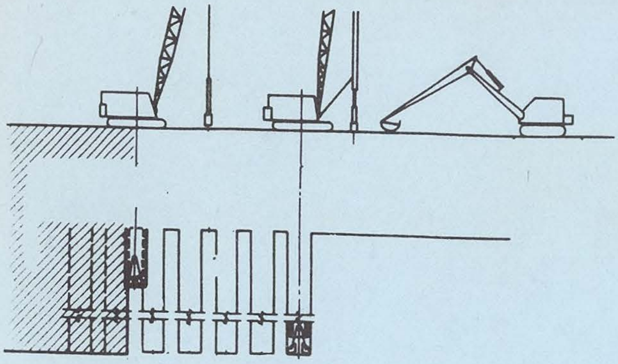
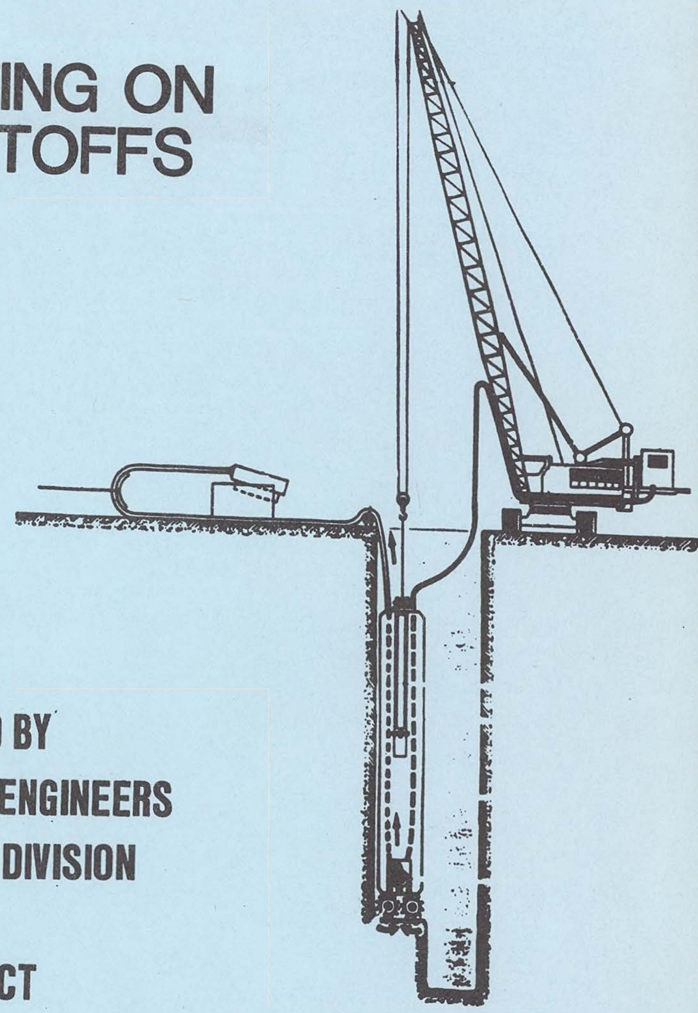
3 HOLDERS



GEOTECHNICAL MEETING ON SLURRY TRENCH CUTOFFS

OCTOBER, 1984
EUFAULA, AL.



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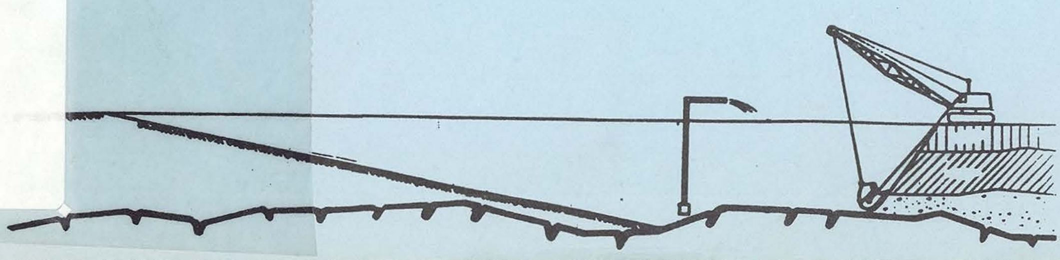
MOBILE DISTRICT

GEOTECHNICAL AND MATERIALS BRANCHES



GEOTECHNICAL MEETING ON SLURRY TRENCH CUTOFFS

INTRODUCTION. The South Atlantic Division (SAD) and Mobile District (SAM) hosted a seminar on the design, construction, and performance of slurry trench cutoffs with emphasis on projects within SAD. Seventeen presentations were made covering aspects of soil-bentonite cutoffs, concrete cutoffs, vibrating beam cutoffs, and cement-bentonite cutoffs. A field trip was also made to observe the installation of the concrete cutoff wall at the Walter F. George Dam. In addition to the presentations, draft guide specifications for soil-bentonite and concrete cutoffs were presented for comments. An agenda and list of attendees is provided in Appendix A.

In 1978, SAM hosted a similar seminar on design and construction of the soil-bentonite cutoffs for the dewatering of a number of projects on the Tennessee-Tombigbee Waterway. Since that time, a significant amount of data has been collected on the performance of soil-bentonite cutoffs and experience has been gained in the use of concrete cutoffs and vibrating beam techniques. The applications of the various types of cutoffs have also increased making cutoffs viable alternatives to the solution of many problems both during design of new projects and in the rehabilitation of existing projects. In order to disseminate information on cutoffs as widely as possible, this seminar was conducted. Details of design, construction, and performance as well as "lessons learned," of projects Corps-wide were presented. The presentations, keyed to the slides used, are included in Appendix B. The slides that accompany these papers are available on loan from SAD. Requests or inquiries should be addressed to SADEN-F.

SOIL-BENTONITE SLURRY TRENCH CUTOFFS. The first soil-bentonite slurry trench (SBST) constructed in the U.S. in connection with a major dam was in 1960 at the Wanapum Dam on the Columbia River. Since then, and in particular in the last 10 to 15 years, geotechnical engineers have become more aware of the advantages, success, and low cost of SBST cutoffs and their use has spread rapidly. In SAD, the first SBST constructed was at the West Point Dam in 1966 to control underseepage. It was 5 feet wide with a maximum depth of 60 feet. Since West Point, they have been utilized on thirteen additional projects in SAD for either construction dewatering or permanent seepage control. Some specific advantages of the SBST cutoffs which have been realized are:

a. They can be constructed in almost any type of ground situation.

b. Continuity of the cutoff is easy to obtain.

c. The soil-bentonite backfill can be made flexible to eliminate cracking due to ground movement.

d. Construction is relatively fast and inexpensive as compared to other alternatives.

In SAD, SBST cutoffs have proven to be very effective for construction dewatering when they have been tied into an impervious stratum such as was the case on the Tennessee-Tombigbee Waterway and Cooper River Projects. On only one of the ten projects which utilized SBST for dewatering did minor seepage occur and it was determined to be coming under the cutoff through lenses of sand. Some of the advantages of using SBST for dewatering in lieu of conventional systems such as well points are:

a. The elimination of operation and maintenance costs associated with normal dewatering systems.

b. Elimination of any risk associated with possible breakdown of a dewatering system or power outages.

c. The elimination of wells and headers around the perimeter of an excavation thus allowing freer access to and from the construction site.

On a number of the Tenn-Tom Waterway projects where SBST were constructed around the perimeter of the site for dewatering, SBST were also installed laterally from the structures out to the dewatering SBST for permanent seepage control. This greatly increased the seepage path around the structure thereby reducing the exit gradients into the river channel downstream to a safe level. To date, all the SBST cutoffs, both temporary for dewatering and permanent, which have been installed in SAD have performed satisfactorily. The performance of SBST cutoffs is highly dependent on the quality of the backfill, continuity of the cutoff, and an adequate tie-in into an underlying impervious stratum. The backfill used on projects in SAD has been controlled with a specified gradation with a maximum size of 3 inches and from 10 to 25 percent passing the No. 200 sieve. Experience on the Tenn-Tom Waterway projects indicates that a backfill with a high percentage of fines and an absence of sand and gravel materials is difficult to mix as well as place. The inclusion of sand and gravel increases productivity without compromising the overall effectiveness of the cutoff. Tests conducted on samples of the backfill and slurry from a number of projects indicate permeabilities in the order of 10-6 to 10-7 cm/sec. The continuity of SBST cutoff is generally easy to attain because of the method of excavation and backfill placement. It is important, however, that the slump of the backfill be maintained around 3 to 6 inches to insure proper placement into the trench. In regards to insuring that the cutoff is adequately keyed into the underlying impervious stratum, it is important to maintain accurate account of the depth, acquire adequate samples, and ensure proper cleaning of the bottom. Experience in SAD has indicated a tendency on behalf of some contractors to resist extensive cleaning because of its time consuming nature. However, the use of an air lift has proved the most satisfactory method of cleaning the trench bottom. The cleaning will reduce or eliminate potential seepage resulting from sand and gravel that was left from the excavation or that settled from the slurry.

During construction of SBST cutoffs, the stability of the trench and overall constructibility are also dependent on the properties of the bentonite slurry and the level of the slurry in the trench. The slurry properties which are normally monitored are viscosity, density, and filtrate loss. Typical values for these during construction are: a minimum viscosity of 40 seconds as measured by the marsh funnel, a density of 64 to 85 PCF, and a maximum filtrate loss of 20 cc in 30 minutes as measured by the standard filter press test. With respect to slurry level in the trench, it should always be maintained a minimum of 2 ft. above the water table and preferably within a few feet of the working surface.

Basic information on the SBST cutoffs constructed in SAD is summarized in Table 1. The cost information shown on the table is the average bid price of the three lowest bidders and the government estimate per square foot of wall. Probably the two most significant factors that affect the cost are the type of subsurface material the cutoff is excavated through and its depth. With the exceptions of the West Point Dam and the Cooper River Project, all of the SBST cutoffs constructed in SAD have been through unconsolidated alluvial flood plain material varying from clays and silts to sand and gravel. At West Point the subsurface materials consisted primarily of residual soils and at Cooper River dense coastal marine sediments. The most common excavating equipment which has been used to install the cutoffs in SAD is the backhoe. There have been significant improvements in the design of backhoes through the years that have increased their capability significantly. In 1974 when the first SBST cutoff was installed in the Tennessee-Tombigbee Waterway the limiting depth for backhoe excavation was approximately 42 ft. In 1981, the deepest trench in the waterway was 72 ft and was excavated with a Koehring Model 1266 backhoe with an extended boom. It is interesting to note by the comparison of projects with similar conditions that there has been little, if any, increase in unit costs per square foot. This can probably be attributed to the improvement in excavating equipment and to the fact that contractors have become more familiar with slurry trench construction which has increased interest among contractors.

CONCRETE CUTOFF WALLS. Concrete cutoff walls have been installed at Corps projects prior to construction to correct known foundation problems and after construction to correct seepage problems that developed after impoundment. At Bonneville 2nd Powerhouse concrete cutoff walls were used to cutoff seepage during dewatering of the excavation and then portions of the walls were used to form a permanent seepage cutoff for the completed powerhouse. Steel Creek Dam has incorporated a cutoff wall under the upstream toe of the dam to block deep foundation seepage anticipated during design. The St. Stephens Powerhouse cutoff walls were installed to control embankment seepage. The pool was drained to facilitate investigations and the walls were installed while there was no differential head. A similar sequence was followed at Mill Creek Dam in Walla Walla District. The remaining Corps projects discussed herein remained operational during installation of the cutoffs. At Wolf Creek, Walter F. George and Clemson Upper and Lower Diversion Dams the walls were

TABLE 1 SOIL-BENTONITE SLURRY TRENCHES

Project	District	Bid Date	Est SF Cutoff	Trench Width, FT	Trench Depth, FT	Purpose	Ave. Bid Price/SF **	Gov't Est. **	Equipment
West Point Dam	SAS	2/66	83,000	5	60	Seepage	3.39	2.56	Clamshell
Gainesville	SAM	12/73	30,000	5	30	Seepage	6.33	6.70	Clam/Backhoe
Aliceville L&D	SAM	1/74	239,600	3	30	Seepage/Dewater	4.21	4.10	Backhoe
Columbus L&D	SAM	2/75	211,900	3	30	Seepage/Dewater	4.30	3.76	Backhoe
Aberdeen L&D	SAM	10/76	176,000	3	25	Seepage/Dewater	6.04	4.00	Backhoe
Lock A	SAM	3/77	257,700	3-4	35-40	Seepage/Dewater	2.90	4.33	Backhoe
R.B.Russell Dam	SAS	10/77	15,000	3	75	Dewater	*		Clamshell
Lock B	SAM	12/77	218,800	3	30	Seepage/Dewater	3.33	2.07	Backhoe
Lock C	SAM	9/78	137,600	3	22	Seepage/Dewater	3.08	2.35	Backhoe
Cooper River	SAS/SAC	3/79	260,000	3	70-80	Dewater	15.67	9.07	Clam/Backhoe
Pool D	SAM	8/79	680,100	3	30	Seepage	2.82	3.00	Backhoe
E.Jackson Levee	SAM	9/79	161,500	3	30-35	Seepage	2.80	4.59	Backhoe
Lock E	SAM	2/80	290,000	3	50	Seepage/Dewater	4.52	3.45	Backhoe
Lock D	SAM	10/80	385,400	3	50	Seepage/Dewater	4.88	3.55	Backhoe

*Work done by amendment to dam contract, no bids.
**Bid prices and Gov't estimates have not been normalized for inflation. The average bid represents the average of the three low bidders.

installed while maintaining normal project pools. Except for Wolf Creek, all of the walls were the panel type which would require exposing small segments of the embankment and foundation at a time, thus reducing the risk of failure due to instability of the excavation. Due to the extreme depth of the wall, the foundation conditions, and the high heads involved, the cutoff wall at Wolf Creek was an element type wall which exposed even less foundation and embankment during excavation. Table 2 summarizes pertinent data for the cutoff walls at major Corps projects. The price of the walls varies considerably with the amount of excavation, the amount of rock involved, and the site conditions. Where competitive bids were obtained, the contracts have generally shown a decrease in cost. At Walter George the average price per square foot of wall dropped from $16.43 in the first contract to $8.69 in the final contract. Bonneville 2nd Powerhouse had similar success with a price drop from $40.00 per square foot of wall in the first contract to $20.00 in the final contract. While the trend has been toward lower unit prices for the concrete walls, this method cannot compare favorably with soil-bentonite cutoffs with regard to overall cost and speed of installation. Typically, the concrete cutoffs require more excavation equipment, a concrete batch plant and trucks, tremie equipment, shoulder pipes and equipment to remove the pipes and coring equipment to sample the completed panels. However, concrete cutoffs can reach greater depths, can cutoff seepage problems in rock, and can be installed with little or no interruption of normal project operations.

Design of a concrete cutoff wall is site specific since the project may involve rehabilitation of a dam without interrupting normal operations. At functioning projects a suitable work area has been obtained by degrading the embankment (Water George, Clemson, St. Stephen) or by building a work area platform (Wolf Creek). The work area has generally been the contractor's responsibility to design and maintain. At existing projects the contracts have specified how much the existing embankment could be degraded, but have left the detailed configuration to the contractor. At Walter George, the phase II contractor left a wedge in-place upstream so as not to disturb the rip-rap. This also provided pedestrian access to the entire work area away from the vehicular access as well as an area for mud and water lines out of the working area. The working areas are generally maintained as little as possible and typically are very sloppy due to the heavy traffic during the life of the contract. This could be alleviated during design by specifying not only the degrading of the embankment but also the surface preparation and treatment that will be required.

While most of the work appears routine, a considerable inspection effort is needed to insure the quality of the cutoff. While individual equipment production rates are low, the contractors maintain a high overall production rate by using more equipment. This makes it difficult to inspect since the work will be spread over a large area. Also, the concrete operations greatly increase the variables affecting construction and require continuous close inspection. Contamination, breaking a tremie seal, flowability of the concrete, panel joints, segregation, slump, extracting the shoulder pipes,

TABLE 2 CONCRETE CUTOFF WALLS

PROJECT	DISTRICT	BID DATE	ESTIMATED S.F. WALL	L.F. WALL	PANELS PRI	SEC	RUN	TTL	AVE. LENGTH	AVE. DEPTH	MAX DEPTH	CONTRACT COST/SF WALL*	CONTRACTOR
Bonneville 2nd Powerhouse P-I	NPP	6/74	47,000	360	4	3	13	20	18.0	131	145	40.00	ICOS
Bonneville 2nd Powerhouse P-II	NPP	3/75	211,000	1720	12	12	67	91	18.9	123	146	27.00	ICOS
Wolf Creek Phase I	ORN	6/75	189,500(O) 29,500(R)	1080	221	220		441	2.5	203	277	131.00	ICOS
Bonneville 2nd Powerhouse P-III-IV	NPP	7/76	479,000	3300	22	25	93	140	23.6	145	180	20.00	Bencor
Wolf Creek Phase II	ORN	6/77	211,000(O) 49,700(R)	1245	278	278		556	2.3	210	263	188.00	ICOS
Mill Creek	NPW	11/80	230,000	2260	48	48		96	23.5	102	180	27.98	Soletanche
Walter George Phase I	SAM	4/81	107,000(O) 32,500(R)	1244	31	9	32	72	17.3	112	137	16.43	Soletanche
Clemson Lower Diversion Dam	SAS	12/81	195,000	2560	16	15	78	109	23.5	80	90	17.97	Bencor
Walter George Phase II	SAM	1/83	664,000(O) 297,000(R)	8145	86	86	189	361	22.6	118	187	8.69	Bencor
Clemson Upper Diversion Dam	SAS	6/83	172,000	2264	44	43	3	90	25.2	76	95	12.67	Soletanche
St. Stephen Project	SAS SAC	6/84	78,650	694	19	17		36	19.3	113	117	23.37	Recosol (Soletanche)
Steel Creek Dam	SAS	9/84	210,000	2054	43	46	1	90	22.8	102	145	16.79	ICOS

*Costs shown are the total contract cost divided by the square foot of wall installed. These figures have not been normalized for inflation.

excessive bleeding, and premature set are some of the potential problem areas that require close inspection. Most of the contracts to date have required the use of a traveling plug or "go devil" at the start of the tremie concrete placement. Generally, the contractors have resisted this by ignoring the requirement or by using non-approved substitutes such as mortar balls and beach balls. The mortar balls tend to break apart and the beach balls will compress to 50-60% of the original size. Basketballs have been used on several jobs. While these will flatten under the high pressures at the base of the tremie pipe, they do recover their shape at the surface and are reuseable. Most important there is little evidence that the initial pour is much more severly contaminated using the basketball than when a non-compressable plug is used. Another area of contention on several jobs has been the requirement to use an airlift during the panel bottom clean-up and desanding operations. While this has proved the most effective way to clean the panel bottom, the contractors often resist using the airlift. However, it has been used with good success and is essential to assure a clean contact at the panel base free of sand and/or gravel which could cause a seepage window through the wall.

Test sections have been included on most jobs to evaluate the contractor's procedures in a non-critical area. To properly evaluate the in-place product, an extensive core boring program is needed which will require a built-in delay in the contract. This is expensive as the contractor is idle during this period and will have included these costs in his bid. Also, if any questionable quality concrete is found, the boring program will need to be expanded which can lead to disputes over liability for additional delays. It also requires considerable attention by the Government forces, both design and construction, to examine and evaluate the core and approve or reject the contractor's procedure. Close coordination between design and construction personnel is essential for completion of the evaluation with the allotted time period. In addition to this initial evaluation, a continuous quality assurance program is necessary to insure the integrity of the in-place concrete. This is accomplished by a continuing core boring program. For this to be of value cores must be taken routinely throughout the contract rather than in clusters. This must be specified in the contract or the contractor will not be obligated to keep a drill rig available. By coring more or less continuously during the contract, unacceptable concrete can be detected while the job is in progress. This will allow timely correction of the problem and as well as altering construction procedures, if necessary, to prevent the reoccurrence of the problem. Typically NQ core is adequate for most borings except for joints between panels, which are more accurately represented with 6-inch diameter core. Problems that have been detected with core borings are honeycomb concrete, segregation, lack of cementation, bentonite contamination, and voids in the concrete. Contractors have relied heavily on grouting to remedy most of these problems except where the areas of poor quality concrete were so extensive that grouting was not acceptable. Such problems were corrected by constructing a panel upstream, adjacent to the problem area.

One potential problem area which has been noted at most projects is the upper 10 to 20 feet of concrete panels. This area frequently contains poor quality concrete contaminated with scum and laitance. This is not critical when the wall is being installed through a dam from the crest which places this bad zone above normal pools and areas of high head differentials. However, in walls installed before dam construction or in walls installed at or near the upstream toe, the concrete quality must be acceptable throughout the panel since the entire panel can be exposed to high heads. Overflowing the panels until fresh concrete appears helps reduce the amount of contamination. The length of time required to place an element may affect the quality of the concrete. It has been noted that panels which are completed quickly appear to have good quality concrete throughout, while panels which take several hours to complete often have a concentration of contaminants near the top of the panel. This may be caused by the concrete stiffening or taking its initial set before the placement is completed allowing contaminants to be trapped in the panel. Problems due to slow placement rates would be compounded as the size of the element increases. While the depth of the element is controlled by the site conditions, the length of the panels is usually left to the contractor. While the longer panels are less expensive to construct, they will take more time to place and may lead to the above mentioned problems. Experience in SAD has shown that the maximum length of any completed panel should be 24 feet or less. This will limit the lateral travel of the concrete to 6 feet or less with a two tremie pour. Stiffen ing of the concrete will often cause poor contact at the top of the joints between panels. The stiff concrete is under very little head at that point and will resist flowing to the joint where laitance is often trapped leaving an open path for seepage. Recent investigations have shown a wide range in the quality of the concrete at panel joints. At Walter F. George and Steel Creek several joints had a concentration of scum, honeycombed concrete, and/or uncemented aggregate in the upper 10 to 20 feet which others were essentially tight in the upper portion of the wall. However, at St. Stephen powerhouse coring of the joints indicated a tight, bonded contact. This appears to be the result of the construction method which removed a small amount of the adjacent primary panels when the secondary panel was excavated and the fact that the eight foot secondary panel length limited the horizontal flow of concrete to 4 feet. The reduction of panel volume also increased the placement rate which prevented problems associated with premature set. Where the entire panel panel will be exposed to high head differentials, the quality of the concrete and tightness of the joints throughout must be guaranteed.

The effectiveness of the concrete cutoffs has been dramatic. At Walter F. George, flows from the relief well system ceased as the wall was completed. Piezometric heads upstream of the cutoff have approached headwater while down-stream of the cutoff heads have dropped significantly. A further indication of the effectiveness of this cutoff was demonstrated upon completion of the Phase I wall, which extended 1244 feet landward from the lock wall. The lock was successfully dewatered without triggering sinkhole activity for the first time since construction.

At the Clemson Diversion Dam, similar changes in piezometric head have been observed. At the St. Stephen Powerhouse, the pool was recently impounded and there has been no evidence of a reoccurrence of the detrimental seepage which occurred during initial impoundment.

CEMENT-BENTONITE SLURRY TRENCH CUTOFF WALLS. Since there has been no direct experience with this type of cutoff in SAD, Mr. Wayne Adaska, of the Portland Cement Association was invited to present a talk on various aspects of cement-bentonite (C-B) cutoffs. Some of the apparent advantages over soil-bentonite (S-B) cutoffs are:

a. The C-B method is not dependent on the availabilty or quality of soil for backfill.

b. The construction sequence is more flexible. It permits construction in sections to meet site constraints and adapts to extreme topographic changes where a step-type construction is required.

c. The width of the trench is generally less than that required for a S-B trench. The S-B trench must be wide enough to permit free flow of the backfill material.

d. An area adjacent to the trench is not required for mixing, making C-B more suitable for projects with space limitations.

The uses of C-B cutoffs are essentially the same as S-B cutoffs: underseepage at dams, dewatering, pollution control and containment. Design permeabilities are on the order of 10-6 to 10-7 cm/sec while strengths are generally on the order of 10 to 40 psi (depending on water-cement ratios). Mixing requires hydrating the bentonite prior to introducing the cement to the slurry using mixers similar to those used for S-B slurries. After the cement is mixed in the slurry, the C-B slurry is then introduced directly into the excavation. Excavation can be by backhoe, dragline, or clam buckets though dragline trenches are generally too wide to be economically competitive.

VIBRATING BEAM CUTOFFS. Another application of slurry cutoffs involves installation with a vibrating beam. This involves a specially designed I-beam which is driven into soil to the required depth by use of a vibratory hammer while injecting slurry (either bentonite or cement-bentonite) at the tip. Then the I-beam is withdrawn while slurry is injected to fill the void left by the beam. Two test cells have been installed using this method, one for the Mobile district and one for the Wilmington district. Both tests yeilded unsatisfactory results for the intended purposes. To date, this method has not been used on a project in SAD.

APPENDIX A

Agenda and Attendees

AGENDA

GEOTECHNICAL MEETING ON SLURRY TRENCH CUTOFFS

Tuesday, October 2

1:00*	Opening Remarks	SADEN-F, SAMEN-F, SAMCO
	Session I - Moderator - Jim Erwin	
1:15	Soil-Bentonite Cutoffs - General Design & Construction	Baer/SADEN-FS
2:00	Concrete Cutoff Walls - General Design & Construction	Hallford/SADEN-FG
2:45	Break	
3:00	West Point Slurry Cutoff	Rogers/SASEN-F
3:20	Wolf Creek, Cutoff	Simmons/ORNEN-G
3:50	Bonneville, Cutoff	Bankofier/NPPEN-F
4:20	Clemson Design	Hightower/SASEN-G
4:40	Clemson Construction	Zielonka/SAWEN-GG
5:00	Huxtable Design and Construction	Akers/SADEN
5:30	Questions for Presenters	
6:00	Adjourn/Dinner	
8:00	Walter F. George Hungry Hole - After Dinner	Bryan/SAMEN-F

Wednesday, October 3

	Session 2 - Moderator - Greg Baer	
8:00	Tennessee-Tombigbee Projects Design	Gustin/SAMEN-FS
8:30	Tennessee-Tombigbee Projects Construction	Joiner/SAMCO
9:00	Cooper River Slurry Cutoff, Design & Construction	Lane/SASEN-GS
9:30	Break	
9:45	Cooper River Concrete Cutoff Design	Hightower/SASEN-GG
10:00	Cooper River Concrete Cutoff Construction	Hess/SACRO
10:20	Walter F. George History & Design	McFayden/SAMEN-FG

11:00	Walter F. George Construction	Jangula/SAMCO-W
11:45	Questions for Presenters	
12:00	Lunch	
1:00-5:00	Field Trip to Walter F. George	Jangula/SAMCO-W
8:00	Cooper River Seepage Problem - After Dinner	Erwin/Hess SADEN-F/SACRO

Thursday, October 4

Session 3 - Moderator - Jack Bryan

8:00	Sunny Point Vibrating Beam	Golden/SAWEN-G
8:20	Cement-Bentonite Cutoffs	Adaska/PCA
9:00	Break	
9:15	Guide Specs.	Baer/Hallford/Foster
9:45	Panel	Baer/Hallford/Foster/Fisher/Davidson
10:30	Closing Remarks/Discussion	Erwin/SADEN-F

*All times are Central Daylight Time

Attendees - Lakepoint State Park

Geotechnical Meeting

Adaska, Wayne
Akers, Ken
Arnold, Leroy
Ayers, Fred
Baer, Greg
Ballard, Rod
Bankofier, Duane
Bennett, John
Blackburn, Ed
Boernge, Jim
Brown, Roger
Bryan, Jack
Canning, Charlie
Carter, Don
Couch, Ben
Davidson, Luther
Davidson, Richard
Dodson, Ces
Duster, Clarence
Erwin, Jim
Felder, Bobby
Fisher, Paul
Foster, Byron
Franklin, Binks
Golden, John
Gustin, Ray
Hallford, Chuck
Hannon, Rich
Hartung, Steve
Hess, Charlie
Hightower, Ted
Honeycut, Sam
Hudak, Joe
Jackson, Lawson
Jangula, Terry
Johnson, George
Joiner, Jerry
Keller, Dan
Kraft, Leonard
Lane, David
Lee, Albert
Leach, Roy
Lewis, Jim
Lynch, Bob
Mahtesian, Monty
Matthews, Clarence
McCollum, Ross
McFayden, John
Melnyk, Joe
Mitchell, Brit
Moon, Edger
Moore, Bruce
Novak, Tom
Paris, Jim
Parrillo, Dan
Perry, Ed
Pope, Ellis
Rainer, Jerry
Redell, Carwin
Rogers, Joe
Sams, Clay
Shafer, Paul
Simmons, Marv
Smith, Card
Smith, Joe
Snipes, Bob
Spencer, Bob
Stanley, Stan
Stephenson, Bob
Tyson, Johnny
Tison, Lane
Van Fleet, Steve
Wagner, Ramona
Weaver, Frank
Willig, Karl
Winter, Carroll
Zielonka, Ted

APPENDIX B

Presentation Papers

SOIL-BENTONITE SLURRY TRENCH CUTOFFS
DESIGN, CONSTRUCTION, AND PERFORMANCE

GREG BAER, SOUTH ATLANTIC DIVISION

INTRODUCTION

The "Slurry Wall Method" of excavation, although being relatively new, is really a very simple and innovative method of installing cutoff walls.

SLIDE 1

The construction of "Soil-Bentonite Slurry Trench Cutoffs" or just plain "Slurry Trenches" (ST) as they are often referred to, consists first of excavating a continuous narrow trench (usually 3 to 5 ft. wide), with vertical sides, through an aquifer or potentially pervious stratum and tying it into an underlying impervious stratum. The vertical sides of the trench are stablized during the excavation process by keeping the trench full of a slurry suspension of bentonite and water. Upon completion of excavation, the slurry in the trench is displaced usually with a select blend of soil-bentonite backfill forming a relatively inexpensive and highly effective impervious barrier. The first large ST cutoff in the United States was constructed between 1948 and 1950 at Terminal Island (near Long Beach), California to stop sea water infiltration. It was not until 1960 at the Wanapum Dam on the Columbia River that a ST cutoff was used in connection with major dam construction. Since then, and in particular in the last 10 to 15 years, geotechnical engineers have become more aware of the advantages, success and low costs of ST cutoffs and their use has spread rapidly.

The first ST cutoff constructed in SAD was at West Point Dam on the Chattahoochee River in 1966 to control underseepage. It was 5 ft. wide with a maximum depth of 60 ft. Since then they have been used successfully on 12 additional projects including nine projects on the Tennessee-Tombigbee Waterway, the Richard B. Russell Project, the Cooper River Rediversion Project, and the Jackson Mississippi flood levees.

To begin with, I would like to touch upon the applications and advantages of ST and then present some of the basics for design and construction.

APPLICATIONS

With respect to application, the use of ST cutoffs for construction dewatering and for permanent seepage barriers is now widely accepted.

SLIDE 2

The general benefits associated with their use are:

a. They can be constructed in almost any type of ground situation.

b. Continuity of the cutoff is easy to attain due to the excavation method which essentially eliminates any potential of openings.

c. The backfill can be made flexible to adjust to local ground movements thereby eliminating any possible cracking, and

d. Construction is relatively rapid and inexpensive as compared to other alternatives.

For construction dewatering, ST's are very effective when they can be tied into an impervious stratum such as the case on the projects on the Tennessee-Tombigbee Waterway (TTW).

SLIDE 3

There, as was typical at the Aliceville L&D project, the slurry trenches were constructed around the perimeter of the construction site beneath the cofferdam.

SLIDE 4

Some of the advantages of using the ST for dewatering in lieu of conventional systems such as well points are:

a. Elimination of operation and maintenance costs associated with a normal dewatering system. This can be a very significant savings on projects where a dewatering system would have to remain in operation for long periods of time which is normally the case on large Civil Works projects.

b. Elimination of any risk associated with a possible breakdown of a dewatering system or even power outages. Failure of a dewatering system could of course not only jeopardize the stability of the excavation slopes but also the foundation under certain circumstances.

c. Elimination of damages due to drawdown of the water table in the area surrounding the site. This was really not of major concern of any of the projects on which ST's were used in SAD, however, there are instances where this could be very important. For example, in some areas damage to domestic wells as well as the drying up of agricultural lands could be prevented. Also, in some cases, settlement of structures due to drawdown could be avoided.

d. Elimination of wells and headers around the perimeter of an excavation thus allowing freer access to and from the construction site.

As for seepage barriers, ST's can and are used as pollution control barriers and as permanent cutoffs in connection with water retention structures. As pollution control barriers they have been constructed around sewage treatment plants, chemical wastes, and sanitary landfills to isolate the effluent from the local ground water systems. Some of you may be familiar with the superfund program where ST's are being used. The Wilmington District will be letting a contract possibly this fall to install a ST around a dredged disposal area at MOTSU in North Carolina to control salt water intrusion.

SLIDE 5

On a number of the TTW projects, ST's were constructed laterally from the dewatering slurry trench to provide permanent seepage control.

SLIDE 6

This greatly increased the seepage path around the structure thereby reducing the exit gradients into the river channel downstream.

SLIDE 7

Mobile also used a ST quite successfully to control seepage beneath the flood levees around Jackson, Mississippi. These levees were plagued with underseepage problems since they were constructed. During every major flood numerous boils developed at and beyond the downstream toe.

SLIDE 8

The ST was constructed near the upstream toe and tied into the existing levee with an impervious blanket.

SLIDE 9

It is intersting to note that less than a week after completion of the trench a flood occurred and there were no signs of seepage or boils downstream.

TRENCH LOCATION

As for trench location, there has been a lot of discussion throughout the Corps and particularly in Mobile with respect to whether the slurry trench should be located upstream and tied into an embankment or whether it can be located beneath the embankment. There are pros and cons for both locations.

SLIDE 10

The present Corps policy as outlined in EM 1110-2-2300, "General Design and Construction Considerations for Earth and Rockfill Dams" is to locate ST's upstream and tie them into the core with a blanket. It is recognized, however, that there may be exceptions to this and each case should be judged individually.

BENTONITE SLURRY

Now that I have touched upon some of the advantages and applications, I would like to discuss briefly the bentonite slurry.

Bentonite slurry is a collodial suspension of bentonite and water which has been used in the drilling industry for over 50 years.

SLIDE 11

Similar to its functions in the drilling industry, bentonite slurry is used in slurry trench construction to:

a. Support the face of the trench and prevent the sides from caving;

b. To seal the materials in the sides of the trench by forming what is normally referred to as a filter cake which prevents slurry loss.

c. A third function of the bentonite slurry which is not always addressed in ST construction is the suspension of soil mixed with the slurry during the excavation process which prevents excessive layers of unconsolidated materials from accumulating in the bottom of the trench.

While performing these functions, it is important to remember that, the bentonite slurry must also be workable and remain fluid enough during construction to allow for both the passage of the excavating equipment and the placement of the soil-bentonite backfill.

The type of bentonite normally used in slurry trench construction is generally referred to as "Premium Grade" or "Natural Bentonite". Bentonite is a clay derived from the alteration of volcanic dust and ash consisting primarily of the clay mineral montmorillonite. The chief commercial deposits in the United States are located in the black hills of Wyoming and South Dakota.

Some characteristics of bentonite which are often addressed are its swelling capacity, water absorption, and thixotropy. Bentonite can swell up to 10 to 15 times its original volume and can absorb water up to 7 times its own weight. Thixotropy is the property of a material to undergo an isothermal change from GEL-TO-SOILD-TO-GEL upon agitation and subsequent rest. In other words, when the bentonite suspension is stirred vigorously it flows like free fluids, gradually setting again to a gel which stiffens progressively when stirring is stopped.

The process by which the bentonite slurry functions in ST construction begins when the bentonite is mixed with water and undergoes hydration. Hydration is also referred to as "Swelling" and in simple terms constitutes the attachment of water molecules to the bentonite platelets through electrical bonding. This increases the viscosity of the mixture and the resulting specific gravity is then greater than that of water. The duration of the hydration process is dependent on such things as the type of mixing system used, length of mixing time, grade of bentonite, and chemical properties of the water.

With respect to the water used for the bentonite slurry, it should be good quality containing a minimum amount of impurities and have a neutral consistency. It should be essentially clean, and free from oil, acid, and organic matter.

SLIDE 12

Some recommended standards for the water contained in the guide spec which we have distributed are:

a. A pH greater than or equal to 7.0 $\pm$ 1.0.

b. Total dissolved solids not greater than 500 PPM.

c. Oil, organics, acids, alkali, or other deleterious substances not greater than 50 PPM each.

d. Hardness less than or equal to 50 PPM..

It is realized that it may not be possible to meet these standards in every case and some adjustments may be necessary. However, in all cases the water to be used in the bentonite slurry mixture as well as the local groundwater should be analyzed during design. Some chemicals are known to react with bentonite such that an increase in trench permeability could occur.

FILTER CAKE

SLIDE 13

In the trench, the slurry enters the open pores of the soil on the sides due to the pressure difference caused by the slurry. This same pressure difference also forces the water from the slurry that has impregnated the soil causing a build up of bentonite particles in the voids. As the filtration process continues, more bentonite particles accumulate in the pores and on the sides of the trench forming a tightly packed zone of gelled material referred to as the filter cake. As the filter cake builds up, it becomes impervious offering a resistance to any further loss of slurry as well as water from the slurry through the sides of the trench.

SLIDES 14-20

A good example of this was evident on a small model the lab constructed. Here the bentonite slurry which was dyed red migrated out into the sand as the trench was deepened, however, the zone of migration ceased with time even though the slurry level was maintained well above the water. (A close look at the sides of the trench showed evidence of the filter cake beginning to form.)

SLIDE 21

A classical field example of a filter cake formation was evident at the RBR project when the cofferdam was excavated. The backfill which consisted of a relatively clean sand and bentonite shows up quite clearly.

SLIDE 22

A close up of the upstream side of the trench shows the filter cake.

There is a lot of discussion in slurry trench literature as to the contribution of the filter cake to overall impermeability of the trench. Tests have been conducted which indicate that filter cakes can have a permeability as low as 10-8 cm/sec. The general consensus among most is to ignore any contribution of the filter cake to the imperviousness of the trench. This is a conservative approach and may be justified since the filter cake could be disturbed during excavation and backfilling operations. In addition to providing an impermeable seal, the filter cake also has a plastering effect on the sides of the trench which helps to hold the individual soil particles together. This is particularly important in cohesionless materials which may tend to slough or peel off due to their own weight and the action of the excavation equipment.

MIXING OF BENTONITE SLURRY

In the field there are two basic methods of mixing the bentonite slurry.

SLIDE 23

The most commonly used method involves the use of flash-type mixers (also sometimes referred to as venturi mixtures) in conjunction with circulation ponds. The flash mixer introduces dry bentonite into a highly turbulent water jet.

SLIDE 24

The resultant mix is then stored in a circulation pond until hydration is complete, usually within 24 hours. In many cases, a two-pond operation is used with one pond used for mixing and a second pond for storing the hydrated slurry prior to introduction into the trench.

The second method of mixing is essentially a batch system where given volumes of water and bentonite are fed into a tank and subjected to high speed agitation by such means as a propeller-type mixer. Agitation is continued until hydration is complete, normally a matter of minutes and the slurry is then deposited directly into the trench. The mixing system used at the Wolf Creek project was a batch system.

SLURRY TESTING

The normal slurry testing program includes viscosity, density, and filtrate loss.

Viscosity is generally recognized by most experts in the field of slurry trenches as being the most important property.

SLIDE 25

At the time of introduction into the trench as well as throughout construction, a minimum viscosity of 40 seconds is specified which generally assures trench stability and good filter cake formation. With good mixing conditions, using a premium grade bentonite and reasonable water quality, bentonite consumption will be approximately 5 to 7 percent by weight of slurry to achieve 40 sec. viscosity.

SLIDE 26

Viscosity is normally measured with the marsh funnel. The testing procedure consists simply of measuring the time required for a known volume of slurry to run out of a standard funnel.

The density or specific gravity is also an important slurry property since it effects both the stability of the trench and placement of the backfill. If the slurry becomes too heavy, it is difficult to assure good backfill placement because the backfill may tend to fold over the heavy slurry rather than displace it.

SLIDE 27

Density is normally specified between 70 and 85 PCF during backfill placement. Some experts advocate that as long as the slurry density is 15 PCF less than the backfill then proper backfill placement can be achieved. The density of the bentonite slurry at the time of introduction into the trench is usually around 64 PCF and increases in density as the excavation progresses.

SLIDE 28

The density is measured directly with a mud balance.

The third property of the bentonite slurry which is often monitored is the filtrate loss. It is generally interpreted to give an indication of fluid loss and presumably relate a material to the filter cake characteristics.

SLIDE 29

A maximum filtrate loss of 20 cc in 30 minutes is recommended in the Guide Spec.

SLIDES 30-32

The standard filter press test is used to measure filtrate loss. In the test, a sample of slurry is subjected to a constant pressure (100 psi) and squeezed through filter paper. The amount of filtrate (measured in milliliters or cubic centimeters) escaping over a period of 30 minutes is referred to as the filtrate loss. It is the opinion of many slurry trench experts that the filtrate loss need not be specified as long as the viscosity and density are properly specified.

TRENCH STABILITY

SLIDE 33

The analysis of trench stability assumes the formation of the filter cake on the sides of the trench which enables the slurry to exert its full hydro-static pressure. This hydrostatic pressure is the major stabilizing factor.

The height of the slurry in the trench in relation to the ground water level is critical with respect to stability. A drop of the slurry below the ground water could obviously jeopardize stability.

SLIDES 34 THRU 46.

In the model prepared at the lab, after the trench was excavated, the slurry was lowered below the water level and then the water level raised. As the water level came up a crack developed on the right side of the trench and a progressive failure followed. Failure did not occur immediately on the left side because there was a time lag in the rise of the water on that side. The rise in the water level to near the surface essentially resulted in complete collapse of the sides of the trench. Specifications generally require that the slurry be maintained a minimum of 2 ft above the ground water. It is therefore important to be aware of any potential ground water fluctuations such as those associated with seasonal or tidal changes which may occur during construction. In areas of high ground water, construction of working platforms are often required to maintain the slurry above the ground water. Even when the ground water is low, it is also important to maintain the slurry within a few feet of the top of the trench to ensure overall trench stability.

EXCAVATION PROCEDURE AND EQUIPMENT

As for the type of excavating equipment used in ST construction, it is not of major concern to the designer as long as it is capable of producing a continuous trench to the depth and width specified. The contractor on the other hand is very concerned with the type of equipment since the slurry trench production rates are so dependent on the excavation.

SLIDE 47

Dragline buckets,

SLIDE 48

clamshells, and

SLIDE 49

backhoes are probably the most common pieces of ST excavating equipment, however, other equipment such as rotary drilling rigs may be required to facilitate excavation.

SLIDE 50

For the excavating of the ST at Cooper River, the contractor used a backhoe, followed by auger holes and a clamshell. Selection of excavating equipment is dependent on the depth and width of the trench as well as the type of materials being excavated.

SLIDE 51

Non-perforated buckets are generally preferred because they remove more sand than is possible with a perforated bucket.

SLIDE 52

It is interesting to note that in 1974 when the first ST on the TTW was installed the limiting depth for backhoe excavations was approximately 42 ft.

SLIDES 53-54

In November 1981, the deepest trench on the TTW at Lock E which was 72 ft was excavated with a Koehring Model 1266 backhoe with extended boom. The trenches were both 3 feet wide and excavated through unconsolidated alluvial material.

CLEANING AND SAMPLING OF TRENCH BOTTOM

SLIDES 55-56

Prior to placement of the backfill, the trench bottom is usually cleaned. To eliminate potential seepage resulting from sand and gravel settling out of the slurry, or which may have been left in the trench. Typical cleaning equipment consist either of an air-lift pump or a submersible pump. Sometimes a clamshell bucket is used if the trench has been well keyed into an underlying impervious stratum. The cleaning generally just proceeds the backfill operation. When an air-lift is used, the material removed is generally mixed directly with the backfill. When submersible pumps are used, they are sometimes circulated through sand separators which allow the slurry to be reclaimed.

Subsequent to cleaning, the trench bottom is normally sampled (if possible) to insure that the trench is adequately keyed into the desired impervious stratum.

SLIDES 57-58

Typical samplings devices consist simply of attaching a torpedo shaped spoon sampler to the bucket of a backhoe which lowers it into the trench and pushes it into the clay.

BACKFILL

The soil bentonite backfill to be placed in the trench may consist of material from the trench excavation, material from the trench excavation mixed with borrow material or all borrow material.

SLIDE 59

The primary design parameters ccontained in Chapter 9 - EM 1110-2-1901 for the backfill are blowout requirements, permeability, and strength and compressibility if the trench is to be constructed beneath or in a structure.

SLIDE 60

Consideration for blowout of the backfill into the surrounding materials is critical when the trench is excavated through open work gravel. The factor of safety against blowout is:

$$F = \frac{I\ \text{Allowable}}{I\ \text{Actual}}$$

where:

IAllowable = Allowable hydraulic gradient from laboratory blowout tests (25 to 35 for widely graded gravel containing no sand)

IActual = Actual hydraulic gradient existing on slurry trench

$$I\text{Actual} = \frac{H}{W}$$ where: H=Maximum head
W=Width

Assuming a factor of safety of 3 and an allowable hydraulic gradient of 30.

$$W = \frac{H}{10}$$

In other words, one foot of trench width for every ten ft of head. It is interesting to note that this was the accepted rule of thumb for determining the width for permanent trenches in 1972 during the initial design on the TTW. In general, a minimum trench width is governed by the equipment. Most large backhoes require 2.5 to 3.0 ft. The guide specification recommends 3 ft which accommodates the use of most equipment and does not hinder the placement of the backfill by conventional methods.

SLIDE 61

The permeability of the backfill is dependent on the gradation and quantity of bentonite used as well as on the mixing procedure. Naturally, the lower the permeability of the materials comprising the backfill, the lower will be the permeability of the soil bentonite backfill. Examination of slurry trench literature indicates that the permeability of the soil bentonite backfill is generally in the order of 10-6 to 10-7 cm/sec. To achieve this permeability the backfill must contain a significant percentage of fines (minimum of 10-20 percent passing the #200 sieve) and preferably clay fines.

SLIDE 62

Typical backfill gradations which were specified on some of the projects in SAD the and the Wanapum Dam. Measured permeabilities of the backfill on the slurry trench at the Columbus Lock and Dam project on the TTW were in the order of 10-6 to 10-7 cm/sec. With respect to bentonite content, some experts advocate a minimum of 1 percent bentonite content to achieve a low permeability backfill. The bentonite content was not, however, controlled on any of the projects in SAD.

SLIDE 63

The compressibility of the backfill depends on the gradation. The potential for detrimental differential settlement is dependent on the location of the trench. When a trench is located beneath an embankment, the compressibility should be comparable with the compressibility of the surrounding ground. In general, settlement of ST's are more pronounced in trenches wider than 8 ft. Relatively narrow trenches settle much less, due to arching effects. Settlement in the order of 1 to 6 inches has been measured in trenches 50 to 90 ft. deep. Settlement plates were installed on a number of ST's on the TTW projects.

SLIDE 64

In regards to the shear strength of the backfill, the strength increases with time due to consolidation and thixotropy. For design purposes, the strength is normally assumed to be zero. Undisturbed samples were obtained from both the Cooper River and Columbus projects and the results of lab test are summaried on Table 1.

MIXING OF BACKFILL

SLIDE 65

Material for the backfill, is usually mixed by windrowing, dozing, or blading to meet the required gradation. There have been cases where mechanical mixing off-site has been utilized due to either inadequate mixing room or to insure a high quality uniformally blended backfill. During the normal mixing process, the backfill is sluiced with slurry and mixing is continued until a homogeneous mixture with a slump of 3 to 6 inches and free from lumps larger than 6 inches or pockets or fines or sands and gravels is obtained. The bentonite slurry mixed with the backfill is normally obtained from the trench. It is thicker and contains suspended fine materials that aid in achieving a low permeability backfill. This also allows for introduction of fresh slurry into the trench which reduces the suspended solids load. During the mixing process, small dikes are often constructed parallel to the trench to keep the backfill from flowing into the trench as a result of wave action created by the equipment during mixing. Intermittent holes are constructed in the dike to allow excess slurry to flow back into the trench. To avoid possible sloughing in the trench, dozers are prohibited from operating in a back and forth fashion parallel to the open trench closer than 15 feet from the edge.

PLACEMENT OF BACKFILL

SLIDE 66

Backfill placement starts at the beginning of the trench when sufficient length of trench has been opened such that the toe of the advancing backfill will not infringe upon the excavation. The Guide Specification recommends the excavation slope proceed the toe of the backfill by no less than 50 ft and no more than 150 ft. In general, enough space should be provided behind the excavation for cleaning the trench bottom. Initially, the backfill is lowered to the bottom of the trench using a clamshell bucket and deposited according to the anticipated angle of response, approximately 1V on 7H. This procedure is continued until the backfill emerges at the surface, reaching the top at one end and sloping downward to the other.

SLIDE 67

A bulldozer is then used to push additional backfill into the trench such that the material slides down the backfill without trapping pockets of slurry. Backfill should never be allowed to fall freely through the slurry.

TREATMENT OF THE TOP OF THE TRENCH

SLIDE 68

If a completed slurry trench is left exposed on the surface, it will dry out and tension cracks will form. To prevent this, most specifications require that a blanket of moist material, for example, 2 feet thick and 8 feet wide blanket be placed over the completed trench. The blanket also allows movement of equipment across the trench.

COSTS

ST cutoffs were used on 8 projects on the TTW for construction dewatering. A savings of between 25 to 30 percent per project has been estimated for using the ST cutoffs in lieu of well points.

SLIDE 69

Bid prices per square foot for all the ST constructed in SAD. As can be seen, the unit prices ·ay from 1974 to 1975 for a 3 ft wide trench under similar conditions actually went down somewhat. This can probably be attributed to the fact that contractors became more familiar with slurry trench construction techniques and a greater interest on behalf of more contractors.

Of the 13 projects on which slurry trenches have been used in SAD, there has only been one claim. That is in the Cooper River Project and is for $3.5 million. It is associated with the difficulty the contractor experienced in excavating the trench. The claim is currently unsettled. It is very important to adequately define the material through which the trench passes since production in slurry trench construction is so dependent on the excavation.

SUMMARY

In summary, ST' have achieved wide recognition as effective and economical seepage barriers. They have been used very successfully in SAD for both construction dewatering and the control of underseepage. In ST construction, the stability of the trench and overall constructibility are dependent on the properties of the bentonite slurry and the level of the slurry in the trench. The slurry properties which are normally monitored are viscosity, density, and filtrate loss. Typical values for these during construction are: A minimum viscosity of 40 seconds as measured by the marsh funnel, a density of 64 to 85 PCF, and a maximum filtrate loss of 20 cc in 30 minutes as measured by the standard filter press test. With respect to slurry level in the trench, it should always be maintained a minimum of 2 ft above the water table and preferably within a few feet of the working surface. Design considerations for the backfill include blowout requirements, permeability, compressibility, and shear strength. Permeability is probably the most important parameter and it is highly dependent on the gradation, quantity of bentonite, and mixing procedures. The permeability of a well mixed backfill containing 10-20 percent fines and around 1 percent bentonite will range from 10-6 to 10-7 cm/sec.

SUMMARY OF
LAB TEST RESULTS

							Direct Shear		Q		$\overline{R}$			
Project/Depth	Class	LL	PL	PI	W.C.	K	Ø	C	Ø	C	Ø	C	Ø	C
Columbus L&D														
0.0 - 2.5	SC	31	19	12	18.2	8.0×10^{-7}**	3.90	0.10*	25.5	0.40	28.0	0.40	34.0	0.0
5.0 - 7.5	SC	33	19	14	23.6	1.7×10^{-7}**	31.0	0.30*	0.0	0.33	16.5	0.20	31.0	0.0
10.5 - 12.5	SC	29	20	9	28.8	2.5×10^{-6}	35.5	0.15*						
19.0 - 21.0	SC	32	18	14	23.7	3.0×10^{-7}	32.5	0.20*	1.5	0.15	12.0	0.20	27.0	0.10
Cooper River														
17.0 - 19.9	SC	40	19	21	34.4				0.0	0.25				
38.0 - 40.0	SC	32	19	13	27.2				0.0	0.16				
66.4 - 68.4	SC	34	19	15	25.2				0.0	0.22				

**K_v * Apparent Cohesion

Table 1

CONCRETE CUTOFF WALLS - GENERAL DESIGN AND CONSTRUCTION
CHARLES HALLFORD - SOUTH ATLANTIC DIVISION

SLIDE 1

The purpose of this talk is to cover the general aspects of concrete cutoff walls that are basic to most of the projects that we are going to hear about this week. I will cover those procedures which are common to these projects to allow each speaker to concentrate on site specific features of design and construction.

SLIDE 2

Concrete panel walls have been placed prior to construction to correct known foundation problems. Two examples of this are manicogan III, is a large earth/rock-fill dam that is part of the James Bay Hydropower project in northern Quebec and the Bonneville second powerhouse near Portland Oregon.

SLIDES 3-5

At the Manicogan III project, the damsite is a steep walled canyon with approximately 300 feet of glacial and alluvial material. As this was impractical to remove or treat by conventional methods, it was decided to install a concrete panel wall from the valley floor to the bedrock to cut off the anticipated underseepage.

SLIDE 6

The Bonneville Second Powerhouse is a hybrid application of panel cutoff walls. The cutoff was designed to permit dewatering of the excavation site during construction and then portions were to serve as a permanent structural barrier against underseepage.

This panel wall was installed through a complex variety of materials including recent flood pain alluvium and older alluvium including sands, gravels, cobbles, and boulders which sandwiched a wedge of the cascade landslide.

SLIDE 7

The complex nature of the material made conventional treatment inappropriate for this project. In both cases, the unique foundation problems were recognized early in design which provided a novel solution that proved not only cost effective but most important each wall performed as designed.

Unfortunately, foundation defects are not always detected, recognized, or anticipated during design and/or construction and have resulted in problems of varying magnitude during a project's life. While some have been alleviated by more conventional methods such as grouting, impervious blankets, stability berms, relieve wells, etc., several have been of a nature that precluded such approaches and required a positive cutoff.

SLIDE 8

Map - Wolf Creek Dam, Cumberland River, Kentucky; Walter F. George Lock and Dam, Chattahoochee River, Georgia-Alabama; Clemson University Diversion

Dams, Seneca River, South Carolina; Mill Creek dam, Walla Walla, Washington; are Corps projects where concrete cutoff walls were installed or are being installed.

SLIDE 9

WFG Section - These walls were designed to cut off seepage through alluvial foundation materials and/or provide a positive cut off through highly solutioned limestones underlying earth embankments. With the exception of Mill Creek Dam, each cutoff was installed while the project was in operation.

SLIDE 10

Normal project operation was maintained throughout the cutoff installation at each project except at Wolf Creek which was held below normal seasonal levels during completion of critical sections.

Basically there are two types of cutoff walls which are characterized by those installed at the Bonneville Second Powerhouse and at Wolf Creek Dam. The true panel wall, which is most commonly referred to as a "Bonneville Wall" is constructed of panels typically with a length to width ratios of 4:1 to 12:1. These panels range from 8 to 30 feet in length, 2 to 3 feet in width and have been installed to depths of over 150 feet. The element wall, which is most commonly referred to as a "Wolf Creek Wall," is constructed of discrete circular elements of 26 inches diameter concrete filled steel casing connected by panels with a length to width ratio of approximately 2:1. Of the two types of cutoff walls, the "Bonneville" type is typically much less costly and would be applicable to a wider range of projects.

SLIDE 11

The "Bonneville" cutoff wall has three types of panels which are characterized by the final shape of the concrete panel and sequence of installation. These are primary, secondary, and running. The primary panel is the initial panel in any sequence and is not installed adjacent to an existing panel. The spacing between primary panels varies with the Contractor's operations and may range from 8 feet up to hundreds of feet. The final configuration of the concrete primary panel is concave on both ends. This is the result of removing the shoulder or stop end pipes. These concave ends serve as a guide during the excavation of the adjacent panels. A secondary panel always connects two existing panels that are generally less than 25 feet apart. This panel is excavated using the concave ends of the existing panels as a guide to insure the wall is continuous.

SLIDE 12

This panel will always have convex ends and will complete a segment of cutoff wall.

SLIDE 13

The running panel always extends from a single panel using the concave end of the existing panel for a guide during excavation. The completed panel will have a free end which will be concave while the connecting end will be convex.

When working several segments of wall simultaneously, the Contractor may elect to construct the wall sequentially with running panels.

The "Bonneville" type cutoff has been used to cut off foundation seepage through overburden materials and through soft limestone. Due to the nature of the method of excavation the "Bonneville" type wall is best suited for soft rock and poorly consolidated materials. When hard material is encountered, the production rate typically will drop drastically.

SLIDE 14

Mill Creek Section - Where seepage is through overburden materials above firm to sound rock, the "Bonneville" type cutoff can be keyed a short depth into rock to successfully cut off the seepage. In cases where the rock seepage is through a rock foundation which is soft to moderately hard, a panel wall can be installed quite efficiently and effectively.

SLIDE 15

WFG - An excellent example of this application are the cutoff walls at the Walter F. George project which we hear about and visit tomorrow. Of major significance is that this type of cutoff opens short segments of embankment and/or foundation at a time which limits the area for potential failure to a segment that can be controlled or repaired without risking catastrophic failure of the project. The cutoff penetrates the zone(s) of seepage with a rigid, impermeable barrier capable of withstanding high head differentials even across zones with little lateral support such as filled cavities.

SLIDE 16

Wolf Creek - While the "Bonneville" type wall has three types of panels, the "Wolf Creek" wall has only primaries and secondaries. By the nature of this construction, the "Wolf Creek" wall is more costly and results in a slower placement rate. However, it has the advantages of greater depth, better control of verticality, the ability to penetrate hard rock at an accelerated rate, and the protection of the embankment and overburden with casing. This type of cutoff is best suited for eliminating seepage through competent foundation rock where the danger of piping into the rock is present to considerable depths.

SLIDE 17

The major types of equipment used in construction of a panel cutoff wall are clam buckets, chisels, drills, slurry plants, concrete plants, and spoil removal equipment.

SLIDE 18

Cranes are typically used for most of the project work and will be the key piece of equipment on the job.

SLIDE 19

The clam buckets for a panel wall are the same for each panel type regardless of the material being excavated.

SLIDE 20

The open bucket takes a bite approximately 8 to 10 feet long and

SLIDE 21

2 feet wide. Most are cable activated and use their weight to obtain penetration into the material being excavated.

SLIDE 22

However, a hydraulic bucket on a "Kelly Bar" has been used very successfully to depths of 120 feet. This provided a high degree of control as the clam was guided into the excavation without the free fall effect common the the cable buckets.

SLIDE 23

The clam buckets typically have interlocking teeth welded to the jaws to aid in penetrating the insitu materials.

SLIDE 24

The construction of the cable tools used by most contractors is very similar with the major difference being the weight of the bucket.

SLIDE 25

If penetration was not as good as expected, weights can be added to the frame to increase the momentum at impact. The bucket plate thickness and configuration of the bucket control the amount of material that can be excavated in a bite when the material is soft or loose. Dense or hard material may not be easily removed and several bites with the clam may be necessary to fill be bucket.

SLIDE 26

The basic differences in the clam buckets used for the element wall are the shapes and that a separate type bucket is used for the primary and secondary excavations.

SLIDE 27

The primary bucket takes a circular bite. The diameter depends upon the depth of the excavation and the size of the temporary casing to be installed. These buckets were used to reach top of rock but were not able to excavate the hard limestone.

SLIDE 28

The secondary bucket was fabricated to use the permanent casing of the primary elements as guides to remove the web of insitu material and annular space grout holding the primary casing in place.

SLIDE 29

The secondary bucket served as a chopping bit and had a very limited capacity.

SLIDE 30

Typically when material was encountered that was too hard or massive to remove with a clam bucket, it would be broken by using a star chisel. These are simple percussion tools that are suspended on a cable and dropped on the bottom of the excavation. Typically these will weigh between 5 and 20 tons. Obviously, this would be a very slow, tedious process, but in some instances it is the only means of advancing an element. Once the material is sufficiently broken it can be removed with a bucket.

SLIDE 31

Large diameter drills were used to excavate rock from the primary elements at both Wolf Creek and Manicogan III. At Wolf Creek, a reverse circulation shaft drill was used. At Manicogan III, the Contractor used a nested - self guiding percussion drill. A smaller in shaft was advanced and left in place. Then the larger outer shaft would follow over the inner shaft. This allowed the Contractor to maintain verticality to extremely close tolerances throughout the excavation.

SLIDE 32

The mud plant must supply slurry to the excavation in sufficient quantity to keep the excavation filled at all times.

SLIDE 33

Mud plants can range from a simple mixing plant and mud pit

SLIDE 34

to a sophisticated mixing and storage system. Each must be designed to meet anticipated needs of the project and must be able to supply fresh mud at a fast rate in the event of a slurry loss during excavation.

SLIDE 35

The pumps must be of sufficient capacity to deliver the mud to the excavation because in most instances, the slurry will be mixed some distance from the actual excavation.

SLIDE 36

This will require supply lines from the plant

SLIDE 37

with access to the entire work area.

SLIDE 38

The removal of spoiled excavation is typically done with dump trucks

SLIDE 39

through a garbage bin and lift truck arrangement was used very successfully at Wolf Creek. Of importance is that the removal of spoil should not be at slower rate than it is produced,

SLIDE 40

otherwise, a quagmire of bentonite and spoil will quickly develop.

SLIDE 41

The contractor must provide enough trucks or containers to hold and haul the spoil as it is removed by the clam bucket.

SLIDE 42

A key feature to the wall installation are the guide walls or guide trench which is installed along the entire alignment of the proposed wall.

SLIDE 43

This serves as the initial guide for the clam bucket to start the excavation vertically at the correct station. It serves as a work platform to maintain the shape of the excavation by preventing the bucket from wallowing out the edges during the numerous trips in and out of the trench.

SLIDE 44

The dimensions of the wall guide according to the requirements of the project, though typically a cross-section 1 foot wide and 2.5 feet deep is usually sufficient for most panel walls. Thee are given as minimum dimensions and the Contractor is allowed to increase the section to suit the needs of his equipment. The guide walls are usually the width of the final wall.

The excavation of embankment/overburden materials is often started dry (without bentonite slurry) to a depth where slurry can be added and contained in the excavation with the clam in place. The slurry is added directly into the excavation and maintained within a specified distance of the top of the guide wall. This is critical to maintain the stability of the excavation at depth. Allowing the slurry to drop may result in cave-in of less stable materials. Slurry must be added more or less continuously to compensate for the materials removed. The slurry has been designed to provide maximum support of the excavation while minimizing penetration through the embankment/ overburden materials.

Rock excavation in a panel wall is similar to the embankment/overburden excavation if the rock is soft enough to be removed by the clam bucket.

SLIDE 45

Depending on the nature of the materials below "Top of Rock" the slurry may become contaminated with cuttings such as sand and crushed rock and will often require desanding to maintain specified properties of density or sand content.

Where hard materials are encountered, a star chisel can be used to break or crush the rock for removal with the bucket. Where top of rock is highly irregular the chisel may be used to create a notch to help the bucket penetrate or bite. This becomes more important when the kelly is used as the irregular surface can cause enough stress to bend of even break the kelly and/or its supports.

SLIDE 46

As noted with the "Wolf Creek" wall the rock in the primaries was drilled.

SLIDE 47

Another feature of cutoff walls which must be considered are the ends of the wall. In most cases, this is a point where the seepage path is so long that there is no danger to the structure. This may be the end of the embankment or a point in the foundation where no seepage paths exist. In the cases where the embankment abuts a concrete structure such as a gravity dam, powerhouse, or look at tie-in is required. The tie-in is simple. The panel is excavated to grade along the sloping face of the lock. The contractor scrapes the lock wall clean and pours the concrete panel. The resulting contact can be observed by

SLIDE 48

coring through the contact.

SLIDE 49

In this instance, the hole was pressure tested and no leakage was observed.

SLIDE 50

The major concern during the excavation of an element is hole stability. The slurry must be monitored at all times to detect even gradual losses. During the excavation process the level of slurry will drop as the hole is deepened which will require periodic addition of fresh slurry to maintain the specified elevation. Small gradual losses will be impossible to detect except when the excavation is left at a given depth for a period of time, such as over a weekend or during delays due to equipment failure. While small losses can be expected they must be monitored to be able to prevent the losses from increasing or progressing to a point that could jeopardize the stability of the excavation.

SLIDE 51

Such slurry control is part of the contractor's quality program and should be defined prior to starting excavation. Of greater concern is the partial or total loss of slurry into a void, cavity, or solution channel. Such a loss could have severe detrimental effects on both the embankment and overburden materials as well as on the rock foundation. The loss of the slurry could result in the collapse of the excavation due to lateral pressures. Since the length of the excavation is limited, the possibility of a failure causing a breach of the dam is highly unlikely unless an open channel was encountered that allowed piping of the embankment to occur. Most likely the loss of slurry would occur upon hitting a void where the slurry which would flush into the void. This would require immediate plugging

SLIDE 52

to allow restoring the slurry level before collapse occurs if possible. This can take the form adding additional slurry, cement, bagged bentonite, or spoil. The exact method may be trail and error until the contractor gets the loss under control. Of importance is that the contractor has personal on the job at all times with the experience necessary to react quickly and decisively to control the loss. Once the loss has been controlled, measures must be taken to mitigate further losses when the excavation is continued past the point of loss.

SLIDE 53

This can be accomplished by drilling and grouting in the area of the loss. This defines the feature that caused the loss and treats the feature by filling the void, typically with a weak cement-bentonite grout which not only seals the void but also can be easily excavated when the element is advanced. The use of dumped spoil and grouting has been very successful in halting a slurry loss and allowing completion of the panel without additional slurry loss. However, the actual treatment of such a loss would depend upon the contractor's expertise and materials on hand at the time of the loss. Most contractors would utilize concrete as a last resort due to the difficulty that would be involved with excavating it once it had set. The major point concerning slurry loss is that when placing a cutoff in a water retaining structure with a functioning pool, we rely upon the experience and expertise of the contractor to respond in such a manner as to minimize damage to the structure and eliminate as much risk as possible. This is one of the major reasons why contractors have been pre-qualified to bid on the Corps projects mentioned earlier.

SLIDE 54

The key to the cutoff wall is the concrete that is placed to form the barrier against seepage and piping. By the very nature of the construction technique, the concrete must be placed at some considerable depth, through bentonite slurry in a more or less continuous operation. This has led to an adaptation of tremie concreting.

SLIDE 55

From mix design, to batching, to placement, the goal is a panel of concrete with as little contamination, honeycomb, or segregation as possible. There are several factors which affect the quality of the completed panel, the excavation preparation, the tremie procedure, and the quality of the concrete.

SLIDE 56

The excavation must be introduced through the screen.to catch large lumps.

SLIDE 57

The size of the tremie pipe depends upon the size of the aggregate used and the width of the panel. Typically, 3/4 inch maximum diameter coarse aggregate is used and a 10 inch diameter tremie has proved to be best suited for that size aggregate. The pipe is in convenient lengths - usually 10 feet - with threaded, watertight connections. Centering guides have been used to center the pipe in the excavation but these have not been proved as mandatory, especially in the panel type wall.

SLIDES 58-59

The crane is used to raise and lower the tremie and hopper during and between placement to aid in the flow of the concrete. This must be done with care as contaminants are sometimes drug into the concrete by the couplings on the tremie or by wallowing out the stiffened concrete near the concrete slurry contact. The crane is also used to maintain the pipe embedment in the concrete within the specified depths.

SLIDE 60

When the placement has reached an elevation requiring removal of several sections of pipe, the pipe is dogged off to remove the hopper. Then the amount of pipe necessary is pulled, the string dogged off, and the exposed pipe is removed. The hopper is then replaced and the pour is continued. This operation must be carefully monitored to insure that the bottom of the tremie pipe is maintained below the surface of the concrete at the minimum specified embedment, otherwise a loss of seal will occur which will cause numerous problems with continuing the placement which will be addressed later. The last piece of equipment that should be considered is the traveling plug or go devil. This is the device used to keep the concrete from mixing with the slurry while placement is starting and the tremie pipe is full of slurry. Some plugs that have been used are basketballs, beach balls, wood spheres, concrete balls, grout slugs, and cylindrical pigs with rubber wipers.

SLIDE 61

Some contractors have used nothing at all, plugging the hopper orifice with a shovel, filling the hopper and then pulling the shovel. Of all these methods, only the wood sphere and the "pig" are consistently effective.

SLIDE 62

The rubber ball collapses under the pressure. Some, which have been recovered, are about 50 percent smaller indicating their ineffectiveness as a separating medium.

SLIDE 63

The concrete balls tend to break up and are not considered consistently effective. The grout slug and shovel start will result in mixing in the tremie and increase the possibility unacceptable materials being caught in the placement. The solid, retrievable traveling plug insures that the concrete is not cut by the slurry while the pipe fills and reduces the amount of mixing that will occur when the concrete is allowed to flow out of the tremie. The typical operation is to set the tremie to the bottom of the excavation and then raise it enough to keep the plug in the pipe and still let the slurry escape.

SLIDE 64

The concrete is then added to fill the pipe, at which point the pipe is raised approximately a foot to allow the plug and the concrete to escape. Concrete is then added to the hopper at a rate to eliminate or minimize the amount of free fall to the surface of the concrete in the pipe. Some mixing will occur when the tremie pipe is lifted, however, this will be minimal compared to the other methods.

SLIDE 65

The number of tremies used depends upon the length of the excavation and varies from one to

SLIDE 66

three. The governing factor is that the concrete should not travel more than 7 1/2 feet laterally.

SLIDE 67

Most excavations require two tremies working together. The concrete is started simultaneously and both tremies are fed at rate to keep the surface of the concrete rising uniformly.

SLIDE 68

A weighted line is used to sound the top of the concrete to insure a uniform rise and assure that the bottom of the tremie pipes are maintained within the specified embedment ranges. Due to the physical limitations of the work area and the need to remove pipe, it is impossible to place the concrete continuously and the pour will be interrupted periodically. At these times, the concrete in the tremie will reach a static level somewhat above the level of concrete in the excavation.

SLIDE 69

As the placement continues in a primary or running panel, the concrete in the lower portion will take its initial set. Each shoulder pipe is gradually raised to prevent it from becoming permanently embedded. This is accomplished using hydraulic jacks to raise the pipe(s) in small increments as the upper portion of the panel is being placed. This is continued after the placement is completed until the entire panel has sufficiently to permit complete withdrawal of the shoulder pipe(s).

SLIDE 70

The jacks can be used with a ring with wedges to grip the casing as the jacks lift the ring

SLIDE 71

or with a slotted upper section with a cross beam to jack against.

SLIDE 72

When the placement is nearing completion, a scum of slurry and concrete called laitance will appear in the trench.

SLIDES 73-74

If the placement has been well controlled, the amount of laitance will be small and fresh concrete will appear quickly. This concrete will not be the first concrete placed as that material will have started to set before the placement is complete and will stiffen causing fresh concrete to flow past until the bottom of the tremie is pulled above it. This has been demonstrated by dying separate batches of concrete different colors and then coring the completed element. This has shown that the concrete is not forced uniformly up from the bottom but that it flows in stages sometimes displacing and sometimes bypassing. This is important because less dense contaminants can be trapped and give zones of poor quality concrete at depth with good quality concrete above. This is especially true when a loss of seal occurs, either due to a plugged tremie or because the tremie pipe was raised above the surface of the concrete. At that point, slurry and laitance are often swept into the upper reach of the concrete by the rush of concrete out of the tremie. The pipe then fills with slurry. If the pipe is reinserted into the fresh concrete and the pour restarted, the slurry in the tremie will be forced into the concrete contaminating it. If the pour is started above the surface and then the tremie is inserted, the contaminated zone may stiffen and set in the panel and not float out. The two best methods are a cold joint and a valved tremie. A valved tremie plugs the pipe to prevent the slurry from entering until the tremie is embedded in the fresh concrete. This has the problem of buoyancy which makes this impractical at depth. The cold joint method requires removal of the contaminated concrete in the upper reach and allowing the rest to set. Then the pour is treated as an initial start with a traveling plug.

The last major factor affecting the quality of the panel wall is the concrete. Concrete to be placed as outlined above must have specific characteristics to insure a high quality product in place.

SLIDE 75

Tremie concrete must have high slump, be cohesive, and still flow readily. General guidelines for mixes are included here but the actual mix design must be specific for the project and materials to be used by the contractor. In general terms, the concrete should:

SLIDE 76

1 - Be flowable - that is travel through the tremie and then fill the excavation completely.

2 - be cohesive - not segregate during placement.

3 - be stable - resist mixing with the slurry.

4 - Stiffen at a rate which will allow removal of shoulder pipes and still maintain the shape of pipe form.

The fluidity of the concrete depends upon the cement content and the water/cement ratio. A too high water/cement ratio would promote segregation while a low water/cement ratio would decrease flowability. A low cement content with a good water/cement ratio, while being less expensive, would not flow good and would tend to segregate. A good balance is on the order of 6 to 7 bags of cement per yard with a water/cement ratio near 0.5. Pozzolan has been added to reduce the cement content, decrease segregation, increase flowability, and increase long term strength. Another factor is the shape and size of the coarse aggregate. Most contracts to date have specified 3/4 inch maximum aggregate. Natural aggregate, well rounded, increases flowability and allows use of less cement than an angular manufactured aggregate.

The concrete mix typically is composed of type I or II cement, type F pozzolan, fine aggregate, coarse aggregate, water, and the following admixtures: Air-Entraining, accelerator, water reduces or retarders. Typically the specifications will give certain requirements as to the mix design while still giving the contractor the latitude to adjust the specific mix to accommodate the materials he plans to use. An example of this is the concrete specification for the Clemson Lower Diversion Dam which provided the following requirements:

1. Cementitous material - equivalent volume mix of 564 pounds of cement allowing 20 to 30 percent pozzolan.

2. Maximum water cement ratio of 0.5.

3. Fine aggregate shall be approximately 45 percent of the total aggregates.

4. Air content of 5.0 percent $\pm$ 1.5 percent.

SLIDE 77

5. Slump of 7-1/2 $\pm$ 1 inch.

SLIDE 78

These give the contractor the latitude to develop the best mix that meets the requirements while taking advantage of his expertise. This concrete can be produced by local ready-mix companies or if the job warrants - by an on site batch plant. The mix is typically delivered by truck which allows efficient placement in a timely fashion.

Since, in most applications, the cutoff wall will not be exposed for inspection,

SLIDE 79

the quality of the final product can be observed in selected areas with core borings taken in some of the panels. This, though slow and expensive, is necessary to indicate the quality of the contractor's product.

Unfortunately, this can be done only after the concrete has attained sufficient strength to be cored without damage from the coring operation. Thus the placement operation will be well ahead of the coring and any changes in the operation indicated will be belated. One means of minimizing the effects at the start of a job is to require a test section in a non-critical area to allow the contractor to work out his procedures and, at the same time, provide in-place concrete that can be cored before mass production is in full swing. Then, if major changes are warranted, they can be made early in the project before critical areas are reached.

Core borings are also very useful in examining the concrete in panels that had any difficulties or unusual events during placement. This could be panels that experienced a loss of seal during placement or had questionable quality concrete such as a borderline slump or water/cement ratio. Random sampling of panels is necessary throughout a job but sampling is manditory for each panel that experienced problems during placement. Good quality borings can be recovered for the entire depth of any panel if proper care is exercised in setting up the drill rig to insure the drill string is vertical and to insure that the rig does not move or settle during drilling. The drilling must not be pushed to get high production rate as this often encourages the hole to deviate and exit the side of the panel before the bottom is reached.

As mentioned earlier, the construction of the cutoff wall must be monitored closely throughout the construction sequence. This not only includes the excavation but also associated instrumention such as piezometers and inclinometers, surface monumentation, relief drains, and any areas that can be visually inspected. Piezometers should be monitored regularly, from 1 to 4 times a day, depending on their location and responsiveness. A great deal of information about seepage paths can be learned by observing changes in downstream piezometers as the wall is constructed. Sometimes this may be a gradual drop in piezometric level over a long period of time. Often, however, rapid, distinct changes can be observed.

The final proof of the cutoff is the normal operation of the project with the panel wall functioning as designed or predicted. As most of the walls have been installed to cutoff seepage, the effectiveness of the wall is seen by a drastic drop in seepage, where observed, or in marked changes in piezometric levels after construction. While long term observation will verify the ultimate effectiveness of the cutoff, the rapid response seen at many projects are good indications that the panel walls are very effective seepage barriers. However, the ultimate effectiveness of any wall is determined by the care exercised by the contractor and rigid adherence to the specifications. This work is inherently sloppy but this should not lull one into a sense that extra care is not necessary. Slurry levels must be monitored constantly and kept within specified limits. We have record of one instance where the slurry level was allowed to fall over 30' in a panel which resulted in a partial collapse of the excavation. Almost every instance of loss of the tremie seal can be directly attributed to improper monitoring of the depth to the concrete during placement which resulted in the crane operator lifting the tremie above the concrete/slurry interface. Fortunately, most of these problems have occurred in noncritical areas or at high elevations in wall so the effectiveness is not diminished. However, undetected and untreated defects in a critical area could greatly diminish the effect of the wall, and could result in later costly repairs.

The initial walls were relatively expensive compared to conventional forms of treatment. However, with our recent bidding experiences, it appears that the trend is toward lower costs per square foot of wall making this remedial work even more cost effective.

SOIL-BENTONITE SLURRY TRENCH CUTOFF
WEST POINT DAM
JOE ROGERS-SAVANNAH DISTRICT

SLIDE 1.

The slurry trench constructed as a part of the underseepage protection for the Alabama (west) Earth Embankme nt at the West Point Project was the first soil bentonite slurry trench constructed within the South Atlantic Division. It is one of the first slurry trenches in the United States to be constructed through residual soil.

SLIDE 2.

The West Point Project lies on the Chattahoochee River approximately 3 miles upstream of West Point, Georgia. The site is located on the Alabama/Georgia state line. The town of West Point is located approximately 74 miles southwest of Atlanta.

SLIDES 3-6.

The West Point Project is a multi-purpose project consisting of a concrete gravity dam and earth embankments. The concrete section is approximately 1,006 feet long. The earth embankment has the following configuration:

Georgia Embankment	900 feet long
Alabama Embankment	5,200 feet long
Top of Dam	652 m.s.l.
Stream bed	555 m.s.l.
Maximum Height	75 feet $\pm$
Crest Width	25 feet
Upstream Slope	IV on 3H
Downstream Slope	IV on 2.5H

The project is located in the Piedmont Plateau Physiographic Providence. The maximum relief at the site is approximately 150 feet.

SLIDES 7-8.

There are a number of items of geologic significance which apply to the construction of the West Point slurry trench.

Moving from the river toward the west abutment, the first unit encountered is a deposit of recent floodplain alluvium. This zone is approximately 30 feet thick and has a permeability ranging from 0.03 to 70.0×10^{-4} feet per minute. Groundwater in this area ranges from about .5 to 17 feet deep. There is no artesian pressure in the floodplain alluvium.

Westward of the floodplain alluvium a deposit of ancient floodplain alluvium is encountered. This deposit is 15 to 20 feet thick and has a permeability ranging from aout 40 to 264x10 feet per minute. Groundwater in this area is approximately 20 to 35 feet deep and there is no artesian pressure.

In the area of the ancient floodplain alluvium and westward, residual soil is encountered. This material has a permeability in the ragne of 13×10^{-4} feet per minute. Groundwater in this area is also approximately 20 to 35 feet deep.

The contract for excavation of the slurry trench required the trench to be excavated into weathered rock. Weathered rock was defined in the contract as, "material below soil and above sandrock which refuses 1 feet of penetration by a splitspoon sampler driven by 100 blows of 140 pound hammer falling 30 inches."

SLIDE 9.

During the initial stages of the project design it was determined that under-seepage control will be required for the Alabama Embankment. Based on permeability data it was decided that some type of seepage cutoff was necessary. Two options was investigated:

a. A conventional cutoff trench with dewatering.
b. A slurry trench.

Investigations indicated that a soil bentonite slurry trench would be the less costly of the two alternatives. In considering use of the slurry trench, designers also anticipated that certain parameters must apply to the trench.

SLIDE 10.

For example, in order to reduce settlement and to insure stability of the embankment, it was considered necessary that the trench be located upstream of the embankment. The slurry trench was intended to extend from the bottom of the cutoff trench to "top of rock." The bottom of the cutoff trench was to be governed by the location of the water table in the area.

SLIDES 11-12.

Since the slurry trench was the first constructed by the Savannah District, it was determined that a test section would be appropriate prior to proceeding with design and development of slurry trench plans and specifications. The test section was conducted during July and August of 1965. The purpose of the test section was to determine if a dragline could excavate the ancient alluvium and residual soils and to further evaluate whether trench backfill could be improved by the addition of Portland cement. For construction work a 50 ton P and H mobile crane was utilized. The crane had a three quarter yard bucket. It was believed a large crane with a small bucket would have better success in excavating the weathered rock. Slurry with a viscosity of 15 to 20 centerpoises was to be introduced once the water table was encountered.

Construction of the test section begin in July of 1965. After excavation proceeded for a few days the work was stopped for the weekend. The groundwater table had been encountered and excavation had proceeded a few feet below the water table without slurry being introduced to the trench. The following Tuesday a slope failure occurred in the trench. The failure occurred along an ancient joint in the weathered rock material. This experience indicated that the slurry needed to be introduced into the trench, soon after or before the groundwater table was encountered.

Overall, the test section was successful. It indicated that a trench could be successfully excavated through the site material with a dragline. However, it did indicate that the bucket use must be heavy (large) and that the bucket must be properly rigged in order to dig effectively. Rigging to the back of the bucket, rather than on the sides, as is typical in dragline work, was considered necessary for efficient excavation. The test sections also indicated that slurry must be kept near the top of the trench. The experiment to evaluate the effect of the Portland cement on the slurry trench backfill indicated that the Portland cement made the mix brittle, hard to work, and tended to make the backfill honeycomb.

Based on the results of the test section, plans and specifications were developed and a construction contract was awarded in March 1966. Construction began in July 1966 and was completed in January 1967. The contract was accomplished by Holiday Construction Company of Greenville, Georgia. The slurry work was subcontracted to Industrial Engineering and Equipment Corporation of Los Angeles, California. The cost of the slurry trench was approximately $3.00/square foot. Overall length of the trench was 3,250 feet and the trench had a maximum depth of 60 feet. The procedure for excavation of the slurry trench was to begin in the floodplain and work toward the high ground on the right (west) abutment.

SLIDES 13-16.

The procedures used were those which have become typical for the slurry trench excavation. Excavation was initiated with a dragline and a bentonite slurry was introduced into the trench. As the excavation proceeded the base of the excavation was cleaned up by utilization of an air lift. After the air lifting was accomplished, backfilling of the trench began. Initial backfill was placed using a clamshell with the backfill being placed into the trench until it rose to the top of the slurry. Subsequent backfill was placed by bulldozing slurry progressively into the trench causing the material to fail and slump down into the trench. A minimum of distance 100 feet was specified between the toe of the backfill and the toe of the excavation surface. Onsite materials were used for backfill of the trench.

SLIDE 17.

The backfill consisted of a natural clayey gravel with coarse gravel and sand added in order to reach proper gradation. The gradation and properties of the bentonite slurry are contained in Table I, a copy of which is attached.

SLIDES 18-22.

Equipment used for the excavation of the slurry trench consisted of a slurry plant, excavating equipment, trench cleaning equipment, and slurry cleaning equipment. Specific equipment utilized is contained in Table II.

SLIDES 23-25.

Of particularly interest in the excavation of the slurry trench is the fact that a special dragline bucket was fabricated for the job. The bucket had teeth for use in normal excavation and an interchangeable scraper blade which could be used in the latter stages of digging to help clean up the bottom of the trench. The bucket initially weighed 3,800 pounds. However, during excavation it was determined that a heavier bucket was necessary and weights were added to the bucket to bring its final weight up to approximately 5,700 pounds.

SLIDE 26.

Trench cleaning was accomplished by use of a 6-inch air lift supplied by 600 cfm compressor. This device proved to be a delaying factor in the construction progress and subsequently an additional 4-inch air lift supplied by 400 cfm compressor was added to expedite the construction operation. Construction procedures consisted of excavation, cleaning and backfilling of the trench as indicated previously. Careful testing was conducted on the slurry to determine and maintain its properties.

SLIDES 27-30.

The bentonite was brought to the site in bags. The bentonite was introduced into a flash type mixing system and the resulting slurry stored in ponds until the completion of hydration. The slurry was placed in the trench from the ponds.

SLIDES 31-32.

Soundings and samples were taken within the slurry trench to assure the proper depth was obtained.

SLIDES 33-36.

Test performed included; viscosity measurements by use of the Marsh Funnel, filtration measurements using the filter press, density testing utilizing a mud balance and cation exchange capacity using the methylene blue absorption test. Production rate for advance of the slurry trench was generally controlled by the production rate of the air lift.

SLIDE 37.

As indicated in Table III the excavation capability in recent alluvium was 20 linear feet per day. Twenty-nine linear feet per day could be backfilled and mixed. However, only 14 linear feet per day could be cleaned using the 6-inch air lift. Subsequently, an additional air lift was added and in the residual soil the production rate increased. The cleaning rate increased to 24 linear feet per day as an average whereas excavation rates remained approximately the same as in the recent alluvium, that is 23 linear feet per day. Mixing and backfilling for the residual soil could be done at approximately 50 linear feet per day.

SLIDES 38-39.

The contractor stockpiled backfill adjacent to the excavation during the contract for mixture with slurry in order to expedite the backfilling operations. Extensive testing was done on the slurry both prior to placement in the trench and in the trench.

SLIDE 40.

Table IV indicates the average properties of slurry in the trench.

SLIDE 41.

Table V shows a summary of slurry test results during the construction contract. An evaluation of the effectiveness of the slurry trench was conducted. A boring was taken in the slurry trench and into the underlying weathered rock materials. This boring indicated a good clean contact between the slurry trench backfill and the weathered rock zone.

SLIDE 42.

During the conducting of the boring undisturbed samples were taken. Copies of test results performed on the undisturbed samples are contained in test data summary which is included in Table IV.

SLIDES 43-44.

The overall design included a grout curtain placed beneath the slurry trench. It was decided that grout curtain would be installed prior to excavation of the trench. During drilling of grout holes the top of weathered rock and top of sound rock elevations were obtained. This data is contained in the plot shown on the slide attached to this talk. Also indicated on this slide is the bottom of the slurry trench. With the equipment used, it was generally possible to excavate into the top of weathered rock; however, in only a few instances did the slurry trench extend to the top of sound rock. This experience is consistent which has occurred in the Savannah District subsequent to the West Point slurry trench, that is, excavation equipment can typically excavate materials which give split spoon refusal but cannot penetrate more than several feet into these materials without difficulty.

A pump test was conducted in order to determine the effectiveness of the slurry trench. The pump well was placed approximately 9 feet downstream of the trench. Piezometers were installed both upstream and downstream. When the pump test was run it indicated drawdowns ranging from approximately 2.4 to 5.1 feet, depending on the location relative to the pumping well. After the slurry trench passed the well, the pump test was again performed. After pumping it was noted that there was no response to the piezometers located upstream on the cutoff wall even though the wall did not penetrate totally into the weathered rock zone into which the pump and well was installed. These results indicated that the trench had formed an impervious barrier.

SLIDE 45.

After completion of the slurry trench an evaluation was made of test data, the effectiveness of the excavation equipment, and other considerations related to the slurry trench. This evaluation is contained in a report entitled, "Construction of Slurry Trench Cutoff." This report was distributed to all Corps of Engineers offices in May 1968. The conclusions and recommendations reached as a result of the construction of the West Point slurry trench are as follows:

a. The slurry trench cutoff wall was an effective and expedient method of controlling underseepage for the west earth embankment.

b. The equipment utilized was suitable for excavations up to approximately 60 feet in depth; however, 30-feet depth was considered optimum for the equipment used.

c. The construction indicated it was necessary to have experienced and trained personnel familiar with the techniques utilized in order to have effective control of operations during construction.

d. As a result of the project it was recommended the following properties be maintained for the slurry:

   (1) At introduction into the trench viscosity greater than 15 centipoises as measured by Marsh Funnel reading of approximately 40 seconds.

   Water loss of less than 15 cc in 30 minutes.

   (2) At time of placing backfill into the trench, the slurry should have a water loss of less than 20 cc in 30 minutes.

e. It was determined that a processed backfill gives a more uniform job. A slump of approximately 4 inches for backfill was considered to be a reasonable and workable range.

f. It was also recommended that curing time of approximately 2 weeks be allowed to compensate for settlement of the backfill prior to initiating construction fill operations above the slurry trench.

g. When utilizing an air lift it was determined that right angle bends should be avoided and replaced by a general sweep or "el." This will help prevent hanging up of the material to be discharged.

SLIDE 46.

Overall, the construction of the West Point slurry trench was effective and techniques developed and procedures learned during this process have served as a basis for subsequent slurry trench work and for modification as techniques have improved.

TABLE I

MATERIAL PROPERTIES

BACKFILL - NATURAL CLAYEY GRAVEL WITH COARSE GRAVEL AND SAND ADDED

GRADUATION

SLEVE SIZE	PERCENT PASSING
3 INCH	100
3/4 INCH	45-100
NO 4	30-70
NO 30	20-45
NO 200	10-25

SLURRY - BAROID BENTONITE - MINIMUM 20 LB/BARREL OF SLURRY

AT TIME OF INTRODUCTION INTO TRENCH	UNIT WEIGHT 64.5 PCF MINIMUM VISCOSITY >15 CENTIPULSES AT 20°C WATER LOSS <25 CC IN 30 MIN
IN TRENCH	UNIT WEIGHT <80 PCF VISCOSITY <30 CENTIPULSES
MIXED WITH BACKFILL	UNIT WEIGHT <75 PCF WATER LOSS <30 CC IN 30 MIN

TABLE II

EQUIPMENT

SLURRY PLANT	MIXER, INTERMEDIATE SUMP, STORAGE SUMP, PUMPS, SUPPLY PIPES MIXER CAPACITY: 500 GPH WATER DELIVERY 7.5 TONS/HOUR BENTONITE
EXCAVATION	60 TON AMERICAN CRANE W/DRAGLINE, 110 FOOT BOOM BUCKET (SPECIALLY FABRICATED) 2 CY WITH INTERCHANGEABLE TEETH AND SCRAPER BLADE REAR HOIST CHAINS INITIAL WEIGHT - 3,800 POUNDS FINAL WEIGHT - 5,700 POUNDS
TRENCH CLEANING	6" SECTION AIRLIFT WITH 2" SUPPLY 600 CFM COMPRESSOR ADDED 4" AIRLIFT WITH 400 CFM COMPRESSOR
SLURRY CLEANING	10" INCH KREBS CYCLONE 400 GPM CAPACITY

TABLE III

Production Rate

Length - 3250 ft.
Max Depth - 60 ft.

Recent Alluvium - Airlift governed production rate

Capability Cleaning: 14 LF/day (6" Airlift Only)
Excavation: 20 LF/day
Mix & Backfill: 29 LF/day

Residual Soil/- Added 4" Airlift, Max Depth Reached in Residual Soil - 60 ft
Ancient Alluvium

Capability Cleaning: 24 LF/day
Excavation: 23 LF/day
Mix & Backfill: 50 LF/day

TABLE IV

Average Paroperties Of Slurry In Trench

Type Test	Alluvial Area	Residual Area
Viscosity Funnel (Seconds)	56 (20 to 25 Centipoises)	70 (25 to 30 Centipoises)
Density (pcf)	70.7	71.7
Sand Content (% by Volume)	8,9	13.1
Bentonite Content (lbs/Barrel)	18.8	18.1
Filtration Loss (cc In 30 Munites)	11.6	13.1
Filter Cake (32nd Inch)	2	2

TABLE V

Summary of Slurry Test Results

Depth Below Surface	Viscosity Funnel (Seconds)	Density (pcf)	Sand Content (% by Volume)	Bentonite Content (lbs/Barrel)	Filtration Loss (cc in 30 Minutes)	Filter Cake Thickness (32nd Inch)	Number Of Tests
			Alluvial Area				
Fresh slurry	48	64.7	–	19.4	12.3	2	
5'	55	69.8	7.7	19.3	11.6	2	91
10'	67	73.3	13.2	15.7	12.3	2	10
15'	55	70.8	8.9	19.1	11.3	2	80
20'	64	73.6	13.8	16.7	12.1	2	15
			Residual Area				
5'	71	71.5	12.3	18.4	12.9	2	60
10'	70	71.7	12.6	17.7	12.8	2	57
15'	71	70.8	14.6	18.3	13.4	2	36
20'	67	72.0	13.2	18.0	13.1	2	58
25'	76	73.6	14.0	18.0	13.6	2	17

TABLE VI

Summary Of Test Data

Boring Number	Location	Sample Number	Depth (Feet)	Dry Density (pcf)	Moisture Content (%)	Atterberg Limits		
						LL	PL	PI
SG-6	Station 44+50	1	3.0-4.3	98.8	24.5	54	20	34
SG-6	Station 44+50	4	12.3-14.7	99.2	26.2	49	18	31
SG-6	Station 44+50	8	23.8-26.3	98.0	22.8	51	17	34
SG-6A	Station 44+54	1	30.0-32.0	91.2	28.6			
SG-6A	Station 44+54	2	32.0-34.0	90.7	29.7			
SG-6A	Station 44+54	3	34.0-36.0	90.4	30.2			
SG-6A	Station 44+54	4	36.0-38.0	97.2	25.1			

Boring Number	Sample Number	Depth (Feet)	Triaxial Compression Test						Compression Index	Perm-eable (10^{-4} cm/sec)
			Q		R		R			
			0	C	0	C	0	C	CC	
SG-6	4	12.3-14.7	0.0'	0.08 T/SF					19	
SG-6	4	12.3-14.7			9.0'	0.20 T/SF	33.0'	0		
SG-6	4	23.8-26.3							.22	.0005

CONCRETE CUTOFF WALL
WOLF CREEK DAM
MARVIN SIMMONS - NASHVILLE DISTRICT

SLIDE 1.

Wolf Creek Dam is located in South Central Kentucky, to the east of the Cincinnati arch in the highland rim section of the interior low plateau.

SLIDE 2.

It is founded, for the most part, on Ordovician limestones with short sections of the embankment founded on Chattanooga shale and Ft. Payne limestone and shale.

SLIDE 3.

The concrete section ties into the left abutment with the embankment extending across the broad flood plain to the right abutment. There was about 50 feet of alluvium overlying rock near the river channel.

SLIDE 4.

The concrete section is about 1700 feet long with a 6-unit powerhouse. There are 10 radial spillway gates. The earth embankment is about 3900 feet long and is homogeneous rolled earthfill. The embankment is about 205 feet above top of rock with maximum height of concrete above founding level of 258 feet. Foundation work for the embankment was completed prior to World War II.

SLIDE 5.

After about 15 years of successful operation, some muddy water showed up in the tailrace in the fall of 1967. On March 13, 1968, a sinkhole developed near the downstream toe of the embankment.

SLIDE 6.

It was in an area of hand-placed riprap and was about 3 feet in diameter with the top of disturbed material about 10 feet down.

SLIDE 7.

After about a day, it had enlarged to about 8 feet in diameter.

SLIDE 8.

The embankment had been constructed with a cutoff to top of rock. It tied into the upstream corner of the concrete portion of the dam and extended upstream and landward. The reason for the cutoff position is not fully understood but it is believed that a system of solution features was followed in the bedrock when making the excavation for the cutoff.

SLIDE 9.

This is a view of the solutioned limestone near where the cutoff was to tie into the concrete monolith. This area is about 40 feet below top of rock and contains numerous caves and solution features.

SLIDE 10.

Further out the cutoff trench, the bottom steps up to the top of rock. Note that the solution channel filling material had to be bulkheaded to keep it from flowing into the 10 foot wide cutoff trench.

SLIDE 11.

The openings marked 15 and 16 are entrances to caves. These caves were not fully explored or treated at the time of construction. Note the condition of the side slopes of the trench.

SLIDE 12.

This is a solution feature being cleaned - note the overhang it will be rather difficult to backfill beneath that overhanging rock.

SLIDE 13.

It appears that equipment compaction was all that the trench backfill received. The centerline of the trench is to the right of the ladder. Note the irregular, nearly vertical face behind the ladder.

SLIDE 14.

A profile along the centerline of the cutoff where the grout curtain was installed. The single-line grout curtain had windows.

SLIDE 15.

After the sinkhole, explorations were started and the seepage path established. Dye was used as a tracer. This is the same area that the muddy water was appearing.

SLIDE 16.

On April 22, 1968, another sinkhole appeared about 40 feet away from the first. A churn rig had installed the piezometer the day before with no indication of an impending collapse.

SLIDE 17.

About this same time, enough piezometers had been installed to draw these contours. Note the 650 contour in relation to the 660 berm of the dam. There was definitely a high-pressure ridge extending downstream from the crest at the end of the concrete. There was a steep gradient between the location of WO-1 and the area of the sinkhole.

SLIDE 18.

It was decided to isolate the area with triple-line grout curtains extending out from the concrete for about 250 feet. Then construct another down the face of the dam for over 1000 feet. The contours shown here are a geologic interpretation of solution features in rock based on exploration.

SLIDE 19.

This is a geologic interpretation along one of the grout lines extending out from the concrete. It is based on 2.5 center holes so it can't be too far off. The grouting was all gravity type.

SLIDE 20.

After the grouting, the piezometric contours looked like this. You can see the high pressure ridge is gone with the 650 contour back upstream.

SLIDE 21.

It was decided that the grouting was not a permanent fix to the problem of Wolf Creek's foundation. The seepage forces were still at work and the erodible material still present in the solution features was still subject to piping. A more permanent fix such as a concrete diaphragm wall was decided upon. Exploration carried out along the proposed alignment produces a picture like this. The area in white indicated very soft or absent material. This represented a very real danger to the embankment.

SLIDE 22.

To install the diaphragm wall, it was first necessary to construct a work platform atop the 200-foot high earth embankment from the concrete out a distance of about 1000 feet. This platform was eventually extended a total distance of more than 2300 feet. That platform had to serve as the construction work area as well as carry U.S. Highway 127 traffic for a period of more than 4 years. The entire area was paved to provide easy movement of equipment.

SLIDE 23.

Two 9x9 foot guide walls were constructed to provide proper alignment at the top of Dam. A single-line actuated clamshell bucket was used to excavate a 52 inch diameter hole from the top down to about 70 foot depth. At this point, a thin bentonite slurry was added to help stabilize the hole.

SLIDE 24.

A 47 inch diameter casing is set in the bottom of the hole and secured with a casing driver. The hole is continued with a 45 inch clamshell to a depth of about 140 feet.

SLIDE 25.

Upon reaching 140 foot depth, a 42 inch diameter temporary casing is placed telescopically inside the 47 inch casing to the bottom of the hole and advanced to the top of rock.

SLIDE 26.

After the 42 inch casing is seated at top of rock, a rock boring machine is moved on to the hole.

SLIDE 27.

The machine used a roller rock bit to drill a 36 inch diameter hole into rock to a predetermined elevation below the karstic features in the limestone bedrock.

SLIDE 28.

The rock drilling used airlift assisted reverse circulation method. Upon completion of the rock hole, a 26 inch permanent casing is lowered to the bottom of the hole and the temporary casing removed.

SLIDE 29.

The outside of the casing is grouted in place and concrete is placed inside by the tremie method. This constitutes a primary element.

SLIDE 30.

After completion of two adjacent primary elements, a secondary element is constructed using the primaries as guides. The excavation is made with a small clamshell in the overburden and grout.

SLIDE 31.

In the rock portion of the element, the rock is chopped with a chisel and removed with a clamshell.

SLIDE 32.

After excavation is completed between two primaries, the hole is backfilled with tremie concrete. This procedure is followed until a continual wall is finished.

SLIDE 33.

It looks something like this in plan view.

SLIDE 34.

This is the single line actuated clambucket.

SLIDE 35.

The upper portion of the 52-inch hole in the embankment.

SLIDE 36.

The 47-inch temporary casing can be held by the casing driver.

SLIDE 37.

The 45-inch clam bucket operating in the 47-inch temporary casing.

SLIDE 38.

View of work on top of dam during work period.

SLIDE 39.

Some 42-inch casing being placed inside 47-inch casing. Note teeth for setting casing in top of rock.

SLIDE 40.

The temporary casings are held by keys set in a grove.

SLIDE 41.

Rock drill in place and ready to drill.

SLIDE 42.

One of the 36-inch diameter rockbits.

SLIDE 43.

Setting the permanent 26-inch casings.

SLIDE 44.

Tremie concrete being placed through a 10-inch tremie pipe.

SLIDE 45.

Secondary element. Note concave guides that ride the primary elements.

SLIDE 46.

Some of the rock chisels.

SLIDE 47.

Looking down into a secondary element. Note how clean are the primary elements.

SLIDE 48.

Recent piezometric contours.

CONCRETE CUTOFF WALL
CLEMSON DIVERSION DAM
HARTWELL LAKE, GA & SC
TED F. HIGHTOWER - SAVANNAH DISTRICT

SLIDE 1.

The Upper and Lower Clemson Diversion Dams are located in Pickens County, South Carolina, adjacent to the town of Clemson and the property of Clemson University.

SLIDES 2-3.

The dams were constructed in 1960 and 1961 as part of the Hartwell Reservoir project. Both dams are constructed across the Seneca River flood plain and consist predominantly of random earth fill with internal drainage provided by an inclined drain and a horizontal sand drainage blanket. The upper dam has a length of about 2,100 feet and the lower dam a length of about 3,000 feet. They were constructed to prevent flooding of Clemson University lands by the Hartwell Reservoir impoundment. A pumping station was constructed and is operated by the Government at the lower diversion dam to remove runoff and seepage from the protected area.

Crest elevations of both dams are about elevation +680 m.s.l. Normal pool elevation is about elevation +660 m.s.l., thereby providing about 20 feet of freeboard at each dam.

SLIDES 4-11.

Since their completion and before installation of the concrete cutoff walls in 1982-1984, the toe area of both dams had been troubled with seepage. This situation created maintenance and emergency repair situations. Emergency repairs were required to control boils. If these boils had been left uncontrolled, they had the potential for causing possible failure of the dams through piping, and for flooding of the protected area.

In the late 1970's money for the rehabilitation of the Clemson Diversion Dams was obtained through a Corps funding program which provided funds for rehabilitating existing structures.

Because of funding restrictions, the two Clemson Diversion Dams were repaired using two separate contracts. The lower dam was chosen for the first contract. The upper dam cutoff wall was designed while the lower dam contract was under way. This allowed us to refine the second contract based on our experience with the first.

Initially, two concepts were considered as potential remedial measures for the seepage problems at the two dams. The basic concepts were:

- Positive seepage cutoff

- Passive seepage control

Numerous methods and combinations of methods were evaluated, with most being determined to be inappropriate for this application for reasons other than cost. Passive seepage control considered:

- Relief wells

- Stability berms

- Continuous filter trench

After much consideration, the entire concept of a passive solution was abandoned in favor of the positive cutoff approach.

Once the design goal of providing a positive cutoff had been established, various methods for a positive cutoff were considered for installation within berm. Consideration for a positive cutoff on the downstream berm was abandoned on the basis that:

a. A positive cutoff there would have the undesirable result of negating any effectiveness of the existing internal drainage.

b. Piezometric levels along the berm were near the ground surface, indicating the potential for overtopping a positive cutoff at that location. This would require construction of an additional drainage system to collect and remove the flow.

SLIDE 12.

Postive seepage cutoff methods considered were:

SLIDE 13.

Conventional Slurry Wall; and Cement Grout.

SLIDE 14.

Chemical Grout; Piling - both driven and cast-in-place overlapping piling; and Panel Type Slurry Walls (conventional type) - One type, backfilled with lean concrete and another, backfilled with cement-bentonite.

SLIDES 15-18.

- Panel Type Concrete Cutoff Walls.

The panel type concrete cutoff walls were chosen for use at the Clemson Diversion Dams.

SLIDE 19.

The investigation program for these projects consisted of the following:

SLIDE 20.

- Standard penetration test borings

SLIDES 21-22.

- Bedrock Sampling - 4 x 5 1/2 inch core

SLIDE 23.

- Piezometer Program
- Laboratory Testing
- Pumping Tests
- Water Pressure Tests in Bedrock

SLIDES 24-27.

- Chemical Grout Test Program
- Comparison of Original Design to Results of Current Investigation
- Seepage Analysis
- Stability Analysis

One common design consideration for the Clemson Diversion Dams was that all construction was to be accomplished under full reservoir head.

Other particular design considerations and parameters are as follows:

Lower Diversion Dam:

- Dam degraded 5 feet to elevation +675 m.s.l. to provide work platform.

SLIDES 28-29.

- Cutoff walls extends from elevation +675 m.s.l. through all materials to elevations that were established on the contract drawings, with the following exceptions:

  a. If bedrock (granite gneiss) was encountered that could not be excavated with normal overburden excavation equipment at a higher elevation than indicated by the contract drawings, the cutoff wall was to be founded at the higher elevation.

  b. If bedrock (granite gneiss) had not been encountered at the elevations indicated by the contract drawings, the excavation was to be deepened and the cutoff wall founded nominally 3 feet into weathered bedrock (granite gneiss) or on bedrock (granite gneiss) that could not be excavated with normal overburden excavation equipment, whichever was shallower.

SLIDE 30.

- Wall Width - Nominally 2 feet

SLIDE 31.

- Maximum Panel Length Along Alignment - Nominally 25 Feet

- Approximate Maximum Panel Depth - 95 Feet

SLIDE 32.

- Concrete Placement - Tremie

SLIDE 33.

- Concrete Strength (28 Day) - 3000 PSI Minimum

SLIDE 34.

- Acceptance - Concrete strength plus core results

Upper Diversion Dam:

- Dam degraded 5 feet to elevation +675 m.s.l. to provide work platform

- Cutoff wall constructed along centerline of dam from Station 1+00 through 22+50 for a total of 2150 feet.

- Cutoff wall extends from elevation +675 m.s.l. through all materials and keys a minimum of 5 feet into bedrock (granite gneiss).

- Wall Width - Nominally 2 Feet.

SLIDE 35.

- Maximum Panel Length Along Alignment - Nominally 25 Feet

- Approximate Maximum Panel Depth - 95 Feet

SLIDE 36.

- Concrete Placement - Tremie

SLIDES 37-42.

- Concrete Strength (28 Day) - 3000 PSI Minimum

- Acceptance - Concrete strength plus core results

Slurry Limits:

- Bentonite Concentration - 6%

SLIDE 43.

- Minimum Density - 64.3 P.C.F. at 6 Percent

- Maximum Density While Excavating - 80.0 PCF

- Density Before Concrete Placement - 78.0 PCF

SLIDE 44.

- Apparent Viscosity - Minimum of 15 centipoise

- Plastic Viscosity - Not greater than 20 centipoise at the time of concrete placement.

- Gel Strength - Minimum of 15 pounds per 100 square feet.

SLIDE 45.

- Marsh Funnel - Minimum of 34 seconds

- Filtrate - Maximum of 20.0 cubic centimeters

- Alkalinity - pH of 8 - 11

SLIDES 46-48.

- Filter Cake - Less than 2 mm

SLIDES 50-54.

Concrete Proportions:

- Strength (28 Day) - Minimum of 3000 PSI

- Water-Cement Ration - Maximum of 0.50 by weight of equivalent Portland cement

- Cement Content

  a. Without the use of Type F or G Chemical Admixture - Volume of cement and pozzolan shall be about equivalent to a total of absolute volume of 658 pounds of cement per cubic yard (7 Bag mix).

  b. With the use of Type F or G Chemical Admixture - Volume of cement and pozzolan shall be about equivalent to a total of absolute volume of 564 pounds of cement per cubic yard (6 Bag mix).

In a and b above, of the total cementitious materials, about 20 to 25 percent may be pozzolan.

- Fine Aggregate - About 45% plus or minus 5% of the total aggregate volume meeting ASTM C 33

- Coarse Aggregate - Maximum nominal size - 3/4 inch meeting ASTM C 33, size 6

- Air Content - 5% plus minus 1.5%

- Slump - 7.5 inches plus or minus 1.5 inches

Bencor Corp. was the successful bidder for the Clemson Lower Diversion Dam contract. They were given notice to proceed in February 1982. Work on the cutoff wall was completed in December 1982. Work on the project was completed in June 1983. Their bid price for the project was $2,864,800.00. This includes all project costs for constructing 195,000 square feet of concrete cutoff wall surface area. This translates to $14.69 per square foot for construction of the Lower Diversion Dam concrete cutoff wall.

Soletanche and Rodio, Inc. was the successful bidder for the Clemson Upper Diversion Dam contract. They were given notice to proceed in August 1983. Work on the cutoff wall was completed in May 1984. Some work is remaining to be completed in the toe area of the dam. Their bid price for the project was $2,178,900.00. This includes all project costs for constructing 172,000 square feet of concrete cutoff wall surface area. This translates to $12.67 per square foot for construction of the Upper Diversion Dam concrete cutoff wall.

SLIDES 55-57.

The construction of concrete cutoff walls in the Clemson Diversion Dams has resulted in decreased piezometric heads at both sites. Before construction, the lower diversion dam recorded piezometric heads about 3 to 9 feet above the ground surface and the upper diversion dam had recorded piezometric heads about 3 to 6 feet above the ground surface. Since construction has been completed, no individual piezometer currently records a water level above the ground surface. Individual head decreases are measured up to 12 feet with the average being around 6 feet. Flows from relief well manifolds and French Drain systems at both dams also have shown significant decreases since construction has been completed. Additionally, we have been told verbally by the pumping station operators that they have noted significant decreases in the pumping demand since completion of the lower diversion dam cutoff wall.

CONSTRUCTION OF THE CLEMSON SLURRY WALLS
TED ZIELONKA - WILMINGTON DISTRICT

SLIDE 1

The Clemson Upper and Lower Diversion Dams were constructed in 1960 and 1961 as part of the Hartwell Reservoir Project. These random earth fill dams, along with a saddle dike, diversion channel, and pumping station, prevent flooding of Clemson University lands by Hartwell Reservoir. Since the impoundment of Hartwell Lake in 1962, both dams experienced heavy seepage. On the Lower Dam, which experienced the more serious seepage problems, a Portland cement grouting program was carried out with no notable success. Relief wells and graded sand and gravel filters were installed in both structures over the years in various attempts to control the seepage. Several emergency repairs were made to control boils which might have caused dam failure through piping. In 1979, after the development of several boils associated with high lake levels, major rehabilitation was deemed necessary. Through extensive geotechnical exploration and testing from 1979 to 1981, the problems were attributed to the fact that both dams had been founded on flood-plain alluvium without adequate seepage cutoff. It was decided to correct the problem using 2-foot wide concrete panel-type slurry walls, installed through the embankment and alluvium into the bedrock below each dam.

A $2.9 million contract for the Lower Dam panel wall was awarded in late 1981. Actual construction of 195,000 sq. ft. of wall took place in 1982, with reconstruction of the dam completed in 1983.

SLIDE 2

A $2.2 million contract for 172,000 sq. ft. of slurry wall for the Upper Dam was let in 1983. Construction began in the fall of 1983 and site restoration was completed in late 1984.

SLIDE 3

Site work prior to wall construction took 6-10 weeks on each dam. Since the crest width of each dam was only 16 feet, the main element of the site work was the construction of a working platform. The top of the embankment was degraded 5 feet, and the degraded fill material was used to build a wedge on the downstream side of the dam. The riprap from the top 5 feet of the dam was pushed to the upstream side in another wedge.

SLIDE 4

This provided a 60 to 65-foot wide flat platform for the construction equipment. The contractor for the Lower Dam wall decided not to construct a stone-surfaced haul road across the working platform.

SLIDE 5

Construction traffic was routed over the exposed embankment fill alongside the panel wall excavation.

SLIDE 6

However, within a few weeks this route became impassable, with 2-3 feet of mud overlying deep ruts.

SLIDE 7

The Upper Dam contractor left the upstream side of the working platform 2 feet higher than the rest of the work area and built a stone-surfaced haul road on it. This proved to be a much better layout because although the lower part of the working platform did get sloppy, the haul road was free-draining and remained in good shape.

SLIDE 8

Concrete guide walls were installed to keep the top of the excavation stable. We were surprised when the contractor put rebar into the guide walls on his own, but understood later upon seeing the abuse these walls take from the clam buckets and hydraulic jacks. On several panels the guide walls cracked but did not fall into the open excavation because of the rebar. While the guide walls are being installed is probably the best time to set out pins for station markers on a slurry wall project. We requested the Lower Dam contractor do this, but to no avail. As a result, there were foul-ups almost daily in panel stationing.

Each project has a test section in a non-critical area of the dam consisting of 10-11 short panels where the contractor demonstrated his equipment, methods, and procedures before constructing the "production" panels in the balance of the wall. Several changes were made in the specifications as a result of deficiencies noted in these test sections.

SLIDE 9

The Lower Dam specifications required the contractor to test the parameters of the slurry in the storage pond a minimum of three times a day. He was only to test the slurry in the trench after a rainfall or for its sand content before concrete placement. This gave us essentially no control over the slurry at the excavation, while requiring unnecessary testing in the storage pond. We were forced to rely on the contractor to maintain the trench slurry in condition suitable to keep the trench stable. Soon after the construction of the test section, the specifications were modified at no cost to include testing of the slurry in the excavation and to reduce the testing frequency at the storage pond.

SLIDE 10

A filter press was used to measure the fluid loss of the slurry. The Upper Dam specifications initially allowed a fluid loss of 15 cc or less in 10 minutes. In the trench, however, due to mixing with the materials being excavated, it was virtually impossible to maintain this on the slurry without it becoming too thick. The contract was subsequently modified to allow a fluid loss of 20 cc or less in 10 minutes as was previously used on the Lower Dam.

SLIDE 11

The Lower Dam contractor used cranes with free-hanging cable-operated clamshell buckets to excavate the panels. In the beginning, the clam buckets dumped the slurry-contaminated excavated material directly into a dump truck which took it to the disposal area. However, when the owner of the trucks saw what they looked like after 2 weeks of hauling, he refused to rent the trucks to the contractor any longer. To avoid delaying the job, the contractor was allowed to stockpile the excavated material on the platform.

SLIDE 12

This unfortunately contributed significantly to the haul road problem and the deterioration of the working platform in general. The situation worsened when the contractor mobilized a second excavating crane. After a few weeks, the contractor bought a used dump truck and kept it and a large end loader busy cleaning the platform through the end of the project. The Upper Dam specifications were later worded to disallow any stockpiling of excavated material on the working platform.

SLIDE 13

The Upper Dam contractor used a bucket auger on some panels in the test section. This provided relief holes, making excavation easier for the clamshell bucket, and gave him an indication of what equipment to use for the panel excavation.

SLIDE 14

He chose a kelly-mounted clam bucket and did not use the bucket auger on any "production" panels.

To avoid trucking the excavated material to the on-site stockpile, this contractor lined the downstream slope of the dam with plastic sheeting to form a chute.

SLIDE 15

The excavated material was dumped at the top of the slope, but did not slide all the way down the chute. Using heavy equipment to move the material downslope would have torn the plastic. To keep the job going, the contractor built a skip pan for the service crane.

SLIDE 16

The excavating clam dumped into the skip pan and the service crane, sitting on the downstream berm, dumped the skip pan on the stockpile beyond the toe of the dam.

SLIDE 17

The contractor then constructed a steel chute on the slope.

SLIDE 18

Material dumped at the top of this chute slid down to the berm.

SLIDE 19

However, a crane or end loader was still needed on the berm to clean the chute area and move the excavated material to the stockpiles.

SLIDE 20

The chute itself was difficult to move because of its lack of bracing. About halfway through the job it was severely bent while being moved and the contractor went back to using the skip pan.

SLIDE 21

Excavation was unclassified on both projects for payment purposes. The cost of excavation was included in the unit price for a square foot of completed panel wall.

SLIDE 22

Both types of clam buckets easily excavated through the embankment and most alluvial materials. A very dense deposit of fine sand was found in a few panels at the Lower Dam. When the clamshell could not make any headway in this material, a chisel was used to break up the deposit. Much of this went into suspension in the slurry. Excavation of the lower alluvium, which contained sandy gravels and some cobble gravels, was slightly slower than the upper finer-grained alluvium.

SLIDE 23

Wood was frequently encountered in the alluvium. Usually it consisted of just branches and twigs,

SLIDE 24

but some larger limbs and tree trunks were also found.

SLIDE 25

Bedrock at both sites is a granite gneiss. For the purposes of these contracts, saprolite derived from the granite gneiss was considered bedrock. The softer saprolite, with blow counts roughly between 10 and 60, gave the buckets no problem. It was excavated at about the same average rate as the embankment fill and alluvium, around 15 to 20 feet per hour. The buckets reached the limits of their capability in saprolite and intensely weathered rock with blow counts from approximately 50 to in excess of 100.

The specifications defined top of rock as the point at which essentially only bedrock was excavated by the excavating bucket.

SLIDE 26

To determine the top of rock, we would have the crane operator dump a bucket of material on the working platform.

SLIDE 27

The Project Geologist, or, later in the job, some project personnel trained by him, examined the material.

SLIDE 28

When the contents of the bucket were essentially only bedrock, the excavation was sounded using a weighted cable. Excavation continued to the depth required by the specifications and the final excavated depth was determined with the weighted cable.

SLIDE 29

Both contractors used star chisels to excavate the harder rock and to chip out excess concrete. The Lower Dam contractor submitted an excavation plan for advancing the wall using clamshell buckets and chisels.

SLIDE 30

He used a chisel in 8 out of 11 test section panels, where we were evaluating his equipment and methods. When he started the "production panels," however, he refused to use the chisel, stating that it was not "normal overburden excavating equipment."

The specifications stated that the contractor would excavate all materials, regardless of type, to the founding elevation of the wall, the intent being 3 feet into rock or saprolite. However, they also stated that if bedrock was encountered that could not be excavated with "normal overburden excavating equipment" at a higher elevation, the wall would be founded at the higher elevation. Unfortunately, we were not consistently getting the desired 3-foot key into rock in the area being excavated at this time. The contractor was directed to use the chisel and the directive was then rescinded. Before the issue was settled, the contract was modified for $7,700 for chiseling in 3 panels. the phrase "overburden excavating equipment" was not used in the Upper Dam specifications.

SLIDE 31

The specifications required that an airlift be used to clean the panel bottom after the panel excavation reached the founding depth. The airlift used at the Upper Dam was a straight pipe with two 90-degree angles at the top to direct the discharge into a box with a screened bottom.

SLIDE 32

Materials recovered with this airlift ranged up to several inches in diameter.

SLIDE 33

The airlift used at the Lower Dam was a tapered pipe 10 inches in diameter at the bottom and 6 inches at the top. The top was capped and a 3-inch discharge hose exited the side near the top. This hose was connected to the de-sander.

SLIDE 34

The bottom of the airlift has inverted V-shaped notches cut into it. To test the airlift, we dropped some orange-painted crushed stone into a panel excavation. The stone was never seen again because the airlift brought up only sand. We had the contractor try numerous modifications to try to make the airlift work. He replaced his 265 cfm compressor with a 750 cfm compressor to supply more air, disconnected the discharge hose so the airlift spewed sand and slurry all over the platform, thus reducing any back pressure, and even cut the V-notched section off the end to improve suction. No one insisted on shutting down the job until the airlift was working properly and no one could say how to make it work, so it was used as it was, essentially to de-sand the slurry and the panel bottom. Sometime after the job was completed, we found a reference on airlifts that indicated the tapered pipe and the reduction to the small 3-inch discharge hose was likely the problem. What we do know now is that an airlift made of a straight pipe will work. The Upper Dam specifications included a notice to the contractor that his airlift would be performance tested in the test section and that he would not proceed if it failed.

SLIDE 35

The sand content of the slurry was tested after airlifting. If it was 5% or less, concrete could be placed .

SLIDE 36

Joint pipe(s) were set and the panel bottom was sounded again at several points to determine whether any material had sloughed during this operation.

SLIDE 37

The same supplier provided the concrete for both projects. The concrete was delivered to the sites in front-dump concrete trucks and fed through a

SLIDE 38

screened hopper into the 10-inch tremie pipe. The concrete used was a 3,000 psi high-slump mix with a maximum aggregate size of 3/4". As the setting time of the concrete was a critical factor,

SLIDE 39

ice was added to the concrete on both projects to control the temperature.

The drilling subcontractor was the same for the Upper and Lower Dam work.

SLIDE 40

He used a truck-mounted CME 55 drill rig. Under the specifications, payment was made only for those core borings which remained within the panel concrete through the entire depth of the panel. On the Lower Dam, the subcontractor cored a total of 5,200 linear feet of which approximately 2,600 linear feet met the specification for payment. His average remained about the same on the Upper Dam, where about 700 linear feet of the 1,400 linear feet drilled were for payment.

SLIDE 41

The cores generally showed the concrete to be of excellent quality on both projects. Coring the Lower Dam panels turned up a few surprises. The panel depths determined by drilling frequently came up shorter than the sounded depths. The weight used initially on the sounding cable was torpedo-shaped and about 6 inches long. Apparently, it would lie down on the panel bottom before the tension in the cable was relieved, throwing our measurements off by 6 inches. Before construction of the "production panels," this weight was replaced by a short steel cylinder with a square plate welded to its bottom to keep it from turning over.

Below the Lower Dam concrete was a sediment which appeared to be made up of sand and slurry. The thickness of this deposit varied from none to around 2 feet and was usually attributed to poor cleanup of the panel bottom because the airlift never worked properly. However, the presence of some similar sediment under the wall at the Upper Dam, where an effective airlift was used, seems to indicate that this sand dropped out of suspension in the slurry. A surprising amount of sand will settle from a container full of slurry known to contain 5% sand if it sits for about a half-hour. Up to 2 hours were allowed between bottom cleanup and concrete placement under both contracts. The sediment was apparently moved around the panel bottom by the initial concrete placement because core borings taken where the tremie was located during placement usually showed little or no sediment. Core borings taken away from the tremie location usually encountered the greater sediment thicknesses. A lower allowable sand content might be desired on some projects.

SLIDE 42

The lower Dam specifications required the contractor to begin construction of "production" panels on the right abutment. There was a possibility that we might want to extend the panel wall farther into the abutment depending upon its effects on the project instrumentation. A primary panel with a joint at each end was constructed. This allowed the contractor to install the wall as intended while leaving a joint for the possible extension later.

SLIDE 43

Usually the joint pipe was easily removed by the service crane. When there was a delay in the completion of a pour or the concrete temperature was near the upper limit, it would begin to set up around the pipe. In these cases, the contractor used hydraulic jacks to pull the joint pipe.

SLIDE 44

Each project was instrumented with 40-50 open-tube piezometers.

SLIDE 45

The contractor began daily piezometer readings at the beginning of site work and continued until several months after the last panel was installed. The piezometric elevations at all but a few of the piezometers downstream of the wall declined as a result of the wall installation, some by as much as 8-10 feet. A drastic decline was also noted in flows from relief wells and drainage blanket outfalls.

SLIDE 46

Erosion was a problem on both projects as construction of the working platforms left fairly steep bare slopes.

SLIDE 47

After a few rains, there were gullies several feet deep where runoff from the platform was concentrated.

SLIDE 48

A silt fence, gravel drains,

SLIDE 49

berms, and protected drainage courses were used in attempts to control the erosion with varying degrees of success.

The experience gained during construction of the Lower Dam slurry wall contributed to an improved set of specifications for the Upper Dam Wall. Both walls appear to be functioning well to date.

CONSTRUCTION OF SOIL-BENTONITE
SLURRY TRENCH CUTOFFS
TENNESSEE-TOMBIGBEE WATERWAY
GERALD JOINER - MOBILE DISTRICT

This presentation on the slurry trench cutoffs on the Tennessee-Tombigbee Waterway shows a complete sequence of the construction activities needed to carry out the design concepts. You might notice the scenery changing or the equipment looking somewhat different from slide to slide for I've selected slides from anumber of projects that seem to best tell the story.

Without going into any detailed discussion of the Tennessee-Tombigbee project lets take a quick look at where we are for the benefit of those who are not familiar with the Tennessee-Tombigbee.

SLIDES 1-2.

The Tennessee-Tombigbee Waterway is a 232 mile long navigation project running from Pickwick Lake on the Tennessee River system in the NE corner of Mississippi down to Demopolis, Alabama where there is an existing lock and the Tombigbee River is joined by the Black Warrior River. From that point, on a navigation channel already exists to handle traffic on down to the Gulf exiting through Mobile Bay. The Mobile District was in charge of construction of 9 of the 10 locks on the system with Nashville in charge of the northernmost project, Bay Springs L&D. There was some type of slurry trench construction on all of the projects supervised directly by Mobile District. It is this construction that we will now focus on.

SLIDE 3.

To refresh our memory lets look at a typical section of a slurry trench under a cofferdam used to protect the structural excavation area during construction of a lock and dam. Let me highlight some of the features of this typical design---this one happens to be for the Aliceville L&D, the second project awarded by Mobile---for they do have a direct bearing on construction techniques.

First, it is absolutely vital that the moisture content and the compaction of the working surface for the slurry trench construction be right. The working surface is at or near natural ground and the high water tables and surface water conditions mandate that this phase of the operation be carefully controlled.

Next, note that the slurry trench penetrates into a formation called the Eutaw. What actually is to be done is to tie about a foot into an impervious zone of the Eutaw which at Aliceville was right at the top of the Eutaw. We deviate from this rule on the Lock D project where we dug 50 feet and tied into whatever was there. It was necessary for construction personnel to constantly monitor this feature of the operation to assure a good tie-in. We were greatly aided by the accuracy of the boring data provided by engineering in the plans and specs. The specs spelled out specific gradation requirements for the backfill---they were different for the temporary and permanent construction but practically if we met the requirements for temporary construction we also met the requirements for permanent construction. It was necessary that we know ahead of time how to supplement and for waste the material excavated from the trench to meet this requirement.

SLIDE 4.

This typical soil profile, this one happens to be for Lock B, highlights the fact that the top of Eutaw---that is, the elevation of the impervious zone that the slurry trench will tie into---does vary around the perimeter of the cofferdam. I'll postpone any further discussion of how this affects construction until we get into the slides showing the placement of backfill.

SLIDE 5.

I wanted to highlight the problems we have getting a slurry trench started when your construction sites are located in the middle of a swamp, in an area subject to flooding and Notice to Proceed is given in late November when it is virtually impossible in Alabama and Mississippi to do any significant compacted impervious fill work. This happens to be the upper approach to the Lock C site during a January flood of the Tombigbee River right at the time when the contractor was trying to get the site cleared and working surface ready so the slurry trench could be put in and excavation for the structure started. I have a slide a little later that will highlight the problems created by working under these conditions. Without getting into details I will mention that the political climate for this Waterway project was such that it was expedient that contracts be awarded and work be started even when it was recognized that construction conditions would not be "optimum."

SLIDE 6.

Conditions were not always as bad as depicted in the previous slide and here you see a fairly orderly construction of a working surface---usually we have to do mostly fill work with occasionally a little cut to maintain the needed level surface. Width of surface very important, we have found as a practical minimum that the contractor needs about a 50 foot width to work up his backfill. It you have the slurry trench centered in the working surface you're looking at a 100 foot wide section---if its off center a minimum of 70 foot will do.

SLIDE 7.

Sometimes it is necessary to undercut and remove unsatisfactory material and replace it with compacted backfill as you see here. This backfill came from an approved on site borrow area which you see off in the background.

SLIDE 8.

Here is a shot showing the trench excavation getting underway off in the background but what I want to show is the large window of granular material needed to bring the excavated material into the gradation specifications. We are obviously going to waste a lot of material from the excavation. You can also see that the centerline stakes for the trench are off center.

SLIDE 9.

The plant for storing and mixing the bentonite for the slurry is usually very simple. There is a storage tank for the bentonite.

SLIDE 10.

Some means of getting the bentonite from the transporting vehicle to the storage tank such as an auger---remember that we're trying to move a very fine powderlike material.

SLIDE 11.

There is some kind of measured mixing box to assure we can batch 20 lbs. bentonite for every 42 gal. slurry.

SLIDES 12-13.

After some preliminary mixing the slurry is pumped into a holding pond where the mixing action continues. One ingenious contractor is reported to have supplemented his mixing action with a boat and outboard motor.

SLIDE 14.

Single holding pond but-----

SLIDE 15.

Usually, the contractor will have more than one pond. Sometimes on a small operation or because of individual preference a contractor might use a holding tank but the quantity that can be kept stored is rather limited because of tank size. It can take a lot of slurry to keep one of these trenches full. On the Tennessee Tombigbee we had trenches as deep as 70 feet.

SLIDE 16.

This is a shot of another contractor's plant---there are some slight differences but nothing earth shattering.

SLIDE 17.

Also needed are delivery lines which can be rigid or flexible. Since the digging operation is constantly moving, mobility of the lines is an important consideration.

SLIDE 18.

An essential piece of equipment was the backhoe and by far the most popular on the Tennessee-Tombigbee was a Koehring 1066 with special arms and bucket.

SLIDES 19-20.

A Koehring 1266, a much heavier backhoe, was used at Lock E and here you can appreciate the size of the arm needed to allow this machine to dig up to 70 feet. It was necessary to put special counter weights on this backhoe to handle the force exerted when that arm is extended all the way.

SLIDE 21.

Here it is in action. An interesting side note---the 1066 backhoe equipped with counter weights and a special arm for digging the 50 foot deep trench at Lock D tended to get light on the digging end when digging full depth. This wasn't a problem with the 1266 digging at 70 feet. You do have to back way up to start digging and it is rather hard to turn the sharp corners.

SLIDE 22.

This is a shot of a special rock bucket used with the 1266 backhoe at Lock E where the trench went down to rock. I don't have a slide but a special ADCO bucket of a different design was also used at Lock D and performed very well.

SLIDE 23.

Not all the excavation was done with backhoe. Some places a clam bucket such as this special 10 ton model had to be used such as adjacent to the structure or where the depth was too great for the backhoe.

SLIDE 24.

As soon as the digging starts, the slurry is introduced into the trench since the purpose of the slurry is to keep the trench from sloughing. Generally, the slurry level was maintained at the top of the trench or within a foot of the top. At all times it was maintained above the water table.

SLIDES 25-26.

As excavation proceeds, some of the material available may be cast to the side instead of being used as backfill.

SLIDE 27.

Normally, the trenches are dug 3 feet wide but obviously this trench is a little wider than that. The contractor did not properly consolidate this area and consequently once digging started, even with the slurry in the trench, the sides of the trench at the top began to crack. By working off mats the slurry trench subcontractor was able to remove this material near the top and proceed on. Eventually, he was able to eliminate the mats.

SLIDE 28.

Here you see the backhoe still on mats. The trench bottom has to be cleaned of loose material to assure a proper tie-in of the impervious backfill.

SLIDE 29.

We have had the most success with an airlift pipe which is simply an upside down L shaped piece of pipe with an air line running down the side to force the loose material back up the pipe. Its handled by a crane.

SLIDES 30-32.

Here you can see the airlift operation following closely behind the backhoe. An alternate method of cleaning the trench bottom is with a clam.

SLIDES 33-35.

This method was not used as much and probably doesn't give quite as good a result as the airlift. It is necessary that we sample the trench bottom to verify the type material there and the cleanliness.

SLIDE 36.

The specified method was with a splitspoon sampler. Attached to the airlift pipe it could be driven into the trench bottom and a sample successfully recovered.

SLIDES 37-39.

Another type of sampler utilized a solid tube. Attached to a backhoe it could be driven into the trench bottom and the sample then forced out of the tube with an extractor.

SLIDES 40-41.

We have been able to very successfully identify the type of material being excavated by samples from the bucket.

SLIDE 42.

Checking the depth of the trench bottom.

SLIDE 43.

Sampling slurry---required every 10 feet of depth---it is probably enough to require 3 samples for depths over 30 feet---depths vary from 25+ up to 70 feet. Our experience indicates 3 samples adequately describe condition of slurry. The practice depicted is not the safest in the world.

SLIDE 44.

Some of the equipment used to check the properties of the slurry. We're interested in the viscosity, density and water loss.

SLIDE 45.

Mixing of the backfill usually accomplished with dozers working back and forth - away from the trench edge. The dozer has been supplemented to a limited degree with a clam. The slurry is the only fluid used to increase the consistency of the backfill which is usually controlled between 2 and 5 inches of slump as measured by the standard slump cone test.

SLIDES 46-47.

The mixed backfill is pushed into the open trench so it slides down the face of a fairly flat slop such as a 6 or 7 to 1. To do this we must start the backfill with a clam since there can be no free fall of the slurry into the trench.

SLIDE 48.

It is here in the backfill operation that work progress can be affected if trench bottom depth varies particularly if depth increases backfill in extremely deep holes may flow to backhoe and be excavated.

SLIDE 49.

Here you can see the entire sequence of operations, digging, cleaning backfilling.

SLIDE 50.

Maneuvering on 5-inch slump fill can be very treacherous as this dozer operator found. Don't try walking on it unless you have a life line attached.

SLIDE 51.

The backfilled trench tends to crust over and dry very quickly and to protect it one of the final operations is to place a cap over the trench to allow the backfill to cure out for 2 weeks without drying and then proceed on up with the cofferdam embankment.

SLIDE 52.

Our specifications only require a 2 foot thick cap but practically we had to go to 3 feet to successfully bridge over the trench backfill. You will also be interested to know that as far as we can determine the slurry backfill does not settle away from the overlying fill. Very quickly we'll run through some slides showing some permanent uses of slurry trenches and some special applications of slurry.

SLIDE 53.

Dam across river.

SLIDE 54.

Impervious core abuting lock structure with slurry cutoff wall beneath.

SLIDES 55-56.

Concrete cutoff wall around perimeter of lock wall with slurry tie in.

SLIDE 57.

Cement-bentonite slurry injection process using hollow stem H-beam.

SLIDES 58-60.

Telephone cable relocation.

SLIDE 61.

Withstood floods.

Cost of slurry trench construction without trying to tie cost to other methods of dewatering: Bid Prices: 3.10, 4.00, 8.00, 1.70, 4.00, 3.00, 4.00, 5.00
Range of bids 1.64 - 8.00.

SOIL-BENTONITE SLURRY
TRENCH CUTOFF
ST. STEPHEN POWERHOUSE
DAVID LANE - SAVANNAH DISTRICT*

SLIDE 1.

This title slide was used to introduce the presentation and state the purpose of the talk as: the particulars of the Cooper River Slurry Trench design and construction experience. It was pointed out that much of the design and construction was routine and had already been presented by others at this conference.

SLIDE 2.

Location map shows the general location of the project and the specific power plant area. During this slide the history of the project was given, purpose of funding, and the multi-district nature of the design and construction of the St. Stephen Power Plant, including the slurry trench.

SLIDE 3.

The power plant site shows the slurry trench outlined around the power plant. This slide was presented to discuss both the temporary and permanent nature of the slurry trench cutoff as part of dewatering and seepage controls and to show its relationship to the power plant. Stratigraphy was also discussed. The slurry trench was constructed from a working berm elevation of 56 through overburden materials to tie into a shale aquiclude. This shale was excavated inside the slurry trench to found the powerhouse on the underlying limestone.

SLIDE 4.

This is a design introduction which indicates the Philadelphia District designed the slurry trench and presented it in the Powerhouse Foundation Analysis Report in 1976. It points out that the design was based on the Duncan Dam and Saylorville Dam slurry trenches.

SLIDES 5-6.

This presents the slurry trench design requirements based on the Powerhouse Foundation Analysis Report. These are the requirements included in an appendix to the report by Philadelphia District. The requirements are as follows:

* Currently with CH2M Hill, Milwaukee, Wisconsin

1. The trench width must be between 5 feet and 10 feet.

2. Natural Wyoming Sodium Bentonite to be per API Spec 13A.

3. Slurry density prior to placing between 65 pcf and 75 pcf. Slurry density in trench to be between 65 pcf and 90 pcf.

4. Slurry properties going into trench: viscosity must be greater than 15 centipoises at 30 C. Water loss must be less than 20 cubic centimeters in 30 minutes. This insures formation of an adequate filter cake on the trench walls and enables the slurry to hold particles in suspension.

5. Granular Backfill. 12 to 25 percent of the material must pass the number 200, 35 to 70 percent must pass the number 4 sieves, and 100 percent must be less than 6 inches diameter.

6. Slurry Level. Must be a minimum 2 feet above groundwater level and maximum 3 feet below top of the trench at elevation 50.

7. Start backfill with wedge to top of trench and then push down slope of backfill wedge.

SLIDES 7-8.

This shows the plan and sections of the slurry trench, excavation and dewatering wells for the power plant site, respectively. These slides were used to explain the temporary and permanent cutoff requirements of the slurry trench, the overall dewatering scheme used for the overburden and confined aquifers, and to show the relationship of the trench with the excavation and dewatering wells.

SLIDE 9.

This shows the typical overburden section and the three predominant layers of overburden materials. The consistency of these materials were pointed out. The depth of the backhoe excavation was also pointed out and the remaining clamshell operation was shown to be below this depth. Also introduced at this time was the difficult excavation encountered in the lower dense sands and the need to predrill to accomplish that excavation.

SLIDE 10.

This shows the trench profile and operation of the equipment as planned by Moretrench of America (MTA), the subcontractor constructing the slurry trench. It shows the extended arm backhoe followed by two clamshells and an air lift. It was emphasized that this was the intended procedure by MTA.

SLIDE 11.

This shows the trench profile with the equipment that was actually necessary to accomplish the excavation. The extended-arm backhoe was followed by a spiral or bucket auger with a diameter equal to the width of the trench (36 inches). Predrilling was done on 4 to 8 foot centers in order to accomplish the excavation below the reach of the backhoe. The auger was then followed by up to three clamshells at one time in order to accomplish the excavation as quickly as possible, (clamshell operation being the slowest and most difficult). This was then followed by the air lift. It was pointed out that the predrilling was used on close centers because its excavation rate was quicker even than the clamshells.

SLIDES 12-13.

This presents the slurry trench design as specified. It was pointed out that the requirements given in the Powerhouse Foundation Analysis Report by Philadelphia District were modified based upon subsequent experience and expertise in the Corps as obtained by Savannah District during the writing of the specifications. The following points were made:

1. Design trench width was 3 feet minimum.

2. The top of the trench (working surface) elevation was changed from 50 to 56 in order to accommodate a higher than originally expected groundwater table. The slurry was required to be a minimum of 1 foot above groundwater and at the top of the trench.

3. References included the API Code RP13B Spec 13A.

4. The initial slurry mixture placed in the trench had to be a minimum of 20 pounds Wyoming Bentonite per 42 gallon barrel of slurry. This was standard for Corps slurry trenches. The apparent viscosity had to be greater than or equal to 15 centipoises at 30 C and had to be measured by a direct indicating viscometer and correlated to the marsh funnel. Water loss had to be less than or equal to 20 cubic centimeters in 30 minutes measured by the filter press.

5. The trench mixture during backfill had to be greater than 65 pcf and less than 85 pcf. Water loss requirements had to be the same as the initial slurry mixture.

6. Processed granular backfill was required underneath the levees and switchyard because of the required support and undesirability of consolidation beneath these structures. The actual spec requirement was to be 20 feet beyond the toes of the levees and the switchyard. The gradation of this mixture after mixing with slurry had to be 100 percent passing the 3 inch, 35 to 70 percent passing the number 4, 15 to 40 percent passing the number 40 and 10 to 25 percent passing the number 200 sieves.

7. All other areas had to be backfilled with natural backfill which was excavated material blended with slurry. The only requirement here was that 10 to 25 percent had to be passing the number 200 sieve.

SLIDE 14.

This presents the slurry trench excavation requirements as specified. The following points were emphasized:

1. Excavation equipment could be any type of earthmoving and drilling equipment plus chopping bits and ripping blocks. Drilling is emphasized here because the Contractor filed a claim indicating that he did not anticipate drilling would be required, yet he ended up having to use predrilling extensively.

2. The slurry had to be cleaned with a vibratory shaker screen and centrifugal sand separator. The trench bottom had to be cleaned with jet pipes, airlift pumps, etc., because it was too deep to be cleaned with a smooth bucket backhoe.

3. The bottom of the trench was to be probed for potholes and cracks and then sampled before backfilling with a standard splitspoon sampler attached to the clamshell bucket or other equipment. This sampling had to be every 25 feet of trench length.

SLIDE 15.

This presents the slurry trench backfill requirements. These are standard and include thoroughly mixing all backfill into a homogeneous mass with a slump of 2 to 5 inches. Standard backfill procedure of placing a wedge from the bottom of the trench to the top at the start with a clamshell and sliding subsequent backfill down the forward face of the backfill wedge was specified. The excavation toe was to be between 30 feet and 200 feet from the backfill toe. It was pointed out that the maximum distance of 200 feet was extended to 400 feet when the Contractor found that he had to add additional equipment in order to efficiently accomplish the excavation of the trench. The backfill cap was to be 3 feet high and 8 foot wide placed over the completed slurry trench immediately after backfilling to prevent drying out. This cap was to be removed when final backfill was accomplished. At this point it was noted that an earlier presentation concerning the permeability of completed slurry trenches, pointed out that the filter cake formed on the sides of the wall should generally be ignored as a contribution to increasing impedance to flow. I pointed out that this experience could be correlated with experience gained during instrumented caisson load test where the slurry method of excavation was used to install the caissons. The load test results indicated that the caissons developed very high fricitional resistances. This indicates that the process of pouring concrete in a caisson excavated under slurry probably removed the slurry caking from the walls, thus resulting in the friction. It could then be assumed that a similar procedure of backfilling a slurry trench would result in a similar disturbance to filter cake, and probably accounts for the reason that the filter cake can not be relied upon to contribute to the impedance to flow.

SLIDE 16.

This presents the slurry trench testing frequency. Prior to placing slurry in the trench methylene-blue absorption, viscosity, filtration, and density tests must all be run in the mixing ponds twice daily. Slurry in the trench must be monitored at all times and sampled every 15 feet of depth in the trench twice daily and tested for filtration, density, and water analysis. Gradation of backfill must be tested for every 300 cubic yards of material in locations designated by the Contracting Officer and the slump cone test must be made 2 to 5 times daily, depending on the amount of backfill. Those locations are also designated by the Contracting Officer. It was pointed out that all of the slurry and backfill tests run were generally within the requirements presented in the specs and no construction prolems were noted with respect to slurry and backfill properties.

SLIDE 17.

This presents the important slurry trench construction records as experienced on this project. The profile of the trench bottom including the description of the material is very important. In addition, all slurry and backfill test results should be submitted along with the daily construction log. A precise survey of the as-built trench alignment is also important, as there always seems to be a need to locate the trench at some point in the future. The general experience has been that trenches are difficult to locate by visual inspection after having been completed for some time.

SLIDE 18.

This presents the slurry trench bid costs. The top four costs include the Government estimate of $9.07 per square foot and the three top bidders for the overall power plant project. The low (overall) bid by Guy F. Atkinson was $20.00 per square foot for the slurry trench. As has been pointed out previously in this conference, slurry trenches are initial items on most projects, and tend to show high unit prices on the prime bids because of "front-end loading" by the Contractor. Thus, the prime bid price is usually not indicative of the actual cost of installation of the slurry trench. Because on this project there was a significant claim, research by the Government on Contractor's records indicated he received five sub-bidder's prices for the slurry trench. Moretrench's price of $5.10 is significantly lower than the prime bid price. Other prices include Bencor at $5.60, ICOS at $6.75, Case at $8.82, and Soilinc at $11.30. It was pointed out that these square foot prices by the sub-bidders probably reflect to some extent the desirability of the sub-bidder of getting the job, but also reflect the degree of difficulty anticipated by the sub-bidders. The actual prices by Case and Soilinc indicate that they had a better idea of the difficult excavation required than the other lower bidders.

Additional slides show typical slurry trench excavation equipment used at this job, including the extended arm backhoe (Koehring 1066D) that would reach to a depth of approximately 55 feet. Also shown were a typical clamshell, both the spiral and the bucket auger, the airlift, the slurry mixing tank and mixing ponds, and several typical views showing th slurry trench with a lower level of slurry resulting in caving near the top of the trench. It was pointed out that during the construction of this trench there was a nationwide truckers strike which made delivery of the bentonite difficult for a certain period of time. This resulted in the Contractor trying to proceed with the slurry trench with a shortage of bentonite, lowering the levels of slurry 5 to 6 feet at times.

Though there were no slides to present, the suject of the testing and analysis for the slurry trench stability when excavated was briefly discussed. It was explained that the Contractor had fears that the eventual tailrace canal excavation through the slurry trench could result in flow of the slurry backfill out of the trench into the canal excavation. Undisturbed samples were taken and tested. The material was found to be predominantly an SC (clayey sand) material with cohesive strength on the order of 500 psf. Three dimensional analysis were made based upon the literature on trench stabilities and showed there would be no stability problems. Experience today after a minimum of 1 year since excavation indicate that there have been no problems with this trench stability.

Two additional slides were presented from a report by D'Appolonia when he was contracted to help defend the slurry trench claim. The first slide summarized equipment excavation rates and was used to point out the actual excavation rate of equipment compared with Moretrench's expected excavation rate. The slide shows that the expected and actual rate of the backhoe were very close by that Moretrench overestimated the excavatability of the clamshells by 2 to 1 compared to actual experience. The second slide shows Moretrench's experience record in constructing slurry trenches prior to the St. Stephen slurry trench. This shows the types of trenches, depths, types of material, and types of equipment used to construct these trenches. It was pointed out that Moretrench did not have experience in trenches of 80 foot depth such as the one at St. Stephen, nor did they have experience in the ground conditions represented at St. Stephen (very dense sands). It was emphasized that these are two important issues to consider when slurry trench specialist qualifications are considered prior to excavating the trench. Should the responsible engineer find that this particular type of experience may be lacking, it should be called to the attention of the slurry trench subcontractor and careful consideration given to his intended methods and equipment in order to avoid major problems and impact to the schedule which could result in a claim.

CONCRETE CUTOFF WALL
ST. STEPHEN POWERHOUSE
TED F. HIGHTOWER - SAVANNAH DISTRICT

SLIDES 1-2.

The St. Stephen Powerhouse is a part of the Cooper River Rediversion Project in the Charleston District. It is located in a canal which connects Lake Moultrie and the Santee River. The project purpose is to reduce dredging in Charleston Harbor by rediverting a large portion of silt laden flow of the Santee River, which presently flows down the Cooper River, back into the Santee River. The powerhouse was constructed to take advantage of the hydro-power potential available in the diversion canal and to replace the energy lost at the Pinopolis Powerhouse because of the rediversion.

SLIDE 3.

The powerhouse is tied into the levee system on either side of the canal by a north and a south embankment. The invert of the intake canal is elevation +50 m.s.l. The invert of the tailrace canal at the powerhouse is elevation 0.0 m.s.l. The earth embankment has a top elevation of +86 m.s.l. Normal pool is approximately +75 m.s.l. A fishlift is part of the north side of the powerhouse.

The initial contract requirements for the St. Stephen Powerhouse included a steel sheet piling cutoff to be installed beneath the embankment. The pile cutoff was to extend from the base of the embankment impervious core, through in-situ clays, silts, and sands, and tie into underlying shale. The contractor anticipated difficulty in driving the sheet piling based on difficulties experienced with the foundation materials while constructing the dewatering slurry trench surrounding the powerhouse excavation. To evaluate these anticipated problems in driving the specified sheet piling, the contractor retained a consulting engineer to perform pile driving tests.

These tests concluded that no combination of sheet piles and pile drivers could succeed in driving the majority of the sheets to the specified elevations.

SLIDES 4-5.

On 21 April 1981 the contractor submitted a Value Engineering Change Proposal (VECP) to the Resident Office proposing to delete the lower portions of the impervious core and the sheet piling cutoff. Seepage control for the embankment was proposed to be accomplished by the installation of an upstream impervious blanket. After review and evaluation of the VECP, the contract was modified to incorporate changes resulting from the VECP and the contractor was issued a notice to proceed on 1 February 1982.

SLIDE 6.

On 4 November 1983 during filling of the upstream pool, an emergency seepage condition developed in the north embankment immediately adjacent to the powerhouse and fishlift when the pool reached elevation +73.5 m.s.l.

SLIDE 7.

The seepage was first noticed as a strong flow exiting from the north tailrace retaining wall weep holes.

SLIDE 8.

Subsequently, water was observed coming through the bottom of a large planter box placed about midway in the downstream slope of the embankment adjacent to the powerhouse and fishlift wall. Within four to five minutes, the planter box began to overflow. Initial flow was calculated to be approximately 1700 gpm. After half an hour, the flow from the planter box ceased and the water level inside began to drop.

SLIDE 9.

Following this, a sinkhole was discovered on the upstream slope a few feet from the crest of the embankment and soon after, muddy water was observed discharging from the north tailrace retaining wall weep holes. The water from the weep holes cleared up about half an hour later. A pump had been placed in the planter box to control any subsequent flow there. Flow into the planter box continued for some time at a rate of approximately 100 gpm.

The contractor and the Resident Engineer and his staff reacted quickly to bring the problem under control. The first action was to dump bentonite into the lake around the upstream retaining wall. Next, the contractor began placing a surcharge fill on the upstream slope of the dam in an effort to choke the seepage intake. After a few hours, the seepage rate declined significantly. Initially, this was attributed to placement of the surcharge fill; however, when the embankment was later excavated, it was found that the internal zonation of the dam functioned to the extent that it clogged the seepage paths.

SLIDE 10.

A separate action was initiated to close the canal plugs at Lake Moultrie and at the Highway 52 Bridge. After the two plugs were reconstructed across the canal, the upstream pool was drawn down through the fishlift to approximately elevation +66 m.s.l. Further drawdown would have to be through one of the incompleted units in the powerhouse. Preparations were made to do this if necessary; however, the seepage condition stabilized and the decision was made to delay further drawdown until the Highway 52 plug was strengthened and one of the the powerhouse units was completed to a point where no damage would be sustained by passing water through it. Final drawdown was initiated on 22 November and completed on 8 December 1983.

The final drawdown was carefully staged in increments of 1.5 to 3.0 feet with a minimum of two days between each increment to prevent sudden drawdown conditions from developing in the Highway 52 plug which was then serving as a dam.

SLIDE 11.

A formal investigative panel was promptly established for the purpose of performing an independent investigation to discover causes related to the development of seepage. Almost simultaneously, action for rehabilitation was begun. Savannah District's Geotechnical Branch was given primary responsibility for the rehabilitation design.

SLIDE 12.

All considered designs were required to fall within the bounds of certain criteria as follows (no order of priority intended):

- Be compatible with construction completed to date.
- Be positive in character.
- Be cost effective.
- Be approved within the least amount of time by higher authority.
- Be considerate of the project completion date (August 1984).
- Be implemented and completed within a minimum amount of time.
- Be integrated into the existing construction schedule.
- Be administered and inspected by the current staff.

Two courses of action were considered:

- Rebuild the embankment in accordance with the existing plans and specifications.

- Construct two concrete cutoff walls to basically reestablish the original design concept.

The first consideration would be implemented only in case the ongoing investigation revealed the seepage condition to be caused by other than design deficiency. The second consideration would be designed simultaneously with the failure investigation and implemented immediately should the ongoing investigation determine the need.

Savannah District had recent experience in designing and constructing two panel type concrete cutoff walls for the rehabilitation of two earth dams at Hartwell Lake GA & SC.

It was concluded that a panel type concrete cutoff wall constructed through the two embankments at Cooper River would meet all the above stated criteria and, therefore, should be investigated. A contractor's availability and interest inquiry was made, with favorable results. In a series of meetings it was resolved that Savannah District would proceed with the design of a panel type concrete cutoff wall and would have that design completed and ready for implementation within six weeks pending results of the ongoing seepage failure investigation. To expedite its implementation, it was further resolved that, if used, the cutoff wall design could be implemented by modification of the existing contract.

The seepage investigation concluded that the conditions experienced at the Cooper River Project were precipitated by design deficiency; therefore, immediate attention was given to implementing the cutoff wall design.

Design considerations and parameters of the Cooper River Project cutoff walls are as follows:

SLIDES 14-15.

-- Cutoff walls to be constructed along the centerlines of the north and south embankments, extending from the powerhouse, about 375 feet north and 325 feet south, respectively. These lengths terminate each wall about 20 feet beyond the powerhouse slurry trench.

SLIDES 16-17.

-- Each cutoff wall to extend from elevation +80.0 m.s.l. through all materials and key about 3 feet into an impervious shale layer at about elevation -25.0 to -32.0 m.s.l.

-- Wall width - Nominally 2 feet.
-- Maximum panel length along alignment - Nominally 25 feet.
-- Approximate maximum panel depth - 115 feet.
-- Concrete placement - Tremie
-- Concrete strength (28 days) - 3000 psi minimum.

-- Soil-Bentonite panels placed upstream of the cutoff wall at each concrete panel joint and at the cutoff wall-powerhouse contact. The soil-bentonite panel at the cutoff wall-powerhouse contact to be about 30 feet in length. The soil-bentonite panels at other locations to be about 10 feet in length. Soil-bentonite panels were added to the Cooper River Project cutoff wall design to mitigate seismic risk.

-- Acceptance - based on coring results.

Slurry Limits:

- -- Bentonite concentration - 6%.
- -- Minimum Density - 64.3 PCF at 6%.
- -- Maximum Density while excavating - 80.0 PCF.
- -- Density before concrete placement - 78.0 PCF.
- -- Apparent Viscosity - minimum of 15 centipoise.
- -- Plastic Viscosity - not greater than 20 centipoise at the time of concrete placement.
- -- Gel strength - minimum of 15 lbs. per 100 square feet.
- -- Marsh Funnel - Minimum of 34 seconds.
- -- Filtrate - maximum of 20.0 cubic centimeters.
- -- Alkalinity - pH of 8-11.
- -- Filter Cake - less than 2 mm.
- -- Sand Content - less than 5% before discharge into trench or concrete placement.

Concrete Proportions:

- -- Strength (28 days) - minimum of 3000 psi.
- -- Water-cement ratio - maximum of 0.50 by weight of equivalent Portland cement.
- -- Cement content.

  (a) Without the use of Type F or G Chemical Admixture - Volume of cement and pozzolan shall be approximately equivalent to a total absolute volume of 658 pounds of cement per cubic yard.

  (b) With the use of Type F or G Chemical Admixture - Volume of cement and pozzolan shall be approximately equivalent to a total absolute volume of 564 lbs. of cement per cubic yard.

  In (a) and (b) above, of the total cementitious materials, about 20 to 25% may be pozzolan.

- -- Fine aggregate - approximately 45% plus or minus 5% of the total aggregate volume meeting ASTM C 33.

- -- Coarse aggregate - maximum nominal size - 3/4-inch meeting ASTM C 33, size 6.

- -- Air content - 5% plus or minus 1.5%.

- -- Slump - 7.5 inches plus or minus 1.5 inches.

RECOSOL, INC. was the successful negotiator for construction of the cutoff

HYDROFRAISE
INSTALLATION OF CONCRETE CUTOFF WALL
ST. STEPHEN POWERHOUSE
CHARLIE HESS - CHARLESTON DISTRICT

As Mr. Hightower mentioned, it was concluded from the extensive investigations at St. Stephen to install a positive concrete cutoff wall to preclude any future seepage problems.

SLIDE 1.

The wall was constructed through the embankments on each side of the powerhouse extending from the powerhouse to the construction dewatering slurry trench. The length on the north side was approximately 375 feet and on the south side 325 feet.

SLIDE 2.

The top of the wall was at elevation 80 which required degrading of the embankments. This was necessary in order to provide sufficient room for a working platform. The wall was 2 feet wide and was excavated into the underlying clay shale at a depth between 110 and 120 feet. In addition to the concrete cutoff wall, soil-bentonite panels were installed upstream of the joints between the concrete panels to provide an additional measure of safety in case of cracking at the joints during an earthquake.

The prime contractor, Atkinson, solicited bids for the wall from a number of major wall contractors including ICOS, Bencor, Soletanche, and Moretrench.

SLIDES 3-4.

Soletanche who is now known as Recosol was the low bidder at $16.85 per SF and submitted a proposal, which was approved, to use the hydrofraise, a patented drilling machine developed by Soletanche in France. The use of the hydrofraise at St. Stephen marked its first use in the United States.

SLIDE 5.

The hydrofraise consists of a heavy metal frame or body 8 feet wide, 60 feet high and 2 feet wide which servies as a guide and is fitted at the base with two rotary cutters. The cutters are equipped with tungsten carbide teeth and rotate in opposite directions and break up the strata. Each cutterhead can be independently operated to adjust the torgue and verticality.

A large hydraulic pump placed just above the cutters pumps removes the suspended cuttings and slurry from the bottom of the excavation through pipe-lines to the surface and the desander. The slurry with cuttings is continuously filtered and then returned back into the trench.

A heavy crawler crane supports and manipulates the machine. It also carries a 300KW power pack supplying power to three down-the-hole motors, two of them drive the cutters and the third driving the pump.

SLIDES 6-8.

Verticality of the excavation is controlled by aircraft-style bubble inclinometers mounted on the axis of the hydrofraise and monitored by the operator. Corrections are made by varying the speed of the cutters and/or adjusting the boom angle.

SLIDE 9.

The machine is capable of excavating trenches from 2 feet to 5 feet in width with the use of spacers mounted on the frame with associated wider cutters.

SLIDE 10.

The design of the huydrofraise makes it possible to excavate a wide range of material from unconsolidated soils such as silts, sands, gravels, and cobbles up to 10 cm to hard rock of 10,000 psi compressive strength.

Published excavation rates range from more than 26 cu. yds. of wall per hour in the case of unconsolidated soils to 1.3 cu. yds. per hour in the case of very hard limestone. The machine eliminates the need for chiseling.

At St. Stephen the hydrofraise averaged approximately 8 cu. yds. per hour through the embankment and foundation materials. A significant portion of the foundation materials consisted of dense sands (+100 blows/ft). In hard sandstone layers the hydrofraise averaged less than a cu. yd. per hour. In concrete (3,000 psi) the average 2 cu. yds. per hour.

Production rate comparison of the hydrofraise with conventional clamshell/drilling excavation in the same vicinity clearly revealed higher production especially in the more difficult areas. At times, the hydrofraise production more than doubled that of the conventional clam bucket.

SLIDE 11.

The primary panels at St. Stephen were excavated with three complete bites and two secondary bites of approximately 3 feet wide. The distance between primary panels was 2.2 meters which allowed removal with one bite and an overlap of 0.1 meter in each primary panel.

SLIDES 12-13.

This is a slide of the joint between panels. The teeth biting into the adjacent panels essentially formed a tongue and curve situation. Coring at a joint indicated extremely good bond at the joint for the full depth of the panel.

SLIDES 14-15.

A specific advantage of the hydrofraise is the elimination of the mess associated with the normal clam shale operation. The hydrofraise is also essentially free of vibration and shock which makes it very suitable for use on urban sites. Overbreak is also less than conventional systems and verticality is excellent and can be controlled and corrected to less than 0.2% if required.

A special feature of the cutting tool is its ability to cut into the concrete of an adjacent panel, to a thickness of several centimeters, thereby eliminating the need for shoulder pipes when constructing alternate panels.

SLIDES 16-17.

Another important advantage of the hydrofraise is that the drilling mud is constantly screened and desanded during the excavation which allows concreting to be carried out as soon as the required excavation depth is reached which further speeds up the total operation. This does, however, require a much more sophisticated desanding operation.

CONCRETE CUTOFF WALL DESIGN
WALTER F. GEORGE DAM
JOHN McFAYDEN - MOBILE DISTRICT

SLIDE 1.

General geology of the Southeastern U.S.-Walter F. George project is located in the Gulf Coastal Plain physiographic province on the Chattahoochee River on the Georgia/Alabama border. The project is on the outcrop belt of the Eocene age Wilcox formation and the Paleocene age Clayton formation.

SLIDE 2.

The main project features are an 88 foot single lift lock, a 17 gate spillway, a four unit - 32,500 kw powerhouse, and two earth embankments each over one mile in length.

SLIDE 3.

Underseepage problems have plague both embankments since impoundment. Essentially the entire length of Alabama embankment has experienced underseepage. The portion of the Georgia embankment founded in the floodplain has also experienced detrimental underseepage. Landward of station 104+00 there has been no underseepage problems.

SLIDE 4.

Both embankments have an upstream impervious section, a downstream random fill section, and a pervious section at the toe. These were constructed on the natural overburden material-alluvial in the floodplain and residual in the Georgia abutment. Design seepage analysis was based on 5 feet of impervious material upstream of the embankments. Over 60% of the area, the natural clay blanket was supplemented with fill to obtain the required 5 foot thickness 500 feet upstream. In most areas the clay is underlain by 20 to 40 feet of sands that extend to top of rock. In some areas the Wilcox sands and clays are present at the top of rock. In the abutments residual clays from the Wilcox overlay a section of the Wilcox above top of rock. Rock at the site are the Clayton limestones, which has been divided into three members: the earthy limestone; the shell limestone; and the sandy limestone. The earthy limestone is the uppermost member of the Clayton limestones and is approximately 50 feet thick. The earthy limestone is soft to moderately hard, dense, and chalky. Solutioning is concentrated along the two major vertical joint sets and a 2 foot thick horizontal shell layer which bisects the earthy member. Most of the solutioning observed at the project has been in the shell layer and the upper half of the earthy member. The earthy member is underlain by the 40 foot thick, moderately hard to hard, highly porous, shell limestone. This member is not highly jointed with only one definable joint set observed. The lower part of the Clayton is about 35 feet thick and includes a series of thin beds grouped together and called the sandy limestone. It contains calcareous sandstones, sandy limestones, shell limestones, earthy limestones, and unconsolidated sands.

SLIDE 5.

Construction began in 1955 and was completed in 1963. The earth embankments were essentially complete before excavation of the concrete structures was initiated. During the later excavation the joint systems in the earthy and shell limestones were first noted. During this time, several existing sinkholes (shown in red) were documented. In October 1961, during stripping operations two sinks developed near the dam axis. A detailed exploratory drilling program test excavation revealed weathered cavernous zones in the earthy limestone. Because of the extensive solution features found in the excavation, it was decided to grout the earth dike sections of the dam to the top of the shell limestone on 5-foot centers. The curtain grouting program was completed in 1963 with a total grout take of almost one million cubic feet. The total cost of the grouting program was $2.3 million dollars.

SLIDE 6.

The test excavations revealed several solution widened joints that extended to the shell layer in the earthy limestone.

SLIDE 7.

The horizontal shell layer showed different degrees of solution activity.

SLIDE 8.

In December of 1962, when reservoir impoundment reached elevation 166.0, numerous small boils developed in the drainage ditch at the downstream toe of dikes and it was decided that relief wells would have to be installed along lines 40 feet downstream from the dikes.

SLIDE 9.

The relief wells are 6 and 8 inches in diameter and extend through the overburden to the top of rock. A total of 350 wells were installed, generally on 40-foot centers, along the flood plain portion of each dike toe.

SLIDES 10-12.

The flows from the relief wells were channeled in the drainage ditches to allow monitorning of the flows.

SLIDE 13.

Construction of the project was completed in late 1963. From 1963 until 1968, no foundation problems were discovered. In April of 1968, a large spring was discovered flowing from the riprap near the downstream lock guide wall. Removal of the riprap revealed that the water was flowing from an opening in the earthy limestone at a rate of about 1,000 gpm.

SLIDE 14.

At about the same time the spring was discovered, sinkhole activity in the area upstream of the spring began.

SLIDE 15.

In order to intersect the upstream source and cut off the water to the spring, remedial grouting was begun along the Georgia dike and around the perimeter of the larger sinks.

SLIDE 16.

The grouting program was underway when, in 1969, a fathometer survey revealed a large sink in the reservoir. The combination of the spring, the increased sinkhole activity, and the reservoir sink caused considerable concern for the stability of the Georgia dike and the scope of the remedial work underway was increased.

SLIDE 17.

In addition to the grouting along the dikes, the reservoir sink was plugged and a sand filter trench was constructed upstream of the spring.

SLIDES 18-19.

The purpose of the filter trench is to intersect the open solution channels, along which seepage is occurring, and prevent the piping of foundation material. A second reservoir sink was discovered in January 1972, upstream of the Alabama dike. This sink was backfilled with sand and gravel.

SLIDE 20.

The total cost for the remedial work accomplished between April 1968 and April 1972 was $3.2 million. This work was conducted in 8 phases.

SLIDE 21.

A detailed description of this work and the related investigations are contained in the report titled "Foundation Remedial Work 1961-1972." This report was submitted to the Board of Consultants for their review. Based on the information in this report and conferences held with Corps personnel, the Board of Consultants issued a report in September 1973, which concluded that, "the safety of the dam could be met without redesign or major reconstruction, provided a vigorous program of surveillance is established to detect the formation of sinkholes and prompt action is taken to fill and repair sinkholes as they may appear."

SLIDE 22.

In June 1974, a Mobile District Manual entitled "W.F. George - Plans for Surveillance and Emergency Remedial Action" was submitted by the District and approved by higher authority. Since that time the District has followed the surveillance procedures set forth in this manual, which included quarterly inspections of the project by design personnel and semi-annual side scan sonar surveys of the upstream area. It also included potential failure modes and their indicators; action to be initiated, and what equipment was available for emergency use.

SLIDE 23.

In January 1977, the District submitted a supplemental to the 1972 "Foundation Remedial Work" report. The supplementary report updated the subsurface data and reviewed the significant events which occurred since 1972.

SLIDES 24-28.

This included 2 reservoir sinks, 13 sinks downstream of the embankments, pin boils in the drainage ditches, and the installation of 3 new relief walls.

SLIDE 29.

In July 1977, the Board of Consultants met for a second time with Corps personnel to review all the subsurface data and advise the Corps on the necessity of seeking a permanent solution to the foundation and underseepage problems. After a briefing on the subsurface conditions, underseepage problems, and possible methods of constructing a permanent cutoff, the Board found no justification for the permanent cutoff and recommended the present surveillance program be continued. The Corps did not concur and in 1977, OCE directed South Atlantic Division to proceed with investigation and planning of a permanent solution to the underseepage problem at Walter F. George. The Mobile District then proceeded to prepare a Design Memorandum entitled "Study of Permanent Solutions to Underseepage Problems." This memorandum was completed and presented to higher authority in April 1978. This memorandum concluded that the project was still stable, but that the surveillance and repair program did not provide the degree of safety which a positive cutoff would provide.

The memorandum presented several different positive cutoff schemes that had been studied and evaluated. Of these schemes, a Concrete Diaphragm Cutoff Wall along the crest, similar to the one installed at Bonneville Second Powerhouse, was considered the most economically feasible and was therefore recommended as the number one alternative for providing a positive cutoff to the underseepage problem. Other schemes considered were chemical grouting, Wolf Creek Type Cutoff Wall, vibrated beam slurry walls, U.S. & D.S. core trench, a toe berm and concrete cutoff in the relief well ditch and abandonment project.

SLIDE 30.

At that time piezometric contours in the alluvium is indicated uniform areas of high pressure as well as an area of sharp piezometric drop near the downstream end of the lock. To evaluate the effectiveness of a cutoff and serve as a basis for design of the complete section, a 1244 foot test section was designed to extend from the lock along the Georgia embankment.

SLIDE 31.

The test section was designed to extend through the shell layer to cutoff seepage in the area of the lock and permit dewatering the lock for repairs. Past dewaterings had triggered sinkhole activity and this was considered to be a rigorous test of the wall as a permanent solution to the seepage problems. A contract was awarded to Soletanche and Rodeo in August 1981. The 139,500 sq. ft. wall completed in December 1981 at a cost of approximately 2.5 million.

SLIDE 32.

Two of the three typical panel designs were termed primary and secondary. The primaries were excavated first in an area and served as the guide for adjacent elements. The primaries had shoulder pipes placed at each end of the excavation which served as forms for the end guides. The pipes were pulled as the concrete set leaving the guide for the adjacent panels. The secondary panel connected two adjacent completed panels and did not use shoulder pipes.

SLIDE 33.

The third panel design was termed a running or "bastard" panel. This panel extend from an existing primary or running panel with a shoulder pipe in the opposite end to serve as a guide for the next panel. Typically, the contractor would install serveral widely spaced primary panels, fill in an area using running panels and closing with a secondary.

SLIDE 34.

Another area of concern was the tie-in of the cutoff to the lock wall. Here chemical grouting was specified upstream and downstream at the contact to provide a tight bond.

SLIDE 35.

The embankment was degraded to provide a work platform.

SLIDE 36.

Concrete guide walls were installed.

SLIDE 37.

Excavation was performed under bentonite slurry.

SLIDES 38-39.

Both cable activated clamshells and hydraulic clamshells were used to excavate embankment, overburden, and rock.

SLIDES 40-41.

Concrete was tremied into the excavation and the shoulder pipes (when present) were jacked out.

SLIDE 42.

This test section allowed detailed evaluation of the construction method as well as direct observation of the effectiveness of the cutoff on the underseepage. Piezometer contours before the cutoff test section indicated the head differential and flow paths.

SLIDE 43.

After the test section was installed the contours and flow lines were distinctly interrupted. At the toe downstream of the wall there was an 8 foot drop in head; the 44 relief wells, which averaged 20 gpm flow, each, stopped flowing; and there was a 30 to 35 foot head drop across the wall.

SLIDE 44.

Based on the results of the test section evaluation of the district recommended an additional 8100 linear feet of wall, to be installed from the test section through the flood plain in the Georgia embankment and through the entire Alabama embankment. The test section evaluation served as the basis for the GDM for the final remedial measures to remedy the underseepage problems.

walls. Their negotiated cost for the project was $2,385,985.00. This includes all project costs for constructing 76,000 square feet of concrete cutoff wall surface area and 25,000 square feet of soil-bentonite panel surface area. This translates to $27.36 per square foot for construction of the concrete cutoff walls and $11.90 per square foot for the soil-bentonite panels.

After this design had been completed and after RESOCOL had negotiated the job, they revealed to us that they had developed and patented a new excavating machine in Europe for constructing concrete cutoff walls. RECOSOL stated that the machine would be a definite asset in the Cooper River rehabilitation project, particularly with regard to time, and asked permission to use it. At the same time, they also indicated that use of the new equipment would necessitate some changes to the specifications, primarily with regard to the slurry, the use of shoulder pipes, the panel lengths, and the determination of top of shale. Approval was ultimately given for use of the machine (Hydro-fraise) in a test section, with final approval withheld pending completion of the test section.

The following initial specification deviations were approved for the test section:

-- Use of shoulder pipes would not be required.
-- Length of primary panels - nominally 30 feet.
-- Length of secondary panels - nominally 8 feet.

CONCRETE CUTOFF WALL - CONSTRUCTION
WALTER F. GEORGE DAM
JERRY JANGULA - MOBILE DISTRICT

Six bidders were prequalified for this contract. Bencor of Dallas, Texas and Petrifond of Montreal, Canada submitted the joint venture low bid of $8.3 million. The contract was awarded on 9 February 1983, with NTP issued on 4 April 1983, and has a completion date of 22 August 1985.

SLIDE 1.

The contract requires the construction of 1984 lineal feet of a 2 foot thick concrete cutoff wall on the Georgia dike and 6124 lineal feet on the Alabama dike for a total of approximately 1 million square feet of wall.

The first construction is the widening of the 30' dike top to provide a 60' wide work platform. This was done by removing the top 7.7' of a portion of the dike and compacting it on the back slope of the dam.

SLIDE 2.

Two 12" x 30" reinforced concrete guide walls were then installed 27" apart. These walls were blocked and backfilled with 2" x 4" to prevent damage to the walls by the contractor's operations prior to panel excavation.

The work platform on the Alabama dike has a 16' wide road constructed with 6" minus low grade limestone. This road and the work platform in general has been fairly well maintained.

SLIDES 3-5.

The work platform on the Georgia dike was surfaced with clayey sand material and not maintained. Due to the lake of maintenance it deteriorated rapidly after a few rains and became a serious hindrance to both production and safety. I would suggest that you specify the specific work platform surfacing material you desire and that it be sloped to drain and effectively maintained, and put some enforcement teeth in the contract for the resident engineer.

SLIDE 6.

The contractor was furnished profile sheets, showing the minimum wall grade and anticipated geology, along with core boring logs and a narrative description of the different materials to be excavated. The wall ranges in depth from 108 to 191 feet with an average depth of 125 feet. The wall is excavated to the minimum grade shown on the drawing, or until it is embedded into 3 feet of solid Clayton formation material, whichever occurs last.

SLIDES 7-8.

The excavation is performed with an 11 ton, 17 feet high clam bucket with an 11 1/2 foot wide jaw opening operated by a 6-partline block/sheave system.

SLIDE 9.

The excavated material is dumped directly into trucks and hauled to onsite diked disposal areas, which were constructed by the building of dikes from material found within the limits of the disposal area.

SLIDES 10-11.

The Contractor is required to maintain a slurry level in the excavated panel a maximum of 3 feet below the top of the guide walls. The slurry is batch mixed with a high speed grout mixer and stored in storage ponds from which it is pumped to the panels.

SLIDES 12-15.

The contract specifications require the panel to be within 1% of vertical and the centerlines of adjacent panels to be within 6" of each other. This should be changed in the guide specifications, because a 100 foot deep panel could be out 1 foot downstream and be acceptable. However, if the next panel were dug vertical, it would not be acceptable because it would not meet the 6 inches between adjacent panel requirements. The specifications should have the 6 inches between adjacent panel requirements, but should limit the deviation off vertical to no more than 6 inches, because a panel can always be made more vertical, but not less vertical after its excavation has been completed. The verticality of panels is measured by means of a large plumb bob and a crane. The distance between the guide wall and the tip of the crane boom is know. The distance the cable deviates from the center of the panel and the depth of the plumb bob can be measured, and the panel's deviation from vertical can be computed, using the similar triangle method.

SLIDE 16.

If the panel does not meet the verticality requirements, it must chiseled or shaved until it does. The contractor has used three types of chisels on this project: A section of shoulder pipe which has had its down-hole end beveled.

SLIDES 17-18.

A 30 foot long star chisel (this is also the chisel he uses when he encounters material too hard for his clam bucket to dig).

SLIDES 19-20.

And a 25 foot long, 4 foot wide shaving chisel.

SLIDE 21.

There are three serious problems which can occur during the excavation process. The first is called a rapid slurry loss. This occurs when a solution feature is encountered while excavating and the slurry level drops rapidly in the panel. When this occurs, full bentonite bags and truckloads of excavated material are dumped into the panel until the slurry loss has been stopped. After this has been accomplished, a drilling and grouting program is implemented to fill the solution feature prior to re-excavating the panel.

The second is the sticking of chisels and/or buckets in the panel. This can occur either due to operator error, equipment failure, or subsurface irregularities. These items must be extracted with a large hook or clam bucket and a crane or combination or cranes, or concrete placed around them and a coverup panel dug and placed next to them to seal off any windows in the wall which could be created by the buried obstructions.

SLIDE 22.

The guide wall breakage is a third problem. It causes widening of the panel which results in verticality problems, concrete overruns, and a cribbing problem for concrete placing equipment.

SLIDES 23-24.

The shoulder pipes are installed with the aid of the 150 ton service crane and a 400 ton hydraulic jack assembly. The jack has a conical shaped hole in its center. The shoulder pipe is inserted through this hole. A belt of wedges is used to hold the shoulder pipe in the jack while additional sections of pipe are added. The shoulder pipe is made up of varying threaded lengths of 22 inch O.D. pipe having a wall thickness of 3/8 to 1/2 inch.

SLIDES 25-26.

The number of shoulder pipes installed is dependent on the type of panel being constructed. A primary panel, which occurs anywhere from 50 to 200 feet on center, is a starting panel and has a shoulder pipe on each end. A closing panel occurs at the same frequency as a primary panel. It has concrete from a previous panel on each end and therefore needs no shoulder pipes. A running panel is installed on either side of a primary panel when the distance between primary or running panels is greater than 25 feet. This panel has the concrete of a completed panel on only one side of it and therefore needs only one shoulder pipe on the opposite side of the panel. This project has a total of 87 panels on the Georgia dike and 275 panels on the Alabama side. The shortest concreted panel length has been 14 feet, the longest 25 feet, and the average 22.26 feet. To date (20 Oct 84) all the panels on the Georgia dike have been completed and 123 on the Alabama dike.

SLIDE 27.

The contract requires the sand content in the bentonite slurry to be reduced to 2% before placing concrete. The contractor has used three different arrangements to accomplish this. In all three of them an 8 inch diameter pipe is lowered to the bottom of the panel and the slurry is pumped from the bottom using either a centrifugal trash pump or airlift.

SLIDES 28-30.

The most efficient and least wasteful arrangement, when properly maintained, is the commercially available screening/desanding plant. The slurry is pumped from the panel to an intake chamber, which overflows onto the screening plant where large particles are removed and deposited into a truck. The liquid with the finer sands falls through the screen to a holding chamber below, from where a second pump transports the slurry to the hydro cyclones. Here the sands, which are heavier than the slurry, collect on the outside of the cones and fall out the bottom of the cone and into the truck, while the liquid slurry stays in the center of the cone and exits through the top of the cone and flows back to the panel.

SLIDES 31-32.

Another arrangement used by the contractor utilizes a site-fabricated desander. Here the material is merely pumped through a hydro-cyclone which has had the bottom hole enlarged to a 2 inch diameter so it will rarely be plugged by material removed from the panel. This larger hole, although it does not appear to substantially decrease the desanding efficiency of the hydro-cyclone, does drastically increase the amount of bentonite slurry which is wasted. However, this is not at all as wasteful as the third arrangement used by the contractor to desand the panel. Here he opens the by-pass valve next to the hydro-cyclone and pumps all the slurry from the panel directly into a truck. The wasted slurry is then replaced by clean slurry from the slurry ponds.

SLIDE 33.

All of the sand and slurry which is wasted during the desanding operation is deposited in a separate disposal area to prevent it from making the excavation disposal area impassable, due to its liquid condition. All of the disposal areas will have to be stabilized, using cement or dry material, prior to final acceptance of the project.

SLIDE 34.

After the sand content is below two percent, the desanding pipe is systematically moved along the bottom of the panel, to remove any loose material which may have settled out of the slurry. After this has been completed, the contractor must begin concrete placement within four hours, or repeat this operation.

SLIDE 35.

The panel bottom is measured for the first time by the Government after the bottom cleaning is completed unless the contractor was directed to over-excavate the panel. This has caused some Government/Contractor friction in the past, because on several occasions the panel has been found to be too shallow at this time. However, it has made the contractor more aware, at least in this area, of his quality control responsibilities.

SLIDE 36-37.

A tremie pipe consisting of 10 foot threaded sections of 10 inch diameter pipe is installed to within one foot of the bottom. The hopper is screened to remove lumps greater that 3" in diameter. The contract specifications state the hopper must be a minimum of 1/2 cubic yard in volume.

SLIDES 38-40.

The concrete for this project comes from a fully automatic onsite ready mix plant. The plant is controlled by a micro computer which provides a printout of each batch. The concrete is transported to the panel by six 10-cubic yard trucks.

SLIDE 41.

The specifications require the concrete to be placed within 1 1/2 hours after water is induced into the mix if its temperature is below 85 F, within 45 minutes if above 85 F, and not at all if below 40 F or above 90 F. This contractor is currently placing concrete at a rate of around 100 cubic yards per hour. A basketball is used as a go-devil between the slurry and the first batch of concrete to keep the two from mixing.

SLIDE 42.

As concrete is being placed in the panel, the bentonite slurry is pumped to a settlement pond, where the slurry is pumped through a hydro-cyclone before being reused in another excavation.

SLIDES 43-45.

All of the tremie pipe and related equipment is made so it can be easily and quickly handled to prevent problems with concrete placement, concrete quality, or shoulder pipe extraction.

The contract specifications require the tremie pipe to be embedded in the concrete a minimum of 10 feet and a maximum of 30 feet. Close records are kept of the time the concrete from each truck is placed, the depth of the concrete after each set of two trucks, the length of tremie pipe pulled, the time the tremie pipe was pulled, and the length of tremie pipe remaining. These records are used to determine when additional tremie pipe can be pulled and when the shoulder pipes can be pulled.

SLIDE 46.

Two hours after the first section of tremie pipe is pulled, the shoulder pipes are broken loose with the 400 ton hydraulic jack. Thereafter, the shoulder pipes are moved a few inches every few minutes.

SLIDE 47.

After the concrete placement is completed, the service crane is attached to the shoulder pipe to complete its extraction. This is normally completed 3 hours after the last concrete was placed in the panel.

SLIDES 48-49.

The contractor was unable to remove portions or all of 4 shoulder pipes during the concrete placement operation: one, because portions of the shoulder pipe broke off below the surface during their extraction; and two, because the guide wall broke during extraction and the combination of the jack (on a poor foundation) and the service crane could not extract them. One of the portions of shoulder pipe was broken with a chisel and the pieces pulled out with the excavation bucket. A bypass (coverup) panel was placed around the other portion. One shoulder pipe was extracted with a vibratory extractor. The contractor is attempting to form a "Wolf Creek" type of joint at the other stuck shoulder pipe.

SLIDE 50.

The contractor is experiencing a 25.5% concrete overrun on this project. Only a 0.5% overrun is due to directed over-excavation. The 25% is due to the contractor's operation. A minimum of one core hole for every 200 lineal feet of cutoff wall is required by the contract. Payment is on a per lineal foot basis, provided the contractor gets to a minimum depth of 50 feet and has 90% core recovery. This contractor has been able to drill completely through the panel and into the panel foundation a fairly high percentage of the time.

SLIDE 51.

All but one of the panels have shown an excellent quality of concrete. One side of one panel had a poorly cemented zone between 15 and 45 feet down. The reason for this was never explained and a by-pass panel was installed.

SLIDE 52-53.

The contractor is required to read designated relief wells and piezometers within 300 feet of his operations or until they stabilize. Piezometers are located both upstream and downstream of the wall. On the Georgia dike, where the cutoff wall has been completed, the piezometer readings upstream of the wall rose and the downstream fell as a result of the construction of the wall.

SLIDES 54-55.

These are pictures of the relief wells and ditch at the toe of the Alabama dike.

SLIDE 56.

This wier opening is 24 x 36 inches. The water is flowing over it at a depth of 9 inches. This is the same condition which existed at the toe of the Georgia dike before the cutoff wall construction began.

SLIDE 57.

Now the relief well ditch at the Georgia dike toe is completely dry. Not one relief well is producing water.

SLIDE 58.

Two main problems will be encountered by your people during projects similar to ours. The first is congestion. There is no way to avoid this problem if you are working in a confined area such as a dike top. This is just a problem which the government and contractor will have to learn to manage and handle.

The second problem is that we have treated cutoff wall contractors as the experts for too long. There is another complicated or mysterious about this type of work. You merely dig a ditch and fill it with soil bentonite, concrete, etc. in accordance with a set of specifications. The Corps of Engineers has a vast amount of experience in this work. We need to draw on it and instill in our field people the confidence that this construction is no different from any other type of construction. In talking to people who had administered contracts similar to ours before me, I kept being told "They're (the contractors) the "EXPERTS." I think we'll obtain a better end product and reduce contractor/government friction when this attitude is overcome by both the Government and contractors.

VIBRATING BEAM SLURRY WALL TEST CELL
JOHN GOLDEN - WILMINGTON DISTRICT

SLIDE 1.

The vibrating beam method of constructing a cement-bentonite slurry wall was tested by Wilmington District at the Military Ocean Terminal (MOTSU) near Sunny Point, NC, about 20 miles south of Wilmington, NC.

SLIDE 2.

The test cell construction was accomplished in August 1983 by contractor, American Foundations, in 3 days at a cost of $19,000.

SLIDE 3.

The slurry was injected into the fine sand and clay soils through three nozzles welded to the web of a wide flange beam (11 1/2x30) 70 feet long.

SLIDE 4.

The beam was driven to a depth of 68 feet by a 12-ton vibrating hammer to refusal in limestone bedrock.

SLIDE 5.

The cell in plan was a square measuring 20 feet on each side.

SLIDES 6-7.

Construction features are critical to continuity of the wall, such as the guide fin on the beam flange, chipping teeth on the bottom end of the beam, a controlled raising of the beam during slurry injection and control of vertical beam alignment.

SLIDE 8.

The cement-bentonite slurry mix design was formulated in the laboratory by the contractor using groundwater sampled at the test cell site.

SLIDE 9.

The slurry was mixed on site in a trailer-mounted batch plant in 4 cubic yard batches as follows:

Bentonite	400 lbs.	Tamol	5 lbs.
Fly Ash	425 lbs.	Soda Ash	8-10 lbs.
Cement	94 lbs.	Water	56.5 lbs.

Pumps and observation wells were installed in the upper and lower sand aquifers inside the test cell and observation wells were installed outside the test cell to perform field pumping tests for permeability and to determine drawdown/rebound characteristics across the walls of the cell. Pump testing took place in September-December 1983 starting with non-steady state procedures then confirming results with steady state procedures. Slurry wall permeabilities were calculated in the range of $1x10^{-3}$to $1x10^{-5}$cm/sec. The permeabilities of the sands were $1x10^{-3}$cm/sec previously determined by field pumping tests. Drawdown/rebound data showed that the slurry walls had holes or leaks.

SLIDE 10.

The upper 10 feet of the slurry wall on two sides of the test cell was excavated by backhoe and careful hand digging in February 1984.

SLIDE 11.

The total length of wall excavated was about 25 feet including one corner of the test cell.

SLIDE 12.

At several locations the wall thickness pinched down to less than one inch.

SLIDES 13-14.

In one location a 2-foot by 2-foot section of wall had sheared and displaced such that there was an opening at a depth of about 8 feet.

SLIDE 15.

In the corner one of the walls pinched out over most of the excavated depth.

SLIDE 16.

Examination of the slurry showed no filter cake had formed at the slurry/soil interface, this the contractor attributed to the use of Tamol in the mix.

Conclusions.

1. The continuity of the wall cannot be verified by inspection of construction procedures; therefore, the vibrating beam method is not considered reliable until a method to test or verify

continuity has been developed.

2. The thin wall increases the consequences of continuity defects.

3. The vibrating beam method should be tested at the site for the use intended.

Reference: SAWEN-GS Report, "Groundwater Control System," MOTSU, 20 August 1984.

CEMENT-BENTONITE SLURRY TRENCH DESIGN AND CONSTRUCTION[1]

WAYNE S. ADASKA - PORTLAND CEMENT ASSOCIATION

SLIDE 1.

Waste leachates can be contained by using slurry trenches to create a zero flow condition. A slurry trench is a non-structural underground wall that serves as a barrier to the horizontal flow of water and other fluids. It is constructed with a viscous stabilizing fluid called slurry. In the cement-bentonite (C-B) method, cement is added to a bentonite-water slurry before introduction into the trench. This slurry serves as stabilizing fluid to maintain an open trench during excavation and also remains to set up and form the permanent cutoff.

SLIDE 2.

The applications for C-B are essentially the same as for soil-bentonite (S-B) cutoffs. A simple use is to prevent migration of lighter than water waste, such as oil or chemicals. In such an application the barrier must extend below the water table but does not need to tie into an impervious layer.

SLIDE 3.

While C-B must utilize more expensive materials (the cement) than S-B, there are several advantages. The C-B method is not dependent on the availability or the quality of soil for backfill. The C-B method is more suitable in trenching through weak soils where trench stability may be concern. The C-B slurry has a higher density than S-B slurry and begins to set within hours after excavation, thereby reducing the chance of failure. The C-B slurry sets up to a stiff claylike consistency. Trenches may be cut through the wall without sloughing. Construction traffic may cross the trench after a few days. The construction sequence is more flexible. The C-B method permits trench construction in sections to meet site constraints. It adapts to hilly surfaces where a step-type construction can be performed. With the S-B method the long open trench necessary to accommodate the flat slope of the backfill normally requires trenching continuously in one direction at a constant elevation. With a C-B slurry trench, construction may proceed during sub-freezing temperatures. With the S-B method, special precautions are required to keep the backfill from freezing.

[1]This paper was complied from notes and Portland Cement Association publications on this subject.

The width of a C-B trench is generally less than for a S-B trench. For the S-B method, the trench must be wide enough to permit free flow of the backfill material. With the C-B method an area adjacent to the trench is not required for mixing, making it more suitable on projects with space limitations such as the crest of a dam. Also, cleanup is easier with the C-B method.

SLIDE 4.

Restrictive site conditions require a high degree of flexibility in the method which is available with C-B trenches.

SLIDE 5.

The permeability of C-B cutoffs, have ranged from 0.09 to 12×10^{-6}cm/sec.

SLIDE 6.

Since a C-B slurry trench is not intended to support bending moments or significant shear stresses, strength usually is not a primary consideration. The trench is generally designed to achieve a strength equivalent to that of the surrounding soil. However, on projects where slurry trenches are constructed through unstable material such as peats and mine spoils, trench stability, especially during excavation, is a critical consideration. The cement-water ratio has a significant effect on the strength of the C-B slurry trench. Also, as with concrete, strength increases with age.

SLIDE 7.

The deformability or compressibility of a slurry trench is important when considering its application beneath large dams or in seismic areas where displacements may occur. The slurry trench must be able to accommodate the displacements without cracking. A major factor that affects the deformability of C-B slurry trenches is the cement-water ratio. Laboratory tests indicate that higher strength, or a higher cement-water ratio, results in a stiffer, less deformable wall. The high strain capacity of the C-B slurry is significant even for uniaxial compressive strengths of 50 psi. (0.3MPa)

SLIDE 8.

At Commonwealth Edison's Braidwood Nuclear Power Station, Braidwood, Ill. slurry trenches were used both for excavation dewatering and as a cutoff through and beneath the exterior dikes of the 2640-acre (1070-ha) cooling water reservoir. Prior to foundation excavation a cement-bentonite slurry trench was constructed along the perimeter of the main plant. The trench was constructed through 30 feet (9 m) of fine to medium sand and keyed into the underlying glacial till. Considering the length of time the excavation would need to be dewatered, a slurry trench proved much more economical than a conventional pumping well system. In addition, the slurry trench eliminated the need for headers and other obstructions. Any water entering the excavation was removed by intermittent use of a sump pump.

SLIDE 9.

For the cooling water reservoir both C-B and S-B slurry trench methods were used. A large portion of the reservoir is situated over abandoned coal strip-mining operations. This strip-mining area consists of spoil piles and hydraulic fills of low shear strength with depths of 120 feet (37 m). A test cell was installed to evaluate the feasibility of excavating through the mine spoils and also to determine the adequacy of both the S-B and C-B slurry trench methods. The C-B trench was also used beneath the spillway and makeup/discharge structures located in the undisturbed portion of the dike. For the remainder of this portion, an S-B slurry trench was used.

SLIDE 10.

At Braidwood the panel method of excavation using a backhoe and clamshells was utilized. The upper 40 to 70 (12 to 21 m) of trench is excavated with a backhoe and the deeper portions with clamshells. An "alternating panel" method is employed. A series of primary panels are initially excavated. Following completion of at least two adjacent primary panels, excavation of the secondary panels can begin. Secondary panels are narrower to allow a minimum overlap into the primary panels and assure continuity of the trench.

SLIDE 11.

Here the trench was 2 feet wide and 180 feet deep and was excavated using a cable activated clamshell. As with S-B guide walls are not used which can result in an irregular trench at the surface. While there was no limitation on how much trench could be open at a time, usually four panels primary panels - about 80 to 100 feet in length (to full depth) were open at a time.