RECLAMATION
Managing Water in the West

Design Standards No. 13

Embankment Dams

Chapter 16: Cutoff Walls
Phase 4 Final

U.S. Department of the Interior
Bureau of Reclamation

July 2014

Mission Statements

The U.S. Department of the Interior protects America's natural resources and heritage, honors our cultures and tribal communities, and supplies the energy to power our future.

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

Design Standards Signature Sheet

Design Standards No. 13

Embankment Dams

DS-13(16)-14: Phase 4 Final
July 2014

Chapter 16: Cutoff Walls

Foreword

Purpose

The Bureau of Reclamation (Reclamation) design standards present technical requirements and processes to enable design professionals to prepare design documents and reports necessary to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public. Compliance with these design standards assists in the development and improvement of Reclamation facilities in a way that protects the public's health, safety, and welfare; recognizes needs of all stakeholders; and achieves lasting value and functionality necessary for Reclamation facilities. Responsible designers accomplish this goal through compliance with these design standards and all other applicable technical codes, as well as incorporation of the stakeholders' vision and values, that are then reflected in the constructed facilities.

Application of Design Standards

Reclamation design activities, whether performed by Reclamation or by a non-Reclamation entity, must be performed in accordance with established Reclamation design criteria and standards, and approved national design standards, if applicable. Exceptions to this requirement shall be in accordance with provisions of *Reclamation Manual Policy*, Performing Design and Construction Activities, FAC P03.

In addition to these design standards, designers shall integrate sound engineering judgment, applicable national codes and design standards, site-specific technical considerations, and project-specific considerations to ensure suitable designs are produced that protect the public's investment and safety. Designers shall use the most current edition of national codes and design standards consistent with Reclamation design standards. Reclamation design standards may include exceptions to requirements of national codes and design standards.

Proposed Revisions

Reclamation designers should inform the Technical Service Center (TSC), via Reclamation's Design Standards Website notification procedure, of any recommended updates or changes to Reclamation design standards to meet current and/or improved design practices.

Chapter Signature Sheet
Bureau of Reclamation
Technical Service Center

Design Standards No. 13

Embankment Dams

Chapter 16: Cutoff Walls

DS-13(16)-14:[1] Phase 4 Final
July 2014

Chapter 16, "Cutoff Walls," is an existing chapter within *Design Standards No. 13 – Embankment Dams*, and was revised to include the following changes and additions:

- Updates to recommended specification standards for slurry materials
- Additional discussions on alternative forms of cutoff walls including secant pile, geomembrane, sheet pile, deep-soil-mixing (DSM), and jet grouted cutoff walls
- Updated summary table of Reclamation cutoff wall experience including type of cutoff wall, key design parameters, and dam name
- Updated references

[1] DS-13(16)14 refers to Design Standards No. 13, chapter 16, revision 14.

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Chapter 16

Cutoff Walls

16.1 Introduction

16.1.1 Purpose

Seepage barriers, or cutoff walls, can be an effective means for controlling seepage through an earthfill dam or its foundation. Since approximately the early 1970s, the various types of cutoff walls available to embankment designers have increased with advances in construction methodologies and technology. The industry has made significant improvements in the equipment used to construct cutoff walls and in the efficiency of their operation. Increasingly, cutoff walls are being considered as a primary modification component within existing earthfill embankments that have a history of chronic seepage related problems or that undergo an acute seepage incident. With the wide array of cutoff wall alternatives, they can be the technically preferred and most cost-efficient method to remediate an identified seepage problem given the difficulties of creating an earthfill cutoff within an existing embankment with an active reservoir.

Construction of conventional rolled earthfill cutoff trenches, whether in a new earthfill dam or as a component to the modification of an existing dam, can be a prohibitively expensive operation, depending on the required depth, the existing natural groundwater conditions, and the availability of suitable impervious material. Cutoff walls can be a viable alternative for reducing seepage through embankment dams and their foundations. Cutoff walls can be constructed by a variety of methods that do not require foundation dewatering and that greatly reduce the amount of excavation from that required for a rolled earthfill cutoff. Another advantage to cutoff walls is that they can often be constructed in very limited working spaces. This chapter is intended to present general design considerations and guidance for the most widely used types of cutoff walls currently accepted as viable alternatives to the more conventional rolled earthfill cutoff trenches or for use as seepage barriers where remediation of adverse seepage conditions may be required.

16.1.2 Scope

This chapter provides general considerations/guidance for the design and construction of the most commonly employed types of cutoff walls currently used in embankment dam applications. Detailed design criteria have been included when appropriate. However, in keeping with the purpose of this chapter, emphasis has been placed on providing general considerations and information that will assist the designer in developing the most cost-effective design for seepage conditions at a given site. This chapter is not intended to be a step-by-step design guideline. It is the designer's responsibility to closely evaluate the site-specific conditions and select the most cost-effective type of cutoff wall that will satisfy the technical requirements for acceptable seepage control. This guideline is not intended to be a substitute for sound and thorough site-specific design investigations and evaluation of site-specific design parameters. Whenever possible, specific design details and references are included as additional tools for the reader. This chapter is intended to present the most recent state-of-practice methodologies for cutoff wall design and construction. However, the industry is constantly changing, with improvement to both the design and construction practices for cutoff walls. The reader is therefore encouraged to seek out the most recent literature on design of cutoff walls and case histories of their implementation in the field to gain the broadest knowledge available.

16.1.3 Deviations from Standard

Designers of embankment dams within the Bureau of Reclamation (Reclamation) should adhere to concepts and methodologies presented in this design standard. Many of these concepts and methodologies have withstood the rigors of construction and operation in the field and have been proven successful. Rationale for deviation from the standard should be presented in the technical documentation for the dam and should be approved by appropriate line supervisors and managers. In addition, as required in Reclamation, any design should be independently reviewed by known experts in the field, outside of Reclamation, as part of the design process.

16.1.4 Revisions of Standard

This standard will be revised periodically as its use indicates the need. Comments and/or suggestions should be sent to the Bureau of Reclamation, Technical Service Center, Denver, CO, Attention: Geotechnical Services Division, Code: 85-830000, Denver Federal Center, Denver, Colorado, 80225.

16.1.5 Applicability

The guidelines presented in this chapter should be applied to all cutoff walls used for control of seepage where installation would result in a significant differential gradient across the cutoff wall. Examples include the use of cutoff walls to control flow beneath or around hydraulic structures such as earth or rockfill dams. In addition, the use of cutoff walls is occurring more frequently along existing canal levees where seepage barrier design features are often less robust than those used for an embankment dam. The use of erosion-resistant cutoff walls should also be considered as a structural feature when concerns exist for particle migration and/or backwards erosion within an embankment or foundation. Cutoff walls can also aid in dewatering by limiting inflows into excavations and reducing pumping requirements. These guidelines are applicable to any of the cases cited above, but they should not be applied directly to structural support walls. Structural support walls may use similar construction methods, but they will generally have very different applied loadings and performance criteria which are not covered in this design standard. In each specific case, the designer must be aware of the overall performance and durability requirements for each particular application. Use of cutoff walls to contain and isolate hazardous waste requires that more attention be given to reducing permeability and increasing chemical resistance of the cutoff wall and is beyond the scope of this standard. In all cases, the designer should perform a literature review and be aware of new technologies, methodologies, and state-of-the-practice changes in the design and construction of cutoff walls.

16.2 Design Principles

16.2.1 General

The primary use of cutoff walls with embankment dams is to control seepage through either the embankment or the foundation. Cutoff walls can seldom be considered completely effective (due to the inability to see into the subsurface during their construction). Thus, they are most often incorporated into an overall system of seepage control that includes both seepage reduction and drainage features with properly designed filter elements. In essence, this is the same approach that would be used if designing a new state-of-practice embankment dam because no form of cutoff can be considered 100-percent effective.

16.2.1.1 General Principles of Cutoff Wall Design

Cutoff walls are used to reduce seepage by dissipating seepage energy at locations generally upstream of the dam centerline, where high pore water pressure and high seepage gradients are much less likely to have detrimental impacts on overall performance of the dam. This does not imply that other geometries are less desirable. For many existing structures, the degree of reservoir drawdown required to construct an upstream cutoff wall on an existing embankment may prevent consideration of the upstream geometry alternative. The same issue may also exist when considering the use of cutoff walls in canal levees where limitations to unwatering the canal during construction may prevent consideration of the upstream cutoff wall geometry.

Cutoff walls through foundations in Reclamation structures have been installed both as part of the original construction and as modifications to existing structures. In most applications of cutoff walls within Reclamation embankments, the cutoff wall has been installed as a modification with a few exceptions. This is mainly due to the benefit of an engineered earthfill cutoff trench and development of new, more economical, and technically sound cutoff wall options starting in the

1960s. Many modifications to existing embankments (Navajo Dam, New Mexico; Fontenelle Dam, Wyoming; Meeks Cabin Dam, Wyoming; A.V. Watkins Dam, Utah), and limited cases of new embankment construction (Diamond Creek Dike, Wyoming; New Waddell Dam, Arizona; Coquille Dams, Oregon), have cutoff walls near the center section of the dam and have performed successfully.

Another use of cutoff walls is to reduce seepage inflow from recharge sources when dewatering an excavation (Bradbury Dam, California; Virginia Smith [Calamus] Dam, Nebraska; New Waddell Dam, Arizona), which can significantly reduce the costs of dewatering. Cutoff wall construction methods can also be used for structural applications, as was the case in the key block construction at Mormon Island Auxiliary Dike (MIAD) in California. Secant piles were constructed as structural elements to frame the four-sided vertical excavation of potentially liquefiable soils at the downstream toe of MIAD. The secant piles were reinforced with steel for additional bending strength and provided a seepage cutoff against groundwater intrusion. The secant piles served as structural retaining walls that allowed full excavation to bedrock and backfilling of each key block with concrete and compacted soil.

Any preliminary design for a cutoff wall must include a sound geologic understanding of the subsurface materials including groundwater conditions. To minimize the risk of construction failures, the designer should have a good understanding of the groundwater conditions, soil types, and bedrock characteristics (for example, karst conditions, fracture density, etc.). This can include, but is not limited to, the presence of large boulders, cemented zones, artesian water conditions, or soft, clay layers in the overburden. Bedrock characterizations should include bedrock surface details (i.e., variation in surface elevation including the presence of scour/erosion surfaces and alluvium-filled channels), weathering profile, fracture and joint orientation, density, and hardness. With these data, the designer can more easily focus on the best type of cutoff wall to achieve the design objectives, at the least cost, while minimizing the possibility of construction issues that can result in significant cost increases, ineffective seepage control, and schedule delays.

16.2.1.2 Risk-Informed Decision in Selection of Cutoff Wall Type

The selection of cutoff wall type should always include consideration of the range of seepage-related failure modes that the wall is being designed to protect against. Each design, by necessity, must consider the technical requirements, availability of local materials, bentonite and cement supply, and construction restrictions when selecting the most cost-effective type of cutoff wall to achieve the design objectives. These factors will always impact the potential costs. However, cost alone should not be the only criterion used as the basis for selection of the best alternative at a given site. The current practice within Reclamation is to also evaluate each alternative within a risk-based framework. This requires a team of technically qualified persons to identify each potentially reasonable failure mode and, through the use of event tree analyses, quantitatively estimate the amount of risk reduction each alternative is likely to achieve. Computing risk reduction requires that a baseline risk estimate be determined initially for a structure without the cutoff wall. A risk-based alternatives study allows the designer to go beyond cost alone and include an assessment of technical strengths and weaknesses of each alternative in terms of the degree of risk reduction each alternative can achieve compared to the baseline. When combined with estimated costs for each alternative, the designer and decisionmakers are then able to include these factors when selecting the cutoff wall type. A discussion of the methodologies to estimate and quantify risk reduction is beyond the scope of this guideline. However, the reader should be aware that the risk-informed approach to alternative selection is an integral part of Reclamation's state-of-practice at the time of this writing and is a useful tool in the decisionmaking process when considering the purpose and/or type of cutoff wall.

16.2.1.3 Performance Considerations

To be the most effective, cutoff walls must fully penetrate pervious strata. Figures 16.2.1.3-1 and 16.2.1.3-2 illustrate a general schematic cross section of fully and partially penetrating cutoff walls through an embankment, respectively. Based on flow net analyses of partially penetrating cutoffs by Cedergren [1], a cutoff penetrating 50 percent of a pervious stratum would typically be expected to provide only an approximate 20-percent reduction in the quantity of seepage. However, partially penetrating cutoff walls in pervious strata are often used as a defensive measure to cut off any undiscovered highly pervious zones in the upper and mid-portions of the strata. Cutting off these pervious zones can greatly reduce the chance that the capacity of an engineered embankment drainage system, based on the more general characteristics of the pervious strata, will be exceeded. In some cases, a partially penetrating cutoff wall composed of more erosion resistant material such as cement-bentonite (CB), driven sheet pile, or plastic concrete might be used to introduce a vertical, erosion resistant structural element through a permeable zone where there are concerns for backward piping erosion. The use of a partially penetrating cutoff wall for this purpose should carefully consider gradients at the wall interface and around the ends of the wall. Gradients are known to generally increase at the end of a cutoff wall unless it is laterally keyed into a more impermeable stratum. Past research, as well as some more recent research, has shown that soil-bentonite (SB) and even CB walls can be erodible under certain seepage conditions (critical velocity) and gradients [3]. Erosion would generally be most likely to occur at a crack or defect in the wall, or at the ends of a wall, where the velocity through or around the wall would be higher. The designer should always consider the likelihood that a cutoff wall is cracked or that flaws exist. The impact of this on confidence in the performance of the cutoff wall can be accounted for in the risk reduction analyses.

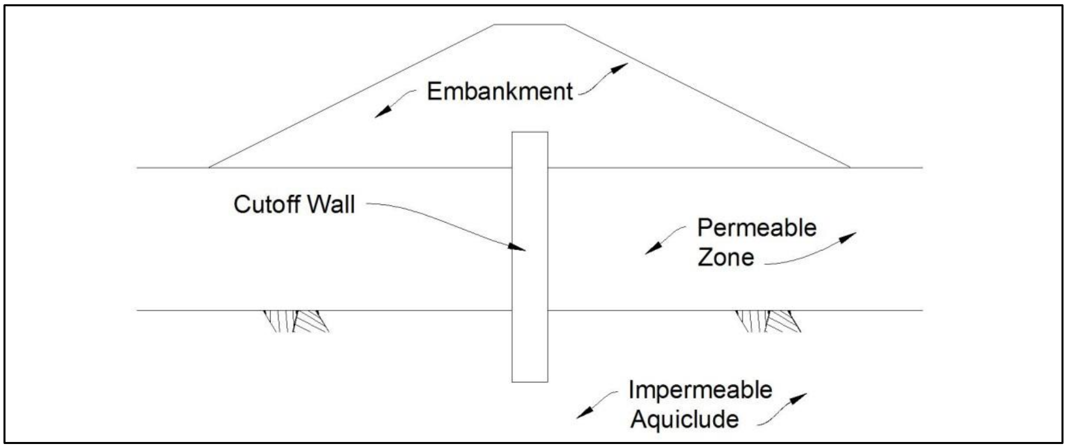

Figure 16.2.1.3-1. Fully penetrating cutoff wall (blocks seepage).

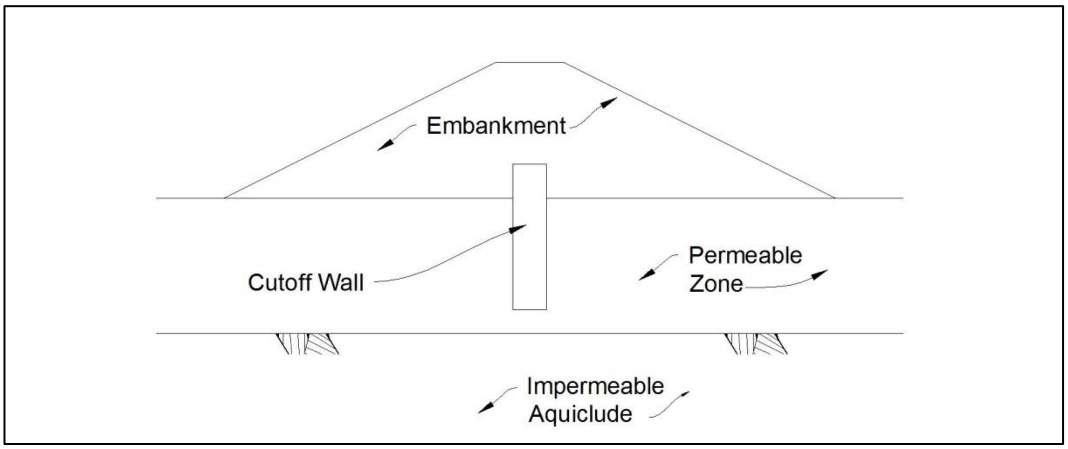

Figure 16.2.1.3-2. Partially penetrating cutoff wall (lengthens seepage path).

Partially penetrating cutoff walls are even more inefficient at reducing uplift pressures and exit gradients than they are at controlling seepage under a dam. Generally, a partially penetrating cutoff in a pervious stratum should not be considered to have any significant impact on exit gradients, unless it is relatively deep with respect to the length of the longitudinal seepage path. Additional seepage control measures, such as filtered foundation drains or toe drains, should generally be provided in such applications. If a fully penetrating cutoff is terminated in a semi-pervious stratum, the depth of penetration into the semi-pervious strata and its effect on actual reduction in exit gradients should be carefully considered. Methods of investigating the effectiveness of a particular cutoff application are presented in *Design Standards No. 13 – Embankment Dams*, "Chapter 8, Seepage."

In addition to concerns for high seepage gradients to form beneath and/or around a cutoff wall, design consideration must often be given to the connection of a new cutoff wall with an existing structure that penetrates an embankment. This could be a spillway or outlet works conduit or a pipe penetration. Without a well-designed connection at these structures, high seepage gradients can form at the contact. High gradients at these contacts can be especially problematic in that these are also points where low stresses or poor compaction may exist. A poor connection could potentially increase the risk of initiation of internal erosion due to increased seepage gradients. Some examples of connections that have been used in Reclamation cutoff wall designs are included in this document.

16.2.1.4 Field Explorations

Field explorations required for cutoff wall design will vary from site to site. However, at a minimum, sufficient drilling and sampling along the alignment of the proposed cutoff wall should be completed to characterize the geologic profile. This should include (but not necessarily be limited to):

- Complete delineation of all soil types, and spatial distribution of soil types, beneath and adjacent to the alignment of the proposed cutoff wall

- Depth to bedrock and/or aquiclude along the entire alignment (for evaluating a fully penetrating cutoff wall)

- Depth to groundwater and annual variation of groundwater depth

- Coring of bedrock to determine hardness, weathering profile, bedding, etc.

- Identification of the overburden material types, gradation, oversize, and presence of potential cemented and/or hardpan layers

- Delineation of potentially weak layers such as soft clays, peat, etc.

16.3 Types of Cutoff Walls

There are many different types of cutoff walls that have practical applications in combination with embankment dams or canal levees. Some types are more universally applicable to earthfill structures, for a variety of reasons that must be considered by the designer and client, and are outlined throughout this document. Other types may not be as common, but they have strong advantages under site-specific conditions. For the guidelines within this design standard, the various types of cutoff walls have been divided into five specific categories and one general category:

A. Earth-backfilled slurry trench cutoff walls (SB)
B. CB cutoff walls and soil-cement-bentonite (SCB)
C. Concrete cutoff walls including plastic concrete cutoff walls
D. Geomembrane cutoff walls

E. Deep soil mixing (DSM) cutoff walls
F. Others: Includes secant pile, sheet pile, and jet grouted cutoff walls

Each of the different types of cutoff walls will be addressed in the following sections of this design standard. In addition, general guidance will be provided for contract specifications documents such as suggested submittals, test section requirements, and contracting methodologies that may be the most desirable for a particular type of cutoff wall. Table 16.3.1 provides a summary of Reclamation's experience with various cutoff wall types since approximately the mid-1970s.

In theory, all of the major types of cutoff walls can be designed to provide a positive seepage cutoff. However, there are major uncertainties in the design and construction of all types of cutoff walls. The designer should have a thorough understanding of the strengths and weaknesses of each type of cutoff wall as a basis for selecting the type of cutoff wall at any site. Each application of a cutoff wall to a given site is unique and will depend largely on the intended function of the wall, in situ foundation conditions, hydraulic gradients anticipated across the wall, constructability issues, groundwater conditions (i.e., fresh or salt water), and economic considerations. Excluding cost, anticipated hydraulic gradient across the cutoff wall and depth of cutoff wall are the two considerations that are most likely to influence the selection of preferred cutoff wall type. At high enough gradients (producing high concentrated seepage velocities through a defect), and with the right soil and/or bedrock conditions adjacent to the wall, both SB and CB cutoff walls are potentially erodible if cracks are present. Plastic concrete and traditional concrete cutoff walls can generally be considered nonerodible, except possibly under very extreme conditions. Such conditions would include very high velocity concentrated seepage flow in alignment with voids or open-work coarse soils that could serve as a repository for eroded material and prevent attenuation of seepage velocity. Laboratory testing has shown that SB, SCB, and CB can erode under relatively low-velocity flows through cracks [3]. For this to happen, soil conditions adjacent to the wall, both upstream and downstream, must not permit self-filtering through these adjacent soils, or sufficient void space must be available, for erosion to progress. However, under high expected seepage gradients across the wall, the potential for erosion must be considered and will impact the selected cutoff wall type.

Table 16.3-1. Reclamation Cutoff Wall Projects[1]

Number	Project	Feature	Solicitation/ Specification Number	Cutoff Wall Material	Wall Dimensions Width (ft)	Wall Dimensions Length (ft)	USCS – Gradation Backfill	Maximum Depth (ft)
1	Pick-Sloan Missouri Basin Program, North Dakota	Wintering Dam	DC-7142	SB	5	1,160	GC-CW	95
2	Seedskadee, Wyoming	Fontenelle Dam Test Section	4-SP-40-02380	Concrete (4,000 lb/in^2)	2	1,315	N/A	172
3	Seedskadee, Wyoming	Fontenelle Dam Phase II - Modification	4-SP-40-04900/ DC-7710	Concrete	2	4,615	N/A	166
4	Central Arizona, Arizona	New Waddell Dam	RFP 6-SP-30-04690	Concrete	3.25 3.25	80 400	N/A	82 170
5	Central Arizona, Arizona	New Waddell Dam	N/A	Secant pile (concrete) (five walls)	Variable	Variable	N/A	130
6	Central Arizona, Arizona	New Waddell Dam	9-SI-32-00930 DC-7797	SB	4	1,460	NA	45
7	Colorado River Storage, New Mexico	Navajo Dam	RFP 6-SP-40-03900	Concrete	3.25	450	N/A	400
8	Pick-Sloan Missouri Basin Program, Nebraska	Calamus Dam (two widths)	7D-C7469	SB	3 5	4,000 3,000	SM	115
9	Lyman, Wyoming	Stateline Dam	No specification	CB			N/A	

Table 16.3-1. Reclamation Cutoff Wall Projects (continued)

Number	Specification Total Area (ft^2)	Total Cost as Bid ($\$/ft^2$)	Wall Construction Date Start	Wall Construction Date Complete	Average Rate of Construction (ft^2/day)	Excavation Method	B/W for Slurry by Weight (%)	C/W by Weight (%)	Additives % of C or B by Weight	Notes
1	N/A	5.98	9/18/75	11/18/75 Trench failure	N/A	Drag line	20 lb bentonite to 40 gal slurry	N/A	Soda ash added to water (3 lb/300 gal)	See regional report, "Construction of Slurry Trench Cutoff for Wintering Dam" [37]
2	160,538	32.30	9/85	10/86	2,290	Hydrofraise (rockmill)	N/A	2.32	N/A	Deepest production use of a rockmill at the time
3	726,236	26.44	8/88	8/89	2,370	Hydrofraise (rockmill)	N/A	N/A	N/A	Rockmill is the generic name for Hydrofraise
4	5,500 50,000	134.22 78.08	10/86	7/87	A = 51.2 B = 11.7 C = 4.3 D = 2.4	Clamshell (blasting)	5	N/A	N/A	First application of a cutoff wall in a new dam by Reclamation
5						Drill	N/A	N/A		Five separate walls were constructed to cut off weathered bedrock and buried alluvial channels
6						Backhoe				
7	132,000	75.00	5/87	4/88	Not available	Hydrofraise	5	N/A	N/A	Deepest concrete diaphragm wall built in the United States on an existing embankment dam at the time (January 1990)
8	439,000	3.80 6.90	7/82	7/84	1,500	Dragline, backhoe, clamshell	5	N/A	N/A	
9										Information not available

Table 16.3-1. Reclamation Cutoff Wall Projects (continued)

Number	Project	Feature	Solicitation/ Specification Number	Cutoff Wall Material	Wall Dimensions		USCS – Gradation Backfill	Maximum. Depth (ft)
					Width	Length		
10	Shoshone, Wyoming	Diamond Creek Dike	DC-7779	CB	2	2,500	N/A	156
11	Shoshone, Wyoming	Diamond Creek Dike	DC-7779	SB	4	5,190		
12	Minidoka, Wyoming	Jackson Lake Dam	DC-7695	Soil-mix wall (DSM)	31 (average)	3,985	Alluvium mixture of sands and gravels with silt	100 62 (average)
13	Newlands, California	Lake Tahoe Dam	20-CO299	Secant pile	2	90	Concrete with steel beams in primary piles	21
14	San Angelo, Texas	Twin Buttes Dam	1425-5-SP-60-07610 60-CO339	SCB	2.5	21,000	(SM)g	100
15	Weber Basin, Utah	A.V. Watkins Dam		CB	2.5	32,000	N/A	65
16	Central Arizona, Arizona	Reach 11 Dikes	2-SP-30-09520 DC-7880	Geomembrane	80 mil	66,000	ASTM C33 concrete sand	65
17	Lyman, Wyoming	Meeks Cabin Dam	DC-7881	Plastic, concrete	3	850	N/A	170
18	Cachuma, California	Bradbury	CO431/1425-5-CC-20-03270	SB	4	930	(SC-SM)g	85
19	Coquille, Oregon (BIA)	Tarheel and Fourth Creek Dams	03SP101521	Composite sheet pile	N/A	920 (two dams)	N/A	40
20	Central Valley, California	Mormon Island Auxiliary Dike	R10PS20114	Secant pile	1 meter diameter	2735	N/A	43 -79
21	Yakima Project, Washington	Keechelus Dam	02SP101485	SB	4	750	(SC-SM)g	75

Table 16.3-1. Reclamation cutoff wall projects (continued)

Number	Specification Total Area (ft^2)	Total Cost as Bid ($\$/ft^2$)	Wall Construction Date Start	Wall Construction Date Complete	Average Rate of Construction (ft^2/day)	Excavation Method	B/W for Slurry by Weight (%)	C/W by Weight (%)	Additives % of C or B by Weight	Notes
10			11/16/89	06/30/90		Backhoe	5.6	17.5 – 19.5 ~ 18%	Spersene - 3 gal/1,000 gal water	CB cutoff wall in embankment keyed 20 ft into bedrock and constructed in 30-ft vertical segments. UCS = 40 lb/in^2
11	171,981		08/14/89	10/12/89	3,300 – 4,800	Backhoe	5	NA		Excavated 5 ft through working platform and 5 ft into bedrock
12	248,312		07/14/87	07/19/88		N/A				W/C ratio of injected grout equal to 1.25
13	1,890		1988	1988		26-inch bucket auger				4,000 lb/in^2 concrete with reinforcement (W14x38).
14	1,400,000		1996	1999		Kelly and cable suspended grabs and hydromill	5+1 for bentonite slurry plus backfill	34		Maximum size 1-1/2 inches in backfill SCB. Cementitious material included flyash and cement. Tremie placement.
15	1,570,000			11/21/2008	19,000	Backhoe	6	18	Lignosulfonate	Lignosulfonate added as a set retarder and plasticizer
16	2,727,000	8.45 [1]	12/1993	02/1995	15,000	Backhoe	N/A	N/A	N/A	Excavated using biopolymer slurry for trench support (EZ Mud)[1]
17	125,000	17.38	05/03/94	06/15/95	115 ft^2/hr, based on cutter wheel	Clamshell Hydrofraise	5	15 cement	3 bentonite	Coarse to fine aggregate ratio of 1:1. Target plastic concrete strength = 400 lb/in^2 UCS at 28 days.
18	47,400	19.05	5/1/1995	5/25/1995	1,975	Backhoe	5	N/A	100 lb soda ash/9-12 tons B	Backfill was imported from commercial source.
19	31,000 (two dams)	15.30	8/2004	11/2004		N/A	N/A	N/A	N/A	39-ft-long, fiber reinforced, composite polymer driven 20 ft into foundation with 19 ft left above surface and embankment built around it
20	71,100 lin ft	$207/LF	02/2011	01/2013		Drill				
21	51,000	8.10	9/21/2002	10/16/2002	2,200	Backhoe	5	N/A		

[1] Includes filter and geomembrane.

Note: B/W = bentonite to water ratio, CB = cement bentonite, C/W =cement to water ratio, DSM = deep soil mixing, ft = feet, ft^2 = square feet, $/ft2 = dollars per square foot, ft^2/day = square feet per day, ft^2/hr = square feet per hour, gal = gallons, lb = pounds, lb/in^2 = pounds per square inch, lin ft = linear feet, N/A = not available, SB = soil bentonite, SCB = soil cement bentonite, USCS = United States Conservation Service, W/C = water to cement.

The required depth of the cutoff wall will also be a primary consideration in the type of cutoff wall that is selected. Although the vertical reach of long-stick backhoes has increased over time, limitations in depth of excavation capability still exist. In addition, they can become inefficient at digging tough or coarse soils at the outer range of their capability. Once the proposed depth of cutoff exceeds the reasonable capability of a backhoe, the method of excavation will generally be limited to dragline, clamshell, or rock cutter and will typically be constructed in individual panels (discussed later in this design standard under Section 16.7, "Concrete Cutoff Walls"). Deep cutoff walls that are excavated using panel-type construction sequencing must be tremie backfilled due to their depth and limited length. This method of placement is more applicable to higher density backfill, such as concrete or plastic concrete, due to the requirement that the backfill be heavy enough to displace the trench support fluid (typically bentonite slurry).

Other design considerations are too numerous to describe in detail but may nevertheless be important considerations in the appropriate type of cutoff wall for a particular application or site, and the designer should be aware of these variables. Some of these other variables will become apparent in the following paragraphs that describe other various cutoff wall types.

16.4 Earth-Backfilled Slurry Trench Cutoff Walls (Soil-Bentonite)

An earth-backfill slurry trench cutoff wall is constructed by excavating a narrow vertical trench that is typically 2 to 5 feet wide. If a positive cutoff is required, the excavation is generally carried through the materials requiring a cutoff to a relatively impervious underlying stratum and keyed into that stratum a minimum of 3 to 5 feet. This may include excavating the cutoff through an existing embankment and through a pervious foundation. The trench is filled with a slurry suspension of bentonite in water during the excavation. Fresh slurry is added as required during the excavation to maintain a constant level of slurry near the top for stability of the trench. The water from the slurry suspension bleeds into the sides of the trench and leaves behind a thin densely packed layer of colloidal particles that acts as a membrane, commonly referred to as a "filter cake." The hydrostatic force of the slurry, acting in combination with the filter cake, provides stability to the sides of the trench. The filter cake also provides the primary contribution to the overall low permeability of the completed cutoff wall [4]. Figures 16.4-1 and 16.4-2 show excavation of a SB cutoff wall at Bradbury Dam and the equipment used to mix the bentonite slurry, respectively.

Figure 16.4-1. Excavation of SB cutoff wall using extended reach backhoe. Note inspector using weighted tape to measure depth (Bradbury Dam, California).

Figure 16.4-2. SB equipment for mixing slurry (high-speed colloidal mixer shown) (Bradbury Dam, California).

After the trench has been excavated to final grade, the trench is backfilled. The backfill is generally made by mixing excavated spoil or material from required excavation from the trench with slurry and earth materials (soil) from additional sources, as required to obtain the desired engineering properties. The backfill is placed by use of tremie, clamshells, or pushing the mixture with a bulldozer into the trench, while displacing the slurry suspension (see figure 16.4-3 for schematic of SB cutoff wall construction). In many cases, the initial placement of backfill is made using tremie methods until the backfill material appears above the slurry in the trench, and a natural slope of the backfill in the slurry-filled trench is achieved. Once this occurs, the backfill is generally pushed into the trench with a bulldozer. Careful measurements of the top of backfill and bottom of the trench are required to ensure that the toe of the backfill does not encroach upon the excavation operation, or the excavated material could mix with the backfill and result in unmixed pockets.

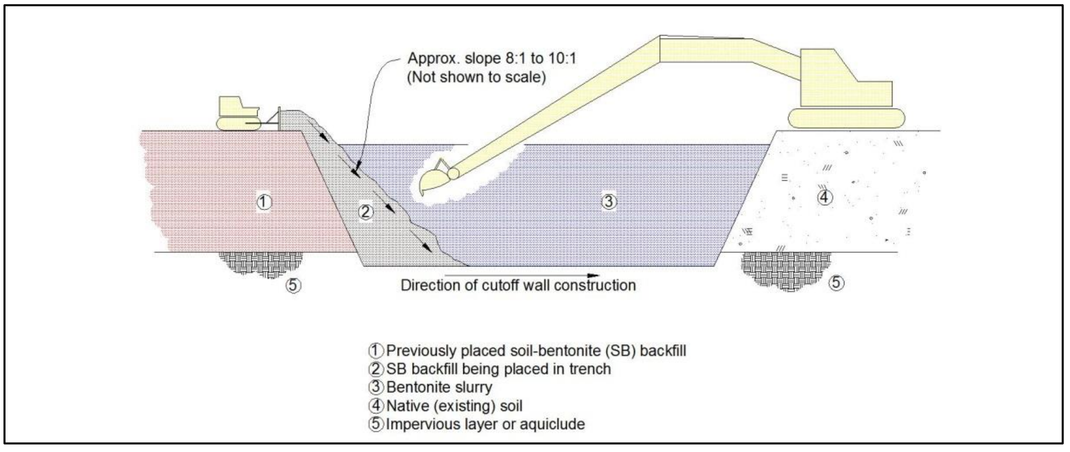

Figure 16.4-3. Schematic of SB trench construction.

16.4.1 Bentonite Slurry Mix Design

Bentonite slurry mixtures are used as temporary trench support for different cutoff wall applications including soil-backfilled slurry trench cutoff walls, concrete cutoff walls, and plastic concrete cutoff walls. When bentonite slurry support for cutoff wall construction is required, the general design principles outlined in the following paragraphs are applicable.

Bentonite slurry should generally consist of a colloidal suspension of pulverized bentonite in water. The bentonite should be a naturally powdered, and similar to pure, premium grade Wyoming-type, sodium cation-base bentonite with high swelling characteristics. Bentonite in pellet form should not be used. Currently, there are very few commercial bentonites available that do not have some form of inorganic additives or extenders added to increase yield. Both chemical and physical additives may be used to improve viscosity, density, gel strength, and fluid loss. Use of these products is generally considered acceptable, depending on the particular application. Some of these additives, and their purpose, can be found in reference [5]. However, the designer should give consideration to potential detrimental effects of overuse of additives (particularly organic additives such as carboxymethyl cellulose [CMC] used to increase viscosity) on the long-term performance of the cutoff wall. This particular organic based product may lead to a long-term increase in the permeability of the bentonite filter cake at the trench wall through decomposition within the filter cake itself. Other additives are too numerous to cite here, and new additives are being developed at any given time. The designer cannot be expected to know each possible additive that may be proposed by a contractor and its potential effects on the long-term cutoff wall performance. However, the designer should be aware that additives may be used and should research any available sources that provide data on the potential consequences of their use. The designer should always require the contractor to list proposed additives in the bentonite within the submittal requirements of the specifications so that any potential conflicts can be researched and/or identified and evaluated (see Section 16.6.1, "Submittal Requirements for Cement-Bentonite and Soil-Cement-Bentonite Cutoff Walls").

The concentration of bentonite in the slurry (measured as the weight of dry bentonite to the weight of water) should generally be on the order of 5 percent, depending on the type of additives used and the required properties of the slurry.

16.4.1.1 Slurry Properties

The five slurry properties most significant to satisfactory performance are presented in the following paragraphs to familiarize the designer with the impact each property has on the construction and long-term performance of the cutoff wall. However, only two of these properties (density and viscosity) are readily measured in the field. In addition, effecting changes in these properties generally requires addition of additives that may have a detrimental impact on the long-term performance of the cutoff wall. General practice should be to specify only the grade of bentonite and bentonite content of the slurry prior to introduction into the trench. Laboratory tests for other slurry properties can be performed on slurry trial mixes during the design phase.

A. **Density.** The density or unit weight of the slurry provides the hydrostatic force necessary to provide stability to the sides of the trench during excavation. Density of the slurry is determined by the weight of water plus the amount of colloidal and noncolloidal solids in the suspension. The density of a bentonite slurry for a typical 5-percent (by weight) concentration is on the order of 65.5 pounds per cubic foot (lb/ft^3). Once the slurry is in the trench, it suspends noncolloidal solids (silts and sands), and slurry density can easily exceed 80 lb/ft^3. In general, a 5-percent bentonite concentration can easily maintain a 70- to 75-lb/ft^3 density once it is in the trench. Stability analyses should be performed using the 70- to 75-lb/ft^3 density. If additional hydrostatic pressure is required for stability, provisions for increasing the slurry head should be provided in the design. The designer should not specify use of additives or increased agitation in the trench to increase densities above these values. A maximum value should also be placed on the density of the slurry in the trench. D'Appolonia [4] has shown that this value should be at least 15 lb/ft^3 lower than the maximum total unit weight of the backfill (typically 105 lb/ft^3) but should not be overly restrictive. Generally, a maximum slurry density of 90 lb/ft^3 is considered acceptable. This density ensures the displacement of the slurry by the backfill during placement. Slurry with a high sand content has a high density. Because the effective placement of backfill relies on the difference in densities of the slurry and the backfill, a high slurry density may result in unsatisfactory backfill placement and/or consistency. Also, a high sand content may result in more material settling to the bottom of the trench, thereby affecting placement and creating a permeable zone at the base of the cutoff wall. The designer should verify the suitability of the slurry in the trench to its full depth prior to backfill placement. This is best accomplished by testing the slurry in the trench, especially near the bottom, for sand content and requiring the use of desanders, if necessary, to remove sand prior to backfill placement. This concern applies to any type of cutoff wall that requires slurry support and displacement of the slurry by the backfill material used for the cutoff wall. These requirements are part of standard specification requirements in Reclamation practice.

B. **Viscosity.** The slurry suspension should have sufficient viscosity to limit penetration of slurry into the in situ soil structure prior to formation of the filter cake. Initially, hydrostatic pressure is dissipated across the distance of slurry penetration. In general, the farther the penetration, the lower the percentage of hydrostatic pressure actually acting on the potential failure wedge. Once the filter cake has been established, the hydrostatic force is transmitted to the filter cake on the face of the trench, and the original distance of penetration loses its significance. Viscosity of the slurry generally becomes more important in coarser sand and gravel deposits. However, the role played by suspended noncolloidal particles in plugging off voids and limiting the depth of slurry penetration is not clearly understood. Generally, a Marsh funnel viscosity (time required for 946 milliliters [mL] of slurry to drain from a standard Marsh funnel (American Society for Testing Materials [ASTM] D6910/ASTM D6910M-09) is between 40 and 50 seconds for a 5-percent bentonite concentration slurry suspension prior to introduction into the trench. Figure 16.4.1.1-1 shows a Marsh funnel test being performed on fresh bentonite slurry. This value of 40 to 50 seconds is an acceptable level for most applications. The designer should avoid specifying Marsh funnel viscosities for slurry in the trench, as the presence of suspended solids from the excavation will significantly affect measured values.

Figure 16.4.1.1-1. Marsh funnel viscosity test being performed on fresh bentonite slurry.

C. **Gel Strength.** The slurry suspension should have sufficient gel strength (minimum shear stress required to produce flow) to maintain a sufficient amount of noncolloidal solids in suspension for the required slurry density. Theoretically, the maximum particle size that can be maintained in suspension is directly related to gel strength of the slurry suspension. However, the actual amount of noncolloidal solids in suspension is greatly influenced by the amount of agitation provided by excavation operations at the time. Bentonite slurries exhibit thixotropic properties. Thixotropic fluids exhibit viscous properties under static conditions and become fluid, or undergo reduced viscosity, when agitated or undergoing shear. With bentonite clays, this property is enhanced with the interactive particle forces that create a flocculated system of particle association. This means that gel strength is not constant but increases with time under static conditions. The most common measure of gel strength in the field has been the 10-minute gel strength. A 5-percent bentonite concentration slurry suspension generally produces a 10-minute gel strength of 10 to 15 pounds per 100 square feet (ft^2). This is generally sufficient to produce a slurry suspension with a density of 70 to 75 lb/ft^3 with normal excavation techniques. Actual measurement of 10-minute gel strength in the field is severely limited by the presence of the noncolloidal materials in suspension. The designer should avoid specifying gel strength for slurry in the trench.

Specification of initial gel strengths, above those obtained with the typical 5-percent bentonite concentration slurry, generally leads to densities higher than desired. This, in turn, requires extensive desanding operations to lower densities and results in the increase of additives to the slurry that may have a detrimental impact on long-term performance of the trench. If excavation operations are suspended for a long period of time, mechanical agitation is the preferred method of maintaining required densities.

D. **Filtrate Loss.** As discussed in the subparagraph on viscosity, formation of the relatively impermeable filter cake is the primary means by which full hydrostatic force of the slurry is applied to the sides of the trench. In addition, the filter cake is one of the components that contribute to the long-term low permeability of the cutoff wall. The filter cake should be thin enough that it is not significantly damaged by excavation equipment and still provides the necessary flow restriction to ensure full hydrostatic force on the face of the trench. The current test used in practice to measure formation characteristics of the filter cake is the filtrate loss test (measurement of slurry losses through a filter paper in 30 minutes at a constant applied pressure (American Petroleum Institute [API] Specification 13A, using ASTM D5891). As with the majority of tests, this test is significantly impacted by the presence of suspended noncolloidal solids and should not be used as a control for slurry in the trench. The test can be performed in the laboratory or in the field. The designer should use this test only as a measure or index of filtrate properties of the fresh slurry prior to introduction into the trench. The typical 5-percent bentonite concentration slurry suspension will generally have less than 20 cubic centimeters (cm^3) slurry loss per 30 minutes under 100 pounds per square inch (lb/in^2) of pressure. A typical allowable is between 15 cm^3 and 25 cm^3 to ensure good filter cake development. Actual filter cake formation on the sides of the trench depends largely on the nature of suspended solids in the slurry and the gradation of the native material on the sidewalls of the trench. However, a typical bentonite concentration of approximately 5 percent will generally provide adequate filter cake formation.

E. **Water for Bentonite Slurry.** A large volume of water is typically required to complete a SB cutoff wall. Water is used in both the slurry preparation and the backfill. Contaminants in the water, both natural or due to the presence of wastes in the ground, can inhibit full bentonite hydration or cause flocculation, leading to weaker filter cake formation and lower viscosity. Potential sources of water should be identified for use in the bentonite slurry and identified in the specifications for use by the contractor. If possible, multiple sources should be identified that can supply the required volume. Each source should be tested for standard water quality indicators, and the test data sheets should be provided in the specifications. This will allow the contractor to determine any water treatment requirements necessary to maximize the properties of both the slurry and the SB backfill. In general terms, the water should be of good quality, contain a minimum amount of impurities, and be free from oil, acid, alkali, organic matter, or other deleterious substances. The use of hard water will require more bentonite and longer mixing times. Salt water or brackish water should also be avoided. Sodium chloride in excess of 500 parts per million (ppm) or calcium salt in excess of 100 ppm is likely to reduce the swelling ability of bentonite and produce a low viscosity gel. The full extent of testing required is beyond the scope of this chapter but should include, as a minimum, the following property tests: (1) pH (test method: API RP 13B-1); (2) hardness (test method: API RP 13B-1); (3) total dissolved solids (test method: U.S. Environmental Protection Agency [EPA] 600/4-79/020); and (4) other tests which may be required if contaminants are suspected. In addition, bidders should be encouraged to perform their own testing on potential water sources for use in the preparation of their bids. Often, the addition of soda ash to the mixing water will be used to alter the pH of the water and create the conditions for full hydration of the bentonite, especially in the presence of water containing significant amounts of calcium and/or magnesium.

16.4.2 Trench Stability During Construction

The designer should ensure that static stability of the open trench is adequate for the range of loading conditions that may reasonably be expected at the site. The combinations of loading conditions to be considered should include equipment surcharge, groundwater levels, expected slurry levels, and slurry densities. If spoil material from the excavated trench is to be placed next to the trench, the designer should also ensure that the surcharge loading from the excavated material is accounted for in the trench stability calculations. The following discussion of trench stability is applicable to long, continuous trenches. For slurry-supported panel construction, the stability can be benefitted by three-dimensional effects and arching that should be considered.

16.4.2.1 Slurry Trench Stability Analysis

The subject of trench stability is very broad, and only general guidelines are provided in this section. The reader should refer to the numerous cited references for details on slurry trench stability methodologies and assumptions [4, 5]. Generally, the slurry level should be maintained at a maximum of 2 feet below the top of the trench and a minimum of 3 feet above natural groundwater elevation. The stability of the trench may be analyzed by any method that correctly applies the hydrostatic force of the slurry to the sides of the trench. A simple wedge analysis is generally adequate. In most pervious foundations where slurry trenches are required, the use of drained strengths in the stability analysis is appropriate. If substantial deposits of silty sands or soft clays are encountered, use of undrained strength parameters should be considered. The designer should avoid specifying an increase in slurry density to provide the required open trench stability. In practical terms, the use of higher densities makes it difficult to maintain densities in the lower portions of the trench below the maximum value without continuous agitation. Where the calculated stability is shown to be inadequate, a common solution is for the designer to specify the use of berms to elevate the working surface to provide additional hydrostatic force resulting from a higher slurry level.

A key component of stability calculations of a continuous trench is the estimate of the level of expected slurry penetration into the sidewalls of the trench and the expected mechanism that will form the seal across the trench wall face. This will influence the magnitude of the resisting force applied by the slurry. In cases of slurry trench excavation in composite finer soils such as clays, silts, or sands which include gravel, the hydrated bentonite slurry suspension will bleed into the sidewalls, gradually building up the thin, densely packed membrane of colloidal particles referred to as a "filter cake." The impermeable property of the filter cake will allow the full hydrostatic force of the slurry suspension in the trench to develop on the sidewalls and resist the active sliding wedge. The filter cake is also a primary contributor to the overall low permeability of the trench. The general depths of bentonite slurry penetration into sandy soils can range from 1 to 3 inches, 3 to 6 inches in sand and gravel mixtures, and up to 1 foot in gravels, depending on the gradation. In each of these cases, a filter cake develops at the trench face, and further penetration is halted.

In cases where the excavated soils are coarser, such as coarse gravels and cobbles (possibly with isolated boulders), trench stability calculations become more critical, especially if the water table is high. In these soils, the slurry is likely to penetrate much further into the trench sidewalls and may not actually develop a traditional filter cake as in the finer soils. In these cases, the slurry flows directly into the porous formation until it is restrained by its own shear strength. This is often called rheological blocking. Slurry penetration into coarse formations in this manner can be many feet. This also results in a greater volume of slurry needed to produce a stable trench and the possibility of sudden or rapid slurry loss. The process is characterized by the gradual gelation of the slurry in the zone of penetration. The result of the penetration can produce a reduction in the resisting force acting on the sidewall of the trench compared to the hydrostatic force computed by using the full weight or density of the slurry in the trench and assuming that the wall of the trench is essentially an impermeable membrane. In practice, the reduction in the resisting thrust of the slurry may be difficult to compute. This is because the actual length of penetration of the slurry into the formation, at the bottom of the trench, must be known to compute the resisting force. A variable known as the stagnation gradient, i_o, (the quotient of the height of the slurry above the water table, h, divided by the length of penetration, l) is required to compute the resisting force [5].

16.4.2.2 Slurry Trench Width

In general, the location, layout, profile, and depth of cutoff are determined primarily by geology, project requirements, and local site conditions. However, the width of the SB cutoff wall must be established to provide adequate safety against blowout failure for the seepage gradient under the maximum differential head. Blowout is defined as a failure of the cutoff backfill material where the slurry and/or the fines are forced out of the backfill and into the formation due to the gradient across the trench (seepage forces exceed the slurry gel strength). This results in increased permeability and possible creation of windows in the cutoff wall. Due to many uncertainties involved in the determination of the blowout gradient, the recommended factor of safety against blowout should not be less than four to compensate for construction imperfections and other limitations. In addition, the designer should be aware that the actual trench width and, subsequently, the cutoff wall, will be effectively wider than designed due to overbreak caused by the excavation bucket. However, it is not recommended that this additional width be included in calculation of the blowout gradient. Where the differential head across the SB cutoff wall is not critical, the minimum trench width should be based on practical considerations such as the type of excavation equipment to be used and standard widths of the equipment. The use of nonstandard bucket widths may increase costs.

The hydraulic gradient required to overcome the gel strength of the bentonite slurry in the backfill and force the bentonite out of the backfill, as described above, is commonly measured by the blowout gradient test. This test, as described by D'Appolonia [4] and Xanthakos [5], consists of placing a hydraulic gradient across a 1-foot -thick specimen contained in a pressure device. In general, for cutoff backfill gradations typical of SB cutoff walls (minus 3-inch particle size with 10- to 30-percent fines [minus No. 200 sieve]), blowout failure appears to occur at gradients between 30 and 40. It has been suggested that, for preliminary estimates, a gradient of 32 may be used. Using an assumed factor of safety of four, as suggested, then implies that the approximate cutoff wall thickness can be found by dividing the differential head by eight. This method can be overly conservative and should be used with caution when the differential head across the trench is anticipated to be very high. Conventional SB cutoff walls have been constructed where the differential head across the wall has exceeded 40 to 50 feet (40 feet would imply that a 5-foot-wide cutoff is required, using the above criteria). In such cases, there may be justification to perform blowout testing on a proposed backfill to verify adequate performance at higher gradients. Another alternative would be to consider the use of stronger backfill materials such as CB. However, the designer should keep in mind that translation of laboratory results to field application is highly questionable. Although this test may give reasonable values of the gradient required to overcome gel strength of the slurry, the mechanism of failure for backfill in the field would be different. Actual confining pressures on backfill in the trench can be severely limited by the effects of arching. Experience [4] has shown that hydraulic fracturing of inplace backfill material is possible. This is a different failure mode than blowout of the slurry and fines into the surrounding formation material. The filter cake may help in preventing hydraulic fracture; however, the role of the filter cake in this regard is not well understood. Given the uncertainties associated with the potential for hydraulic fracturing and blowout, it is suggested that a conservative approach be taken by the designer in designing the backfill mix (see Section 16.4.5, "Backfill Mix Design," below). Typically, the width of the trench will be determined by the estimated maximum differential hydraulic head across the trench and, to a lesser extent, excavation equipment requirements. Uncertainties associated with performance of the backfill preclude a more refined calculation of required width. Because there are numerous cases of SB cutoff walls performing well that may not satisfy the above factor of safety criteria, it is the designer's responsibility to consider other factors such as backfill gradation, as compared to the formation material, and case histories when designing the trench width.

16.4.2.3 Additional Stability Considerations

In all design cases, the designer should pay careful attention to the local geology and ensure that sufficient investigations have been completed to fully understand the soil variability, structure, and expected changes in the groundwater depth over the time period of construction because these factors can have a significant effect on the calculated trench stability. Cutoff walls are often placed within permeable alluvial deposits, which are typically heterogeneous and may contain horizontally bedded, higher-permeability lenses or horizontally continuous weak layers. As an example, a coarse, clean gravel lens within a finer alluvial deposit could exert higher water pressures at the trench wall and/or allow for higher slurry penetration into the lens itself. The designer should be aware of this variability and consider the possibility of a more localized, partial trench failure when evaluating trench stability. Similarly, if a lens of unconsolidated, weak, or high plasticity saturated clay is present within the trench wall, the change in lateral stress during excavation could result in increases in pore pressures within the clay that can reduce the available shear strength due to undrained loading. In this case, the use of an undrained strength analysis may be appropriate along with in situ or laboratory testing of the clay.

Other factors the designer should consider to achieve overall trench stability are construction sequencing, slurry quality control, excavation method, and the potential for rapid slurry loss due to hydraulic fracturing or the presence of preexisting cracks. Construction sequencing would include restrictions on the length of open trench that can exist at any given time by controlling the rate of excavation and backfilling. This can effectively limit the distance between the toe of the trench backfill and the head of the excavation, minimizing the length of time that any portion of the trench wall is left open with slurry support only.

Slurry quality control is critical because introducing slurry into the trench that is not fully hydrated can result in reduced hydrostatic force on the trench wall. This can result from a number of quality control issues that will be discussed later in Section 16.4.7, "Construction Control." If trench stability is questionable or the soil variability cannot be fully characterized, the designer may want to consider panel construction, which not only limits the length of any section of open trench, but includes three-dimensional effects that may increase the overall stability of a given panel. However, panel construction can add significantly to the cost of a cutoff wall and will be discussed in more detail in Section 16.7, "Concrete Cutoff Walls."

Another key factor that can lead to trench stability issues is rapid slurry loss, which reduces the head of slurry on the trench wall faster than it can be replenished. Rapid slurry loss can result from inadequate hydration of the bentonite, hydraulic fracturing, or loss of slurry through very coarse lenses, preexisting cracks, or defects. The designer should always consider the possibility of low stress zones within the trench excavation that can lead to hydraulic fracture and rapid slurry loss. Low stress zones may exist at foundation irregularities where steep irregularities in the bedrock surface cause arching of the overlying soils or within embankments in the vicinity of structures where poor compaction of overlying and/or adjacent soils may create low stress zones. Other features that can contribute to slurry loss are rodent holes and other types of animal burrows that can often extend for long distances within an existing embankment or foundation. The presence of burrows can be determined through good visual examination of the area where trenching will take place during geotechnical investigations for design. Other potential locations for slurry loss include desiccation cracks or cracks caused by differential settlements, open jointing in bedrock, stress relief cracks, coarse gravel or cobble zones, and karst foundation conditions. Slurry loss is discussed in more detail in Section 16.7.9, "Slurry Losses in Panels or Trenches."

16.4.3 Test Sections

Whenever possible, and especially when questionable trench stability exists, a full-scale test section should be considered to evaluate the proposed slurry, excavation method, and trench stability. This is especially true when the trench is to be excavated through weak soils and/or an embankment where preexisting cracks may be present due to desiccation or differential foundation settlement. At the same time, slurry properties can be evaluated along with the proposed construction methods. The length of a test section will depend on numerous factors but should be of sufficient length to test the contractor's entire sequence of construction, including: (1) excavation equipment, (2) mixing equipment and methods, and (3) cutoff wall construction (backfill methods) in the site-specific formation materials.

16.4.4 Cutoff Wall Permeability

Permeability of an earth-backfill cutoff wall is a function of individual permeabilities of both filter cakes formed on the sides of the trench, and the gradation of the earth backfill placed in the trench. The relative contribution of individual permeabilities to overall permeability of the cutoff wall is largely a function of the relative thickness of the two constituents. Overall cutoff permeability is controlled by the backfill when the backfill permeability is low and by the filter cakes when backfill permeability is high. D'Appolonia [4] has suggested that the upper limit of overall expected permeability of an earth-filled slurry trench cutoff wall of typical width is about 1×10^{-6} centimeters per second (cm/s), even for very pervious backfill, due to the low permeability of the filter cake. On the low side, permeability values as low as 1×10^{-8} cm/s (and even lower) have been reported [5, 6]. For the range of permeabilities cited here, there must be sufficient fines within the backfill to fill the pore space of the larger particles. A well-graded backfill is more likely to meet this requirement; however, the designer must consider internal stability of the backfill if it is too broadly graded.

16.4.5 Backfill Mix Design

Permeability of an earth-backfill material depends on both soil gradation and quantity of bentonite used in blending. Typically, permeability of the backfill is reduced an order of magnitude for each 1-percent increase in bentonite concentration by dry weight of backfill. Standard practice is to blend the soil used for the backfill with slurry (which generally contains 5-percent bentonite by dry weight). If the soil is initially dry, the bentonite content of the backfill will be a maximum of about 2 percent, by dry weight, of the mixture when sufficient slurry to obtain a 2- to 6-inch (50- to 150-millimeter [mm]) slump is added (slump test based on ASTM C143/C143M-12) [7]. A 2- to 6-inch slump will generally form a slope between 10:1 horizontal:vertical (H:V) and 15:1 under typical placement conditions and is in the range generally accepted by the industry. Water content has a direct relationship to the slump of the backfill. A moisture content of 20 to 30 percent will generally result in a slump within the 2- to 6-inch range. Soil material used for backfill that is excavated from below the water table may typically retain 10- to 20-percent water content, even with drainage. Therefore, the resulting bentonite content with sufficient slurry added to obtain the required slump may be only 0.5 to 1.5 percent.

To reduce the potential for internal erosion associated with any potential hydraulic fracturing, the designer should apply the filter criteria proposed by Sherard et al. and Reclamation [20, 21, 22] between trench backfill and the coarsest in situ materials. These criteria were based on prevention of internal erosion in cracked embankment core and are directly applicable to design of a cutoff wall backfill. A backfill gradation containing 1-percent bentonite and 20-percent fines (minus No. 200 sieve material) in a well-graded sand/gravel mixture will generally be suitable for most applications. The maximum size of gravel particles should be limited to 1 to 1.5 inches (25 to 38 mm) to minimize segregation during placement. However, in many cases, larger maximum sizes have been used with successful results as shown in table 16.4.5-1. As a general guide, the following gradation ranges that have been used in Europe, Australia, and the United States can be a guide to the designer when determining the appropriate backfill.

Table 16.4.5-1. Typical Gradation Limits for Soil-Bentonite Backfills (Europe, Australia, United States)

Screen size	Europe and Australia Percent passing by weight	United States Percent passing by weight
3 inches	80 - 100	80 - 100
3/4 inch	40 - 100	50 - 100
No. 4	30 - 70	30 - 70
No. 30	20 - 50	25 - 60
No. 200	10 - 25	10 - 30

The designer should understand that these typical gradations are only general guides. Each case will be different, and filter compatibility of the trench backfill and in situ material should be satisfied, especially if high gradients are anticipated across the cutoff wall. In some cases where a cutoff wall is excavated through very coarse material, the excavated material mixed with bentonite slurry will not be an acceptable backfill because it may be too coarse or internally unstable. In some instances, a borrow source or commercial source of soil material can provide an acceptable backfill material that will produce a lower permeability and meet filter criteria for the surrounding in situ material. This was the case at Reclamation’s Diamond Creek Dike [23], Keechelus Dam [39], and Bradbury Dam [24]. The excavated material mixed with bentonite slurry was too coarse and gap-graded to serve as adequate backfill. Therefore, a more compatible backfill was imported, mixed with fresh bentonite slurry, and placed into the trench. If the risk of filter incompatibility leading to loss of backfill is high and/or blowout is a concern, due to high seepage gradients, the designer may want to consider the use of a different type of cutoff wall backfill.

16.4.6 Embankment Core/Earth Backfilled Trench Connection

The connection of the embankment core with the top of the cutoff wall requires particular attention if the integrity of the system is to be maintained. In general, arching effects will severely limit the amount of backfill consolidation that occurs under its own weight. Measured settlements of an inch to less than a few inches are common. Increased settlement, due to embankment loading on the trench, will also be limited to the upper portion of backfill due to arching. However, some precautions are required to ensure that a separation or low stress zone between the embankment and the top of the cutoff wall does not develop. A transition zone with 2:1 (H:V) or flatter slopes may be placed over the top of the completed cutoff to facilitate transfer of stress to the cutoff wall.

The transition typically consists of the same material as the impervious core, but placed slightly wet of optimum to increase its moisture content, thereby providing softer strain characteristics prior to saturation by the reservoir. A general schematic of an embankment core/earth backfilled trench transition section is shown on figure 16.4.6-1. This schematic can vary significantly, and the designer should always consider site-specific requirements.

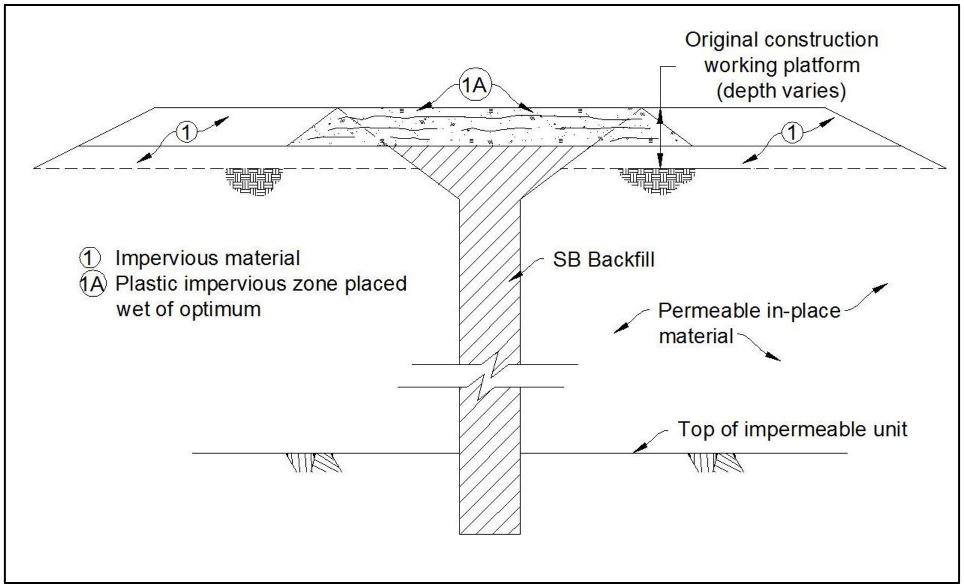

Figure 16.4.6-1. Typical earth backfill/embankment connection detail.

Another concern for the interface between an overlying embankment and cutoff wall is the possible presence of desiccated and/or cracked SB backfill in the upper section of the SB backfill. Invariably, the upper few inches to a couple of feet of the SB cutoff wall will be compromised and/or contaminated, no matter how much care is taken to protect the surface. This should be accounted for in the contract specifications for placing the overlying embankment materials. The specifications should require removal of at least the top foot of the constructed wall, prior to placement of the transition section described above, to ensure that the transition section is being placed over sound, uniform, uncracked or desiccated SB backfill. The removal of this unsuitable zone of material should not occur until the contractor is ready to place the transition material over the final surface of the SB backfill to minimize exposure to air drying. The final surface of the SB backfill should be thoroughly inspected and approved by the designer prior to placement of the transition section.

Any connection detail of a cutoff wall to an overlying embankment needs to be made with consideration given to the specific geometry, the type of backfill material in the cutoff wall, and the surrounding foundation.

16.4.7 Construction Control

The designer must be highly involved in selection and control of procedures employed by the contractor to construct the earth-backfill cutoff wall. In addition, the designer should have a clear understanding of what is, and what is not, important and avoid specifications that unduly restrict the contractor and draw attention away from the more critical aspects of construction. Most slurry trench contractors have a significant amount of construction experience in this specialized field.

A. **Slurry Mix**. As previously discussed in the paragraphs on mix design, physical properties of the slurry are very difficult to measure in the trench. In addition, correlation of specific values to a defined performance level during construction is not generally possible due to the large number of nonquantifiable factors involved. Experience has shown that a slurry with a 5- to 6-percent bentonite concentration (depending on the type of additives used) will generally have an initial Marsh funnel viscosity of around 40 seconds. A slurry with these properties will generally produce satisfactory results. The designer should generally limit specification requirements for slurry properties to the following:

   1. **Grade of Bentonite.** Naturally produced, premium grade, Wyoming-type, sodium-cation-based bentonite displaying high swelling characteristics. The bentonite should meet all of the standards set forth in the current American Petroleum Institute (API) specifications 13A, Section 9, "Specifications for Drilling-Fluid Materials," for bentonite and bear the API stamp of approval on the containers [25]. Bentonites are typically composed of montmorillonite clay minerals that have a high swelling capacity when exposed to water. In Reclamation, the bentonite is typically specified to be a naturally powdered, pure, premium grade, Wyoming-type, sodium cation-based bentonite with high swelling characteristics needed for slurry trench construction. However, this does not mean that the bentonite must come from Wyoming. Other sources outside of Wyoming can produce bentonite with the required properties and have been used in Reclamation work. In the past, Reclamation has typically specified that bentonite must be composed of at least 90-percent montmorillonite. Bentonite with this percentage may be difficult to find due to the increased use of bentonite products worldwide.

A value of 85 percent has been used successfully (A.V. Watkins Dam [26]) and is generally acceptable. The designer should consider this when specifying the bentonite and include a required submittal, for approval, listing the bentonite supplier and bentonite characteristics as part of the contract specifications.

2. **Bentonite Concentration**. Manufacturers or suppliers of bentonite generally offer two or three grades of bentonite (all meeting API standards) requiring a higher or lower concentration of bentonite to produce the desired slurry properties. The various grades of bentonite are generally made by adding inorganic additives or extenders that boost the yield of the bentonite. Currently, all grades of bentonite that are used in slurry trench construction are extended. However, the designer may consider using bentonite grades with lower yields (less additives) to ensure satisfactory long-term performance of the cutoff wall. The designer should establish specific concentrations for each brand and grade of bentonite proposed by the contractor prior to construction. These concentrations should be based on those required to obtain a Marsh funnel viscosity of at least 40 seconds.

3. **Filtrate Loss.** This can be an important test to run on the prepared, hydrated slurry, prior to introducing the slurry into the trench. It can provide a quantitative measure of the ability of the slurry to form a filter cake and allow the full hydrostatic head of the slurry to act on the walls of the trench excavation. Following the test procedure outlined in ASTM D5891 [27], and using the recommended maximum filtrate volume measured from the test as shown in API Specification 13A, should provide greater confidence in the ability of the slurry to form the filter cake.

   Field control of slurry should concentrate on ensuring that the slurry mix is prepared with the specified bentonite concentration. The specification of a 40-second Marsh funnel viscosity for the slurry after hydration in a containment area, and prior to introduction into the trench, may be helpful as an indirect means of controlling the bentonite concentration. However, the main emphasis should be on ensuring that the specified weight (barrels, bags, etc.) of bentonite is physically added to the correct amount of water to achieve the specified concentration for the grade of bentonite used. The designer should limit testing in the trench to that required to ensure that maximum density limits are not exceeded. The same density testing could be used to check minimum density limits in the trench if they are specified.

B. **Backfill Mix.** Construction control of the backfill mix should concentrate on ensuring that the mix has the specified gradation, consists of a homogeneous mass, and has the desired consistency required for placement and bentonite content. The two most applicable field tests are the gradation analysis and conventional concrete slump test. Backfill is typically mixed in batches using dozers and other equipment in flat areas adjacent to the trench. The designer should specify that at least one gradation analysis and one concrete slump test be performed on each batch of backfill prior to clearance for placement in the trench.

C. **General Construction Considerations.** Construction of slurry trenches generally requires construction of berms to obtain level working surfaces in order to retain the slurry in the trench. The elevations of these working surfaces can be selected to increase stability of the excavation by increasing the differential head between the slurry and natural groundwater. Staged berms or working surfaces can also be used to construct slurry trenches where there is a significant change in elevation across the site, as is typical in many river valleys. To minimize the potential for slurry loss through hydraulic fracture of an adjacent, previously completed section or panel, staged construction of the slurry trench should always be from higher to lower elevations. Delays in slurry trench construction can often have significant impact on the overall completion schedule of the dam. The designer should select both the elevation of working surfaces and the sequence of construction required to ensure that construction of the slurry trench will proceed as smoothly as possible. After backfilling of the trench is completed in a given location, the trench is generally covered with a loose superficial cover of soil materials to minimize surface drying and cracking of the top of the SB trench.

During excavation, the slurry within the trench can become too dense as a result of solids such as silt and sand becoming suspended in the slurry. This can be especially problematic near the bottom of the trench. The construction staff should regularly take slurry samples within the trench at varying depths and measure total density. If the slurry becomes too dense, it can prevent displacement by the SB backfill. The degree to which this occurs depends on a number of factors including gel strength of the slurry, method of excavation, and gradation of the surrounding soil. There are numerous types of slurry samplers that are commercially available for use in sampling at depth within the trench. If the slurry becomes too dense, the use of desanding equipment will likely be required to separate and remove suspended solids from the slurry.

In some slurry trench excavations, it is common to encounter nested boulders and/or difficult zones to excavate such as cemented gravels or hardpan layers. If it is anticipated that this may occur, the specifications paragraphs should clearly delineate this possibility. Typically, the contractor will provide a Kelly bar equipped with a chisel. Individual boulders can sometimes be picked out of the trench with a cable-suspended grab or clamshell bucket. If cemented materials or embedded boulders cannot be picked out, the chisel can be used to break the cemented zone, or boulder, into smaller pieces that can be lifted out. If the cutoff wall is to be keyed into a less permeable hard zone, such as bedrock, the excavator may not be able to excavate into this material unless it is weathered or soft, or contains teeth on the back of the bucket, which can rip the bedrock surface. In some cases, a chisel can be useful in achieving a key into rock. During construction of an SB cutoff wall at Reclamation's Keechelus Dam in Washington, a large boulder erratic, estimated to be 10-20 feet in diameter, was encountered. In this case, the trench was rerouted around the boulder, while excavation took place on either side of the boulder (but not beneath), in order to encapsulate the boulder and surround it with soil-bentonite.

Slurry trench excavation and cutoff wall construction operations can result in a messy work site. Areas of greatest concern are where bentonite is mixed and hydrated, as well as where excavation and backfilling occur in the trench. As backfilling proceeds along the trench alignment, bentonite slurry in the trench is displaced and must be handled properly. Confidence that slurry handling techniques will result in a clean work site can be increased with submittal requirements in the contract that require the contractor to describe in detail the mixing, excavation, and backfill placement operations.

D. Verification of Trench Depth. Verification of trench depth is usually performed by dropping a weighted measuring tape to the bottom of the trench. Verifying continuity can be achieved by passing the clamshell or excavator bucket vertically and horizontally within the section of trench to be backfilled. Often, when the cutoff wall is keyed into an aquiclude, visual observation of the excavation and cuttings will confirm penetration into the aquiclude. Confirmation that the full depth of penetration into the aquiclude has been achieved can then be verified with a weighted tape measure. Additional confidence that the trench is fully embedded into the aquiclude can be enhanced with sufficient drilling along the alignment of the trench to identify the bedrock profile.

Figure 16.4.7-1 Large embedded boulders removed from SB cutoff wall at Keechelus Dam, Washington.

Figure 16.4.7-2 SB backfill being mixed with bentonite slurry adjacent to the trench at Keechelus Dam, Washington.

16.4.8 Measurement for Payment

Measurement for payment of earth-backfill slurry trench cutoff walls is generally made for the finished area in square feet, on a vertical plane through the centerline, from top to bottom of the cutoff wall. The unit price per square foot should include all costs of plant, labor, equipment, and materials to excavate and construct the cutoff wall. Consideration should be given to providing a separate cost item for construction of berms required to establish level working surfaces if those surfaces can be well defined during design. In some cases of deeper cutoff walls, different methods of excavation may be required for the shallow and deeper sections (i.e., backhoe for upper section and clamshell for deeper section). In this case, separate bid items for each method of excavation may be appropriate. Also, if an offsite commercial source or borrow area is to be used to supply the backfill, a separate line item should be used.

16.4.9 Submittal Requirements

The requirements for submittals are likely to vary by job and are primarily the responsibility of the designer and the field construction staff. The designer should work closely with the construction field staff during the preparation of the specification contract documents to develop this list. The following is a suggested list of required submittals for approval when constructing a soil backfilled cutoff wall:

- Qualifications:
    - Onsite supervisor resume
- Soil-Bentonite Cutoff Wall Plan:
    - Bar chart construction sequence drawing showing dates of anticipated cutoff wall construction and completion
    - Anticipated production rates in square feet per day
    - Methods and frequency of monitoring trench alignment, depth, and verticality
    - Description of proposed method of trench excavation, including equipment and sequence of constructing cutoff wall
    - Method for desanding, including description of equipment
    - Method of mixing SB and method of placement

- Method of stopping rapid slurry loss during trench excavation
- Bentonite-water mixing method, equipment, storage area size and locations, and hydration period
- Source of water used for construction
- Pumping equipment, capacity, and backup equipment
- Type of additives to be used and conditions under which they will be used
- Source of backfill, gradation, and physical properties (if backfill is from a source other than the excavated trench)

- Method of wasting the following:
    - Bentonite slurry in pump lines
    - Slurry removed during trench bottom cleaning
    - Method of flushing pump lines and disposal of flushing water after use and conditions under which flushing would be done
- Method of sampling bottom of slurry trench.
- Method of excavating cemented soils, cobbles, and/or boulders, and description of equipment.
- Method of winter operation of slurry system, including protection of exposed slurry surfaces and prevention of freezing water in lines and pumps (if applicable).
- Haul traffic plan
- Material approval data:
    - Name and manufacturer of bentonite material and additives
    - Source of bentonite and approval certification that it meets minimum requirements in specifications
- Calibration certifications:
    - Calibration certifications for all equipment used for measuring and mixing bentonite slurry

This list is a suggested list of potential required submittals, and the designer should consider the site-specific requirements which may include submittals not listed here.

16.5 Cement-Bentonite Slurry Trench Cutoff Walls

16.5.1 General

A CB slurry trench cutoff wall is constructed by excavating a narrow vertical trench that is typically 2 to 3 feet wide. Unlike the earth-backfill slurry trench, the stabilizing fluid used during excavation is a CB-water mixture that remains in place after excavation to form a relative erosion-resistant and impervious barrier when solidified. The use of this methodology does not require a separate operation, and the necessary space, to mix the backfill going into the trench. The CB trench can be excavated in a continuous manner similarly to the earth-backfilled type of slurry trench for limited depths. However, many applications of this method make use of the alternate panel excavation technique. This technique consists of excavating a set of primary panels 10 to 20 feet in length to final grade in the presence of the CB slurry suspension. After the initial setup of the CB suspension in the primary panels, the secondary panel is excavated between (and several feet into) the primary panels. However, many CB cutoff walls have been excavated successfully using continuous trenching methods which are typically less costly. A 6-mile long CB cutoff wall was constructed at A.V. Watkins Dam in Utah using continuous trenching. The specifications required that for continuity of the trench, the new excavation each morning should be keyed a minimum of 5 feet into the previous day’s CB trench backfill for the full depth of the trench. Given the very low strength of the CB backfill after 1 day, this can be accomplished easily with a backhoe. Often, additives such as lignosulfonate, which serves as a plasticizer, will also act as a set retarder, which allows for easier excavation when keying into the previous day’s CB backfill.

Excavation of a CB cutoff wall will generate a significant amount of spoil material because the excavated material is not placed back into the trench. The spoil consists of excavated soil, bentonite, and cement, plus any minor additives. Because these materials are nonhazardous, the spoil material can be wasted (usually buried in a borrow pit) or used for other portions of the work. If possible, the designer should consider the use of the material. At A. V. Watkins Dam, the excavated material was placed directly onto the downstream face of the embankment, allowed to dry, and then formed into a flat berm to increase downstream stability. The berm was covered with topsoil and seeded.

Although CB cutoff walls are more commonly installed in existing embankments as a seepage remedial measure, they can also be incorporated into new dam embankment construction, which was done at Reclamation's Diamond Creek Dike in Wyoming. In this application, the CB cutoff wall was installed in 30-foot vertical segments. This was accomplished by building the new earthfill embankment in 30-foot segments, then excavating the CB cutoff wall in the newly placed embankment continuously from abutment to abutment. The top of the trench (wall) was flared to help ensure that the bottom of the next 30-foot vertical segment would have vertical continuity with the previous segment.

16.5.2 Cement-Bentonite Slurry Mix Design

The CB slurry should be composed of a bentonite slurry (Section 16.4.1, "Bentonite Slurry Mix Design") and either a Type I, II, or V Portland cement suitable to the local environment. Typically, the bentonite concentration used will vary from 3 to 6 percent. At the upper end of this range, the mixture becomes more difficult to pump, and the contractor may elect to use a plasticizer to allow pumping of the mixture for longer distances. The primary function of the bentonite slurry is to help maintain the cement in suspension until the initial set occurs. The cement should be added after the bentonite is fully hydrated, and simultaneous addition of bentonite and cement to water should be prohibited. For the bentonite slurry suggested in section 16.4.1, the cement-water ratio will be the controlling factor in determining the strength, deformability, and permeability of the backfill.

The most common mix for CB slurries is a three-bag mix (approximately 300 pounds per cubic yard [lb/yd^3]) of cement added to a typical bentonite slurry composed of 6-percent bentonite by weight of water (approximately 100 lb/yd^3). This mix is generally regarded as providing the desired combination of strength and deformability and equates to an approximate 18-percent cement mix (by weight of water). Generally, a higher cement-to-water ratio will increase the strength and stiffness of the wall. A lower bentonite concentration will typically result in a higher permeability and a cutoff wall of greater stiffness. Laboratory tests should always be performed to relate the cement-water ratio for a given bentonite slurry to the degree of stiffness, deformability, and permeability needed for a particular application. The design must also account for the significant volume of cement that is wasted as part of the excavation process. Since CB slurry provides trench stability during excavation, a large percentage of the cement and bentonite is wasted with the excavated spoil. Although the amount of waste that can result varies, the total volume of the cement and bentonite used to complete the construction may be as much as 40 percent higher than computed using the trench dimensions alone.

Pozzolanic materials are sometimes used as a replacement for cement. Blast furnace slag and fly ash are two of the materials that are sometimes substituted for up to 90 percent and 70 percent (respectively) of ordinary Portland cement in the slurry mix. When using these replacements, two important slurry properties are altered: (1) the slurry's set time is extended, allowing the slurry to remain fluid and workable longer; and (2) the viscosity, gel strength, and ability to form a filter cake (properties of the bentonite slurry) are each less altered than with a full cement mix. Blast furnace slag and fly ash do not damage the bentonite as much as the cement. Other effects of these additives include:

- Reduced bleed rates
- Maintenance of setting ability, even though agitated for long periods
- Lowered permeabilities in the completed wall
- Reduced susceptibility to chemical attack

The engineer must carefully consider the predominant properties being sought in a cutoff wall when considering the use of cement substitutes. In some cases, the use of a substitute can provide enhanced properties, while also reducing the overall cost.

The CB slurry begins to set within a few hours. Retarders can be used to delay the set, improve workability of the slurry, and allow the slurry to be pumped for longer distances. Increasing the distance that the CB slurry can be pumped reduces the number of plant setups required for longer trenches, which can reduce costs. In addition, a longer set time can be beneficial in other aspects such as increased penetration into heavily fractured strata or where voids may be present. This may enhance the long-term performance of the wall by increased filling of these features due to the longer time that the slurry is in a fluid state. The designer should investigate any potentially adverse effects of the use of any particular retarder on the long-term performance of the cutoff wall before approving its use. Lignosulfonate is a common additive used to increase fluidity and set time. Lignosulfonate is a byproduct from the production of wood pulp using sulfite piping, and including it in the mix allows for a reduction in water in the mix, while maintaining the flow characteristics.

16.5.3 Permeability of Cement-Bentonite

For a typical mix of cement and bentonite (i.e., 5 to 6 percent, by weight, bentonite and 18-percent cement, an in situ permeability of 10^{-5} to 10^{-6} cm/s), can be expected in the finished wall. CB cutoff walls that contain 2.5- to 7.5 percent blast furnace slag have exhibited permeabilities of approximately one order of magnitude lower.

16.5.4 Strength of Cement-Bentonite

CB cutoff walls are weak compared to other cement mixtures. For a typical mix described above, the cutoff wall could reasonably be expected to have a 28-day unconfined compressive strength (UCS) of 10-20 lb/in^2. Although strengths can be significantly higher with variations in the mix design and the use of additives, ultimately, the unconfined compressive strength is likely to be less than 50 lb/in^2. This gives the CB cutoff wall the added property of flexibility and greater resistance to cracking when undergoing strains due to embankment stresses. Testing has indicated that typical CB cutoff walls are capable of undergoing strains of several percent without cracking. It also gives the CB cutoff wall greater compatibility with the surrounding soil in terms of strength, stiffness, and modulus of elasticity. Too much stiffness and the cutoff wall may be more susceptible to cracking due to smaller strains within the surrounding soils.

16.5.5 Construction of Cement-Bentonite Cutoff Walls

Construction of CB cutoff walls can be performed either in panels or with continuous trenching methods. Both methods have been used successfully, and each method has benefits. In general, the continuous trenching method will be much less expensive. The designer must consider the total depth of the wall, stability, possibility of slurry loss, and set times for the mixture when either evaluating a contractor's proposed method of excavation or when selecting the construction method. In general, for continuous trenching and a typical mix, as described herein, the strength of the CB backfill will be low enough that excavation by backhoe can be easily accomplished within a couple of days. As described, the use of set retarders can extend this time. CB mixtures containing high cement-water ratios may be more difficult to excavate in continuous excavation CB cutoff wall construction.

For optimum CB properties, the bentonite should be fully hydrated prior to introduction of the cement. Cement should be added only to the fully hydrated bentonite slurry, as quickly as possible, with the use of a high-speed colloidal shear mixer. A high-speed shear mixer (>1,400 revolutions per minute [rpm]) will generally result in a wall with lower permeability and higher unconfined compressive strength. Figures 16.5.5-1 and 16.5.5-2 show construction of a CB cutoff wall at Reclamation's A.V. Watkins Dam near Ogden, Utah.

The primary concern when constructing CB cutoff walls is good continuity of the wall. The panel construction method can be applied with or without the use of stop-end tubes. Stop-end tubes are generally made from steel pipe with a diameter that is equivalent to the width of the trench and has a length equivalent to the depth of the excavated trench. The tubes are inserted vertically at the end of a panel, or at the end of a daily shift, and serve as an end point for backfill. Once an initial set of the backfill has occurred, the stop-end tubes are pulled out, usually by hydraulic jacking. This leaves a clean and sound vertical surface that facilitates continuity with the adjacent new panel. Whether excavation is accomplished using stop-end tubes in panel construction or the continuous trenching method, it is important that each new panel be keyed into the adjacent panels (panel construction) or the previous day's construction (continuous trenching). This can be accomplished when using continuous trenching by ensuring removal of a portion of the previous day's construction. With continuous trenching, it is important that a portion of the previous day's backfill is removed for the full height of the cutoff wall. Typically, in Reclamation, 5 feet of the previous day's panel is removed.

Figure 16.5.5-1. Continuous excavation of CB cutoff wall (A.V. Watkins Dam, Utah).

Figure 16.5.5-2. CB mixing plant (A.V. Watkins Dam, Utah).

During construction of a CB cutoff wall, rapid slurry loss is a key concern for trench stability. This same concern exists for any slurry supported trench or excavation. This is covered in more detail in Section 16.7.9, “Slurry Losses in Panels or Trenches.”

CB cutoff walls will crack near the surface (figures 16.5.5-3 and 16.5.5-4) due to drying and shrinkage after initial set. These cracks are usually a few inches deep. The designer must ensure that the cracked zone is removed down to uncracked, homogeneous CB prior to placement of any overlying embankment and provide the proper cutoff wall/embankment connection at this interface. If the top elevation of the cracked zone is above the top of the reservoir elevation, as was the case at A.V. Watkins Dam, the zone of cracking should still be removed in order to provide an acceptable surface for any overlying embankment

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Figure 16.5.5.-3. CB cutoff wall after initial set (showing cracking) (A.V. Watkins Dam, Utah).

Figure 16.5.5-4. CB cutoff wall after initial set (A.V. Watkins Dam, Utah).

16.6 Soil-Cement-Bentonite Cutoff Walls

The SCB cutoff wall is a special variation of both the SB and CB cutoff walls and has been used with increasing frequency in more recent years. This type of wall incorporates elements of the construction and placement methods of both cutoff walls.

An SCB cutoff wall is specifically designed with a controlled amount of soil, bentonite, and cement content to create different properties than a standard SB or CB cutoff wall. In general, the use of SCB cutoff walls is restricted to applications where a higher strength is desired, compared to standard SB or CB. Also, the permeability of SCB material will usually be slightly higher than SB backfill because the Portland cement prevents the bentonite from achieving its full swelling potential. The advantage of the SCB cutoff wall over the CB cutoff wall is that the cost will generally be less due to the lower volume of cement used in the mix and the fact that cement is not wasted with the excavated spoil. It can also have the advantage of the SB cutoff wall in that the excavated soil can be used as the backfill.

Construction methodology used for the SCB cutoff wall is more similar to that of the SB cutoff wall than the CB cutoff wall; however, the final properties may be more like the CB cutoff wall. The excavation of the trench is accomplished using a standard bentonite slurry support. The excavated soil, if acceptable for use as backfill, is typically transported to a mixing area. The acceptability of the excavated soil as backfill is judged using the same considerations as for the SB cutoff wall backfill in terms of general gradation (table 16.4.5-1) and compatibility with the surrounding soil. In addition, an excavated soil containing pockets of organic materials or zones of plastic clay may not be suitable because it would not produce a homogeneous mix. The mixing area is often within a contained area, or large box, in order to maintain good control over the mix proportions of the soil, cement, and bentonite and to achieve a homogeneous mix. In this manner, a known volume of concentrated bentonite slurry and cement grout can be added to create the backfill. In some cases, the bentonite slurry may be added to the mixture as a concentrated bentonite slurry (> 5-10%) with the addition of a plasticizer, such as lignosulfonate, to reduce viscosity. Although the cement is generally introduced to the backfill in grout form (water and cement), it can be added as dry cement powder. Backfill soil, either from slurry trench excavation or borrow, is first added as a known volume to the mixing area. Hydrated bentonite slurry or concentrated bentonite slurry and cement are then added, usually through a flowmeter, to provide a means of controlling the proportions of each material. The proportions of soil, cement, and bentonite are predetermined from mix design testing in the laboratory. Mixing is typically accomplished with an excavator, or dozers, until a homogeneous mix is developed that is free of unmixed soil inclusions and has a slump of approximately 6 inches. The presence of unmixed soil can be one of the biggest concerns when constructing an SCB cutoff wall. If the soil contains fines with some plasticity or clay seams, it is more likely to contain inclusions of unmixed soil. This could create problems with the finished wall that vary from higher permeability windows to weaker zones that could be potentially more erodible than other portions of the wall.

Placement of the mixed backfill into the excavated trench is usually accomplished in the same manner as a SB backfill using an excavator or dozer to place the material at the head of the trench backfill and allow it to displace the slurry as it is advanced into and along the trench, generally at a slope of between 5:1 and 8:1 (horizontal:vertical) can be achieved. This can usually be accomplished with a typical minimum 6- to 7-inch slump in the mixture before backfilling. The backfill is kept a minimum of 50 feet from the toe of the excavation proceeding ahead of it. This minimizes the amount of trench open at any given time.

An SCB cutoff wall constructed at Reclamation's Twin Buttes Dam near San Angelo, Texas, was constructed using a guidewall, primary/secondary panel wall construction, and a combination of rockmill and clamshell excavation equipment (see Section 16.7, "Concrete Cutoff Walls)." This application of an SCB cutoff wall was the first and only cutoff wall of this type constructed by Reclamation to date. In this case, the SCB backfill mix was tremied into each panel, similarly to the process used in constructing a concrete cutoff wall. It was necessary to use a rockmill for portions of the excavation, due to the presence of caliche and caliche well-cemented sands and gravels in the foundation with unconfined compressive strengths up to 15,000 lb/in^2. In addition, concerns existed regarding trench stability due to large slurry losses in the foundation materials which, in some cases, had permeabilities of 5 x10^5 feet per year. The clamshell was used in portions of the trench where cemented soils were not present. A traditional backhoe would have been unable to complete the excavation in the caliche-cemented materials, and the maximum cutoff wall depth of 100 feet was beyond the capability of even long reach backhoes. Figure 16.6-1 shows the rockmill used at Twin Buttes Dam, and figure 16.6-2 shows the rockmill, clamshell, and guidewall used as a template in the excavation of the cutoff wall. Figure 16.6-3 shows the tremie placement of the SCB.

The permeability and strength of an SCB cutoff wall can vary considerably, depending on the volumes and ratios of the various constituents. Two examples are discussed below:

(1) In the case of a SCB wall in California [8], a permeability of approximately 5 by 10^{-7} cm/s, with a minimum unconfined compressive strength of 15 lb/in^2 at 28 days was achieved with:

- A silty sand mixture containing 40 to 50 percent material finer than a No. 200 sieve

- A ratio of water to cement (W/C) of approximately 0.6

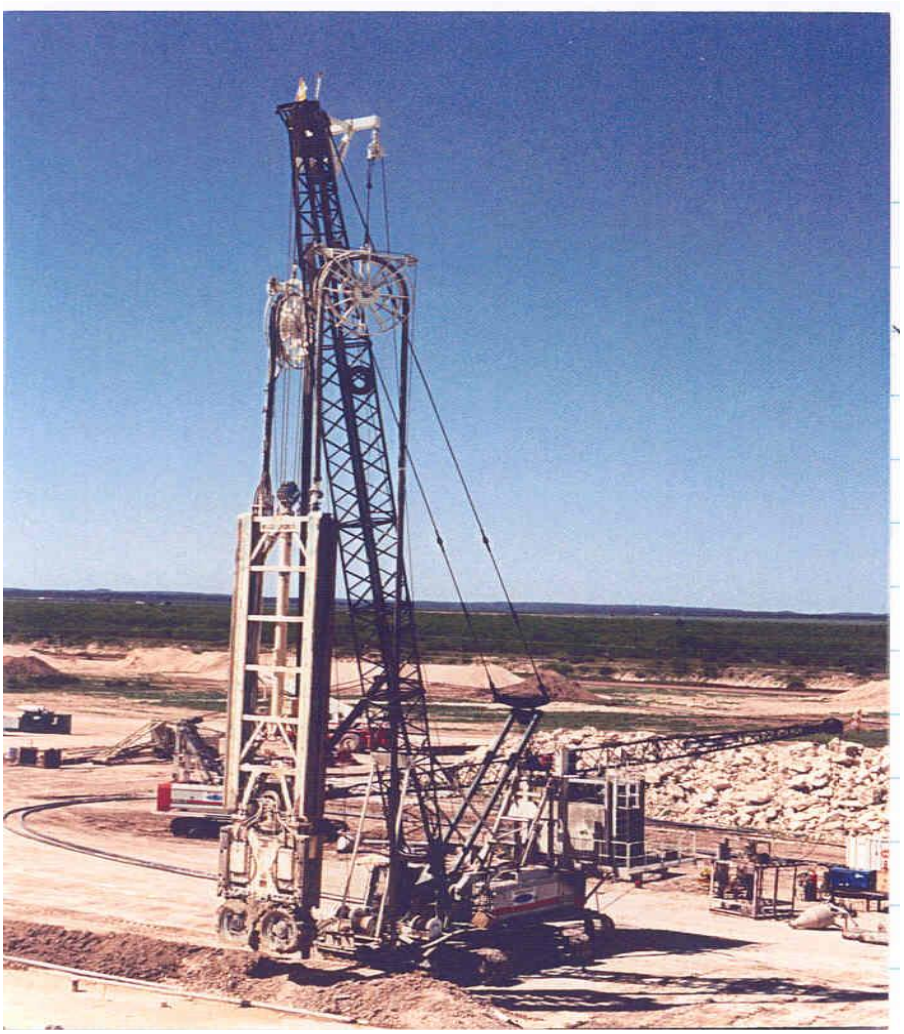

Figure 16.6-1. Rockmill used for the SCB cutoff wall excavation (Twin Buttes Dam, Texas).

(2) In the case of a SCB cutoff wall constructed at Reclamation's Twin Buttes Dam near San Angelo, Texas [13], the target mix in situ properties were 100 lb/in^2 (UCS) at 28 days and a permeability of 1×10^{-6} cm/s at 28 days. The final mix design consisted of 180 pounds of cement, 51 pounds of flyash, 2,542 pounds of soil (dry weight), 50 gallons of bentonite slurry, and 31.2 gallons of reservoir water. The mix resulted in 9 percent cementitious materials, a water/cementitious material ratio (W/C) of 0.34, and 0.9 percent bentonite. The allowable soil gradation for the mixture ranged from a silty gravel with sand (GM)s to a silty sand with gravel (SM)g and a maximum particle size of 1-1/2 inches. The strength was critical in this case, with potentially high gradients across the wall of up to 48 (120 ft per 2.5 ft).

Figure 16.6-2. Rockmill, clamshell, excavating the SCB cutoff wall (Twin Buttes Dam, Texas).

Figure 16.2-3. Guidewall and tremie backfill placement of SCB into the trench (Twin Buttes Dam, Texas).

The average 28-day UCS for the mix proved to be 158 lb/in2, and average 90-day strength was 191 lb/in2. Due to flowability issues, the mix design was altered during construction to add an additional 94 pounds of cementitious materials (73 pounds of cement and 21 pounds of flyash). Various other modifications were made to the coarse/fine ratio of soil in the mix and to the amount of water. The 28-day UCS of the various mixes ranged from 85 to 195 lb/in2, with laboratory permeability ranging from approximately 1 x 10-7 to 2.6 x 10-8 cm/s [40].

These are just two examples of a mix design. It is important that a comprehensive laboratory testing program be completed to evaluate various mix designs to determine the most efficient ratios of soil, cement, and bentonite to achieve the targeted design strength and permeability parameters required by the designer. This testing program will also provide data on the workability of a given mix design that is important during construction. For most SCB mixes, the designer can expect the strength of the SCB backfill to increase over time. However, the permeability of the SCB mixture is unlikely to change significantly with time. Another factor to consider in the design strength of the mix is the in situ stresses (especially shearing stresses) that the wall may be subjected to. In the case of Twin Buttes Dam, the cutoff wall was constructed at the toe of the dam, which will generally produce higher shear stresses on the wall due to the presence of the embankment. These loads may control the target design strength.

16.6.1 Submittal Requirements for Cement-Bentonite and Soil-Cement-Bentonite Cutoff Walls

When contracting for a CB or SCB cutoff wall, the engineer should require submittals for approval, before work begins, that indicate the contractor has a sound approach to mixing the backfill constituents and placing them within the trench. A requirement should also be included to show that experienced persons will be in charge of the work onsite. The suggested submittal requirements in Section 16.4.9, “Submittal Requirements,” for SB typically apply to CB and SCB cutoff wall construction, with the obvious adjustments for the type of planned construction. Additional submittal requirements that may be considered by the designer are listed below. Other submittals may be necessary or desired and should be determined based on job-specific needs.

- Name and manufacturer of cementitious material
- Cementitious materials certifications and test reports
- Set retarders
- Proposed use of cement replacements
- Methods for wasting CB or SCB (and cleaning pump lines (CB))
- Methods to flush lines and disposal of flushing water (CB)
- Proposed plan to mix SCB backfill material

- Method of backfill placement into the trench
- Onsite supervisor

16.7 Concrete Cutoff Walls

Concrete cutoff walls are sometimes used to provide positive seepage cutoff through embankments and/or pervious foundations of rock or soil. The bottom of the concrete cutoff wall should be extended into an impervious stratum. Concrete cutoff walls are almost always formed by constructing cast-in-place or, less likely, precast concrete segments in a series of slurry trenches. The trenches are generally of limited length (known as panels), noncontinuous, and constructed in an alternating sequence. Continuous trenching construction is almost never used for concrete cutoff walls, due to the difficulty of keying into a previous segment of wall with standard trenching equipment. Cutoff wall contractors often use specially designed equipment (described in the next section) that is capable of cutting into hard rock at the bottom of a panel or into an adjacent concrete panel to provide a key. These types of equipment are also capable of depths of excavation far exceeding that of even modified backhoes. With the use of panel construction and specialty equipment, cutoff walls exceeding 400 feet have been successfully constructed. The slurry provides stability for the trench during excavation and prior to concrete placement. The tremie method is used to place concrete in the trench from bottom to top, displacing the slurry. Methods to handle the displaced slurry are then required. The workability or flowability of the concrete is the most important property in constructing a high-quality, cast-in-place concrete cutoff wall. If analysis indicates high stresses within the wall, reinforcing steel may be necessary. Generally, for rehabilitation of existing dams, inclusion of reinforcing steel has not been necessary. If needed, bentonite can be added to the mix to create a more flexible concrete wall, commonly referred to as plastic concrete.

16.7.1 Applications

Concrete cutoff walls have a broad range of applications. In general, construction of concrete cutoff walls will be more expensive than SB, CB, or SCB cutoff walls. Most of the increased cost is due to the necessity to build this type of wall using panel construction methods and the increased cost of cement, due to the higher cement content. In general, trenches for these walls can be excavated using backhoes, draglines, clamshells, and cable-suspended hydrocutters (sometimes referred to as rock mills or hydromills). However, since concrete cutoff walls are generally installed using panel construction techniques that require vertical cuts, draglines and backhoes have limited applications for these walls. In most cases, either a clamshell or a hydrocutter is used because they are capable of going to greater depths and can be maintained under tight tolerances when verticality is critical. Concrete cutoff walls can be used in embankments, embankment foundations, and in areas where depths exceed what a dragline or backhoe is capable of excavating, or in cases where the spacing is limited. Hydrocutters are capable of cutting into hard rock foundations which gives them an added advantage over other excavation equipment. In cases where the concrete cutoff wall is excavated into bedrock, the depth of the effectiveness of the cutoff can be enhanced using standard grouting methods in the underlying rock, below the depth of the wall. Concrete cutoff walls also have a greater strength than SB, CB, and SCB cutoff walls. This makes these walls much stiffer and more susceptible to cracking, especially under bending strains or seismic loading conditions. If cracked, however, concrete cutoff walls are very resistant to seepage erosion. The aperture of cracking can be reduced, and the shearing resistance increased, with the use of steel reinforcement in the wall. In particular cases where concerns for cracking may be significant, a less stiff cutoff wall can be constructed that uses the addition of bentonite to create what is termed “plastic concrete.” Plastic concrete cutoff walls can undergo greater strains before cracking compared to traditional concrete walls. Both types of concrete cutoff walls are discussed in the following sections.

16.7.2 Field Explorations

Subsurface exploratory borings required to design and construct a concrete cutoff wall may vary from those required for the typical SB, CB, or SCB cutoff wall described in Section 16.2.1.4, “Field Explorations.” It is the responsibility of the designer to identify specific explorations that will determine the character of materials to be excavated and the required depth of the wall. Samples should be obtained to determine the properties of the materials to be excavated, especially if the excavation for the cutoff wall will be keyed into rock. Strength tests and rock hardness tests are useful in selecting equipment and construction methods to properly excavate the wall and construct the key. Explorations should be at intervals, and to depths, that will thoroughly define geologic conditions. Bedrock variables that can impact the constructability of the cutoff wall include weathering; hardness; variations in bedrock surface geometry, due to the presence of scour and erosion features or alluvial filled channels; and depth to bedrock. Variables that could impact the excavation through the overburden include the presence of layered soft or fat clays in the overburden, groundwater conditions (i.e., artesian conditions), the presence of boulders or cemented zones in the overburden, and zones of high permeability that could result in slurry loss.

16.7.3 Slurry Properties and Slurry Mix Design

The bentonite slurry provides support for the excavated trench/panel prior to placement of the concrete. The slurry properties, slurry mix design, and materials tests described in Sections 16.4.1, “Bentonite Slurry Mix Design,” and 16.4.1.1, “Slurry Properties,” on earth-backfill slurry also apply to the slurry used in construction of concrete cutoff walls. The bentonite should be a naturally occurring, pure, premium-grade, Wyoming-type, sodium-cation-based bentonite with high swelling characteristics.

16.7.3.1 Density

To facilitate concrete placement, the density of the slurry in the trench/panel at the time of concrete placement should be less than 78 to 80 lb/ft^3 [14]. This density is equivalent to a specific gravity of 1.25 and allows the fresh concrete to initially displace the slurry in the bottom of the excavation by lateral flow of the fresh concrete, and then by displacing the slurry upward in the trench/panel.

16.7.3.2 Sand Content

In the literature and on previous concrete cutoff wall construction projects, the sand content of the slurry prior to concrete placement has generally been limited to a maximum of 5 percent when tested in accordance with American Petroleum Institute (API) Standard RP 13B [28]. Slurry with a high sand content may have a density that is too high for successful placement of tremie concrete. As with placement of SB backfill, the successful displacement of the slurry by the concrete relies on the difference between the density of the slurry and the concrete. In addition, too much sand in the slurry can result in sand settling out of the slurry at the bottom and creating a pervious window. The use of desanders may be necessary for removal of sand from the slurry if it becomes too heavy with suspended particles.

16.7.4 Excavation

The excavation of concrete cutoff walls can be accomplished with many types of equipment including backhoes, clamshells, drag lines, or hydrocutters, sometimes referred to as rock mills. Sometimes, a combination of excavation equipment is the most economical. Concrete cutoff walls are typically constructed in panels which require a vertical cut on both ends of the panel. Typically, primary and secondary panel sequencing is used (as shown in figure 16.7.7-1, which appears later in this report). In special cases, they may be constructed using continuous trenching equipment, although this is not recommended primarily due to issues with continuity between placements of concrete. In such cases, the continuous placement of concrete is important such that new concrete placement occurs on top of previously placed concrete before the previously placed concrete has time to set. Otherwise, a defect may be created at the joint. Although mentioned here for completeness, continuous trenching does not allow for any construction delays. In most cases, this is not considered a realistic or viable construction methodology for application to dams. For panel construction methods, the clamshell (figure 16.7.4-1) or rock mill (figure 16.7.4-2) is the preferred excavation equipment over backhoes and/or draglines, which have limited excavation depths and are less capable of creating clean, vertical joints. This is a key consideration in concrete cutoff walls because a poorly constructed joint interface between panels can be the source of considerable concentrated leakage.

The purpose of panel construction, using vertical ends, is to reduce the amount of open trench at any given point and to allow a clean, continuous joint to be constructed. In continuous trenching, the toe of the newly tremied concrete must be kept as close as practicable to the excavation to minimize the amount of open trench at any given time. In most applications, the use of guide walls is incorporated into the construction to facilitate alignment of the concrete cutoff wall. Guide walls are described in Section 16.7.5, “Guide Walls.” Clamshells can be either cable suspended or equipped with a Kelly bar. The Kelly bar is used to guide and control the vertical line of the excavation and to provide additional weight to assist in the closing of the clamshell. If oversize rock is expected, sometimes a cable-supported clamshell bucket has more flexibility in terms of grabbing the rock for removal. If needed, chisels can be used to break up oversize rock within the excavation.

Figure 16.7.4-1. Clamshell excavation of portions of plastic concrete cutoff wall (Meeks Cabin Dam, Utah).

Rock mills or hydrocutters are generally the most expensive excavation tools to use; however, they can be very efficient in particular applications. These excavators consist of hydraulically operated, rotating cutters capable of cutting dense soils and milling or grinding hard rock to achieve excavation. In addition, rock mills are capable of excavating through previously placed concrete when excavating secondary panels in between previously constructed adjacent primary panels. In most cases they consist of down-hole motors within a large frame with reverse mud circulation. Excavation is accomplished by cutting the soil or grinding/crushing the rock which suspends the cuttings in the bentonite slurry. The slurry with the suspended cuttings is continuously circulated to the surface and through desanding equipment to remove the cuttings and then returned to the trench. The desanding equipment will consist of an air lift with the suction end placed near the bottom of the panel, where most of the suspended cuttings will accumulate, along with loose material. The slurry with the suspended cuttings is drawn to the surface, where it goes through mechanical separators. Generally, a vibrating screen is used to collect gravel and coarse sand particles, while a cyclone is used to separate out the fine sand sizes. The cleaned slurry is then returned to the trench.

Figure 16.7.4-2. Rock mill used to excavate portions of plastic concrete cutoff wall (Meeks Cabin Dam, Utah).

16.7.5 Guide Walls

Guide walls are lightly reinforced concrete sections constructed to grade along the alignment of the trench on each side of the trench. The functions of the guide walls are to control the line and grade of the trench, enable excavation to begin below grade, provide stability for heavy construction equipment, and allow operation near the edge of the slurry trench, as shown in figure 16.7.5-1. The guide walls may also be needed to support shoulder pipes that hold the stop-end pipes in place. Guide walls minimize overexcavation by providing support to the sides of the slurry trench where the trench excavation intersects the ground surface. The rigidity provided by the guide walls at the slurry-trench/ ground-surface interface may reduce the amount of spoil inadvertently entering the trench during excavation. The spacing between guide walls should equal the width of the concrete wall, plus 2 inches of additional clearance on each side of the wall, to allow access for excavation equipment. The designer should verify clearance requirements because required clearances depend on excavating equipment. The base of the guide wall should be constructed on firm, compact material. Typical dimensions for guide walls are 1 foot wide by 3 feet deep [9].

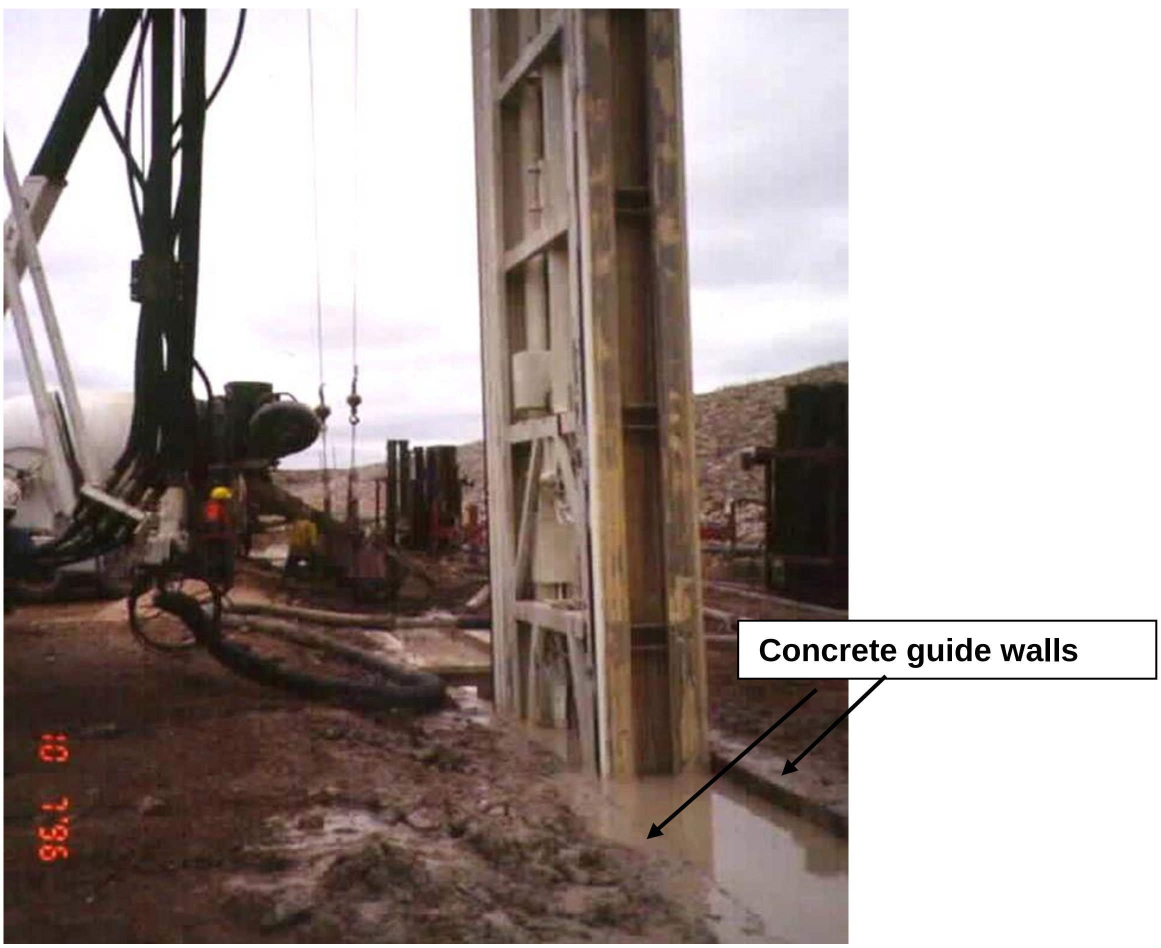

Figure 16.7.5-1. Typical guidewall used to construct a concrete cutoff wall (Meeks Cabin Dam, Utah).

16.7.6 Panel Connection (Joints)

The joint between adjacent panels is generally constructed using shoulder pipes or by removal of concrete from an adjacent primary panel by downhole excavating equipment [2, 14]. Shoulder pipes are placed at the ends of primary panels prior to placing concrete. After the concrete has attained sufficient strength to maintain the form of the pipe and not collapse into the void created by removing the shoulder pipe, shoulder pipes are usually first rotated to break any bond with the concrete, and then they are jacked out of the panel, leaving a concrete surface against which the adjacent, or secondary, panel can be constructed. Excavation for the secondary panels exposes the semicircular surface against which concrete is placed. In some cases, the cutter assembly is capable of excavating a joint into the adjacent primary panel on either side of the secondary panel. In some cases, colored concrete has been used for primary panels to make it much easier to visually determine and verify that a primary panel has been intersected during excavation of the secondary panels.

Another method of constructing the joint between adjacent panels uses downhole excavating equipment. Downhole excavating equipment is used to remove up to 6 to 8 inches of concrete from the primary panels as the secondary panel is excavated. The surface of the primary panels on both ends of the secondary panel is exposed. The joint is accomplished by concrete in the secondary panel flowing against the primary panel. Significant leakage seldom occurs through these joints, due to bentonite in the joint and the impregnation of soil by slurry in the immediate vicinity of the joint. When a rock mill is used, the rotating bottom cutters are typically configured so that they are capable of cutting into the adjacent panel to remove up to 6 to 8 inches of concrete. This creates a fresh surface against which the concrete placed in the secondary panel can bond to form a tight joint.

16.7.7 Panel Thickness, Length, and Sequence

Panel thickness is dependent on equipment type, tremie equipment restrictions, and wall tolerances. A thin wall is not necessarily an economical wall because it could require specialized excavating and tremie placement equipment. Thin walls (less than 2 feet) may create concerns for flowability of the concrete placement, arching effects, and connectivity of the joints. Thin walls require tighter tolerances and sound field control methodologies to achieve and verify joint continuity. If arching of the backfill concrete within a thin cutoff wall occurs, there is a greater concern for cracking due to tensile stresses and strains caused by bending.

Panel length is based on stability of the trench, concrete placement requirements, and construction equipment used to excavate the trench. In general, longer panel lengths reduce the number of vertical joints that are required. This is an advantage because these joints can be sources of discontinuity in the wall. A practical maximum panel length is 30 feet, which is the panel length at which concrete can effectively be placed with two tremie pipes [5] in walls at least 24 inches thick. The depth of the panel and the production capacity of the concrete batch plant are additional considerations in choosing panel length. The concrete should be supplied to the placement so that cold joints are avoided. Short panels are better than long ones when it is necessary to rely on the arching effect of soil for trench stability, or when there is danger of slurry loss through open cavities underground. Panels are constructed by first excavating trenches for primary panels of maximum chosen panel length with spaces between them of a length approximately equal to that which can be removed later by one pass of the excavating equipment. Concrete is then placed in the primary panel, and after sufficient set time, the secondary trenches in the space between primary panels are excavated and concrete placed in them. Secondary panels are generally 4 to 8 feet long. Figure 16.7.7-1 shows panel sequencing for concrete cutoff wall construction.

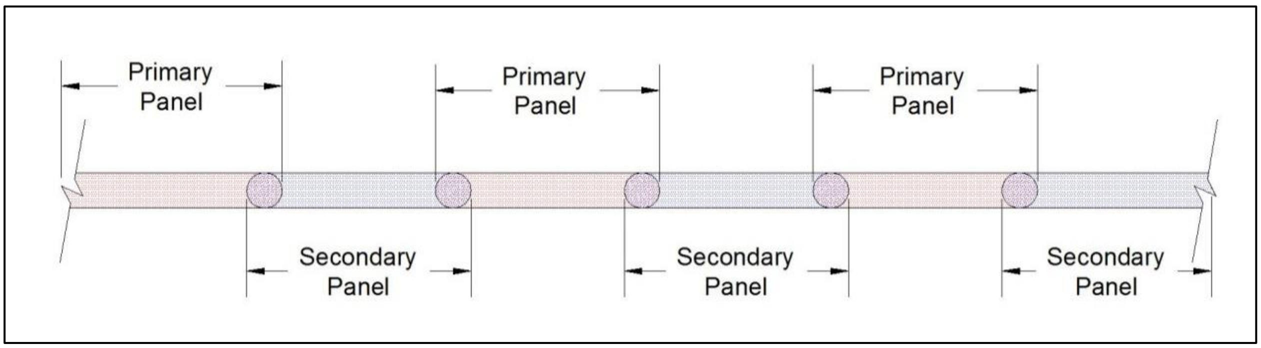

Figure 16.7.7-1. Schematic of panel sequencing for construction of concrete cutoff wall.

16.7.8 Tie-in with Existing Structures

In cases where cutoff walls are installed through an existing embankment, the alignment of the cutoff wall will often intersect or connect to structures that penetrate the embankment. These structures can include outlet works, spillways, and conduits. In other cases, the end of a cutoff wall may need to be connected to one of these types of structures. This connection can be critical to the successful performance of the cutoff because these points, if not properly designed and constructed, can be locations of concentrated seepage and higher gradients. In addition, because they can also be locations where low stress zones may exist or flaws and defects in embankment compaction can occur, the detrimental effect of concentrated seepage at such a location may be exacerbated. Therefore, significant thought must be exercised when such connections are required. In addition, because each structure and embankment are different, there is no generic connection detail that works in every case; each connection must be considered independently. Another primary consideration is the potential to damage an existing structure during the process of constructing a cutoff wall connection. The work is usually performed through a slurry-supported trenching operation with heavy equipment and no direct visual observation of the existing structure, except possibly at the surface. This often requires tight tolerances on equipment movement to minimize the potential for damage.

In most cases where a cutoff wall is excavated through an existing embankment and must connect to an existing structure, the structure will be concrete. In some cases, especially with pipes, the conduit may be steel. In almost all cases, the connection will be nonstructural but must be impervious. This will require cleaning the surface to create a sound bond with the cutoff wall. The surface of the existing structure should be scraped and cleaned with specialized equipment to remove any soil adhering to the surface. In some cases, however, construction of a direct contact connection between an existing structure and the cutoff wall may be too difficult or involve too much risk of damaging the existing structure. In these cases, other connection methods have been used to minimize damage to the existing structure. Because every connection is unique, two examples of connections between existing structures and a new cutoff wall, used in Reclamation practice, are provided in this chapter.

In the case of Reclamation's Fontenelle Dam [10], a concrete cutoff wall was constructed through the embankment after the near failure of the dam in 1965. The alignment of the cutoff wall intersected the outlet works conduit. The detail of the connection at the outlet works conduit is shown in figure 16.7.7-2.

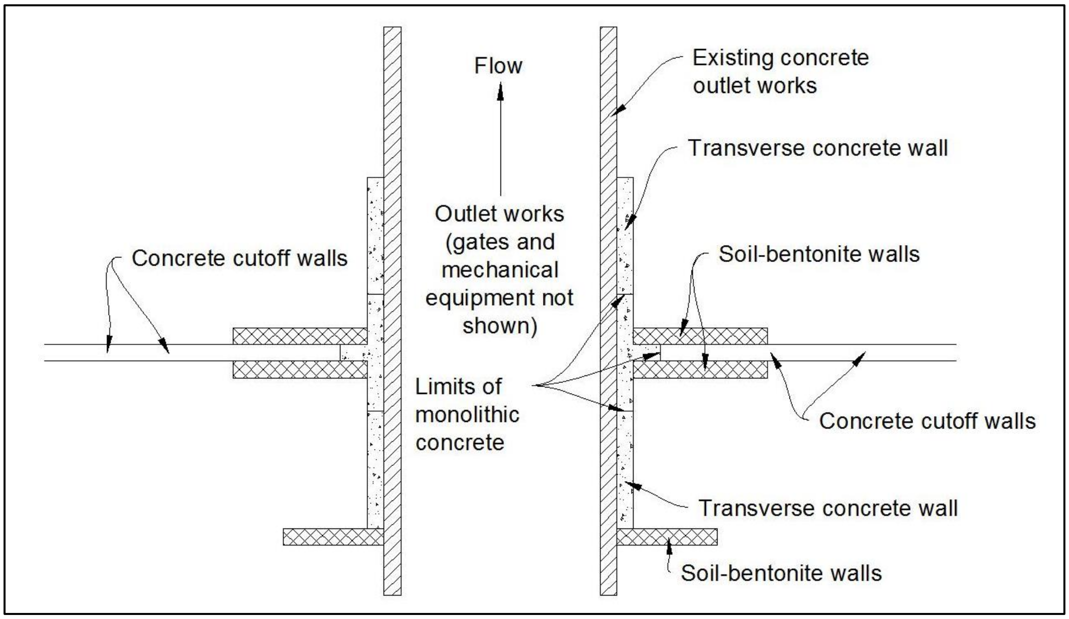

Figure 16.7.7.-2. Example of concrete cutoff wall connection with outlet works conduit structure at Fontenelle Dam.

In this case, a monolithic concrete wall was first constructed adjacent and parallel to the existing wall of the outlet works structure. Then, two parallel SB panel walls were constructed transversely to the monolithic wall. This allowed for the new concrete cutoff wall to be constructed between the SB panels and tie directly into the monolithic wall. The purpose of the monolithic wall was to provide a concrete surface to connect with the new cutoff wall that, if damaged slightly with the excavating equipment, would not compromise the integrity of the outlet structure itself. In addition, a transverse SB panel cutoff wall was constructed at the upstream end of the monolithic wall to provide a secondary barrier to seepage at this critical contact.

In another case, at Reclamation's Reach 11 Dikes [29], a geomembrane cutoff wall was constructed which intersected four different concrete outlet works structures. The connection in that case is discussed in Section 16.8, "Geomembrane Cutoff Walls."

16.7.9 Slurry Losses in Panels or Trenches

Slurry losses are dangerous to panel or trench stability. This is true for any slurry supported excavation, including CB cutoff walls. If large slurry losses occur, immediate action is necessary to maintain the slurry level within 2 to 3 feet from the top of the trench or guide walls. The quantity of fully hydrated slurry available for pumping to a panel should be at least two times the volume required to fill all excavated and unconcreted portions of the excavated panel at any time. The quantity of slurry available for pumping to a continuous trench may vary but should be approximately twice the estimated volume of open slurry supported trench, considering the slope of the backfill and the distance from the toe of the backfill to the excavation. Sufficient pumping capacity should be available to pump all of the slurry to the panel within 60 minutes. Small slurry losses can generally be handled using this reserve. Large or rapid slurry losses, whether in a single panel or a continuous trench, should be considered very serious, and steps should be taken immediately to stabilize the panel or trench. The panel should immediately be filled with granular material, bentonite slurry, sand, or any material available from trench excavation. The panel must be made stable, and the source of the losses must be determined, prior to continued excavation. Given that many deep panel-type concrete cutoff walls are excavated through existing embankments to control seepage, consideration must be given, during the design phase, to the possibility of slurry loss into existing defects or the creation of defects due to hydraulic fracture caused by the slurry itself. Fracture can initiate at zones of low lateral effective stresses due to arching of the fill, such as near steep abutment contacts or near structures where compaction may have been poor. Embankments may also contain preexisting defects that can lead to slurry losses. These defects may be due to preexisting seepage piping conduits, horizontal and/or transverse cracks due to excessive settlement or the presence of collapsible foundation soils, high permeability zones that result from poor compaction or coarse zones, localized defects such as animal burrows, or the presence of voids in certain foundation rock types. Voids in foundation rock should be considered likely if the rock is of volcanic origin, such as basalt flows, or consists of rock that may be solutioned by seepage over time, such as gypsum or limestone. In these cases, thorough geologic investigations should be performed. Other remedial actions could include pressure grouting of the foundation rock prior to installation of the cutoff wall.

16.7.10 Verticality

Verticality requirements should be dictated by the specific conditions at the site. The ability to maintain verticality within a particular soil or rock depends on factors such as the composition of the soil, presence of oversize cobbles and boulders, and type of excavation equipment. The designer should consider these factors when specifying a vertical tolerance. In general, the usual construction tolerance is 1.0 percent of the panel height. In addition to achieving a vertical panel, maintaining proper tolerances will improve the joint connectivity between primary and secondary panels and reduce the likelihood of a flawed joint. New generation rockmills or hydrocutters are typically equipped with hydraulic jacks and levelers located on the cutter frame that sense deviations in verticality and make appropriate adjustments to regain the proper tolerances. Guide walls help establish alignment and aid in maintaining verticality. Trench sidewalls can be monitored using geophysical methods, such as sonar or mechanical calipers, to measure the verticality of the side walls.

16.7.11 Concrete Mix

The concrete mix design for a given concrete cutoff wall depends on the desired properties that the designer is seeking. In general, for applications within embankments, the primary desired property of the wall is very low permeability. Although the strength of the concrete may be a secondary design property, it may still be important in particular applications where the wall will also perform as a structural element. In addition, flexibility and resistance to cracking may also be important design properties to be considered. This section is a general guide to the mix design that is divided into two types of wall: concrete and plastic concrete.

A. **Concrete.** It is important that the concrete have a high degree of fluidity and workability so that it can readily flow through the tremie pipes and force the previously placed concrete upward without plugging the tremie pipe. The concrete should also not be subject to segregation. Mix designs should be in accordance with American Concrete Institute (ACI) procedures. To achieve the necessary fluidity for tremie placement, a slump of between 6 to 9 inches is desirable. A maximum water-cement ratio is generally 0.5 to 0.6 by weight. With this water-cement ratio, an unconfined compressive strength of 3,000 to 4,500 lb/in^2 (28-day strength) is generally obtained [5]. This will produce a very strong cutoff wall but also a stiff wall that may be more susceptible to cracking at small strains.

1. **Aggregates.** The tendency for concrete to segregate can be reduced and the workability of the mix can be improved by limiting the maximum aggregate size to 1 inch. The aggregate should also be well-graded, and the workability of the concrete can be improved by the use of rounded aggregate from natural river deposits.

2. **Air Entrainment.** Air entrainment can be used to improve the workability of the mix. Because air entrainment reduces the strength of the concrete, the designer must decide on the degree to which the strength can be sacrificed in the interest of improved workability. An air content ranging from 4 to 7 percent is generally used in the mix design [5].

3. **Retarders.** Retarders have been used to prevent premature stiffening of the concrete or to delay the stiffening where difficult placement conditions may be encountered, or where the wall is very deep and more time is needed to place the backfill concrete. The designer should check the setting time against the time necessary to complete the concrete pour before allowing the use of retarders. Super plasticizers have been used to retard set time for concrete and to make the concrete more workable. Their use is dependent on the depth of the concrete placement. Their use may also result in a reduction in cracking of the wall.

4. **Cement.** Type I cement is generally used in cutoff wall concrete. The designer should investigate the potential for chemical attack on the concrete; if required, Type II or Type V cement should be used where sulfate resistance is needed.

B. **Plastic Concrete.** Plastic concrete is a variation of traditional concrete in that it also contains bentonite as a partial cement replacement along with the other concrete mix components of gravel, sand, cement, and water. Plastic concrete was primarily developed for use in embankments or soft ground conditions where more flexibility (strain to failure) is desired, due to expected settlements or bending, and where strength may not be the primary design characteristic. Plastic concrete is generally considered concrete containing bentonite that has an ultimate unconfined compressive strength less than 1,400 lb/in^2. As a result of the reduced strength and greater flexibility, the modulus of the plastic concrete is much lower than traditional concrete. In addition, plastic concrete will behave elastically at lower stress ranges, and it is generally the secant modulus or Young's modulus (when the stress/strain curve is linear in the elastic range) that is typically used for analysis purposes.

The use of plastic concrete for a concrete cutoff wall should be considered in highly seismic zones where significant bending strains may be induced in an embankment during ground shaking and in embankments that are subject to large annual reservoir fluctuations that can lead to stress/strain changes. Although concrete cutoff walls have been installed at a number of Reclamation dams, including Fontenelle Dam, Navajo Dam, New Waddell Dam, and Meeks Cabin Dam, only the cutoff wall at Meeks Cabin Dam is composed of plastic concrete. Relatively short plastic concrete wing walls were constructed at Reclamation's Reach 11 Flood Detention Dikes in Phoenix and Scottsdale, Arizona, in order to create a tie-in connection with a geomembrane cutoff wall at each dike's inlet/outlet structure [29]. When considering the use of plastic concrete for a cutoff wall, a careful laboratory testing program should be completed to evaluate alternative mix designs to optimize the desired characteristics of the wall. When considering a plastic concrete mix, the following parameters are typically used: (1) cement factor, (2) bentonite content, (3) water-cement ratio, and (4) coarse-fine aggregate ratio. Each of these terms is defined below. Table 16.7.11-1 lists design mixes for plastic concrete that have been used in other jobs. It is up to the designer to determine the mix design and properties for a particular design.

Table 16.7.11-1. Design Mixes for Plastic Concrete Cutoff Walls Used in Past Jobs

Dam name	Cement factor (lb/cubic yard)	Bentonite content (% of cement)	W/C ratio	Coarse/fine aggregate ratio	28-day UCS (lb/in^2)
Island Copper Mine	239	22	2.78	1.0	220
Wister Dam	350	10	1.0	1.39	598
Meeks Cabin	255	15	1.85	1.0	415 [1]

[1] Based on average for the mix shown.

1. **Cement Factor.** This is defined as the total amount, by weight, of cement and bentonite in a cubic yard of plastic concrete. This factor can vary considerably depending on the desired plastic concrete properties. It also includes pozzolan substitutes such as fly ash or blast furnace slag.

2. **Bentonite Content.** Percentage of cement factor (including pozzolans), by weight, which is bentonite. The designer should carefully consider how the bentonite is mixed with the other components. Adding water to the dry cement and bentonite can be problematic when the bentonite is not already fully hydrated because the bentonite will continue to hydrate in water, which can significantly reduce the slump of the mix, preventing it from having the proper flow characteristics in the tremie pipe and panel. At Island Copper Mine, the bentonite and water were first mixed in a high-speed colloidal mixer and placed into a ready-mix truck that allowed the bentonite to hydrate in the truck before adding the cement and aggregate [14]. This issue becomes more important with higher bentonite content.

3. **Water-Cement Ratio.** Weight of water/(dry weight of cement plus bentonite). This value can vary considerably and is the most significant factor affecting the overall strength of the plastic concrete mix. Because so many variables affect the water-cement ratio, it should be based on the specific design properties required, laboratory test results, and construction requirements.

4. **Coarse-Fine Aggregate Ratio.** The ratio, by weight, of fine to coarse aggregate. This variable will have impact on the density of the cutoff wall and, to a lesser extent, the overall strength. A commonly used ratio of 1:1 will produce a broadly graded and dense mix resulting in less overall consolidation. The maximum particle size is usually limited to 3/4 inch or 1 inch.

The slump of a plastic concrete mix is also an important characteristic. A slump of approximately 8 inches is generally specified to ensure that the plastic concrete backfill flows through the tremie pipe and displaces the trench or panel supporting fluid (generally bentonite slurry). Most plastic concrete designs specify a slump of 8 inches or a range of 7 to 9 inches. A higher slump can create a mix that is too fluid and will not displace the bentonite slurry fully, which can result in a high water-cement ratio that reduces the strength.

Another construction consideration that may influence the properties of a plastic concrete cutoff wall is consolidation of the backfill mix in the trench. As the mix consolidates before setting, it tends to squeeze out water, which reduces the water-cement ratio and increases the cement factor. This will produce a higher strength wall near the bottom of the trench, where consolidation stresses are the highest. At some point, this effect is likely to be reduced or halted as the plastic concrete sets up in the trench and arching effects, due to side friction, begin to develop.

The permeability of plastic concrete can be expected to be between 1 x 10^{-7} cm/s and 1 x 10^{-9} cm/s, depending on many different factors. In general: at a constant water-cement ratio, the addition of bentonite can decrease the permeability; permeability will increase as the water-cement ratio increases; permeability will decrease with increasing confining stress; and permeability will decrease with increasing age. The addition of more bentonite will not necessarily have a corresponding influence on reduced permeability. This is generally due to the necessity of adding more water and increasing the water-cement ratio. The addition of cement substitutes, such as blast furnace slag, will generally result in a reduction in overall permeability compared to regular concrete.

16.7.12 Reinforced Concrete Cutoff Walls

Because concrete cutoff walls within embankments can crack, due to settlement and/or bending loads, steel reinforcement is sometimes placed into the trench or panel to create a reinforced wall that is more resistant to cracking. Reinforcement can be in the form of steel reinforcement cages or individual H or WF steel beam sections. Panel construction of the cutoff wall is more suited to a reinforced wall because the reinforcement frame is typically constructed onsite, lifted, and lowered into the slurry-supported trench prior to placement of the concrete backfill. Reinforcement frames can be heavy and cumbersome, which can lead to significant constructability issues. Restricting the reinforcement frame to one panel length limits the weight and size of the frame and helps to reduce constructability issues. Other difficult issues to overcome when constructing reinforced concrete cutoff walls are installing reinforcement within deep panels, preventing distortion during lifting and insertion, maintaining verticality and clearance in the trench, allowing room for tremie pipes and vibration equipment, and providing continuous reinforcement across joints. The designer should consider each of these issues when evaluating the necessity of reinforcement within the cutoff wall. Each particular issue has solutions and adds additional costs or time, which should be carefully considered. The designer may find it valuable to collaborate with contractors in the industry regarding potential methodologies to overcome some of the obstacles cited here.

16.7.13 Concrete Placement

Prior to concrete placement, the trench bottom must be cleaned of any loose material. No placement of concrete should be allowed until an adequate supply of concrete is on hand at the site to keep the tremie pipe full of concrete at all times, even in the event of concrete losses. The concrete placement should be completed before the initially placed concrete starts to set. Concreting should normally be completed within 4 to 6 hours [5].

A. **Transportation.** Concrete should be transported to the placement in mixer trucks to prevent segregation of the material.

B. **Tremie Method.** Cutoff wall concrete is placed by the tremie method [30]. The recommended practice is described in ACI Standard 304, chapter 8, on tremie concrete [30]. Concrete is poured through a tremie pipe and displaces the slurry by gravity, as shown in figure 16.7.13-1. Displacement of the slurry relies on the difference in density of the two materials. Procedures must also be in place to collect the displaced slurry as the tremie placement is made. Tremie pipes are usually 6 to 10 inches in diameter. A rule of thumb is to select the diameter of tremie pipe to be at least eight times the maximum size aggregate to prevent blocking of the pipe. Steel pipe is recommended for tremie concrete because contamination from contact with aluminum pipe has been reported to cause serious weakening of the concrete. The maximum panel length that can be poured from one tremie pipe is 15 feet. Longer panel lengths require the use of more tremie pipes. Trench stability and concrete batch plant capacity usually limit the panel length to that which can be placed through two tremie pipes. To avoid cold joints, the concrete pour must be simultaneous through both tremies.

The placement begins with insertion of a plug called a “go-devil.” The plug is pushed down the pipe by the fresh concrete. The go-devil can consist of a cement mortar or other materials like vermiculite, which should have a design that prevents intermixing of concrete and bentonite slurry, does not collapse, and returns (floats) to the surface after lifting the discharge pipe.

During concrete placement, the end of the tremie pipes are to be kept submerged below the surface of placed concrete to avoid mixing the concrete with slurry. Except at the beginning of the placement, the end of the tremie pipe should be submerged in the concrete a minimum of 10 feet. Withdrawal of tremie pipe from the placed concrete, causing slurry entrapment, should be prevented, as well as excessive embedment within the mass concrete, which could result in entrapment of the tremie pipe in the concrete. To avoid cold joints, concrete placement should proceed without interruption until it has reached the required elevation. If a steel reinforced cutoff wall is to be built, the designer must provide clearances within the reinforcement cage for tremie pipes.

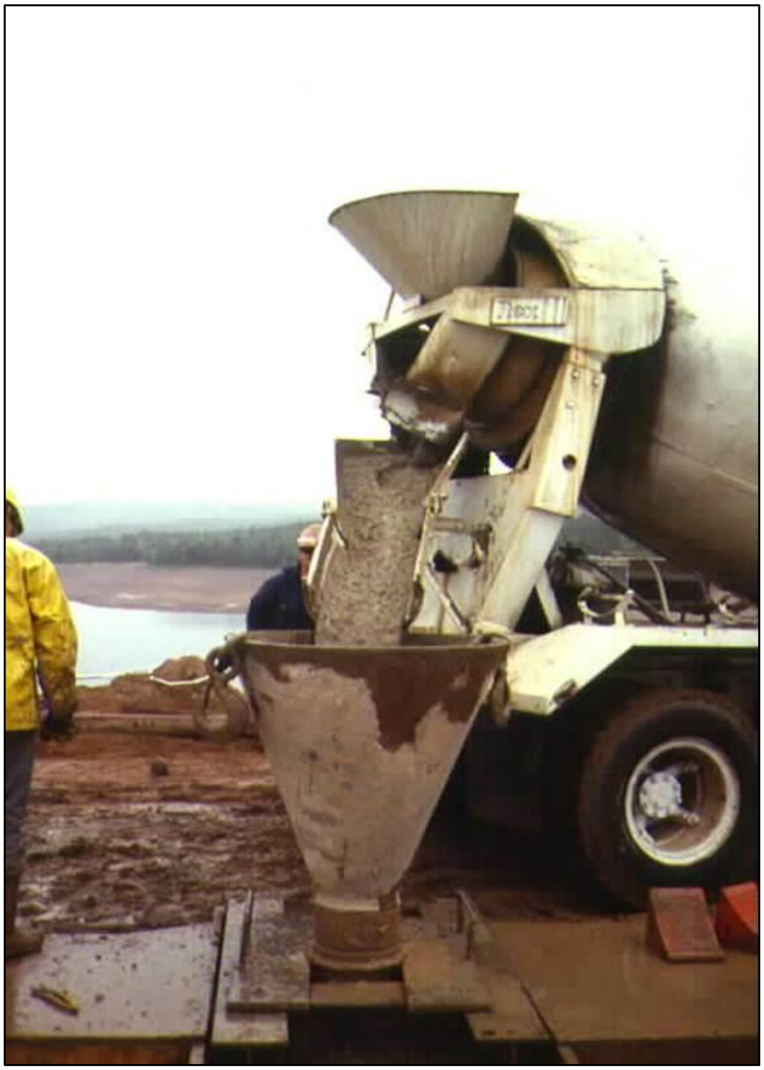

Figure 16.7.13-1. Tremie concrete placement in a concrete cutoff wall (Meeks Cabin Dam, Utah).

C. **Defects.** The typical defects that can result from early withdrawal of the tremie pipes are cold joints and zones of segregated or contaminated concrete. If the concrete does not flow properly, slurry can contaminate the concrete, and cavities can occur at the end of panels. All contaminated concrete associated with cold joints should be removed prior to continued concrete placement.

D. **Construction Control.** Construction control of slurry is discussed in Sections 16.4.1.1, "Slurry Properties," and 16.4.2, "Trench Stability During Construction." For the successful placement of concrete, the density of the slurry and the sand content must be monitored, especially within deep panels. The displacement of the slurry is critical to the achievement of a high quality wall. Desanding equipment can be used to reduce the density of the slurry. The hydrocutter or rockmill can be used to lift the slurry from the bottom of the trench for desanding. Samples of the trench or panel bottom are necessary to ensure removal of all sediments from the bottom. Suction pumps or air lifts are commonly used to remove sediments from the trench or panel. Suction pumps are most efficient to depths of approximately 100 feet. At depths greater than 100 feet, air lifts are more efficient. As stated above, sufficient slurry reservoirs should always be available in the case of large slurry loss.

Construction control of concrete includes monitoring and testing of the mix (i.e., slump testing, temperature, etc.), testing of concrete aggregates, and testing of concrete cylinders. High quality assurance and quality control are important. The trench or panel should be sounded to determine that the design bottom elevation of the trench or panel has been reached and to provide data to estimate the volume of concrete expected to be placed. This allows the onsite inspector to be sure that enough concrete is available to complete a full pour and reduces the likelihood that a cold joint will occur. Once the slurry has been desanded and is at the correct density, and concrete placement has begun, the trench or panel should be periodically sounded to ensure that the tremie pipe remains embedded in the concrete to the specified depth.

During tremie placement, the displaced bentonite slurry will also need to be handled at the top of the panel to keep the working surface clean.

16.7.14 Submittal Requirements

With something as technically challenging as a deep concrete cutoff wall, especially if constructed through an embankment, the designer should use a negotiated procurement process that allows technical proposals to be submitted for review and consideration prior to award. Either during the proposal process or prior to beginning construction, after award, the specifications should require a comprehensive list of submittals. In addition to the suggested submittal requirements cited in Section 16.4.9, “Submittal Requirements,” for earth-backfilled slurry cutoff walls, additional submittals should be considered that are specific to concrete cutoff walls. These may include:

Qualifications:

- Onsite supervisor resume
- Previous concrete cutoff wall jobs of similar size and complexity

Concrete Cutoff Wall Plan:

- Bar chart construction sequence drawing showing dates of anticipated cutoff wall construction and completion.
- Anticipated production rates, in square feet per day.
- Methods and frequency of monitoring trench alignment, depth, and verticality.
- Description of proposed method of trench excavation, including equipment and sequence of constructing cutoff wall. Include panel dimensions and sequencing.
- Method of mixing bentonite slurry and volume of hydrated slurry to be maintained onsite during excavation if rapid slurry loss occurs.
- Method of stopping rapid slurry loss during trench excavation.
- Method of sampling slurry at the bottom of the trench or panel.
- Method for desanding, including method used to remove heavy slurry and description of equipment.
- Method for breaking and/or excavating cemented soils and/or boulders in the trench, and description of equipment.
- Method of creating joints and ensuring joint continuity.
- Tremie equipment and tremie placement methodology, including the handling of displaced slurry during concrete placement

16.7.15 Contract Requirements

As mentioned in the previous section, the negotiated procurement process is well-suited as a contracting methodology for a concrete cutoff wall.

16.8 Geomembrane Cutoff Walls

Although geomembrane cutoff walls are not commonly used in embankments, there are cases of their use, not only in embankments but as seepage cutoffs for contaminated groundwater collection trenches. In general, geomembrane cutoff walls may not provide as much confidence in the uniformity and effectiveness of the constructed wall due to installation complexities and joint details. However, careful planning, proper selection of geomembrane wall type, and construction methodologies can significantly mitigate these concerns. Costs may not be competitive with other cutoff wall types because geomembrane cutoff walls require the excavation of a trench in order to install the geomembrane, including backfill. The primary advantage of a geomembrane cutoff wall is that the geomembrane can be installed in the trench to provide a seepage cutoff, while the trench can be backfilled with a sandy or gravelly material to serve as a filter and/or drain. This was accomplished for approximately 12.5 miles of the Reach 11 Dikes in Phoenix, Arizona [29]. The installation of the geomembrane and a filter sand created both a cutoff and a filter zone for these dikes that were originally constructed as homogeneous embankments on top of collapsible foundation soils consisting of erodible and dispersive materials. For detailed specific information on geomembrane types and properties, refer to Reclamation's *Design Standard No. 13 – Embankment Dams*, Chapter 20, "Geomembranes." [38]

16.8.1 General Description

Geomembrane cutoff walls are generally installed within vertical excavated trenches supported with biopolymer slurry, as discussed in Section 16.8.2, "Biopolymer Slurry," below. A bentonite slurry would typically not be used to install a geomembrane cutoff wall because the trench would then be backfilled with soil, creating a redundancy that is generally not required (i.e., a geomembrane cutoff wall inside of a SB cutoff wall). However, such an application may be beneficial under certain conditions due to potentially better trench stability with the use of bentonite and the redundancy of the SB and geomembrane. Generally, however, a geomembrane cutoff wall trench is backfilled with a filter sand or gravel material that can serve either as a filter zone or as a permeable zone to collect water, respectively; thus, bentonite is not acceptable. The geomembrane cutoff may be installed on either the upstream or downstream side wall of the trench, depending on the purpose. Geomembrane used in the cutoff can be installed in prefabricated panels with interlocking joints, similar to sheet piles, or it can be installed as a continuous panel using field welding of adjacent sheets. This latter process is more complicated and requires close field inspection, and installation in the trench is difficult to control to ensure an adequate cutoff.

16.8.2 Biopolymer Slurry

Various types of biopolymer fluids are currently available for use in constructing trenches. Biopolymers are high molecular weight, organic chemicals that, like bentonite, will swell in water and increase the viscosity of the water. These slurries may be composed of organic, manmade chemicals or natural products such as ground guar beans, water, and proprietary degradable additives.

Because the installations involving geomembrane cutoffs are often performed for the purpose of collecting groundwater seepage, or, as in the case of the Reach 11 Dikes in Arizona, to act as a water barrier, the trenches excavated for their installation must be supported or stabilized with slurry as shown in figure 16.8.2-1. In some cases, the geomembrane may be inserted within a bentonite slurry trench, which is then backfilled with SB or CB, forming what has been called a composite cutoff wall. However, in most cases, degradable biopolymer slurry is used. These slurries, which have been used in the drilling industry for a long time, are divided into two general categories: natural or synthetic. The properties of each slurry vary considerably, and biopolymer slurry properties are different from bentonite slurry. Degradable biopolymer slurry is designed to revert either naturally, or with the addition of chlorine or hypochlorite solutions, to the viscosity of water so that it can drain naturally out of the trench or be recovered. This leaves an uncontaminated backfill capable of performing the function of filtering or drainage. Biopolymer slurry also has a lower viscosity than bentonite slurry and does not create stability by the formation of a filter cake that allows hydrostatic head to develop on the walls of the trench. The biopolymer slurry does tend to temporarily seal the trench wall, but instead of a filter cake (as with bentonite slurry), a very thin, slimy, gelatin-like substance bridges over the voids of the formation to support the trench with its high gel strength and allow hydrostatic head to develop on the trench wall.

Figure 16.8.2-1. Biopolymer slurry trench (Reach 11 Dikes, Arizona).

Biopolymer slurries should be used with caution in coarse formation soils with high void ratios due to the lower viscosity of the slurry and the tendency to degrade over time. This can result in trench instability and collapse. One primary limitation of guar gum slurry is that its effective life is 1 or 2 days. Unchecked, guar gum slurry naturally deteriorates, due to enzyme action from the soil and local groundwater, leaving only water and simple sugars, which are consumed by microorganisms in the soil. However, biopolymer slurry suspends sediment during excavation, and the density will increase. The success of biopolymer slurry depends on maintaining the slurry in an active state (viscous) during excavation, and this can prolong the functional properties of the slurry for up to 2 weeks.

Testing of biopolymer slurry properties is similar to testing performed on bentonite slurry. Primary quality control parameters for the slurry are viscosity, density, filtrate loss, and pH. Chemical additives can be used to adjust the properties of the fresh slurry and slurry in the trench to extend the working life.

A. **Slurry Density.** The density or unit weight of the slurry provides the hydrostatic force necessary to provide stability to the sides of the trench during excavation. The density of a typical biopolymer slurry is essentially that of water, varying between 62.4-63.0 lb/ft^3. During excavation, the slurry in the trench suspends noncolloidal solids (silts and sands), and the slurry density can easily exceed 70 to 75 lb/ft^3. Stability analyses should be performed using the 63.0 lb/ft^3 density. If additional hydrostatic pressure is required for stability, provisions for increasing the slurry head should be provided in the design. As with bentonite slurry, a maximum value should also be placed on the density of the slurry in the trench if sand or gravel backfill is to be placed in the trench. This maximum value may depend on the type of biopolymer slurry used and the expected maximum densities of the slurry in the trench. The purpose of a maximum density is the same as for bentonite slurry: to achieve adequate difference in slurry density versus the density of the backfill material so that the backfill displaces the slurry. If the backfill serves as a drain or filter, it becomes even more critical to displace the slurry because it may contain colloidal and noncolloidal size particles that may decrease the permeability of the backfill. Figure 16.8.2-2 shows the installation of a trench drain used to assist in dewatering at A.V. Watkins Dam. The trench was supported with biopolymer slurry. Once the trench was fully excavated, an ASTM C33 concrete sand was placed in the trench in addition to perforated drain piping and pumps. It was critical that suspended silt and sand be minimized to reduce slurry density for backfill placement and ensure adequate permeability.

Figure 16.8.2-2. Biopolymer slurry trench (A.V. Watkins Dam, Utah).

B. **Viscosity.** The viscosity of the slurry should be monitored to ensure that the gel strength is sufficiently high so that the slurry will bridge over the void spaces within the formation material. Generally, a Marsh funnel viscosity measurement is used similar to bentonite slurry. This is the time required for 946 mL (equivalent to 1 U.S. quart) of slurry to drain from a standard Marsh funnel. As with bentonite slurry, the designer should avoid specifying Marsh funnel viscosities for slurry in the trench because the presence of suspended solids from the excavation will significantly affect measured values. Because biopolymers can vary significantly and are proprietary, the required minimum Marsh funnel viscosity for successful performance in a trench is not well defined. One contractor has recommended a target plastic viscosity of 40 centipoise (cP) for guar biopolymer slurry. In the Xanthakos book [5] on slurry walls, a correlation is presented that relates plastic viscosity (in cP) to Marsh funnel viscosity. Using this relationship, a target plastic viscosity of 40 cP would be equivalent to a Marsh funnel viscosity of 125 seconds. Other biopolymer manufacturers indicate that in a dry sand or gravel, a Marsh funnel viscosity of about 60 seconds is usually adequate to prevent fluid loss through the trench walls. Personal communication with other biopolymer experts has indicated that an optimum viscosity of 34 to 80 Marsh seconds is acceptable; however, the polymer used and the formation soil type must be taken into consideration.

C. **Gel Strength**. The slurry suspension should have sufficient gel strength (minimum shear stress required to produce flow) to maintain a sufficient amount of noncolloidal solids in suspension for the required slurry density. Theoretically, the maximum particle size that can be maintained in suspension is directly related to gel strength of the slurry suspension. As a general rule, natural guar biopolymer slurries will have higher viscosity (greater gel strength) than synthetic biopolymer slurries.

D. **Filtrate Loss.** Although biopolymer fluids do not form a filter cake like bentonite slurry, the filtrate loss test (measurement of slurry losses through a filter paper in 30 minutes at a constant applied pressure) can still be used as an indicator of slurry performance. As with the majority of field tests, this test is significantly impacted by the presence of suspended noncolloidal solids and should not be used as a control for slurry in the trench. The designer should use this test only as a measure or index of filtrate properties of the fresh slurry prior to introduction into the trench. The allowable filtrate loss, as measured in this test, will typically be greater for biopolymer slurry than the typical 5-percent bentonite slurry. Because many biopolymer fluids are proprietary, along with additives, the designer may have to rely on the contractor to propose the quality control values. Even if this is the case, a high filtrate loss value should raise concerns for trench stability. Guar-based biopolymers will generally have a significantly lower filtrate loss than synthetic polymers.

E. **pH.** Biopolymer slurries generally work best at a pH range of 7 to 10. In some cases, the pH must be maintained at 8.0 or higher to limit the breakdown of the slurry by naturally occurring enzymes in the soil. Once the trench is complete, the breakdown of the slurry is usually accelerated by the addition of chemicals such as hypochlorite solutions and proprietary additives. The pH of the slurry thus becomes very important to its successful performance during excavation and requires the presence at all times of an experienced biopolymer engineer.

F. **Water for Biopolymer Slurry.** Water quality can play a significant role in the performance of a particular biopolymer slurry. General guidelines are that the pH should be between 6 and 8, and total dissolved solids should be less than 1,000 milligrams per liter (mg/L). However, the contractor should test anticipated water sources to ensure that the water meets the requirements for a given biopolymer slurry. These requirements can typically be obtained from the manufacturer.

16.8.3 Geomembrane

The type and physical properties of the geomembrane used in a cutoff wall application should be determined by the designer using available published data, manufacturer's specifications, and evaluation of design and performance requirements. The designer should also consider the physical dimensions of the trench when selecting the method of installation. For shallow trenches, continuous geomembrane panels have been used by performing field welding of joints at the site. Weights are often attached to the bottom of these geomembranes before they are placed into the trench so that they will fall through the slurry to the bottom. When using this procedure, they must be supported at the top of the trench to avoid the loss of the geomembrane into the trench. This method of installation has less reliability and requires close monitoring of installation to achieve a successful cutoff.

For deep installations, it is recommended that interlocking geomembrane panel construction be used. In this methodology, the geomembrane panels have interlocking joints (typically factory produced and delivered to the site). Each panel is stretched across a steel frame, horizontally, on the ground surface, and then lowered vertically into the trench, with the interlocking joint of the new panel attached to the joint of the panel in place (see figures 16.8-3-1 and 16.8.3-3). Continuity of the joint can be verified by wiring installed in each joint that creates a circuit when the joint of the new panel reaches full depth, and connects with the wire contact in the previously installed panel. Figure 16.8.3-1 shows typical geomembrane panel connection joints that have been used. The particular joint shown was used by SLT Inc. on the Reach 11 Dikes project. One advantage of the interlocking vertical joints on the geomembrane panels is the limited flexibility the panels have to move vertically, due to sliding along a joint. In the case of the Reach 11 Dikes, this was considered a key design factor because the major cause of serious cracking of the dikes is collapse of foundation soils when wetted and resultant differential settlements.

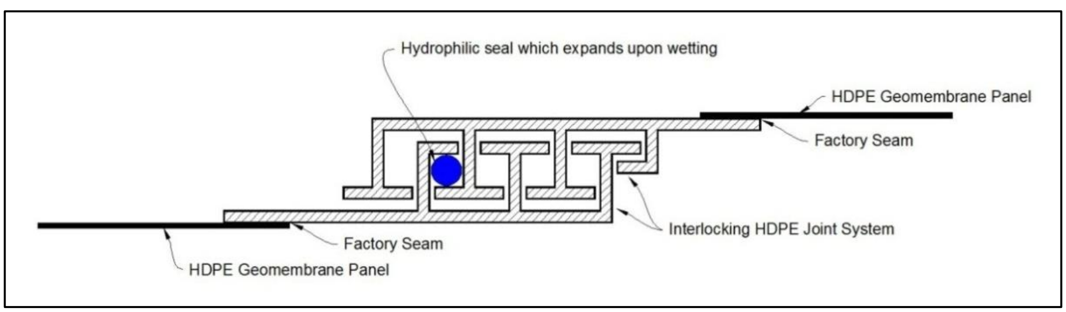

Figure 16.8.3-1. Typical schematic of GSE, Inc., curtain wall interlock joint used in geomembrane panel cutoff wall construction at Reach 11 Dikes, Arizona.

In the case of the Reach 11 Dikes, the geomembrane cutoff wall was connected to four outlet works structures along the alignment. Each outlet works structure had vertical sidewalls. Because the compaction against these sidewalls may not have been as good as the dike embankment soil, no filters are present, and the dike soils are highly erodible, this contact was considered a critical location for piping to initiate. The designers did not want to attach the geomembrane to the outlet works wall, due to concerns about separation, and a filter was considered an essential element at this contact. Figure 16.8.3-2 shows the connection detail that was used at the outlet works structures. A 20-foot-long plastic concrete cutoff wing wall was first constructed transversely to each of the concrete walls of the outlet works to the full depth of the geomembrane cutoff wall. The plastic concrete cutoff wing walls were constructed within a bentonite slurry-supported trench. Later, during biopolymer trenching for the vertical geomembrane installation, the trench was carried to the outlet works wall immediately downstream and adjacent to the plastic concrete cutoff wall. The biopolymer trench was also widened from 2 to 4 feet at this location. The geomembrane was carried adjacent to the plastic concrete cutoff wall for 10 feet to ensure overlap with the wall. This connection detail provides a cutoff and a widened filter zone at the critical interface of the embankment with the outlet works wall.

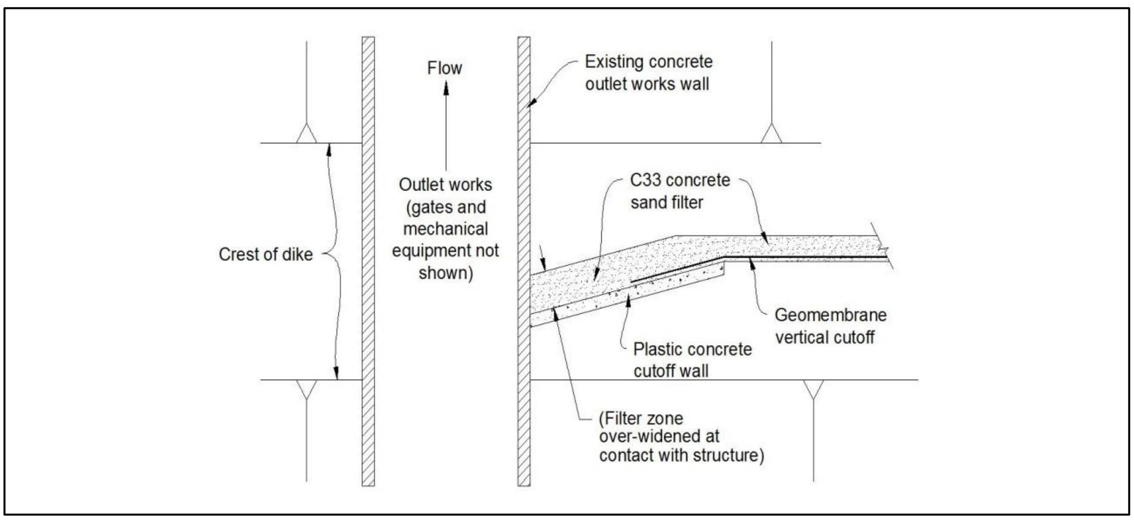

Figure 16.8.3-2. Connection detail for vertical geomembrane cutoff wall with outlet works structures at Reach 11 Dikes, Arizona.

Figure 16.8.3-3. Installation of geomembrane cutoff wall into biopolymer slurry trench (Reach 11 Dikes, Arizona).

16.9 Sheet Pile Cutoffs

Sheet pile cutoff walls have been used throughout Reclamation's history as primary seepage cutoff features, temporary cutoff elements in cofferdams, and sometimes as structural support for concrete overflow weir structures. The most recent use of sheet pile by Reclamation was as a seepage cutoff for the modification of Tarheel and Fourth Creek Dams on the Coquille Indian Reservation (Bureau of Indian Affairs), Oregon, in 2004 [31]. In some cases, such as Rye Patch Dam in Nevada, constructed in 1936, steel sheet piles were driven through the silty sand and sandy alluvium foundation soils, providing a seepage cutoff. These sheet piles were embedded into an underlying, very weakly cemented lacustrine deposit to provide a positive cutoff.

16.9.1 General Description

Sheet pile cutoff walls have traditionally been made of steel. Steel sheet piles can be very strong and hold up well to being hammer driven during installation. Their strength also allows them to be driven into harder materials for embedment, and they have good resistance to bending. However, historically, the types of sheet piles used in embankments have varied from wood, to steel, to more recent flexible vinyl materials and much stiffer reinforced polymer composite, fiber reinforced materials, such as those used at Tarheel and Fourth Creek Dams, and shown in figure 16.9-1. Both dams are located on the Coquille Indian Reservation. Sheet piles can have different shapes and differing joint details, depending on the manufacturer, but are all designed for strength, joint continuity, and other strength characteristics such as modulus of elasticity and shear modulus of elasticity. They can be considered essentially impermeable with respect to soils.

Figure 16.9-1. Installation of sheet pile cutoff wall (Coquille Dam, Oregon).

16.9.2 Design

When considering the use of sheet piles, there are numerous design factors that must be considered. These include:

A. **Strength.** The designer should consider the loads that will be applied to the wall when selecting the type of sheet pile. Loads that should be considered are vertical loads, lateral shear loads, and bending. In addition, when determining the best type of sheet pile, consideration must be given to the types of soils and/or materials that the sheet pile will be driven through and/or into. The driving loads must also be considered.

B. **Loading Conditions.** When selecting sheet pile type, the loading conditions that the sheet pile is likely to be subjected to are critical. Under static conditions, the loading will be dependent on the location of the sheet pile within the embankment. A centrally placed sheet pile through the centerline of the embankment and/or foundation will have a generally symmetric load placed on it. If the sheet pile wall is offset upstream or downstream, it will have lateral loads that will create shear across the wall and produce bending. These loads may be exacerbated by differential settlements, annual drawdown and filling cycles of the reservoir, and whether the bottom of the sheet pile is keyed into a harder, stiffer soil or rock.

Seismic loadings can cause increases in vertical stresses due to settlement, increases in the lateral loads due to shaking, and increases in bending loading if slope movement or differential movement occurs. Careful consideration of all types of loadings should be applied to the selection of the sheet pile type.

C. **Special Design Considerations.** Other design considerations are the method of installation of the piles, the types of soils that the sheet pile will be driven through, and the environment of the surrounding soils and groundwater.

1. **Installation.** Sheet piles are generally installed by a pneumatic vibratory hammer. For some sheet pile applications, the installation can be aided by water-jetting; however, jetting should never be used in an embankment or foundation due to the risks of damaging the embankment or foundation. The installer should not be allowed to continue the vibratory hammer installation when pile penetration stops or slows to a specified rate. This indicates the presence of an obstacle or very hard layer, and different requirements should be specified for such cases. These requirements can include attaching special shoes to the bottom of the sheet pile to allow further penetration, increasing the hammer force, and/or completely removing the pile and replacing it with a sheet pile more capable of penetrating the obstacle. As an example, at Tarheel and Fourth Creek Dams (Coquille Indian Reservation), the contractor was allowed to replace an obstructed

reinforced polymer composite pile with a steel sheet pile equipped with a sharp shoe to penetrate through buried trees in the foundation soils. In some cases, a decision could be made to allow an adjustment to the alignment of the sheet pile wall to allow the wall to go around the obstruction. In instances where large, very hard boulders may be embedded in the soils, the use of sheet piles may not be practical, or provisions must be made to adjust the depth or alignment of the sheet pile wall.

Another method of installation of weaker and more flexible sheet piles involves the use of a much stiffer (often steel) mandrel to which the sheet pile is attached. This mandrel, or shoe, is then vibrated to the required depth and extracted, leaving the sheet pile in place. There are many differing and proprietary types of mandrels and installation methods that can be used in this type of installation. If considering the use of more flexible sheet piles, the designer must also take into consideration the type of installation methods that may be employed.

2. **Soil Types.** In general, the use of sheet piles is going to be limited to less dense and softer soils and finer-grained soils. Alluvial soils that contain numerous cobbles and/or boulders may not be suitable for piles. Also, alluvial and fluvial lacustrine soils, which contain layers of hard pan or strongly cemented zones, may not be good applications for sheet piles. When considering these soil types, it may be appropriate to construct a test installation to evaluate the feasibility of a given type of sheet pile.

3. **Soil and Groundwater Considerations.** The designer should consider soil and groundwater chemistry when it might impact the type of pile used. Steel sheet piles, or poly-coated steel type sheet piles, may corrode in the presence of brackish or salt water. For most Reclamation embankment dams, this is not an issue. In any case, a corrosion expert should be consulted for any application.

16.10 Secant Pile Cutoffs

Secant pile walls are formed by constructing a series of overlapping, concrete-filled drill holes to form a continuous barrier to seepage. Secant pile walls can also be used to create a structural wall for excavation support and groundwater seepage cutoff. If additional strength is required, secant piles can also be reinforced. The advantage of secant piles is that they can be constructed through loose, cohesionless soils below the groundwater table, cobbles and boulders, and through rock. Reclamation has used a secant pile cutoff wall to intercept a buried alluvial channel within the right abutment bedrock of New

Waddell Dam in Arizona [32]. At Mormon Island Auxiliary Dam in California, Reclamation used secant pile walls to serve as ground support for vertical excavations through silt, sand, gravel, cobbles, and boulders from gold dredging operations that included penetration into the underlying bedrock of amphibolite schist [33].

16.10.1 General Description

The construction of a secant pile cutoff wall is similar to that of a concrete cutoff wall in that it is built using alternate primary (initial) piles and secondary (closure) piles (figure 16.10.1-1). The primary piles are installed first, followed by the secondary piles. The spacing between primary and secondary piles is designed so that the secondary piles cut into (overlap) the primary piles, creating a continuous cutoff. Secant piles are round because they are installed using drilling methods. Typical drilling methods include “Kelly drilling,” which allows different types of drilling tools to be used within a cased hole and helps to maintain verticality. Kelly drilling generally refers to any type of polygonal pipe section that passes through a mated bushing or rotary drive, or table. This bushing is rotated via the rotary table, which turns the drill string and allows vertical movement of the drill string. Casing is advanced concurrently with the drill bit, which maintains hole stability and stiffens the drill string. Top drive rotary crawler drills are well-suited and can use an oscillator attachment to help advance the casing and assist in the extraction of the casing (see figures 16.10.1-2 and 16.10.1-3). Typical hole diameters used in secant pile construction range from 24 to 48 inches. Once the hole has reached the design elevation, reinforcement in the form of H-piles, wide flange steel sections, or rebar cages can be lowered into the casing prior to backfilling with concrete.

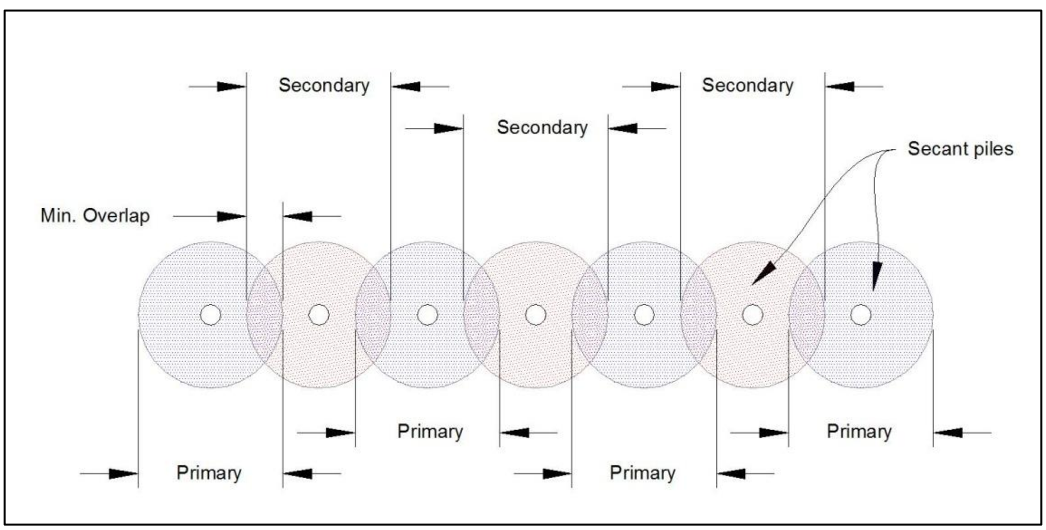

Figure 16.10.1-1. Schematic of general sequence of secant pile cutoff wall segments.

Figure 16.10.1-2. Drill bit used for installation of secant pile cutoff wall (Mormon Island Auxiliary Dam, California).

Figure 16.10.1-3. Installation of secant pile cutoff wall (Mormon Island Auxiliary Dam, California).

16.10.2 Design

A secant pile wall design is similar to the design of a concrete cutoff wall. Critical design features are strength, permeability, and constructability issues including alignment (verticality) and continuity.

A. **Strength.** The strength of the backfill concrete used in the secant pile wall must be designed based on the site-specific needs and the intended function of the wall. Because the secant pile wall is a type of concrete cutoff wall, the same considerations for strength apply to this type of wall, as outlined in Section 16.7.11, “Concrete Mix.” The intended purpose of the cutoff wall will primarily define the strength requirements including evaluation of the potential for differential loading, settlement, bending, cracking, and seasonal variations in loading due to reservoir filling and drawdown. These loading conditions will determine strength requirements and influence the need for things like steel reinforcement or the use of strength altering additives, such as bentonite, to make plastic concrete in the secant pile backfill.

B. **Permeability.** Since secant piles typically use concrete backfill, the permeability of a secant pile cutoff wall will be affected by the same factors that affect the permeability of concrete. These factors include the volume of cement, water-cement ratio, gradation of the coarse and fine aggregates, use of pozzolan substitutes, addition of bentonite, continuity of overlap, and the permeability of the aggregates used in the mix. In general, the permeability of a concrete secant pile cutoff wall will be very small ($< 10^{-9}$ cm/s), which is essentially the permeability of concrete for a wall without any flaws. The continuity of the overlap in secant pile walls may be the most critical factor in the final in situ permeability. The actual permeability of the wall could be a few orders of magnitude higher due to imperfections or flaws created during the construction process and/or due to faulty joint contacts. If very low permeability values are needed, a testing program is recommended to evaluate various mix alternatives, and careful quality control is required.

C. **Constructability.** The most critical aspect of constructing an effective secant pile cutoff wall is continuity of the wall. This requires accurate drilling of each pile and the ability to achieve and verify accurate overlap of each secondary and primary pile. In addition to vertical accuracy, the lateral accuracy, as measured along the ground surface, is also important. Lateral accuracy can be achieved in a way that is similar to panel-constructed concrete cutoff walls: by construction of guide walls or, when more accuracy is required, the use of a template guide trench that actually outlines the top of each individual pile with the use of concrete or steel forms.

For vertical accuracy, it is important to verify plumbness of the drill string continuously during the drilling process. The use of a Kelly bar and casing, which stiffens the drill string, can assist in this process. Various downhole survey techniques are available to verify vertical accuracy. The type of instruments used for verticality assurance and control are sometimes proprietary, such as Sonicaliper or downhole survey techniques, and are similar to inclinometer probes. In the secant pile cutoff wall at Reclamation's New Waddell Dam, the drilled piles were mostly constructed through rock and very coarse terrace gravels with cobbles and boulders, above groundwater, and, therefore, in the dry. These secant piles were of small diameter (15 inches) with a specification requirement that the intersection of each pile has a minimum thickness of 8 inches. A downhole video camera was used in every secondary hole to visually verify that each side of the secondary secant pile hole intersected the previously placed concrete of each adjacent primary secant pile. This proved quite effective in the verification of continuity. If a secant pile is deemed out of the vertical tolerance range or does not meet the specified minimum overlap requirements with the adjacent columns, the pile can be backfilled with lean concrete and redrilled. However, this method does not work if the hole is slurry supported, which is often the case.

Backfilling of secant piles is accomplished using tremie methods (figure 16.10.2-1). This requires close control of the concrete slump and embedment depth of the tremie pipe within the concrete backfill. The same requirements outlined in Section 16.7.13, "Concrete Placement," apply to placement of concrete within secant piles. In some cases, secant piles are installed using steel casing, above any local groundwater. In other cases, water or bentonite slurry may fill the casing if the secant pile is below groundwater and if slurry support is used. In these cases, the same concerns exist for full displacement of water or slurry as they do for concrete cutoff walls.

When the secant piles contain reinforcement, the designer must ensure that sufficient lateral space exists within the pile to extend the tremie pipe completely to the bottom of the secant pile. In past secant pile cutoff walls, reinforcement has consisted of both prebuilt reinforcement cages and individual steel wide flange sections. Wide flange steel sections were installed in every other secant pile at Mormon Island

Auxiliary Dam to increase shear and bending strength for each key block wall. Figure 16.10.2-2 shows the secant piles after excavation of one of the key blocks, and the wide flange steel sections can be seen exposed in some of the secant piles.

Figure 16.10.2-1. Tremie backfilling of secant piles (Mormon Island Auxiliary Dam, California).

Figure 16.10.2-2. Secant piles exposed in the wall key block excavation showing exposed steel wide flange reinforcement (Mormon Island Auxiliary Dam, California).

16.10.3 Submittals

A secant pile cutoff wall installation requires a skilled and experienced contractor that understands drilling and concrete placement methods. Because the successful performance of secant pile cutoff walls (as well concrete cutoff walls) depends on continuity of the joints vertically, they must be installed with tight tolerances. For such specialty work, the designer should consider a negotiated procurement process that allows technical proposals to be submitted for review and consideration prior to award. Either during the proposal process or prior to beginning construction, after award, the specifications should require a comprehensive list of submittals. In addition to the suggested submittal requirements cited in Section 16.7.14, "Submittal Requirements," for concrete cutoff walls, additional submittals should be considered that are specific to secant pile cutoff walls. These may include:

Qualifications:

- Onsite supervisor resume
- Previous jobs of similar size and complexity

Secant Pile Cutoff Wall Plan:

- Bar chart construction sequence drawing that shows the dates of anticipated cutoff wall construction and completion.
- Anticipated production rates in linear feet per day.
- Methods for ensuring alignment, depth, verticality, and overlap.
- Description of proposed drilling equipment and sequence of constructing the secant piles. Include secant pile diameters, width at overlapping intersection of piles, and methods of drilling through the specific materials at the site. Include methods of cleaning pile overlap joints. Include a description of proposed method to correct or replace secant piles that are deemed out of alignment tolerance or specified overlap requirements.
- Tremie equipment and tremie concrete placement methodology.
- Methods to handle slurry and slurry waste.
- Source of water for concrete mix and water chemistry test results.
- If reinforced secant piles are required, the contractor must show how the vertical and lateral alignment of the reinforcement will be maintained during concrete placement, type, and sequence of spacers.

16.11 Deep Soil Mixing Cutoff Walls

Deep soil mixing (DSM), synonymously referred to as soil mix walls (SMW), is a soil treatment methodology by which soil is blended and mixed with cementitious and or other agents to treat soils in situ to create a seepage barrier wall or foundation elements, or to improve strength and reduce compressibility. Although the application cited here is for cutoff walls to control groundwater seepage, DSM technology has been used for many other applications, such as seismic remediation, which is beyond the scope of this document. The DSM process has been used since the 1950s and was initially developed for treatment of soft clays. The method has been refined to produce in situ soilcrete wall elements and is used extensively in Japan, Europe, and the United States. Reclamation first used DSM to construct an upstream cutoff wall at Jackson Lake Dam in Wyoming in the 1980s. Also, as part of the modifications at Jackson Lake Dam, DSM was used to create cellular confinement of potentially liquefiable soils, in the form of honeycomb structures, at the upstream and downstream ends of the embankment [34].

16.11.1 General Description

DSM technology and equipment vary from contractor to contractor, similar to other forms of specialty geotechnical work such as jet grouting. In most cases, a water/cement grout (wet method) is introduced into the soil, using powerful, often overlapping multiple augers to mix the water-cement grout with the in situ soil to create a soil-cement structure consisting of overlapping soilcrete columns as shown in figures 16.11.1-1 and 16.11.1-2. As with other types of cement use, pozzolan substitutes can also be used to replace cement and alter the properties of the in situ soilcrete. In addition, bentonite can be added to create a lower permeability wall.

Figure 16.11.1-1. Deep soil mixing auger system (Jackson Lake Dam, Wyoming).

Another technique is to introduce dry cement (dry method) into the soil if it contains sufficient water to react with the cement. In this case, compressed air is used to convey the cement to the subsurface soil. Basic DSM treatment patterns consist of individual columns, linear rows of overlapping columns, or a grid-type pattern of soilcrete columns. Any of these types of typical patterns can be incorporated to create improvements to embankment dams for seepage barriers, foundation improvement, and seismic reinforcement. A typical cutoff wall could either be constructed of a single linear wall of overlapping soilcrete columns, or a block of two or more rows of overlapping columns, to create a wider wall.

Figure 16.11.1-2. Deep soil mixing (multiple augers).

Modern DSM equipment generally consists of multiple rotating, overlapping augers, or cutters, supported by a single frame and operated through a Kelly bar system (figure 16.11.1-2). This allows for high productivity rates and good vertical and lateral accuracy of the installed soilcrete columns and cutoff wall. DSM is most effective in finer-grained soils with no plasticity such as silts (ML), silty sands (SM), silty sands with gravels (SM)g, silty gravels (GM), etc. However, DSM has been used successfully with clayey soils for certain applications. In addition, the use of DSM to create a seepage cutoff is less likely to be required in a clayey, plastic soil, where permeabilities should already be low. Due to the overlapping augers, the efficiency of the system is going to be affected by coarse soils containing cobbles and boulders, and will produce a less effective mix.

A typical production sequence would consist of slow rotation of the augers into the ground (~10 to 20 rpm) and advancing at a rate of 2 to 6 feet per minute (ft/min). During the rotation and advancement, cement slurry is injected into the soil through the hollow drill stem feeding out of the tip of the auger. Mixing paddles are aligned along the shaft and above the auger to blend and mix the cement grout with the soil. The cement slurry also acts as a lubricant and a conveyance fluid for spoils coming to the surface. These spoils are similar to the soilcrete mix being generated in the subsurface. Once the final depth of the column is reached, the mixing is continued for a short time at the bottom of the hole (up to 2 minutes typically). The entire auger system is then slowly raised at approximately twice the rate of the downward mixing, and cement slurry pumping is continued at a reduced rate until the top of the hole is reached.

Another DSM system that uses newer technology consists of a cutter wheel assembly that cuts and mixes a soilcrete rectangular panel, rather than overlapping soilcrete columns mixed by augers (figure 16.11.1-3). In effect, this is a hybrid methodology of the concrete cutoff wall panel construction, which ordinarily uses slurry support, and the overlapping auger system. The advantage of the cutter wheel system is that it is capable of going through stiffer soils containing oversize particles (up to 8 inches), and the cutter wheels can be equipped to cut into rock up to 5,000-lb/in^2 unconfined compressive strength to form a key. It also has the advantage of the DSM method of mixing the soilcrete in place and eliminating the need for slurry supported panels and tremie backfilling of the panel. Both methodologies construct the cutoff wall in a primary/secondary panel geometry similar to concrete cutoff walls. Figure 16.11.1-4 shows a typical construction sequence using an overlapping triple auger system.

Figure 16.11.1-3. Deep soil mixing.

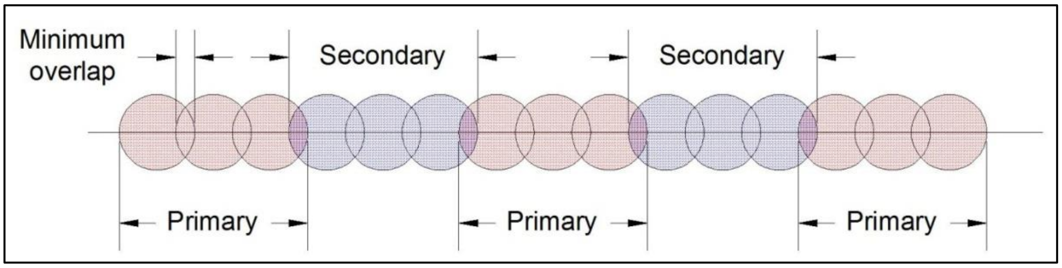

Figure 16.11.1-4. Typical construction sequence of primary and secondary columns for a DSM cutoff wall using an overlapping triple auger system.

16.11.2 Design

When considering the use of DSM for a cutoff wall, the primary considerations will be the permeability of the finished wall, soilcrete homogeneity, and the effective width of the wall required to reduce the seepage gradient. As with most seepage cutoff walls, the effectiveness of the wall will be optimized if it can be keyed into a low permeability soil or rock at its base. The DSM auger system will have some limitations in its ability to create a key in strong, hard soils or rock, which should be considered in the design process if such a key is required. The strength of the soilcrete created by the DSM process will be a secondary design aspect when the primary use will be for seepage control. Similarly to the discussions concerning very stiff concrete walls in Section 16.7.11, "Concrete Mix," a more deformable, less stiff cutoff may be desirable. The depth of the cutoff wall is another consideration due to limitations of the DSM equipment. DSM walls have been constructed to depths greater than 100 feet, and as the technology improves, the capability of greater depths is likely to increase.

The use of the DSM methodology is not well suited to construction of a wall within an existing embankment and is not generally recommended for this purpose. The auger system will naturally disrupt the integrity of the embankment soil around the perimeter of the augers and could leave zones of unmixed and disturbed soil within an existing embankment. Another design consideration for DSM is that it works best when the soilcrete column is to be mixed in place from the bottom elevation of the hole to the ground surface. Because the auger drilling system initiates at the ground surface, progressing downward, the system is not well suited to modification of an isolated zone at depth. Some disturbance of in situ soils between the ground surface and the top of the zone to be treated will occur during the drilling process and extraction of the auger system.

Key components for the design of a successful DSM cutoff wall are a comprehensive understanding of subsurface geology, identification of soil types, in situ soil moisture contents, water table elevation, and retrieval of soil samples for laboratory mix design testing. As with other proprietary ground improvement methods, it is incumbent upon the designer to work closely with the contractor so that the contractor can make the necessary adjustments to achieve the desired soil properties in the field. For best results and to minimize risks, it is recommended that a test section be constructed to evaluate the various parameters and optimize the installation variables required to achieve the desired results. As with other in situ ground modification techniques, much of the quality control/quality assurance (QA/QC) will depend on core drilling of the DSM columns to retrieve samples for laboratory testing and to confirm the continuity and homogeneity of the installation. In addition, the designer must be aware that even with sound, good quality drilling and sample recovery, as well as laboratory testing, some portion of the quality assurance will always depend on a subjective interpretation and evaluation of the results.

A. **Strength.** The strength of the in situ soilcrete created by the mixing process depends on a number of factors including soil type, volume of cement added per unit volume of soil, water-cement ratio, pozzolan substitutes, bentonite addition, and size of soil particles in the soil matrix. Unconfined compressive strengths of 600 lb/in^2 and greater can be achieved with the right soil, water, and mixing conditions and strength; and, in general, they will range between 1/5 and 1/10 of normal concrete unconfined compressive strengths. Strengths in the higher range are generally associated with water-cement ratios less than 1.0 (in the range of 0.7) and cement contents of at least 500 lb/yd^3 of soil. Lower strengths can be achieved with water cement ratios of 1.0 to 1.5. Thorough laboratory mix design testing using the in situ soils, and a subsequent test section, are the best ways to understand the actual strengths achievable at a given site and reduce the risk of construction issues during production. Evaluation of uniformity, strength, and permeability of the in situ mix will involve careful core drilling and sampling and extrapolation of the results to the entire wall.

   Modulus values for DSM soilcrete will be much less than that of traditional concrete and have modulus values similar to that obtained for SCB mixes. Modulus values will vary considerably with the mix design and will generally range from 1/5 to 1/10 of that used for normal concrete. These values should be determined when performing mix designs in the laboratory.

B. **Permeability.** Permeability values of soilcrete from DSM can be quite variable and should be judged by mix design testing in the laboratory to gain the best understanding. Laboratory samples should be tested using the test methodology in ASTM D 5084-03, "Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter." The designer must keep in mind that laboratory results will define the permeability of a laboratory sample. The actual in situ permeability depends on the uniformity and quality of the mixing achieved by the contractor's equipment and mixing technology. Permeability is generally similar to that of soil cement and in a similar range of permeabilities of cutoff walls created by other slurry wall techniques (in the range of 1 x 10-5 to 1 x 10-7 cm/sec) depending on soil type, volume of cement and/or bentonite, and quality of mixing. The quality and homogeneity of soil mixing depend on many factors. These include soil homogeneity, plasticity, groundwater conditions, density, gradation, etc. The variability of natural soils and the actual performance of the DSM equipment make any assumption of final in situ permeability difficult to predict. The actual permeability may vary spatially, both horizontally and vertically, which tends to decrease the certainty in estimating the overall permeability of the cutoff wall as a whole. Each wall is somewhat unique, and the quality of mixing and in situ permeability should be confirmed by a test section, if possible, combined with sampling of the in situ soils and laboratory testing of those samples. Unlike some cutoff wall backfill materials in which the in situ permeability can be reasonably estimated using laboratory mixed samples, DSM soilcrete permeability must be estimated from laboratory testing of cored samples to obtain the best results. This is due to the difficulty of predicting, before mixing, what the final ratios of the mix components will be in the actual soilcrete.

16.11.3 Constructability

DSM cutoff walls have been successfully installed throughout many parts of the world and have proven to be a reliable technology. The successful installation depends on continuity and uniformity to achieve the design intent of seepage reduction. Therefore, quality control is critical to ensuring that the soilcrete columns created by the technology are continuous, uniformly mixed, and intersecting. The use of guide trenches or ground surface templates can help ensure lateral continuity. Tight vertical tolerances, verifiable in the field with the use of equipment instrumentation, are essential in achieving vertical continuity and alignment. However, the best assessment of constructability is sound laboratory testing of mix designs and construction of a test section. For seepage cutoff walls, the method works best in noncohesive granular soils with limited oversize cobbles and boulders that could bind the augers or deflect the drill string. Zones of highly plastic clay can also be problematic because they will not mix uniformly with the cement, which can result in unmixed soil inclusions.
DSM equipment is capable of creating a fully penetrating cutoff into soft rock, weathered rock, and harder rock, depending on the type of equipment and tip design of the auger system. The primary limitations on the use of DSM technology are depth and the DSM equipment's ability to maintain alignment and verticality. DSM equipment has limited capability to mix isolated zones because the auger system injects volume (grout) into the soil, and the soil/grout mixture moves up the auger column during auger rotation. The possibility of non-uniform mixing is also a concern as this could leave pervious windows in the wall that may not be discovered in core drilling.

Quality assurance and control of DSM installations relies primarily on core drilling to verify continuity, uniformity, and to obtain samples for laboratory testing of strength and permeability. In some cases, wet samples of the soilcrete mixture can be obtained at depth while mixing is occurring; however, this method has had limited success. It also does not provide data on continuity and uniformity the way continuous coring can. Therefore, the constructability of DSM relies heavily on the ability of the contractor or client to perform good quality core drilling of the DSM columns, whether it be during construction of a test section or during the production work. The designer should take this matter into account when considering this method.

16.11.4 Required Submittals

The use DSM technology to construct a cutoff wall is a proven method for certain applications and can be cost effective. As with any cutoff wall, the successful performance depends on vertical continuity of the joints. For such specialty work, the designer should consider a negotiated procurement process that allows technical proposals to be submitted for review and consideration prior to award. Either during the proposal process or prior to beginning construction, after award, the specifications should require a comprehensive list of submittals. In addition to the suggested submittal requirements cited in Section 16.7.14, “Submittal Requirements,” for concrete cutoff walls, additional submittals should be considered that are specific to DSM cutoff walls. These may include:

Qualifications:

- Onsite supervisor resume
- Previous jobs of similar size and complexity

DSM Cutoff Wall Plan:

- Bar chart construction sequence drawing showing dates of anticipated cutoff wall construction and completion.

- Anticipated production rates, in linear feet of column per day or square foot of wall per day.

- Methods for ensuring alignment, depth, verticality, and overlap.

- Description of proposed drilling equipment and sequence of construction, whether linearly continuous or constructed in primary and secondary panels. Include DSM column diameters, width at overlapping intersection of columns, and any concerns with drilling through the specific materials at the site. Include a description of proposed method to correct or replace DSM columns that are deemed out of alignment tolerance or specified overlap requirements.

- Describe wet sampling methods.

- Plan to inject cement. Dry or wet method, rate of injection, injection pressure, including recommended weight of cement per cubic yard of mixed soil.

- Include a description of any planned additives or pozzolan substitutes.

- Describe cement mixing and batch plant and conveyance methods to point of injection.

- Identify source of water for cement grout and water chemistry test results.

16.12 Jet Grout Cutoff Walls

Jet grouting may have an advantage over some other alternatives where a laterally continuous, but vertically isolated, permeable zone is present in a foundation. The jet grouting method has the capability to create a soilcrete cutoff within isolated zones, which can result is potential cost savings compared with other methodologies. Very effective seepage barriers have been constructed at many sites throughout the world using the jet grouting method. However, jet grouting has significant limitations and concerns when used near an embankment dam due to the possibility for ground fracturing beyond the jet grout column. This effect has been noted at numerous sites when jet grouting was used. Therefore, this method should not be used within an existing embankment, and careful evaluation should be applied when considering its use within the foundation near an existing embankment. It should only be considered a possible alternative beneath an existing embankment, with careful evaluation for ground fracturing concerns, as well as acknowledgement that this method requires drilling a large-diameter hole for each installed jet grout column. The large-diameter hole is necessary to allow for spoil return (mixed soil, air, water, and cement grout) during the grouting process. The spoil travels up the annulus space between the drill stem and hole during the jet grouting process to prevent pressurization of the hole due to the high pressure and volume of injected fluids at the point of subsurface jetting. When completed, this leaves a column of soilcrete from the top of the column to the surface that would be unacceptable within an existing embankment under almost any condition due to the disturbed soil. The method could be appropriate if a foundation jet grout cutoff is constructed prior to constructing an overlying embankment, and if the appropriate defensive design features for seepage collection and filtering of seepage can be incorporated.

Based on the above considerations, there are significant limitations to using jet grouting for a seepage cutoff for an embankment. However, it is included in this standard because there may be applications where a jet grout cutoff would be economically and technically feasible.

16.12.1 General Description

Jet grouting is an in situ soil mixing technology that uses cement-grout and/or water and/or air injected under high pressure within the subsurface to create a column of soilcrete. The grout, air, and/or water used in jet grouting are synonymously referred to as fluids or phases. It is essentially a hydrodynamic mix-in-place technology unlike traditional rock grouting. The technology used in jet grouting has been available since the 1970s in Japan and Europe. The technique is now used around the world, and like many ground modification methodologies, each jet grouting contractor uses their own proprietary equipment, nozzle design, grout mixes, and injection parameters to achieve an acceptable soilcrete product that meets the designer's specifications. The section of the jet grout drill string that contains the injection nozzles and drill bit is referred to as the monitor. Because of the proprietary nature of many of the tools and operational mechanics used in jet grouting, the following guidelines are generally less specific than for other cutoff wall construction methodologies.

Jet grouting is usually performed to create round soilcrete columns within the subsoil. However, when diametrically opposed nozzles are used, cutoff walls can be constructed by slowly rotating the monitor back and forth (without full rotation), while raising the monitor and creating a panel that is shaped like a bow tie when viewed from above. By using the primary and secondary panel construction method, a linearly continuous wall can be created (figure 16.12.1-1).

In the performance of jet grouting, three types of grout injection systems are used: (1) single phase, (2) double phase, and (3) triple phase. Sometimes, the term "fluid" is used synonymously with the term "phase." These systems vary in how they utilize cement grout, water, and/or air to construct a soilcrete column. Some systems may use a single nozzle, while other systems may use two nozzles diametrically (and sometimes vertically) opposed to each other on the monitor. Triple fluid systems use nozzles separated vertically on the monitor. Each system is generically shown in figure 16.12.1-2. Single phase systems use only cement grout, injected under high pressure, to erode the surrounding soil and mix the cement grout with the soil to create soilcrete. A double phase system uses both air and cement grout. In this system, cement grout is injected under high pressure, while a ring of compressed air is simultaneously injected from a concentric ring nozzle surrounding the cement grout nozzle. The compressed air surrounds, or shrouds, the cement grout stream, reducing friction and confining the cement grout stream such that soil erosion is more efficient and can occur at larger distances from the nozzle.

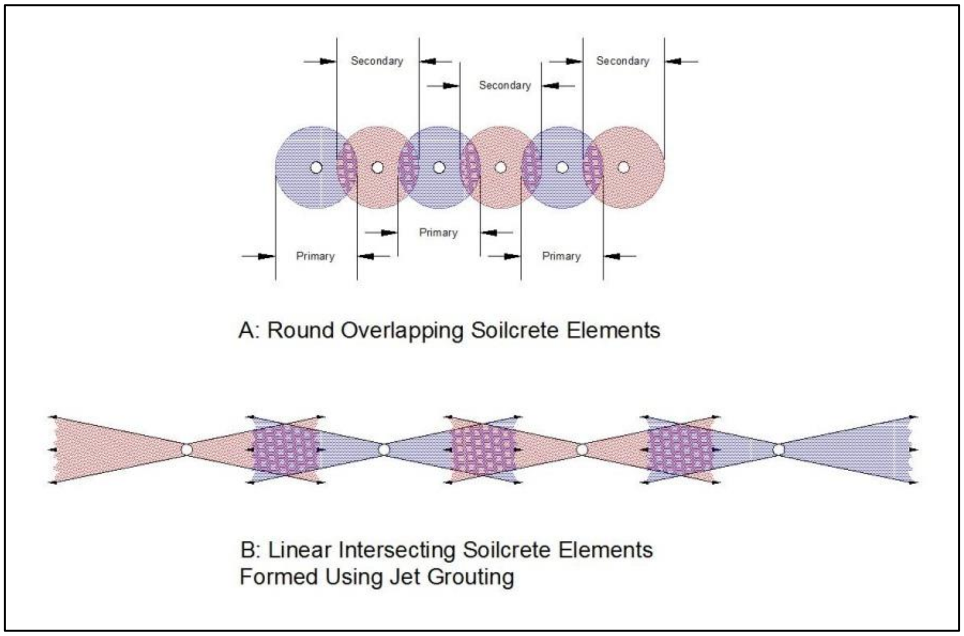

Figure 16.12.1-1. Types of jet grout cutoff walls – general plan view.

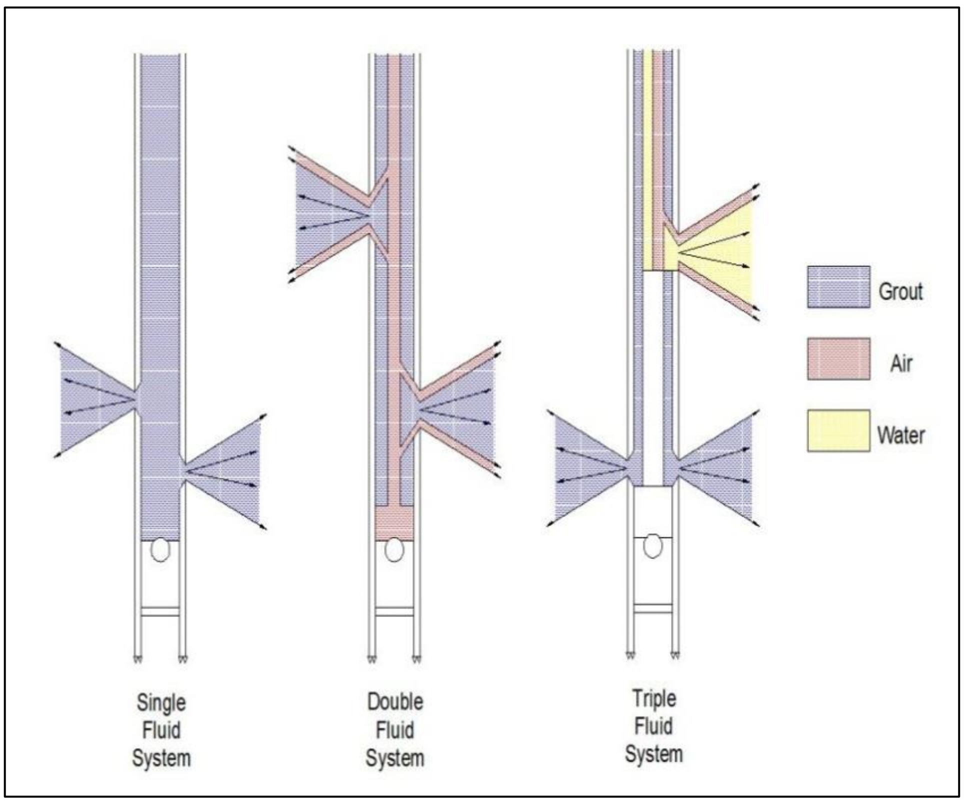

Figure 16.12.1-2. Schematic of general jet grout systems.

Triple phase jet grouting systems use cement grout, air, and water to create soilcrete. Generally, on the monitor of a triple phase grout system, a nozzle injecting water under high pressure, surrounded by a concentric nozzle of high pressure air, is located vertically above a nozzle to inject cement grout. Since jet grouting is performed from the bottom of the hole upward, the water/air nozzles serve to precut and destroy the soil matrix, producing an air-lifting effect, which evacuates much of the soil cuttings. The cement grout injected at the lower nozzle then serves almost as a soil replacement within the column and can be injected at lower pressures than typically utilized for single and double phase systems. In some cases with triple fluid grouting, polymer drilling fluids are combined with the water to help suspend the cuttings to lift them to the surface. At the time of this writing, there have been some concerns about the effect of these polymers on the cement grout and whether they can negatively affect its strength. This particular concern arose during the post-construction evaluation of the jet grouting test section at MIAD in California. As a result of this issue, and other concerns about the effectiveness of jet grouting performance in the dredged mine tailings in the downstream foundation at MIAD, jet grouting was eliminated as a foundation modification alternative. Additional study is taking place in this area, and the designer should consider this factor and be aware of the current state-of-the-practice when proposing the use of triple fluid jet grouting.

At all times during the jet grouting process, spoil material consisting of soil, cement grout, water, and air flows up within the annulus space between the drill stem and the drilled grout hole, and to the ground surface, where it is channeled into a trench and into a waste pit under gravity flow. At locations where soils may be contaminated or space is highly limited, the jet grout spoil may be collected by other means, such as pumping it into a temporary storage tank for disposal. The spoil is left to set overnight in the waste pit, in most cases, and then it is excavated and removed to a suitable disposal site. The spoil develops into a semi-solid consistency overnight, which makes it easy to excavate and mix with in situ soils or waste if necessary. In many cases, the soilcrete spoil is buried in a nearby waste area. However, in some cases, the spoil may be usable as fill. This was the case at Reclamation’s Wickiup Dam, where very large quantities of spoil were created by using jet grouting for seismic remediation. After sitting overnight in the waste pit, the jet grout spoil was hauled to a borrow area, where it was disked and blended with silty sand and clayey sand at a 1:1 ratio and then used as miscellaneous fill for a downstream berm that was part of the embankment modification.

Due to the variety of components that can be injected during jet grouting and differing injection pressures, and the in situ mixing required, the number of pieces of equipment needed to perform the process increases in complexity with each additional fluid (air, cement grout, and water). Both the tooling and design of the nozzles is proprietary. Double-walled and triple-walled piping is used for double phase and triple phase jet grouting, respectively, to transmit the fluids to the nozzles. In addition, the use of high pressures for the various fluids requires equipment capable of producing those pressures, as well as a high degree of safety against failure. Although injection pressures for the different fluids vary considerably due to proprietary factors, soil type, jet grout system, and column diameter, table 16.12.1-1 below lists general ranges that can provide a guide for illustration; however, it should not be considered absolute. Each jet-grout contractor sets their own injection pressures based on their experience, site-specific conditions, and requirements of the client for the finished product.

Table 16.12.1-1. General Fluid Pressures Used in Jet Grouting

Jet grout system	Fluid component injection pressure (lb/in^2)		
	Air	Cement grout	Water
Single phase	N/A	4,000 – >10,000	N/A
Double phase	100 – >200	4,000 – >10,000	N/A
Triple phase	100 – >200	700 – >1,500	4,000 – >10,000

A key restriction with the use of jet grouting, which the designer should be aware of, is the limitations of jet grouting near the ground surface. This construction method cannot be performed from the bottom of the proposed cutoff wall to the ground surface due to the high injection pressures and the danger to persons nearby if the high pressure jetting gets too near the ground surface. The method can work very well for confined pervious zones and at depth below the ground surface. If the cutoff wall needs to be extended to the ground surface, the designer will need to incorporate other components into the upper part of the cutoff wall, near the ground surface. This is, in fact, how the jet grouting method has been used in many cases.

Another consideration for jet grouting is the depth of the cutoff wall. Because jet grouting is performed at the end of a drill string, the depths that can be achieved are significant. However, confining stresses at large depths will impact the jet grouting parameters necessary to create a particular design geometry. Jet grouting has been performed successfully at depths exceeding 100 feet; therefore, it has the potential to be applicable to many sites.

16.12.2 Design

Due to the proprietary nature of jet grouting equipment and grouting parameters, in most cases, the designer (or client) will primarily determine the required geometry including width, length, and depth and desired finished permeability and/or strength properties. It is also critical to have sufficient field exploration, sampling, and laboratory testing of the soils to be jet grouted in order to obtain a thorough knowledge of the soil properties and groundwater conditions. This includes the soil-type variability, both laterally and vertically, of the zone to be jet grouted and the presence of oversize particles such as gravel, cobbles, and boulders. Rock sizes larger than gravel can cause a shadow effect that prevents grout from traveling beyond the particle. If the wall is to be keyed into an impervious layer, such as clay or rock, the physical properties of the impervious zone should be identified, including gradation, plasticity, inplace density, and shear strength. If the cutoff wall is to be keyed into bedrock, important properties to be determined are the rock type, hardness, weathering profile, and surface elevation profile.

After these parameters have been defined, it is normally the responsibility of the jet grout contractor to select the most suitable jet-grout method and design the grouting parameters to achieve the designer's cutoff wall dimensions, uniformity, and physical property requirements. The jet-grout contractor will determine the most suitable water-cement ratio for the grout mix and will establish the jetting parameters including injection pressures, water-cement ratio, flow rates, injection volume (for all fluids used including air), monitor rotation speed, monitor lift rate, and expected column diameter and spacing needed to achieve overlap. The parameters selected by the contractor will be based on the jet-grout contractor's experience and equipment.

A. **Test Section.** A test section for jet grouting is almost an essential component for any cutoff wall design and should always be considered. Without the benefit of a test section, the designer is risking time and money. Jet grouting effectiveness can be sensitive to certain changes in the subsurface soils such as soil plasticity, density, and the presence of cobbles and boulders. Therefore, achievement of a uniform, impermeable cutoff wall relies heavily on the experience of the contractor. However, the contractor will appreciate the ability to perform a test section and gain an understanding of the site-specific conditions and jet grouting challenges including evaluation of different water-cement ratios and jetting parameters. The test section can be used to evaluate different column spacing, lift rates, rotation speeds, and jetting pressures to determine the most efficient geometry and jetting parameters. This allows these adjustments to be made prior to the start of production work. The test section also allows the client time to core the soilcrete and evaluate the effectiveness of the jet grout parameters being used in relation to the specified target properties and geometry. The collection of wet grab samples from the spoil stream during jet grouting of the test section also provides the first opportunity to form molds of the fresh soilcrete for curing and testing.

   As described, coring of the in situ soilcrete will be required, as a minimum, to evaluate the effectiveness of the treatment. Coring of the soilcrete columns in the ground should not be started until the soilcrete has had an opportunity to set and reach a UCS of approximately

50 lb/in^2 to allow good quality coring. In addition to coring, three-dimensional (3-D) subsurface seismic tomography has been used at some test sections to develop a 3-D image of the grout in the subsurface. This method was used at Reclamation's Wickiup Dam with limited success, but this information has been useful at other sites. This method also requires open cased drill holes surrounding the area to use for cross-hole shear wave measurements and the introduction of a seismic source. Measurements are made of shear waves passing through the soil and soilcrete structure created by the jet grouting process. The velocity of the s waves depends on the rheology of the material (density and elasticity), and these data can theoretically be used to construct a 3-D image of the subsurface soilcrete elements. If this methodology is used, the designer needs to work closely with the geophysicist who will perform the work to get the additional drill holes installed and in the right locations. At some jet grout test sections, the soilcrete columns have been located within a test area that allows excavation around the columns to expose them. This is one of the best ways to evaluate jet grouting because it allows visual observation of the geometry and direct sampling of the in situ soilcrete. The soilcrete columns can also be excavated or cut in order to view them in cross section, which provides excellent visualization and verification of effectiveness. A good partnership between the client and the jet grout contractor is very essential at the test section stage. It provides an opportunity for the client to share the information gained from investigations of the test section and to work with the contractor to make the necessary adjustments to the jet grouting parameters and /or physical dimensions of the soilcrete column layout.

The test section can determine if jet grouting is feasible at a given site and provides an opportunity to adjust parameters before production work begins, which reduces the risk of cost overruns or schedule delays.

B. **Strength.** The strength of the in situ soilcrete created by the jet grouting process depends on a number of factors including soil type, volume of cement added per unit volume of soil (water/cement ratio, lift rate, and rotation speed), water-cement ratio, pozzolan substitutes, monitor and nozzle design, and the size of soil particles in the soil matrix. UCS similar to concrete can be achieved under the right soil, water, and mixing conditions. Higher strengths are possible with the use of jet grouting, compared to some other forms of in situ mixing, due to the partial and near full replacement of the soil matrix with cement grout and mixed soil. The in situ UCS achievable with jet grouting spans a very broad range and can be influenced by the addition of common pozzolan substitutes such as ground granulated blast furnace slag. At Reclamation's Wickiup Dam in Oregon, large blocks of soilcrete were created in generally nonplastic silty materials for seismic strengthening of the foundation using a double fluid system [36]. The work was performed downstream of the dam at depths of 90 feet or more below the ground surface to approximately 20 feet below the ground surface. The specified target 28-day UCS was 200 lb/in^2, and the cores extracted from the in situ soilcrete columns averaged a 28-day UCS of approximately 600 lb/in^2. These strengths were achieved using a water-cement grout varying from 1.15:1 to 1.25:1. In these specific soils, using the proprietary double fluid system of the contractor, column diameters of 15 to 16 feet were achieved. However, the designer must keep in mind that the strength and column geometry achieved at any given site will depend on many other factors (cited above) other than just the water-cement ratio of the injected grout.

Modulus values for jet-grouted soilcrete also vary considerably. Although bentonite is not commonly used in jet grouting, there are applications where it has been part of the injected grout mix. Similar to the effect it has on other cement mixes, the use of bentonite tends to reduce the modulus value and increase the strain to failure. Some research in jet grouting has established empirical relationships between the UCS of jet grouted soilcrete and the secant modulus [34]. Although the data is scattered, the general trend is one of increasing secant modulus with increasing UCS. The relationship varies depending on the soil type (clay or sand) and water-cement ratio used.

C. **Permeability.** The in situ permeability of the jet grout soilcrete resulting from jet grouting can be very low. However, it is difficult to predict the resultant permeability without construction of a test section to evaluate different water-cement ratios and jetting parameters. Unlike some cutoff wall backfill materials in which the in situ permeability can be reasonably estimated using laboratory mixed samples, jet grouting permeability must be estimated from laboratory testing of cored samples to obtain the best results. This is due to the difficulty of predicting, before jet grouting, what the final ratios of the jet grout mix components will be in the actual soilcrete.

16.12.3 Constructability

Jet grouted cutoff walls have been successfully installed throughout many parts of the world and have proven to be a reliable technology. However, their use as cutoff walls for embankment dams is very limited. The primary reason for this is the concern over the use of jet grouting within an existing embankment and/or foundation and the likelihood of fracture occurring from the use of high pressure fluids and/or air. When considered as a foundation seepage cutoff alternative within a new embankment, it is likely that other types of seepage cutoff alternatives would be preferable due to easier (and possibly lower cost) quality assurance and more data on long-term historical performance or reliability.

One major constructability issue associated with jet grouting is ensuring the uniformity of the column diameter. When considering the use of jet grouting as a seepage cutoff within an embankment foundation, the cutoff wall is likely to be constructed within alluvium or fluvio-lacustrine soils. The soils are generally heterogeneous and, in the case of lacustrine deposits, may contain horizontally bedded layers with various stiffness. Jet grouting is typically performed using a constant lift rate and rotation speed on the monitor for one entire column. Therefore, when passing through the soils, the effective erosion radius, away from the monitor, will change with material stiffness, plasticity, and the presence of coarse particles. This invariably results in necking and uneven soilcrete diameters as the jetting nozzles move vertically upward. To achieve overlap of each column for a seepage cutoff, the center-to-center distance between jet grout columns must be conservatively designed (based on the estimated column diameter achievable with a particular system and set of foundation conditions) to ensure that each column intersects the adjacent column on either side. Otherwise, the risk of an ungrouted window increases. Such windows can be remediated by localized remediation grouting, as long as they can all be identified. The risk is that the quality assurance core drilling will not be extensive enough to identify every ungrouted window. Often, the denser soils, or more plastic soils that cause necking to occur, are not the critical soil layers to be jet grouted. In such cases, some necking of the columns and/or the presence of nonoverlapping adjacent columns may be acceptable. This was the case with very dense silt layers at Wickiup Dam [36], which were sandwiched between very loose diatomaceous silts. Necking in the dense silts was predicted and observed but did not require remediation due to the already dense state of this zone. The designer must be aware that this condition may occur and plan accordingly.

The successful installation of a jet grouted cutoff wall depends on continuity and uniformity to achieve seepage reduction. Therefore, quality control is critical to ensuring that the soilcrete columns created by the technology are continuous, uniformly mixed, and intersecting. Based on the results of a jet grouting test section, the spacing of the jet grout columns and subsequent minimum overlap can be determined for the production columns. The location of the center of each column is then staked in the field by survey. Tight vertical tolerances for each soilcrete column are verifiable in the field with the use of equipment instrumentation placed on the drill string and boom system. For seepage cutoff walls, the method works best in noncohesive, fine-grained silty, sandy, and/or gravelly soils with little to no cobbles and boulders. Jet grouting coarse soils can be successful, but it can be more problematic because the particle size increases due to “shadowing” effects of cobbles and boulders, as well as the possibility of groundwater washing away the grout if large volumes of seepage are present within high permeability layers. The limited success of jet grouting in coarse soils containing cobbles and boulders can be a negative factor when considering its use in these soils. Coarse soils can be significant contributors to risk from seepage-related failure modes, and jet grouting may not provide the level of confidence in the risk reduction being sought.

Jet grouting equipment is capable of creating a fully penetrating cutoff keyed into dense soils, clays, soft rock, and weathered rock, depending on the type of equipment and jetting parameters. Within these materials, the full diameter will not be achievable, and considerable necking (i.e., column diameter reduction) will occur. A primary advantage of jet grouting is that isolated zones can be treated. This reduces waste and overall cost.

Jet grouted cutoff walls are typically created using primary, secondary, tertiary, or more jet grout elements (i.e., skipping adjacent columns in a defined pattern) to avoid jet grouting a new soilcrete column adjacent to another newly installed column and eroding it. Figure 16.12.1-1 shows the sequence of constructing circular and linear jet grout cutoff walls using primary and secondary elements. The sequencing will depend on the anticipated soilcrete strength and should be responsibility of the jet grouting contractor.

QA/QC of jet grouted cutoff wall installations relies mostly on core drilling to verify continuity, uniformity, and to obtain samples for laboratory testing of strength and permeability. The collection of wet grab samples of the spoil stream exiting the ground surface around the drill string during jetting is typically the first opportunity to gain actual laboratory data of soilcrete properties. This will not provide data on continuity and uniformity of the in situ soilcrete, which is why continuous coring must be performed.

A secondary issue to be considered with jet grouting is the disturbance and contamination of the ground surface that occur during grouting. After completion of the jet grouting, the area must be fully stripped to remove all contaminated and disturbed material. If an overlying embankment or structure is to be placed, the foundation surface will have to be treated as required for the specific structure. Refer to *Design Standards No. 13 – Embankment Dams*, Chapter 3, “Foundation Surface Treatment,” for further guidance.

If a full seepage cutoff to the surface is required, or the full depth of cutoff wall required exceeds the depths achievable by common excavation equipment, the designer might consider the use of another cutoff wall methodology in combination with jet grouting to create a composite cutoff wall. The sequence of construction then becomes a factor. In such a case, the jet grouted cutoff could be constructed first from the bottom depth upward, to a given depth below the ground surface, and verification completed. Then, a secondary seepage cutoff could be constructed from the ground surface to the top of the jet grouted soilcrete. Most of the seepage cutoff wall methodologies cited herein could be viable alternatives for this upper section of cutoff wall. Appraisal level designs and cost estimates will be required to determine if such a composite cutoff wall is economical.

16.12.4 Contracting and Submittals

Jet grouting, like other specialty work, should only be contracted using a negotiated procurement process that allows technical proposals to be submitted for review and consideration prior to award. In addition, a test section is strongly recommended and should be considered mandatory unless the job is small. The designer should design the geometry of the jet grouted seepage cutoff wall and the required physical properties of the in situ soilcrete. A thorough review of jet grouting literature and previous jet grouting projects can give the designer a general idea as to column diameters, cement contents, strengths, waste volume, and permeabilities that are possible at a given site for estimating purposes. The test section contract should specify the geometry and properties of the final cutoff wall and allow the contractors to propose the types of equipment, spacing, and grouting parameters they plan to use. It is also advantageous to allow variations in the spacing, jetting parameters, grout mix, etc., to evaluate different combinations of these variables. This will provide more information to fine-tune these variables such that the most efficient combination of parameters and column geometry can be determined for the production work to meet the goals of the project. The test section will require time to evaluate and study the results before finalization of the full-scale production work. The test section contract should always be issued as a separate contract and should not be linked to the production work. This will increase the cost for the test section, but it allows the designer the maximum flexibility to adjust the methodology or eliminate jet grouting as an alternative if the required final product is deficient or unable to be constructed, respectively.

Quality assurance and quality control can be monitored during construction of the test section and will typically involve observation of the contractor’s drilling, jet grouting, and waste handling procedures. Below the ground surface, core drilling is typically required to evaluate the quality of the in situ soilcrete. In certain cases, excavation to the jet grout columns can be made to expose the columns for observation and testing. With core drilling alone, good quality coring is essential. Also, it should be recognized by the designer that even with good quality core recovery, the overall assessment of mixing, uniformity, and in situ properties of the soilcrete will always be somewhat subjective.

Both the test section and production contract specifications should require a comprehensive list of submittals. Submittals for cement materials and pozzolan substitutes can be found in Section 16.6.1, “Submittal Requirements for Cement-Bentonite and Soil-Cement-Bentonite Cutoff Walls.” Suggested submittals for inclusion in the jet grouting contract should include as a minimum the following list.

Qualifications:

- Onsite jet grout supervisor resume
- Previous jobs of similar size and complexity

Jet Grout Cutoff Wall Plan:

- Bar chart construction sequence

- Jet grouting method:

    - Plant, equipment, jet grouting system, nozzle geometry, and material descriptions (i.e., grout, water, air, and additives)

    - Layout and arrangement of grout mixing and injection equipment

    - Planned drilling equipment and methodology (including suitability for drilling through site-specific difficult materials, if needed)

    - Grout mix design, material sources, and material data demonstrating compliance with the specifications

    - Column construction sequence, pattern, and schedule

    - Anticipated column diameter and spacing

    - Cement delivery, storage, and handling plan

    - Jet grout waste management plan

- Jet grouting records:

    The contractor should propose the procedures and methodologies for which records of the jet grouting will be obtained and provided for review. These records are provided through automated data acquisition equipment and should be provided at intervals no greater than 0.2 foot of column and include:

    - Identification of all equipment used per column
    - Time and date
    - Depth
    - Density of injected grout
    - Cumulative injected volume of grout

- Grout injection pressure
- Grout injection rate of flow
- Air injection pressure and rate of flow (if double fluid system is used)
- Water injection pressure and rate of flow (if triple fluid system is used)
- Lift rate and rotation speed
- Drilling time per hole
- Total grout injection time and total volume per column
- Total weight of cement injected per column
- Top and bottom elevation of each column
- Estimated volume of waste return for each column

16.13 Instrumentation of Cutoff Walls

Instrumentation needs for a cutoff wall installation will vary considerably. Primarily, piezometers located upstream and downstream from the wall will likely provide the best information as to the effectiveness of the seepage cutoff. The downstream instrumentation should have piezometer tips isolated in bedrock, permeable foundation material such as alluvium, and in the overlying embankment. If seepage is creating higher downstream water levels than anticipated, a thorough review of geologic data and drilling records, along with separate isolated instruments, may help determine if the problem is due to seepage through the wall joints or in fractured bedrock passing beneath the wall. Inclinometers and settlement gauges can be installed to monitor deformations in the wall. Instrumentation is critical to evaluation of the overall performance of a cutoff wall but the types and density of instrumentation will depend on available funding, design needs, and other factors too numerous to cite here in detail. However, in the design process, the designer, along with the design team, should anticipate and plan for sufficient instrumentation to evaluate performance. General guidance on instrumentation can be found in *Design Standards No. 13 - Embankment Dams*, Chapter 11, "Instrumentation and Monitoring."

16.14 Overall Chapter Review

This cutoff wall design summary chapter is meant to be a general guideline to assist the designer when selecting an appropriate seepage cutoff element for an embankment dam and/or foundation. The summary includes most types of cutoff walls that are commonly used in the industry for embankment dams and other geotechnical features. As much as possible, both advantages and disadvantages of each cutoff wall type and construction methodology are provided to aid the designer in the selection of the most technically effective, cost-efficient alternative. This standard is not meant to be a design manual that guides the designer step by step through the design process. Each cutoff wall site and application is unique, and it is imperative that the designer ensures that sufficient geologic/geotechnical and laboratory investigations are completed to develop a sound design. In addition, the equipment used to construct seepage cutoff walls is constantly evolving, improving, and changing. It is the responsibility of the designer to be aware of the state-of-the-art changes and improvements in technology. In many cases, the references included in this chapter of the design standards, and past Reclamation cutoff wall designs and specifications, can be of additional benefit to the designer. Communications with other designers, experts, consultants, and contractors that use a particular methodology can also provide useful information.

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16.16 Glossary of Terms

Biopolymer fluids	A polymer substance (as a protein or polysaccharide) formed in a biological system or a synthetic cellulosic derivative which combines an antiswelling agent, a biodegradation inhibitor, and a pH booster base. Biopolymers, when mixed with water, are capable of increasing the viscosity of water and can act as biodegradable slurry for trench support.
Blast furnace slag	A glassy, granular product produced by quenching molten iron slag (a byproduct of iron and steel making) from a blast furnace, in water or steam, which is then dried and ground into a fine powder.
Carboxymethyl cellulose (CMC)	The sodium salt of carboxymethyl cellulose, which is derived from cellulous (the chief constituent of the cell walls of plants) and made water soluble by introducing carboxymethyl groups along the cellulose chain and makes hydration of the molecule possible. It is used as a viscosity modifier.
Colloidal suspension	A homogeneous dispersion of clay particles, typically bentonite, in water.
Cutoff wall	Any formed seepage barrier within a dam embankment or foundation.
Deep-soil-mixing (DSM)	*See soil-mix-wall.*
Desander	Solid control equipment that separates sand and silt from slurry fluids using hydrocyclones.
Flocculation	The process in which colloids form larger size clusters, or flocs, through contact and electrical attraction but do not precipitate from the suspension.
Fly ahs	Ash produced during the combustion of coal, which contains substantial amounts of silicon dioxide (SiO_2) and calcium oxide (CaO).
Jet grouting	An in situ ground modification methodology in which cement-grout, air, and/or water are injected at high pressure by a rotating drill string (monitor) in order to mix, modify, or replace existing soils and create soil-cement elements.
Lignosulfonate	Water-soluble polymers that are byproducts from the production of wood pulp.

Pozzolan	A siliceous and/or aluminous material which, in itself, possesses little or no cementitious value but will, in finely divided form and in the presence of water, react chemically with calcium hydroxide to form compounds possessing cementitious properties.
Secant piles	Closely spaced bored piles in which the construction sequence involves boring primary piles with centers spaced less than two diameters apart, followed by in-filled secondary piles that overlap each primary pile, creating a continuous vertical barrier.
Slurry	A thin mixture of liquid, especially water, which includes the suspension of any of several finely divided substances such as clay, cement, or proprietary organic and inorganic substances.
Soil-mix-wall (SMW)	Mixed in-place vertical walls that typically use cement grout slurry injected into the in situ soil, usually through nozzles located within overlapping auger drill strings or rotating cutter wheel systems.