

Soil-Cement Guide for Water Resources Applications

Dennis L. Richards and Hans R. Hadley, WEST Consultants, Inc.



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By Dennis L. Richards and Hans R. Hadley, WEST Consultants, Inc.



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An organization of cement companies to improve and extend the uses of portland cement and concrete through market development, engineering, research, education, and public affairs work.

Abstract: For more than 50 years, soil-cement has proven to be effective and economical construction material for use in water resources applications including streambank protection, slope protection, channel and pond linings, and grade control structures. Materials for soil-cement, mix proportioning, design, construction, and quality control topics are discussed.

Keywords: Soil-cement, streambank protection, erosion control, grade control structures, levee protection, slope protection, channel lining, upstream slope protection.

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Cover photo credits:

Top: Soil-cement upstream slope protection at Jackson Lake, Colorado.

Middle: Soil-cement bank protection at Rillito River, Tucson, Arizona.

Bottom: Soil-cement upstream slope protection at Tampa Bay Regional Water Reservoir.

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1 INTRODUCTION

Soil-cement, as defined in ACI 230 State-of-the-Art Report on Soil-Cement, is a mixture of soil and measured amounts of portland cement and water compacted to a high density. It can be further defined as a material produced by blending, compacting, and curing a mixture of soil/aggregate, portland cement, possibly admixtures including pozzolans, and water to form a hardened material with specific engineering properties. Soil-cement has proven to be an effective and economical construction material for use in water resource applications including streambank protection, slope protection, channel and pond linings, and grade control structures.

A wide variety of soils can be used to make durable soil-cement slope protection. For maximum economy and most efficient construction, it is recommended that (1) the soil contain no material retained in a 2-in. (50-mm) sieve; (2) at least 55% of the material pass the No. 4 (4.75-mm) sieve; (3) between 5% and 35% pass the No. 200 (0.075-mm) sieve; and (4) the plasticity index (PI) not be greater than 8. Material meeting these requirements is readily available at most sites. If the amount of material passing the No. 200 (0.075-mm) sieve exceeds 35%, the addition of a coarser material may be justified. Soils containing more than 35% material passing the No. 200 (0.075-mm) sieve may be used if the fines are nonplastic and can be mixed adequately with cement.

For slopes exposed to moderate to severe wave action or debris carrying rapid flowing water, the soil-cement usually is placed in successive horizontal layers adjacent to the slope. This is referred to as “stair-step” slope protection. For less severe applications, such as small reservoirs, ditches, and lagoons, the slope protection may consist of one or more layers of soil-cement placed parallel to the slope face. This method is often referred to as “plating.”

1.1. Background

Following World War II, a rapid expansion of water resources projects occurred in the Great Plains region of the United States. Rock riprap of satisfactory quality for upstream slope protection was not locally available for many of these proj-

ects, and the high costs of transporting the material from distant quarries made the economic feasibility of some projects questionable. The U.S. Bureau of Reclamation (USBR) initiated a major research effort to study the suitability of soil-cement as an alternative to riprap.

The USBR constructed a full-scale test section in 1951 following successful laboratory studies that showed soil-cement made of sandy soils was durable in a water-erosive environment. A location along the southeast shore line of Bonny Reservoir in eastern Colorado was selected because of natural severe service conditions created by large waves, ice, and more than 100 freeze-thaw cycles per year (Figure 1-1). The USBR was convinced after evaluating the test section for 10 years that soil-cement was suitable for slope protection.



Figure 1-1. Bonny Reservoir soil-cement test section in 2001 after 50 years of service.

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1.2. Applications

Since its initial application as slope protection in reservoirs by the USBR, soil-cement was found to provide several additional uses in the water resources field. The following sections summarize the use of soil-cement in water resources applications including streambank protection, slope protection, channel linings, pond linings, and grade control structures. More detailed descriptions and examples are provided later in the document.

1.2.1. Slope Protection

Soil-cement has proven to be an effective and economical method for providing slope protection for many dams and other types of embankments. These include dikes, levees, channels, coastal shorelines, highway and railroad embankments, and embankments that form cooling-water reservoirs for electric generating plants.

The typical design section used for slope protection depends on the severity of its intended application. For slopes exposed to moderate-to-severe wave action the soil-cement usually is placed in successive horizontal layers 8 to 9 ft (2.4 to 2.7 m) wide by 6 to 12 in. (150 to 300 mm) thick adjacent to the slope (Figure 1-2). This is referred to as “stair-step” slope protection. The “steps” that are created help to dissipate the energy in the waves and reduce the height of the wave runup. Figure 1-3 shows a completed “stair-step” slope protection project on a cooling water reservoir in Texas.

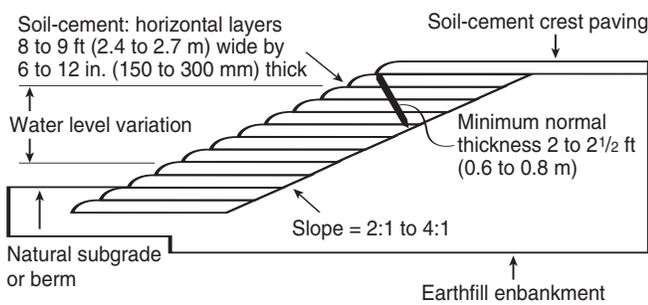


Figure 1-2. Typical section for soil-cement slope protection.

For less severe applications, slope protection may consist of a 6- to 18-in.- (150- to 450-mm-) thick layer of soil-cement placed parallel to the slope face. If the final thickness is greater than 12 in. (300 mm), the soil-cement should be placed in more than one lift. This method, referred to as



Figure 1-3. Completed “stair-step” slope protection project, Texas.

“plating,” uses less soil-cement than the stair-step method but cannot be successfully placed on slopes steeper than 2.5:1 (H:V) and provides little resistance to wave runup. Figure 1-4 shows the installation of plated soil-cement slope protection.

At the Tampa Bay reservoir in Florida, designers used both plating and stair-step methods to optimize the quantity and cost of soil-cement slope protection. The embankment height and wave run-up amongst other design parameters were considered in the optimization process. Plating method was used for the soil-cement that would normally be submerged. Stair-step method was used for the upper portion of the slope where wave action is expected and limiting wave run-up is desired (Figure 1-5).



Figure 1-4. Installation of plated soil-cement slope protection.



IMG19599

Figure 1-5. Plated and stair-stepped soil-cement slope protection at Tampa Bay Reservoir, Florida, at beginning stages of reservoir first filling.

1.2.2. Streambank Protection

Building on the success of soil-cement for slope protection for wave action, engineers transferred this knowledge to protecting streambanks from lateral erosion during flood events. The success of soil-cement in this application was demonstrated during two significant flood events in Tucson, Arizona, in 1978 and 1983. Even though overtopped, the soil-cement bank protection prevented millions of dollars in property damage. In Tucson, more than 74 miles (119 km) of streams, rivers, and washes now are protected with soil-cement. In 2002, Los Angeles County, California, formally adopted soil-cement as an acceptable material for bank stabilization in the county. A pilot project, West Creek near Santa Clarita, was completed as the first soil-cement project where the county assumed control over the project following construction. The first soil-cement bank protection in Palmdale, California, was completed in 2003 on Amargosa Creek (Figure 1-6).



IMG19600

Figure 1-6. Amargosa Creek, California, bank protection.

A typical section consists of 8- to 9-ft- (2.4- to 2.7-m-) wide horizontal layers placed in stair-step fashion along stream bank slopes (Figure 1-7). If the design calls for a “soft” channel bottom, the soil-cement is carried below the existing channel invert elevation to a depth equal to the maximum scour depth that could be expected over the life of the project. For a “hard” bottom design that would have either a soil-cement or concrete-lined invert, the soil-cement bank protection provides sufficient overlap with the lined invert as shown in Figure 1-8. At the terminus of the soil-cement reach, the soil-cement protection is turned perpendicular to the channel into the banks approximately 50 ft (15 m) to prevent head-cutting erosion from occurring behind the soil-cement. Figure 1-9 shows an example of a completed bank protection project that uses a hard-bottom design. Alternatively, Figure 1-10 shows an example of a soft-bottom design.

The exposed slope facing can be trimmed “smooth” as shown in Figure 1-11, left natural with loose overbuild soil-cement remaining in place as shown in Figure 1-12, or rough steps can be created using forms or edge compaction techniques as shown in Figures 1-3 and 1-9. Slope facing that is left natural with varying amounts of overbuild can create the illusion that the soil-cement is not durable because portions of the loose overbuild will be eroded by floodwaters.

To withstand the abrasive force of stormwater flows at velocities up to 20 ft/sec (6m/sec), the soil-cement typically is designed in stair-step fashion with a minimum 7-day compressive strength between 600 and 750 psi (4.2 and 5.2 MPa).

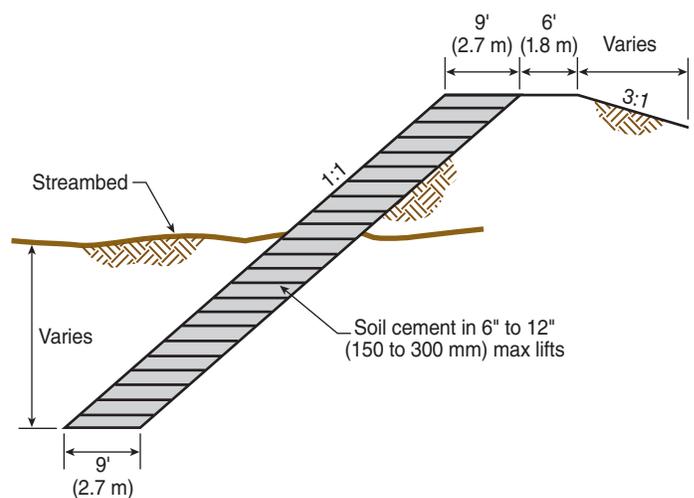


Figure 1-7. Typical section for soil-cement streambank protection.

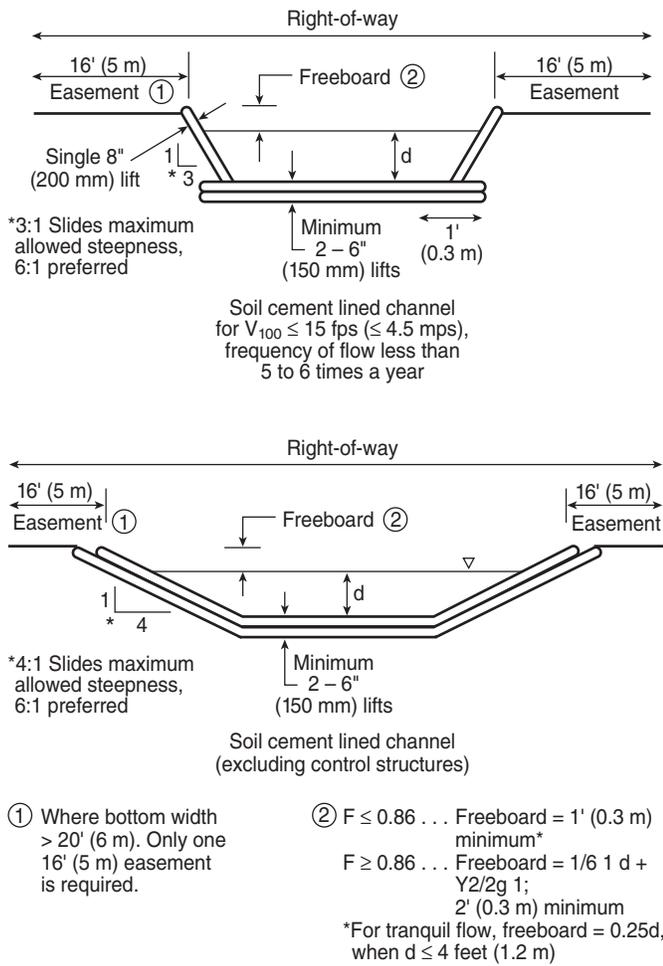


Figure 1-8. Typical section for soil-cement channel linings
 V = Average channel velocity and F = Froude Number
 (GLHN Architects and Engineers, Inc).



Figure 1-9. Example of hard-bottom channel and rough steps along San Antonio Arroyo, New Mexico.



Figure 1-10. Example of soft-bottom channel at Sand Creek, Colorado.



Figure 1-11. Example of trimmed smooth soil-cement bank protection along San Diego Creek in Irvine, California.



Figure 1-12. Bank protection along Las Vegas Wash, Nevada.

1.2.3. Channels

The use of soil-cement as a lining for open flumes was investigated as early as 1943, when a test flume was constructed at the Stillwater Hydraulics Laboratory at Oklahoma State University in Stillwater, Oklahoma (PCA, 1943). The soil-cement was a mixture of sandy loam (60% sand and 40% clay) and 8% cement. The flume was tested with clean water flowing at a rate of 150 cfs (4.2 cu m per second) with a velocity of 28 ft (8.5 m) per second for six days. No appreciable wear or erosion was produced. In addition to providing erosion protection, soil-cement also minimizes water loss due to seepage. Figure 1-8 shows typical sections used for channel linings.

A common application of soil-cement channel linings is for flumes carrying cooling water to and from power plants. Figure 1-13 shows an example of a channel constructed in 1971 at the Florida Power and Light Company plant in Sanford, Florida. The soil-cement was central plant mixed, placed on 3:1 (H:V) side slopes, and compacted by a bulldozer equipped with street pads operating up and down the slope.



IMG19605

Figure 1-13. Channel lining for Florida Power & Light Company plant in Sanford, Florida.

1.2.4. Linings

Soil-cement has served as a low permeability lining material for over 50 years. In 1945, soil-cement was used to line the bottom of a 12-million-gallon (45-million-L) reservoir in Port Isabel, Texas. The reservoir had been constructed initially with concrete faced side slopes but no bottom lining. The soil-cement was mixed-in-place, using 12% cement by volume, and compacted to a thickness of 4 in. (100 mm).

In 1955, engineers with the Soil Conservation Service began lining small 1- to 2-acre (0.4- to 0.8-hectare) farm reservoirs in southern California with soil-cement. A study by the Soil Conservation Service in 1970 found that the average seepage loss (change in water depth) of three soil-cement-lined farm reservoirs was 0.06 ft/day (18 mm/day), which is considerably less than similarly lined clay reservoirs.

In 1967, the 135-acre (55-hectare) bottom of Lake Cahuilla, a terminal regulating reservoir for the Coachella Valley County Water District irrigation system in the Mojave Desert of southern California, was lined with 6-in.-thick soil-cement. The banks were protected by 2-ft- (0.6-m-) thick (normal to slope) stair-stepped soil-cement. Without the soil-cement liner, water could not be preserved in the reservoir for subsequent use.

Soil-cement also has been used in conjunction with geomembrane liners. In 2001 the City of Kennedale, in Tarrant County, Texas, relined its Cedar Creek raw water balancing reservoirs (Figure 1-14). A composite liner system was used consisting of a High Density Polyethylene (HDPE) liner covered with 6 in. (15 cm) of soil-cement. The soil-cement served the purpose of protecting the liner from the elements and allowed rubber-tire equipment to operate in the reservoir to remove accumulated sediments. Soil-cement also has been used for lining wastewater treatment lagoons (Figure 1-15), sludge drying beds, and ash settling ponds.



IMG19606

Figure 1-14. Soil-cement placement and compaction over HDPE liner at Cedar Creek, Texas, balancing reservoir.



IMG19607

Figure 1-15. Soil-cement lined sludge lagoon in Austin, Texas.

1.2.5. Grade Control Structures

Reducing the velocity of flowing water is one approach to preventing the degradation of stream channels. The volume of water is difficult to change but modifying the effective slope of the channel can be accomplished by creating small soil-cement structures that raise the normal water surface elevation across a reach of the channel. The velocity of the water is decreased in the reach above the grade control structure, reducing the potential for erosion. The water is allowed to spill over the grade control structure and back into the natural channel. Figure 1-16 illustrates a typical cross section of a grade control structure. Often there will be a series of grade control structures when the channel gradient is fairly steep. Where bedload sediment transport is frequent and/or consists of larger sediment sizes such as gravels, cobbles, and boulders, grade control structures should be constructed with more erosion resistant material such as conventional concrete or roller compacted concrete (RCC), rather than soil-cement.

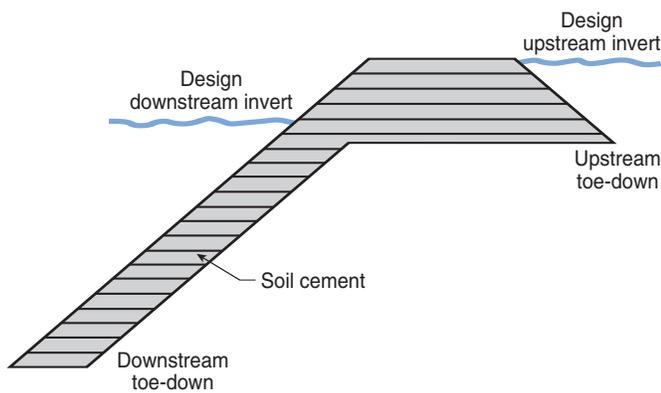


Figure 1-16. Typical section of soil-cement grade control structure.

In the Highlands Ranch Development south of Denver, soil-cement drop structures were used along Marcy Gulch (Figure 1-17 and Figure 1-18). The material for the soil-cement came from the overbank deposits and was mixed with cement on site in a portable pugmill plant. The structures are designed and constructed to look natural and blend in with the park setting.



IMG19608

Figure 1-17. Soil-cement drop structure constructed along Marcy Gulch, Colorado.



IMG19609

Figure 1-18. Water flowing over drop structure in Marcy Gulch, Colorado.

1.3. Erosion and Abrasion Resistance Research

Some research has been conducted on erosion and abrasion resistance of soil-cement to provide a basis for design of water resources applications. The earliest tests on soil-cement were conducted by the Civil Engineering Department at Oklahoma A & M College (now Oklahoma State University) (PCA, 1943). The test consisted of applying a continuous flow of clean water at a velocity of 28 ft/sec (8.5 m/sec) in a

10-foot- (3-m-) wide soil-cement-lined flume for a period of six days. The soil-cement was composed of approximately 60% sand, 40% silt and clay, and 8% cement by volume. No appreciable erosion of the 4.5-in.- (114-mm-) thick soil-cement lining occurred.

Nussbaum and Colley (1971) conducted laboratory experiments that tested the erosion resistance of soil-cement to flow carrying 1/8-in.- to 1/4-in.- (3-mm- to 6-mm-) size gravel. Soil-cements developed from two different soil types were exposed to an abrasion erosion test after fog curing for 7 days. An AASHTO A-1-b (sand-gravel) soil and A-4 (silt) soil were mixed with varying percentages of cement. About 8,000 gallons (30,000 l) of water carrying 4.2 tons (3.8 tonne) of gravel flowed at a velocity of 3.8 ft/sec (1.1 m/sec) over a small soil-cement sample each day until the depth of erosion in the sample totaled 1 in. (25 mm). Results of the tests indicated that increasing the cement content of the soil-cement increased its erosion resistance. However, the size of the aggregate in the soil-cement had an even greater effect on its erosion resistance. The abrasion resistance of the stabilized sand-gravel (A-1-b soil) was extremely good and superior to the stabilized silt (A-4) soil for all cement contents tested. The time required to erode a depth of 1 in. (25 mm) from the stabilized silt was less than 2 days even with cement contents as high as 13.5%. By contrast, the stabilized sand-gravel soil lasted 15 days with a cement content of 5% (Figure 1-19). Additional tests were conducted on soil-cement samples composed of silt aggregate by adding gravel greater than 1/4 in. (6 mm) in size. This was done to determine how much of an increase in erosion resistance could be gained by adding larger particles to fine material. Results of the tests indicated that the addition of gravel to the fine soil increased its resistance to erosion significantly when the gravel component was greater than 20% by weight. At 30% gravel content and 9.5% cement, the modified silt (A-4 soil) was nearly as resistant to abrasion as the sand-gravel (A-1-b soil) with 5% cement (Figure 1-19). The overall conclusions of the study were that the erosion abrasion resistance of soil-cement exposed to waterborne particles can be improved by (1) using coarser material as the aggregate, (2) adding gravel to a finer soil, or by (3) increasing the cement content.

Litton and Lohnes (1982) conducted research at Iowa State University to determine if locally derived soils could be stabilized sufficiently with cement to construct drop structures that would control degradation in stream channels. The local soil was a loess-derived alluvium (loam) from western Iowa. The local soil was mixed with a coarser material (sand) in proportions of 20%, 40%, and 55% to increase erosion

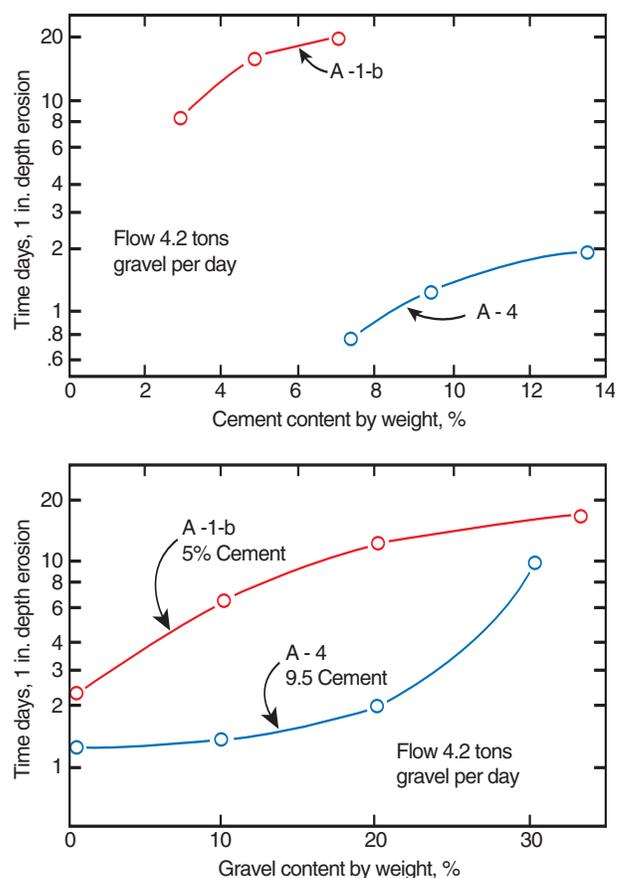


Figure 1-19. Erosion resistance of stabilized soils (Nussbaum, 1971).

resistance as found by Nussbaum and Colley (1971). Specimens containing 100% sand also were tested. The researchers also wanted to develop an alternative method for determining adequate cement content for erosion resistance of soil-cement exposed to a water jet. Results of the water jet experiments were then compared to results of standard durability brush tests. It was determined that for water jet velocities less than 20 ft/sec (6.1 m/sec), the erosion tests produced lower weight losses than that produced by brush tests. This provided designers with further evidence that soil-cement designed to provide adequate durability could withstand water velocities up to 20 ft/sec (6.1 m/sec) with little deterioration.

Litton and Lohnes (1983) later conducted additional experiments using the water jet apparatus to test the erosion resistance of soil-cement specimens after being subjected to 12 freeze-thaw cycles. The researchers found that for any given flow rate, the rate of erosion was relatively large at first and then quickly reached a stable configuration after which there was relatively minimal material loss.

Oswell and Joshi (1986) developed a test apparatus that combined the impacting forces of waves and waterborne debris. The research was conducted as part of a program to investigate soil-cement as a possible method for providing shore protection of artificial islands planned for oil drilling in the Alaskan and Canadian Beaufort Sea. The researchers created plastic soil-cement specimens, which are molded at higher water contents than for compacted soil-cement. As a result, the plastic soil-cement has less strength than compacted soil-cement. The mix was composed of masonry sand, Class G oil well cement (similar to Type V portland cement), and artificial sea water. Six different mixes were tested: two containing cement and four containing cement plus fly ash. All of the specimens were moist cured for either 1, 3, or 7 days to obtain samples of varying compressive strength. The specimens were tested using a variable pressure water jet that impinged directly on the specimen as well as stones contained in a wire cage. The stones were activated by the water jet, resulting in impact on the sample causing additional abrasion. The researchers found that the erosion loss was reduced with increasing compressive strength. For the soil-cement mixture used in their study, the researchers concluded that a compressive strength of 2,300 psi (16 MPa) was necessary to reduce erosion or abrasion erosion to a negligible amount when subjected to a jet with a water pressure of 10 psi (70 kPa) (PCA, R&D Serial No. 2436, 2001).

The Los Angeles District of the U.S. Army Corps of Engineers used an underwater abrasion erosion testing apparatus developed by Liu (1980) on both soil-cement and roller compacted concrete (RCC) (ASTM C 1138). This test was later expanded by Omoregie et al. (1994) to include conventional portland cement concrete (PCC). The results of the tests confirmed previous studies of abrasion erosion that wear progression is nonlinear. Greater wear occurred earlier in the test than later. This is the result of the cement paste and sand mortar on the surface eroding away and exposing the more resistant aggregate. With the soil-cement specimens, this occurred within the first few hours of testing. The PCC and RCC samples took 12 to 24 hours for the initial erosion of the paste and mortar. From the results of their work, the researchers concluded that abrasion resistance of the cement-stabilized materials was primarily a function of the aggregate hardness and secondarily a function of the strength of the cement paste.

The soil-cement had considerably less resistance than either the RCC or PCC. Therefore, if a typical soil-cement mixture is to be used to withstand abrasion from waterborne particles, the soil-cement needs to be stronger or of a greater thickness than either of the other two materials. The greater thickness is typical of soil-cement bank protection placed in stair-step fashion on a relatively steep bank. Typically in the stair-step design the soil-cement layer has a horizontal width of at least 8 ft (2.4 m). If thinner soil-cement sections are to be used such as for plating a channel, consideration needs to be given to increasing the soil-cement strength through the addition of larger stone particles to the mixture, increasing cement content, or both.

AGRA Earth & Environmental (1998) applied the underwater abrasion test (ASTM C 1138) to soil-cement mixtures planned for a project in Scottsdale, Arizona, for the Maricopa County Flood Control District. The soil-cement specimens contained cement contents between 5% and 10% by dry weight of aggregate in 1% increments. In all cases, increased cement content produced higher 7-day compressive strengths. However, for specimens with cement contents above 6%, the wear resulting from the 48 hours of abrasion testing was nearly identical. The researchers concluded that the abrasion resistance of the aggregate controls once a certain level of compressive strength is reached. In this case it was 1,100 psi (7.6 MPa) at 7 days. Results of the work described in this section as well as other abrasion erosion testing are given in the PCA publication *Erosion and Abrasion Resistance of Soil-Cement and Roller-Compacted Concrete*, Research and Development Bulletin RD126 (PCA, 2002)

1.4. Completed Projects (1997 – 2005)

The following table summarizes details of some water resources soil-cement projects completed between 1997 and 2005 including project location, bid date, construction year, volume of soil-cement used, and total cost per cubic yard of soil-cement.

Table 1-1. Completed Water Resources Soil-Cement Projects (1997 – 2005)

Project	Application	Bid date	Construction year	Soil-cement volume cu yds (m ³)	Soil-cement unit cost \$/cu yd (\$/m ³)
Arizona ANG Access Road, Phoenix, AZ	—	01/07/1997	1997	9,250 (7,075)	29.61 (38.75)
67th Ave Bridge, Skunk Creek, Phoenix, AZ	—	01/07/1997	1997	11,625 (8,890)	29.05 (38.05)
Sky Harbor Third Runway, Phoenix, AZ	Bank Protection	01/14/1997	1997	42,882 (32,800)	30.00 (39.24)
Avra Valley Road Bridge, Tucson, AZ	—	02/04/1997	1997	23,315 (17,835)	23.03 (30.12)
91st Ave Wastewater Plant, Tempe, AZ	Bank Protection	03/18/1997	1997	340,450 (260,445)	40.92 (53.52)
San Antonio Arroyo, Albuquerque, NM	Channel Liner	12/11/1996	1997	31,440 (24,050)	26.94 (35.24)
Tropicana Detention Basin, Las Vegas, NV	Slope Protection	09/19/1996	1997	11,800 (9,030)	38.14 (49.89)
SR 95 Levee, Bullhead City, AZ	Levee Protection	07/31/1997	1998	23,543 (18,010)	40.12 (52.48)
North Domingo Boca, Albuquerque, NM	Channel Liner	09/03/1997	1998	16,200 (12,395)	27.32 (35.73)
East Mesa Phase 1, Las Cruces, NM	—	04/08/1998	1998	24,500 (18,740)	51.57 (67.45)
Diascund Dam, Newport News, VA	Slope Protection at Emergency Spillway	03/03/1998	1998	4,400 (3,365)	52.00 (68.02)
Calabacillas Arroyo, Albuquerque, NM	Bank Protection	01/27/1998	1998	38,820 (29,700)	27.20 (35.58)
Camelback Ranch Phase 1, Phoenix, AZ	Bank Protection	1998	1998	38,800 (29,680)	32.95 (43.10)
Brahman Road Channel, Las Cruces, NM	Liner	08/25/1998	1998	11,140 (8,520)	38.66 (50.57)
Lower Santa Cruz, Tucson, AZ	Bank Protection	10/1998	1999	308,000 (235,620)	19.85 (25.96)
San Diego Creek – Barranca Pkwy, Irvine, CA	Bank Protection	06/1998	1999	36,689 (28,070)	20.45 (31.98)
San Diego Creek Phase 2, Irvine, CA	Bank Protection	09/1998	1999	17,750 (13,580)	25.65 (33.55)
Santa Margarita Levee, Camp Pendleton, CA	New Levee Protection	08/1998	1999	190,000 (145,350)	25.00 (32.70)
Bridgeport – Santa Clara, Santa Clara, CA	Bank Protection	10/1998	1999	100,000 (75,500)	31.62 (41.36)
Camelback Ranch Levee, North Glendale Airport, Phoenix, AZ	Bank Protection	12/08/1998	1999	73,938 (56,560)	28.09 (36.74)

Table 1-1. Completed Water Resources Soil-Cement Projects (1997 – 2005) (continued)

Project	Application	Bid date	Construction year	Soil-cement volume cu yds (m ³)	Soil-cement unit cost \$/cu yd (\$/m ³)
Black Mountain Detention Basin, Henderson, NV	Slope Protection	04/29/1999	1999	25,000 (19,125)	30.00 (39.24)
Norco Bluffs Zone 2, Riverside County, CA	Bank Protection	04/13/1999	1999	56,715 (43,390)	42.96 (56.19)
Lower Duck Creek Dam, Las Vegas, NV	Slope Protection	06/24/1999	1999	7,500 (5,740)	33.41 (43.70)
Lytle Creek, San Bernadino, CA (cement supplied by owner)	Bank Protection	—	1999	95,000 (72,675)	24.45 (31.98)
Cedar Creek Lake Balancing Reservoir, Fort Worth, TX	Liner with membrane	08/11/1999	1999	45,000 (34,425)	27.73 (36.27)
Many Farms Dam, Many Farms, AZ	Slope Protection	07/29/1999	1999	22,600 (17,290)	48.85 (63.90)
Landfill Levee, Glendale, AZ	Bank Protection	09/1999	1999	38,900 (29,760)	28.28 (36.99)
Wershban Lake, Omaha, NE	Slope Protection	1999	1999	6,500 (4,970)	45.00 (58.86)
Windmill Wash, Bunkerville, NV	Slope Protection for Dam	05/13/1998	1999	11,900 (9,100)	—
Hassayampa River Bank, Phoenix, AZ	Bank Protection	06/06/2000	2000	18,650 (14,270)	30.18 (39.48)
Pantano Wash, Tucson, AZ	Bank Protection	01/19/2000	2000	42,400 (32,435)	23.11 (30.23)
Marcy Gulch Phase 1, Highlands Ranch, CO	Drop Structures	09/2000	2000	2,570 (1,965)	97.35 (127.33)
Greystone Homes Tract #5099, Valencia, CA	Bank Protection	10/2000	2000	15,500 (11,860)	—
Cardinal Development, Santa Clarita, CA	Bank Protection	08/23/1999	1999	6,850 (5,240)	49.00 (64.09)
Creekside, San Francisquito Creek, Santa Clara, CA	Bank Protection	07/05/2000	2000	30,552 (23,370)	27.66 (36.18)
Hidden Creek, San Francisquito Creek, Santa Clara, CA	Bank Protection	07/05/2000	2000	19,560 (14,965)	26.20 (34.27)
Greystone Tract #5099, Filmore, CA	Bank Protection	02/24/2000	2000	13,500 (10,330)	47.75 (62.46)
Soledad Canyon Road, Santa Clarita, CA	Bank Protection	09/09/2000	2000	9,100 (6,690)	53.54 (70.03)
San Patricio Raw Water Storage Reservoir, TX	Liner	1/19/2000	2000	7,750 (5,930)	50.98 (66.68)
San Patricio Raw Water Storage Reservoir, TX	Slope Protection	1/2001	2001	8,800 (6,730)	44.90 (58.73)

Table 1-1. Completed Water Resources Soil-Cement Projects (1997 – 2005) (continued)

Project	Application	Bid date	Construction year	Soil-cement volume cu yds (m³)	Soil-cement unit cost \$/cu yd (\$/cu m³)
Marcy Gulch Phase 2, Highlands Ranch, CO	Drop Structure	08/30/2001	2001	7,820 (5,980)	75.50 (98.75)
Jackson Lake Dam, Wiggins, CO	Slope Protection for Dam	07/01/2001	2001	32,300 (24,710)	47.92 (62.68)
Santa Cruz River (Grant to Ft. Lowell), Tucson, AZ	Bank Protection	09/2001	2002	115,558 (88,400)	23.37 (30.57)
Moss Creek Spillway Repairs, TX	Drop Structure/ Channel Lining	10/5/2001	2002	9,900 (7,575)	49.24 (64.40)
West Creek, San Francisquito Creek, Santa Clara, CA	Bank Protection	07/05/2001	2002	27,345 (20,920)	27.66 (36.18)
San Timoteo Creek, CA	Bank Protection	9/17/2002	2003	222,000 (169,830)	27.82 (36.39)
Sand Creek Detention Pond, Colorado Springs, CO	Channel Lining	10/2002	2003	15,150 (11,590)	53.17 (69.60)
Tampa Bay Reservoir, Tampa, FL	Slope Protection for Dam	7/2002	2004	365,000 (279,000)	23.83 (31.17)
McMicken Dam, Phoenix, AZ	Embankment Protection	2/2005	2005	25,970 (19,850)	40.34 (52.76)
Mission Lakes, Riverside County, CA	Bank Protection	2/2005	2005	32,000 (24,470)	32.32 (42.27)

2 MATERIALS

Soil-cement is composed of a mixture of soil/aggregates, portland cement, and water. Chemical or mineral admixtures may be added to make up for a lack of fines, improve workability of the soil-cement during construction, or modify the characteristics of the plastic and final hardened properties of the soil-cement.

2.1. Soil Aggregates

Soil aggregates make up approximately 90% of the dry mixture and generally are derived from the project site or nearby location. This helps to reduce the impact to the environment, minimize construction costs, and provide a final product that has a similar appearance, texture, and color to the surrounding landscape.

2.1.1. Source

Soil aggregate for use in soil-cement may be produced by processing, screening, crushing, and/or blending soils obtained from the required excavations, and/or may be furnished from borrow. Soil aggregate for soil-cement should not contain any deleterious material. Before mixing as soil-cement, the soils should be stockpiled and sampled. The distribution and gradation of materials in the soil-cement should not result in lenses, pockets, streaks, or layers of

material differing substantially in texture or gradation from surrounding material. Table 2-1 shows the typical range of cement requirements for different soil types. Because of the more severe exposure conditions, cement contents in Table 2-1 have been increased by two percentage points over typical cement requirements for soil-cement base pavement stabilization from which the table was derived.

2.1.2. Stockpiling

Soil aggregate should be stored at the site of the mixing plant in accordance with *ACI Guide for Measuring, Mixing, Transporting, and Placing Concrete*, ACI 304 in such a manner as to avoid segregation or contamination by foreign materials. Soil aggregate should remain in free-draining storage for at least 24 hours immediately prior to use. Sufficient soil aggregate should be maintained at the site at all times to permit continuous, uninterrupted operation of the mixing plant at the time soil-cement is being placed.

2.1.3. Gradation

Soil aggregate for use in soil-cement construction, when tested in accordance with ASTM C 136 and ASTM C 117, should conform to the gradation indicated in Table 2-2 and should be free of any deleterious materials. Screening or blending may be needed to produce soil aggregate materials meeting the gradation requirement. Soil aggregate for soil-cement should not contain clay/silt lumps larger than 1 in. (25 mm). The plasticity index (PI) should be less than 8. With higher PIs, the tendency for the development of clay balls increases.

Table 2-1. Normal Range of Cement Requirements for Soil-Cement Slope Protection (PCA, EB052, 1992)

AASHTO soil group	% by volume of soil	% by weight of dry soil
A-1-a	7 – 9	5 – 7
A-1-b	9 – 11	7 – 10
A-2-4	9 – 12	7 – 11
A-2-5	9 – 12	7 – 11
A-2-6	9 – 12	7 – 11
A-2-7	9 – 12	7 – 11
A-3	10 – 14	9 – 13

Table 2-2. Typical Gradation for Aggregate Used in Soil-Cement

Standard sieve size	% passing by weight*
2 in. (51 mm)	100
1½ in. (37.5 mm)	98 – 100
No. 4 (4.75 mm)	50 – 80
No. 40 (425 µm)	40 – 70
No. 200 (75 µm)	5 – 20

*The maximum plasticity index (PI) for soil-cement aggregate materials is typically less than 8 when determined in accordance with ASTM D 4318.

2.1.4. Water (Moisture) Content

The water content of the soil is defined as percent of water in a given mass of soil in proportion to the mass of solid particles. The water content is determined by dividing the mass of water by the mass of oven-dried soil and is expressed as a percentage as follows:

$$\omega = \frac{\omega_w}{\omega_s} \times 100$$

ω = water (moisture) content in percent

ω_w = mass of water

ω_s = mass of oven-dried soil

Moisture content of stock-piled soil (aggregate) is typically in the range of 0.5% to 8%. This value is important to know as the water in the soil must be accounted for when determining the additional water needed to achieve the optimum moisture content of the soil-cement mixture that produces the maximum density when compacted. Moisture content determinations generally are made in accordance with ASTM D 4643, ASTM C 556, or ASTM C 70. Testing of soil and soil-cement mixtures is covered in Chapter 3.

2.2. Portland Cement

For most applications, portland cement conforming to ASTM Specification C 150 or C 1157 is used. Type I portland cement typically is used for most soil-cement applications. For soil-cement where the water-soluble sulfate (as SO_4) in the soil borrow or soil adjacent to the soil-cement is between 0.10% and 0.20% or the water in the reservoir or channel contains between 150 and 1500 parts per million sulfate, it is recommended that Type II portland cement with a maximum tricalcium aluminate (C_3A) content of 8% be specified. For severe sulfate exposures where the water soluble sulfate exceeds 0.20% in the soil or 1500 parts per million in the

water, a Type V portland cement with a C_3A content of not more than 5% should be used.

Immediately upon delivery to the work site, the portland cement should be stored in a dry and properly ventilated structure. All storage facilities should provide easy access for inspection and identification. A sufficient supply of material should be stored on site to sustain continuous operation of the mixing plant while the soil-cement is being placed.

2.3. Water

Water should be clean, fresh, and free from injurious amounts of oil, acid, salt, alkali, organic matter, and other substances deleterious to the hardening of the soil-cement. Non-potable water may be considered as a source for part or all of the water, provided such water does not adversely affect the quality of the soil-cement. In some cases seawater has been used satisfactorily, provided such water does not adversely affect the quality of the soil-cement. However, the presence of chlorides may increase early strengths (Hansen and Avera, 1978).

2.4. Admixtures

Admixtures can be in the form of either mineral or chemical. Mineral admixtures used in soil-cement shall conform to the requirements of ASTM C 618 (fly ash), ASTM C 989 (slag), and ASTM C 1240 (silica fume). Prior laboratory tests, using the desired replacement amount of admixture, should be performed to demonstrate that the specified 7-day compressive strength can be met, or a 28-day strength may be specified if the cementitious properties of the mineral admixture are to contribute. Very little durability testing has been performed on soil-cement containing admixtures. If wet-dry or freeze-thaw conditions can be expected in the field, it is recommended that these durability tests be performed in the laboratory with the admixtures proposed in the mix. Mineral admixtures can be beneficial where the soil aggregate lacks fines. A clean sand with no fines can benefit from the addition of a mineral admixture to fill voids

Chemical admixtures rarely are used in soil-cement. Although research has been conducted in this area, it has been limited to laboratory investigations (Wang, J.W.H., 1973 and Wang, M-C., 1976).

3 SOIL-CEMENT TESTING

One of the key factors that accounts for the successful use of soil-cement in water resources applications is careful predetermination of engineering control factors in the laboratory and their application during construction. The composition of soils varies considerably. These variations affect the manner in which the soils react when combined with portland cement and water. The way a soil reacts with cement is determined by simple laboratory tests made on mixtures of cement with soil. These tests determine the three fundamental requirements for soil-cement:

1. the minimum cement content needed to harden the soil to meet compressive strength and durability requirements,
2. the proper moisture content necessary to sufficiently compact the soil-cement, and
3. the density to which the soil-cement must be compacted.

Established test methods from the American Society for Testing and Materials (ASTM) and the American Association of State Highway and Transportation Officials (AASHTO) are used to determine these factors. Invaluable as these standards are, they require considerable time to obtain the factors needed for construction. To reduce this time, the Portland Cement Association (PCA) has developed a special shortcut test method for determining cement factors for sandy soils. For more detailed information about specific testing procedures, the reader is referred to the PCA publication *Soil-Cement Laboratory Handbook*, EB052.

3.1. Sampling

A representative sample of soil must be collected from the proposed construction site or borrow location (if adequate material is not found on-site). On large projects, multiple samples may be required to insure proper representation of

all potential sources. About 75 lb (34 kg) of soil are usually sufficient to run a complete series of soil-cement tests. When necessary, the sample is first dried until it is friable under a trowel. Drying may be accomplished by either air-drying or by using a drying apparatus that limits the temperature of the sample to 140°F (60°C). The soil is then separated on the 2-in. (51-mm), ¾-in. (19-mm), and No. 4 (4.75-mm) sieves. All clods are broken up or pulverized in such a way as to avoid reducing the natural size of individual particles. The soil passing the No. 4 (4.75-mm) sieve should be well mixed and stored in a covered container throughout the duration of the tests to prevent any major changes in moisture content. The quantity of material larger than 2 in. (51 mm) is not included in calculations of grain size distribution but is noted prior to being discarded. The material larger than ¾ in. (19 mm) is stored until soil-cement test specimens have been molded, after which it is usually discarded. The remainder of the material is used for testing purposes.

3.2. Aggregate Gradation

Aggregate gradations are determined using standard dry sieve analysis for material coarser than a No. 200 sieve (75 µm) using ASTM C 136. For material finer than a No. 200 sieve (75 µm), gradations are determined using ASTM C 117.

3.3. Plasticity Index

The plasticity index (PI) is defined as “the numerical difference between the liquid limit and the plastic limit; the range of moisture content within which the soil remains plastic.” The “plastic limit” is, in turn, defined as the moisture content at which a soil changes from semisolid to plastic, and the “liquid limit” as the moisture content at which the soil passes from a plastic to a liquid state. The lower the PI the less sensitive the soil is to variations in water content. Clay balls have a strong tendency to form when the plasticity index is greater than 8. For pavements and other applications not directly exposed to the environment, the presence of occasional clay balls may not be detrimental to performance. For slope protection or other applications where soil-cement is exposed to weathering, the clay balls tend to wash out of the soil-cement, leaving a “swiss cheese” appearance which can weaken the soil-cement structure (Figure 3-1).

Most soil-cement specifications call for a plasticity index to be no greater than 8. Procedures for determining liquid limit, plastic limit, and plasticity index for soils are given in ASTM D 4318.



Figure 3-1. “Swiss cheese” appearance of soil-cement with clay balls.

3.4. Moisture-Density Test

The proper moisture content and density (termed the optimum moisture content and maximum density) for soil-cement usually is determined by a moisture-density test (ASTM D 558). The test is similar to the “standard” Proctor test for soils (ASTM D 698). The test also is used in the field during construction to determine the quantity of water to be added and the density to which the mixture should be compacted. Multiple specimens with varying moisture contents are compacted in the laboratory. The corresponding moisture content

and dry density of each specimen are plotted on a graph. The plotted points are connected with a smooth line to produce a curve as shown in Figure 3-2. The moisture content at which the maximum density is obtained is called the “optimum moisture content” of the soil-cement mixture. The oven-dry weight per cubic foot (cubic meter) of the mixture at optimum moisture content is called the “maximum dry density.” The optimum moisture and maximum dry density are used for molding samples for compressive strength testing and design of freeze-thaw and wet-dry test specimens.

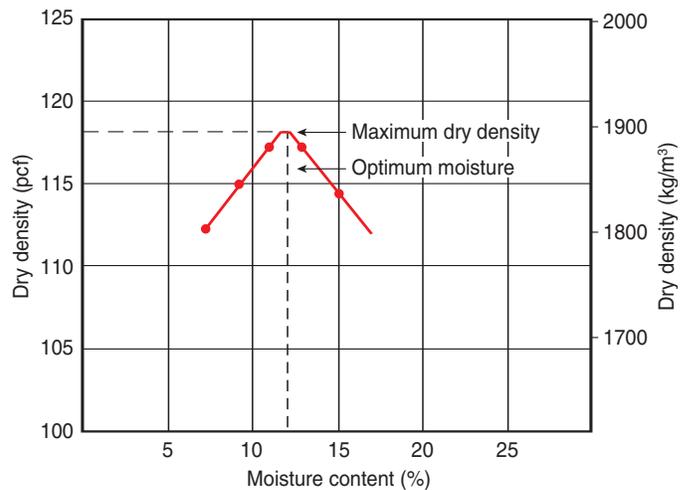


Figure 3-2. Standard Proctor curve for soil-cement.

For the same soil the “modified” Proctor test (ASTM D 1557) usually will result in a higher maximum dry density at a slightly lower optimum moisture content than the “standard” Proctor test. For granular soil the “modified” method usually will give a better correlation to what densities can be achieved in the field using vibratory rollers. Figure 3-3 shows a comparison of moisture density curves developed using both the “standard” and “modified” Proctor moisture-density tests for the same soil. The “modified” Proctor moisture-density test typically is used when the coarse fraction of the soil aggregate is such that over 30% is retained on the No. 4 (4.75-mm) sieve.

When soil, cement, and water are being mixed, the soil becomes coagulated, which causes an increase in internal friction. The moisture-density relation of the soil-cement mixture will vary slightly as a result of the soil coagulation and the partial hydration of the cement that take place during damp-mixing. These effects are noted as an increase in the optimum moisture content and a decrease in the maximum density of the soil-cement mixture as the damp-mixing time increases. For this reason, moisture-density tests on the soil-cement mixture are made in the laboratory as

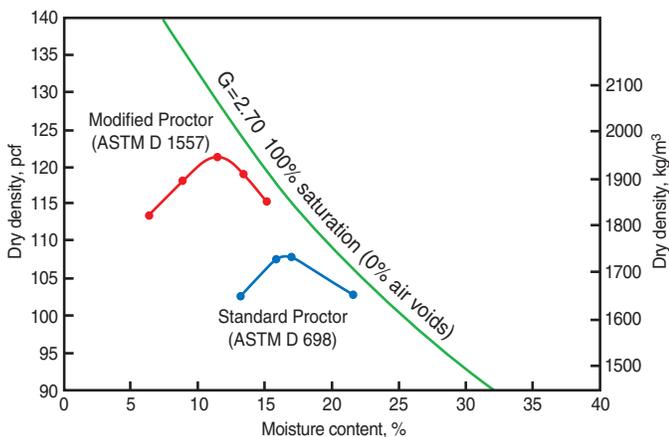


Figure 3-3. Comparison of “standard” and “modified” Proctor moisture-density curves for the same soil-cement mixture.

rapidly as possible. This is necessary because test specimens, which are designed from the laboratory test data, are molded after a few minutes of mixing soil, cement, and water and before cement hydration.

Specifications for soil-cement construction require that moisture-density relations be established in the field toward the end of damp-mixing, with soil-cement taken directly from the area being constructed. Guidelines for determining moisture-density relations of soil-cement mixtures are provided in ASTM D 558 and AASHTO T 134.

3.5. Cement Content

The cement content required will vary depending on the type of soil and its gradation. Additionally, the abrasion resistance of soil-cement is only partially dependent on cement content. Laboratory tests conducted by AGRA Earth & Environmental (1998) for the Maricopa County Flood Control District (see Section 1.3) indicate that abrasion resistance increases with increasing cement content up to a certain value, at which point the abrasion resistance levels off even as the compressive strength continues to increase. In the development of standard tests for soil-cement, it was found that repeated cycles of wetting and drying or freezing and thawing could produce forces of expansion and contraction in the soil-cement specimens. Thus the wet-dry and freeze-thaw durability tests were evolved to reproduce in the laboratory volume changes that can be encountered in the field. This helps determine whether the soil-cement will stay hard, or whether it will expand and contract excessively and soften with moisture variations and alternate freezing and thawing – conditions that produce disastrous results in untreated soils.

3.5.1. Freeze-Thaw and Wet-Dry Tests

In order to determine adequate cement content, durability tests are conducted on test specimens of soil-cement mixtures prepared at optimum moisture content. The tests are conducted using an estimated cement content and also on mixtures with cement contents two percentage points above and below the estimated cement content. Total testing time is approximately one month.

The freeze-thaw tests consist of 24 hours of freezing at -10°F (-5°C) and 24 hours of thawing at 70°F (21°C) and 100% relative humidity. Then the specimens are brushed with a wire scratch brush over the entire surface and placed back in the freezer to start another freeze-thaw cycle (48 hours). A total of 12 cycles are conducted on the specimens, after which the specimens are dried to a constant weight at 230°F (110°C) and weighed to determine their oven-dry weights. The oven-dry weight includes water used for cement hydration. The oven-dry weight must be corrected for this retained water. Chapter 3, “Details of Soil-Cement Test Methods,” of PCA publication EB052, *Soil-Cement Laboratory Handbook* includes an example of oven-dry weight correction.

The wet-dry tests are conducted in a similar fashion to the freeze-thaw test except that the specimens are submerged in tap water at room temperature for a period of 5 hours and then removed. Then the specimens are placed in an oven at 160°F (71°C) for 43 hours and removed and brushed in the same manner as for the freeze-thaw test. Again, the samples are subjected to a total of 12 cycles (48 hours each) and then dried to a constant weight at 230°F (110°C) and weighed to determine the oven-dry weight (corrected for cement hydration).

The freeze-thaw and wet-dry tests determine the minimum cement content required to produce a structural material that will resist volume changes produced by changes in moisture and temperature. Since moisture and temperature changes occur in varying degrees in all climatic and geographic areas, use of both the freeze-thaw and wet-dry tests assures that an adequately hardened, durable material is produced for any climate area. If the soil-cement project is to be located in a geographic area with little or no freeze-thaw cycles, then the freeze-thaw test is optional, unless an unusual soil or soil gradation is encountered. Guidelines for conducting wet-dry and freeze-thaw tests of soil-cement mixtures are provided in ASTM D 559 and D 560 (AASHTO T 135 and T 136), respectively.

PCA has established an acceptance criteria for these durability tests based on various soil types (see Table 4-1).

These criteria are applicable for soil-cement used for base stabilization where the soil-cement is not directly exposed. For water resource applications where the soil-cement is directly exposed to the weather and effects of wave action and flowing water, cement contents typically are increased two percentage points to account for the exposure conditions.

3.5.2. PCA Shortcut Method

Procedures have been developed to determine adequate cement contents for sandy soils (PCA, 1992). The procedures are based on data developed from previous tests of similar soils. This eliminates some of the tests and reduces the amount of work required. The only laboratory tests required are a grain-size analysis, a moisture-density test, and a compressive-strength test. Relatively small soil samples are needed, and all the work can be completed in about one week. These procedures do not always allow the determination of the minimum cement content, but almost always provide a safe cement factor that is generally close to that indicated by standard ASTM or AASHTO wet-dry and freeze-thaw tests.

Similar to the wet-dry and freeze-thaw tests described above, the PCA shortcut method was developed for soil-cement base stabilization applications. As a result, cement contents determined from this method need to be increased two percentage points for water resource applications to allow for the more severe exposure conditions.

3.6. Pozzolan Content

Limited data are available on the effect of pozzolan — including fly ash and slag — in soil-cement on short- and long-term compressive strength and durability. It is recommended that a complete series of tests including standard wet-dry and freeze-thaw durability tests be performed during the mix design phase to verify that the samples meet the minimum performance standards. Also, because of the possible variation in fly ash characteristics, it is recommended that the fly ash used in the laboratory testing be the same as that proposed for the project.

3.7. Compressive Strength

Compressive strength tests generally are made subsequent to the freeze-thaw and wet-dry tests. Compressive strength specimens commonly are broken in compression at ages 3, 7, and sometimes 28 days. Tests are run at 3 days so that during construction the quality control engineer will have an early indication if problems are developing. They are stored at room temperature in an atmosphere of approximately

100% humidity until the day of testing. Each specimen is soaked in water for 4 hours before testing. The compression load is applied at a rate of 20 psi (0.14 MPa) per second. Results of the compressive strength tests are used to determine the rate of hardening and to determine whether the soil is reacting normally. Guidelines for making and testing soil-cement compressive test specimens are given in ASTM D 1632 and D 1633.

The influence of cement in producing compressive strength in compacted soil-cement mixtures can be analyzed from two viewpoints. The cement influences will be evidenced by increases in strength with increases in age and by increases in strength with increases in cement content. The 7-day compressive strengths of saturated specimens at the minimum cement content that produces adequately hardened soil-cement will generally be between 300 and 800 psi (2.1 and 5.5 MPa). No direct relationships exist between compressive strength and erosion resistance due to flowing water or wave action. However, specifications for soil-cement used in water resources applications typically have required minimum in-place 7-day compressive strengths between 600 and 750 psi (4.2 and 5.2 MPa). For liners, the strength requirements can be as low as 500 psi (3.4 MPa), for grade control structures 1,000 psi (6.9 MPa), and for spillways up to 2,000 psi (13.8 MPa). The exact strength requirements should be based on climate conditions, abrasion requirements, frequency of use, and bedload sediment transport conditions.

3.8. Shear Strength

Shear strength tests typically are not performed on soil-cement for most water resource related structures. However, when considering the construction of an earthen dam made from soil-cement, these tests are invaluable for design. Triaxial shear data were developed by Nussbaum and Colley (1971) for use in considering the construction of an earth dam in which all the soils are stabilized with cement. The tests were made on 2.8 x 5.6-in. (7.1 x 14.2-cm) cylindrical specimens that were compacted to standard density at optimum moisture content. Specimens were cured in a fog room at 72°F (22°C) until the tests were conducted. The tests were undrained with a rate of loading such that the test was completed in about 10 minutes. Data from the triaxial tests are shown in Table 3-1.

The cement-stabilized soils showed substantial increases in the coefficient of internal friction and cohesion when compared to the untreated soils. It was noted that cohesion increased with either cement content or curing time. Also, considerable increases in the angle of internal friction values

Table 3-1. Triaxial Strength and Cement Content after 28 Days Curing (Nussbaum and Colley, 1971)

AASHTO soil type	Percent cement by wt.	Cohesion, psi (MPa)	Slope Angle, degrees
A-2-4	0	20 (0.14)	29
	2	50 (0.34)	41
	3	58 (0.4)	44
	4	70 (0.48)	44
	6	90 (0.62)	48
	8	100 (0.69)	49
A-1-b	0	10 (0.07)	38
	1	27 (0.19)	45
	2	37 (0.26)	49
	3	50 (0.34)	51
	4	72 (0.5)	52
	5	95 (0.66)	55
A-4	0	5 (0.03)	37
	2.5	30 (0.21)	46
	5.5	65 (0.45)	45
	7.5	85 (0.59)	45
	9.5	125 (0.86)	45

were observed when the stabilized soils were compared with untreated soils; however, only small changes were noted when the amount of cement used to stabilize the soil was varied. The increase in cohesion with increased amounts of cement is significant in consideration of slope steepness and stability.

The data from the triaxial shear strength tests performed after 28 days of curing were used to compute the slope angle permissible for cement-stabilized earthen dam construction. Compared to a dam made of unstabilized material with a typical 3:1 (H:V) side slope, the volume of material used to construct a dam of the same height using 2% cement by weight to treat the soil would be approximately 60% – 70% less.

3.9. Poorly Reacting Sandy Soils

In soil-cement construction, certain types of sandy soils (typically acidic or high organic content soils) are found that cannot be treated successfully with normal amounts of portland cement. While the use of additive soils, calcium chloride, and other chemicals effectively can neutralize this adverse reaction, it is impossible to detect the presence of this type of sandy soil until after standard laboratory tests (which typically take one week to complete) have been performed. A test method, taking approximately one hour, was developed by Robbins and Mueller (1960) to detect poorly reacting sandy soils.

The test involves determining the ability of the soil to immobilize or absorb calcium. The greater the absorption, the poorer the reaction with cement. In order to measure the absorption of calcium by a soil, calcium ions must be placed in contact with the soil grains. A standard saturated and carbonate-free solution of calcium hydroxide is used as the agent. When a standard solution of calcium hydroxide is mixed with a predetermined amount of soil, the amount of calcium actually absorbed by the soil can be determined by titrating the calcium remaining in solution. This amount of calcium, when compared with that available in the standard solution, provides a direct measure of the calcium absorption ability of the soil.

Organic content and low pH do not in themselves constitute an indication of a poorly reacting sand. However, a sandy soil with an organic content greater than 2% or having a pH lower than 5.3 will in all probability not react normally with cement. Typically these poorly reacting sandy soils are located in the top few feet of the surface. Oftentimes these soils can be stripped and wasted and the underlying soils used in the soil-cement. Also, the use of a small percentage of calcium chloride (CaCl_2) on the order of 0.6 to 1.0 percent by weight of dry soil or blending normally-reacting aggregate into the mixture have been effective in minimizing the adverse reaction from poorly reacting sandy soils.

4 MIX PROPORTIONS

4.1. Mixture Design Criteria

The principal structural requirements of a hardened soil-cement mixture are based on adequate strength and durability such that it withstands exposure to the elements. Various criteria are used by different organizations to determine acceptable mix proportions. The following are soil-cement mix guidelines developed by the Portland Cement Association, U.S. Army Corps of Engineers, U.S. Bureau of Reclamation, Maricopa County, AZ, Pima County, AZ, and Orange County, CA.

4.1.1. Portland Cement Association

The primary basis of design of soil-cement mixtures is the cement content required to produce a mixture that will withstand the stresses induced by the wet-dry and freeze-thaw tests. Soil-cement has been used as base course for pavements for more than 70 years. Mixture designs for many of these projects have been based on the wet-dry and freeze-thaw tests or the PCA shortcut procedures. The service record of these projects in use proves the reliability of the results based upon these tests and the criteria in Table 4-1.

These criteria provide the minimum cement content necessary to produce a hard, durable soil-cement suitable for pavement base coarse construction or for water resources applications that are 5 ft (1.5 m) or more below the minimum water surface elevation. For water resources applications above this elevation, the minimum cement content should be increased by 2% unless laboratory tests show otherwise. Additionally, the maximum volume change during the durability tests should be less than 2% of the initial volume, and the compressive strength should increase with the age of the specimen.

Soil for use in soil-cement construction should not contain any material retained on a 2-in. (50-mm) sieve, nor more than 45% retained on a No. 4 (4.75-mm) sieve, nor more than 35% or less than 5% passing the No. 200 (0.075-mm) sieve. The distribution and gradation of materials in the soil-cement shall not result in lenses, pockets, streaks, or layers of material differing substantially in texture or gradation from surrounding material.

Table 4-1. Maximum Allowable Weight Loss Criteria by Soil Type from Wet-Dry and Freeze-Thaw Tests (PCA 1992)

PCA criteria for soil-cement as indicated by wet-dry and freeze-thaw durability tests		
AASHTO soil group	Unified soil group	Maximum allowable weight loss (%)
A-1-a	GW, GP, GM, SW, SP, SM	14
A-1-b	GM, GP, SM, SP	14
A-2*	GM, GC, SM, SC	14
A-3	SP	14
A-4	CL, ML	10
A-5	ML, MH, CH	10
A-6	CL, CH	7

*10% is maximum allowable weight loss for A-2-6 and A-2-7 soils

4.1.2. U.S. Army Corps of Engineers

The U.S. Army Corps of Engineers Technical Manual *Soil Stabilization for Pavements*, TM 5-822-4, and Engineering Manual *Design and Construction of Levees*, EM 1110-2-1913, provide mixture design criteria for soil-cement. The durability requirements for portland cement stabilization are given in Table 4-2.

Table 4-2. Maximum Allowable Weight Loss Criteria by Soil Type from Wet-Dry and Freeze-Thaw Tests (USACE)

Type of soil stabilized*	Maximum allowable weight loss after 12 wet-dry or freeze-thaw cycles (% of initial specimen weight)
Granular, PI<10	11
Granular, PI>10	8
Silt	8
Clays	6

*Refer to USACE Documents MIL-STD-619B and MIL-STD-621A.

The USACE frequently increases the cement content by 1% or 2% over that determined from the durability requirements to account for variations in the field.

The minimum design requirements for soil-cement bank protection shall be such that it has a minimum compressive strength of 600 psi (4.2 MPa) at the end of 7 days and 875 psi (6 MPa) at the end of 28 days as tested in accordance with ASTM D 1633. A project completed by the Los Angeles District Corps of Engineers at Norco Bluffs in southern California called for a minimum compressive strength of 725 psi (5 MPa) and a maximum compressive strength of 1,000 psi (6.9 MPa) at the end of 7 days.

The cementitious portion of the soil-cement mix shall consist of portland cement conforming to the requirements of ASTM C 150 for Type I or Type II cement unless exposed to high sulfates. The use of fly ash generally is not recommended especially when early-age strength and durability are required. However, in the recently completed Norco Bluffs project, the specifications allowed for fly ash to be mixed with the portland cement, but it could not exceed 15% by weight of the total cementitious material and had to conform to the requirements of ASTM C 618, Class F.

At the Norco Bluffs project, the soil used in the soil-cement mix had to conform to the gradation shown in Table 4-3 when tested in accordance with ASTM C 136 and C 117 and had to be free of any deleterious material. Appendix G in EM 1110-2-1913 shows the USACE typical gradation limits for soil-cement.

Table 4-3. USACE Gradation Requirements for Aggregate Used in Soil-Cement

Standard sieve size	% passing (dry weight)	
	Minimum	Maximum
1½ inch (37.5 mm)	98	100
No. 4 (4.75 mm)	60	90
No. 200 (75 µm)	5	10

The maximum plasticity index (PI) for soil-cement aggregate material can be as high as 12 (see EM 1110-2-1913) when determined in accordance with ASTM D 4318, but generally better quality performance is achieved with non-plastic soils. At Norco Bluffs, specifications called for a PI of not greater than 3.

Retarding admixtures, if used, shall conform to ASTM C 494, Type B or D. Water shall conform to the requirements of COE CRD-C 400 and shall be clean, fresh, and free from injurious amounts of oil, acid, salt, alkali, organic matter, and other substances deleterious to the hardening of the soil-cement.

4.1.3. U.S. Bureau of Reclamation

There are no typical guide specifications for soil-cement available from the Bureau of Reclamation. However, a summary report documenting the performance of soil-cement projects constructed by the Bureau between 1963 and 1969 is available (DeGroot, May 1971). This document provided a basis for the general specifications given in the following paragraphs. Many were derived from the specifications developed from the Bonny Reservoir Test Section completed in 1951.

Soil-cement for use as slope protection should have a minimum compressive strength of 600 psi (4.2 MPa) at the end of 7 days and 875 psi (6 MPa) at the end of 28 days. The cementitious portion of the soil-cement mix should be Type I or Type II portland cement.

Very little soil-cement work has been done by the Bureau on clay soils which have high plastic fines. The Bureau recommends a maximum PI of 8 for soil used in soil-cement. In addition, the soil aggregate should not contain clay or silt lumps larger than 1 in. (25 mm).

4.1.4. Maricopa County, Arizona

The design specifications for soil-cement bank protection require a minimum compressive strength of 750 psi (5.2 MPa) at the end of 7 days when tested in accordance with requirements of Arizona Test Method 241a. This method determines the percentage of cement required in the soil-cement mixture by determination of compressive strength of molded specimens at varying cement contents. No durability tests are performed for determining the cement content. The cementitious portion of the soil-cement mix shall consist of either 100% portland cement conforming to the requirements of ASTM C 150 for Type I or Type II cement or no less than 85% portland cement and no more than 15% fly ash. Fly ash shall meet specific requirements for Class F pozzolan as specified in ASTM C 618. An additional 2% cementitious material may be required to be added above that which was determined during the initial mix design to compensate for variations in field construction conditions. The soil used in the soil-cement mix shall not contain any material retained on the 2-in. (51-mm) sieve. The soil shall conform to the gradation requirements in Table 4-4 when tested in accordance with ASTM C 136 (AASHTO T-27) and ASTM C 117 (AASHTO T-11).

The plasticity index (PI) shall not exceed 3 when tested in accordance with the requirements of ASTM D 318 (AASHTO T-90). The soil aggregate shall not contain clay or silt lumps larger than 1 in. (25 mm). Water shall be clear and free from injurious amounts of oil, acid, alkali, organic matter, or other deleterious substances. Water shall contain not more than 1,000 parts per million of chlorides as Cl or of sulfates as SO₄. Water shall be sampled and tested in accordance with the requirements of AASHTO T-26.

Table 4-4. Maricopa County Gradation Requirements for Aggregate Used in Soil-Cement

Standard sieve size	% passing (dry weight)	
	Minimum	Maximum
2 inch (51 mm)	–	100
No. 4 (4.75 mm)	50	80
No. 200 (75 µm)	5	20

4.1.5. Pima County, Arizona

The design specifications for soil-cement bank protection require a minimum compressive strength of 750 psi (5.2 MPa), unless otherwise specified, at the end of 7 days. The cementitious portion of the soil-cement mix shall consist of either 100% portland cement conforming to the requirements of ASTM C 150 for Type I or Type II cement or no less than 85% portland cement and no more than 15% fly ash. The design shall be performed in accordance with Pima County Department of Transportation and Flood Control District's procedure *Determination of Cement Content Required for Soil-Cement Mixtures, Modified Arizona 220 Test Method* (PCDOTFCD, 1987). This method determines the percentage of cement required in the soil-cement mixture by determination of compressive strength of molded specimens at varying cement contents. No durability tests are performed for determining the cement content. Also, Pima County no longer requires a 2% increase in cement content for water resource projects as long as the target compressive strength of 750 psi (5.2 MPa) at 7 days is met.

The soil used in the soil-cement mix shall not contain any material retained on the 2-in. (51-mm) sieve, nor any deleterious material. The soil shall conform to the gradation requirements in Table 4-5. The plasticity index (PI) shall be a maximum of 3. Clay and silt lumps larger than ½ in. (12 mm) shall be unacceptable, and screening will be required whenever this type of material is encountered. The water used shall be free from injurious amounts of oil, acid, alkali, clay, vegetable matter, silt, or other harmful matter. Water shall contain no more than 1,000 parts per million of chlorides as Cl or of sulfates as SO₄.

Table 4-5. Pima County Gradation Requirements for Aggregate Used in Soil-Cement

Standard sieve size	% passing (dry weight)	
	Minimum	Maximum
2 inch (51 mm)	95	100
No. 4 (4.75 mm)	50	90
No. 200 (75 µm)	5	15

4.1.6. Orange County, California

The design specifications for soil-cement bank protection require a minimum compressive strength of 750 psi (5.2 MPa) at the end of 7 days plus 2% additional cement added for erosion protection. For grade control structures, the design specifications for soil-cement require a minimum compressive strength of 1,000 psi (6.9 MPa) at the end of 7 days, plus 2% additional cement for erosion protection. The cementitious portion of the soil-cement mix shall consist of 100% portland cement conforming to the requirements of ASTM C 150 for Type II cement. Fly ash or other substitutes for portland cement shall not be permitted. The design shall be performed in accordance with California Test Method 312, Part IV-B. Preliminary design samples shall be used for determining the percentage of portland cement required in developing soil-cement mixtures. The compressive strength of molded specimens at varying cement contents shall be determined by California Test Method 312, Part V, *Determining Compressive Strength*. No durability tests are performed for determining the cement content.

The soil used in the soil-cement mix shall not contain any material retained on the 1.5-in. (37.5 mm) sieve, nor any deleterious material. The soil shall conform to the gradation requirements in Table 4-6. The sand equivalent shall be a minimum of 15 with a minimum moving average of 20. The sand equivalent shall be determined by California Test Method 217. The distribution and gradation of materials in the soil-cement lining shall not result in lenses, pockets, streaks, or layers of material differing substantially in texture or gradation from surrounding material. The water used shall be free from injurious amounts of oil, acid, alkali, organic matter, or other deleterious substances. Water shall contain no more than 1,000 parts per million of chlorides as Cl or of sulfates as SO₄. Water shall be sampled and tested in accordance with the requirements of AASHTO T-26.

Table 4-6. Orange County Gradation Requirements for Aggregate Used in Soil-Cement

Standard sieve size	% passing (dry weight)	
	Minimum	Maximum
1.5 inch (37.5 mm)	98	100
No. 4 (4.75 mm)	60	90
No. 200 (75 µm)	5	20

4.2. Laboratory Tests

In order to achieve the proper combination of aggregate, cementitious material, and water to develop a strong erosion-resistant soil-cement that can be used in water resources applications, extensive laboratory tests are conducted using the materials planned for use. These tests are discussed in Section 3 and include moisture-density test (ASTM D 558 or D 1557) to determine the optimum moisture to achieve the maximum density of the compacted soil-cement mixture, the wet-dry and freeze-thaw tests (ASTM D 559 and D 560) to determine the minimum cement content that provides the proper durability, and the compressive strength test (ASTM D 1633), also used to determine the minimum cement content.

While the durability tests are still the most valid for determining cement content for soil-cement mixtures, running both series of tests takes about one month each and tends to be expensive. To reduce time and costs associated with soil-cement testing, it was determined that for granular soils, the freeze-thaw test typically produces a greater weight-loss than the wet-dry test; therefore, use of only the freeze-thaw test provides the most conservative results.

The Portland Cement Association conducted over 1,700 sets of durability tests on a wide range of soils, developing a relationship between 7-day compressive strength and durability. Figure 4-1 shows the percent of samples passing the standard ASTM wet-dry and freeze-thaw tests plotted against

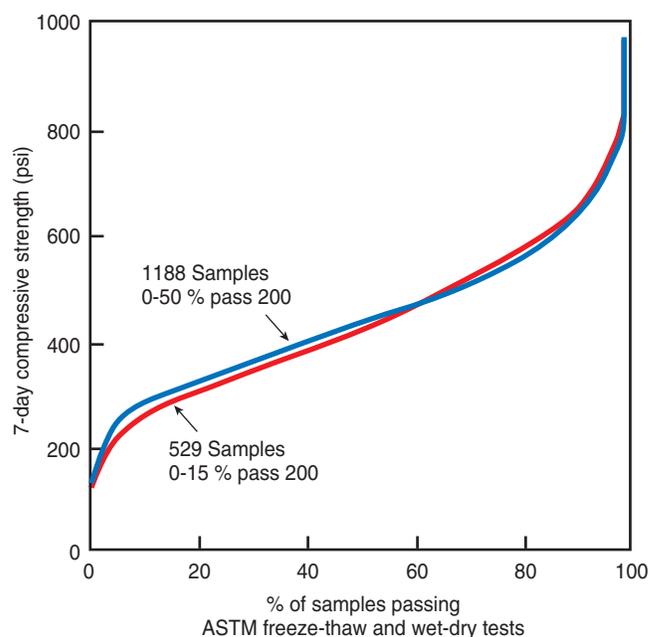


Figure 4-1. Compressive strength vs. durability curve (ACI, 1990).

the 7-day compressive strength of the lab specimens. This figure indicates that, on average, 87% of sandy soil samples that achieve a 600 psi (4.1 MPa) 7-day compressive strength will pass the durability tests. Similarly, about 97% of the soils that achieve a 750 psi (5.2 MPa) 7-day compressive strength also will pass the durability tests. Results of the testing indicate that soils which have a high compressive strength and a less than adequate durability are coarse mixtures with a low amount of, or poorly graded, fine sand. The strength versus durability relationship led to the adoption of the 7-day compressive strength criteria for determining adequate cement content in the laboratory.

4.3. Physical Properties

4.3.1. Strength

The strength of soil-cement will depend upon the properties of the aggregate (soil), the cement content, compaction, and curing conditions. Soil-cement strength is measured by compressive strength tests. Guidelines for making and testing soil-cement compressive strength test specimens are given in ASTM D 1632 and D 1633. Curing time influences strength gain differently depending on the type of soil. Granular soils typically have a greater strength increase than fine-grained soils over the same curing period. Also, compressive strength generally increases with increasing cement content. Compressive strength requirements will vary by project but typically range between 500 and 750 psi (3.4 and 5.2 MPa) at the end of 7 days for bank protection and 1,000 psi (6.9 MPa) for grade control structures. Soil-cement will continue to gain strength over time. Figure 4-2 shows results from tests conducted on cores of soil-cement taken from 1 to 10 years after project completion. They indicate that the compressive strength is nearly double the 28-day compressive strength.

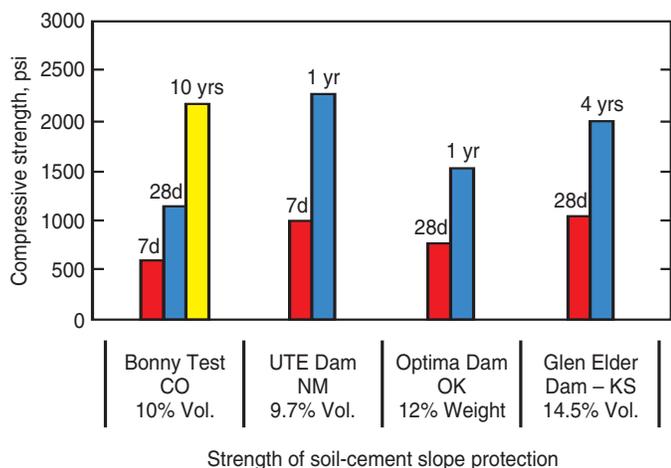


Figure 4-2. Long-term compressive strength test results.

4.3.2. Erosion and Abrasion Resistance

Erosion resistance of soil-cement is defined as the ability to resist disintegration by the action of water. Abrasion resistance is the ability to resist disintegration by the action of water and waterborne solid particles acting upon exposed surfaces. Results of numerous evaluations (see Section 1.3) suggest that erosion abrasion resistance of soil-cement exposed to waterborne particles can be improved by using coarser material as the aggregate, adding gravel to a finer sandy soil, and/or by increasing the cement content.

4.3.3. Shrinkage

Cement-treated soils undergo shrinkage during hydration and drying. The shrinkage and subsequent cracking depend on several factors including cement content, soil type, water content, degree of compaction, and curing conditions. Research by Marchall (1954) indicates that soil-cement made with clays develops higher total shrinkage, but crack widths are smaller and individual cracks are more closely spaced. Soil-cement made with granular soils produces less shrinkage, but larger cracks spaced at larger intervals. Methods suggested for reducing or minimizing shrinkage cracks include reducing the amount of fine material in the mixture, optimizing the cement content (not using more cement than necessary), keeping the soil-cement surface moist beyond the normal curing period, and placing the soil-cement at slightly below the optimum moisture content.

4.3.4. Permeability

Permeability of most soils is reduced by the addition of cement. Laboratory tests conducted by the Portland Cement Association were made on a variety of sandy soil types (ACI, 1990). Results of the tests indicate that for many sandy soils, the permeability of soil-cement can be reduced dramatically with permeabilities as low as 10^{-8} cm/sec., using the amount of cement required for durability acceptance. However, overall permeability considers not only permeability of the material but includes permeability through shrinkage cracks and construction joints in the soil-cement and permeability along lift surfaces of multiple-lift construction. A large scale seepage test was conducted by the U.S. Bureau of Reclamation on a section of layered stair-step soil-cement facing the Lubbock Regulating Reservoir in Texas. Results by DeGroot (February 1971) indicated a decrease in permeability with time, possibly due to shrinkage cracks in the soil-cement filling with sediment and the tendency of the cracks to self-heal. Seepage was as much as 10 times greater in the cold winter months than the hot summer months. The reduced

summer seepage was probably caused by thermal expansion which narrowed the crack widths and by the presence of algae growth in the cracks.

In multiple-lift construction, higher permeability generally can be expected along the horizontal surfaces of the lifts than perpendicular to the lifts. Research by Nussbaum and Colley (1971) indicated that permeabilities for flow parallel to the compaction plane were 2 to 20 times higher than values for flow normal to the compaction plane.

5 DESIGN ISSUES

5.1. General

Soil-cement has been utilized for many water resources applications. However, design issues are different for each application and must be addressed accordingly. The different applications and design aspects of each are discussed in the following sections.

5.2. Bank and Levee Protection

Design issues that must be addressed for soil-cement bank protection include: establishing bank alignment and section dimensions; computing top-of-bank and toe-down elevations; evaluating upstream and downstream river and floodplain conditions and how those conditions may impact the proposed channel; addressing impacts of existing and proposed sand and gravel mining operations; and additional design features such as providing for access ramps into the channel, providing for tributary and side drainage into the channel, providing penetrations for structures or utilities, assessing the impact of high groundwater and resulting hydrostatic pressures, and adequate tie-ins for grade-control structures and terminations of bank protection. Figure 5-1 shows soil-cement bank protection along Rillito River, Tucson, Arizona.

5.2.1. Location/Alignment

The bank alignment is determined by a variety of factors including: required hydraulic section, right-of-way constraints, environmental constraints, and location of facilities to be protected.

5.2.2. Hydraulic Conditions

The hydraulic conditions in the channel will determine the water depth and design velocities to which the bank protection will be exposed. This information is also important for



Figure 5-1. Bank Protection at Rillito River in Tucson, Arizona.

determining potential scour that must be accounted for in the bank protection design if a “soft” bottom alternative is selected.

5.2.2.1. Water Elevation

The water elevation, determined by hydraulic modeling, discharge measurements, stage-discharge rating curves, or other means will determine the minimum height for bank protection. The design water elevation will be based on a design discharge. This is the discharge for which the bank protection will be designed not to fail and is usually the 100-year recurrence interval flood event. However, Los Angeles County requires bank protection be designed to withstand the Capital Flood, which is in excess of the 500-year recurrence interval flood.

5.2.2.2. Water Velocity

The water velocity is needed to determine whether the soil-cement bank protection will be adequate to resist potential excessive velocities (and erosive forces) in the channel.

Velocity is also an important factor in determining the shear stress on the channel bed that is responsible for scour and potential undermining of the bank protection.

5.2.2.3. Flow-Duration

The time period or duration that soil-cement bank protection is exposed to high flows is an important consideration for design. The longer the period of exposure to high flows, the greater the potential for seepage to increase hydrostatic pressure landward of the bank protection, potentially affecting its stability when flows in the channel are reduced. Additionally, the more frequently high flows occur, the more often the soil-cement is exposed to abrasion by sediment-carrying flows.

5.2.3. Section Dimensions

Section dimensions of soil-cement bank protection include the height, toe-down, bank slope, layer thickness, and horizontal width. The height and toe-down are determined by a variety of factors and are presented in the following paragraphs. The bank slope is typically 1:1 to 3:1 (H:V) depending on right-of-way constraints, stability of the cut slope, and hydraulic efficiency requirements. The compacted layer thickness for stairstep soil-cement construction generally has been 6 to 12 in. (150 to 300 mm) and is determined by the ability of the compacting equipment to achieve the proper density in the soil-cement layer. The horizontal width usually is determined by the equipment used to place and compact the soil-cement. Most soil-cement bank protection projects utilize standard road construction equipment that requires the horizontal width to be at least 8 ft (2.4 m).

5.2.3.1. Height

The top-of-bank elevation is established based on the design water surface elevation plus super elevation, freeboard requirements, and potential future aggradation in the channel. Some designs, such as San Diego Creek in Irvine, California, use a two-tiered approach such that near the main channel the soil-cement protects for up to the 2-year flood, while further away from the channel another section of soil-cement protects up to the 100-year flood (see Figure 5-2). Similar in design, the Bridgeport Project in Santa Clarita, California (Figure 5-3), used multiple sections of soil-cement bank protection with lateral drop structures between the bank protection sections to prevent erosion along the floodplain. The area between the sections of soil-cement bank protection was backfilled, burying the upper soil-cement section, to give a more natural appearance.

- **Water Surface Elevation** — The design water surface elevation is established by adding the water depth as discussed in Section 5.2.2.1 to the design invert elevation of the channel.

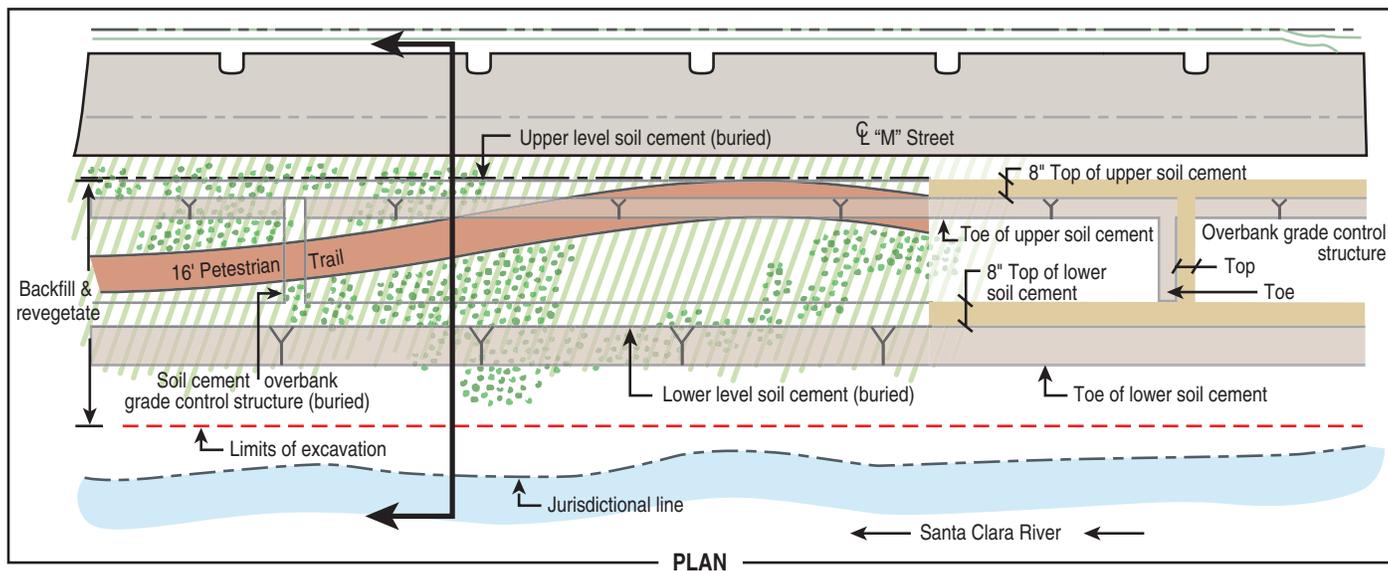
- **Super Elevation** — Super elevation occurs as a result of the centrifugal force induced by flow around a curve. This causes a rise in the water surface along the outside of the curve and a depression in the water surface along the inside of the curve. The amount of super elevation is usually small for the channel size and curvature commonly used in the design of tranquil flow channels. The main problem in channels designed for rapid flow is standing waves generated by simple curves (Figure 5-4). These waves not only affect the curved flow region but exist over long distances downstream. The total rise in water surface for rapid flow has been found experimentally to be about twice that of tranquil flow. Super elevation can be estimated using guidelines developed by the Corps of Engineers and presented in Engineering Manual EM 1110-2-1601 (USACE, 1994).

- **Freeboard** — The top-of-bank elevation is generally based on the Federal Emergency Management Agency (FEMA) guidelines requiring 3 ft (0.9 m) of freeboard above the 100-year water surface elevation. In the highly alluvial rivers of the desert southwest, establishment of the top-of-bank elevation must also address the potential for deposition in the channel. Insufficient freeboard could lead to overtopping and failure of the material landward of the soil-cement bank protection (Figure 5-5).



Figure 5-2. Two-tiered bank protection along San Diego Creek in Irvine, California.

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Patent No.: US6,168,349—issued: 01/02/01

Figure 5-3. Typical plan and section of Bridgeport Project in Santa Clarita, California (Advanced Pacific Civil Engineering).



Figure 5-4. Standing waves along Santa Cruz River, Arizona.



Figure 5-5. Erosion of bank material landward of soil-cement due to overtopping by flood flows, Santa Cruz River, Arizona.

- **Aggradation**—Aggradation occurs when the sediment transport capacity of the channel cannot keep up with the supply from upstream. This most often occurs at locations where the channel slope transitions from steep to mild, resulting in a reduction in the sediment transport capacity. These locations generally have a history of aggradation and are often the site of gravel mines. Aggradation also may be a result of an increase in sediment supply from the watershed. This can occur as a result of a sudden disturbance in the watershed such as a fire or long-term disturbance such as increased development. Both local and watershed scale evaluations should be conducted to determine the potential for aggradation in the channel over the design life of the bank protection.

5.2.3.2. Toe-down

The toe of the soil-cement bank protection will almost always need to be placed below the channel invert. If the bed of the channel is left undisturbed or will consist of natural materials (sand and gravel), it will be subject to both aggradation and degradation during large flow events. Severe short-term degradation or scour can occur resulting in the undermining and subsequent failure of the bank protection if the toe is not placed at sufficient depth.

The toe-down elevation for soil-cement bank protection is determined by subtracting from the design invert elevation the estimated total depth of scour for the design event, typically the 100-year event.



Figure 5-6. Toe excavation at upstream terminus of Tri-City Landfill soil-cement bank protection, Arizona.

5.2.4. Scour

The total depth of scour is established by summing the applicable scour components which include general scour, low-flow incisement, bend scour, long-term aggradation or degradation, bedform scour, and local scour (i.e., grade control structures, bridge piers, utilities, etc.) and a safety factor. The safety factor usually is computed as 30% of general scour, bed form, bend scour, and local scour. The safety factor accounts for the nonuniform flow distributions that are typical of alluvial channels.

5.2.4.1. General Scour

General scour refers to the vertical lowering of the channel bed over relatively short time periods, for example, the scour in a given reach during passage of a single flood event. The general scour component is calculated as the difference between the minimum bed elevation that occurs during the passage of the flood hydrograph and the initial bed elevation. A sediment routing model such as HEC-6 can be used to estimate the amount of material either transported or deposited.

5.2.4.2. Low-flow Incisement

Low-flow incisement refers to the small incised channel which forms within the bed of the main channel and in which low discharges are conveyed. Low-flow incisement occurs when the width/depth ratio of the main channel is large. Rather than flow in a very wide, shallow depth, low flows will develop a low-flow channel in order to provide more efficient conveyance of these discharges. There are no rigorous methodologies for the prediction of low-flow channel incisement. A review of existing field conditions and experience from other projects in the vicinity may assist in estimating this component

5.2.4.3. Bend Scour

Bends associated with meandering channels will induce transverse or “secondary” currents which will scour sediment from the outside of the bend and cause it to be deposited along the inside of the bend. Bend scour is calculated using a relationship presented in Arizona Department of Water Resources (ADWR) 1985 manual titled *Design Manual for Engineering Analysis of Fluvial Systems*. (ADWR, 1985)

5.2.4.4. Long-term Aggradation or Degradation

Long-term aggradation/degradation is a function of the dominant discharge. Dominant discharge is defined as the

discharge which, if allowed to flow constantly, would have the same overall channel-shaping effect as the natural fluctuating discharges. In the southwestern United States, the dominant discharge typically is taken as the 10-year event. The long-term scour component may be limited by one of two processes: armoring or equilibrium slope. The governing process may be determined by using procedures in Pemberton and Lara (1984).

5.2.4.5. Bed Form Scour

Bed forms result from the interaction of the flowing water and bed material in the channel. These bed forms can deform a channel bed, potentially undermine flood control structures, and significantly change the hydraulic characteristics of a channel in terms of its hydraulic roughness and sediment transport capability. Bed forms may consist of either dunes or antidunes, and bed form scour is estimated by calculating the dune or antidune height. The actual type of bed form present is a function of the flow regime. Bed form scour due to dunes is estimated to be one-half the dune height as determined by Allen's relationship (Simons & Senturk, 1992). Antidune scour is determined using a simplified form of Kennedy's 1963 equation. Kennedy's equation is based on an anticipated scour depth equal to one-half the height of an antidune, measured from trough to crest.

5.2.4.6. Local Scour

Local scour occurs when flow-induced vortices remove sediment from a localized region around an obstructing object. Local scour will occur around piers, grade-control structures, and storm drain outlets. Procedures and equations found in the Federal Highway Administration's (FHWA) Hydraulic Engineering Circular #18 "Evaluating Scour at Bridges" (FHWA, 2001) should be used to determine contributions from local scour at bridges. Local scour downstream of grade-control structures and storm drain outlets, etc. should be estimated from scour prediction methods applicable to the given structure.

5.2.4.7. Mining

Sand and gravel mining is an additional consideration when establishing bank protection toe-down depths. The scour and deposition problems associated with sand and gravel mining are very complicated and an increase in toe-down depths generally is required when mining activity is occurring or has taken place in the vicinity of the proposed bank protection. Each situation is unique and requires independent analysis. No standard equation or formula can be adopted which is universally applicable to all situations.

5.2.5. Transitions/Penetrations

After determining the section dimensions, special features need to be included in the design. These features may include: access ramps, penetrations, local scour enhancements, tie-ins and tie-backs, grade-control structures, construction around existing features, and transitions into existing bank protection. Ramps are typically 12 to 16 ft (3.7 to 4.9 m) wide and should be sloping downstream. Penetrations for existing or proposed piers, storm drain outlets (Figure 5-7), and utilities must be taken into consideration. Storm drain penetrations may require increases in toe-down depths, and rip rap or concrete aprons to prevent undermining the embankment toes.



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Figure 5-7. Construction of storm drain pipe penetration in soil-cement bank protection.

Piers located near embankment toes may require remedial measures, for example, shelves around piers to protect the bank protection against local scour influences (Figure 5-8).

Grade-control structures must be tied into the bank protection or keyed into the natural watercourse bank at an adequate distance. Bank protection toe-downs on the downstream side of the grade-control structure must be increased to account for local scour influences. Upstream and downstream termini of the bank protection must be properly keyed into high ground to prevent flow from outflanking the soil-cement stabilized bank and causing erosion behind the bank protection. Tie-ins and tie-backs of the upstream and downstream terminations must consider right-of-way constraints, outflanking by channel migration, and constructability issues. Also, consideration must be given to existing bank protection. Soil-cement bank protection should make a smooth transition into any existing bank protection (Figure 5-9).

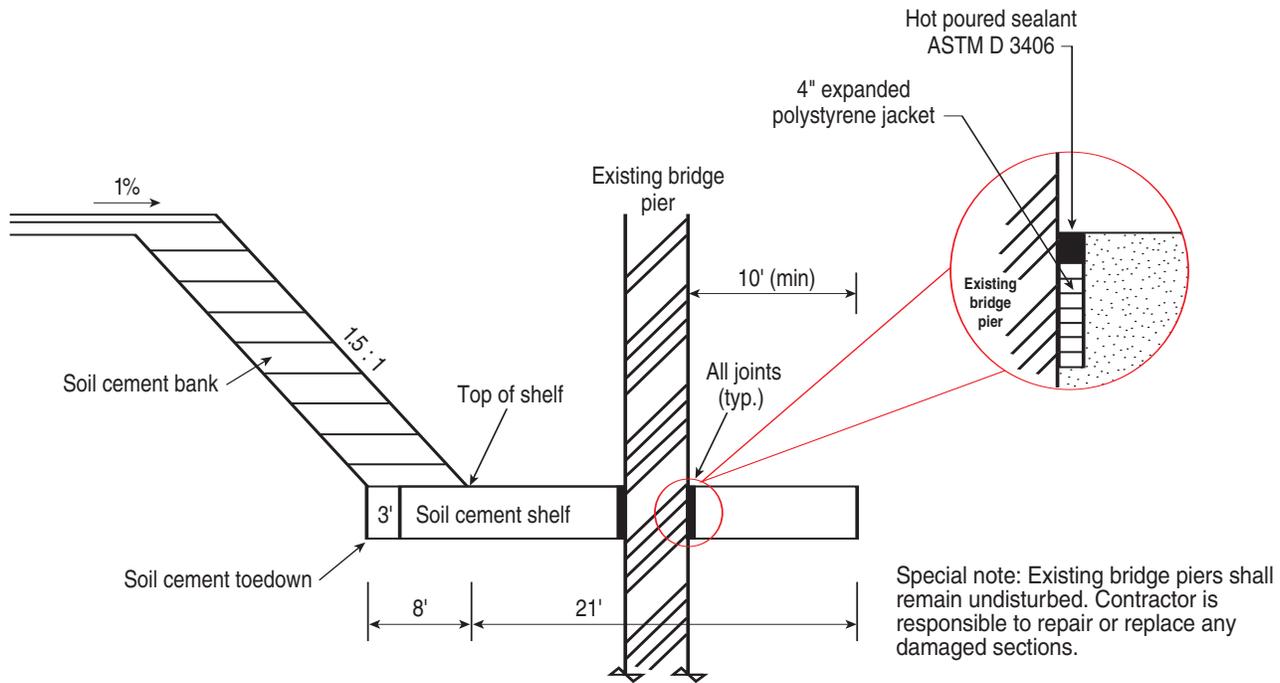


Figure 5-8. Example of pier shelf detail (Richards and Pauley, 1996).



Figure 5-9. Transition from soil-cement to concrete block bank protection, San Diego Creek, Irvine, California.

Bank protection for bridge abutments and approaches should extend both upstream and downstream from the bridge so that the soil-cement is not outflanked during a storm event. This distance may vary between 200 to 500 ft (60 to 150 m) depending on velocities, duration, and topography.

5.2.6. Upstream/Downstream

The river system's responses to the effects of bank armoring and channelization on the reaches upstream and downstream of the project need to be assessed. The effect of bank armoring, improved cross-sectional geometry, and channel straightening will impact the sediment transport characteristics of the system well beyond the limits of the project.

The continuity of the sediment supply from upstream may be changed as a result of the channel improvements. This could result in either additional aggradation or degradation within the improved channel reach. Also, reaches downstream of the channel improvements may see increases in erosion of unprotected banks or increased channel degradation while reaches upstream of the channel improvements may see changes in lateral migration trends (Hansen and Lynch, 1995).

5.3. Slope Protection

Design issues that must be addressed for soil-cement slope protection include: slope angle, facing dimensions (plate or stair step construction), freeboard (that accounts for wave runup), geotechnical stability, seepage, and interior drainage requirements.

In 1966 soil-cement was used as upstream slope protection for Eagle Creek Dam in Indianapolis, Indiana. In addition to



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Figure 5-10. Soil-cement slope protection on Eagle Creek Dam, Indianapolis, Indiana. Note compaction marks from roller still visible 12 years after construction.

wave action from the 1500 acre (607 hectare) reservoir, the soil-cement experiences many freeze-thaw cycles annually.

5.3.1. Slope Angle

The slope angle for soil-cement slope protection for dams and reservoirs is generally much flatter than that of channel bank protection. Reservoir slopes generally range from 2.5:1 to 4:1 (H:V) depending on the embankment design.

5.3.2. Facing Dimensions

Soil-cement may be placed either in horizontal “stair-step” layers on slopes as steep as 1H:1V or parallel to the slope in one or more layers on slopes of 2.5H:1V or flatter. The parallel method is referred to as “plating.” For stair-step construction, soil-cement generally is placed in 8-ft- (2.4-m-) wide horizontal layers as this width generally will accommodate the placing equipment including dump trucks, spreaders, and compactors. Figure 5-11 shows the relationship between the slope of facing, thickness of compacted horizontal layers, horizontal layer width, and minimum facing thickness measured normal to the slope. For plating construction, a minimum compacted thickness of 8 in. (200 mm) typically is used. For reservoirs with large waves, the thickness should be increased. However, individual lift thickness should be no more than 12 in. (300 mm) to allow for adequate compaction. When wave action is severe, the slope should be constructed using the stair-step method with a 2 ft (0.6 m) minimum thickness measured normal to the slope.

For many channel bank protection projects where the slope of the channel is steep, i.e., 2H:1V, the thickness of the soil-cement can be as much as 3.2 ft (1.0m). It is not usually necessary for “stair-step” construction to have such a large final compacted (normal to the slope) thickness. This usually

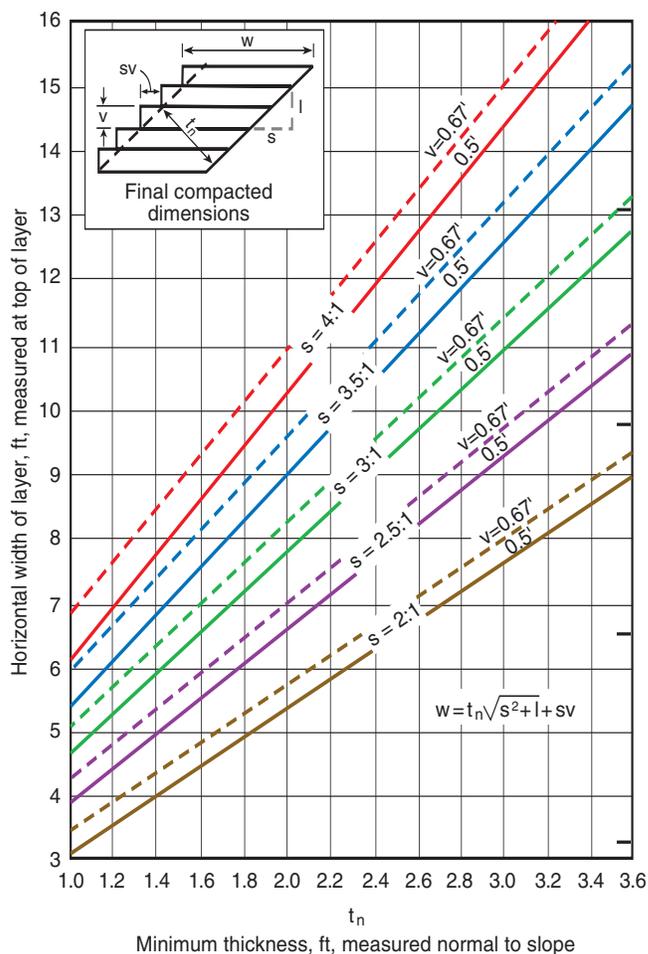


Figure 5-11. Relationship of slope, facing thickness, layer thickness, and horizontal layer width (PCA, 1991).

occurs because of the necessity of accommodating large construction equipment such as dump trucks. However, at Jackson Lake Dam in Colorado, the contract was bid with horizontal widths of 6 ft (1.8 m) and 8 ft (2.4 m) with the 6-ft (1.8-m) widths coming in less expensive. To accommodate the narrower width, the contractor developed a plan where trucks dumped soil-cement into a skid bucket. A large excavator retrieved the soil-cement from the bucket and placed it on the lift surface. Figure 5-12 shows the placement scheme at Jackson Lake Dam. A similar approach for bidding was done in Pima County, but the bid prices for the two different widths were basically the same because the reduction in material costs for the narrower width was offset by the reduced production rates compared to the standard width of 8 ft (2.4 m).

Another common practice of narrowing the width of soil-cement layers is to use a conveyor belt system to deliver material to the placement area. Conveyor belts can operate



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Figure 5-12. Placement of soil-cement at Jackson Lake Dam, Colorado.

from the top or bottom of the embankment. Trucks deposit the mixture into a hopper that feeds a conveyor belt leading to the spreader box. Figure 5-13 shows conveyor placement at the South Texas Nuclear Power Plant reservoir where each layer was placed 6.75 ft (2 m) wide by 9 in. (230 mm) thick. Conveyor placement also eliminates the need for earth ramps that are required to provide access for dump trucks hauling soil-cement to the placement area.

A method of reducing the amount of soil-cement while still utilizing standard dump trucks on the lift surface is to place the layers of soil-cement on a slight slope (not to exceed 8H:1V) toward the reservoir rather than in horizontal layers. Sloping the layers provides a greater working surface while reducing the thickness of the soil-cement normal to the slope.

When soil-cement is placed parallel to the slope, it is generally placed up and down the slope rather than cross slope (Figure 5-14). However, techniques have been developed for



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Figure 5-13. Conveyor system reduces layer width and eliminates need of earth ramps for truck travel up and down the slope.

successfully placing soil-cement across the slope (Figure 5-15). This technique utilizes dump trucks to discharge the soil-cement into the hopper of a transfer device. Then the soil-cement is transferred to a conveyor belt and then to the hopper of a paving machine. The paving machine spreads the materials across the slope. (Peltz, R. et al., 1999) While the plating method of design and construction allows for thinner facings and eliminates layer joints exposed to the water, greater wave runup will occur due to the smooth compacted face and adequate compaction is more difficult to achieve.

Stair-step construction generally is used where it is necessary either to have steep slope angles or when wave runup is likely to be excessive. Alternatively, plating construction generally is used when the available reservoir area is sufficient to allow for mild slope angles and when wave runup is not considered to be excessive. Some slope protection designs have included a combination of plating and stair-step techniques such as Tampa Bay Reservoir (Figure 5-16 and Figure 1-5. The



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Figure 5-14. Up and down slope soil-cement placement.



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Figure 5-15. Soil-cement placement across the slope using a paving machine.

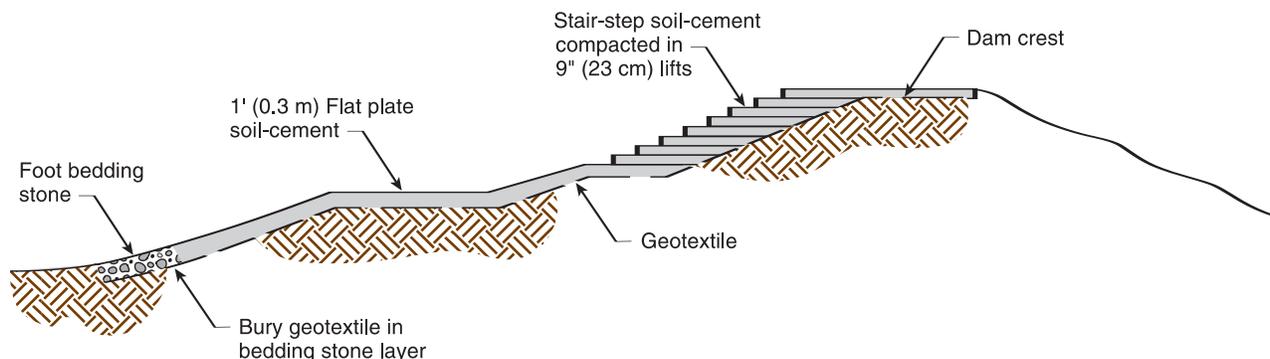


Figure 5-16. Typical section showing the combination plate and stair-step construction on Tampa Bay Reservoir, Florida.

cooling ponds for Florida Power Corporation’s Hines Energy Complex had a series of notches cut into the finished plated soil-cement section near the crest to reduce wave runoff.

Some of the regional flood control detention basins in Clark County, Nevada, have used soil-cement to plate the inside slopes of the embankments (Figure 5-17). Quality riprap is expensive and because of the arid environment, vegetation is impossible to establish. Plating is the proper choice because the detention facilities are designed to hold water for 72 hours or less so wave action will be insignificant.



Figure 5-17. Plated soil-cement slope protection at Upper Duck Detention Basin in Clark County, Nevada.

A freeboard of 5 ft (1.5 m) commonly is used for all but the largest reservoirs exposed to high velocity winds. This value is generally greater and therefore more conservative than most calculated freeboard using a value of 1.2 to 1.4 times the maximum wave height. Damage to the embankment by overtopping can be minimized by continuing the soil-cement over the entire crest which also can serve as a service road or public pathway.

5.3.3.1. Wave Runup

The height of waves will depend on the reservoir fetch, wind velocity, duration of wind, depth of water, width of reservoir, and wave characteristics. When a wave contacts the dam’s upstream face, it will run up the slope a distance governed by factors such as slope, roughness, permeability of the facing, and the wave steepness factor.

Results of wave tank tests by Nussbaum and Colley (1971) using scaled stair-stepped slope protection are given in Figure 5-18, Figure 5-19, and Figure 5-20. These nondimensional plots show the wave runup factor (R/H) versus wave steepness (H/T^2) where L , the wave length, is equal to $5.12T^2$. The researchers tested soil-cement slope facing configurations

H = Wave height L = Wave length
 R = Wave run-up T = Wave period
 S = Slope

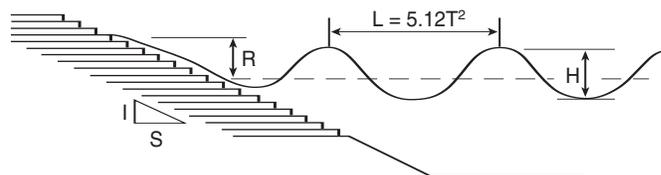


Figure 5-18. Wave runup measurements.

5.3.3. Freeboard

Freeboard is the vertical distance from the top of the embankment to the maximum reservoir water surface during flood conditions. The freeboard should be sufficient to prevent waves from overtopping the dam or embankment or from reaching areas of the crest that may be damaged.

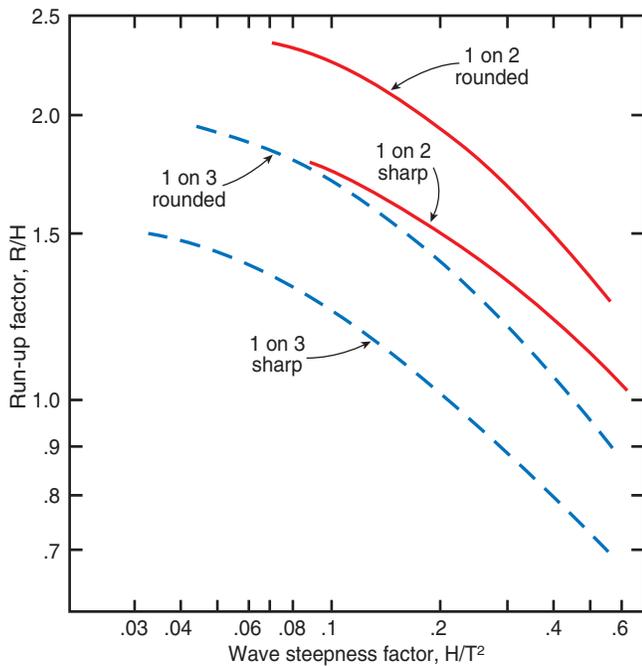


Figure 5-19. Wave runup on soil-cement slope facing.

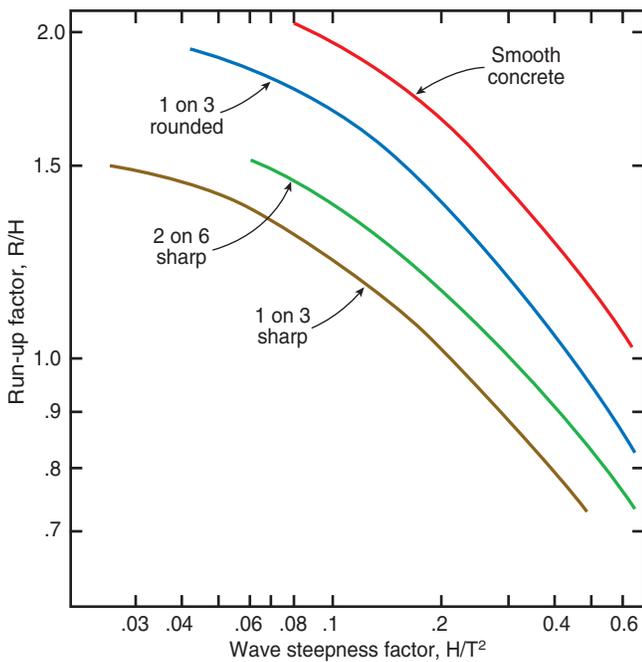


Figure 5-20. Effect of steepness on wave runup.

representing different degrees of surface roughness and embankment slopes of 1V:3H and 1V:2H. The 1V:3H slope was constructed in increments of 1-in. (25-mm-) thick layers, with a 3-in. (75 mm) setback for each successive layer. In addition, tests were made using 2-in. (50 mm) layers with 6-in. (150 mm) setbacks. Surface roughness varied as a func-

tion of the type of edge on the slope; both a sharp-edge and a rounded-edge condition were tested. A facing made of concrete finished with a wood float was considered representative of a smooth surface, and data from this test were used as a basis for comparison with data obtained from the soil-cement facings. As shown in Figure 5-19 and Figure 5-20, the sharp edged configuration was more effective than the rounded edge at reducing wave runup. Also as shown in Figure 5-20, larger steps (2 on 6) were less effective in reducing runup than smaller more frequent steps (1 on 3) for the same overall embankment slope.

To calculate the wave runup for soil-cement slope protection, the wave runup value based on riprap protection is first calculated and then multiplied by a factor based on the type and condition of the soil-cement slope protection. Calculation of wave runup for riprap is discussed in the Corps of Engineers manual EM-1110-2-1614, *Design of Coastal Revetments, Seawalls, and Bulkheads* (1995). Freeboard allowance of 1.2 times the wave runup is reasonable for soil-cement facing constructed of horizontal layers, stair-stepped up the slope, and finished with nearly vertical square steps. An allowance of 1.3 times the wave runup would be reasonable for the usual rounded step facing which results from standard stair-step placing techniques and natural weathering. A freeboard of 1.4 times the wave runup would be suitable for soil-cement placed parallel to the slope.

If wave runup cannot be controlled by the roughness of the soil-cement alone, a parapet wall can be used along the upstream edge of the crest. Jackson Lake in Colorado now has a combined soil-cement stepped protection with a parapet wall (Figure 5-21).



Figure 5-21. Soil-cement embankment protection with parapet wall in background, Jackson Lake, Colorado.

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5.3.4. Geotechnical Stability

The following sections briefly describe the procedures that should be conducted to determine the stability of soil-cement slope protection, bank protection, and grade control structures. Analyses should include determination of the factor of safety against sliding, overturning, soil bearing capacity, soil piping potential, uplift from hydrostatic pressures (floating), and seismic analysis.

5.3.4.1. Sliding

The ability of soil-cement to resist sliding is based upon its shear capacity. The shear capacity is dependent upon the physical properties and physical dimensions of the soil-cement and is discussed in Section 3.6. Shear strength represents the ability of the soil-cement to resist forces that would tend to slide parts past each other. These forces include hydrostatic pressure that tends to force individual particles apart causing a reduction in internal friction, horizontal pressure forces imposed by the difference between the upstream and downstream water surface elevations (head differential), buoyancy forces which cause vertical uplift, and forces due to acceleration (such as flowing water). As confining pressures are reduced, sliding resistance is reduced. This fact is especially important for the design of drop structures. For drop structures or other applications where horizontal loads are imposed, the sliding resistance of one layer of soil-cement over another must be analyzed along with the contact of the soil-cement with the foundation material. Oftentimes, a bonding mortar is used between the upper layers of soil-cement in grade control structures to increase the sliding resistance in these less confined layers. Further information on evaluation of sliding stability may be obtained from *Design of Small Dams* by the U.S. Bureau of Reclamation (1977) and *Sliding Stability for Concrete Structures* ETL 1110-2-256 by the U.S. Army Corps of Engineers (1981).

5.3.4.2. Overturning

The ability of soil-cement to resist overturning is based on its moment-resisting capacity. In this case it represents the ability of the soil-cement slope/bank protection to withstand laterally applied loads which would tend to cause the soil-cement liner to fail by rotating about its toe. The higher the resisting moment, the larger the forces that can be resisted by the soil-cement slope/bank protection. Similar to forces that would cause sliding, overturning forces include horizontal pressure forces and acceleration forces. The greatest force that would tend to resist overturning is gravity acting on the mass of soil-cement.

5.3.4.3. Bearing Capacity

The ability of the underlying soil to support the weight of the soil-cement above without failure or significant settling is determined by the soil's bearing capacity. Guidelines for determining a soil's bearing capacity can be found in the Corps of Engineers manual *Bearing Capacity of Soils*, EM 1110-1-1905.

5.3.4.4. Soil Piping

Soil piping is subsurface erosion of soil by percolating waters that produce pipe-like conduits underground. Soil piping can affect materials ranging from clay-size particles (less than 0.002 mm) to gravels (several centimeters), but is most common in fine-grained soils such as fine sand, silt, and coarse clay. The resulting "pipes" are commonly fractions of an inch wide but can expand up to several inches in size and may grow to several feet (meters) or more in diameter.

Soil piping most often will occur where backfills have been inadequately compacted or rodent burrows are present. Soil piping is most prevalent where seepage tends to concentrate along the outer surfaces of structures such as buried pipes or culverts. Adequate compaction of backfill along buried structures is essential to reduce the potential for soil piping. Guidelines for reducing soil piping potential in levee embankments can be found in the Corps of Engineers manual *Design and Construction of Levees*, EM 1110-2-1913.

5.3.4.5. Uplift and Hydrostatic Pressures

Excessive hydrostatic pressure may develop behind and/or beneath soil-cement in situations such as rapid drawdown of a reservoir. In this case, the water trapped in the soil beneath the relatively impermeable soil-cement layer may create pressures greater than those that can be counterbalanced by the weight of the overlying soil-cement, increasing the potential for uplift and displacement (flotation). When sudden drawdown is expected, embankment design should include features that reduce the potential for excessive hydrostatic pressures to develop such as those discussed in Section 5.3.5. Similarly, hydrostatic pressure will be exerted on the upstream face of the grade control structure. These structures typically are evaluated in a similar manner to dams, which is considered conservative given the hydraulic conductivity typical of the river bed and banks that support the structure. Further information on evaluation of uplift and hydrostatic pressure may be obtained from *Design of Small Dams* by the U.S. Bureau of Reclamation (1977) and *Flotation Stability Criteria for Concrete Hydraulic Structures* ETL 1110-2-307 by the U.S. Army Corps of Engineers (1987).

5.3.4.6. Seismic

A slope stability analysis compares the driving and resisting forces acting on the materials within the slope and returns a ratio, termed the factor of safety. For static conditions, a factor of safety of at least 1.5 usually is required. The seismic stability of slopes usually is evaluated using a pseudostatic analysis. For a pseudostatic analysis, a horizontal load (specified by a seismic coefficient) is applied to the slope. A factor of safety of at least 1.1 using a seismic coefficient relevant for the region of the country where the project is located usually is required for a slope to be considered seismically stable. Regional seismic coefficients may be obtained from *Earthquake Design and Evaluation for Civil Works Projects* ER 1110-2-1806 by the U.S. Army Corps of Engineers (1995) and also may be found in local building codes.

5.3.5. Interior Drainage

In most cases, soil-cement slope protection does not require interior drainage features. Only in cases where a reservoir undergoes rapid drawdown, exceeding 15 ft (4.6 m) or more within a few days, could hydrostatic pressures theoretically be produced from moisture trapped in the embankment against the back of the soil-cement facing. Three methods are available that can be used to prevent damage due to rapid drawdown. The first method is to design the embankment so that the least permeable zone is immediately adjacent to the soil-cement facing. The second method is to determine that the weight of the soil-cement facing is sufficient to resist uplift pressures. The third method is to provide free drainage behind and through or under the soil-cement facing to prevent uplift. In some cases a geotextile or filter fabric is needed between the soil-cement and embankment material. Sometimes cracks in the soil-cement can be large enough to allow wave action to pull soil particles out from behind the soil-cement, leaving voids. At Tampa Bay Reservoir in Florida, a nonwoven geotextile fabric was placed on the embankment slope before being plated with soil-cement (Figure 5-22). Similarly, soil-cement was placed on geotextile fabric at Ten Mile Creek Reservoir, Fort Pierce, Florida.

At the Florida Power and Light Sanford Plant, voids have developed behind the soil-cement where cracks occurred. Wave action would allow water to go in and out of the cracks, removing the fine materials. Periodic inspections are used to locate the voids, which are filled with a cement grout.



IMG19621

Figure 5-22. Placement of soil-cement over a geotextile fabric at Tampa Bay Reservoir, Florida.

5.4. Channels

Design issues that must be addressed for soil-cement lined channels include: establishing bank alignment and section dimensions, channel hydraulics, freeboard, providing adequate tie-ins at upstream and downstream terminations of channel, channel access, providing for tributary and side drainage into the channel, providing penetrations for structures or utilities, and consideration of effects upstream and downstream from the project.

The majority of the design issues associated with soil-cement lined channels are handled the same as those of soil-cement bank protection. However, scour within the lined channel will not be an issue given that the bottom of the channel will be protected by soil-cement. However, if the lined channel transitions from or to a natural channel, adequate toe-down should be provided at these locations. Figure 5-23 shows a soil-cement channel in a dense residential area in Albuquerque, New Mexico.



IMG19622

Figure 5-23. North Domingo Boca Channel in Albuquerque, New Mexico.

5.4.1. Abrasion

If a soil-cement lined channel is expected to convey sediment as bed load, it should be designed to resist the additional abrasion that results from the sediment as it impacts the bottom of the channel during transport. Increasing the abrasion resistance of soil-cement typically is done by increasing the amount of larger aggregate and/or increasing the amount of cement in the mix. Further information on abrasion resistance of soil-cement can be found in the PCA publication RD126 *Erosion and Abrasion Resistance of Soil-Cement and Roller-Compacted Concrete* by K.D. Hansen.

5.4.2. High Groundwater

Where high groundwater is a concern, the design of a soil-cement lined channel will need to account for the additional hydrostatic uplift that occurs. A thorough geotechnical evaluation should be done to assure the stability of the constructed soil-cement liner. The design will either need to account for the additional uplift forces or allow for drainage to reduce these forces.

5.5. Linings

Design issues that must be addressed for soil-cement linings for ponds, lagoons, and reservoirs include: slope angle, facing dimensions (plate or stair-step construction), free-board, provisions for inlet/outlet works, potential integration with a geomembrane, and allowable seepage. Of these, the first four are addressed in the same as for bank protection and slope protection, as discussed in Sections 5.2 and 5.3, respectively.

5.5.1. Geomembrane

The geomembrane may be placed either underneath or on top of the layer(s) of soil-cement. If placed underneath, the liner will prevent seepage loss of any liquid that has penetrated the soil-cement, allow for easy maintenance of the soil-cement lined pond or flume, and be protected from exposure to waves and sunlight. If placed on top of the soil-cement, the liner has a more stable base for prevention of tears, can be inspected periodically, can be repaired or replaced easily, and provides a barrier between the liquid and the soil-cement to prevent any chemical breakdown caused by chemical incompatibilities.

When placing soil-cement over a geomembrane, the largest particle size of the soil aggregate is generally kept below $\frac{3}{4}$ in. (19 mm) to prevent punctures. Also, the soil-cement should have a compacted thickness of at least 6 in. (150 mm).

5.5.2. Seepage

As discussed in Section 4.3.4, soil-cement can be used successfully as a low permeability liner either on its own or in conjunction with other seepage reduction techniques including low permeability embankment cores and synthetic membrane liners. The amount of seepage will depend on the construction technique (plating versus stair-step). Stair-step construction generally will result in greater opportunity for seepage as the permeability between the horizontal layers was found to be 2 to 20 times greater than for flow normal to the layering (Nussbaum and Colley, 1971). Seepage also occurs as the result of shrinkage cracks that develop in the soil-cement. Field studies indicate that these cracks tend to be less than $\frac{1}{8}$ -in. (3 mm) wide and spaced approximately 10 to 20 ft (3 to 6 m) apart in soil-cement made from granular soils (Highway Research Board, 1961). Adequate curing also helps to reduce shrinkage cracks. Inundating the soil-cement as soon as possible provides an excellent method of curing and helps minimize shrinkage.

5.6. Grade Control Structures

Design issues that must be addressed for soil-cement grade control structures include: type of structure, channel hydraulics, equilibrium channel slope, longitudinal spacing, height of drop, stilling structures, toe-down depth, tie-in to banks, upstream channel alignment, local inflows, downstream impacts, and geotechnical stability.

5.6.1. Type of Structure

The two main types of grade control structures are bed control structures and hydraulic control structures. The purpose of a bed control structure is to maintain the existing bed of the upstream channel by forcing the degradation zone to occur at the structure where nonerodible materials will prevent scour of the bed. The weir crest of the bed control structure generally is constructed at or near the existing channel bed with a cross section that approximates the upstream channel dimensions. Hydraulic control structures function by reducing the upstream energy slope, causing a reduction in velocities and shear stress. This usually will eliminate upstream erosion or, if desired, cause deposition of sediment upstream of the structure. Hydraulic control structures often are used to reestablish a previously degraded channel. Figure 5-24 shows an example of a soil-cement hydraulic grade control structure at Shop Creek, Colorado.



IMG19623

Figure 5-24. Soil-cement grade control structure, Shop Creek, Colorado.

5.6.2. Hydraulics

Grade control structures that are designed and constructed for the purpose of modifying the channel hydraulics do so by modifying the channel slope. Therefore, the channel hydraulic conditions must be determined for each iteration of the grade control design to make sure design water depths and velocities are not exceeded. Designs that significantly change the channel hydraulics could lead to increased overbank flooding upstream of the structure and have the potential for causing erosion downstream where overbank flows return to the main channel. Additionally, if the channel contains migratory fish species, the local hydraulics at the grade control structure could create an obstruction to the fish passage.

Outside of hydraulic and sediment transport considerations, one of the primary concerns in the design of a grade control structure is safety. Grade control structures with a significant drop in invert elevation can form a hydraulic jump and possibly a dangerous hydraulic roller. Hydraulic rollers are caused by flow recirculation around a hydraulic jump. These rollers can be particularly menacing if the hydraulic jump is submerged, lulling the public into thinking the waters are safe with quiescent surface conditions. The potential presence of reverse hydraulic rollers can be evaluated using procedures developed by Carriaga and Deschamps (1999) and the U.S. Bureau of Reclamation (USBR, 1977).

5.6.3. Equilibrium Channel Slope

One of the most important factors when locating grade control structures is the determination of the equilibrium slope. Unfortunately, it is also one of the most difficult parameters to define with any reliability. Failure to properly define the

equilibrium slope can lead to many problems. If the final channel slope is flatter than the predicted equilibrium slope, then the structures may be spaced too far apart, potentially leading to unacceptable degradation of the bed and inadequate design of the upstream structure which can lead to damage or possible failure of the structure. Conversely, if a steeper slope develops, then the structures may be spaced too close together. In this instance, the hydraulic performance of the upstream structure will be impaired by sediment deposition.

The primary factors affecting the final equilibrium slope upstream of a structure include: the incoming sediment load, channel characteristics, and the hydraulic effect of the structure. An additional complicating factor is the amount of time required for the equilibrium slope to develop. In some cases it may occur within a few high flow events while in others it may take multiple high flow events over several years.

There are many different methods of determining the equilibrium slope of a channel. These can range from detailed sediment transport modeling to less elaborate procedures involving empirical or process-based relationships such as regime analysis, tractive stress, or minimum permissible velocity. In some cases the equilibrium slope may be based solely on field experience with similar channels in the area. Regardless of the procedures used, the limitations of that procedure must be recognized before it is applied to a particular situation. Additional information on determining equilibrium slope is presented in Pemberton and Lara (1984) and ASCE (2002).

5.6.4. Longitudinal Spacing

The longitudinal spacing of grade control structures is a critical element of the design process, particularly when a series of structures is planned. The design of each structure is based on the anticipated tailwater or downstream bed elevation which, in turn, is a function of the next downstream structure. Heede and Mulich (1973) suggested that the optimum spacing of structures be such that the upstream structure does not interfere with the deposition zone of the next downstream structure. Mussetter (1983) showed that the optimum spacing should be the length of the deposition above the structure which is a function of the deposition slope. Johnson and Minaker (1944) recommend extending a line from the top of the downstream structure at a slope equal to the maximum equilibrium slope of sediment upstream until it reaches the original stream bed. The theoretical hydraulic spacing of grade control structures is given by Goitom and Zeller (1989):

$$H = (S_0 - S_f) x$$

H = Drop to be removed from reach

S_0 = Slope of the channel bed

S_f = Final or equilibrium slope

x = Length of reach

The corresponding number of structures (N) required for a given reach is given by:

$$N = H/h$$

h = Structure height

5.6.5. Drop Height

Once the total drop for the reach is determined, several factors must be considered when determining the height and number of individual structures within the reach. The potential for causing overbank flooding will increase as the height of an individual structure increases. More structures with smaller drops typically will have less impact on flood elevations than fewer structures with larger drops. The number of locations with appropriate alignment for the structures will limit the total number of structures for a reach. Additionally, the economics associated with building a few large structures compared to more small structures will be an important consideration in the overall design. Soil-cement grade control structures are most applicable when used as a series of small drops in lieu of a single large drop structure (Gemma et al., 1982). Experience has indicated that a maximum drop height for these structures is on the order of 3 ft (0.9 m).

5.6.6. Toe-down

Toe-down of the grade control structure is based on the expected local scour below the structure. The method used to estimate local scour at grade control structures is dependent on the type of grade control structure. Methods for estimating local scour below grade control structures can be found in Pemberton and Lara (1984) and ASCE (1998), and Simons, Li & Associates, Inc. (1986). As an example, local scour at the grade control structure included in Simons, Li & Associates, Inc. can be estimated from the following equation:

$$D_s = 0.54 q^{0.67} (h/y_t)^{0.158} S_r^{-0.134}$$

$h/y_t < 0.99$

D_s = Depth of local scour (ft)

q = Discharge per unit width (cfs/ft)

h = Drop height (ft)

y_t = Downstream flow depth (ft)

S_r = Relative submergence

The longitudinal extent of the scour hole created by either a free or submerged overfall is represented by the distances shown in Figure 5-25. Bank protection toe-downs downstream of the grade control structure extend to the computed depth of scour for a distance equal to $6 D_s$ beyond the grade control structure. The bank protection toe-down then tapers back to the downstream channel bank protection toe-down depth within a total distance downstream of the grade control structure equal to $12 D_s$.

If estimated scour depths below grade control structures are excessive, they may be reduced by decreasing the drop height or modifying the downstream toe-down depth.

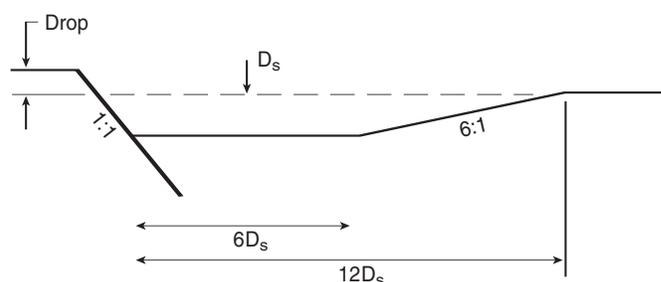


Figure 5-25. Grade control structure scour hole.

5.6.7. Bank Tie-in

The grade control structure should be tied or keyed into the bank sufficiently to prevent flanking as a result of lateral instability. If bank erosion is considered to be excessive or available right-of-way is insufficient to provide adequate tie-in, then protection of the bank should be considered in the design for the grade control structure.

5.6.8. Upstream Channel Alignment

A stable upstream alignment that provides a straight approach into the structure is critical. Since failure to stabilize the upstream approach may lead to excessive scour and possible flanking of the structure, it is desirable to locate the structure in a straight reach. If this is not possible, it may be

necessary to realign the channel to provide an adequate approach. Stabilization of the realigned channel may be required to ensure that the approach is maintained. Even if the structure is built in a straight reach, the possibility of upstream meanders migrating into the structure must be considered. In this case, the upstream meanders may require stabilization prior to or concurrent with the construction of the grade control structure.

5.6.9. Local Inflows

Local inflows from tributaries, field drains, roadside ditches, or other sources often will be an important consideration when sizing and locating grade control structures. Ideally, the structure should be located to avoid local drainage problems. However, there may be some situations where this is not possible. In these instances, the local drainage should either be redirected away from the structure or incorporated into the design.

The effect of main stream structures on tributaries should be considered when sizing and locating grade control structures. As degradation on a main stream channel migrates upstream, it will branch up into the tributary streams. If possible, main stream structures should be placed downstream of tributary confluences. This allows one structure to provide grade control to both the main stream and the tributary.

5.6.10. Downstream Impacts

Since grade control structures affect the sediment delivery to downstream reaches, it is necessary to consider the potential impacts to the downstream channel when grade control structures are planned. Bed control structures reduce the downstream sediment loading by preventing the erosion of the bed and banks, while hydraulic grade control structures have the added effect of trapping sediment. The ultimate response of the downstream channel to the reduction in sediment supply will vary from site to site. The magnitude of impacts downstream may be significant enough to warrant a change in the design. Changes could include reducing the amount of sediment being trapped by the structures or adding additional structures downstream.

5.6.11. Geotechnical Stability

Geotechnical stability of grade control structures is evaluated in a similar manner to that of bank protection as discussed in Section 5.3.4. Generally hydrostatic uplift, bearing capacity,

pipng potential beneath the structure, and sliding stability must be evaluated. The geotechnical analysis is very similar to that of a small earthen or concrete dam.

5.7. Access

When channels or reservoirs are sufficiently large to require vehicular access, ramps may be constructed within the soil-cement bank protection for this purpose (Figure 5-26). In Pima County, Arizona (Tucson), for example, access ramps in streams typically are constructed at one-half mile intervals on alternating sides of the channel. Whenever possible, the ramps should be located on the inside of channel bends and should be oriented so that their downward slope is in the same direction as the flow in the channel.



Figure 5-26. Access ramp oriented downstream, San Diego Creek, Irvine, California.

5.8. Safety

A standard design feature of all soil-cement bank protection projects in Tucson, Arizona, is the installation of a pipe railing along the stream-side top edge of the soil-cement to mitigate safety concerns to pedestrians utilizing the linear river parks which have developed as a result of the bank stabilization and the 8-ft- (2.4-m-) wide “pathways” provided by the soil-cement (Figure 5-27). Additional safety features can be designed into the soil-cement bank protection. These include access ramps, mild side slopes, or leaving the “stair-step” edge formed during the placement of the successive layers “as is” to facilitate egress from the channel during floods. “Stair-steps” were incorporated at Sand Creek in Aurora, Colorado, as shown in Figure 5-28.



IMG19625

Figure 5-27. Pedestrian railing along top of soil-cement bank protection, Rillito Creek in Tucson, Arizona.



IMG19626

Figure 5-28. "Stair-step" edges along Sand Creek in Aurora, Colorado.

5.9. Joints

Construction joints should be made to ensure continuity in smoothness and grade between old and new sections of soil-cement. All joints should have the same texture, full-depth density, and smoothness. Regardless of age, contact surfaces of previously constructed layers that have become coated with dust, sand, or other objectionable material should be cleaned by power brushing or other approved methods before placement of additional soil-cement.

5.9.1. Transverse Construction Joints

At the end of each day's work, or whenever construction operations are interrupted for more than two hours, a transverse construction joint should be formed by cutting back into the completed work to form a full-depth near-vertical face. If construction joints are formed at more than one layer, the construction joint for each layer should be stag-

gered at least 8 ft (2.4 m) horizontally from the construction joint of the layer below. Figure 5-29 shows a proper transverse construction joint.



IMG19627

Figure 5-29. Proper construction of a transverse construction joint.

5.9.2. Horizontal Lift Joints

Whenever the contractor's operations are interrupted such that the subsequent lift can not be placed within 12 hours or if the surface has been allowed to dry out or become contaminated, the surface should be cleaned with a power broom (Figure 5-30) or compressed air, moistened, and then covered with a bonding slurry or mortar (Figure 5-31). This requirement may not be necessary if the surface is sufficiently roughened to provide a mechanical bond. The roughened surface then should be cleaned using a high-pressure air or other approved method to completely free the surface of all loose material prior to the placement of the subsequent lift.



IMG19628

Figure 5-30. Power broom used to clean surface of soil-cement.



IMG19629

Figure 5-31. Applying cement slurry to previously placed soil-cement surface immediately prior to placement of subsequent lift.



IMG19630

Figure 5-32. Ribbed drum used to create indentation in surface of soil-cement.

The limiting number of hours defining a horizontal cold joint will vary depending on the ambient weather conditions. Hot windy conditions may reduce the time down to 6 hours while cool conditions may allow more than 12 hours. If the design is based on the frictional resistance at the lift joints and does not rely on cohesion then the number of hours between lifts is not that important.

Some designs will specify that the final pass of the compaction equipment over a soil-cement layer create indentations in the surface of the soil-cement to create a mechanical bond. Indentations can be created by welding several metal bands around a smooth drum (creating inch-deep grooves parallel to the direction of the lift) and passing the heavy drum (with projecting ridges) over the lift surface just after the last pass of the smooth roller (Figure 5-32). Alternatively, a set of dimpled rollers can be used to create the desired indentations (Figure 5-33).

For grade control structures, weirs, and spillways, the top three lifts always should use a bedding mortar between the lift surfaces to improve cohesion between the lifts and prevent delaminating.

Both dry cement and cement slurry lift bonding have been used and evaluated in USBR test sections. A slurry mix of cement, water, and sometimes sand should have a water-to-cement ratio of about 0.70 to 0.80. Dry cement applications have the disadvantage of being susceptible to wind and producing nonuniform coverage. When using dry cement scattered over the compacted layer of soil-cement, it should be moistened just prior to placement of the next lift. Whichever method is used, the material should be applied immediately before the placement of the next lift in short reaches



IMG19631

Figure 5-33. Set of dimpled rollers used to create indentations in surface of soil-cement.

so as not to dry out. A typical application rate for the dry cement approach is 0.5-1.0 pounds per square foot (2.4-4.8 kg/m²).

5.10. Finishes

For multiple horizontal lift soil-cement bank protection projects, several finishing options are available. The finish used will depend on hydraulic requirements such as channel roughness as well as aesthetic and access requirements. During placement of the soil-cement layers, the outer edge of each layer may be treated in several ways. To produce a smooth surface conforming to a preferred bank slope, the edge may be compacted with special equipment (Figure 5-34) or simply trimmed back to the required grade. To produce a rougher surface, the outer edge may be left as is, leaving a ragged stair-step appearance. This outer exposed portion of the soil-cement usually is compacted to less than optimum density due



IMG15508

Figure 5-34. Compacting edges with compactor attached to bucket of backhoe.

to limitation of the equipment and typically is considered sacrificial for erosion purposes. Another option is to roll the top edge over with a special flange on the vibratory roller that compacts and smooths the outer edge (Figure 5-35). Finally, the outer edges also may be trimmed to conform to vertical stair-step faces. A combination of finishes may be utilized to achieve the final desired appearance and functionality.

5.11. Aesthetics

For in-stream projects such as bank protection and drop structures, the final appearance or aesthetics are often an important design consideration. Because soil-cement generally is made from native aggregate found on site, the final appearance, texture, and color are generally similar to native bank materials. This is particularly true in locations where optimal soil gradations are encountered which allow for cement contents below 10 percent. This feature of soil-cement gives



IMG15509

Figure 5-35. Compacting edges of soil-cement bank protection with rounded flange attached to still drum roller.

it a much more “natural” look compared to the typical riprap alternative. Soil-cement banks and drop structures also can be curved to follow and blend in with the surrounding topography. Additionally, the layered and stepped appearance of the soil-cement has been likened to layers of sandstone that often are exposed along stream channels. Artificial color also can be applied after construction and used to mimic the natural desert appearance. Figure 5-36 shows an example of soil-cement constructed in offset layers that give the appearance of geologic folds in layers of sandstone.

At this same location along Calabacillas, special artwork was used. Colored shotcrete was used above the soil-cement and precast dinosaur bones were placed into the shotcrete (Figure 5-37).



IMG19632

Figure 5-36. Construction of simulated geologic fault in soil-cement layers along Calabacillas Arroyo, New Mexico.



IMG19633

Figure 5-37. Precast dinosaur bones placed in colored shotcrete above soil-cement bank protection at Calabacillas Arroyo, New Mexico.

For areas outside of the active channel, but still in the active channel migration zone, the soil-cement may be buried, allowing for natural landscaping and trails. This was done at the Bridgeport project along the Santa Clara River in California (Figure 5-38).



Figure 5-38. Buried soil-cement bank protection along Santa Clara River, California.

5.12. Chemical Compatibility

Soil-cement has been tested both for its compatibility with the soil that is used to create it and its ability to resist certain chemicals after curing. Certain soils are not compatible with portland cement for making a durable soil-cement. Also, laboratory tests have been conducted on hardened soil-cement to determine its resistance to certain types of chemicals, including various hazardous wastes, for use as a liner.

5.12.1. Poorly Reacting Sandy Soils

As discussed in Section 3.9, certain types of sandy soils are encountered that cannot be treated successfully with normal amounts of portland cement. Early research by Robbins and Mueller (1960) showed that organic material of an acidic nature usually had an adverse effect on soil-cement. The study concluded that organic content greater than 2% or a pH value lower than 5.3 most likely will result in an abnormal reaction with cement. These types of soils require additional studies prior to use in soil-cement.

5.12.2. Sulfate Resistance

As with conventional concrete, sulfates generally will attack soil-cement. Studies by Cordon (1962) and Sherwood (1962) indicated that the resistance to sulfate attack differs for cement-treated coarse grained and fine grained soils and is

related directly to the concentrations of clay and sulfate. The studies showed that sulfate-clay reactions are more detrimental than sulfate-cement reactions, resulting in deterioration of soil-cement utilizing fine grained soil more rapidly than coarse grained soil. They also concluded that increasing the cement content of the soil-cement mixture may be more beneficial than using a sulfate-resistant type of cement.

5.12.3. Resistance to Hazardous Waste

The U.S. Environmental Protection Agency (EPA) sponsored laboratory tests to evaluate the compatibility of a number of lining materials exposed to various wastes. The tests indicated that after one year of exposure to leachate from municipal solid wastes, the soil-cement hardened considerably and cored like portland cement concrete. In addition, it became less permeable during the exposure period (Office of Solid Waste and Emergency Resources, 1983).

The soil-cement also was exposed to toxic pesticide formulations, oil refinery sludges, toxic pharmaceutical wastes, and rubber and plastic wastes. Results showed that for these hazardous wastes, no seepage had occurred through the soil-cement following 2½ years of exposure. Also, the compressive strength of the soil-cement exceeded that of similar soil-cement that had not been exposed to the wastes. Soil-cement was not exposed to acid wastes. It was rated “fair” in containing caustic petroleum sludge, indicating that the specific combination of soil-cement and certain waste materials should be tested and evaluated prior to design.

6 CONSTRUCTION



IMG19635

Figure 6-1. Soil-cement stream bank protection at beginning stages of construction along Santa Clara River in Valencia, California, with dewatering system in the background.

6.1. Subgrade Preparation

Soil-cement cannot be placed under water or on a wet or soft subgrade that will not allow the soil-cement to be compacted to meet minimum density requirements. If there is water in the channel or a good chance of flow occurring in the channel, temporary earthen dikes should be constructed to keep water away from the construction area. This is especially important during initial placement of the soil-cement below grade in the toe-down area. Where a wet subgrade exists or where excavation releases water from the banks, it may be necessary to devise a dewatering system to protect the initial soil-cement placement operations (Figure 6-1). Overexcavation to facilitate the placement of a gravel layer below the first lift of soil-cement is one potential solution for wet subgrade conditions. Another option is to place a waste layer of soil-cement which is minimally compacted and allowed to set prior

to soil-cement placement. Where a weak subgrade exists, a concrete mud mat may be placed to provide an adequate surface for soil-cement placement. This was done on portions of San Diego Creek in Irvine, California.

6.2. Mixing

Soil-cement either may be mixed in a central mixing plant or mixed in place using traveling mixing machines. The central plant method of mixing is recommended, especially for stair-step and multilayer construction, because it provides more accurate and uniform mixing than the mix-in-place technique. Except for very small projects, the central plant method is also more economical and generally results in faster production of soil-cement.

There are two basic types of central plant mixers – pugmill mixers (either continuous flow or batch feed) and rotary drum mixers. Although batch feed pugmills and rotary drum mixers have been used successfully for mixing soil-cement, the most common central plant mixing method, especially for large projects, is the continuous flow pugmill mixer (Figure 6-2). A typical plant consists of a soil bin or stockpile,

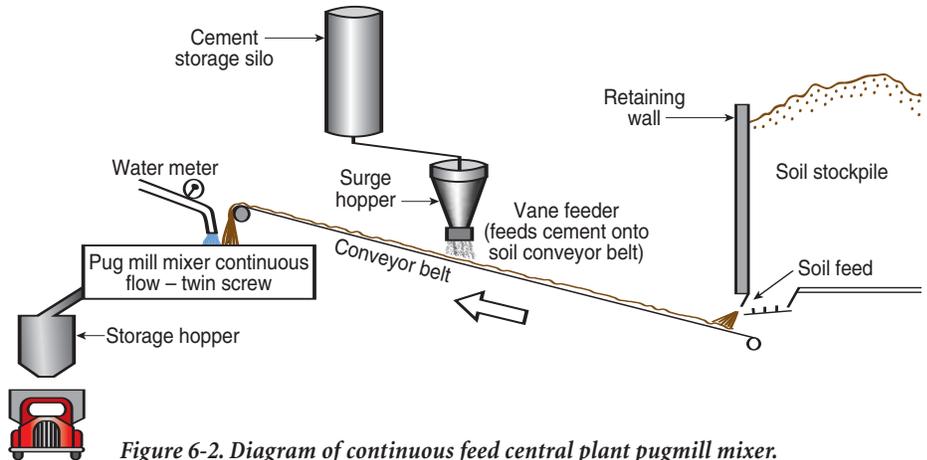


Figure 6-2. Diagram of continuous feed central plant pugmill mixer.

a cement silo with surge hopper, a conveyor belt to deliver the soil and cement to the pugmill mixing chamber, a pugmill mixing chamber, a water storage tank for adding water during mixing, and a holding or gob hopper to temporarily store the mixed soil-cement prior to loading.

Some soil-cement projects require the use of large storage hoppers. Storage hoppers like the one shown in Figure 6-3 allow the mixer to run continuously so production is not interrupted by truck delays.

Mixing plants control the proportions of water, aggregate, and cementitious materials by monitoring either the weight or volume of each product. In order to maintain a consistent product, the mixing plant should have batching or feeding tolerances of $\pm 2\%$ for cementitious material and water, and $\pm 3\%$ for soil aggregate. Most portable pugmill plants have capacities between 200 and 500 tons per hour. The capacity of the pugmill should be at least 25% more than the anticipated placement production volumes. In addition, because large amounts of cement will be used in a short duration, storage “pigs” should be used to maintain enough cement on site for at least a single shift.

The aggregates and cementitious materials should be conveyed to the mixer in proportions as required. The water content may be varied at frequent intervals as necessary and as considered appropriate because of placing and compacting operations and shall, in general, be based on the action of the rollers on the freshly placed materials, on the field density test results, and on the surface texture being produced. Additionally, the proportions may be changed as necessary to adjust for variations in materials, plastic, and hardened properties. Adjustments to mix proportions other than water can be as frequent as once per shift.



IMG19636

Figure 6-3. Central plant pugmill mixer with bottom dump trailer used for storage hopper at Santa Cruz River, Arizona.

In batch mixing, aggregates and cementitious materials should be charged into the mixer and dry-mixed at least 15 seconds before water is added. After water is added, mixing should continue as required to obtain a homogeneous mixture. When a continuous mixer is employed, mixing time should be sufficiently long to obtain a homogeneous mixture but in no case should it be less than 15 seconds. Shaft speed of the pugmill should be maintained at the speed recommended by the manufacturer. Mixer and mixer paddle surfaces should be kept free of hardened soil-cement and other contamination. Mixer paddles worn down more than 10% from new paddles of the same type and manufacture should be replaced.

6.3. Transportation

Equipment for transporting the soil-cement mixture should have clean, smooth beds and protective covers for use in unfavorable weather. No track vehicles should be allowed to transport the soil-cement onto the lift surfaces. The equipment should dump directly onto the final location or as near the final location as possible. Deliveries should be scheduled so that soil-cement will be spread and rolled within the time limit specified in Section 6.5. Loads that have crusts of partially hardened material or have become wet by rain should not be used. Rubber tired vehicles transporting soil-cement may travel across compacted soil-cement only for the purposes of depositing the subsequent lift. Vehicles transporting soil-cement should be operated to prevent sudden stops, sharp turns, or other operations that damage the surface of the previously compacted lift. When dumping soil-cement, it should not free fall more than 5 ft (1.5 m) or be piled higher than 5 ft (1.5 m).

Soil-cement usually is transported from the central mixing plant using either end-dump or bottom-dump trucks (Figure 6-4) and placed directly on the previous layer of soil-cement or into a paving machine. In the case of Calabacillas Arroyo, New Mexico, the trucks dumped the soil-cement into a skip bucket (Figure 6-5) from which an excavator placed the soil-cement onto the previous lift where it was spread by a dozer and compacted by a vibratory roller.

All necessary precautions should be taken to prevent damage to completed soil-cement by the equipment and to prevent the deposition of raw earth or foreign materials between layers of soil-cement. Earth ramps crossing completed soil-cement must have at least 2 ft (0.6 m) of compacted thickness. Where ramps are constructed over soil-cement that is



IMG19637

Figure 6-4. Bottom dump truck placing soil-cement, Bridgeport Project, Santa Clara River, California.



IMG19639

Figure 6-6. Motor grader spreading soil-cement.



IMG19638

Figure 6-5. Skip bucket and excavator used to place soil-cement along Calabacillas Arroyo, New Mexico.

not to final grade, all foreign materials must be removed prior to continuation of the soil-cement construction.

6.4. Spreading

The equipment for spreading the soil-cement mixture should be suitable for the purpose and be capable of discharging the mixture in layers to produce reasonably smooth uniform surfaces (Figure 6-6). The equipment should be controllable so as to produce layers which when compacted will each be approximately of the specified thickness. Under no condition should tracked vehicles be operated on other than fresh uncompact soil-cement. Soil-cement mixtures should be placed and distributed in such a manner as to produce a reasonably smooth and uniform surface in layers of such uncompact thickness that when compacted each layer will be no less than 6 in. (150 mm) nor typically more than 12 in. (300 mm) in thickness. The contractor must demonstrate that proper densities could be obtained to full depth in a test section. The soil-cement should be placed and compacted as

rapidly as possible. Soil-cement should be placed in successive horizontal layers for most applications. Placing soil-cement during excessively hot, cold, or wet weather should be avoided where possible.

Placement of stair-stepped soil-cement should be limited to a height of 4 ft (1.2 m) in a single 8 to 10 hour construction shift. Instability of the soil-cement mass as evidenced by a bulging of the outer face can occur if too much soil-cement is placed before the lower layers have attained sufficient stability to support the weight of the material and equipment above.

The active working area must be limited such that no more than 1½ hours have passed since the soil-cement was mixed until final compaction is achieved. Figure 6-7 shows that because of the vertical length of the embankment slope, the contractor placed the bottom half of the soil-cement first and is now beginning the placement of the top half of the slope protection.



IMG19640

Figure 6-7. Dozer spreading soil-cement up a slope.

Soil-cement typically is spread with a dozer or motor grader, but also has been successfully spread with paving machines. Some pavers are equipped with one or more tamping bars which provide initial compaction. In the case of Florida Power Corporation's Hines Energy Complex (Figure 6-8), the tamping bars were the only means used to compact the soil-cement. Although full compaction of the soil-cement could not be achieved readily, a mixture of 11% cement and compacted dry densities as low as 92% of ASTM D 558 allowed for 7-day compressive strengths of 700 to 800 psi (4.8 to 5.5 MPa).



Figure 6-8. Paving machine used to spread soil-cement at Hines Energy Complex, Florida.

6.5. Compaction

Compaction is accomplished by tamping rollers, pneumatic-tired rollers, steel-wheeled rollers, vibratory compactors, or other approved equipment well suited to the material being compacted (Figure 6-9). The equipment should provide uniform compaction across the width of the equipment for each pass. Trucks, tractors, or other hauling equipment should not be used for compaction of soil-cement mixtures. Light, walk-behind, or similar sized vibratory rollers and mechanical tampers should be used for compacting areas inaccessible to the large rollers (Figure 6-10). This equipment must be able to compact the soil-cement to the required densities. At the start of compaction, the mixture should be in a uniform, loose condition throughout its full depth. No section should be left undisturbed for longer than 30 minutes during compaction operations. Compaction of each layer should be done in such a manner as to produce a dense surface, free of compaction planes, within 1½ hours from the time cement is added to the mixture.

When using stair-step construction it is sometimes desirable to compact the vertical edge or face of each soil-cement lift (Figure 6-11). This may be done for aesthetic reasons or as a means to strengthen the vertical edges and form steps that will resist abrasion and erosion, and provide for safe egress.



Figure 6-9. Vibratory roller compacting soil-cement.



Figure 6-10. Remote-controlled vibratory compactor for use in inaccessible areas.



Figure 6-11. Steel plate attached to bucket of small excavator used to create and compact the vertical face of each soil-cement lift at Calabacillas Arroyo, New Mexico.

Readings to determine the density of compacted soil-cement should be made using a single probe nuclear density gauge. The number of tests per lift will vary depending on the size of the project and the surface area of each lift. Generally, density tests should be taken approximately every 500 cu yds (380 cu m) of soil-cement placed or at least one test per work shift if less than 500 cu yds (380 cu m) are placed during the shift. For small structures, such as grade control structures, one density test per lift should be sufficient. The moving average of any five consecutive in-place density tests should not be less than 98% of the maximum density obtained by ASTM D 558, with no individual test less than 95%.

6.6. Finishing

After compaction to the required lines and grades, the soil-cement surface shall be reasonably smooth and free of surface tears and rock pockets. Any trimming to remove loose soil-cement, or as indicated on project drawings, must be completed within 6 hours from the time of mixing. It should be noted that a conventional concrete finish is not attainable with soil-cement. The larger the maximum size aggregate allowed in the mix, the rougher the finished surface appearance.

An acceptable finished appearance of soil-cement will vary depending on the aesthetic requirements of the project. For bank protection, some designers will specify slopes that are trimmed of overbuild back to the design neat lines while others will specify leaving any overbuild and loose soil-cement on the slopes to create a more “natural” look.

If unformed “steps” are designed as part of the slope surface they will vary in appearance. These steps often are used to mimic a layered geologic formation and to blend into the surrounding area.

6.7. Curing

Temporarily exposed surfaces of soil-cement that will be in contact with succeeding layers of soil-cement should be kept continuously moist by water curing until placement of the subsequent layer. Curing of permanently exposed surfaces should begin immediately after compaction and shall continue for at least 7 days. Soil-cement should be cured and protected from premature drying, extremes in temperature, rapid temperature change, freezing, mechanical damage, and exposure to rain or flowing water. All equipment needed for adequate curing and protection should be on hand and ready to install before actual placement begins. All soil-

cement should be protected from the damaging effects of rain for 8 hours and of flowing water for 7 days.

Water for curing should comply with the requirements of Section 2.3. Horizontal surfaces that will be exposed permanently may be cured by covering with a minimum uniform thickness of 6 in. (150 mm) of continuously saturated sand. Temporarily exposed surfaces should not be cured by saturated sand. Saturated sand was used at the Tropocania Detention Basin in Las Vegas (Figure 6-12).



IMG19645

Figure 6-12. Water truck used to prevent soil-cement surfaces from drying out.

Water trucks may be used to keep surfaces wet at all times (Figure 6-12). The water truck should be supplemented, as necessary, by mists from hand-held hoses. The spray should cover all portions of the soil-cement surface. All spray nozzles both on the trucks and the hand-held hoses should be of a type that produces a true fog spray without any concentrated streams of water. The mist should not be applied in a channelized or pressurized manner that in any way erodes the surface of the soil-cement. It also should be applied at a rate which does not cause ponding at the surface.

An approved sprinkler system consisting of pipelines and rotating or other approved types of sprinklers may be used. Sprinklers should deliver a fine mist of water and should not cause any erosion to the surface of the soil-cement. The sprinkler system should cover all portions of the soil-cement surface, and keep the surface wet at all times.

7 QUALITY CONTROL INSPECTION AND TESTING

The engineer, with assistance and cooperation from the contractor, should perform the inspection and tests described in the following paragraphs. Based upon the results of these inspections and tests, the contractor should take actions necessary to correct any items that are not in compliance with the specifications. Where applicable in this guide, designations of standard specifications and test methods are listed. For additional information, the user should refer to ASTM and AASHTO standard test methods and specifications listed in Appendix A.

7.1. Mixture Uniformity

The uniformity of the soil-cement mixture can vary depending on the uniformity of the stockpiled aggregates, type of equipment used to mix it, and the duration of mixing time. The uniformity usually is checked visually as it comes out of the plant. It also can be checked at the placement area by digging trenches or a series of holes at regular intervals for the full depth of treatment and inspecting the color and texture of the exposed material. When the mixture is of uniform color and texture from top to bottom, the mixture is satisfactory.

The mixing time necessary to achieve an intimate uniform mixture will depend on the soil gradation and the type of mixing plant used. Mixing shall be sufficient to secure a

homogeneous, intimate, uniform mixture of the soil and water within the specified tolerances (Table 7-1). Soil and cementitious material shall be mixed sufficiently to prevent cementitious balls from forming when water is added.

Batching or feeding should conform to the mixture proportions directed within the tolerances shown in Table 7-2.

Table 7-2. Batching or Feeding Tolerances

Material	Plant
Cementitious material	± 2.0%
Water	± 2.0%
Soil aggregate	± 3.0%

Table 7-1. Soil-Cement Uniformity Test Tolerances

Test	Allowable maximum difference*	ASTM standard
Water content of mixture	15%	D 2216
Maximum density	2%	D 558 D1557
Compressive strength at 7 days	25%	C 39 D1633

*Maximum allowable difference = (maximum value minus minimum value, divided by three tests).

For batch-type plants, the variation is in percent by weight from batch weight of each material, based on the mixture proportions specified. For continuous feeding and mixing plants, the variation is in percent by weight from the mixture proportions of each material designed to be in a total timed sample obtained from a designated location in the plant.

Whenever it is found that either the weighing or the batching accuracy does not comply with specification requirements, the plant should be shut down until necessary adjustments or repairs have been made. Discrepancies in recording should be corrected immediately.

7.1.1. Batch-Mixing Plants

Accuracy of the batching equipment should be checked by the contractor for amount of cementitious materials and aggregate at the beginning of operations and at least once every 10 shifts in the presence of the engineer. Such checks also should be made whenever there are variations in properties of the fresh soil-cement, which could be the result of batching errors. Standard test weights accurate to 0.1% should be provided for checking plant scales.

7.1.2. Continuous-Mixing Plants

Accuracy of proportioning of the continuous-mixing plant should be checked by the contractor for the cementitious material, and for each aggregate at the beginning of construction and after every 10 shifts. The accuracy of proportioning should be checked by simultaneously securing timed samples of the cementitious materials and the aggregate as they are fed to the mixer, and weighing each as appropriate.

7.1.3. Monitoring Cementitious Content

The contractor should, at least once per day, take a sample of soil-cement and determine the cement content in accordance with ASTM D 5982 – Heat of Neutralization Method. At the end of each shift a mass balance check should be made on all cementitious materials. The amount of cementitious materials stored on-site and delivered that shift should be compared to the amount of cementitious materials used during the shift.

7.2. Aggregate

7.2.1. Stockpiled Aggregate

Samples from stockpiles for moisture-density tests and mix design should be obtained by the engineer by taking a minimum of four face cuts at different locations on a stockpile and blending the sampled material. A minimum of one moisture-density relation should be determined in accordance with ASTM D 558 for each 10,000 cu yd (7,500 cu m) of stockpiled soil aggregate or one per stockpile, whichever is less. In addition, one sieve analysis should be performed in accordance with ASTM C 136 and ASTM C 117 and one Atterberg limit test in accordance with ASTM D 4318 for each of the four face cuts of stockpiled aggregate, and one for the blend of the four face cuts.

7.2.2. Batched Aggregate

At least once during each shift in which soil-cement is being placed, there should be one sieve analysis performed on soil aggregate from the batching plant in accordance with ASTM C 136 for aggregates. When two consecutive tests show the soil aggregate to be deficient in grading, the mixing operation should be stopped until acceptable material is furnished for delivery to the mixer. Each time a moisture-density relation is determined, an additional gradation analysis in conformance with ASTM C 136 should be performed, corresponding to the material used in the moisture-density relation.

When the amount passing any sieve is outside the specification limits or tolerances, the aggregate should be immediately resampled and retested. If the second sample fails on the same sieve, immediate steps should be taken to correct the grading.

7.2.3. Moisture Content

The contractor should conduct at least two tests for moisture content in accordance with either ASTM C 70 or ASTM C 566 during each 8-hour period of mixing plant operation. The first test should be at the beginning of each shift and subsequent tests should be selected randomly within the 8-hour period. Additional tests should be made whenever there is excessive variation in workability of the soil-cement.

7.3. Placement Area

The contractor should inspect foundation or construction joints prior to each lift of soil-cement placement. The contractor should obtain approval from the engineer prior to placing soil-cement at the start of each shift and at previously placed joints.

The contractor should not permit placement to begin until there is adequate equipment, manned by competent operators, available for compacting the soil-cement. Placement should be temporarily discontinued if any lift is inadequately compacted until such time as that lift has been reworked, retested, and meets the density requirements.

The contractor's quality control personnel should supervise all placement operations and should be responsible for measuring and recording soil-cement temperature, air temperature, weather conditions, time of placement, yardage placed, and method and location of placement. The engineer will provide quality assurance over placing operations.

7.4. Curing

The curing operation should be inspected to assure that the surface of the permanently exposed soil-cement is kept moist (or wet) continuously until the end of the curing period. All temporarily exposed soil-cement should be kept moist.

The engineer should be notified when any soil-cement surface is allowed to dry before the end of the curing period, and immediate steps should be taken to correct the situation.

7.5. Density

Field density testing should be performed by the engineer on the soil-cement in accordance with ASTM C 1040 as soon as possible, but within 30 minutes, after the completion of vibratory rolling. Only wet density should be used for evaluation. The test should be performed using a single probe nuclear density gauge operating in the direct transmission mode so density of the full depth of the soil-cement can be measured. Each test should include readings near the bottom of lift and at 3-in. (76-mm) increments; however, only the deepest reading should be used to evaluate the density. Both wet and dry densities should be reported, as well as individual readings. Periodically, densities should be determined by the sand core method (ASTM D 1556) as a check against the results obtained by the nuclear density gauge.

If any nuclear density gauge reading is below 96%, another test will be performed within a 5 to 8 ft (1.5 to 2.5 m) radius of the previous testing location. If the adjacent reading is also below the density requirements, additional vibratory roller passes should be made across the full lane width between the last testing location that produced an acceptable reading and the placement operations. If additional vibratory roller passes cause the density to decrease or cause the surface texture and appearance to deteriorate in the opinion of the engineer, the placing operation should be discontinued until appropriate adjustments are made to the moisture content of the mixture, placement operations,

rolling procedures, or other operations to assure that the specified density and surface requirements can be achieved.

As mentioned in Section 6.5, the number of tests per lift will vary depending on the size of the project and the surface area of each lift. Generally, density tests should be taken approximately every 500 cu yds (380 cu m) or at least one test per work shift if less than 500 cu yds (380 cu m) are placed per shift. For small structures where less than 500 cu yds (380 cu m) per lift are placed, one density test per lift should be sufficient. Additional tests should be made during start-up and when problems with attaining required density occur.

7.6. Moisture Content

The moisture content should be determined in accordance with ASTM D 3017 at the same depths as the density testing and in accordance with ASTM C 566 at least once a day.

7.7. Compressive Strength

The engineer should make at least three compressive strength specimens for each 1,000 cu yd (750 cu m) or less of soil-cement placed. A "test" is defined as the average of two companion soil-cement cylinders. Samples should be taken from the placement area prior to compaction. Tests should determine the 1-day and 7-day compressive strengths in accordance with ASTM D 1633, Method A, except that curing of specimens in the mold will be required only for the length of time necessary to satisfactorily remove the specimens from the mold without damage to the specimens. Some engineers also will determine the 3-day compressive strength. The 1-day compressive strength test can be used to monitor the daily production of the central mixing plant and to adjust soil-cement mixture proportions. Table 7-3 shows the typical relationship of the 1-day, 3-day, and 28-day compressive strengths to the 7-day design strength. The 7-day compressive strength test typically is used for final acceptance of the soil-cement.

Table 7-3. Relationship of Various Ages as Percentage of 7-Day Compressive Strength (7-Day Design Strength of 750 psi [5.2 MPa])

1-day		3-day		28-day	
Range %	Average %	Range %	Average %	Range %	Average %
40-60	50	60-80	70	120-150	130

(Results based on 12 projects in Los Angeles County, CA-Pacific Advanced Civil Engineering) (PACE)

7.8. Temperature

The temperature of the soil-cement mixture usually is taken at the mixing plant and at the placement area on randomly selected batches of each mix design of soil-cement used per shift of placement. Additional tests usually are made when rapid set time or workability loss is reported. The engineer should be notified whenever the soil-cement temperature falls below 40°F (4°C) or is above 90°F (32°C). In such cases remedial action or even discontinuation of placement may be required.

8 COST ESTIMATING

The in-place cost of soil-cement for water resources applications depends on a number of factors. These factors include availability of aggregate, ease of construction, competition, cement content, cost of cement, and volume of soil-cement to be placed. As might be expected, lower bid prices for soil-cement are obtained when the aggregate is on-site and requires little processing, easy access is provided between the mixing plant and the placement area, there is little need for control of ground or surface water, there are no unusual placement requirements, larger placement volumes are involved, and there are a greater number of qualified bidders.

A study of the bids received on 30 projects between 1995 and 2002 found the unit cost of soil-cement, including cement, ranged from \$19.85 to \$97.35/cu yd (\$25.96 to \$127.32/cu m) in place. The projects studied were located in seven states and ranged in size from 1,195 to 340,450 cu yd (915 to 260,310 cu m). The average unweighted unit cost for the soil-cement was \$37.00/cu yd (\$48.29/cu m). As expected, the lower unit costs for soil-cement are associated with the larger volume projects.

The total cost is equal to the cost of processing plus the cost of the cement. The cost of processing depends upon the

quantity of soil-cement to be placed. The cost of processing includes cost of materials, hauling, central plant mixing, water for mixing and curing, transporting mixed soil-cement to placement area, spreading, compacting, and curing. This depends to some extent on the quantity of soil-cement to be placed. If the soil to be mixed with the cement must be hauled from off-site or if extensive processing of the on-site soils is required, an additional cost must be added to the cost of processing the soil-cement. Table 8-1 provides guidelines for estimating the cost of processing based on quantity and degree of difficulty (2002 prices).

Table 8-1. Estimated Cost of Processing Soil-Cement (Cement Costs Not Included)

Volume of soil-cement yd ³ (m ³)	Degree of construction difficulty		
	Easy \$/yd ³ (\$/m ³)	Average \$/yd ³ (\$/m ³)	Difficult \$/yd ³ (\$/m ³)
< 7,000 (5,400)	22.00 (29.00)	31.00 (41.00)	46.00 (60.00)
7,000 – 40,000 (5,400 – 30,000)	14.00 (18.00)	19.00 (25.00)	26.00 (34.00)
40,000 – 100,000 (30,000 – 75,000)	12.00 (16.00)	17.00 (22.00)	22.00 (29.00)
> 100,000 (75,000)	10.00 (13.00)	15.00 (20.00)	19.00 (25.00)

The above estimated unit costs apply best to soil-cement protection for earth dams and bank protection. For construction in years after 2002, the estimated unit costs should be increased to include increases in the costs of labor, equipment, and materials. A contingency factor may be applied to the above costs for estimating purposes.

The quantity of cement per cu yd (cu m) required depends on results of durability testing using soil aggregate proposed for the job. The unit cost of the cement depends on the mill base price and cost of delivery plus the contractor's cost for handling, overhead, and profit. If no soil-cement tests have been conducted and no cement prices are available, assume 300 lb of cement per cubic yard (390 kg per cubic meter) and obtain cement cost information from the latest Engineering News-Record or contact cement producers local to the area.

9 PERFORMANCE

9.1. Streambank Protection

Soil-cement streambank protection projects were severely tested during the large flows of 1983 and 1993 in Arizona. During these major flood events, soil-cement performed very well with little damage.

9.1.1. Tucson

In September 1983, Tucson, Arizona, received 6.7 in. (170 mm) of rain in a six-day period. This was more than 60% of the area's average annual precipitation of 11.05 in. (280 mm). The heavy rains caused the Santa Cruz River through downtown Tucson to reach a record flow estimated at 45,000 cfs (1,260 cu m per second). The water velocity was estimated to be greater than 20 ft/sec (6 m/sec). This was determined to be greater than the 100-year flood event. The flow on the Rillito River northeast of Tucson was estimated at 25,000 cfs (700 cu m per second), or greater than the 50-year flood event. During this event, there was severe damage to banks not protected by soil-cement (Figure 9-1).



IMG19646

Figure 9-1. Erosion of nonprotected bank causing damage to office complex adjacent to Rillito River during 1983 flood.



IMG19647

Figure 9-2. Bank erosion on Rillito River and soil-cement protected bank in foreground during 1983 flood.



IMG62039

Figure 9-3. Soil-cement shown in foreground performed well as the 1983 flood event nears its peak flow.

The performance of the more than 30 soil-cement streambank protection projects in the area was very good. The soil-cement withstood the flows with little to no damage (Figures 9-2 to 9-5). The only noticeable problem was in a few cases where overtopping of the soil-cement resulted in loss of soil directly behind the soil-cement (Figure 5-5).



IMG49266

Figure 9-4. Bridge over Santa Cruz River protected by soil-cement (shown in background) experienced no damage.



IMG55701

Figure 9-5. Condition of soil-cement bank protection as water recedes following 1983 flood.

9.1.2. Phoenix

In January 1993, fourteen consecutive days of rain produced an estimated discharge of 124,000 cfs (3,470 cu m per second) within a reach of the Salt River where soil-cement bank protection construction recently had been completed. The 100-year discharge for this reach was 210,000 cfs (5,880 cu m per second). The soil-cement performed well when subjected to high, turbulent flows containing a significant amount of sand and gravel. Because soil-cement performed well during the 1993 flood, soil-cement is now the predominant type of bank protection being used along the Salt River, the New River, and the Agua Fria River systems in the Phoenix metropolitan area.

The Salt River differs from the streams in Tucson and other parts of Maricopa County in several ways. Discharges on the Salt River are typically higher. This, along with its relatively steep gradient, allows it to transport sediment up to 15 in. (400 mm) in diameter. Approximately 33% of the bed mate-

rial in the Salt River is greater than 1.5 in. (38 mm), typically the maximum size used in soil-cement. In order to produce a more economical material capable of withstanding the erosive forces of the Salt River's large diameter sediment, cement stabilized alluvium (CSA) was developed by the Arizona Department of Transportation (ADOT) engineers and their consultants. CSA uses aggregate up to 3 in. (76 mm) in diameter.

In the Phoenix metropolitan area, the Salt River Channelization from 40th Street to Mill Avenue was completed in 1990. This was the first CSA project designed and constructed along the Salt River. The project included three new river crossings and three CSA grade control structures, with the channel alignment set to avoid landfill boundaries. The bank protection was a composite section with CSA utilized to the 10-year water surface elevation and gabions on the upper section.

Since the 1993 flood several noteworthy projects have been built in the Phoenix area. The Tri-City Landfill bank protection was completed for the Salt River Pima-Maricopa Indian Community in 1994 (Figure 9-6). The project protected two landfills, one on each bank, with approximately 10,280 lineal ft (3,130 m) of CSA. Toe-downs were adjusted during design to account for potential future mining downstream of the project. The upstream termination of the north bank measured 58 ft from the toe-down to top of bank, and accounted for 7.8 ft (2.4 m) of bend scour and 1.4 ft (0.4 m) of super elevation in addition to other scour and hydraulic components. The Tri-City Landfill project was the first CSA project on the Salt River constructed with 1:1 slopes.



IMG20815

Figure 9-6. Tri-City Landfill CSA bank protection along the Salt River.

The Sky Harbor International Airport Bank Protection and Reconstruction along the north bank of the Salt River was completed in 1996 (Figure 9-7). The project replaced earthen and gabion protection constructed in the early 1980s which was damaged during the January 1993 flow events.

Bank protection for the 19th Avenue landfill project extends from 19th Avenue to 15th Avenue. Construction was completed in March 1996 (Figure 9-8). The project provides protection to Cell A (200 acres) [81 hectares] on the north bank of the Salt River and Cell A-1 (15 acres) [6 hectares] located along the south bank. The project required construction of CSA bank protection under the existing 19th Avenue bridge. A grade control structure was constructed immediately downstream of the bridge.



IMG20816

Figure 9-7. CSA bank protection along the Salt River at Sky Harbor International Airport.



IMG19650

Figure 9-8. Upstream end of CSA bank protection along Salt River at 19th Ave.

9.2. Slope Protection for Dams

The Bonny Reservoir test section, built in eastern Colorado in 1951 by the Bureau of Reclamation. The test section was built to determine if soil-cement could be a viable and more economical alternative to rock riprap for upstream slope protection.

Two different soils were used at the Bonny test section. A fine silty sand required 12% cement by volume while a coarser silty fine to medium sand used 10% cement by volume. The average 28-day laboratory compressive strengths were 1140 psi and 880 psi (7.9 MPa and 6.1 MPa), respectively. After 10 years, cores drilled from the facing had approximately doubled in strength, averaging 2,000 psi and 2,160 psi (13.8 MPa and 14.9 MPa), respectively (Hansen 2001).

After 54 years of exposure to not only wave action, but also an average of 140 freeze-thaw cycles per year, the soil-cement remains hard and durable. Erosion of the compacted soil-cement surface has been minor. The test section has been subjected to an average of 140 freeze-thaw cycles per year. Except for some minor surface weathering after its first years of exposure, additional erosion of the test section has been insignificant (Figures 1-1 and 9-9).

Following a successful ten year monitoring period of the Bonny Reservoir test section, the Bureau began specifying soil-cement slope protection in 1963. Although most of the soil-cement projects built by the Bureau to date have performed well, two of the earliest projects — Merritt Dam in Nebraska completed in 1963 and Cheney Dam in Kansas completed in 1964 have experienced some damage (Figure 9-10). The damage that occurred was apparently due to two factors: (1) lack of bonding between soil-cement lifts, and (2) severity of the weather and wave action on the facing. The intense



IMG19652

Figure 9-9. Bonny Reservoir test section in 2005 after 54 years of exposure. Drought conditions in recent years has caused the reservoir water level to drop below the toe of the soil-cement.



IMG20682

Figure 9-10. Section of soil-cement slope protection on Cheney Dam, Kansas, heavily damaged following a major storm in 1971.

wave action and lack of bond caused the exposed individual soil-cement layers to pick-up and break apart from the embedded soil-cement. Repairs consisted of placing lean concrete patches in the effected areas (Figure 9-11) (Casias and Howard).

Another example of long-term performance of soil-cement slope protection for dams is the Castaic Dam in Castaic, California. A total of 250,000 cu yd (191,000 cu m) of soil-cement were placed in 1969-1970. Cement content was specified at 11.3% by volume. Thirty five years later, the soil-cement bank protection continues to provide excellent service with no appreciable erosion (Figure 9-12 and Figure 9-13).



IMG 19659

Figure 9-12. General view of Castaic Dam in Castaic, California, in 2006, 35 years after construction.



IMG 19656

Figure 9-13. Close-up of soil-cement bank protection at Castaic Dam, California, 35 years after construction.



IMG20681

Figure 9-11. Lean concrete used to repair damaged soil-cement slope protection on Cheney Dam, Kansas.

9.3. Soil-Cement Plating

The plating method although not as prevalent as the stair-step method for soil-cement slope protection has had many years of successful performance. As described in Chapter 1, the plating method is generally used in less severe applications. It is placed in single or multiple layers parallel to the slope. The plating method has been used successfully as slope protection on channels and small reservoirs. It has also been used for the emergency (auxiliary) spillways of medium sized earth dams.

9.3.1. Florida Power and Light, Sanford Plant

In the early 1970s soil-cement was used for the upstream slope protection for the Sanford Plant's cooling water reservoir and for the intake and discharge channels. The overall condition of the soil-cement is quite good after more than 30 years of service.

The intake and discharge channels at the plant were lined by the plating method (Figure 1-13). These channels are about 1 mile (1.6 km) long with a 20 ft (6 m) bottom width. The single 6 in. (150 mm) lining was placed by depositing centrally-mixed soil-cement on one side of the channel. This material was then pushed with bulldozers from the top on one side down the 3H:1V slope across the bottom and up the other 3H:1V slope. Compaction was accomplished using the placing bulldozer equipped with street pads to track over the soil-cement. Areas along the waterline have experienced some deterioration. It appears that the flowing water washes the sand subgrade out through cracks or openings in the soil-cement causing undermining and eventual breakage of the hardened soil-cement. These areas have been repaired by removing the loose material and filling the void with tremie concrete (Adaska 1980).



IMG19654

Figure 9-14. Soil-cement bank protection (plating method) for Manatee County Dam, Bradenton, Florida, 39 years after construction.

9.3.2. Manatee County Reservoir

Soil-cement was constructed in 1966 to protect the upstream slope of the dam at Manatee County Reservoir near Tampa, Florida. The plating method was used to build the soil-cement slope protection on the 3H:1V slope. The liner consisted of two 6-in. (300 mm) layers placed parallel to the slope. Track dozers were used to compact the soil-cement by making multiple passes up and down the slope. Over the past 39 years, only minor repairs were made which mainly consisted of covering soil-cement cracks with grout (Figure 9-14). The soil-cement's excellent durability is evidenced by the presence of original dozer compaction track marks observed in December 2005.

APPENDIX A

Table A-1. Summary of ASTM and AASHTO Specifications and Test Methods Used for Soil-Cement Construction

Document No.	Document Subject Title
ASTM C 33	Concrete Aggregates
ASTM C 42	Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
ASTM C 70	Surface Moisture in Fine Aggregates
ASTM C 94 AASHTO T 26	Concrete Uniformity Requirements
ASTM C 117	Material Finer than 75 Micrometer Sieve in Mineral Aggregates by Washing
ASTM C 136	Sieve Analysis for Fine and Coarse Aggregates
ASTM C 150	Portland Cement
ASTM C 171	Sheet Material for Curing Concrete
ASTM C 309	Liquid Membrane-Forming Compounds for Curing Concrete
ASTM C 566	Total Moisture Content of Aggregate by Drying
ASTM C 595	Blended Hydraulic Cements
ASTM C 618	Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete
ASTM C 989	Ground Granulated Blast-Furnace Slag
ASTM C 1040	Density of Unhardened and Hardened Concrete In-Place by Nuclear Methods
ASTM C 1138	Standard Method for Abrasion Resistance of Concrete (Underwater Method)
ASTM C 1157	Performance Specification for Hydraulic Cement
ASTM C 1240	Silica Fume
ASTM D 558 AASHTO T 134	Moisture-Density Relations of Soil-Cement Mixtures

Table A-1. Summary of ASTM and AASHTO Specifications and Test Methods Used for Soil-Cement Construction (continued)

Document No.	Document Subject Title
ASTM D 559 AASHTO T 135	Wetting-and-Drying Tests of Compacted Soil-Cement Mixtures
ASTM D 560 AASHTO T 136	Freezing-and-Thawing Tests of Compacted Soil-Cement Mixtures
ASTM D 698	Laboratory Compaction Characteristics of Soil Using Standard Effort
ASTM D 1556	Density and Unit Weight of Soil In-Place by the Sand-Cone Method
ASTM D 1557	Laboratory Compaction Characteristics of Soil Using Modified Effort
ASTM D 1632	Making and Curing Soil-Cement Compression and Flexural Test Specimens in the Laboratory
ASTM D 1633	Compressive Strength of Molded Soil-Cement Cylinders
ASTM D 1635	Flexural Strength of Soil-Cement Cylinders
ASTM D 2167	Density and Unit Weight of Soil In-Place by the Rubber Balloon Method
ASTM D 2901	Cement Content of Freshly Mixed Soil-Cement
ASTM D 2922	Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth)
ASTM D 3017	Water Content of Soil and Rock In-Place by Nuclear Methods
ASTM D 4318 AASHTO T 90	Liquid Limit, Plastic Limit and Plasticity Index of Soils
ASTM D 4643	Water (Moisture) Content of Soil by the Microwave Oven Method
ASTM D 5982	Cement Content of Fresh Soil-Cement (Heat of Neutralization Method)

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Selected Conversion Factors to SI Units

To Convert	Into	Multiply by
Square yard (yd ²)	Square meter (m ²)	0.8361
Square foot (ft ²)	Square meter (m ²)	0.0929
Foot (ft)	Meter (m)	0.3048
Inch (in.)	Millimeter (mm)	25.4
Ton (2000 lb)	Kilogram (kg)	907.185
Pound (lb)	Kilogram (kg)	0.45359
Pounds per square inch (psi)	Kilopascals (kPa)	6.8948
Cubic yard (yd ³)	Cubic meter (m ³)	0.7646
Horsepower (HP)	Kilowatt (kW)	0.7457
Fahrenheit (°F)	Celsius (°C)	$\frac{5}{9} (°F - 32)$
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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