnaissance. Guidelines have been prepared for conducting inspection of dam embankments, for example, USBR (1983), FEMA (1987), and some state dam safety agencies. These guidelines include standard forms for evaluating the dam embankment and the foundation downstream of the dam.

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 apply to RCC overtopping protection investigation. Additional subsurface investigation methods include Cone Penetration Testing (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 overtopping structure and appurtenant facilities are planned. Investigations must be completed without jeopardizing the safety of the dam. 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 could also be present. Subsurface investigations generally need to be conducted 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 below. 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 encountered, and samples are needed for laboratory testing. Logs of test holes and pits should be prepared in accordance with the locally accepted standard of practice.

Test pits should be backfilled following sampling and logging. Generally, the excavated material will provide suitable backfill. Test holes also need to be backfilled, usually with 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 holes using instruments such as a standpipe piezometer or inclinometers.

Field sampling and testing are a function of the soil types encountered, so some expectation of the soil types to be encountered is needed for planning the investigation, and flexibility is needed in the sampling and testing scope. Generally, soil without sufficient clay and silt content 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.

Samples are 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 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 samples are generally needed for Proctor compaction testing. 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 Unified Soil Classification System (USCS) designation should be recorded (based on visual classification) during drilling or test pitting 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 exceptions to the sampling and testing shown in the table should be expected. It may not be necessary to have all of the indicated sampling and testing performed, because the properties and parameters required for each project varies. 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.

Laboratory testing should be performed in accordance with the American Society for Testing and Materials (ASTM) Standards shown in Table 3.1, or another established testing standard.

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: D 2488
WC	Water content: D 2216
WC/DUW	Water content for the determination of dry unit weight: D 2216
LL/PL	Liquid and plastic limits: D 4318
SA	Sieve analysis: D 422, or C 136 and C 117 (ASTM Vol. 04.02)
HYD	Hydrometer analysis: D 422
WA	Wash analysis or percent fines determination: D 1140 or C 117 (ASTM Vol. 04.02)
GS	Specific gravity, specify sieves: D 854 or C 127, C 128 (ASTM Vol. 04.02)
COMP	Compaction effort, procedures A, B, or C, moist or dry preparation: D 698 or D 1557 or C 1435 (ASTM Vol. 04.02)
UC	Unconfined compression: D 2166 or C 39 (ASTM Vol. 04.02)
S/C	Swell/Collapse potential: D 4546 (Swell & Settlement) or D 5333 (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

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 conducted 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 the project. 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 above. An investigation would typically include contacting local contractors and suppliers, county road departments, and state departments of transportation 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, 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 (minus 200 sieve size) 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 judgement can be made on whether additional sources of aggregate should be investigated. Investigations of aggregate sources should be conducted to confirm a minimum of twice the quantity of material needed for the project. 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 projects: large volume sampling and test blasting. Large volume sampling is (1) based on obtaining the maximum size of the material (ASTM D 75), 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 more fully evaluate borrow sources with large size aggregates. A test blast investigation would include excavation and processing 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 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, and sliding.

Certain projects may have special aspects that require analyses of specific conditions not described here. In some instances, more sophisticated models, such as finite element or finite difference models of deformation may be warranted. However, the standard analyses described here is 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.

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 Section 5.0 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

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 – 1.4	1.3 – 1.4
Steady State Seepage, Normal Pool	1.5	1.5	1.5
Steady State Seepage, Flood Pool	1.5	1.4	1.4
Steady State Seepage, Earthquake	>1.0	1.0	>1.0
Rapid Drawdown	1.3	1.2	1.2

[1] U.S. Bureau of Reclamation (USBR) 1987a
[2] U.S. Army Corps of Engineers (USACE) 1970
[3] Federal Energy Regulatory Commission (FERC) 1991

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 the eroded geometry, should be 1.5 or greater.

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. Bishop's Modified 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, 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 surface 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 water content and phreatic levels that could have adverse impacts if they are not considered in the design. The paragraphs below 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 overconsolidated 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. Consolidation/settlement can be expected to occur gradually, over several months or years.

Settlement can also occur as a result of collapse from wetting. 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 can result from the swelling of some types of clayey soil. Heave resulting from unloading of saturated clayey soil is generally not large, and considering the limited amount of excavation associated with typical RCC overtopping projects, is often insignificant. Heave resulting from increased moisture content in partially saturated clays and weathered claystone can represent a volume increase of 10 percent or more. The degree of heave 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.

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 minimizing volume change. Free draining bedding material is recommended where conditions for frost heave exist.

Bearing Capacity

Bearing capacity is generally not of significant concern because of the light loads typically applied. 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.

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 in 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 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).

CHAPTER 5

Seepage Analysis

5.1 SEEPAGE CONSIDERATIONS

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

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 or foundation under normal reservoir conditions.
- To limit uplift pressures that could develop under the RCC as a result of floods.
- To collect and control infiltration of water through cracks and joints.

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

Seepage – Under normal reservoir levels, seepage can develop through the embankment and foundation and also through the foundation beneath a spillway. RCC is essentially unreinforced concrete, and 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 embank-

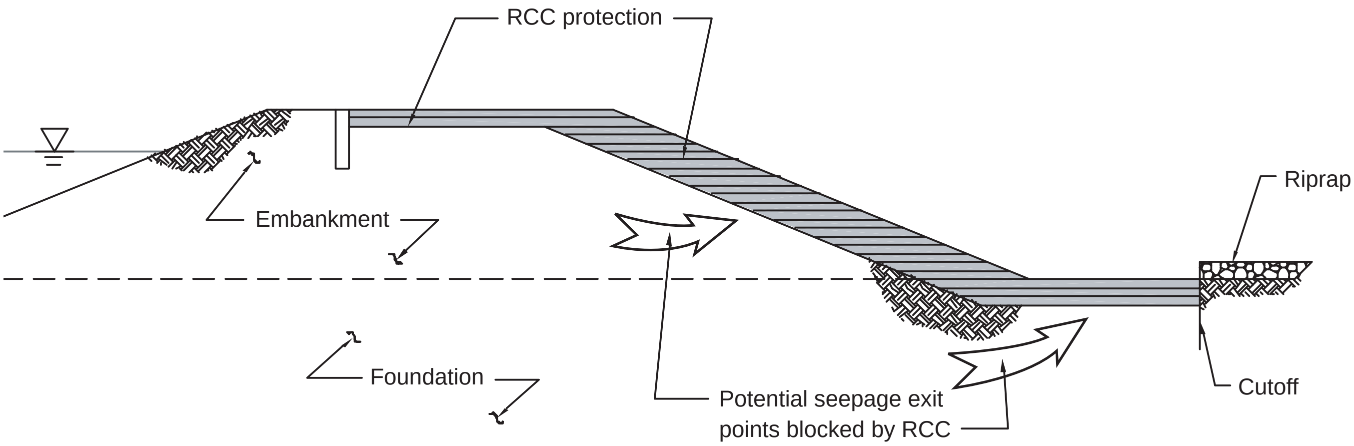


Figure 5.1. Seepage exit points blocked by RCC overtopping protection.

ment and foundation soil which in turn would decrease the stability of the embankment.

If the existing embankment or foundation includes adequate seepage collection and control features upstream of the location of the RCC, then it may not be necessary to include seepage collection and control features in the RCC design. 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 clay 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 system may not be needed. It is possible that the amount of seepage that reaches the face is sufficiently small that it evaporates in the open air, but could build up under the RCC. 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 protection design. Rather, it is recommended that field investigations and instrumentation readings be used to confirm the actual seepage condition in the embankment and foundation before that decision is made.

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 storm 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, 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 protection for seepage under normal pool conditions, it may be advisable to include a geotextile or 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 below.

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 piping 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 pressures to build up beneath the RCC layer. These pressures can build up in a permeable drainage layer beneath the RCC or at the boundary between the RCC and less permeable underlying strata (if no drainage layer is present). If the pressures that build up exceed the combined weight of the overlying RCC and the water on top of the RCC, the RCC could be lifted up and damaged. Movement of the RCC layer during flow over the RCC can lead to erosion, undermining, and failure of the RCC protection.

Pressure can build up beneath RCC overtopping 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 the reservoir is of particular concern, because of the potential to transmit the full reservoir head to the area beneath the RCC.

For pressures to build up under large areas of the RCC slab due to flow in the spillway, hydrostatic 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, conditions during an overtopping event, and conditions 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 drain 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 heave of the RCC could result. In general, heave of the RCC 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 needs to assess how serious this potential condition is and how it should to be addressed in the design.

For low height 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 higher structures, it may be

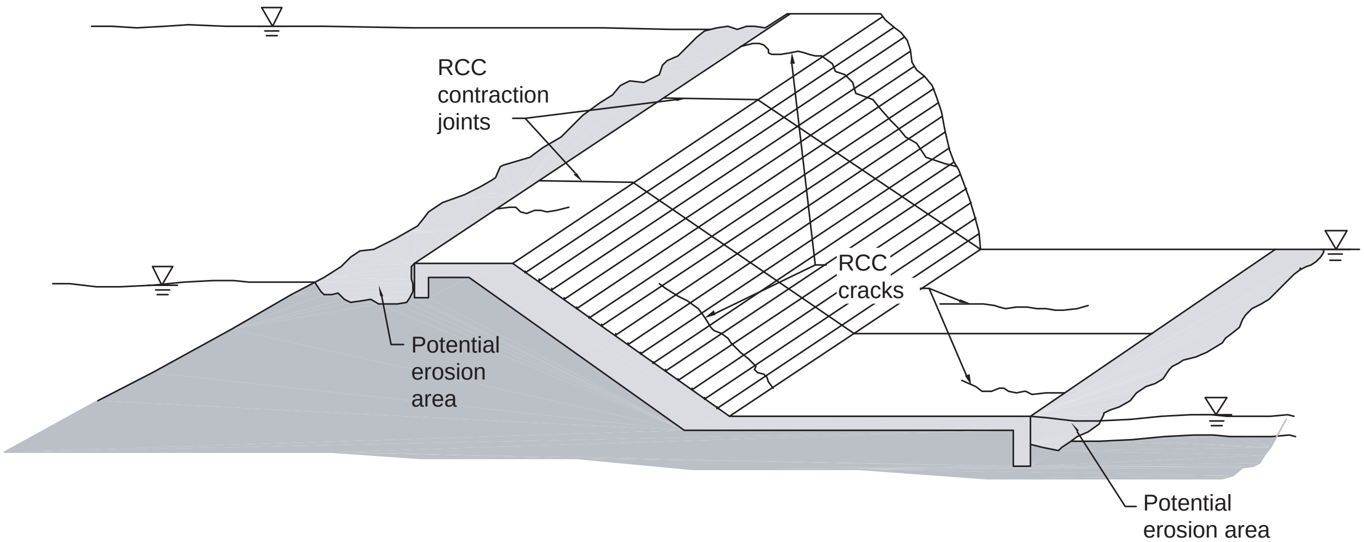


Figure 5.2. Cracks, joints, and potential erosion where water pressure can be transmitted to the area beneath the RCC.

necessary to include specific design features to address the uplift concerns. Conventional concrete spillway slab design includes reinforced concrete with waterstop 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 build up of unbalanced pressures. A typical cross-section for an under-drain beneath RCC overtopping protection 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 piping 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, waterstop, and anchors used in conventional concrete 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 the discussion above, 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 the three stages described above are discussed in the following sections.

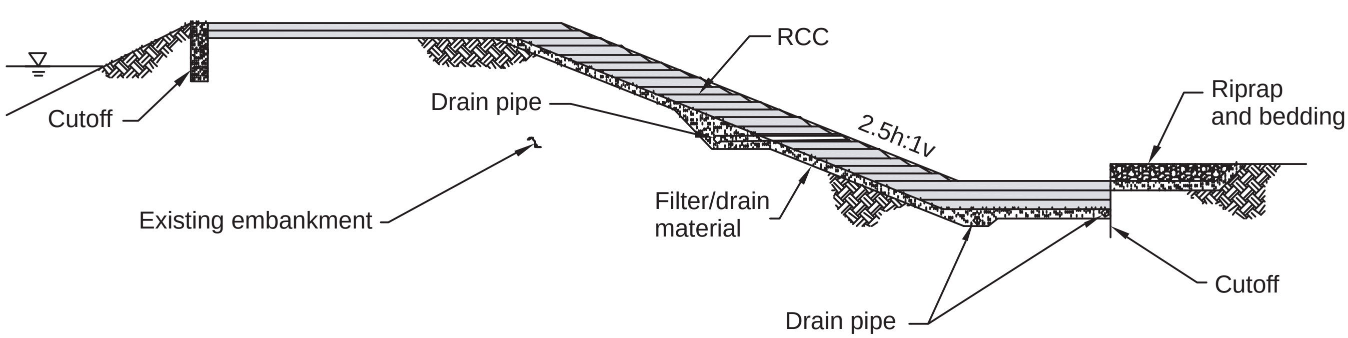


Figure 5.3. Typical RCC overtopping section with underdrain (not to scale).

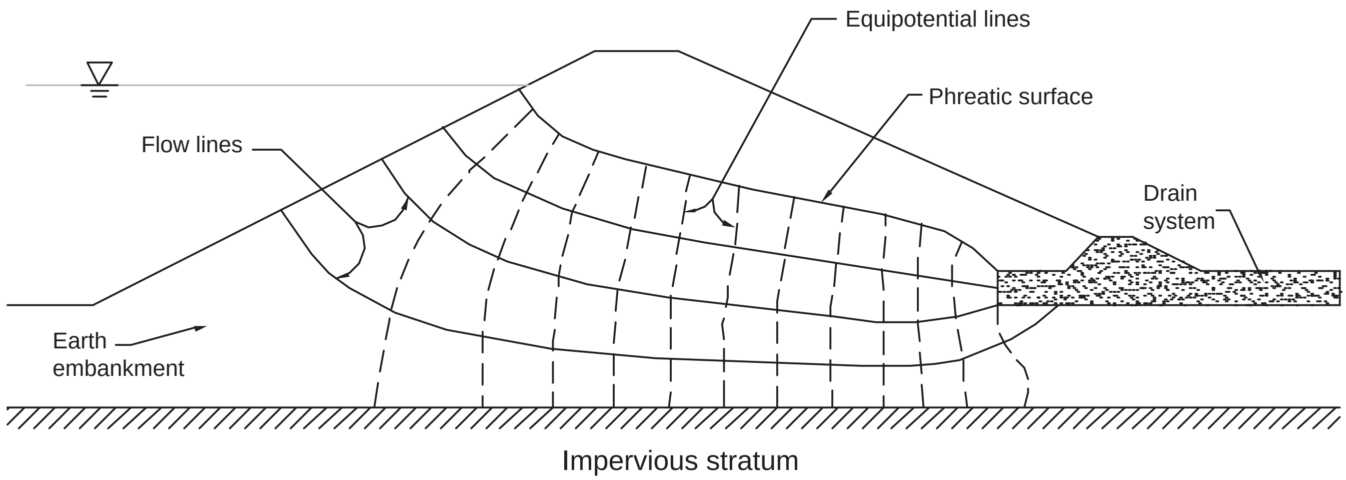


Figure 5.4. Example of a flow net solution for an embankment with a toe drain.

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

As the use and availability of computers has increased, programs have been developed for the analysis of steady-state seepage. However, for many years, these computer programs were not widely used for the analysis of steady-state seepage for dams. Within the past decade, personal computer (PC)-based finite element computer programs 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. 1992). With the advent of the PC-based programs, they 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 is performed using a graphical flow net or a computer program, it must be recognized that the resulting computed flow quantities are 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 5 to 10 for sizing of drainage system components (e.g., sand filter layers, gravel drain layers, collector pipes, etc.).

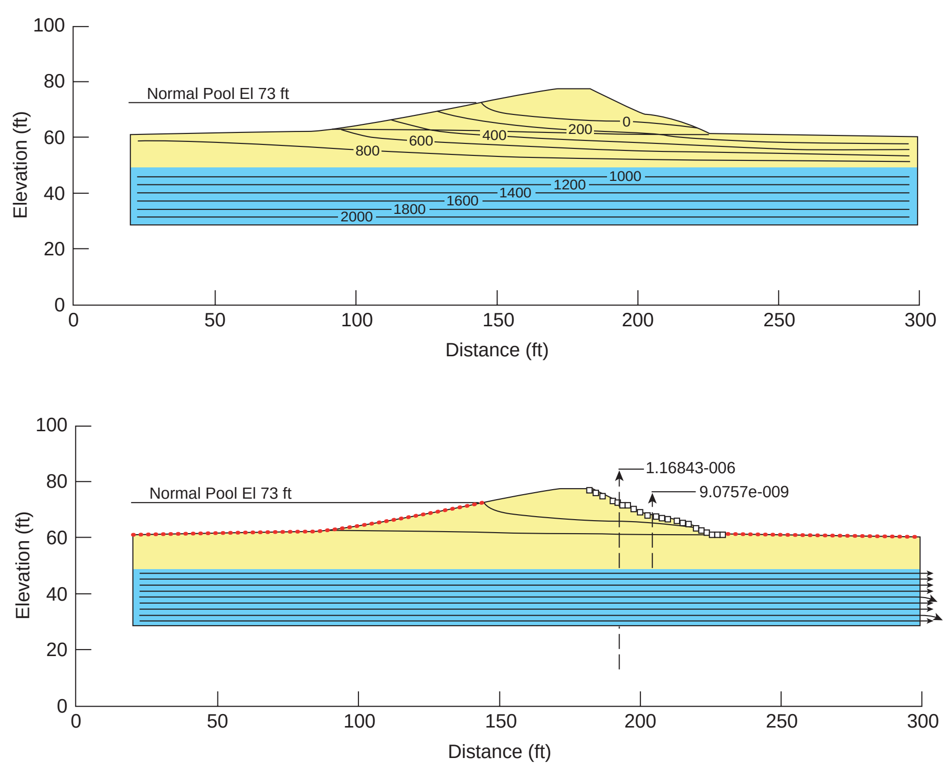
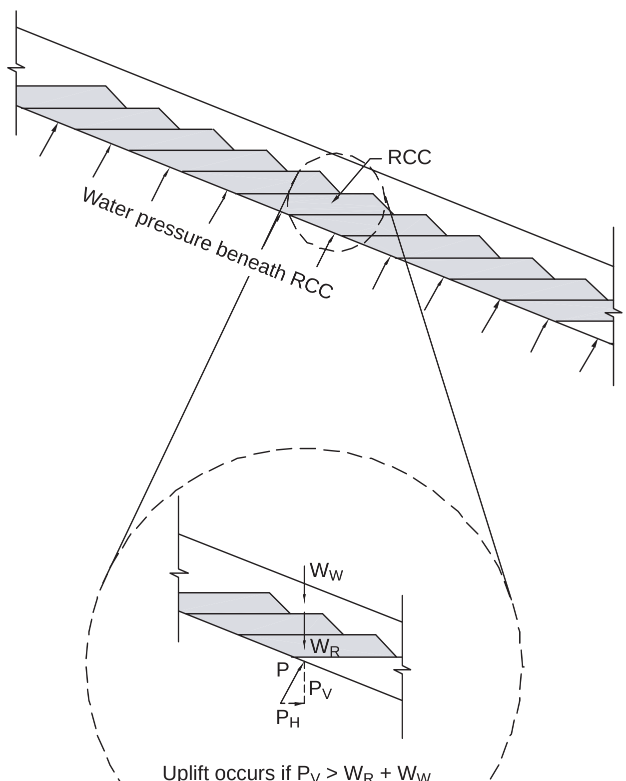


Figure 5.5. Example of results from SEEP/W finite element seepage analysis.

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 the RCC is not well established. The physics of 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.



W_R = Weight of RCC
W_W = Weight of water above RCC
P_R = Water pressure beneath RCC
P_V = Vertical component of P
P_H = Horizontal component of P

Fig. 5.6. Physics of uplift for RCC overtopping protection.

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 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 question 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. Following this school of thought, unbalanced uplift pressures would not build up significantly and no special design features would be needed to address this concern. Another approach suggests that high pressures will build up in 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 protection 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. It is likely that more appropriate analysis methods will be developed as installed systems are tested by overtopping 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, 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 protection installations include 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 spillways and overtopping protection sections, as is customarily done with 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, October 1994; USACE 1993; U.S. Department of Interior, Bureau of Reclamation, June 28, 1999). The specific criteria are not repeated herein.

In principle, filter compatibility guidelines were 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 soil. Published criteria for filter compatibility of slots and perforations are provided in the references noted above.

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 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 from 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 geotextile should be considered as a filter layer directly beneath the RCC. Steady-state seepage is most critical of the sources because the sustained duration seepage from this condition could lead to a piping 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. Filter compatibility beneath a crack should be analyzed using the same methods used for slotted pipes, as described above.

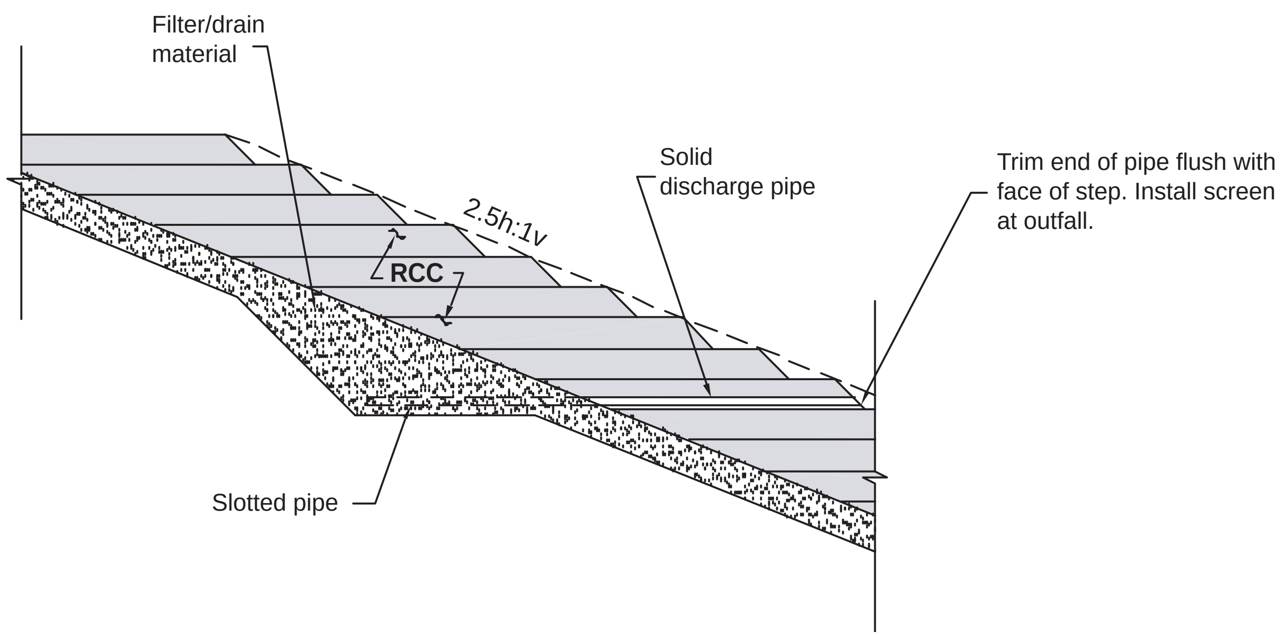


Figure 5.7. Typical details of an underdrain and outlet pipe.

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. 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 limited access to the geotextile for repair or replacement in the future.

CHAPTER 6

Overtopping Spillway Design

6.1 INTRODUCTION

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

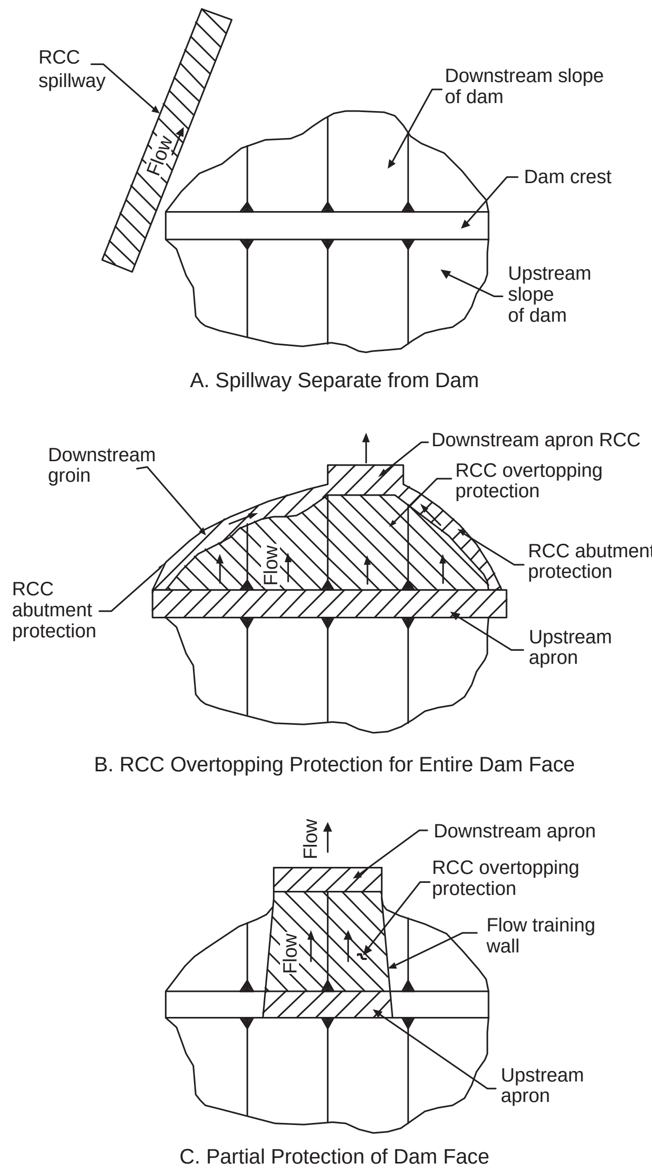


Figure 6.1. Alternative spillway locations.

6.2 SPILLWAY LOCATION

An RCC spillway can be located in three general areas: (1) as a spillway separate from the dam embankment, (2) as spillway overtopping protection over the entire dam embankment, and (3) as spillway overtopping protection over a portion of a dam embankment. The general design locations are shown schematically in Figure 6.1.

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

1. Is separate from the dam embankment when possible.
2. Would not cause excessive erosion at the downstream embankment groins, or at the downstream toe of the dam.
3. 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 contribute to 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. During the 16th to 18th centuries, large stepped fountains were built in Europe and India. Some were 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, Chanson 1995). Most were masonry or concrete structures with granite or concrete blocks protecting the downstream face. Since the beginning of the 20th century, stepped 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 size of the energy dissipation structure at the base of the dam and the potential for scouring of channel and/or foundation material.

The modern development of the RCC construction technique has renewed interest in stepped channels and spillways. 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 overtopping protection provide energy dissipation/reduction. The reduction in energy on the steps leads to a reduction in the size of the required energy dissipation basin, and consequently, the overall cost of the project is typically reduced. The unit discharge, width and height of the steps, overall slope of the overtopping 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). In addition, other published papers include "Model Study of A Roller Compacted Concrete Stepped Spillway" (Rice and Kadavy 1996), and "Design and Laboratory Testing of a Labyrinth Weir and Stepped Spillway System" (Trollope et al. 2001). These papers provide additional information on energy loss associated with stepped spillways.

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 the face of the dam. 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 as shown in 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 two feet, the duration of overtopping is short, and the 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 hori-

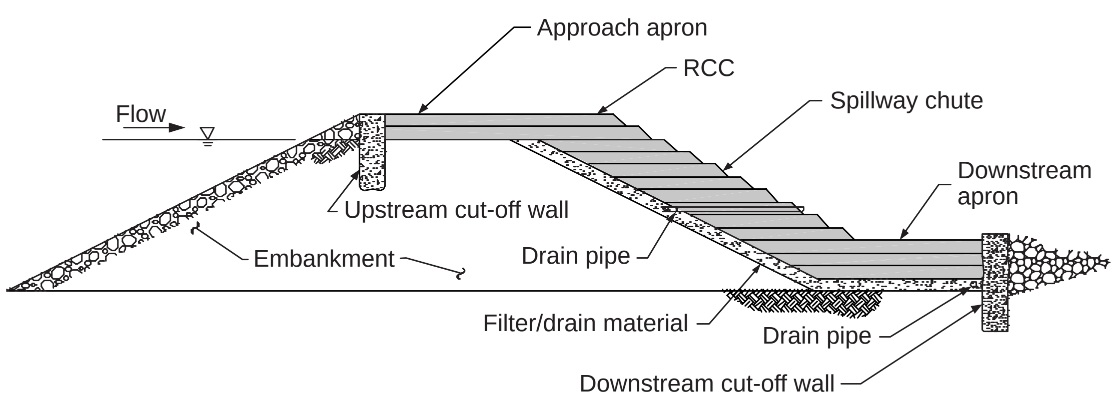


Figure 6.2. Typical section: RCC overtopping protection.

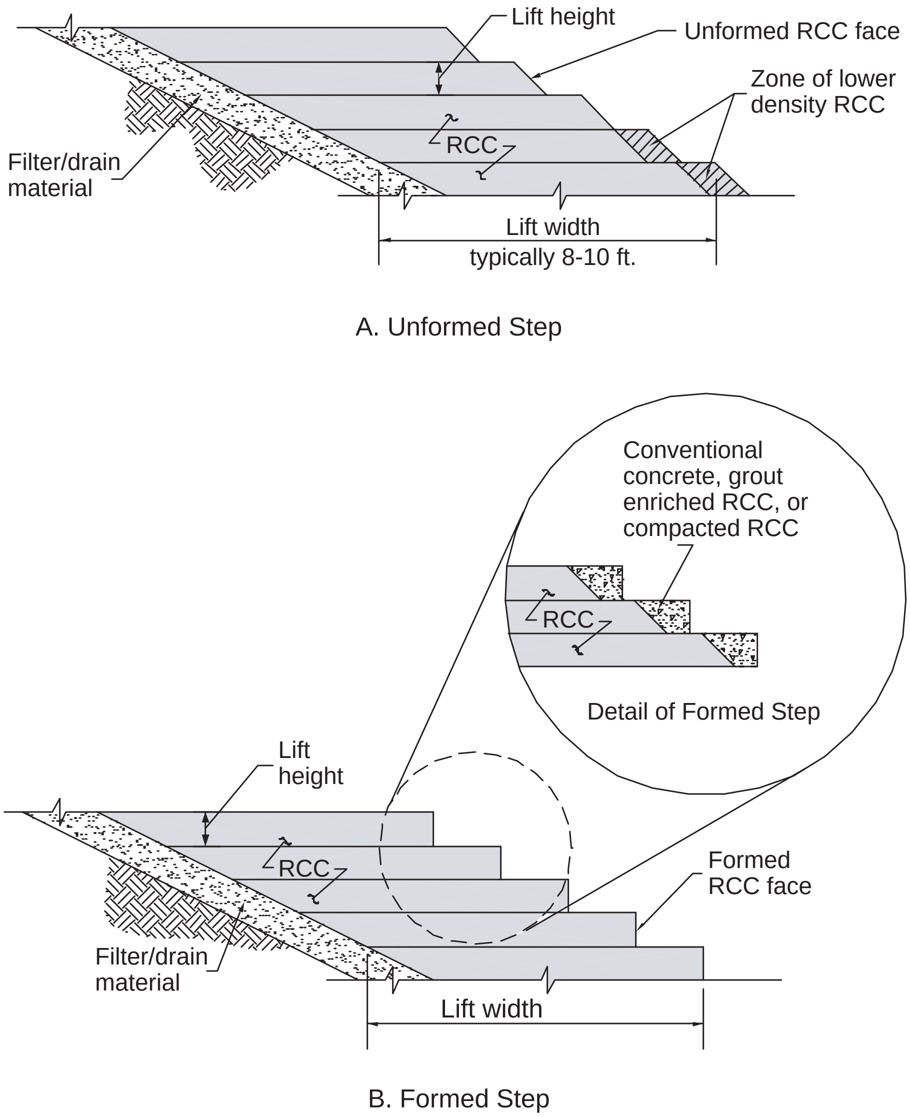


Figure 6.3. Downstream slope geometry of RCC overtopping section.

zontal lifts. The plating configuration also dissipates less energy on the RCC surface than a stepped section, which would therefore require more energy dissipation. One plating application was at Toutle River where one of the primary design considerations was to provide a structure that would allow debris from the Mount St. Helens volcano to flow through the spillway.

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

Unformed Chute

Unformed RCC chutes are usually less expensive and take less time to construct than formed RCC chutes, and therefore, are used more commonly. Unformed RCC is usually end dumped by trucks or placed by a loader and spread by a dozer. 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 has 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

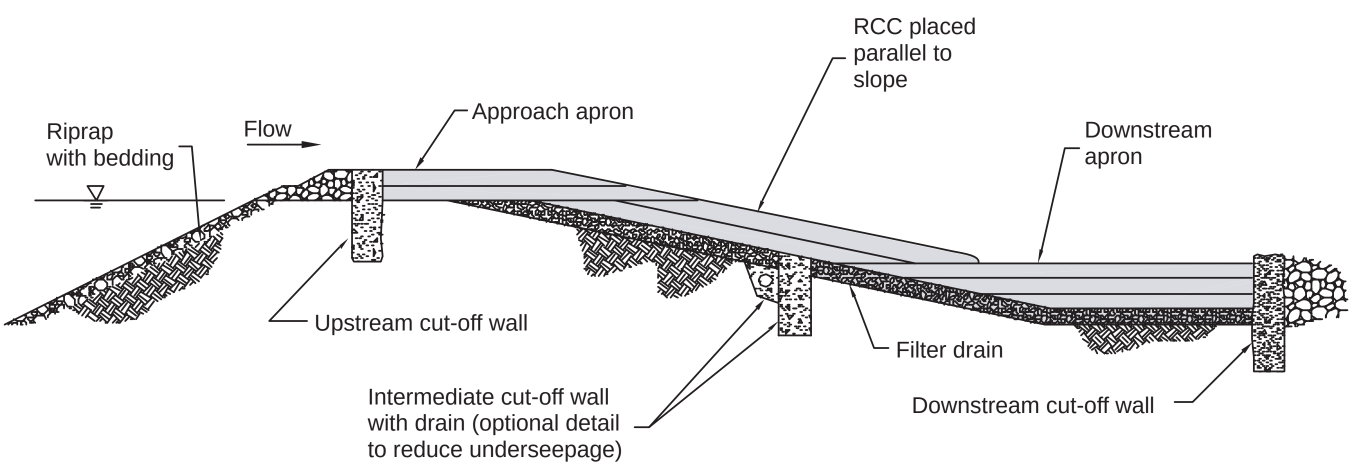


Figure 6.4. Overtopping protection with RCC placed parallel to chute (plating).

RCC density should be considered as "sacrificial concrete" by the designer. A conservative design approach would be to not consider the lower density RCC as part of the wearing surface, and not include the material in the mass for the stability analysis computations.

An unformed RCC face can have the appearance of rough, irregular shaped concrete 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 concrete. Examples of the appearance of unformed, uncompacted downstream RCC face are shown in Figures 6.5 and 6.6. To others, the rough, irregular appearance blends into the natural surroundings. If a smoother finish surface is an important project requirement, the exposed RCC edge can be compacted or trimmed to give a more uniform conventional concrete 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 low Vebe time 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 face are shown in Figures 6.7 and 6.8. Methods of trimming and compacting the outside edge are discussed in Chapter 9.

Figure 6.5. Uncompacted, unformed RCC downstream face (Bishop Creek Dam No. 2, California).

Figure 6.6. Uncompacted, unformed RCC downstream face (Mona Dam, Utah).

Figure 6.7. Unformed RCC chute with compacted RCC (Fawell Dam, Illinois).

Figure 6.8. Unformed RCC chute with compacted RCC (Smith Lake Dam, Virginia).

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 less 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 of 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 of the outside edge, reduces raveling, and increases freeze-thaw resistance; and (3) the appearance of a formed surface (when well constructed).

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 papers by Rice and Kadavey (1996), Houston (1987), Christodoulou (1993), Chanson (1995), and Trollope et al. (2001). Technical information is available on the hydraulics of steeper sloped spillways associated with concrete gravity dams. However, the data will not directly apply to RCC embankment overtopping projects.

The step height can affect the cost, constructability, energy dissipation, and public access to the spillway area. Step heights for RCC spillways generally use 1- to 2-foot high vertical forms. 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. Greater step heights can also result in larger RCC volumes as shown in Figure 6.9.

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 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) the use of pozzolan or additives, and (3) increasing the water/cement ratio. 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. Examples of completed projects with a formed RCC chute are shown in Figures 6.10, 6.11, and 6.12.

The disadvantages of forming include: (1) decreased RCC placement rates; (2) increased requirements for labor-

Figure 6.10. Formed RCC chute (Goose Lake, Colorado).

Figure 6.11. Formed RCC chute (Big Beaver Dam, Colorado).

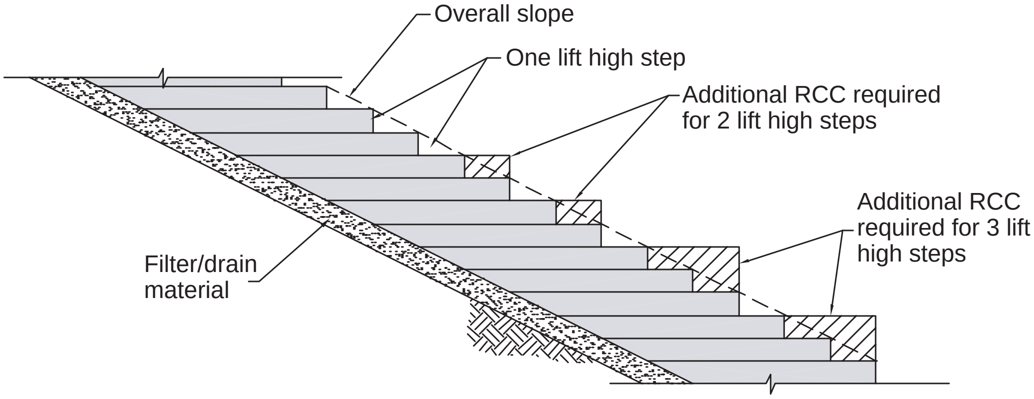


Figure 6.9. Step height comparison of RCC volume.

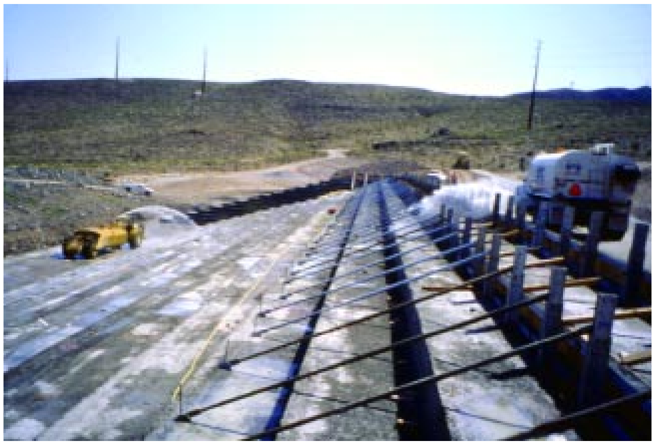


Figure 6.12. Formed RCC chute, 2 foot high steps (Anthem No. 2, Nevada).

ers 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.

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. These relationships are graphically shown in Figure 6.13.

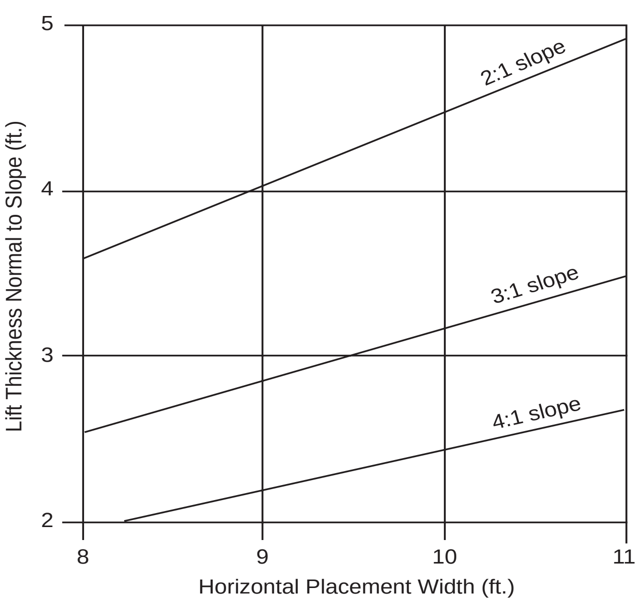


Figure 6.13. Thickness of RCC on the slope versus the width of the lift.

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 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 PROTECTION

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

- Energy dissipation — A wider spillway will usually improve spillway performance by decreasing the depth of flow, decreasing 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, conventional concrete service spillway or a spillway in bedrock.)
- Extending the RCC overtopping protection 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.
- Conversely, the designer may want to limit the crest length of the overtopping protection 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 protection will usually increase the RCC volume, weir crest and stilling basin concrete, and project cost.
- Converging spillway — A converging spillway can be used to provide a wider crest length and better fit a narrow downstream channel. This design configuration requires consideration of the effects of wall convergence on spillway cross-waves and wall overtopping.

The transition from the overtopping width to the downstream floodplain and channel is also important from both operation and maintenance perspectives, and for property considerations. The width required is integral with the crest and control structure design which is described in the next section.

6.6 CREST AND CONTROL STRUCTURES

RCC spillway crests for embankment overtopping protection commonly follow the shape of the embankment crest such as shown in Figure 6.14, which has a relatively low hydraulic efficiency. This can result in simplified construction, but a spillway conforming to the crest of an embankment dam would be considered a broad crest weir, and has a low discharge coefficient, especially for lower depths of overtopping relative to the crest width. Increasing the efficiency of the spillway crest can reduce the required width of the spillway and/or flow depth. Project costs are often reduced using a more efficient (i.e., a crest section with a higher discharge coefficient) crest. Therefore, it is important for the designer to be aware of alternative spillway crest designs and how crest geometry can affect spillway performance and cost.

Conventional concrete can be used to shape a crest control section, such as an ogee crest, a flat curved crest, or sharp crest weir to improve the discharge coefficient and reduce the upstream water surface. Adding a more efficient weir crest requires more conventional concrete and forming to the project, than a broad crest weir with RCC, and will add to the project cost and limit future access. The designer should be aware that the discharge coefficient of all the weirs will vary with the approach channel conditions, approach depth conditions, depth of flow over the weir, and tailwater conditions. Refer to general design references (USBR 1987a, USACE 1996) for a discussion of these effects.

Constructing highly efficient crest designs such as an ogee shape often requires significant steel reinforcement and highly skilled labor to provide the smooth curved surface (Figures 6.15 and 6.21b), and will increase the cost of the concrete placement. Crest designs such as sharp crest weirs or modified ogee/flat curve shapes can be used to improve spillway hydraulics with lower unit costs than an ogee section. A significant technical benefit of a more efficient crest design is the decrease in required width of the spillway, and/or reduction in the maximum water surface, which typically reduces material quantities. A narrower spillway chute width can also better match the downstream channel geometry. The following general guidelines should be considered when selecting the crest design:

- Broad Crest Weir Design — This design configuration consists of paving the crest with RCC as shown on Figures 6.14, 6.16a. The designer should be aware that the efficiency of this type of crest improves as the ratio of the depth of flow to the crest width increases. The discharge coefficient is affected by the approach conditions to the crest and the tailwater conditions below the crest.
- Sharp Crested Weir Design — A sharp crested weir can be constructed as an extension of an upstream cut-off wall as shown on Figures 6.16b and 6.18. The sharp crested weir can significantly increase the efficiency of the spillway and can be designed with minimal effect on the efficient placement of the RCC as shown in Figure 6.17.
- Ogee Crest and other curved crest designs — The ogee shape design is a highly efficient crest section (Brater and King 1976) as shown in Figure 6.16c. Refer to the "Design of Small Dams" (USBR 1987a) for a discussion on ogee crest spillways. Modified crest designs (flat curves, etc.), as shown in Figures 6.16d, and 6.19

Figure 6.14. Broad crest weir construction of RCC (Saklado Dam No. 10, Texas).

Figure 6.15. Installing reinforcement steel for modified ogee crest weir (Tellico Dam, Tennessee).

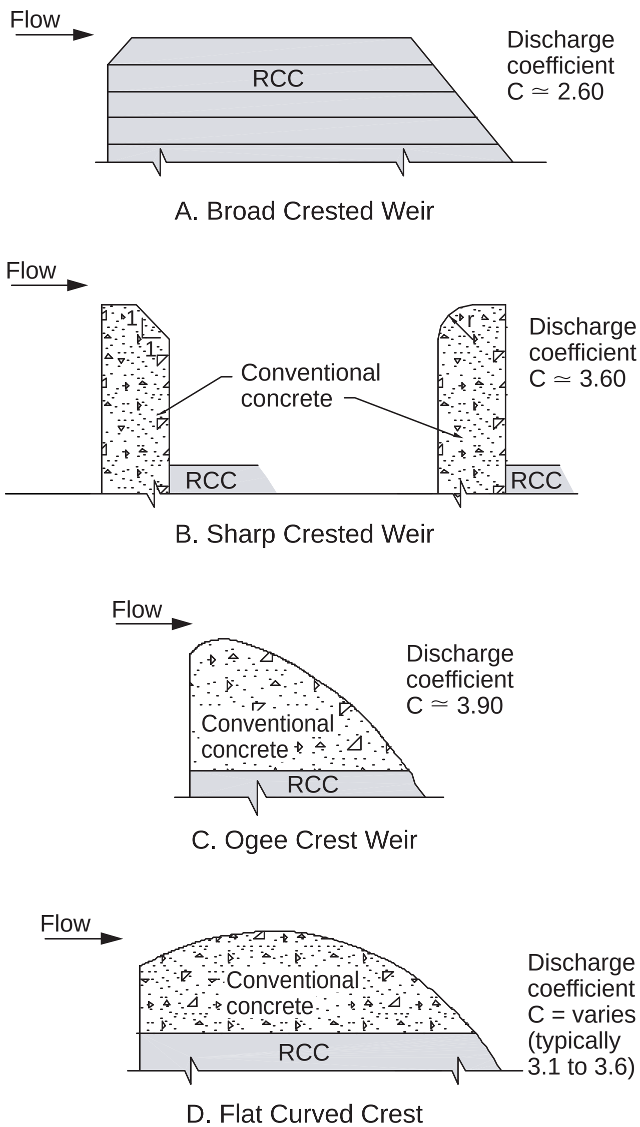


Figure 6.16. Alternative weir crest shapes.

Figure 6.17. Sharp crest weir wall under construction (Smith Lake Dam, Virginia).

Figure 6.18. Sharp crest weir and with completed RCC overtopping section (Smith Lake Dam, Virginia).

Figure 6.19. Sloping ogee crest weir constructed with conventional concrete (Windmill Wash, Nevada).

Figure 6.20a. Modified ogee crest weir (Coal Ridge, Colorado).

through 6.21, can be used which can have improved crest efficiency when compared to broad crested weirs. Engineering Monograph No. 9 (USBR 1952) can be used as a design reference for non-standard curved crests.

6.7 APPROACH APRON 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 increase the under-

Figure 6.20b. Modified ogee crest weir. RCC encapsulated by conventional concrete (Smith Lake Dam, Virginia).

Figure 6.21a. Modified ogee crest weir constructed of unreinforced concrete placed in segments (Blue Diamond, Nevada).

Figure 6.21b. Completed ogee crest weir (Tellico Dam, Tennessee).

seepage path, and reduce the seepage that could occur from the reservoir under the spillway chute (Figure 6.22).

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 or excessive seepage from occurring through the dam, under the apron slab and crest section, and under the sloped RCC chute. An upstream cut-off wall is an important design feature for increasing the seepage path under the approach apron and also to prevent erosion of the upstream edge of the RCC apron. Design of the cut-off 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 cut-off 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 two feet, the designer should consider increasing the minimum thickness or installing a gravel underdrain beneath the apron.

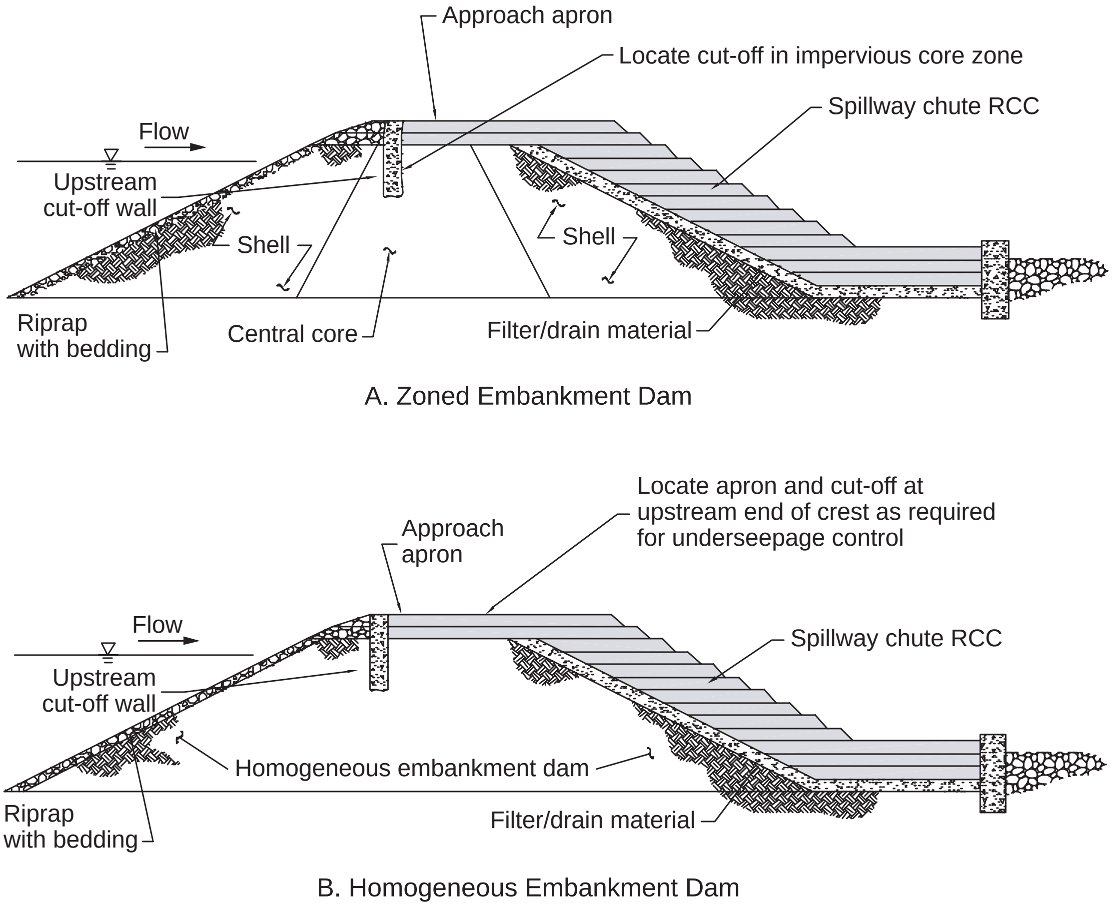


Figure 6.22. Approach aprons.

6.8 DOWNSTREAM APRON SLAB

General

The primary function of the downstream apron is to protect the RCC spillway and the dam embankment from erosion caused by spillway flow. The length and thickness of the downstream apron depends upon energy dissipation and erosion control features of the energy dissipator design.

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. The apron can also be located at adequate depth below tailwater, and with adequate length, so that a hydraulic jump will form on the apron.

The designer must also determine the erosion potential of the soil or rock downstream of the apron. The estimated depth of erosion and channel degradation can then 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. 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 modeling may be required.

Examples of the downstream run-out apron, and an alternative stilling basin, are shown in Figures 6.23 and 6.24, respectively.

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

Figure 6.23. Run-out apron, end sill and riprap transition section under construction (Smith Lake Dam, Virginia).

Figure 6.24. Stilling basin constructed of RCC (Lake Tholocco Dam, Alabama).

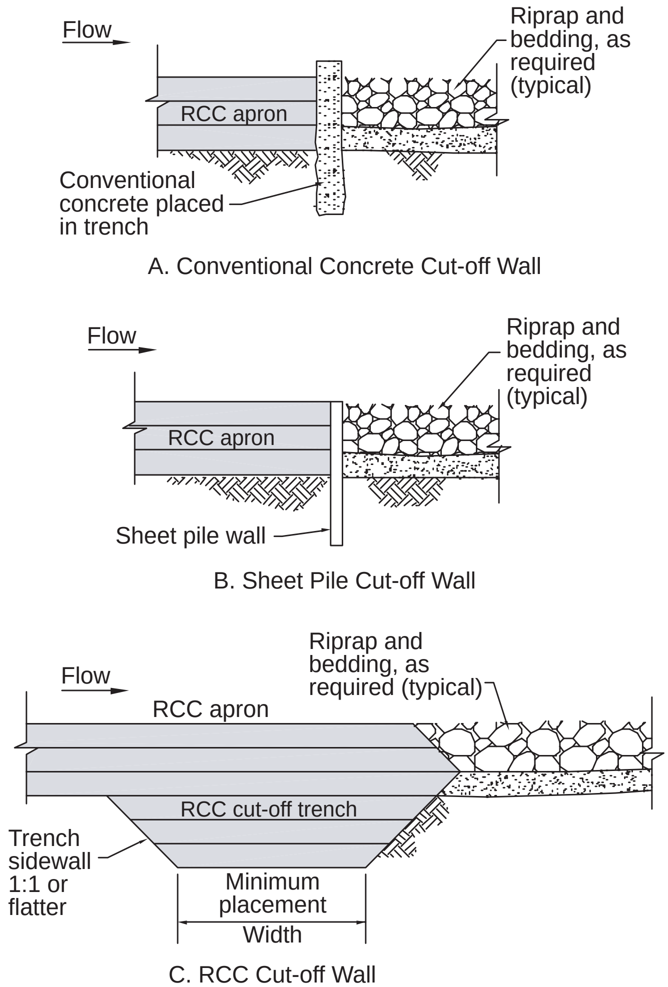


Figure 6.25. Cut-off wall details.

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 three feet should be considered as a minimum thickness for most projects.

6.9 CUT-OFF WALLS

Cut-off walls are typically located at the upstream and downstream ends of the RCC spillway as shown in Figures 6.2 and 6.25. 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 degradation. The depth of the downstream cut-off wall should extend to a competent bedrock, or to below the estimated depth of erosion that could occur from the spillway design flow. 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. Cut-off walls can be constructed of conventional concrete, RCC, or steel sheet piling.

Conventional Concrete Cut-off Walls

Cut-off walls can be designed as non-structural elements constructed by excavating a trench and backfilling the trench with concrete. 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 concrete walls. Formed wall construction requires a larger excavation than trenched wall construction because of the excavation required for the installation of the form work. A formed wall design will require that

Figure 6.26. Overtopping spillway — upstream cut-off wall during concrete placement.

the excavated slopes be laid back as required for trench safety, and then backfilled and compacted to grade. Construction of a typical upstream concrete wall is shown in Figure 6.26.

Sheet Pile Cut-off Wall

Sheet piling can be used to construct upstream or downstream cut-off walls (see Figure 6.27). Some advantages of sheet piling are that trench excavation, dewatering, and placement of compacted fill in the trench are not required.

Figure 6.27. Steel sheet pile for downstream cut-off wall (McKinney Lake, North Carolina).

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.

RCC Cut-off Wall

Construction of RCC cut-off walls require a larger trench excavation than conventional concrete cut-off walls because of the minimum width requirements for placing and compacting the RCC (see Figure 6.25c.) 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 conventional concrete would not otherwise be required.

RCC can also be placed over the entire crest of the dam and extend down the upstream face of the dam. This design serves as a cut-off wall as well as minimizing 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 dissipater and shorten the apron length. The designer is cautioned that if a hydraulic jump type energy dissipator feature is used, adequate tailwater will be required for these features to function as designed. If chute or impact blocks are used, "capping" the apron with a conventional concrete slab to expedite construction of the blocks can also be considered. The end sill can easily be incorporated with a conventional concrete or RCC cut-off wall (Figure 6.28). 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 succeeding 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 to the design of horizontal RCC lifts is to require that a bedding mix be used between each lift of the approach apron slab and the downstream apron slab, and treated with a bedding mix. Bond should also be developed between the sloped chute lift joints. With proper curing and maintaining a clean lift surface during construction, bonding of lift surfaces usually occurs. However, horizontal lift joint treatment in the spillway chute section is less standardized and the extent of lift bond has not been documented in overtopping structures.

An important part of the design of the spillway chute is obtaining a large monolithic mass. This serves the purpose of providing few paths for water to seep beneath the chute during spillway flows, and provides 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 joint 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 with the exception of one project in the southwest (apparent delamination occurred between the top two lifts of an in-stream grade control structure). Chapter 9.0 describes procedures for joint treatment. A second approach would be to require bedding mix concrete between each successive lift.

Factors favoring treatment of joint surfaces between succeeding 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 designed drainage blanket and drainage system beneath the RCC chute.

The decision to require bond on cold joint lift surfaces of RCC spillways is, at present, dependent upon project requirements and the engineering judgment. The minimum joint treatment recommended at this time would be: (1) placement of a bedding mix on joint surfaces more than 24 hours old, and between each lift of the approach apron and downstream apron; (2) cleaning of the surface using compressed air prior to placement of a succeeding lift; (3) removal of contaminants, laitance, damaged RCC or RCC that is not properly cured; and (4) evaluate the need to provide a bedding mix on joints that are more than 12 hours old.

Construction Joints

Construction joints for RCC overtopping protection 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 to fully compacted RCC, clean the joints to expose the coarse aggregate, and to place a bedding concrete on the joint surface prior to the placement of fresh RCC. Transverse construction joints locations should be documented, and joints should be staggered by at least 20 feet longitudinally when transverse joints are required in succeeding lifts. Methods for joint treatment are discussed in Chapter 9.

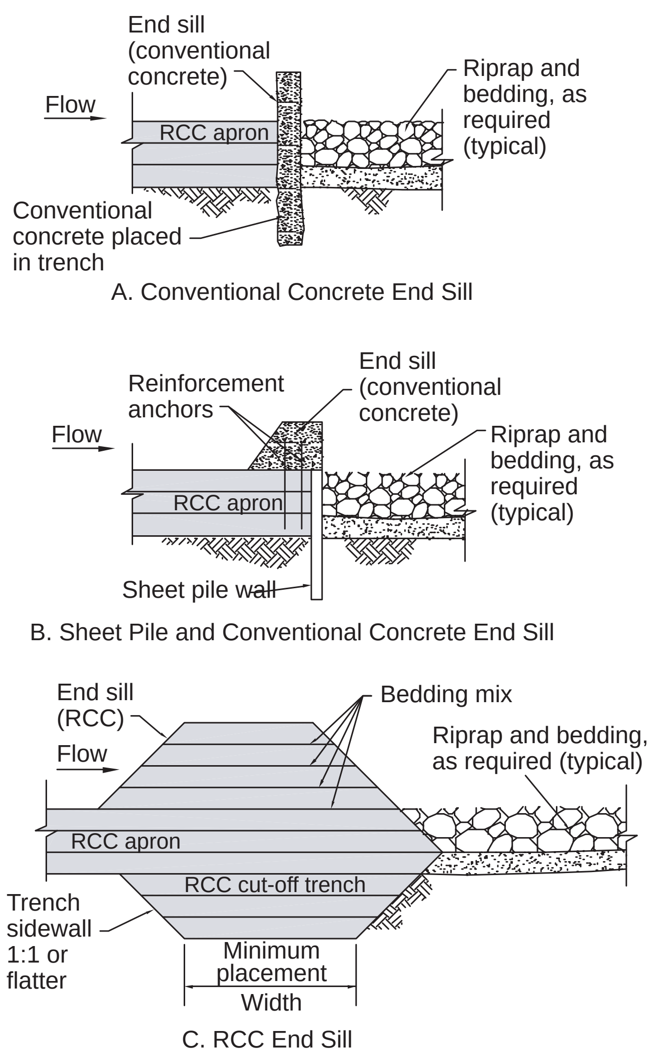


Figure 6.28. Alternative end sill details.

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. Most RCC overtopping projects have not been designed using contraction joints and have been allowed to crack freely (see Figure 6.29). Performance histories have not been compiled on the effectiveness of using contraction joints.

Spacing between contraction joints should be determined based upon the exposure conditions of the project and performance of other similar projects. Where contraction joints have been constructed for RCC overtopping projects, transverse (upstream:downstream) joints have been installed. Typical spacing of contraction joints has been from 100 to 300 feet.

Longitudinal (abutment to abutment) joints have typically not been installed. Longitudinal contraction joints 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 fairly straight contraction 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. A typical section at a contraction joint is shown in Figures 6.30 and 6.31. Methods used to construct contraction joints are discussed in Chapter 10.

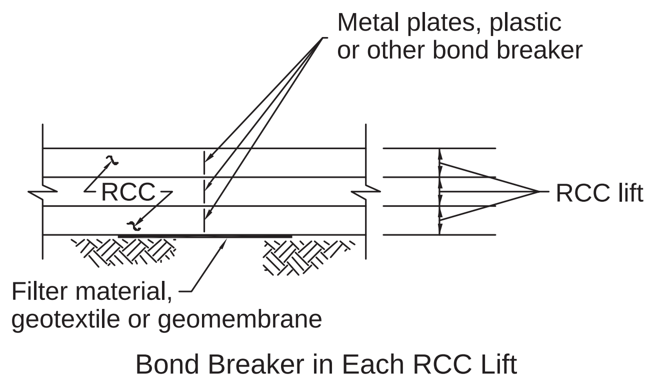


Figure 6.30. Typical section at contraction joint.

Figure 6.29. Naturally occurring shrinkage crack in RCC.

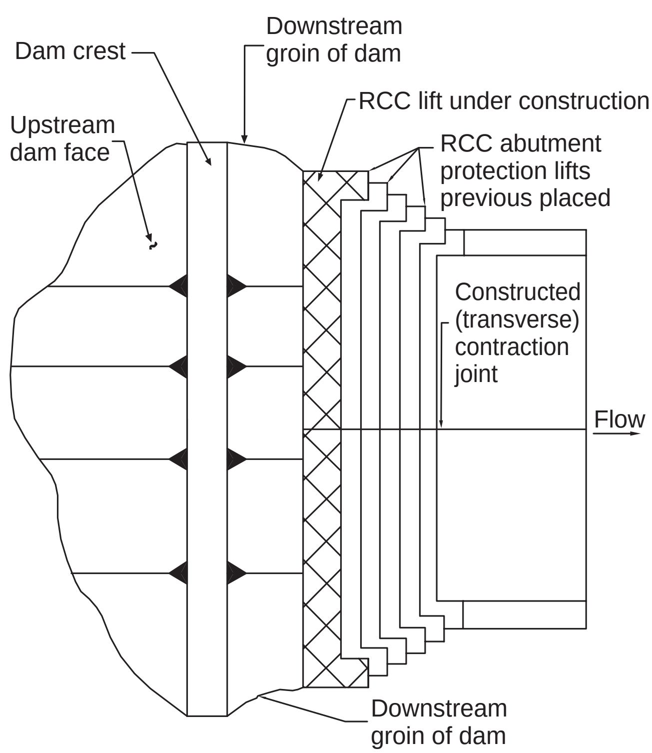


Figure 6.31. Plan of RCC overtopping and abutment protection partially constructed.

6.11 DRAIN OUTLETS

Drainage is installed beneath the RCC chute (see Figures 6.32 and 6.33). Underdrainage piping is sometimes required for the filter and drainage system for spillways. If underdrains are included as part of the design, methods for cleaning and maintaining the system should be provided. Drain outlets can range from pipes daylighting through the RCC steps (Figure 6.34) to substantial concrete channels (Figure 6.35). 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.

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. Reverse flow can cause excessive uplift pressure on the RCC spillway slab. Providing two access points to drain lines can facilitate inspection and maintenance activities.

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.

Figure 6.34. Pipe outlet.

Figure 6.32. Trench drain construction prior to RCC placement (McBride Dam, Ohio).

Figure 6.35. Concrete drain outlet structure (South Prong Dam, Texas).

Figure 6.33. Blanket drain construction prior to RCC placement (Douthat Dam, Virginia).

6.12 TRAINING WALLS AND ABUTMENT PROTECTION

General

Spillway flow training walls contain spillway flow within the RCC chute and protect the dam and abutments from potential erosion. Overtopping of the spillway walls or the abutment protection can result in 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 6.36, 6.37, 6.38, and 6.39.

Figure 6.36. RCC training wall constructed using vertical forms (Salado No. 10, Texas).

Figure 6.37. RCC training walls constructed using formed sloping RCC wall (Anthem No. 2, Nevada).

Figure 6.38. RCC training wall (Kyle Canyon Dam, Nevada).

Figure 6.39. RCC training wall in background (Black Rock Dam, New Mexico).

Abutment Protection

Abutment protection is required for all overtopping designs (Figure 6.1). 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. 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. The potential could lead to dam failure.

The hydraulic analysis of flow depth and velocity for the overtopping spillway design, should provide a design that protects the abutments from erosion and safely convey the flow away from the dam. 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 hydraulic model studies. 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

Flow training walls are constructed along the RCC chute to contain the spillway flow. 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 width for the length of the spillway, parallel to the flow direction, or they can be designed to contract (converge) from the spillway crest to the base of the spillway. The geometry of both configurations are shown in Figure 6.40. An example of a overtopping section with conventional concrete training walls is shown in Figures 6.41 and 6.42.

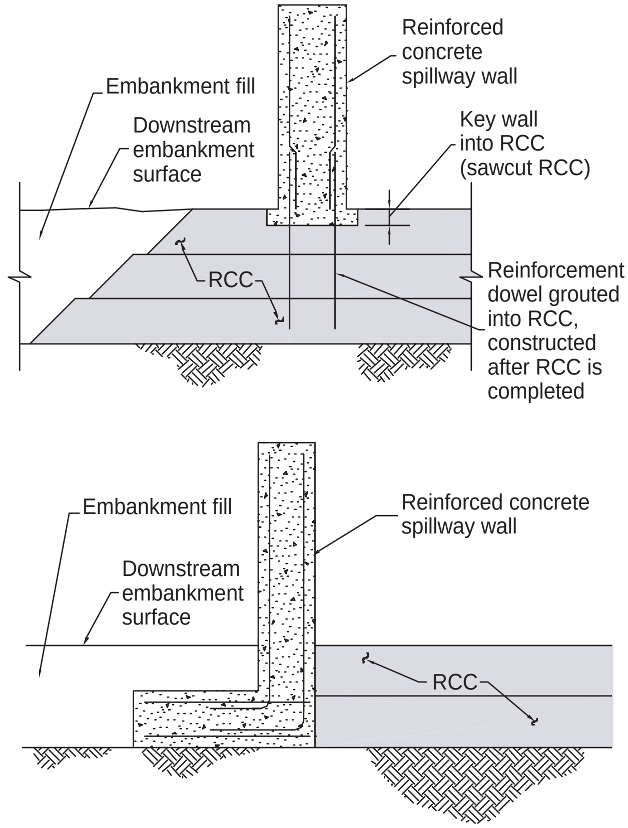


Figure 6.41. Reinforced concrete training wall (section looking downstream).

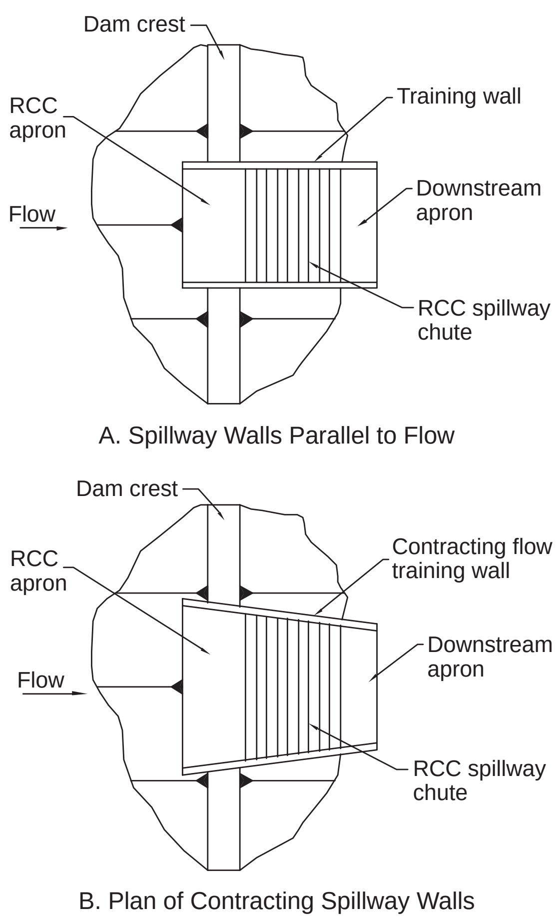


Figure 6.40. Spillway flow training walls.

Figure 6.42. Conventional concrete training wall being constructed on completed RCC (Leyden Dam, Colorado).

Flow training walls constructed on the downstream face can mitigate the need for abutment groin protection.

The height of the flow training walls is 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) and "Hydraulic Design of

Spillways" (USACE 1990). The designer should be aware that RCC spillway surfaces are typically rougher than conventional concrete chutes, and bulking of flow due to greater air entrained in the flow must be considered in determining the depth of flow.

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, the reader is referred to references described above. 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, as shown in Figures 6.31, and 6.36 through 6.39. A benefit of structural concrete 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 structural concrete training walls if the spillway width is narrow.

6.13 SOIL COVER FOR RCC SPILLWAYS

A number of RCC spillways have been covered with soil and grass (see Figure 6.43a and 6.43b). 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. Freeze-thaw protection of the RCC can also be a consideration in wet climates subject to freeze-thaw conditions. The typical soil cover thickness has been about 2 feet.

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 at 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 cover may occur due to concentrated runoff from precipitation, developing erosion channels in the soil cover down to the RCC.
- Seepage outlet drains must extend through the soil cover and large quantities of seepage can cause erosion of the soil cover.

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.

Figure 6.43a. RCC spillway before soil cover is placed (Philipsburg Dam 3, Pennsylvania).

Figure 6.43b. RCC spillway after soil cover is placed (Philipsburg Dam 3, Pennsylvania).

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 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 (compactability) of a mix. The compressive stresses in an RCC overtopping spillway structure are typically low. However, compressive strength is generally specified as an indirect indicator of the durability of the RCC mix. The workability of a mix (such as slump in conventional concrete) is also important in obtaining the desired in-place properties. A uniform measure of workability of RCC, which by definition is a no-slump concrete, has not yet been established. But there is sufficient information to provide the necessary guidance for RCC applications. We have also seen a third property that should be considered in developing RCC mixes — uniformity. Mix designs, and more importantly developing criteria for specifying mix designs, are discussed below.

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 concrete. 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 concrete 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 concrete mix proportioning procedures. However, since RCC is spread and compacted with earthmoving equipment (dozers, surface compactors, etc.), conventional concrete 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. In contrast, fill control for earthwork placement is generally performed using a compaction test (such as ASTM D 1557). A good correlation has been developed between compaction test properties and surface compactors.

The compaction test establishes the workability (compactability) 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 conventional concrete and consequently can have a higher permeability, and lower durability and compressive strength, relatively, than conventional concrete. 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 concrete 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 D 1557) process or conventional concrete 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 percent above optimum moisture content (ASTM D 1557).

7.2 SOIL COMPACTION METHOD OF MIX DESIGN

In general, the ASTM D 1557 test (modified Proctor compaction test) procedure is typically used for fill control of granular material. The standard test procedure involves compaction of the minus 3/4-inch fraction of the fill material in a 6-inch diameter by 4.584-inch high steel mold. The material is placed in 5 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 a placement moisture content is specified based on the compaction characteristics. In granular fill, such as roadbase, fill placement requirements are then specified. A typical specification for placement of roadbase would include the following requirements:

- Maximum loose lift thickness: 8 inches
- Placement moisture: optimum moisture plus or minus 2 percent
- Compaction: Minimum of 95 percent 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 for granular fill control. In addition, the void content of RCC compacted at optimum moisture is higher than conventional concrete due to entrapped air content. Historically, conventional concrete has shown that with an air void content of 5 percent due to incomplete consolidation, loss of strength (as much as 30 percent) 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 percent 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.

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 D 1557). 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 percent above optimum.

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), and that will provide the required strength and durability. 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
- Aggregate quality requirements
- Cement type and pozzolan (if used) and properties
- Aggregate gradation

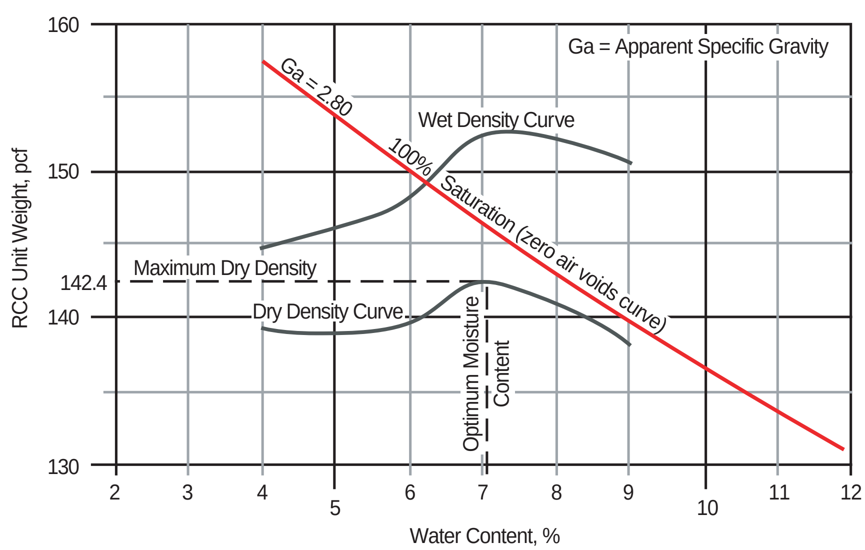


Figure 7.1. Typical RCC moisture-density relationship (ASTM D 1557).

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 protection, a 1 inch maximum size is preferred for RCC overtopping structures. Aggregate is selected that fits a design grading band (such as the examples shown in Figure 7.2) or using conventional concrete mix design procedures for combining fine and coarse aggregate contents (as described in the next section). Aggregate from several completed projects is 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

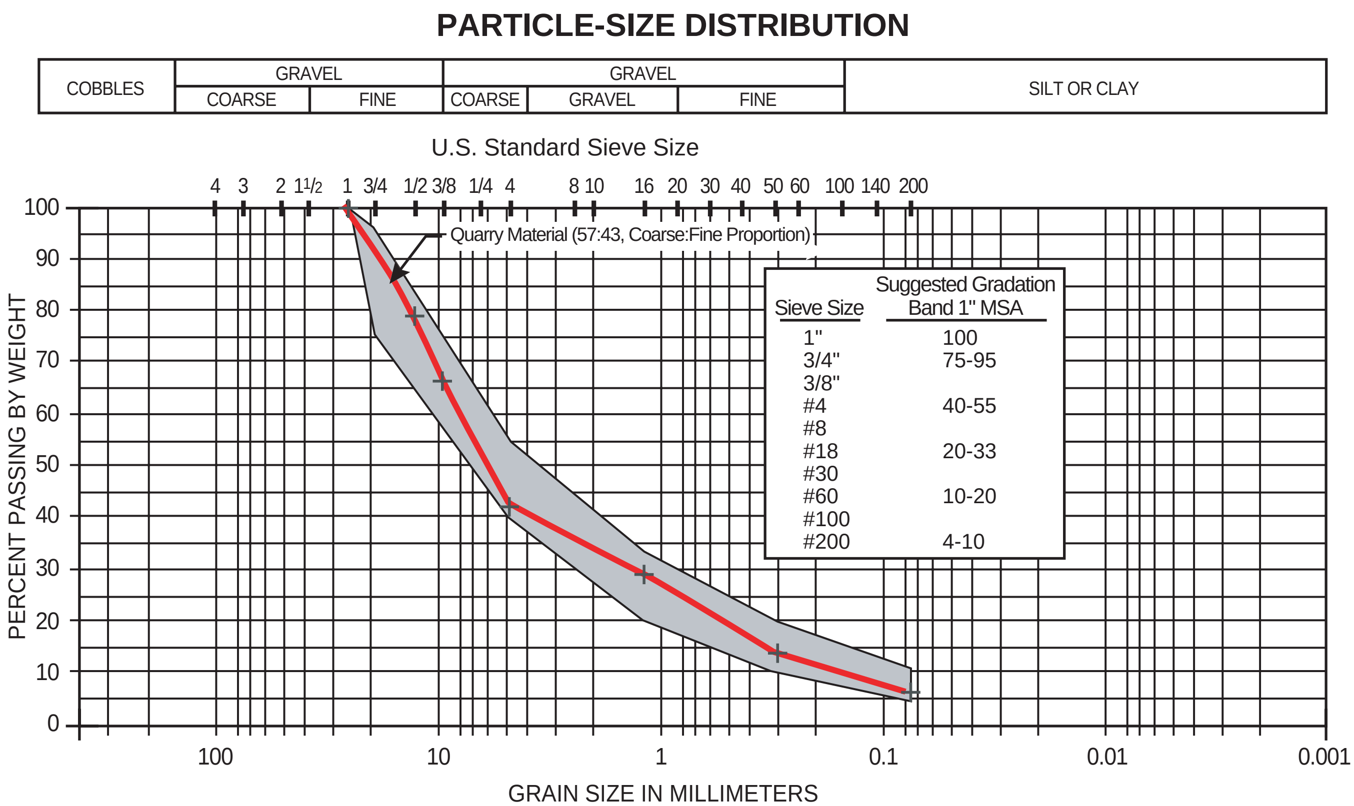


Figure 7.2. RCC aggregate design gradation bands.

Table 7.1. RCC Gradations

Sieve Size	Fawell Dam, Illinois	Mona Dam, Utah	Smith Lake Dam, Virginia	Leyden Dam, Colorado	Lake Tholocco Dam, Alabama	Saddle Dam, Indiana
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	350-60
#8			35	28	38	25-50
#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

critically saturated, even in severe climates, but may be susceptible to freeze-thaw damage when critically saturated. Efforts to entrain air in RCC has to date met with debatable success. In particular, air entrained RCC in field production has not been consistently produced. The current practice for design of RCC for overtopping protection is to specify a minimum compressive strength of 3,000 psi, and a maximum water: cementitious ratio 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 conventional air-entrained concrete in areas of moderate to 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 will 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, use 400 pounds per yd^3 of cement).

Step 4.

Develop the modified Proctor compaction curve (ASTM D 1557) using the RCC mix at the mid-range cement content. If the aggregate contains material greater than ¾-inch, the ASTM D 1557 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 RCC of 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 D 2216). 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 percent increments, and allow the samples to "season" for 24 hours in sealed containers without cement. The cement is then manually mixed in to 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 D 2216 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 percent above optimum water 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 C 1435, and other testing deemed appropriate, for the mid-range cement content for compression testing over the range of design age with two or 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 C 138 or C 231. The entrapped air content should range between 1 and 2 percent. If the entrapped air content is higher than about 3 percent, 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 at different cement contents using ASTM C 1435. Vary the cement content by increments of 10 to 15 percent, above and below the mid-range cement content.

Step 8.

Prepare a semi-log 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 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 conventional concrete.

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 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 condition (i.e., weight of aggregate in a saturated surface dry 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 saturated surface dry condition]). It is critical that the engineer understands how the plant operate so that the design mix proportions can be correctly conveyed to the contractor. At a minimum, constituents to be provided are:

- Cement type, and pozzolan (if used), content
- Aggregate content (SSD) (coarse and fine)
- Water content (above SSD)
- Total water content
- Specific gravity (SSD)
- Absorption
- Air content
- Water:Cement + Fly ash 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.

Mix design calculations for an Example Project are shown in Appendix A.

7.3 CONVENTIONAL CONCRETE METHOD OF MIX DESIGN

Conventional concrete mix proportioning looks at achieving a gradation that can be densely consolidated with internal vibrators. Workability for conventional concrete is usually measured by a slump test (ASTM C 143). However, the slump test is not a suitable method for indicating workability (compactability) 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 C 1170) is commonly referred to as the Vebe test.

Mixture proportioning using the conventional concrete 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, Mixture Proportioning." Proportioning methods include both weight and absolute volume methods. The absolute volume method is more accurate and is the method summarized below.

RCC is designed with a consistency that is sufficiently stiff to support vibrating rollers. A key to the design of RCC mixtures using conventional concrete 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 and pozzolan (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 recommended for the RCC overtopping section design. Compare gradings of combined coarse and fine aggregate with other gradings such as Tables 3.1 and 3.2 (USACE EM 1110-2-2006) (see Appendix A).

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 3.3 (EM 1110-2-2006) (see Appendix A) and select the required cement content from Figure 3.2 (EM 1110-2-2006) (see Appendix A) 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 percent).

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 3.3 (EM 1110–2-2006).

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 air (USACE EM 1110-2-2006, Table 3.3) (see 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 C 1170 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" significantly 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. A good target Vebe time would be between 25 and 35 seconds.

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 C 1176 or C 1435 (two or three cylinders for each age are tested). Vary the cement content by increments of between 10 and 15 percent 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 C 138 or C 231, and compare with the 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 C 1176) or by an electric hammer (ASTM C 1435). The entrapped air content should range between 1 and 2 percent.

Step 11.

Prepare a semi-log 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 conventional concrete.

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, and pozzolan (if used), content
- Aggregate content (SSD) (coarse and fine)
- Water content (above SSD)
- Total water content
- Specific gravity (SSD)
- Absorption
- Air content
- Water:Cement + Fly ash 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 required during production placement.

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

CHAPTER 8

Instrumentation and Monitoring

The construction of overtopping projects essentially entails placing an RCC layer on the 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.
- Permanent survey points to monitor embankment movement.
- Water level gauges or piezometers to monitor the internal phreatic level and reservoir level.

CHAPTER 9
Construction Considerations

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

9.1 CONSTRUCTION ACCESS/SITE LAYOUT

In design of an RCC overtopping project, site access issues must be addressed. Site access, location, and layout will greatly influence the contractor's ability to successfully complete construction of a project. Some of the issues that should be considered in the planning and preparation of the construction documents are as follows:

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. Often, because of limited site 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. 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 fly ash if used) at the site. This entails locating cement (and fly ash) silos adjacent to the plant and providing sufficient room for transport trucks to maneuver and off load the materials and cement (and fly ash).

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 deal with varying traffic impacts in urban areas. The lack of on-site space for material storage will increase the construction cost of an RCC overtopping project.

Waste Areas – Typically, an RCC overtopping project 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 is utilized where possible.

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.

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 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 requirements of ASTM C 94. Sediment contained in water used for curing can cause staining of RCC and affect the aesthetics of a completed project.

Dams in Urban Areas – Constructing an overtopping project in an urban area adds some complications that are less of a concern at more remote sites. These issues include:

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 dam. 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.

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 gravely 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 contractors 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 and ditches provide suitable groundwater control. A typical dewatering sump 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 typically removed, or a concrete mud slab is 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 layer, 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 vacuum type equipment as shown in Figure 9.2, and/or compressed air or compressed air combined with water. Bedding mortar is often applied

Figure 9.2. Foundation preparation and cleaning using a vacuum device.

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.

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 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 700 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, 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 mobil plant, similar to a Con-E-Co all-pro batch plant, and mixed in transit ready-mix trucks. This method has a relatively low production capacity, and control of the uniformity of the RCC can be difficult.

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 fly ash), and degree of segregation will vary throughout the batch.

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 conventional concrete.
- Build up of hardened RCC is more rapid and requires more labor to maintain than observed for conventional concrete.
- The mixing time to obtain uniform RCC is typically greater than that for conventional concrete using a drum mixer.
- With conventional concrete, 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 of 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 nonstandard issues or due to commitments to ongoing customers.
- Type II and Type I/II cement, and occasionally fly ash, is typically specified for overtopping projects. The type of cement and fly ash can vary regionally, and also can vary over time as the product from powerplant varies. Project costs can be affected if a separate silo is required to provide a cement or fly ash that differs from the typical usage at an existing ready-mix plant.

RCC Mix Temperature – The temperature of RCC for overtopping projects is usually similar to conventional concrete requirements. Unlike mass RCC gravity dam projects, where temperature control is critical to control cracking, overtopping projects are 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. 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 concrete.

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 contractors 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 – Motorized vehicle systems, including 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. They have a long history of satisfactory usage on soil cement used for upstream slope protection on dams and channel protection. 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.

Care must be taken to limit segregation of the RCC mix when motorized vehicle systems are used. Modification of the truckbed 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. 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 RCC delivery process as shown in Figures 9.3 and 9.4. Truck beds, spreader

Figure 9.4. RCC delivery with a long stick track hoe (Hayes Dam, South Dakota).

boxes, and skid boxes that are used to haul or temporarily hold RCC will usually experience build up of dried RCC. Equipment used for hauling, conveying, and spreading RCC will require periodic cleaning to keep the dried RCC from contaminating the placement.

Conveyor Systems – Conveyor systems are often used when RCC is placed in steep valleys where access to the site 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 9.5. Long placement areas typically require long, multi-conveyor systems or frequent equipment moves. Conveyors can typically operate at slopes of 30 plus or minus degrees. Therefore, multiple conveyor segments with transfer points can be required for delivery over the full height of the structure. An alternate conveyor

Figure 9.3. RCC delivery with a track hoe (Fawell Dam, Illinois).

Figure 9.5. RCC delivery using conveyor system (Blue Diamond Dam, Nevada).

Figure 9.6. RCC delivery using conveyors and mobile super swinger (Black Rock Dam, New Mexico).

Figure 9.7. RCC spreading and compaction (Lake Tholocco Dam, Alabama).

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 (see Figure 9.6).

Conveyor/Motorized Vehicle Systems – Often times the site geometry will dictate that a combination of a conveyor and wheeled 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, to transport the RCC to the placement area.

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 on uncompacted rather than compacted RCC or placed in windrows by an elephant trunk 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.

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 9.7.

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 will result. Often thin lifts, 6-inches or less, and hand placement (Figure 9.8) are required to eliminate segregation and the formation of rock pockets in formed RCC faces. Use of a smaller, maximum size aggregate (such as 1 inch) and/or a "wetter" RCC mixture can also reduce the segregation potential.

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 dozer. An optimal size dozer for an RCC overtopping project is usually equivalent to a Caterpillar Model D4, John Deere Model 5503 or Case Model 850 track-type dozer. Dozers larger than these sizes tend to be too large for the work area available for most overtopping protection projects.

Other types of machinery that have been used for the spreading of RCC include dozer-mounted spreader boxes and paving machines. Hydraulic excavators, backhoes, and

Figure 9.8. Hand compaction of RCC adjacent to step form.

loaders are also used with hand labor to spread RCC in tight areas not accessible to larger spreading equipment.

When using track-type dozers 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 overtopping project should consider in preparing construction documents. The lift thickness can best be controlled by the use of laser leveling techniques, whether hand operated or mounted to the dozer blade. The lift geometry can be controlled using a string line such as that used in common concrete and earthwork construction (see Figure 9.9).

Figure 9.9. Compacting downstream RCC slope (note string line for alignment control).

9.6 COMPACTION OF RCC

Equipment – Several sizes of compactors should be specified for an RCC overtopping 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 specified 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 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 two feet from the face of forms because the weight and compactive 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 400 pounds per linear inch of drum width
  - Adjustable frequency with a minimum frequency of 1,700 vibrations per minute
  - Amplitude of 0.025 to 0.035 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 and produce 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, see Figure 9.8, 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, see Figure 9.19. However, they are unable to effectively compact RCC deeper than about 3 inches. The typical requirements for reversible vibratory plates are that they have a minimum gross weight of 1,000 pounds and a minimum centrifugal force of 12,000 pounds.

Figure 9.10. Compacting downstream RCC face with track hoe mounted hydra-plate compactor.

- Compaction with vibratory plates using manual labor or an attachment to a hoe or dozer, has been used to obtain satisfactory compaction of the exposed downstream RCC face as shown in Figures 9.9 and 9.10.

Table 9.1 lists typical effective compaction depths in RCC for various types of compaction equipment.

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, 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 fly ash contents of the mix, the mix moisture content, and the water to cement ratio of the mix.

When RCC Doesn't Make Compaction – With quality control of the aggregate, cement, 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
- 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
  - Aggregate gradation has changed, either from a change in the material source or improper loading of segregated aggregates from the stockpiles
- No cement, or a deficient quantity of cement or fly ash in the RCC mix
- Segregation of the mix during transport and/or placement

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 Bowmag 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

The alternatives available when RCC does not make the desired compaction are 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 may be 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 concrete-like finish on the RCC surface, it is usually necessary to have a mix with a higher moisture and paste content. During compaction, the paste will tend to work to the lift surface. A problem with using a wet RCC mix is that the roller will 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, concrete finish. 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. A typical good quality surface texture is shown in Figure 9.11. If a uniform final surface is required, a concrete paving machine (with tamping bars) should be considered for placing the RCC. Also, MSA of the RCC should not exceed 3/4-inch.

The mixing time during the batching of RCC can be important in controlling the appearance of the final lift surface. When the moisture content in the mix varies, i.e., 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.

Figure 9.11. Appearance of a "good" RCC finish surface.

9.7 CURING OF RCC AND EFFECTS OF CLIMATE

Like conventional concrete, RCC must be properly cured and protected from 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 concrete curing procedures and protection from climatic conditions are also applicable to RCC overtopping 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 should 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 water to cement 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 debonding layer if it is not removed before the next 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.

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 damage 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 to conventional concrete 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°F. 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 overtopping 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.

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 6.0, there are several types of downstream facing available for overtopping projects, each of which has their own merits. The different types of downstream facing can be categorized as: (1) unformed, uncompacted RCC, (2) unformed, compacted RCC, and (3) formed RCC. 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 (see Figures 6.5 and 6.6) 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.

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 above, with the exception that the exposed RCC face is compacted (see Figures 6.7, 6.8, 9.9, and 9.10) with a tampor 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, rock pockets will remain in the compacted face. This system is aesthetically more attractive than the unformed, uncompacted step system, with some increase in cost.

Formed RCC Steps – The construction procedures for this type of facing system involves 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 includes:

- A "wet," more workable RCC mix, 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 tamper (thin lifts ± 6 inches are sometimes required) adjacent to formwork to reduce the potential for the formation of rock pockets and/or deformation of the forms.
- 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 consolidating 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 9.12, 9.13, and 9.14, respectively.

Formed Conventional Concrete Steps – Formed, conventional concrete steps have been included in the design and construction of RCC gravity dams. Of the facing systems described above, 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 conventional concrete. The most important detail the designer must address is the construction of the interface of the RCC and the facing concrete.

Figure 9.12. Example of "good" appearance of RCC placed against a vertical form.

Figure 9.13. Example of "fair" appearance of RCC placed against a vertical form.

Figure 9.14. Example of "poor" appearance of RCC placed against a vertical form.

Debate is ongoing as to the proper sequencing of the placement of the RCC and facing interface. Dams have been constructed using the following procedures:

- A stiff-facing concrete is placed adjacent to the formwork and then vibrated in a restrained condition. RCC is spread adjacent to the facing, and the interface of the facing and RCC are vibrated together. The RCC is then compacted with a large drum vibratory roller.
- Alternately, the RCC is spread and a trough left adjacent to the forms. Facing concrete is placed in the trough and the RCC facing interface is vibrated together using immersion vibrators. The RCC is then compacted.

Both construction procedures have been used successfully for the purpose of construction of downstream steps on gravity dams.

This method has not typically been used on the flatter slopes of RCC overtopping protection projects.

Formed, Grout Enriched RCC Steps – This concept for constructing facing for RCC was generally developed in China, and is relatively new to the U.S. Aesthetically, this construction method is similar to formed, conventional concrete steps. 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 cement grout is then applied to the surface of the uncompacted RCC.
4. Immersion vibrators are then used to mobilize the grout and RCC into a fluid, concrete mixture.

Test trials are needed to refine the construction procedures and to determine the quantity of cement paste per linear foot of facing to add to the uncompacted RCC, to produce the desired engineering properties.

A couple of alternative "grout enriched" placement methods that have shown some promise in a test section trial 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.

9.9 CONTROL JOINTS

The purpose of control joints is to control the location of the 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 metal plates driven into or buried in the lift (see Figures 9.15 and 9.16)
- Plastic sheeting buried into the lift as shown in Figure 9.17
- Saw cuts
- Monolithic construction with bond breakers

The decision as to the type of material to use for construction of a control 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 view point. 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 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.

Figure 9.15. Installation of control joint plate.

Figure 9.16. Control joint plate placed during RCC placing.

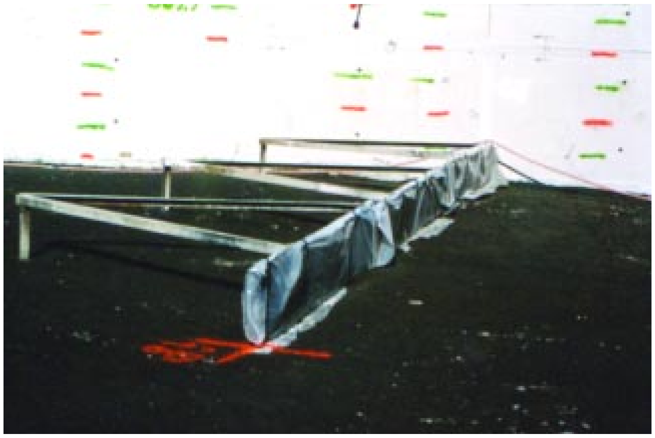


Figure 9.17. Control joint form using plastic sheeting.

Construction control joints should line up vertically in an overtopping section 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.

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) 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 treatment 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. Examples of cold joint preparation and a well prepared joint surface are shown in Figures 9.18 and 9.19.

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 concrete structures (spillway walls, outlet works conduits, etc.)

Figure 9.18. Cold joint preparation.

Figure 9.19. Example of well prepared RCC cold joint surface.

Typically, bedding mortar is a mixture of sand, cement, water, and set retarder. Occasionally, fly ash and air entrainment are added to the mix to improve its workability and reduce segregation. Bedding mortar is spread in thin lifts of about 1/4 to 1/2 inch in thickness. The maximum aggregate size is typically about 3/8-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 9.20. When small quantities of bedding mortar are required, contractors often will utilize small 1/4 yd^3 mixers.

Figure 9.20. Bedding mortar placement.

9.12 LIFT TREATMENT

RCC construction is composed 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 should be implemented: (1) for lift surfaces more than 24 hours old, and (2) between the top three lifts of the overtopping section. Joint treatment for lift surfaces less than 24 hours old is still an area of designer preference, without an obvious standard at this time.

9.13 CONSTRUCTION JOINTS AT WORK STOPPAGES

It is rare that an RCC overtopping 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 to 1 vertical. 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 lanes are typically offset by a distance of 15 to 20 feet to prevent the establishment of preferential seepage paths in the structure.

9.14 CONSTRUCTION OF TRANSITION AREAS

RCC overtopping projects typically consist of 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 concrete structures (see Figure 9.21). Some construction considerations the designer should consider in planning an overtopping project are described below.

Figure 9.21. RCC transition area construction.

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 concrete training wall.

When RCC is "sculpted" at the embankment interface, the designer must consider the methods used to place and compact the RCC, including dozers 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 headcutting and backcutting. 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 conventional concrete 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 concrete walls. Conventional concrete 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 should be placed 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 (see Figure 9.2). If the interface is to be "water tight," such as at or near the crest or abutment interface, a greater degree of preparation should be done. 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 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 placed between the RCC and the structure. 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.

9.15 RCC CONSTRUCTION IN CONFINED AREAS

When starting RCC placement for an overtopping project, 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 is to place a conventional concrete 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.

- 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 conventional concrete can be built to begin RCC placement.

Both of these methods have been used successfully in the construction of existing RCC overtopping projects.

9.16 REPAIR OF RCC

Because of the nature of RCC, when first produced, it behaves as a 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. Judgement 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.

Repair of Fresh RCC – The repair of fresh RCC can be necessitated for some 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
- Lack of the proper quantity of cementitious material

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 jack hammer, 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 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. If possible, it is much preferred by both contractor and designer to remove RCC, if necessary, when it is fresh and not hard. 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 Conventional Concrete – Occasionally during the construction of an overtopping project, the contractor will request to be allowed to repair RCC in tight areas by replacing it with conventional concrete. This substitution is generally adequate but the following must be considered when allowing this substitution:

- The compressive strength of the conventional concrete 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 conventional concrete 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 conventional concrete should be the same as those for repairing fresh RCC or old RCC, as stated above.

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 shall 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 3-4 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. In overtopping structures, the prevention of rock pockets are typically not critical to the structural integrity of the repair; 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 above). 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

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APPENDIX A

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 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.

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

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	Left 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

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 a plasticity-index 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 D 698) 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. 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 based on the lowest construction cost.

The maximum water surface is limited to elevation 614 due to land development upstream. This will provide one 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})

$$q_{h6} = 38.2 \text{ cfs/LF}$$

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.

$$q_{h6} = 57.3 \text{ cfs/LF}$$

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.

$$q_{h6} = 52.9 \text{ cfs/LF}$$

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. Therefore, these quantities and cost will not be included for this comparison.

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

1.3 **Calculate the Comparative Cost for the Three Spillway Crest Types:**

1.3a **Broad Crest Weir:**

Total RCC Required	39,030
RCC Cost (per yd^3) (From Fig. A.5)	x $53
Comparative Cost	**$2,068,590**

1.3b **Ogee Crest Weir:**

Total RCC Required (yd^3)	23,060
RCC Cost (per yd^3) (From Fig. A.5)	x $60
Subtotal Cost	$1,383,600
Reinforced Concrete Ogee (yd^3)	928
Est. Reinforced Concrete Cost (per yd^3)	x $350
Subtotal Cost	$324,800
Comparative Cost	**$1,708,400**

1.3c **Sharp Crested Weir:**

Total RCC Required	25,350
RCC Cost (From Figure A.5)	$60
Subtotal Cost	$1,521,000
Conventional Concrete Crest (yd^3)	220
Est. Concrete Cost (per yd^3)	x $250
Subtotal Cost	$55,000
Comparative Cost	**$1,576,000**

Therefore, with other technical and physical conditions being equal, based on the comparative cost, a 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. For this simplified example, the cost of earthwork excavations, drainage system, and minor items, have not been included in the cost estimate.

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/sec^2)

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.6), and V_1 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 a 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 above table 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.

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

	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

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

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:

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

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

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

$$= 0.407$$

Where:

f_c = Darcy friction factor (assume 0.2)

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

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

$$\Delta h = 16.60 \text{ ft}$$

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:

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

where V_0 is the velocity of water at the top of the stepped spillway, and Z_0 is the elevation of the approach channel (see Figure A.4); and V_1 is the flow velocity at the base of the spillway and Z_1 is the elevation of the stilling basin (see Figure A.4). 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,

$$\frac{(11.94)^2}{2(32.2)} + 605.0 = \frac{V_1^2}{2(32.2)} + 567 + 16.60$$

$$V_1^2 = \left[\frac{(11.94)^2}{2(32.2)} + 605.0 - 567 - 16.60\right] 2(32.2)$$

$$= 1520.70$$

$$V_1 = 39.0\ \text{fps}$$

The flow depth at the toe of the spillway d_1 (see Figure A.6), is:

$$d_1 = \frac{q}{V_1} = \frac{52.9}{39.0} = 1.36\ \text{ft}$$

Step 5:

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

$$d_2 = \frac{\pm 1.36}{2} + \sqrt{\frac{(1.36)^2}{4} + \frac{2(39.0)^2 \times 1.36}{32.2}}$$

$$= \frac{\pm 1.36}{2} + \sqrt{0.46 + 128.5}$$

$$d_2 = 10.7\ \text{ft}$$

$$V_2 = \frac{q}{d_2} = \frac{52.9}{10.7} = 4.94\ \text{fps}$$

The conjugate flow depth after the hydraulic jump is calculated to be 10.7 ft. Therefore the bottom of the stilling basin should be set at an elevation 10.7 ft below the tailwater elevation in order to have a fully developed hydraulic jump in the basin. If the tailwater depth is greater that 10.7 feet, the hydraulic jump will become submerged. If the tailwater is less than 10.7 feet 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 578. 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:

$$= \frac{V_1}{\sqrt{gd_1}} = \frac{39.0}{\sqrt{32.2 \times 1.36}}$$

Based on design charts in Design of Small Dams (USBR 1987a) Figure 267, the length of stilling basin is equal to 4.0 d_2. For a depth (d_2) of 10.7 ft., the length of the stilling basin would be approximately 43 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 = 5.64ft

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.6.

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.6) 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) = 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.4.

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 C-39 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 1557 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 D 1557 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 C 1435.

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 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

Constituent	Weight of Material (lbs/ft^3)	Weight of material (lbs/yd^3)	Specific Gravity	Absolute Volume (ft^3/yd^3)
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

Entrapped Air Content (calculated):

27.0 ft^3 – 26.108 = 0.89 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.8)

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.8, and a target/design strength of 3,000 psi at 28 days, specify a cement content of 412 pounds per yd^3.

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

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

$$= 141.1 \text{ lbs} - \frac{412 \text{pcy cement}}{27 \text{ ft}^3/\text{yd}^3} = 125.8 \text{ lbs}$$

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 ft^3/yd^3 = 1936.2 pcy
SSD = 72.78 pcf x 27 ft^3/yd^3 = 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/yd^3)**	**SDD Weight (lbs/yd^3)**	**Percent of Dry Weight of Aggregate**	**Percent of Dry Soil-Cement 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/yd^3 ÷ 27.0 ft^3/yd^3
= 151.7 lbs/ft^3

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 C-39 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:

	Aggregate Properties	
Property	**Coarse**	**Fine**
Absorption (%)	1.5	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 inches from Table A.2. Use a water content of 253 pcy. Using Figure A.9, 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 x 0.062 (from Figure A.7) = 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 C 1170). Test results indicate a Vebe time equal to 10 seconds. The mixture is well proportioned but is too wet. Adjust mixing water by 3 percent for each 10 second change in Vebe consistency. Calculate new trial mixture with 6 percent (2 x 3 percent) 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.7) = 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 C 1435. 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.10, 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 ft^3
Water:	246 lbs/62.4 pcf = 3.94 ft^3
Air (1.9%):	27 ft^3 x 0.019 = 0.513 ft^3

Calculate Aggregate Volume:

27.0 ft^3 – 2.239 ft^3 – 3.94 ft^3 – 0.513 ft^3
= 20.30 ft^3

Fine Aggregate Content:

20.308 ft^3 x 0.43 = 8.732 ft^3

Coarse Aggregate Content:

20.308 ft^3 – 8.732 ft^3 = 11.576 ft^3

Calculate Aggregate Weight

Fine Aggregate (SSD):

8.732 ft^3 x 2.85 x 62.4 pcf = 1552.9 lbs

Coarse Aggregate (SSD):

11.57 ft^3 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 ft^3/yd^3 = 153.3 lbs/ft^3

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 (300 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 (ft^3)	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 ft^3	N/A	4138

Water: cement ratio = 0.56

	RCC Design Mix Proportions			
Constituent	Dry Weight (lbs/yd^3)	SSD Weight (lbs/yd^3)	Percent of Dry Weight of Aggregate	Percent of Dry Soil-Cement 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. (Ref. USACE Technical Memorandum EM 1110-2-2006, Table 3-3)

	Nominal Maximum Size of Aggregate[a]			
	3/4" (19 mm)		2" (50 mm)	
Contents	Average	Range	Average	Range
Water content[b], lbs/yd^3 (kg/m^3)				
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

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

Table A.4

Ideal Coarse Aggregate Grading

	Cumulative Percent Passing		
Sieve Size	3 in. to No. 4 (4.75 to 75 mm)	2 in. to No. 4 (4.75 to 50 mm)	¾ in. to No. 4 (4.75 to 19.0 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.

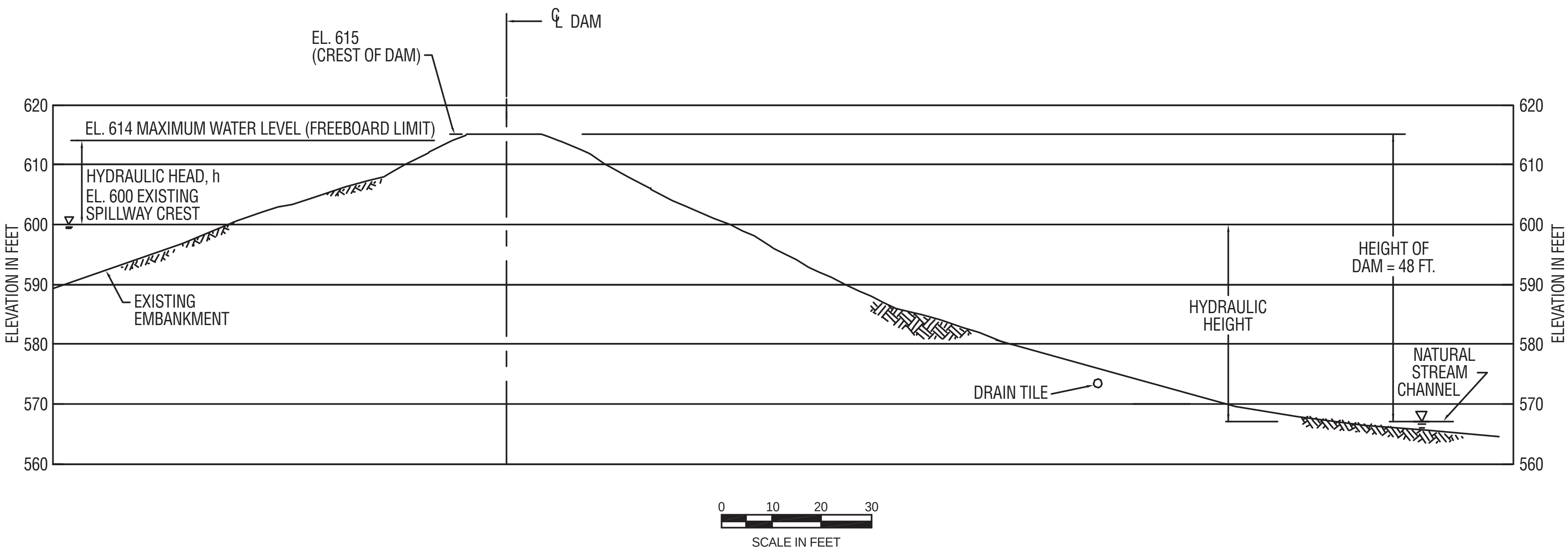

Figure A.1. Typical Dam – Existing cross section

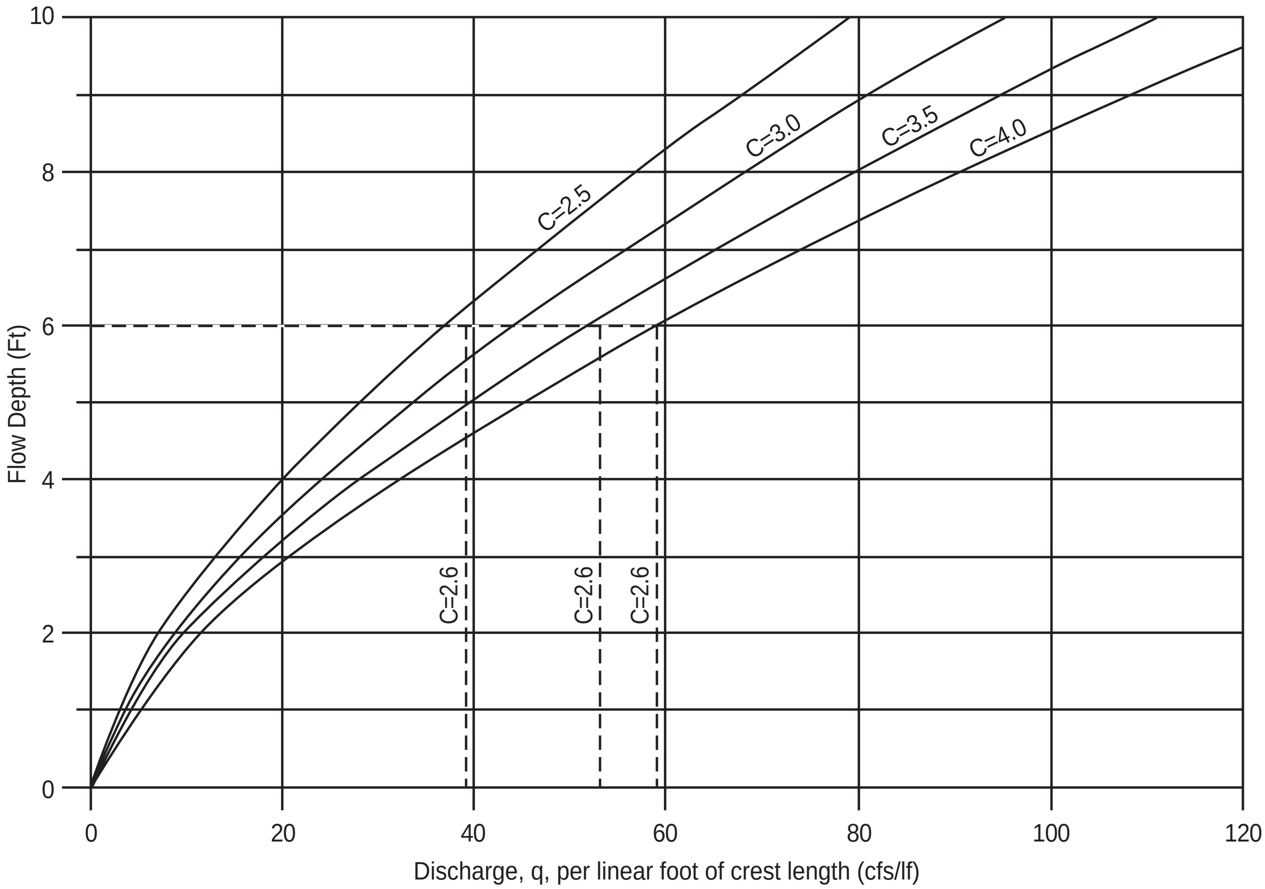

Figure A.2. Discharge versus crest coefficient and flow depth ($q = C*L*h^{3/2}$)

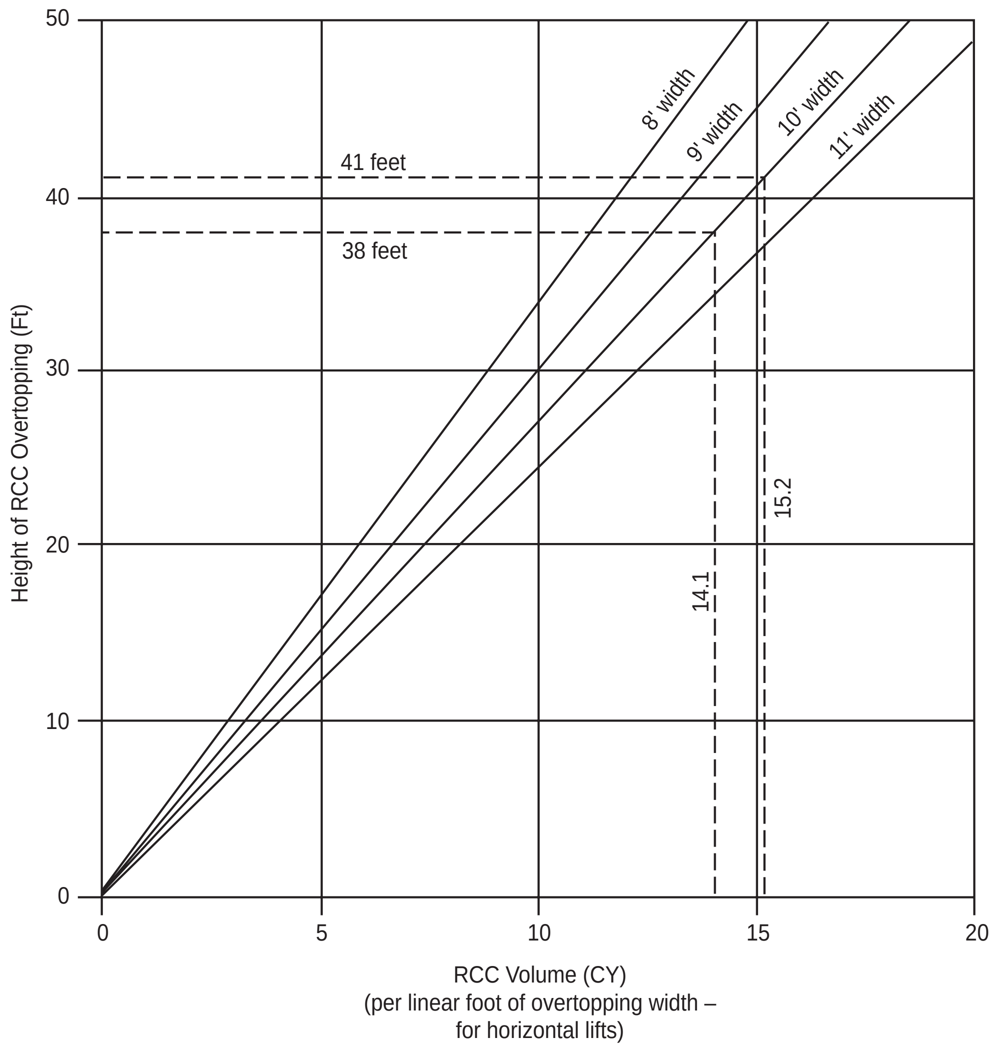

Figure A.3. Height of RCC overtopping versus RCC chute volume

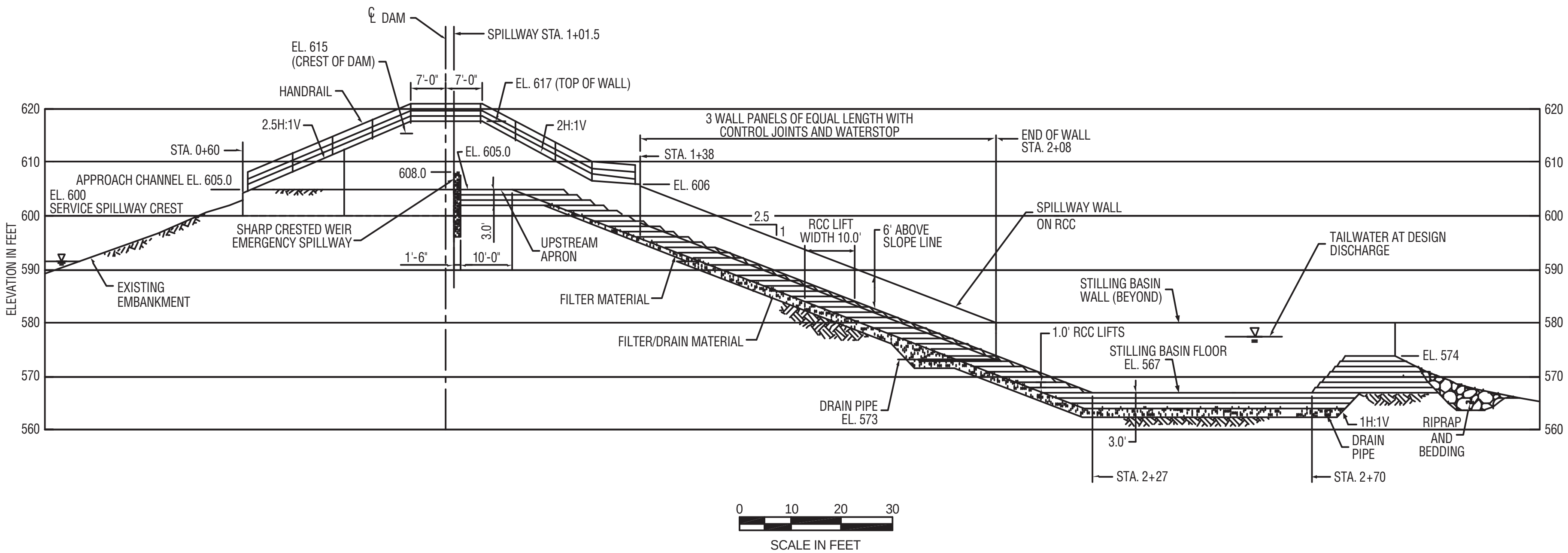

Figure A.4. RCC spillway overtopping section – Typical Dam

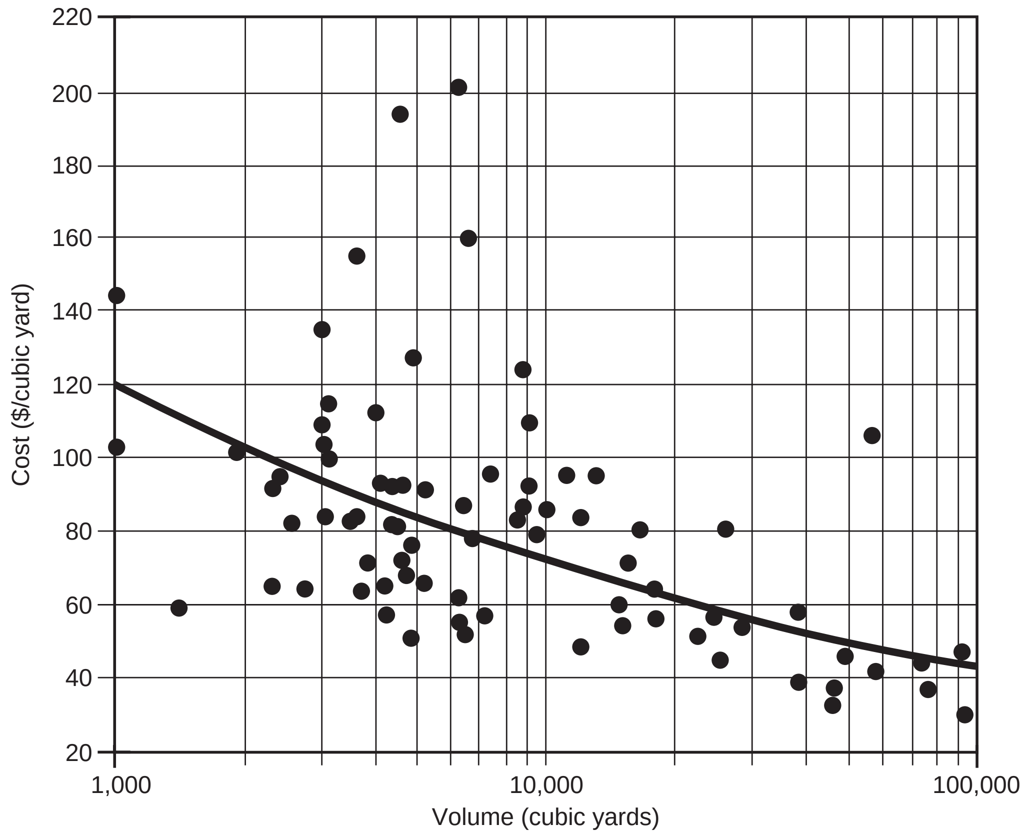

Figure A.5. 2001 Cost curve, RCC overtopping protection projects

(No adjustments have been made to unit costs based on cementitious content)

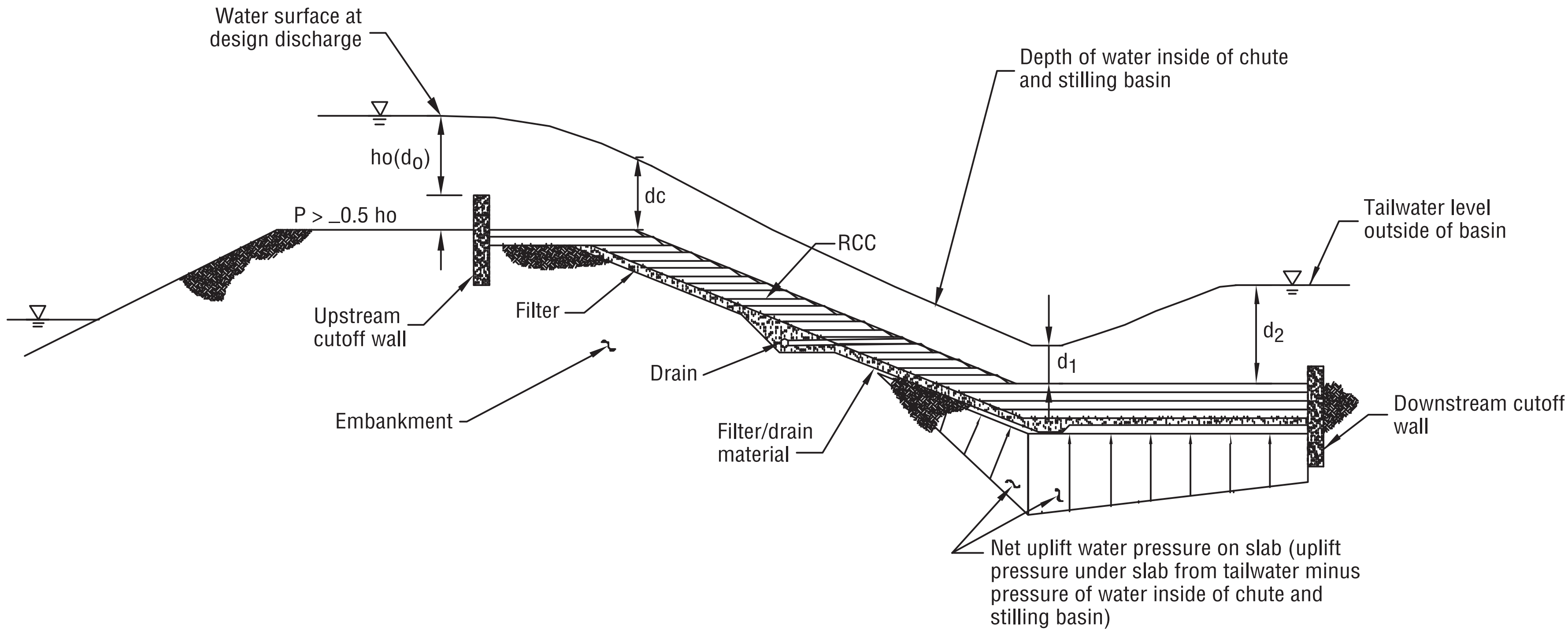

Figure A.6. Hydraulic design parameters

NTS

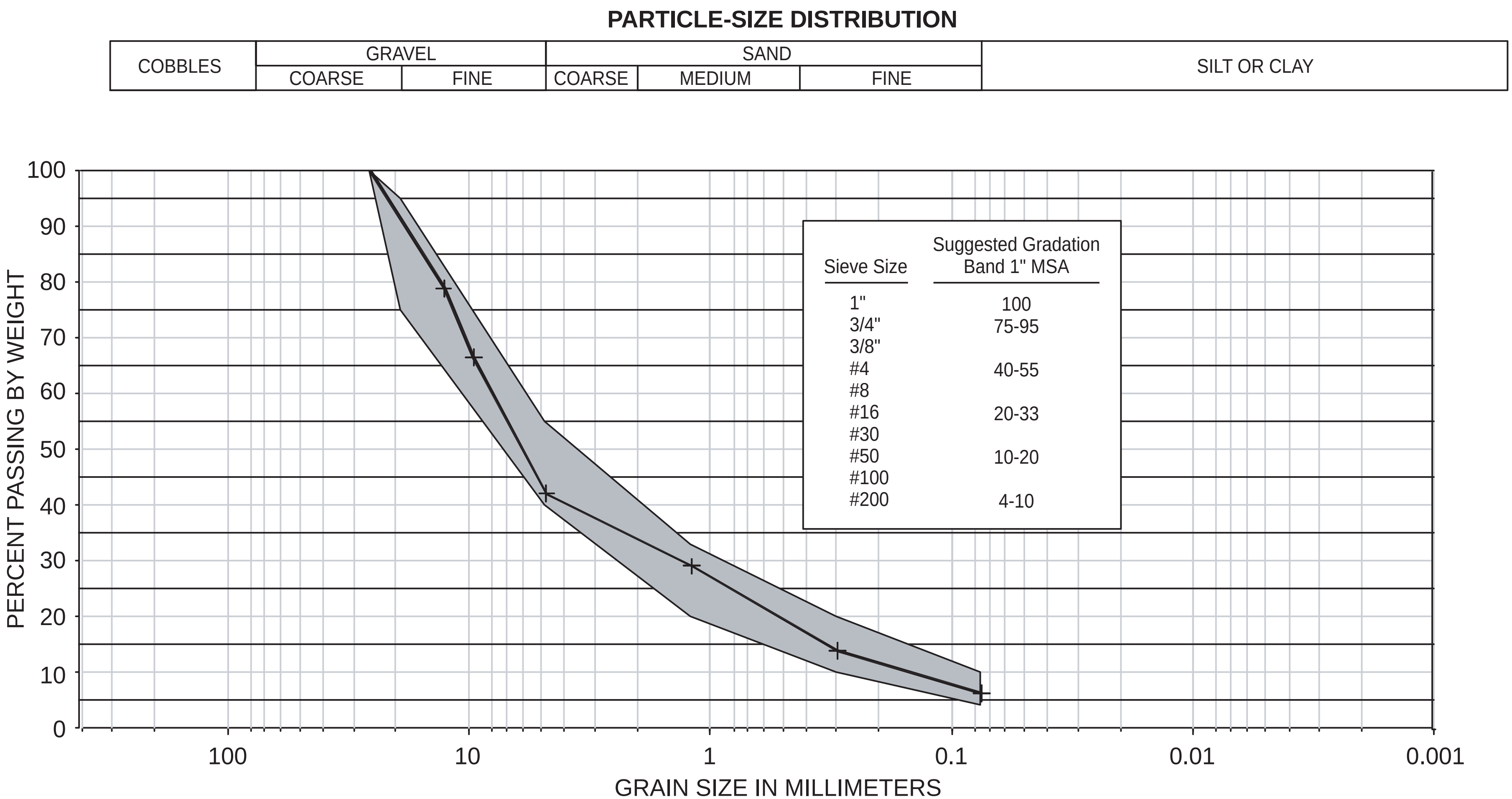

Figure A.7. RCC aggregate gradation example

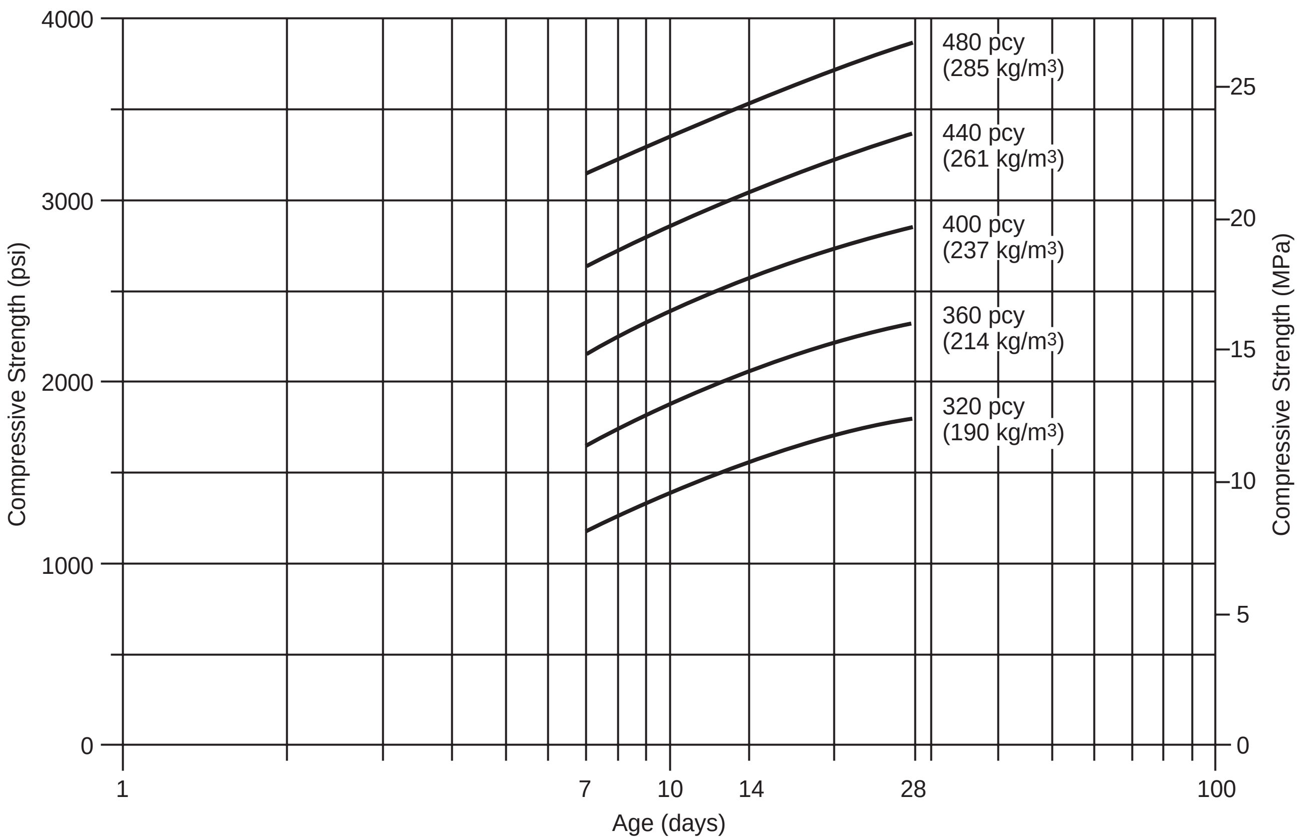

Figure A.8. RCC compressive strength versus age for various cement contents – soil compaction method

(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.)

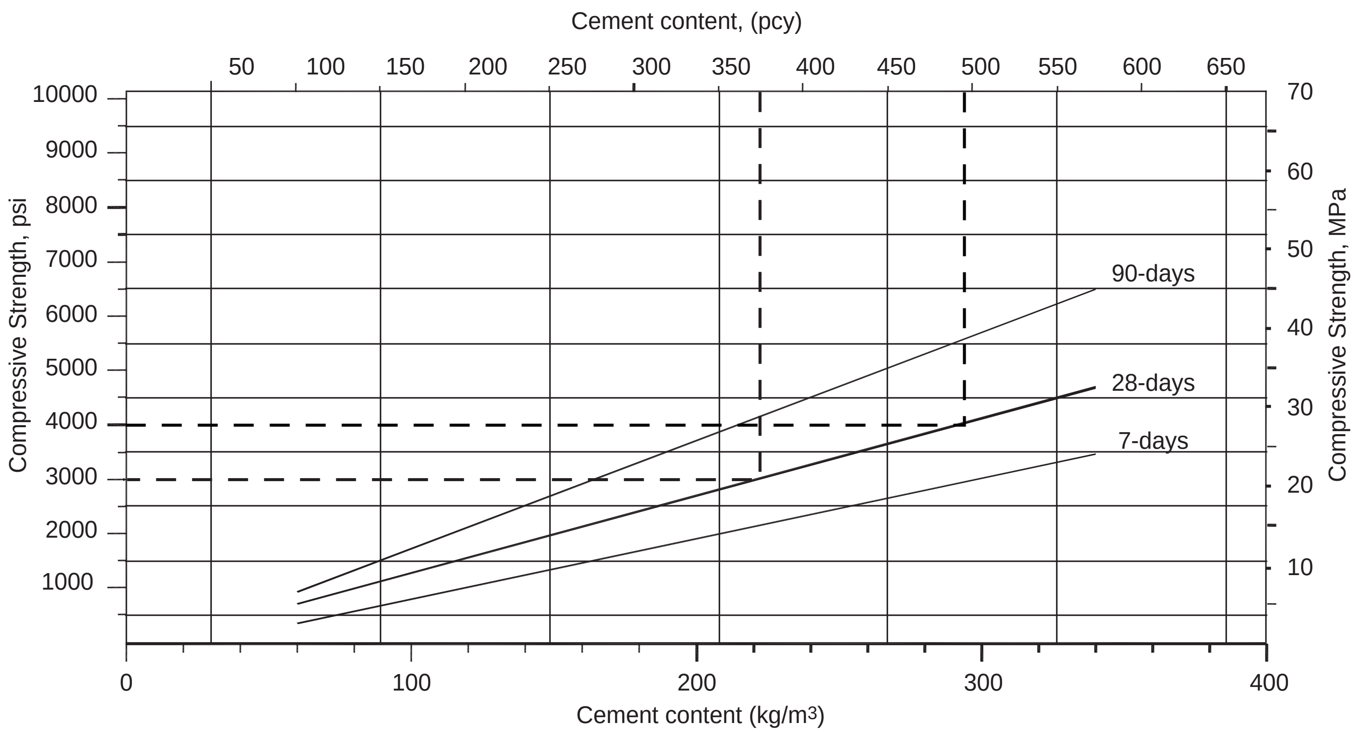

Figure A.9. Compressive strength versus cement content for various ages
(Ref. USACE Technical Memorandum EM 1110 Figure 3.2.)

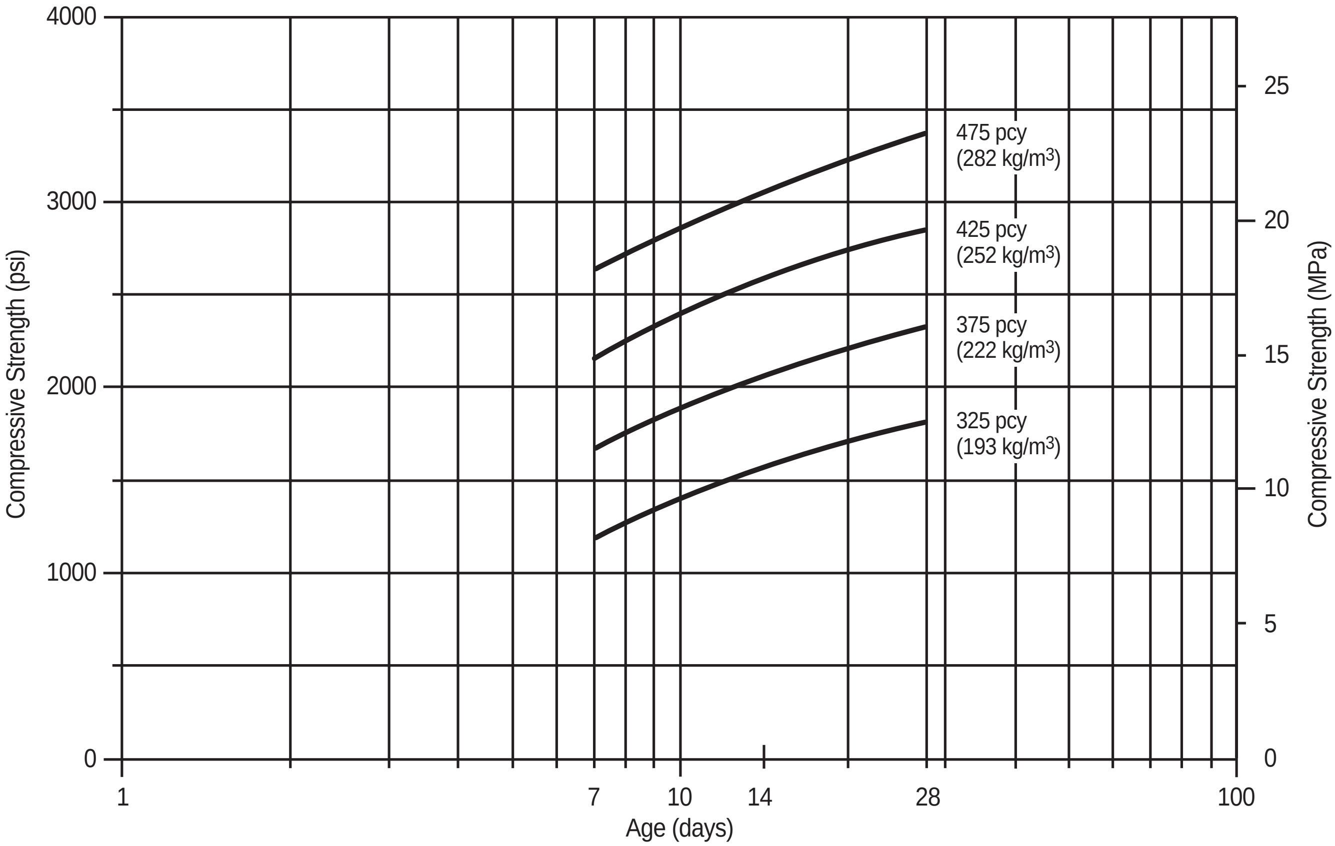

Figure A.10. RCC compressive strength versus age for various cement contents – conventional concrete method

(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.)

ACKNOWLEDGEMENTS

This Guide for RCC Overtopping Protection Design was developed by Terry Arnold (primary author) Principal, and Sal Todaro, Senior Consultant, URS Corporation. This guide was prepared for the Portland Cement Association under the supervision of Randall Bass, and Wayne Adaska.

Contributing authors of various sections of the guide were: John France, Senior Principal; Scott Anderson, Senior Geotechnical Engineer; Mike Miller, Senior Project Engineer; and Frank Lan, Senior Hydraulic Engineer, URS Corporation.

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This document is written in English units. To convert to metric units, use the conversion table presented below.

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 (ºF)	Celsius (ºC)	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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ERRATA

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	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

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

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).

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

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:

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

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

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

$$= 0.407$$

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.6). 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)

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

$$V = 17.63$$

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

$$H_F = 2.0 + 0.64 = 2.64$$

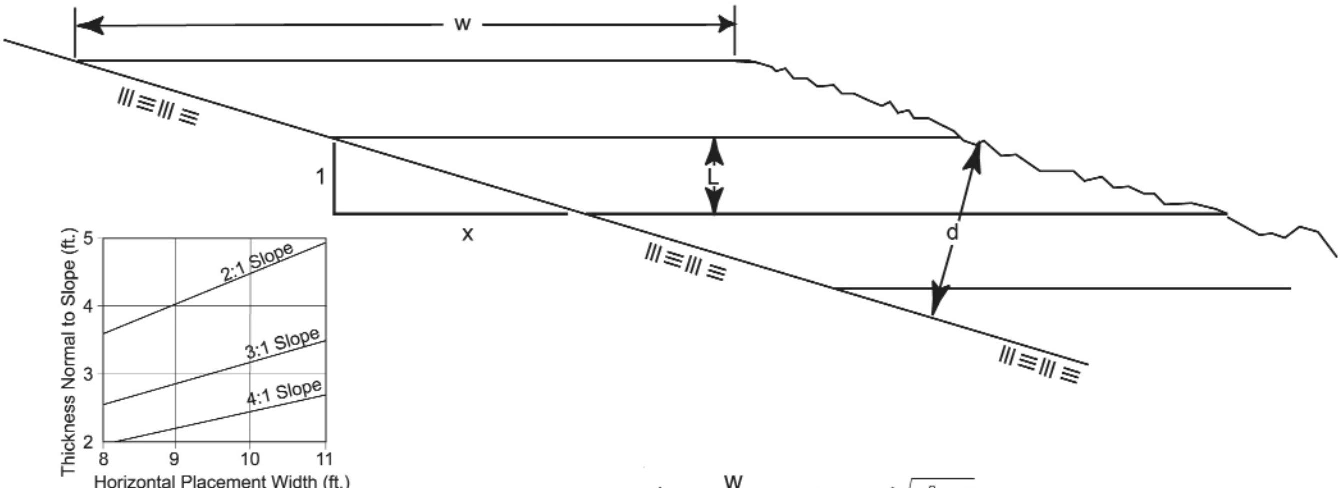
Wall Height (unbulked depth of flow + freeboard)
= 3 + 2.64 = 5.64 ft

Use Wall Height = 6 ft

CLARIFICATION

Page 34, Figure 6.13 changes —The thickness derived from the figure in the publication is for unformed sloping RCC steps. The figure below is an addendum to the current figure and includes thegraphic representation and equation for determining the thickness (d) of the RCC step normal to the embankment slope for both formed and unformed RCC steps.

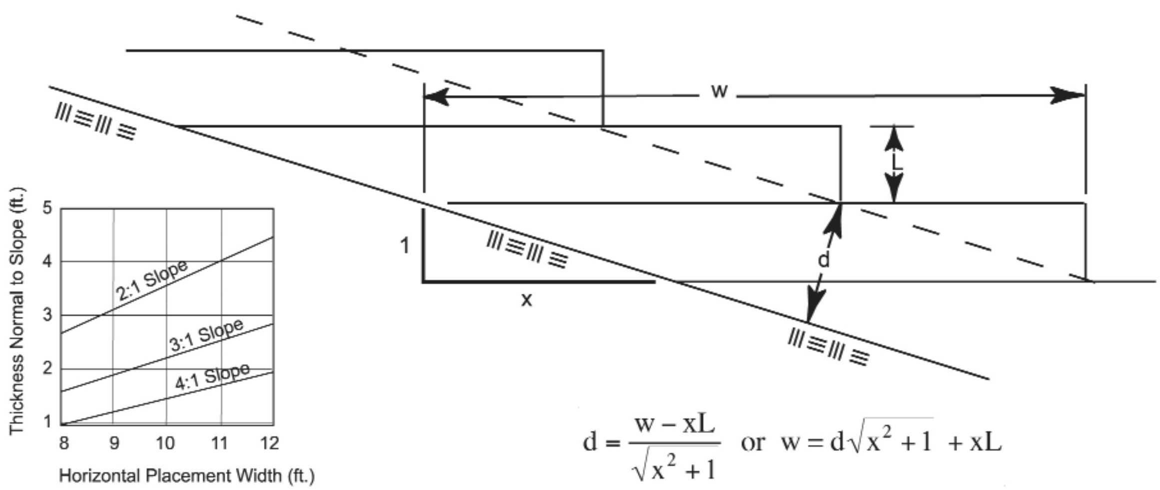
Unformed Sloping Section



$$d = \frac{w}{\sqrt{x^2+1}} \quad \text{or} \quad w = d\sqrt{x^2+1}$$

Figure 6.13. Facing thickness (d) of RCC versus width (w) for a 1-ft thick (L) unformed sloping RCC step.

Formed Stepped Section



$$d = \frac{w - xL}{\sqrt{x^2+1}} \quad \text{or} \quad w = d\sqrt{x^2+1} + xL$$

Figure 6.13a. Facing thickness (d) of RCC versus width (w) for a 1-ft thick (L) unformed RCC step.

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$$V = 17.63$$

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

$$H_F = 2.0 + 0.64 = 2.64$$

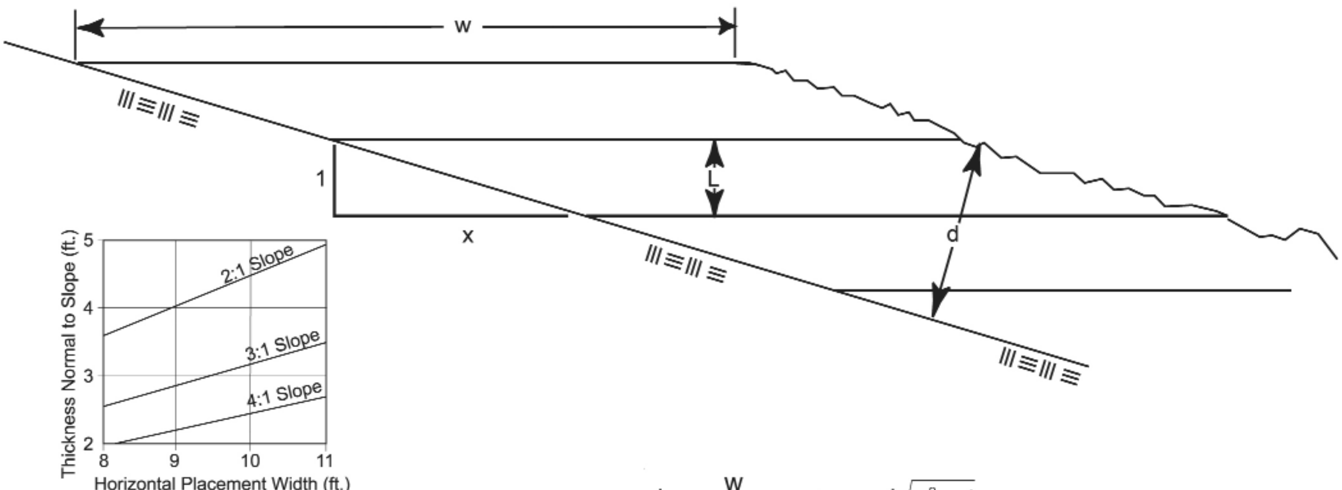
Wall Height (unbulked depth of flow + freeboard)
= 3 + 2.64 = 5.64 ft

Use Wall Height = 6 ft

CLARIFICATION

Page 34, Figure 6.13 changes —The thickness derived from the figure in the publication is for unformed sloping RCC steps. The figure below is an addendum to the current figure and includes thegraphic representation and equation for determining the thickness (d) of the RCC step normal to the embankment slope for both formed and unformed RCC steps.

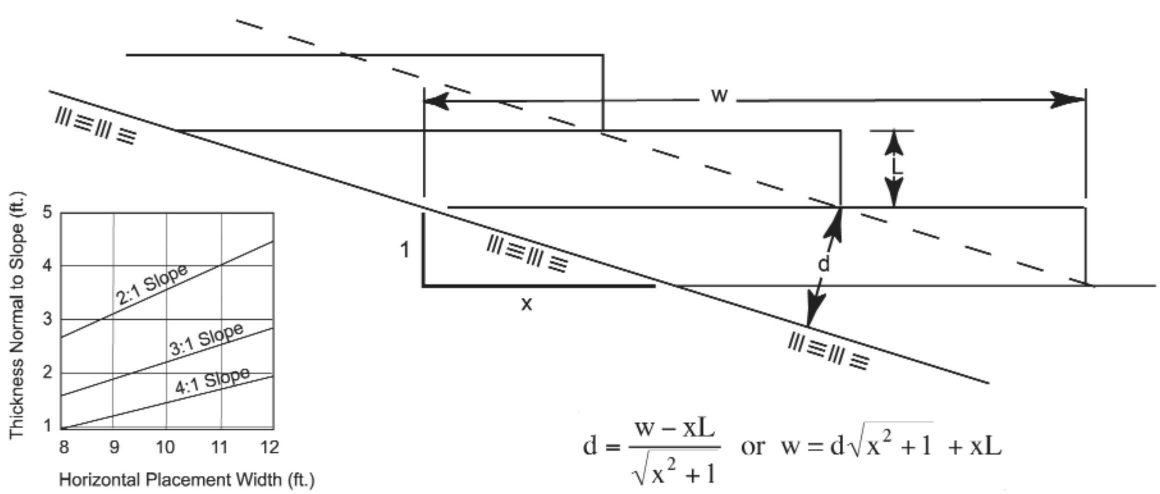
Unformed Sloping Section



$$d = \frac{w}{\sqrt{x^2+1}} \quad \text{or} \quad w = d\sqrt{x^2+1}$$

Figure 6.13. Facing thickness (d) of RCC versus width (w) for a 1-ft thick (L) unformed sloping RCC step.

Formed Stepped Section



$$d = \frac{w - xL}{\sqrt{x^2+1}} \quad \text{or} \quad w = d\sqrt{x^2+1} + xL$$

Figure 6.13a. Facing thickness (d) of RCC versus width (w) for a 1-ft thick (L) unformed RCC step.