SEEPAGE CUTOFFS USING OVERLAPPING CONCRETE PILES

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ABSTRACT

Concrete cutoffs under new dams built on alluvial foundations have been installed around the world for decades. Such diaphragms have provided positive seepage barriers on projects where technical, logistical, political and economic reasons have weighed against the choice of some other method, such as a grout curtain. Concrete diaphragms are now also being installed on a routine basis to repair the foundations of existing dams, and where the problem is predominantly in the underlying rock, excavation using hydromill technology is common. However, there are occasions when even this powerful tool is prevented from operating by the prevailing rock conditions, and an alternative method must be sought. This paper describes the seepage remediation work conducted at Beaver Dam, Arkansas, by forming a concrete diaphragm using overlapping concrete piles. Details are also provided of the first such application, in Thailand.

INTRODUCTION

The use of concrete seepage cutoffs for new and existing dams founded on alluvium or rock is well documented. Several papers at the USCOLD Conference in 1994 on "Dam Safety Modification and Rehabilitation" described typical projects. Within the last decade, their use for remedial applications has grown dramatically in North America, fostered on the one hand by doubts about the effectiveness of grouting, and promoted, on the other hand, by aggressive marketing by certain foreign based specialist contractors. Although there are variations in equipment, methods and materials, most of these cutoffs have been constructed by the slurry trench method.

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For most of these applications, conventional clamshell, or grab methods have been adequate. However, for walls of considerable depth (e.g., Mud Mountain Dam, Washington) or when the cutoff has had to be carried deep into underlying rock (Navajo Dam, New Mexico), the method of choice has been the hydromill (Figure 1). Many successful case histories of this method are cited by Bruce (1990).

Even this excellent technology, however, is defeated by certain geological conditions, principally the hardness and abrasiveness of certain types of rock masses, and the presence of major voids leading to potentially catastrophic sudden and massive losses of slurry.

Dating from the late 1970s, a very limited but crucial, number of dam repairs in such conditions have been effected by an alternative technology - a concrete wall formed by overlapping, large diameter concrete piles. The value of this method has also been exploited to create cutoffs in rock under new dams, for example New Waddell Dam, Arizona, and at the Eastside Dam site in California.

This paper illustrates the method by providing details from the two most significant projects conducted to date, namely Khao Laem Dam, Thailand, and Beaver Dam, Arkansas.

KHAO LAEM DAM, THAILAND

As described by Watakeekul and Coles (1985), this concrete faced rockfill dam is a multipurpose structure 114m high and 1020m long. It is located on the Quae Noi River, 300 km east of Bangkok, and is founded on Ordovician and Permian limestones. These contain major karstic features to a depth of 60 m, and other significant highly permeable zones to over 180m. Conventional grouting was conducted to this depth, but it was decided to reinforce the effectiveness of this effort with a concrete cutoff in three particular places where the solutioning was especially severe. In these areas grouting would have been unacceptably time consuming, labor intensive and expensive to ensure long term protection.

It was originally foreseen to use the then newly-developed hydromill to cut the wall into the rock. However, detailed site investigation confirmed that 83 percent of the core samples gave strengths in excess of 100 MPa: the hydromill would also have been too slow and costly. It was then

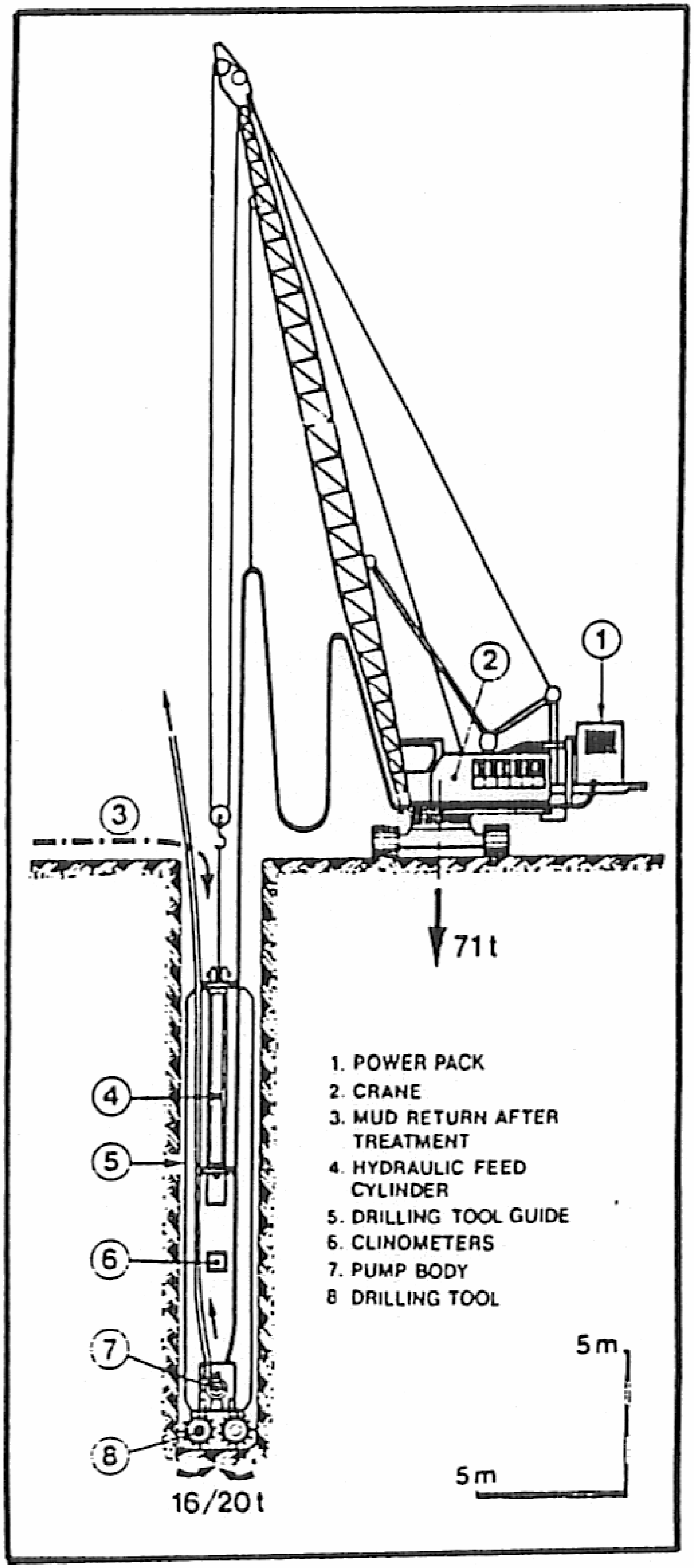


Figure 1. Equipment used for hydromill excavation

decided to use the overlapping pile method, successful during the work at R.D. Bailey Dam in the United States from 1977 to 1979. The concept is illustrated in Figure 2 - Primary holes of 762mm diameter were drilled and concreted at 1230mm centers, followed by intermediate Secondaries 36 hours later. The cutoff then varied from 450 to 762mm in thickness.

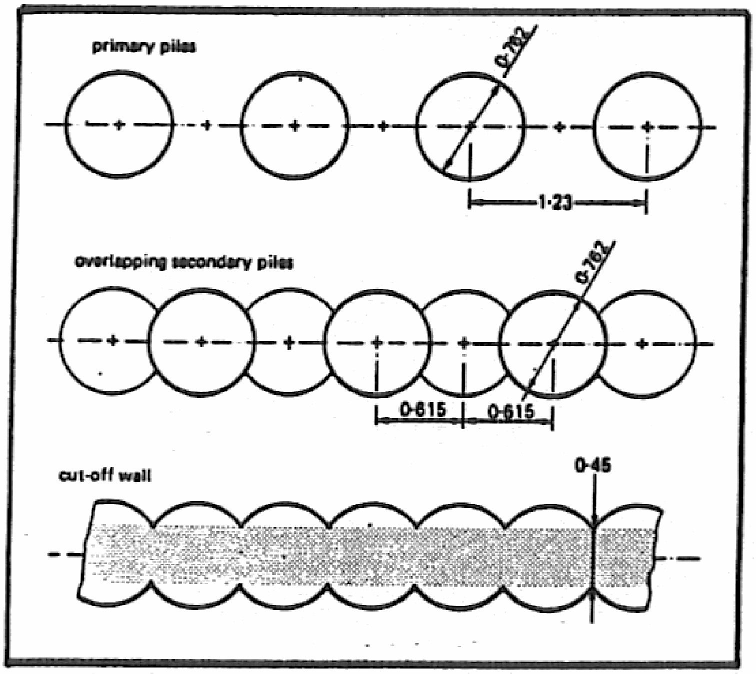


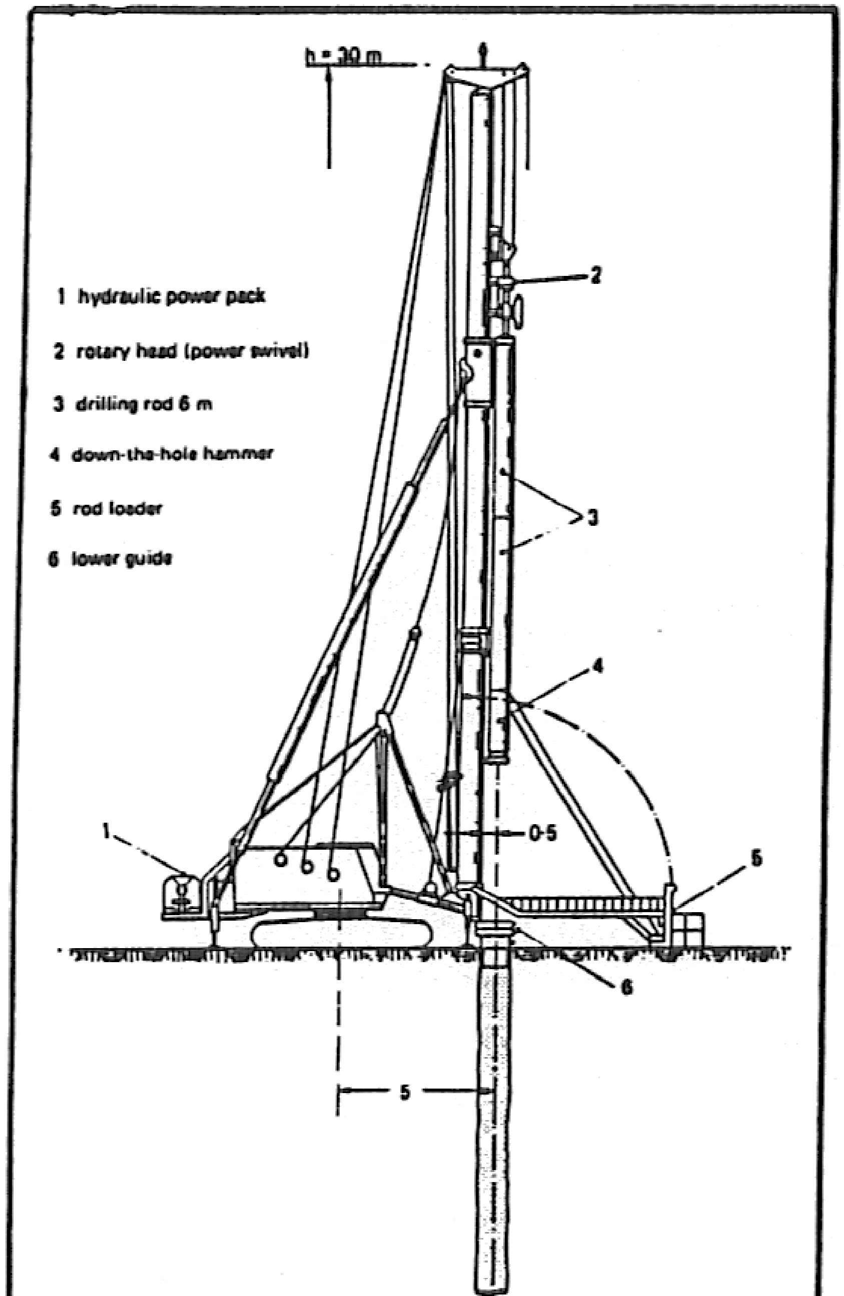
Figure 2. Construction phases of the cut off wall, Khao Laem Dam, Thailand.

The equipment used (Figure 3) featured:

- A 30-m high tubular tower on a 70 tonne crane, as a guide, and support for the head;
- A high torque rotary head (2700 kgm, rotating the drill rods at 2 to 5 rpm);
- A 686mm diameter down-the-hole hammer with a 762mm diameter bit, mounted on 6-m long rods of 686mm o.d. and 152mm i.d.;
- A compressor station providing 90 to 100 m^3/min of compressed air at 1 MPa.

This whole unit, complete with 60m of drill rods, weighed 135 tonnes.

If the hammer found a cavity, other equipment was used to clean out all soft material, by grab and by jetting with air and water. Such cavities were then backfilled with concrete prior to resumption of the pile drilling. Cavities with volumes as much as 50 m^3 were found.



The EC 80 Superdrill, where: 1 = hydraulic power pack; 2 = rotary head (power swivel); 3= drilling rod; 4 = down-the-hole hammer; 5 = rod loader; and, 6 = lower guide.

Figure 3. Equipment for drilling 686mm diameter holes, Khao Laem Dam, Thailand.

Overall, a total cutoff length of 431m was formed in this way, from 15 to 55m deep, and totaling almost 16,000 m^2. Over 18,700 m^3 of concrete was used at an average rate of 0.73 m^3/m of hole. The overall industrial average was 56 m/day, all of this being under the water table. Hole verticality was measured by pendulum and never exceeded 0.3 percent.

The authors concluded the method had no practical limits regarding the hardness of the rock, whereas depth was limited only by the available air pressure and machine torque to possibly 100m. Since construction, no evidence of piping and erosion has been recorded.

A similar, smaller application was conducted in conjunction with grouting at Mangrove Creek Dam, Australia (Mackenzie and McDonald, 1985) to prevent movement of erodible gouge material from the upstream toe of this, another high, concrete-faced rockfill, dam.

BEAVER DAM, ARKANSAS

Beaver Dam is located on the White River in Carroll County, northwest Arkansas, It was constructed for the U.S. Army Corps of Engineers between November 1960 and June 1966. It consists of a concrete gravity section 506m long, rising to a maximum height of 69m above the stream bed, flanked successively to the north by a main zoned embankment 379m long, and three smaller saddle dikes. The top elevation of the flood control pool was originally 344m, and the maximum pool elevation 347m.

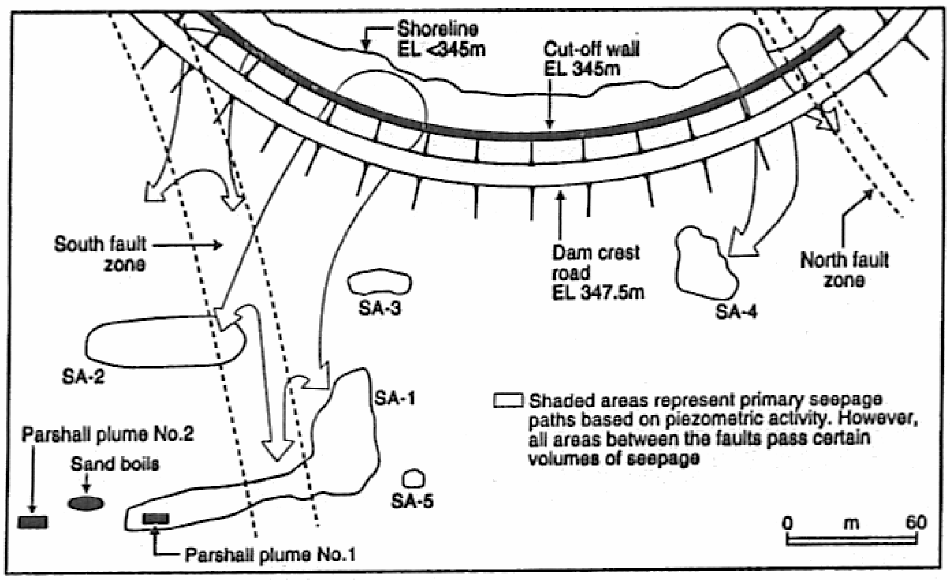


Figure 4. Relative positions of Dike One, the cut off wall and major seepage features.

This paper focuses on Dike One, adjacent to the north end of the main embankment (Figure 4). During the design period, a graben beneath Dike One had been identified as a potential problem source, due to the presence of very permeable, highly weathered Mississippian karstic limestone with clay infilling (Boone Formation). A grout curtain was installed along the center line to contemporary engineering standards. However, soon after initial filling of the reservoir, seepage was observed at several exit points on the downstream face of Dike One, totaling 50 liters/s.

Remedial grouting in 1968 to 1971 succeeded in reducing the flow to about 32 liters/s. Clearly the presence in the Boone Limestone of many open and clay-filled cavities and channels, porous strata, and deep intensely weathered permeable zones, allied to the difficulty of grouting in dynamic water flow conditions, had limited the potential effectiveness of the grouting operation.

The seepage water remained clear, but a new muddy spring was found in December 1984 after a long period of unseasonably heavy rains. Fearing material loss from the dike, the Corps decided to lower the flood control pool level to 344m. This markedly reduced the rate of clear seepage but hardly affected the new muddy flow. In addition, the reduced pool elevation directly affected flood management capacity and restricted generating capacity in the powerhouse in the concrete gravity section. A comprehensive assessment of the seepage issues was published by Llopis et al.(1988).

Concept of Solution

By February 1988, the Corps had designed a "positive" concrete cutoff wall to be installed in the bedrock upstream of the dike, with a depth varying from 24m to 56m. The first attempt to construct a slurry trench type cutoff using a hydromill failed. Apparently, those beds of relatively fresh rock had in situ compressive strengths of over 170 MPa and could not be excavated economically.

In August 1990, the Corps' resolicitation of request for proposals led to the award of a contract based on the concept of forming the wall by secant large-diameter concrete piles. Construction of the wall itself began in October 1992 and lasted for 22 months.

Ground Conditions

The graben underlies Dike One and the contiguous 61m of the northern main embankment (Figure 5). It is downfaulted about 61m between northeast-southwest trending faults, which are now characterized by zones of disturbed material. Some planes were infilled by competent breccias or solution deposits. Other fault planes were open and clean.

Under variable thicknesses of relatively impermeable overburden (typically 5m to 12m), the deeply weathered siliceous and cherty Boone overlies sound rock. The Boone is mainly spongy and chalklike, containing highly irregular tubular and sheet-like cavities. The cavities are mostly infilled with soft clay containing rock fragments and chert concentrations. The sound rock contains a network of interconnecting cavities that locally extend down to elevation 297m.

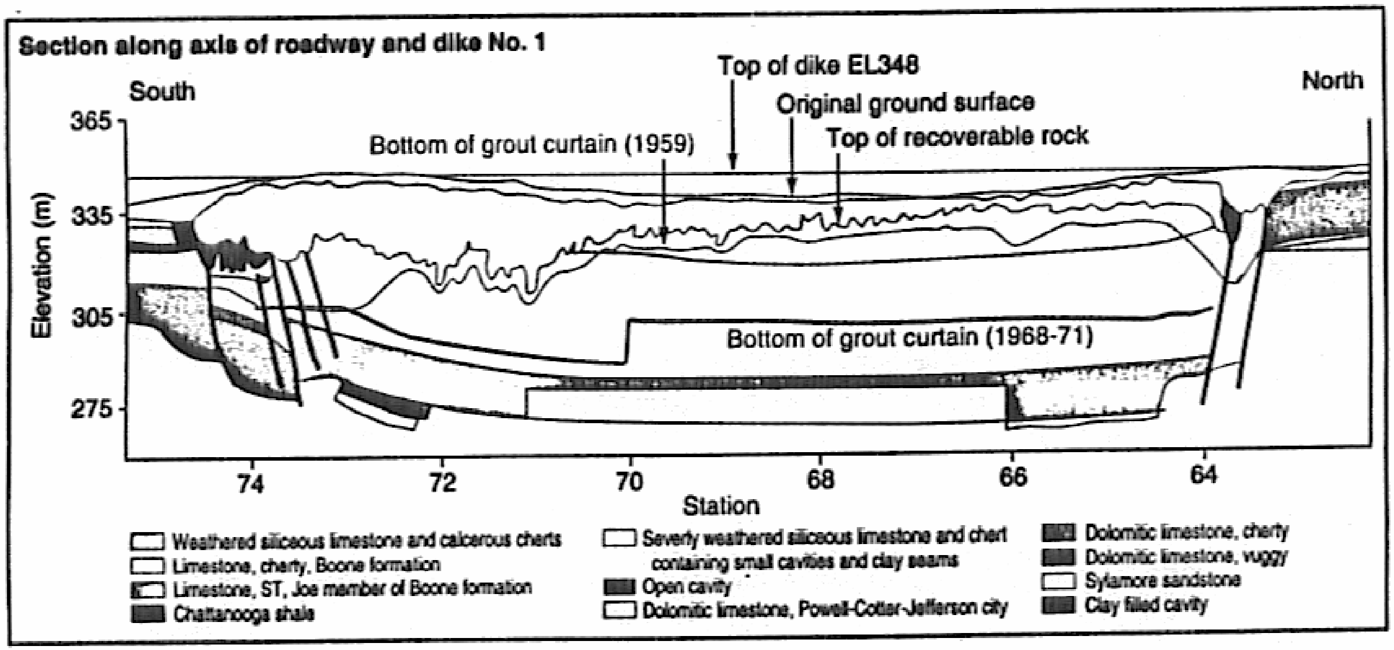


Figure 5. The inferred geology of the graben area underlying Dike One (Llopis et al., 1988).

Prior to drilling for the cutoff, the upper layers of work platform, embankment and overburden materials were excavated (by slurry wall techniques) to the top of weathered rock, and replaced by concrete. This was intended to act as a competent, in situ 1.2m thick, "casing" for the piles as the piles were subsequently advanced through these upper layers. This overburden replacement covered 3941 m^2, and consumed 5360 m^3 of concrete, mainly 21 MPa in strength.

Figure 6 shows the recorded profile of overburden depth, and the lateral subdivision of the wall, into four contiguous "areas," based on the different geological and construction conditions subsequently encountered.

Wall Geometry

As shown in Figure 6, the cutoff wall extends for a total length of 464m from Dike Station 62+00 to 77+22. It is offset 20m upstream of the embankment centerline, and needed a work platform, benched into the upstream face of Dike One at Elevation 344m. This platform was 10m to 23m wide.

The wall depth varied in response to the geological conditions from 24m to 56m although pile 572 reached 66m for exploratory purposes. The total wall area was 19,300m². A total of 24 additional ("conforming") piles were installed, mainly in Areas A and D to assure the required pile overlap at full depth.

The individual piles were located at 610-mm centers, yielding a nominal choral joint width of 610mm (Figure 7). This overlapping pile method was executed in two stages:

- In Stage 1, a series of "primary" piles was drilled and concreted;
- In Stage 2, the intermediate "secondary" piles were installed to complete the cutoff.

The following general rules were observed to avoid disturbing nearby piles being drilled, or which had been recently concreted:

- Drilling was permitted only beyond a distance of 9m from an adjacent open pile not entirely in rock:
- Minimum elapse of 48 hours after completion of concreting in a primary pile before drilling the next successive primary pile;
- Drilling of a secondary pile only when the concrete of the two adjacent primaries had reached at least 14 MPa unconfined compressive strength.

Equipment

Two drill rigs were used for drilling pile holes. The first was originally used at Khao Laem Dam. The second machine was designed and built on site and was generally a more powerful evolution of the original, comprising a Manitowoc 4100 crane, a power pack, a Watson rotary head, and two lateral rod changers.

The drill rods were in 9m lengths, with an outside diameter of 813mm and an inner air passage of 302mm. Each rig could drill 21m in a single pass. Rod rotational speed was varied from 2 to 10 rpm in response to ground conditions.

Different models of down-the-hole hammers - some on trial only - were used. However, the main types, each equipped with a bit 864mm in diameter, were the Ingersoll Rand DHD130A and the Sandvik XL24. Each hammer was equipped with an internal check valve to allow it to operate underwater.

A successful experiment was also made, specifically for penetrating a zone of very abrasive cemented (Sylamore) sandstone, with an Ingersoll Rand DC24-5 cluster drill. This equipment comprised a 610mm diameter shell, housing

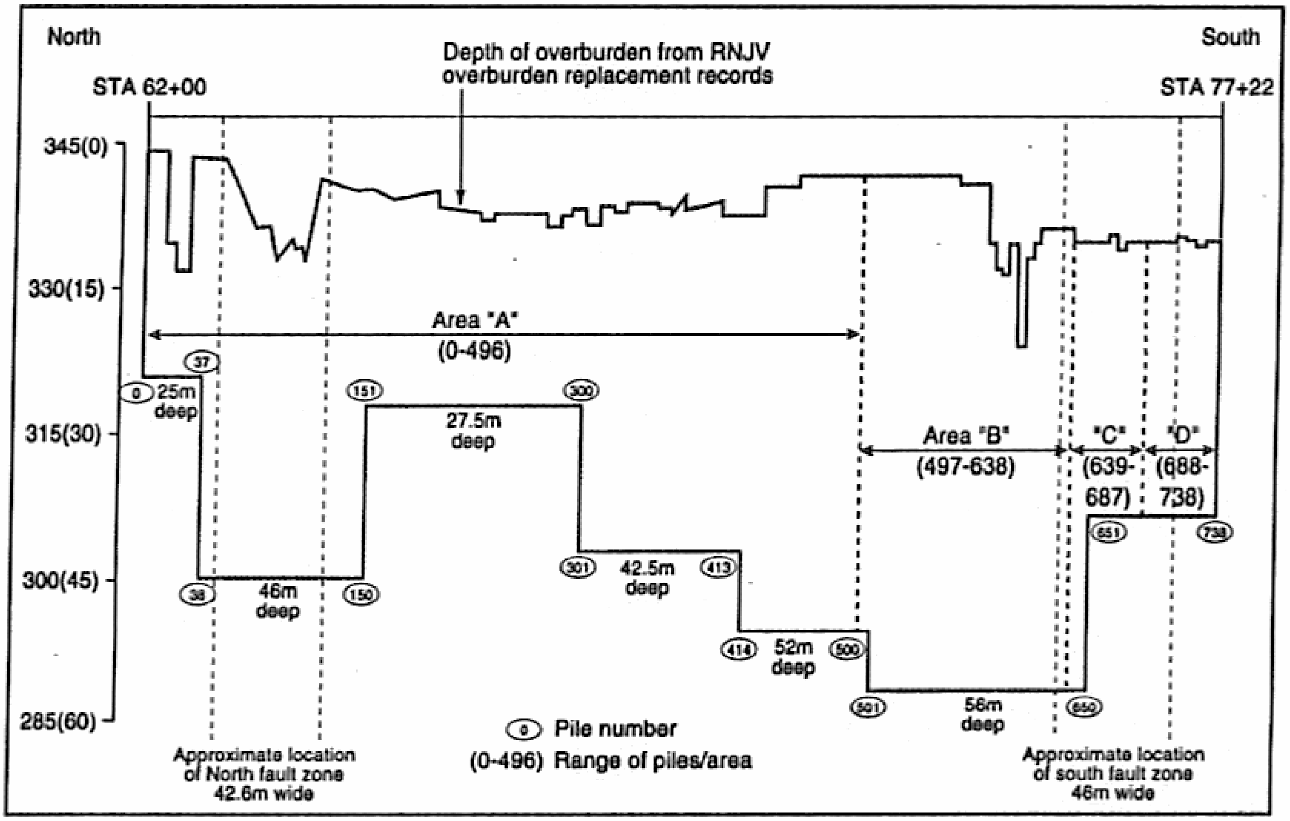


Figure 6. Elevation of the cut off showing main construction parameters.

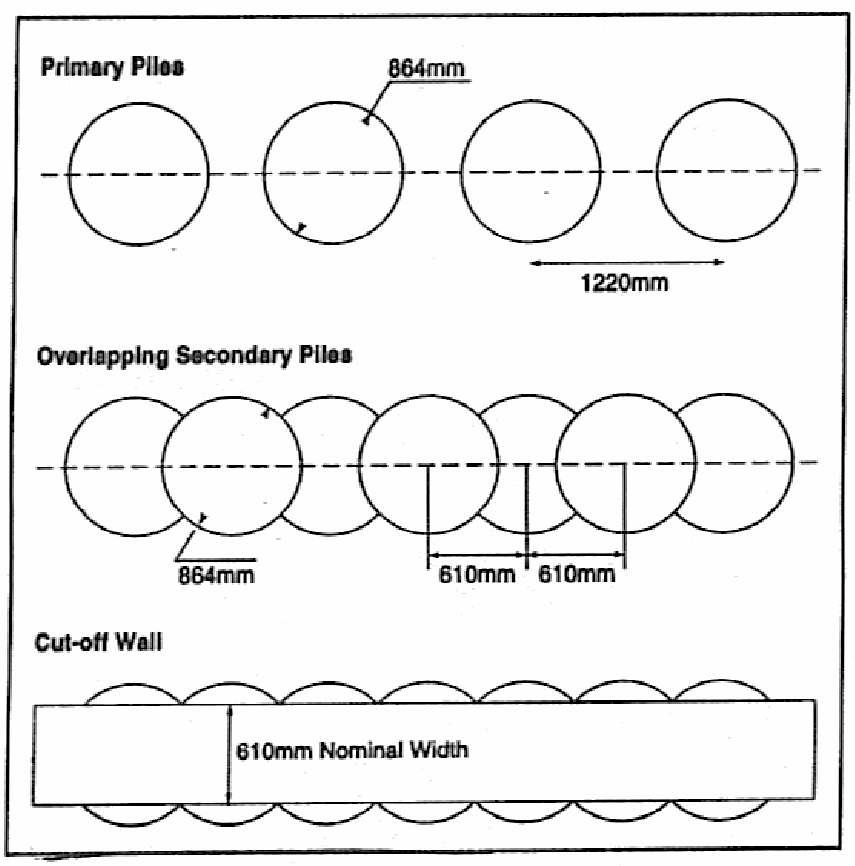


Figure 7. Construction sequence of the cut off wall.

five conventional down-the-hole hammers each of 203mm diameter.

The 1 MPa compressed air supply to each hammer was supplied from a bank of nine, static electric-powered compressors each with a volume capacity of $34m^3$/min at maximum pressure of 2.1 MPa. The compressors were arranged in two groups of four each, with one spare or supplemental, depending on the individual rig requirements.

Drill penetration rates for primary piles ranged from 2.4 m/hr to 6.3 m/hr (average 4.3m/hr), and for secondaries from 4 m/hr to 7 m/hr (average 5.4m/hr). These rates varied considerably from area to area and from rig to rig.

The major item of auxiliary equipment was a Link-Belt LS-338 crawler carne for installing and withdrawing concrete tremie pipes. The nearby concrete batching plant and the fleet of truck mixes were furnished by another major subcontractor.

Other items included:

- A hydraulic crane, for general lifting duties;
- A truckhoe for digging drainage ditches and settlement ponds;
- Various payloaders, backhoes, dump trailers, flatbed trailers, tractors, offices and workshops;
- Miscellaneous equipment for special activities, such as overburden replacement, pressure grouting, drainage of the platform, and so on.

In addition, there were various QA/QC instruments (in addition to material testing equipment), including:

- The survey and laser systems used for setting up the drill rigs and controlling drill string alignment;
- A special device for hole verticality control.

Other activities were subcontracted, and included preliminary site surveying, electrical installations, concrete coring and testing, and site restoration.

As many as 70 management, supervisory and general labor personnel were involved at the peak of construction in mid 1993.

Construction

The standard sequence of pile construction was:

- Setting up the drill rig, using theodolites and lasers;
- Drilling using air pressure commensurate with the local geological conditions. Constant monitoring, and adjustment if necessary, of mast verticality, including after each rod change;
- Extraction of rods, and sounding of exact hole depth;
- Removal, if necessary, by airlift of any soft debris accumulated at the pile toe. This process was enhanced by bucket or grab if larger debris was found;
- Verticality of hole verified by a device called a Submersible Reverse Plum Bob;
- Placement of concrete via 254mm diameter tremie tubes fed by a 1.1m^3 hopper with screen. These tubes were progressively withdrawn during filling, with the toe always embedded 1 to 6m in the concrete.

The details of the method had been reviewed intensively and agreed between the Corps and the contractor. However, as work progressed, many changes were made, both in the interests of progressive improvement and efficiency, and in response to unforeseen site and/or geotechnical problems. The more remarkable changes were as follows:

Down staging: Some problems of ground instability were foreseen while drilling through the weathered rock, i.e., below the overburden replacement and above the bedrock. When these instabilities prevented continuous drilling to full depth, the rods were extracted and the pile depth sounded. Following removal, whenever necessary, of appreciable amounts (more than 0.7m) of loose material, the hole was backfilled from the surface by concrete. No earlier than 24 hours later, the hole was redrilled through the unstable zone. A total of 71 holes in Area A (Figure 6) were completed in this way, some requiring three successive treatments. These piles involved 1855m of redrilling and 1614m^3 of concrete.

Ground pre-treatment by pressure grouting: For a 91m long section in Area A, a layer of coarse gravel was encountered, and a test grouting operation undertaken over a 36m long section. Two rows of holes were installed, 1.2m apart, respectively 0.3m upstream and downstream of the cutoff. Steel casings 178mm in diameter were drilled to the level of "recoverable rock" and cementitious mixes injected during their withdrawal. Totals of 1062m of drilling and $216m^3$ of grout were involved. Thereafter 14 piles were installed in this grouted zone, without the need for downstaging. This trial showed that the principles of grouting could be well employed to fill voids and permeate loose, cohesionless materials in the weathered zone.

Hole stabilization by grouting: During wall construction in Areas B and D, severe problems were posed by the instability of the weathered rock. In Area D, one consequence was settlement of the work platform and the appearance of sinkholes, notably near piles 690 and 714. The sinkholes were excavated, examined and then backfilled with lean mix concrete, while activities on several other piles of construction in this area were suspended. After much discussion, a modified grouting-based method was selected, in which the work platform, embankment, overburden, weathered rock, and the top 0.9m to 1.5m of sound rock were to be treated. Basically, the percussive drilling was interrupted at various depths and the resultant (partial) hole visually inspected. Once the maximum achievable stable depth was identified, the rods were reintroduced, but with a 813mm diameter rock roller bit at their tip. Grout was then pumped through the rods and bit, and simultaneously mixed with the unstable material, while also filling voids. In this way, efficient stabilization was achieved in this section of hole. The method was repeated where necessary until stable bedrock was reached, and it proved highly successful in permitting the standard construction methods to be then used to hole completion. A total of 51 piles in area D were treated in this way, some requiring as many as six (Pile 714) successive treatments. In total, over $1124m^3$ of grout were injected plus $101m^3$ of concrete used in the more conventional downstaging process in certain piles in less problematical sections.

In Area B, the early piles showed the existence of particularly unstable, weathered rock, containing cavities, voids, and very soft clay pockets in places. This zone ranged from 4.5 to 27m deep. Basically the same method proved in Area D was used there also, with similar success. The process was needed in all the holes in the area, at least once, and as often as six times (Piles 534 and 550) and, on one occasion (pile 584), ten times. Totals of $3801m^3$ of grout and $1952m^3$ of downstage concrete were used, both volumes considerably in excess of neat drilled hole volume, emphasizing the very cavitated nature of the ground in this area.

The various concrete mixes used during construction were produced by an automatic batching plant in the immediate project area. It was rated at $150m^3/hr$. Transport to the cutoff was by means of $7m^3$ truck mixers. Mixes were varied during the work in response o experienced gained, and strong QA/QC measures were enforced both at the batching plant and at the point of placement for both fluid and set properties. The most commonly used mixes had the following composition:

Coarse aggregate	950 to 90 kg/m^3
Fine aggregate	760 to 810 kg/m^3
Cement	290 to 240 kg/m^3
Flyash	60 to 80 kg/m^3
Water	160 to 136 $liters/m^3$
Reducer N	0.4 to 0.5 kg/m^3
Reducer L	0.3 to 0 kg/m^3
Air entraining agent	approximately 0.1 kg/m^3
Calcium	0 to 1.0 kg/m^3

Water was heated or chilled, depending on the other material and ambient temperatures. Total quantities of work conducted are summarized in Table 1.

Effectiveness of the Cutoff

Data were recorded from the existing seepage monitoring instrumentation. Of five seepage areas (SA-1 to SA-5), the area of most concern was SA-1, located in a natural gully 94m downstream of the centerline of Dike One at Station 71+00 between Elevations 321 and 315m.

Besides the piezometric network in the three embankments, specific measuring devices were installed to monitor the seepage exiting downstream of Dike One.

Major observations of those devices in the SA-1 area include:

- Parshall Flume Nos. 1 and 2 - the rise in flows in mid June 1993 was associated with pumping excess surface water from the work platform to the other side (downstream) of the dike. By mid March 1994, Area D had been completed and the grout stabilization of Area B commenced: flows decreased. By mid September 1994, after surface pumping had ceased, the remnant seepage was barely 0.3 liters/s, compared to a maximum of over 81 liters/s in September 1993.

Item	Description		Cut-off wall areas and pile numbers Area 'A' 0-496	Area 'B' 497-638	Area 'C' 639-687	Area 'D' 688-738	Total
1	**Cut-off wall construction**						
1.1	Drilling footage design	(ft)	63,543.5	26,255.0	7,030.0	6,630.0	103,458.5
1.2	Drilling footage actual	(ft)	63,922.0	26,259.0	7,030.0	6,639.0	103,850.0
1.3	Surface footage design	(sq. ft)	127,087.0	52,510.0	14,060.0	13,260.0	206,917.0
1.4	Surface footage actual	(sq. ft)	127,844.0	52,518.0	14,060.0	13,278.0	207,700.0
1.5	Set-ups	(no)	497	142	49	51	739.0
1.6	Concrete volume	(cu. yd)	19,057.3	7,069.5	1,950.0	1,815.0	29,891.8
1.7	Concrete design theoretical volume	(cu. yd)	14,869.2	6,143.7	1,645.0	1,551.4	24,209.3
1.8	Concrete OB design volume	(cu. yd)	4,188.1	925.8	305.0	263.6	5,682.5
1.9	Concrete OB design percentage	(%)	28.17%	15.07%	18.54%	16.99%	23.47%
1.10	Concrete actual theoretical volume	(cu. yd)	14,957.7	6,144.6	1,645.0	1,553.5	24,300.8
1.11	Concrete OB actual volume	(cu. yd)	4099.6	924.9	305.0	261.5	5591.0
1.12	Concrete OB actual percentage	(%)	27.41%	15.05%	18.54%	16.83%	23.01%
2	**Grout stabilisation**						
2.1	Drilling footage	(ft)	0.0	10,667.0	0.0	2,974.0	13,641.0
2.2	Set-ups	(no.)	0	143	0	60	203.0
2.3	Grout volume	(cu. yd)	0.0	4,971.0	0.0	1,470.2	6,441.2
2.4	Grout theoretical volume	(cu. yd)	0.0	2,496.1	0.0	695.9	3,192.0
2.5	Grout OB volume	(cu. yd)	0.0	2,474.9	0.0	774.3	3,249.2
2.6	Grout OB percentage	(%)	0.00%	99.15%	0.00%	111.26%	101.79%
3	**Downstaging**						
3.1	Drilling footage	(ft)	4,460.0	7,587.0	348.0	249.0	12,644.0
3.2	Set-ups	(no.)	91	116	4	7	218.0
3.3	Concrete volume	(cu.yd)	2,111.0	2,553.0	199.0	132.0	4,995.0
3.4	Concrete theoretical volume	(cu. yd)	1,043.6	1,775.4	81.4	58.3	2,958.7
3.5	Concrete OB	cu. yd)	1,067.4	777.6	117.6	73.7	2,036.3
3.6	Concrete OB percentage	(%)	102.27%	43.80%	144.38%	126.55%	68.82%
4	**Redrilling**						
4.1	Redrilling footage	(ft)	6,087.0	18,770.0	501.0	3,685.0	29,043.0
4.2	Set-ups		(no.)	98	211	4	69 382.0
5	**Conforming piles**						
5.1	Drilling footage actual	(ft)	2,380.0	1,480.0	0.0	0.0	3,860.0
5.2	Set-ups	(no.)	16	8	0	0	24.0
5.3	Concrete volume	(cu.yd)	623.5	374.0	0.0	0.0	997.5
5.4	Concrete actual theoretical volume	(cu.yd)	556.9	346.3	0.0	0.0	903.2
5.5	Concrete OB actual volume	(cu.yd)	66.6	27.7	0.0	0.0	94.3
5.6	Concrete OB actual percentage	(%)	11.96%	7.99%	0.00%	0.00%	10.44%
6	**Overburden replacement**						
6.1	Surface footage actual	(sq.ft)					42,415
6.2	Volume of lean concrete	(cu.yd)					682.0
6.3	Volume of non-spec concrete	(cu.yd)					6,329.0
6.4	Total volume of concrete	(cu.yd)					7,011.0

Table 1. Summary of major work quantities.
1 ft = 0.3048 m; 1 ft^2 = 0.93 m^2; 1 yd^3 = 0.74 m^3

- V Notch Weir - a sharp decrease (in both seepage and pumped water) also occurred from mid March 1994 but by late August 1994, when the cutoff was completed, it had totally dried up.
- French Drain Weir - until September 1993 the flow was related to lake level stabilizing at 0.2 liters/s when the level fell below 341m. At the beginning of grout stabilization in Area D in later January 1994, a sharp increase in flow occurred - greater than attributable to lake level fluctuation alone. This suggested a redistribution and concentration of flow paths by the treatment. The flow peaked at 0.7 liters/s on March 14, 1994, three days after the completion of Area D. However, by the end of March 1994, the flow had dropped to 0.1 liters/s, and eventually dried up totally two weeks later.
- Artesian Well - reacted, with delay, to lake level, but showed a 12.6m drop in March 1994, despite the rise in lake level. In mid June 1994 it dropped to its "drying up" elevation of 325m.

In general, these measurements showed that, prior to construction, the seepage was tens of liters/s, varying with lake level. In mid March 1994, when Area D was completed and grout stabilization in Area B was commenced, all devices showed a sharp decrease. This trend continued until the completion of the whole wall in August 1994, when all five seepage areas had dried up and the total underseepage was barely 0.3 liters/s.

Final Remarks

This massive and critical dam rehabilitation project was executed in the face of major geological, logistical and QA/QC challenges. It was completed in a timely fashion to high technical standards, with an outstanding safety record and minimal environmental impact. This excellent result reflects the benefits of the formal Partnering process which was systematically pursued throughout the project by the owner and the contractor. The open lines of communication and high levels of mutual respect permitted both parties to resolve issues on a daily basis, and to deal optimally with the major challenges as they evolved.

ACKNOWLEDGMENTS

This paper summarizes the efforts of a large number of dedicated and motivated people over a long period: to single out specific contributors would risk causing offense to the great majority, through omission. The organizations who share the credit for this project include the U.S. Army Corps of Engineers, Rodio, Nicholson Construction Company, and various major subcontractors, including Keystone Drilling Services, Inc. and Beaver Lake Concrete, Inc.

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