CHAPTER 4. CONSTRUCTION OF PILOT-SCALE SOIL-BENTONITE CUTOFF WALLS

4.1 Introduction

This chapter describes the construction of the three pilot-scale soil-bentonite cutoff walls (called W1, W2, and W3) at the SBTF that were tested in this research. Construction techniques, equipment, and materials are described. After this introduction, the chapter is divided into nine additional sections. The first eight correspond to the construction steps outlined in Chapter 3, with the exclusion of Step 8 (hydraulic conductivity testing), which is discussed in detail in Chapter 5. In the last section, 4.10, the key points in this chapter are summarized and conclusions are presented.

For each construction step, the basic construction procedure is described, and specific data for each wall is given as conditions change from wall to wall. Table 4-1 highlights the key differences between the three walls. The main differences were geometry, type of support slurry, and bentonite content in the soil-bentonite. These differences, and reasons for the differences, are briefly described in the following paragraph.

The soil-bentonite for W1 was designed to have a hydraulic conductivity around 1×10^{-6} cm/s so that the flow rate through the wall during measurement of the wall's average hydraulic conductivity would be large enough to measure easily. In addition, the trench was excavated using a biodegradable slurry so that there would be no low-permeability filter cakes during the hydraulic conductivity tests. Unfortunately, a defect existed at the bottom of W1. In an effort to avoid another defect, the CCL in W2 was roughly doubled in thickness (from 0.3 m to 0.6 m) and the width of the trench was doubled in thickness (from 0.3 m to 0.6 m). To make a cutoff wall of similar volume to W1, the length of W2 was shortened. Instead of compacting the clay liner along the entire bottom of the barrier pit, two 1.5-m-thick compacted clay bulkheads were constructed, one vertical and one at a slope of 1:1. The 1:1 slope was for initial placement of the soil-bentonite during backfilling of the trench. Despite these changes, defects existed in W2 also. It was concluded that the defects were attributable to use of the biodegradable slurry. For W3, the trench was excavated using bentonite-water slurry. Because the use of bentonite-water slurry results in low-permeability filter cakes, the hydraulic conductivity of the soil-bentonite was designed to be around 1×10^{-7} cm/s in order to reduce the influence of the filter cakes on the hydraulic condutivity tests. The lower hydraulic conductivity results in a lower flow rate through the wall when trying to measure its average hydraulic conductivity, so it was desired to use the entire length of the barrier pit to increase the area available for flow. Arrangements were made so that twice the volume of W1 could be made for W3; the CCL for W3 was roughly 0.6-m-thick and the thickness of the wall was also 0.6 m. W3 ran the entire length of the barrier pit.

The sections in this chapter are:

Table 4-1. Summary of key differences between the three walls

	Wall number: 1	2	3
Compacted clay liner: Compaction method	4 passes sheepsfoot roller, 1 pass rammer	3 passes rammer	4 passes sheepsfoot roller, 1 pass rammer
Total nominal thickness (m)	0.3	0.6	0.6
Soil-bentonite: Bentonite content (%)	1	1	3
Form of bentonite added	All in slurry form	All in slurry form	Half in slurry form, half in dry form
Support slurry: Slurry type	bioslurry	bioslurry	bentonite-water
Trench excavation: Nominal trench width (cm)	30.5	61.0	61.0

4.2 Step 1: Construction of Compacted Clay Liner (CCL)

4.2.1 Material Used

The compacted clay liners were constructed of a material called Washout, which is a by-product of the production of Light Castle Sand at the Castle Sand Company in New Castle, Virginia. The Washout material is removed from a sand called Dark Castle Sand to make the clean Light Castle Sand.

The engineering firm Froehling and Robertson, Inc. performed lab tests on a sample of the Washout material from October, 1992. Figure 4-1 shows the results of their Standard proctor compaction test (D698 in ASTM, 1992), Atterberg limits test (D4318 in ASTM, 1992), and specific gravity test (D854 in ASTM, 1992). Froehling and Robertson, Inc. classified the material as high plasticity silt, MH, according to the Unified Soil Classification System (USCS). In addition, the hydraulic conductivity of a sample compacted with Standard proctor energy at optimum water content was measured to be 8×10^{-8} cm/s.

The Washout material was also tested in this research. The Standard proctor compaction curve and Atterberg limits are shown in Figure 4-1. The specific gravity of 2.78 from Froehling and Robertson, Inc. was confirmed. The USCS classification is again MH. The hydraulic conductivity of a Washout sample was measured using a flexible-wall permeameter (D5084 in ASTM, 1992). The sample was formed in a Standard proctor compaction mold by placing three lifts and tamping each lift 21 times with the Standard proctor hammer. This procedure resulted in a dry unit weight of 14.2 kN/m^3 for the sample of water content 31.4%. Once in the flexible-wall cell, the sample was back-pressure saturated and then consolidated to an effective stress of 28 kPa. The hydraulic conductivity of the sample, measured using the falling headwater/rising tailwater procedure, was 1.3×10^{-7} cm/s.

4.2.2 Procedure

The Washout material was placed in lifts and compacted in the bottom of the barrier pit. A loose lift thickness of 13 cm was used for all three pilot-scale cutoff walls. After placing a lift in the barrier pit, large clumps of the Washout were broken up with pick axes and shovels. Water was then added to the lift if necessary, and the lift was worked with a rototiller. The rototiller acted to homogenize the lift; it mixed in the added water if necessary or dried out the soil if necessary. For low hydraulic conductivity, a water content wet of optimum, approximately 31%, was desired for compaction.

Two compactors were used for compaction of the Washout material: a rammer ("jumping jack") and a sheepsfoot roller. The rammer was a 61-kg Wacker BS60Y. The sheepsfoot roller was a 61-cm-wide Multiquip Rammax P33/24 FMR Trench Roller. Three different compaction methods were investigated: 1) two passes with the rammer with a 15-cm loose lift thickness; 2) three passes with the rammer with a 15-cm loose lift thickness; and 3) four passes with the sheepsfoot roller and one pass with the rammer with a 13-cm loose lift thickness. Methods 1 and 2 were performed on a CCL that ended up not being a part of W1, W2, or W3. Method 3 was performed on the CCL for W1. Unit weight and water content measurements, shown in Figure 4-2, were made using a Troxler nuclear moisture/density gauge. Water contents from the nuclear gauge were increased by the addition of 2.8% based on a calibration study in which laboratory water content measurements were made on samples of the CCL where the nuclear gauge was used. To check the moist unit weight measured by the nuclear gauge, a Shelby tube was pushed into the CCL where Compaction method 3 was used. The unit weight of the sample in the Shelby tube was 18.5 kN/m^3. All of the moist unit weight measurements

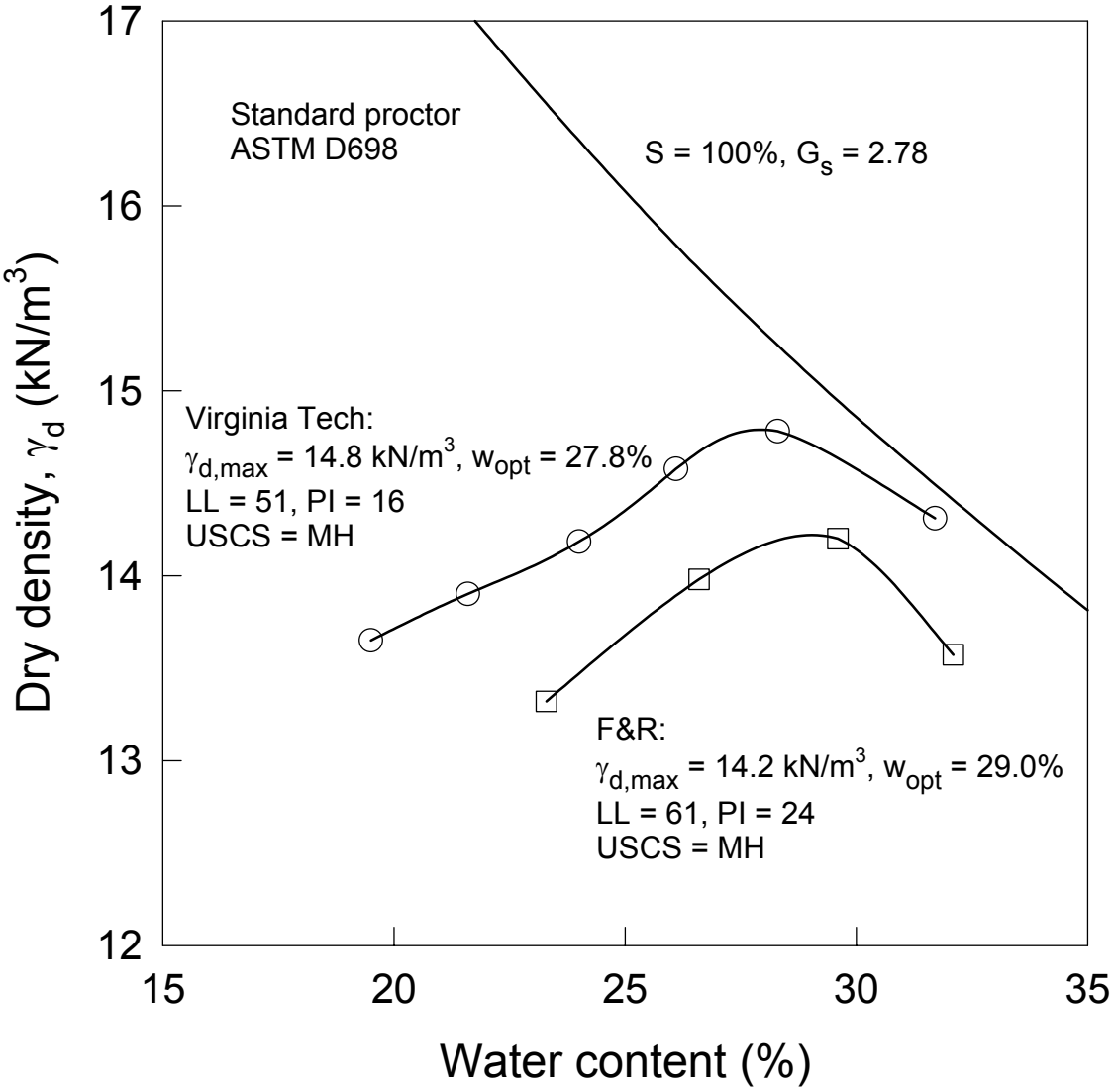

Figure 4-1. Compaction curves for Washout material used for CCL in pilot-scale experiments

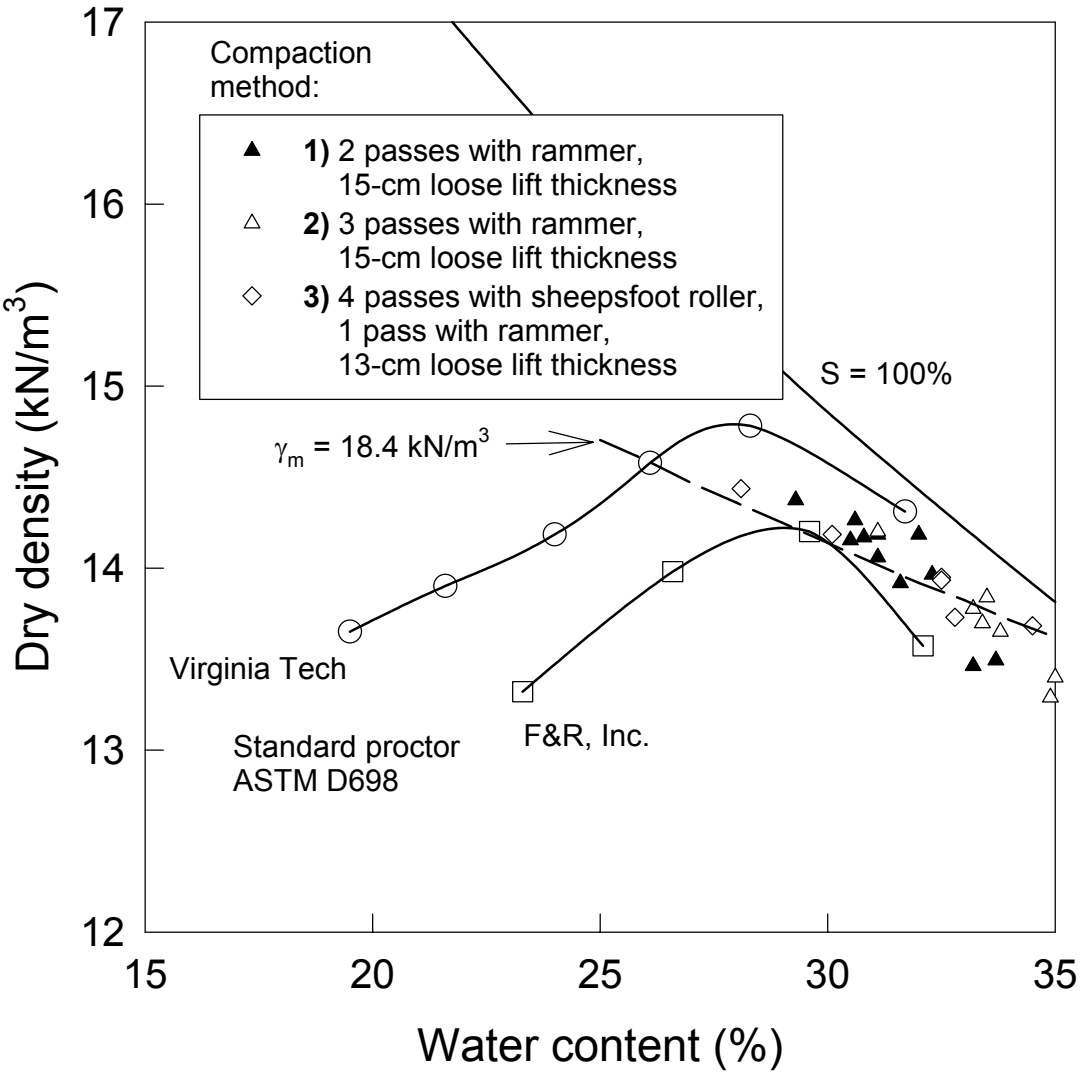

Figure 4-2. Compaction characteristics of Washout material from three different compaction methods

for this compaction method were in a tight range: minimum 18.2, maximum 18.5, and average 18.4. A contour of moist unit weight (γ_m) = 18.4 kN/m^3 is shown in Figure 4-2. With the moist unit weight from the nuclear gauge confirmed by the Shelby tube measurement, dry unit weights were calculated using the nuclear gauge moist unit weights and the nuclear gauge water contents plus 2.8%.

The compaction methods for the CCLs in W1, W2, and W3 are shown in Table 4-2. CCLs for W1 and W3 were constructed using Method 3. Due to the geometry of the CCL for W2, the CCL was constructed using a variation of Method 2: 3 passes with the rammer but with 13-cm loose lift thickness. The CCLs for W1 and W3 were constructed before any sand was compacted in the barrier pit. For the CCL for W2, the bottom portion of the CCL was compacted first, then the bulkheads were extended upward in the following manner: the bulkheads were compacted roughly 30-cm high with supports at their edges, then sand was compacted between the bulkheads to a height just below the top of the bulkheads (to keep sand out of the bulkheads), then the bulkheads were raised another 30 cm, and so on to the top of the barrier pit.

Unit weight measurements were not made for the CCLs for W2 and W3; it was assumed that by controlling the compaction method and water content, unit weights in the range shown for the three compaction methods in Figure 4-2 would be attained. The compacted thickness of the lifts was approximately 10 cm.

The total thicknesses of the CCLs were measured after they were constructed. The total thickness of the CCL for W1 was roughly 0.3 m, and the total thickness of the CCLs for W2 and W3 was roughly 0.6 m. Figures 4-3, 4-4, and 4-5 show the measured total thicknesses of the three CCLs along the length of the barrier pit. Figure 4-6a is a photo of the completed CCL for W1.

Table 4-2. Characteristics of the compacted clay liners

	Wall number:		
	1	2	3
Loose lift thickness (cm)	13	13	13
Compaction method	4 passes sheepsfoot roller, 1 pass rammer	3 passes rammer	4 passes sheepsfoot roller, 1 pass rammer
Total nominal thickness (m)	0.3	0.6	0.6

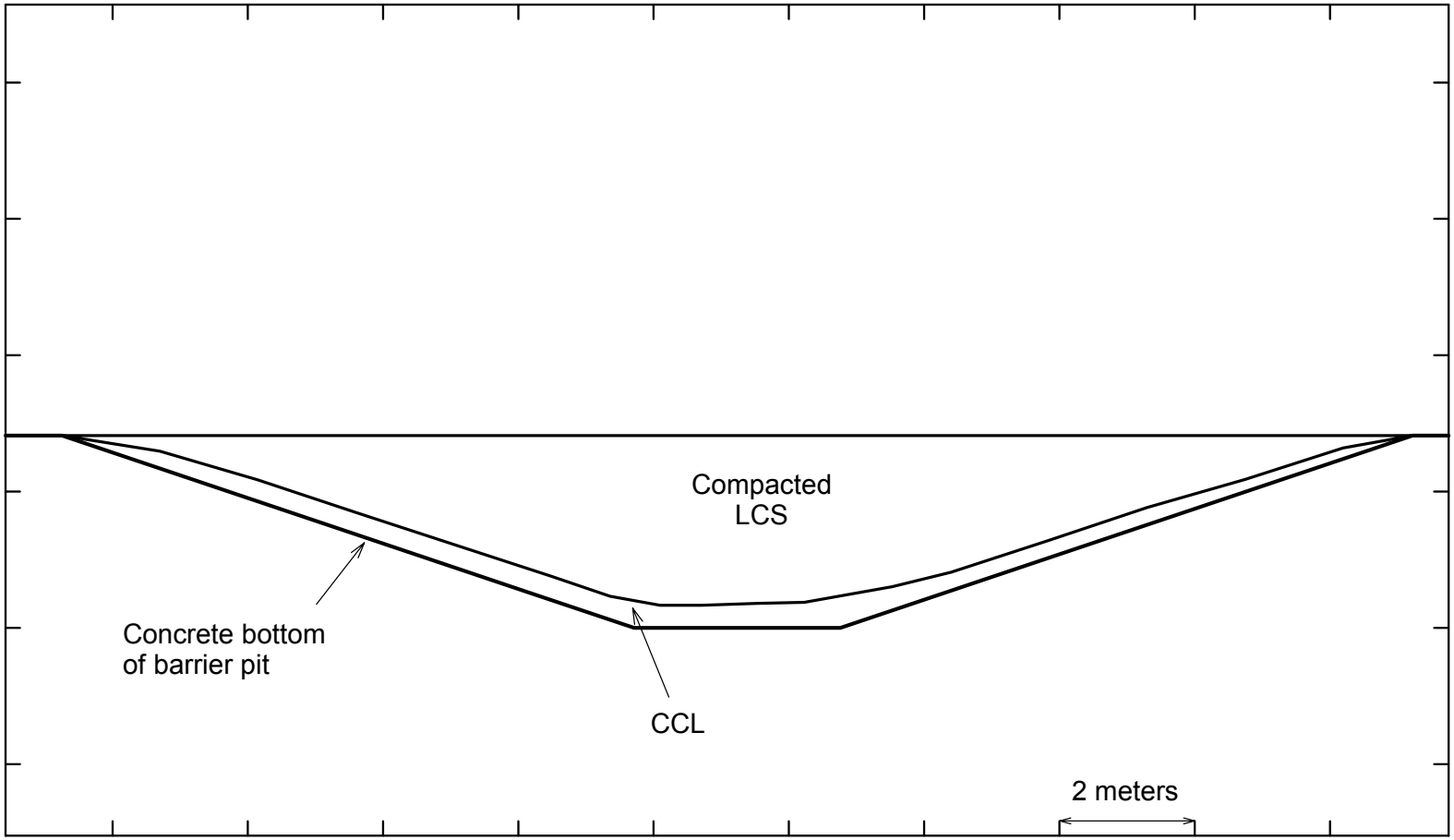

Figure 4-3. CCL and LCS geometry for W1

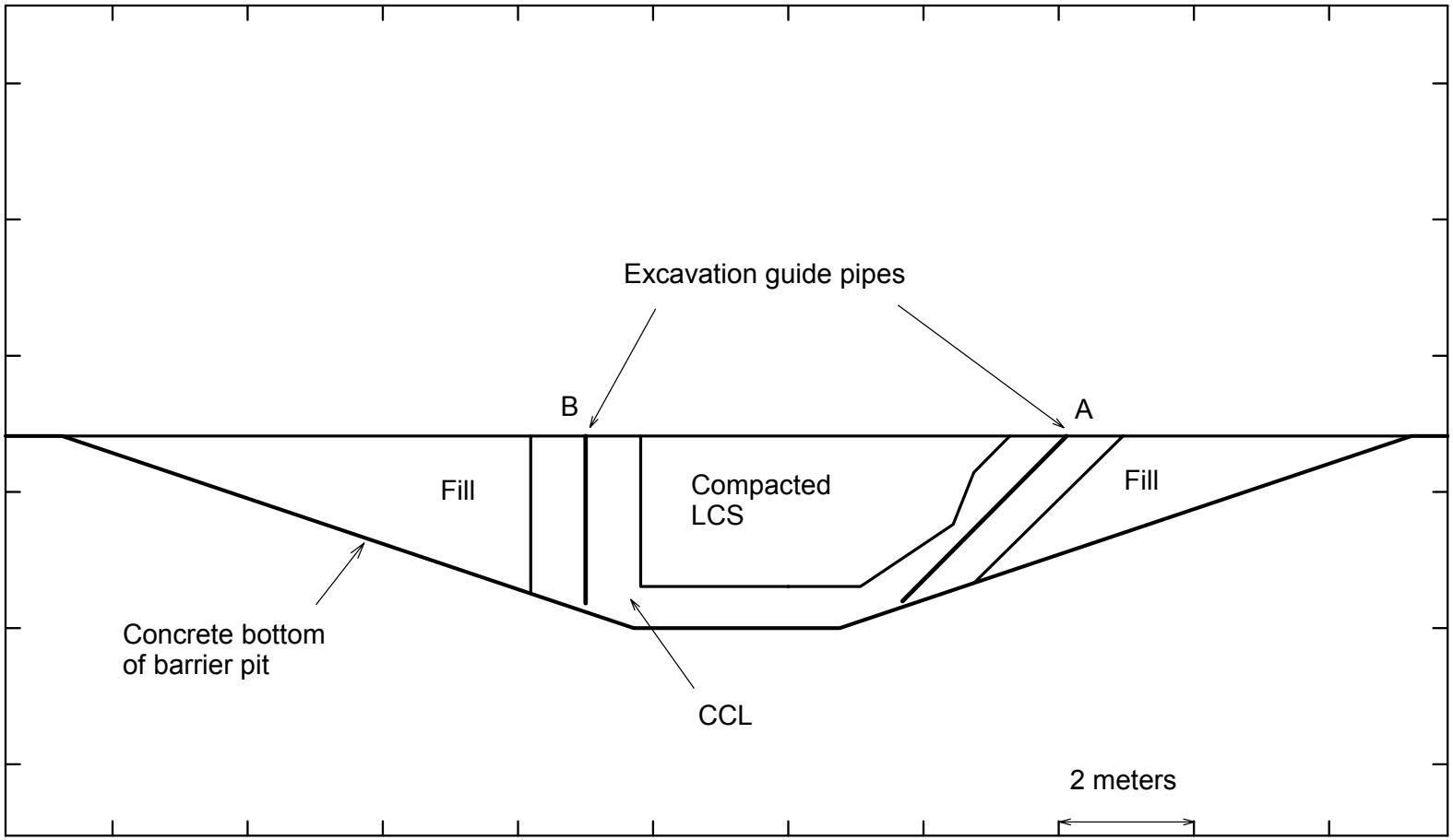

Figure 4-4. CCL and LCS geometry for W2

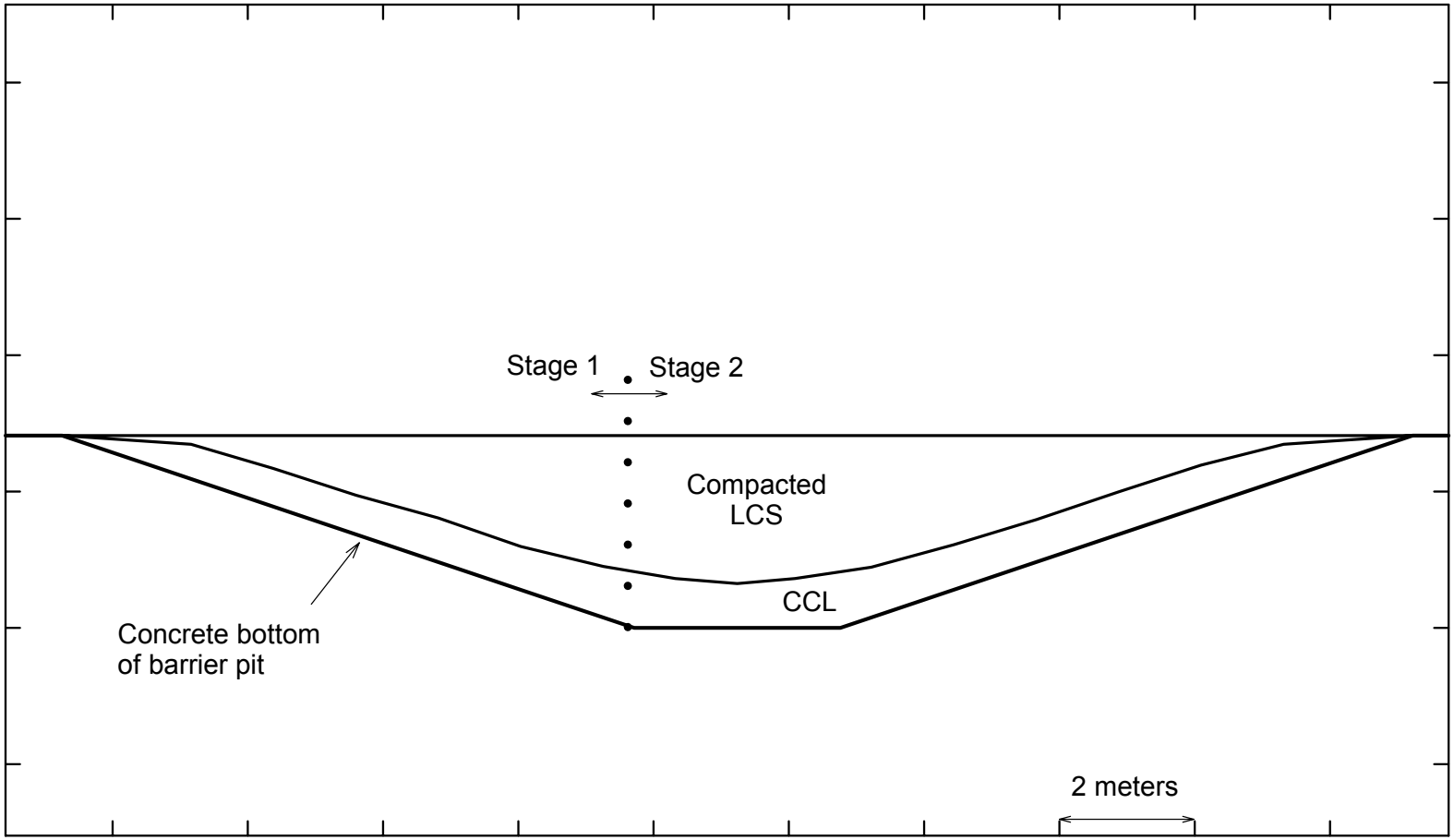

Figure 4-5. CCL and LCS geometry for W3

Figure 4-6. a) left – the finished CCL for W1, b) right – compacting the LCS for W1

4.3 Step 2: Compaction of Sand Simulating an Aquifer

4.3.1 Material Used

The sand that was used to simulate an aquifer in the pilot-scale experiments was Light Castle Sand (LCS). This is a uniform, poorly graded sand, classified as SP in the USCS. Because there is a negligible amount of fines in the sand, as shown by the grain size distribution in Figure 4-7, the hydraulic conductivity of the sand is high: approximately 0.04 to 0.05 cm/s as measured on a specimen at 70% relative density (Muck, 1999). A high hydraulic conductivity was desired for the sand so that head losses in the pilot-scale experiments occur predominantly in the soil-bentonite and may be ignored in the sand.

4.3.2 Procedure

After the CCLs for W1 and W3 were constructed, a lift of LCS was placed above the entire CCL and moistened to prevent drying of the CCL. This was not necessary for the CCL for W2, as it was constructed differently as described above in Section 4.2.2. In general, actions were taken during all three pilot-scale experiments to prevent drying of the CCL.

The LCS was compacted in horizontal lifts of loose thickness 15 cm using a Wacker BPU 2440A vibrating plate compactor as shown in Figure 4-6b. Five passes were used per lift. Density and water content measurements were made for W1 using the Troxler nuclear moisture/density gauge. The relative density of the LCS ranged from 80 to 90% and the water content ranged from 2 to 4%. The compacted lift thicknesses were from 7.5 to 10 cm. No density and water content measurements were made for the LCS for W2 and W3, as the compaction method was the same as for the LCS for W1. For each pilot-scale experiment, the LCS was compacted to the top of the barrier pit.

4.4 Step 3: Production of Soil-Bentonite

4.4.1 Materials Used

Three materials were used to make the soil-bentonite: a base soil, bentonite, and water from a well at the Price's Fork Research Station where the SBTF is located.

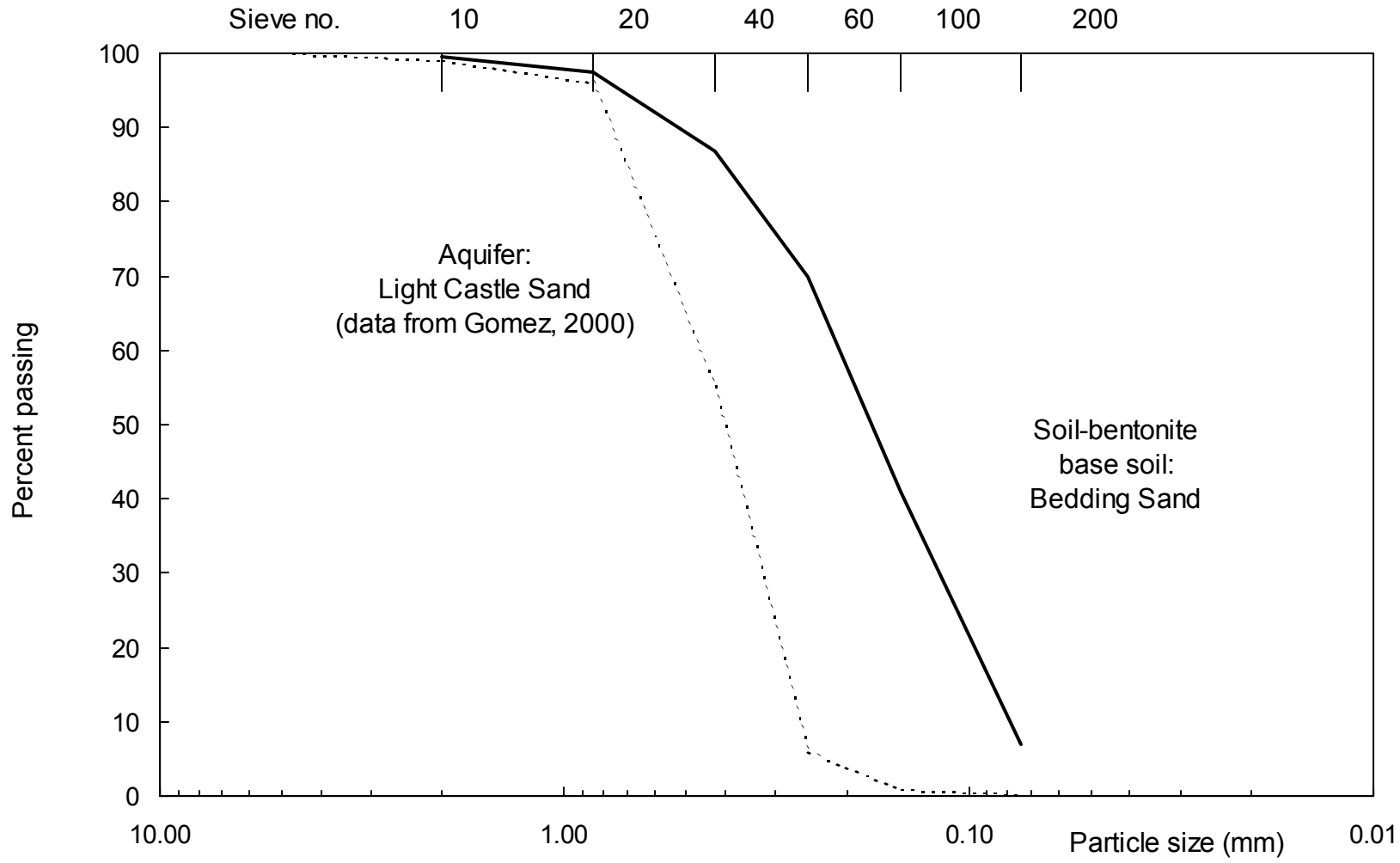

Figure 4-7. Aquifer and soil-bentonite base soil grain size distributions

The base soil was a silty sand called Bedding Sand from the West Sand and Gravel Company in Grottoes, Virginia. The Bedding Sand classifies as SP-SM in the USCS, with a silt content ranging from 6 to 9% as measured from seven samples. The specific gravity of the Bedding Sand was measured to be 2.68. The grain size distribution of the Bedding Sand was measured and can be seen in Figure 4-7. The hydraulic conductivity of the Bedding Sand was measured in a flexible-wall apparatus. The Bedding Sand was mixed at a high water content and then consolidated to an effective stress of 28 kPa. Water was permeated upward through the specimen to dislodge air in the specimen and setup. The hydraulic conductivity was measured to be 2.3×10^{-4} cm/s under a constant hydraulic gradient of 9.9. The Bedding Sand was chosen as the base soil for its availability, consistency over time in composition, and ability to hold the bentonite internally (i.e., bentonite does not wash from the Bedding Sand under the magnitude of hydraulic gradients used in the pilot-scale experiments).

The bentonite used in this research was 90-yield Hydrogel from Wyo-Ben, Inc. in Billings, Montana. The bentonite, which is a sodium montmorillonite, was in powder form. The specific gravity of the bentonite is 2.55.

The water used in all phases of the pilot-scale experiments was well groundwater from the site (Price's Fork water, PFW). A compositional chemical analysis was performed on the water using an ion chromatograph. The results are shown in Table 4-3. The predominant cations in the water are magnesium and calcium. Electroneutrality is likely met by accounting for the bicarbonate and carbonate concentrations in the water, which were not measured parameters in the chemical analysis.

Table 4-3. Chemical composition of Price's Fork water

Ion	Concentration (mg/L)
Anions:	
Fluoride	Not detected
Chloride	0.2
Nitrate	Not detected
Nitrite	Not detected
Phosphate	Not detected
Sulfate	10.5
Cations:	
Sodium	1.7
Ammonium	Not detected
Potassium	2.1
Magnesium	28.9
Calcium	45.2

4.4.2 Soil-Bentonite Mix Design

The soil-bentonite was a mixture of Bedding Sand, bentonite, and PFW. Three different bentonite contents, measured on a dry weight basis, were investigated: 1%, 2%, and 3%. To make each mixture, bentonite was mixed into PFW using a small high-speed mixer. The resulting bentonite-water slurry was then allowed to hydrate for at least 24 hours. After hydration of the bentonite, the slurry was added to the Bedding Sand and mixed by hand until uniform.

The next step was to determine the water content that produced a slump between 10 and 15 cm. The water content of the soil-bentonite mixtures was adjusted by adding PFW or drying, and slump measurements were made according to C143 in ASTM, 1992. For a slump between 10 and 15 cm, the target water contents for the 1%, 2%, and 3% bentonite mixtures were 30%, 35%, and 37%, respectively. When mixed at these water contents, the soil-bentonite mixtures had unit weights of approximately 18.2, 17.9, and 17.3 kN/m^3, respectively.

The hydraulic conductivity of the mixtures was measured using the API test method described in detail in Chapter 5. Figure 4-8 shows the results. An observation from the data is that the dependence of hydraulic conductivity on consolidation pressure is small for these particular mixtures.

Two one-dimensional consolidation tests (D2435 in ASTM, 1992) were performed on soil-bentonite from grab samples from W1. The stress/strain curves for the two consolidation tests are plotted in Figure 4-9. The specimens had similar compression and recompresion ratios, with averages of 0.0285 and 0.00225, respectively. For comparison, from a chart of compression ratio versus fines content from D'Appolonia (1980), a compression ratio of approximately 0.02 applies to fines contents less than 10%.

4.4.3 Procedure

The soil-bentonite for W1 and W2 had 1% bentonite, and the soil-bentonite for W3 had 3% bentonite. The approximate volumes of soil-bentonite produced for each wall are shown in Table 4-4.

The first step in mixing soil-bentonite at the SBTF was making a bentonite-water slurry. One of the slurry tanks was filled to an appropriate volume with PFW. Then the hoses of a 15-cm or 10-cm (hose diameter) trash pump were connected to the outlet at the bottom of the tank and the opening at the top of the tank. A Rantec© mixing eductor was connected to the end of the discharge hose connected to the top of the tank. A schematic of the eductor is shown in Figure 4-10. The eductor sucks bentonite powder from its bag and wets it in the high-velocity jet of discharge water. After all of the bentonite was added using the eductor, the eductor was removed and the slurry was circulated from the bottom outlet of the tank, to the trash pump, and back into the top opening of the tank, using the 15 or 10-cm diameter hoses. After circulating for roughly an hour, the slurry was allowed to hydrate overnight.

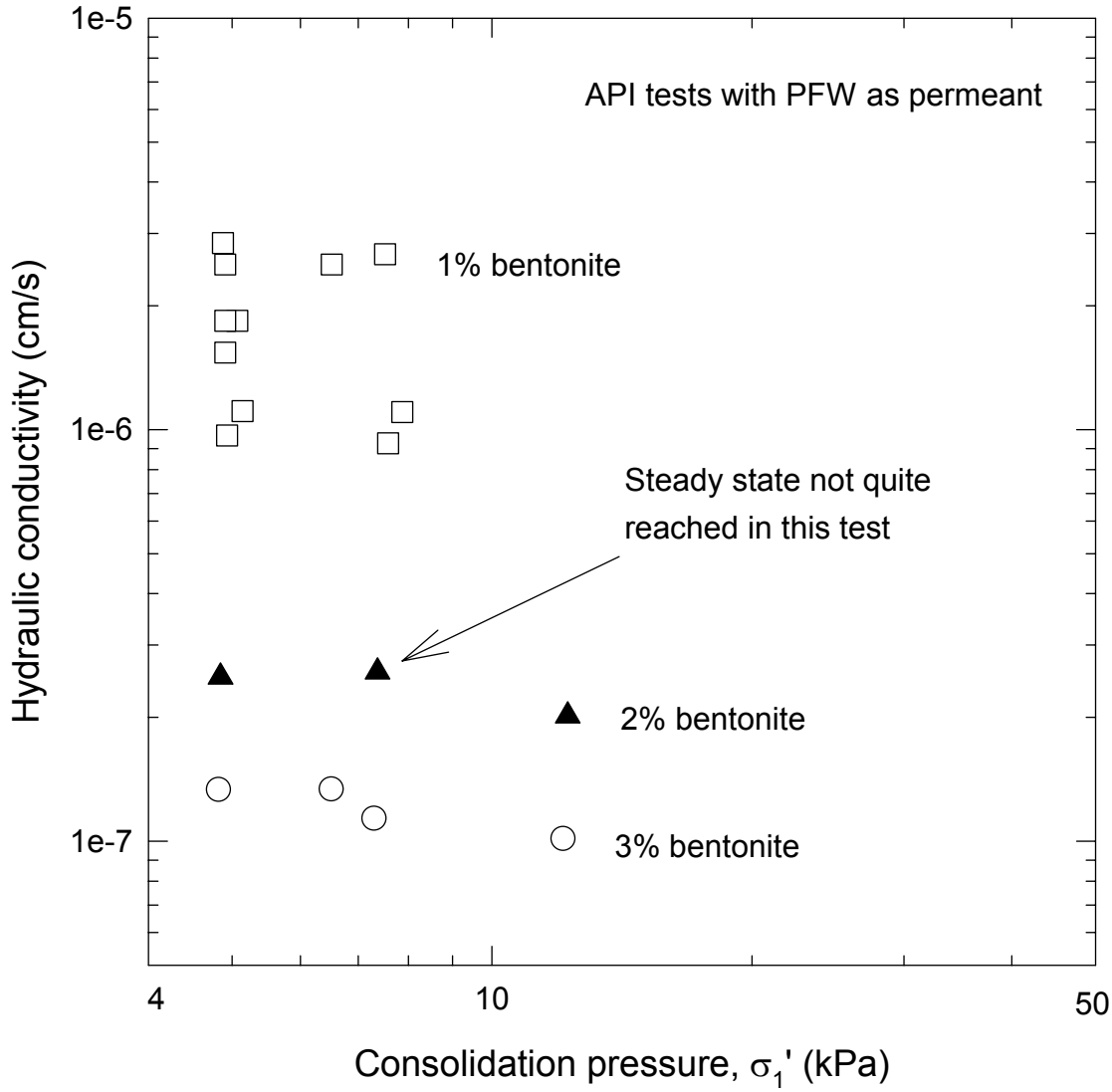

Figure 4-8. Soil-bentonite mix design: hydraulic conductivities of Bedding Sand mixed with three different bentonite contents

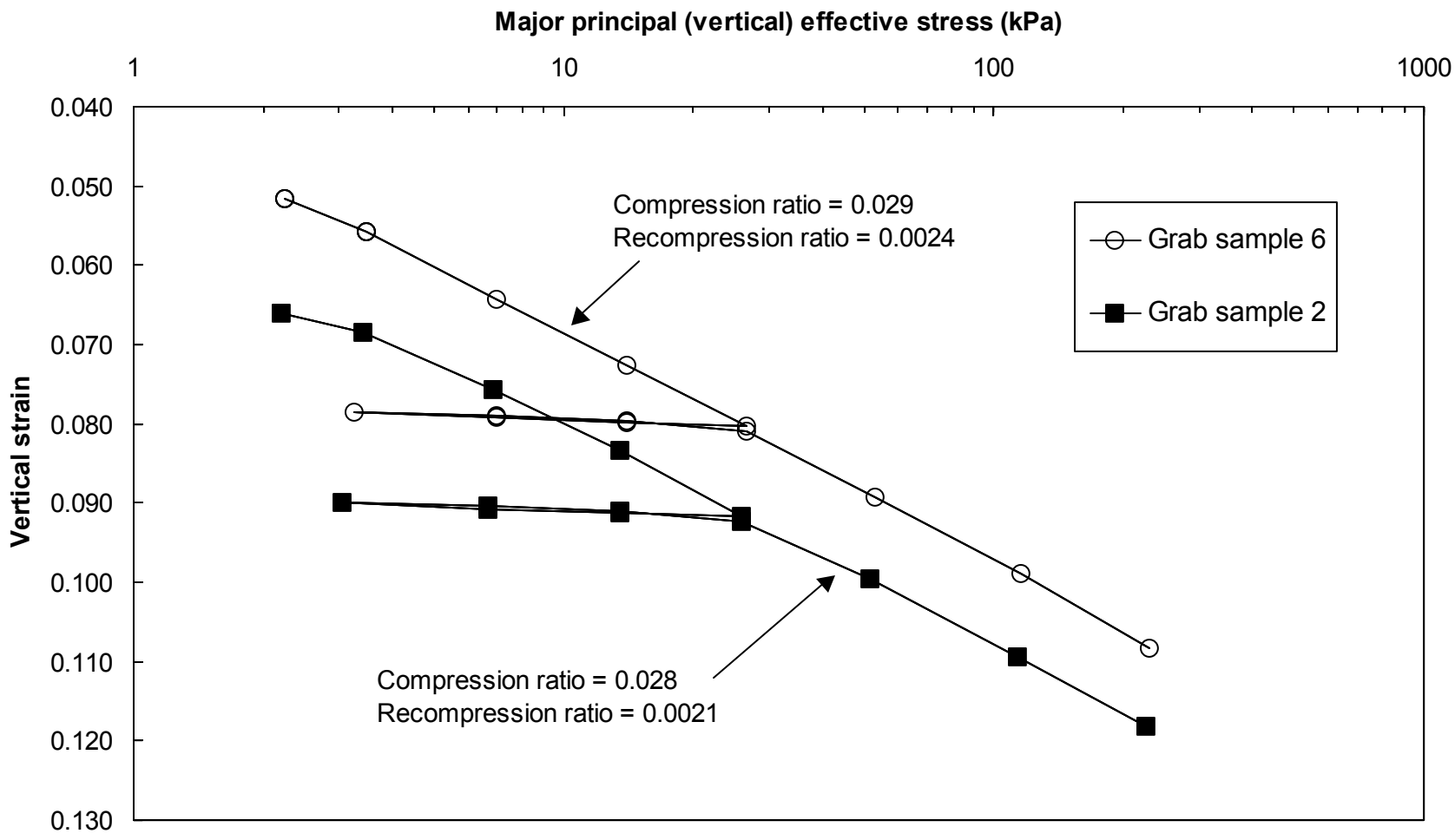

Figure 4-9. Consolidation tests on soil-bentonite grab samples from W1

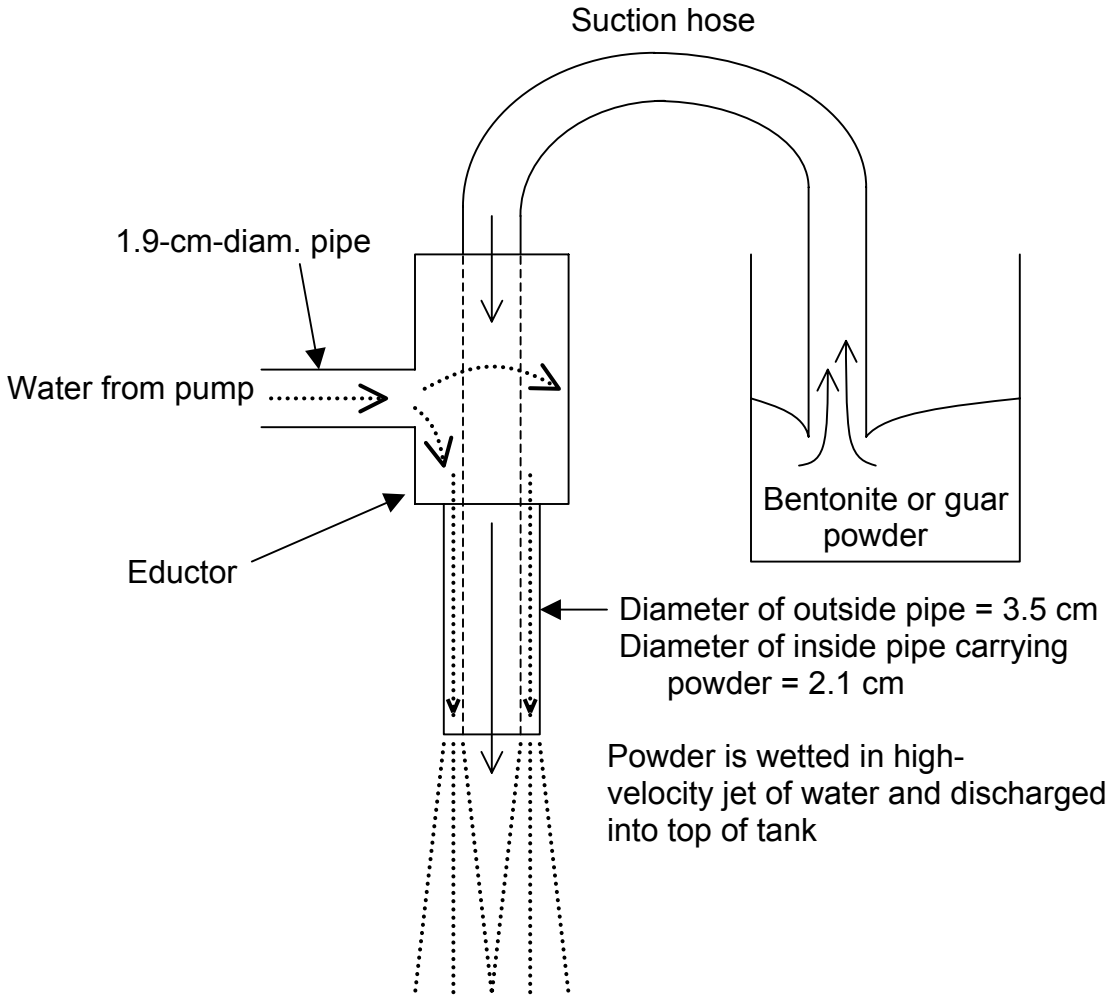

Figure 4-10. Schematic diagram of Rantec© plastic mixing eductor

Table 4-4. Characteristics of the soil-bentonite

	Wall number: 1	2	3
Bentonite content (%)	1	1	3
Soil-bentonite volume produced (m^3)	13.9	13.9	25.8(1)
Form of bentonite added	All in slurry form	All in slurry form	Half in slurry form, half in dry form
Average water content (%)	31.9	32.0	36.0

Notes:
(1) The ratio of soil-bentonite made to trench volume was 13.9/9.9 = 1.4 for W1 and 25.8/19.8 = 1.3 for W3. It was found through experience from W1 to W3 that less soil-bentonite needed to be made per volume of trench to accomodate soil-bentonite losses during mixing, placing, etc.

The bentonite content of the slurry (weight of bentonite divided by weight of slurry) for W1 and W2 was approximately 5%, and for W3 was approximately 7%. The bentonite content of 7% was about the maximum that could be produced with the slurry mixing equipment. There was a limit as to how much slurry could be made for each soil-bentonite mixture, because it was desired to have an initial soil-bentonite water content less than the target value required for the proper slump. If too much water was added in slurry form, it would be very difficult to dry the soil-bentonite to the proper water content in a timely manner. For W1 and W2, all of the bentonite could be added to the Bedding Sand in slurry form while satisfying this water content requirement. For W3, however, approximately one-half of the bentonite could be added in slurry form, and the rest had to be added in dry form as described below.

Once the bentonite-water slurry was ready, the Bedding Sand was spread out on the concrete apron at the facility. The slurry was then added to the Bedding Sand. The slurry was mixed into the Bedding Sand using a Bobcat loader. The loader on the Bobcat was used to move the materials around (see Figure 4-11) while PFW was added to approach the target water content. As soon as the soil-bentonite became wet enough, it was spread thin (30 to 60-cm-thick) and driven through with the Bobcat. By pushing the soil-bentonite around during this driving, especially when turning, the Bedding Sand, bentonite, and PFW became well mixed. Mixing was performed until the soil-bentonite appeared uniform by visual inspection.

The process of adding the slurry and mixing the soil-bentonite took about 10 hours for W1 and W2. For W3, due to the larger total volume of soil-bentonite produced and other considerations[1], three batches of slurry were made and added to three separate piles of Bedding Sand. The slurry adding and soil-bentonite mixing time period for each batch was approximately 10 hours. In addition, for each batch for W3, roughly one-half of the bentonite was added in dry form by spreading out the Bedding Sand after addition of the slurry and sprinkling the dry bentonite over the Bedding Sand.

[1] It worked out best to receive three truck-loads of Bedding Sand of known, measured weight, and therefore add three separate known amounts of bentonite to each load of Bedding Sand to achieve the right bentonite content.

Figure 4-11. Soil-bentonite mixing

Shortly before backfilling, PFW was mixed into the soil-bentonite using the Bobcat until the slump of the soil-bentonite was measured to be within 10 to 15 cm. The average water contents of the soil-bentonite mixtures, measured from the grab samples described in Section 4.7, are shown in Table 4-4.

4.5 Step 4: Production of Support Slurry

4.5.1 Materials Used

Two different types of slurry were used for trench support during excavation: bentonite-water slurry and a biodegradable slurry (bioslurry). The trenches for W1 and W2 were excavated under bioslurry and the trench for W3 was excavated under bentonite-water slurry. Both slurries were made using PFW.

The bentonite-water support slurry was made with the same type of bentonite used to make the soil-bentonite backfill: 90-yield Hydrogel.

Before describing the bioslurry, the reason for its use will be discussed. The soil-bentonite in W1 and W2 was made with 1% bentonite, giving it a hydraulic conductivity on the order of 10^{-6} cm/s. If bentonite-water slurry had been used for trench excavation, then there could have been intact filter cakes on the trench walls with hydraulic conductivity on the order of 10^{-8} cm/s. Even though the thickness of the filter cakes may be small, their influence on the hydraulic performance of the cutoff wall may be significant. For example, for a ratio of soil-bentonite k to filter cake k of 100, and a reasonable ratio of two times filter cake thickness to trench thickness of 0.01, the equivalent hydraulic conductivity of the cutoff wall system is one-half that of the soil-bentonite. The equivalent hydraulic conductivity is the hydraulic conductivity of the filter cake/soil-bentonite/filter cake system in series, and is what controls the flow rate through the cutoff wall. This magnitude of effect on the performance of the cutoff wall is undesirable for testing the performance of soil-bentonite mixtures. If there were defects in the wall, then the effect would be even larger. In addition, it is uncertain whether or not the filter cakes remain intact during the backfilling operation. It is not hard to imagine a variability in integrity and influence of the filter cake from place to place in the wall, making interpretation of hydraulic conductivity tests difficult. To avoid these issues altogether, a slurry was desired that would stabilize the trench during excavation but leave no permanent low-permeability filter cake.

The bioslurry used in this research was made with Rantec G150 biopolymer from the Rantec Corporation in Ranchester, Wyoming. The biopolymer in G150, which was in powder form, is a carbohydrate called Galactomannan, or guar gum. The guar gum molecule is a linear, long-chain polysaccharide that 1) attracts and weakly captures water and 2) physically tangles with other guar molecules to produce a viscous solution.

The sealing system formed by the guar is comprised of a cake at the trench wall and a slurry penetration distance into the sand. The cake portion is formed from 1) insoluble components of the slurry, such as insoluble proteins and polysaccharides, and 2) adsorption of guar to soil particles and subsequent bridging of guar between the particles. The high viscosity of the slurry controls the rate at which the slurry leaks from the trench under the head above that lost through the cake.

The Rantec Catalog (1997) states that "water found on location often contains micro-organisms that will breakdown the G150 after 24 to 48 hours exposure." The first step is for enzymes to reduce the guar molecules to large sugar molecules, or oligosaccharides. The second step is reduction of the oligosaccharides to carbon dioxide and water through bio-reduction.

Lab tests were performed on the bioslurry prior to its use in the pilot-scale experiments. The bioslurry for the lab tests was mixed at a G150 to water ratio of 0.6% by weight. The measured unit weight of the freshly mixed bioslurry was approximately equal to the unit weight of water. The filtrate loss of the fresh bioslurry, measured in an API Filter Press test (see Section 3 in American Petroleum Institute, 1985 for a description of this test), was 15 ml. For comparison, a common requirement for bentonite-water slurries is a filtrate loss less than 20 to 25 ml. After the filter press test, the cake thickness on the filter paper was measured to be approximately 0.25 to 0.5 mm.

Next, lab tests were performed to verify that the sealing system formed by the bioslurry would degrade. The first level of investigation was to mix up a batch of bioslurry using PFW and observe the degradation of the bioslurry with time. Degradation was assessed in a qualitative manner by visual inspection of the mix and by stirring the mix with a spatula and observing its consistency. From this simple experiment, it appeared that degradation occurred in approximately one week.

In the second experiment, LCS was placed in a compaction mold permeameter and PFW was permeated downward through the LCS under a hydraulic gradient of 10. After measuring the flow rate under this condition (the permittivity of the system with the LCS was approximately 5.5/hr), a bioslurry cake was formed on top of the LCS under a formation head of 70 cm. The same hydraulic gradient was applied to the system and the flow rate was measured with time. Initially, the flow rate was negligible. As the cake degraded, the flow rate increased[2]. At steady state, the flow rate through the system with the cake (now significantly degraded) was 65% of the flow rate through the system prior to formation of the cake. The steady state permittivity of the degraded cake was approximately 10.4/hr. Even in its degraded state, the cake had an effect on the permittivity of the system, lowering it from 5.5/hr before formation of the cake to approximately 3.6/hr with the degraded cake. However, when compared to a 30-cm-wide soil-bentonite cutoff wall, the permittivity of 10.4/hr for one degraded cake is insignificant. The permittivity of a 30-cm-wide cutoff wall with $k = 2 \times 10^{-6}$ cm/s is 2.4 ×

[2] The bottom of the compaction mold permeameter was closed most of the time so that there was no flow, in order to minimize maintenance of the experiment. The bottom was opened for flow measurements at various times. As a result, the cake essentially degraded under zero hydraulic gradient.

10^{-4}/hr, which is much smaller. In the lab test, it took approximately one day for the cake to degrade to a permittivity of 10.4/hr. As a result, the permittivity of a 30-cm-wide cutoff wall with two degrading cakes should approach the permittivity of a 30-cm-wide cutoff wall alone in approximately one day.

Based on the above tests and considerations, it was concluded that the influence of the bioslurry sealing system would be insignificant after a time period of one week in the pilot-scale experiments. This is a conservative estimation of the degradation time; the sealing system may be insignificant after a much shorter time period.

4.5.2 Procedure

Both types of slurry were mixed in the same way as described in Section 4.4.3 for the bentonite-water slurry for soil-bentonite production. An extra step was required for bioslurry production. After a batch of bioslurry was made and emptied from a slurry tank, there remained a residual amount of guar and an accompanying micro-organism population. When a new batch of bioslurry was made in the tank, the existence of the micro-organism population greatly accelerated degradation of the bioslurry, to the point where it did not last long enough to excavate the trench. To avoid rapid bioslurry degradation, the slurry tanks and slurry mixing equipment were disinfected prior to production of each new batch of bioslurry to kill the existing micro-organisms. The disinfecting process involved circulating and holding a solution of Cl_2 at 100 parts per million and a pH around 5.5 to 6.0 in the tanks and mixing equipment for a time period of 24 hours. The disinfecting solution was made by completely filling the tanks with water and adding bleach and muriatic acid. The tanks were rinsed after disinfection.

For W1 and W2, the bioslurry was mixed the afternoon before excavation in both tanks, yielding a slurry volume of 22.7 m^3. The bioslurry was mixed at a guar to water ratio of 0.6% by weight. The Marsh funnel viscosity[3] (see Section 2 in American Petroleum Institue, 1985 for a description of this test) of the bioslurry prior to excavation was in the range of 47 to 55 seconds, and its fresh unit weight was approximately the same as the unit weight of water. The viscosity and unit weight of the bioslurry in the trench were not measured.

For W3, the first batch of bentonite-water slurry was mixed the day before excavation in both tanks. More slurry was made later for W3, as described in Section 4.6.1.3 on excavation below. The total slurry volume produced was 44.5 m^3. The slurry was mixed at a bentonite to slurry ratio of 6% by weight. The Marsh funnel viscosity of the slurry prior to excavation was 43 seconds, and its fresh unit weight was 10.2 kN/m^3. During excavation, the viscosity of the slurry in the trench was measured to be 49 seconds and the unit weight was measured to be 10.5 kN/m^3.

[3] For reference, Schneider (1994) suggests a miminum required fresh bentonite-water slurry Marsh funnel viscosity of 40 seconds for trench excavation.

Table 4-5 summarizes the characteristics of the support slurry for the three walls.

Table 4-5. Characteristics of the support slurry

	Wall number: 1	2	3
Slurry type	bioslurry	bioslurry	bentonite-water
Guar or bentonite content on weight basis	0.6% guar to water	0.6% guar to water	6% bentonite to slurry
Slurry volume made (m^3)	22.7	22.7	44.5

4.6 Step 5: Trench Excavation

4.6.1 Procedure

Each trench was excavated with a Bobcat track-mounted 331 excavator. This excavator has a maximum digging depth of 3.08 m. The trench for W1 was excavated with a 30.5 cm bucket and the trenches for W2 and W3 were excavated with a 61.0 cm bucket. The teeth were removed from the buckets so that the trenches could be excavated all the way to the concrete bottom of the barrier pit.

Before excavating the trenches, the water level in the compacted LCS was lowered to approximately the bottom of the barrier pit to increase the stability of the excavations. The LCS was moistened at the surface to give it an apparent cohesion, which helped prevent sloughing of the LCS above the slurry level during excavation.

The outside to outside width of the excavator tracks was slightly less than 1.8 m, which is the inside width of the barrier pit. The general procedure was to excavate from one end of the barrier pit to the other, with the excavator driving backward, ahead of the advancing front of the excavation, as in typical slurry cutoff wall practice. A photograph of the excavation of the trench for W1 is shown in Figure 4-12. Deviations from this general procedure are discussed in Sections 4.6.1.1, 4.6.1.2, and 4.6.1.3, when specific details of each excavation are described. As each trench was excavated, support slurry was drained from the slurry tanks into the trench. The slurry level in the trench was kept as close to the top of the trench as possible. As compacted LCS and Washout material were excavated from the trench, they were placed beside the barrier pit and periodically removed with the Bobcat loader. Each trench was excavated all the way through the CCL to the concrete bottom of the barrier pit.

Figure 4-12. Excavating the trench for W1 using bioslurry for excavation support

Throughout the excavation process, the bottom of the trench was probed for loose material. The probing device consisted of a 1.3-cm-diameter steel rod with a pointed bottom and a 3.8-cm-diameter PVC pipe with a capped bottom. The steel rod and PVC pipe were loosely attached using two hose clamps so that they could slide relative to one another but remain in contact. To measure the height of loose material at the bottom of the trench, the steel rod with the pointed bottom was pushed through the loose material to the concrete bottom of the barrier pit and the PVC pipe with the capped bottom was rested on top of the loose material. The difference in elevation between the steel rod and PVC pipe was taken as the height of loose material.

4.6.1.1 Trench for W1

After the trench was excavated from one end of the barrier pit to the other, an unacceptable amount of loose material, on the order of 10 – 15 cm in height, was measured at some places along the bottom of the trench. To remove this material, 3.8-cm-tall by 14-cm-wide wood boards were placed across the width of the barrier pit, resting on the concrete walls of the pit and not on the compacted LCS, which was lowered slightly in elevation so that the top of the LCS was just below the top of the barrier pit. The excavator was then able to drive back over the open trench, with its weight transferred to the concrete walls of the pit and not the LCS trench walls. The excavator was slowly driven from one end of the barrier pit to the other, with its bucket continuously scraping along the bottom of the trench. At the end of the pit, the bucket was removed from the trench with the collected loose material. The loose material was predominantly LCS, with small amounts of Washout material. The loose material is believed to have come from 1) spilling from the bucket while raising the bucket upward through the slurry and 2) settlement of material through the bioslurry. After cleaning the bottom as described above, the maximum height of loose material was measured to be approximately 5 cm.

4.6.1.2 Trench for W2

Prior to excavation of the trench for W2, the guide pipes shown in Figure 4-4 were installed in the CCL. A hand auger was used to auger two holes in the CCL bulkheads: a vertical hole at Location *B* in Figure 4-4 and a hole at a slope of 1:1 at Location *A*. Then 3.2-cm-diameter steel pipes were inserted into the holes extending from the top to the bottom of the barrier pit. Each length of pipe had a 4.8-cm-diameter coupling roughly one-third of the way down the pipe from the top of the pit. These couplings proved troublesome during excavation, as described below.

Excavation of the trench started at the top of Pipe *B*. With the excavator located to the right of *B* in Figure 4-4, the vertical CCL bulkhead was excavated downward with the bucket digging along Pipe *B* as an excavation guide. After excavating the vertical CCL approximately half-way down, the excavator was turned around and digging began at the top of Pipe *A*. The trench was excavated to a depth in the 1:1 bulkhead controlled by the location of Pipe *A*. Once the 1:1 bulkhead and LCS above it were excavated, excavation continued in the same direction along the deep part of the barrier pit to the bottom of Pipe *B*. To get over the open trench already excavated at the top of Pipe *B*, the wood boards were placed across

the pit as described above for the trench for W1. The excavation progressing from the right finally met the previous excavation started at the top of Pipe *B*, and the entire trench was then open. This order of excavation was done to avoid upward excavation of the CCL along the top of Pipe *B*, which, due to lack of confinement at the top of the CCL, could have lead to the removal of chunks of the CCL wider than 61 cm.

The excavation lines in the CCL bulkheads were not exactly along the original locations of the guide pipes due to movement of the guide pipes during excavation. While digging along the pipes, the excavator bucket caught the couplings described above, and caused Pipe *A* to move downward and eventually break[4], and Pipe *B* to move upward and eventually out of the pit. The effect this had on the geometry of the soil-bentonite cutoff wall key is described in Section 4.9 (Destructive Evaluation).

After the excavation sequence described above was finished, a procedure similar to that for the trench for W1 was used to remove loose material from the bottom of the trench. The bottom of the trench was only scraped clean along the concrete (from the bottom of the original location of Pipe *A* to the bottom of the original location of Pipe *B*), however, because there was nothing to scrape against in the 1:1 bulkhead due to the removal of Pipe *A*. Measurement of the height of loose material on the 1:1 slope was difficult; it was estimated that there was less than 2.5 to 5 cm of loose material on this slope. Along the concrete bottom of the trench, the maximum height of loose material after cleaning was approximately 5 cm.

4.6.1.3 Trench for W3

After the trench for W3 had been excavated to roughly the dotted line shown in Figure 4-5, the outlet of the larger slurry tank broke and the bentonite-water slurry needed to finish the excavation was lost. A time period of 13 days passed before new bentonite was obtained and the bentonite-water slurry needed to finish the excavation was ready. During this time period, the open trench excavated in Stage 1 was covered and the water level in the compacted LCS was kept at the bottom of the barrier pit. Before starting Stage 2 of the excavation, the bottom of the trench excavated in Stage 1 was probed and a negligible amount of loose material was measured. After the entire excavation was finished, a negligible amount of loose material was measured on both slopes of the trench bottom and a maximum of 5 cm was measured in the deepest, central part of the trench. This was considered acceptable, so no further trench bottom cleaning was performed.

Due to the longer time that Stage 1 was open compared to Stage 2, the filter cakes were expected to be thicker in the Stage 1 area. In Nash's (1974) expression for filter cake thickness, the thickness is proportional to the square root of time. Considering Stage 1 to be open for roughly 13 days (312 hours) and Stage 2 to be open for roughly 8 hours, the thickness of the cake in Stage 1 would be $(312/8)^{0.5} = 6$ times the thickness of the cake in Stage 2.

[4] Pipe *A* was removed from the trench prior to backfilling.

4.7 Step 6: Trench Backfilling

4.7.1 Procedure

Soil-bentonite backfilling started at the end of one slope of the trench. Backfilling started at the end of the 1:1 slope for the trench for W2. The Bobcat loader was used to place the soil-bentonite in the trench. The loader was able to place soil-bentonite in the trench with the front wheels of the Bobcat staying on the concrete wall of the barrier pit and not applying weight to the LCS trench walls. The initial soil-bentonite was placed at the end of the slope and moved down the slope through the support slurry. As soon as the soil-bentonite accumulated to the top of the trench, new loads of soil-bentonite were placed on top of existing soil-benonite in the trench.

For the trench for W1, of nominal width 30.5 cm, a piece of plywood was rested against the top of the trench wall opposite the loader to help control placement of the soil-bentonite into the narrow trench. This was not required for the wider trenches for W2 and W3. For each of the three trenches, it was necessary to work the soil-bentonite at the surface with a shovel (by moving the shovel up and down in the soil-bentonite) to help it flow into the trench. It was significantly more difficult getting the soil-bentonite to flow into the 30.5-cm trench compared to the 61.0-cm trenches.

The slurry level in the trench was controlled with a 5-cm trash pump. The suction hose for the pump was placed in the trench at the end opposite of where backfilling started, and the slurry was periodically pumped from the trench into the evaporation pond as the soil-bentonite was placed in the trench. The slurry level was kept close to the top of the trench during backfilling; there were no stability problems during this operation.

The slope of the soil-bentonite in the trench was measured for W2 and W3 by probing the soil-bentonite surface along the length of the trench. For W2, the slope was 3H:1V on average. For W3, the slope was 2.8H:1V on average.

Six grab samples of soil-bentonite were taken during the course of backfilling each of the three trenches. The sampling was spaced out from the beginning to the end of backfilling. For W3, two samples were taken from the first batch of soil-bentonite placed in the trench, three samples from the second batch, and one sample from the third batch.

Finally, after each trench was filled to the top with soil-bentonite and all of the support slurry was displaced into the evaporation pond, an additional amount of soil-bentonite roughly 20-cm-high was placed above the trench. This additional soil-bentonite was placed to accomodate future consolidation settlement of the soil-bentonite in the trench.

4.8 Step 7: Final Treatment of Cutoff Wall

An effort was made to prevent desiccation of the top of the soil-bentonite cutoff walls. Approximately 20 – 30 cm of moist sand was placed above the barrier pit and the sand was covered with plastic sheeting.

After construction of the cutoff walls, the water levels in the compacted LCS were lowered to approximately the bottom of the barrier pit. This was done to consolidate the walls under the highest possible effective stress governed by the water levels in the LCS. The predicted vertical effective stresses in the walls, evaluated using arching theory with estimated values of effective friction angle (ϕ') = 32°, horizontal earth pressure coefficient (K_h) = 0.42, cohesion = 0, and unit weight (γ) = 18.1 kN/m^3, are shown in Figure 4-13 for the case where the water levels are at the bottom of the barrier pit[5,6]. During in situ hydraulic conductivity testing, the water levels in the LCS are higher than the bottom of the pit. Under these conditions, the effective stresses in the walls due to the higher water levels are less than the consolidation pressures shown in Figure 4-13.

Time was allowed for the walls to consolidate under the conditions described above[7]. The consolidation times for W1, W2, and W3 were approximately 2 weeks, 2 weeks, and 4 weeks, respectively.

4.9 Step 9: Destructive Evaluation

A series of in situ tests were performed in each cutoff wall as described in Chapter 5. After these tests were completed, each cutoff wall was evaluated as it was excavated. In Section 4.9.1, the general destructive evaluation procedure is briefly described. Details are given for each pilot-scale cutoff wall in Sections 4.9.1.1, 4.9.1.2, and 4.9.1.3.

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[5] Baxter (2000) found through a literature review of soil-bentonite properties that "all of the reported values of effective friction angle for soil-bentonite mixtures are between 31 and 33 degrees" and the effective cohesion of soil-bentonite is zero. The value of K_h was estimated from the Brooker and Ireland (1965) relationship for at-rest earth pressure $K_0 = 0.95 - \sin \phi'$, which was used by Evans et al. (1995) for estimating stresses in soil-bentonite cutoff walls from dilatometer data.

[6] In arching theory, the pore pressure above the water table is assumed to be zero. In reality, there may be negative pore pressures above the water table, which would tend to increase the effective stress in the cutoff wall.

[7] There is no established procedure for predicting the time-rate of consolidation of soil-bentonite cutoff walls. For the pilot-scale experiments, the time to allow for consolidation was estimated from simple analyses using Terzaghi's consolidation theory.

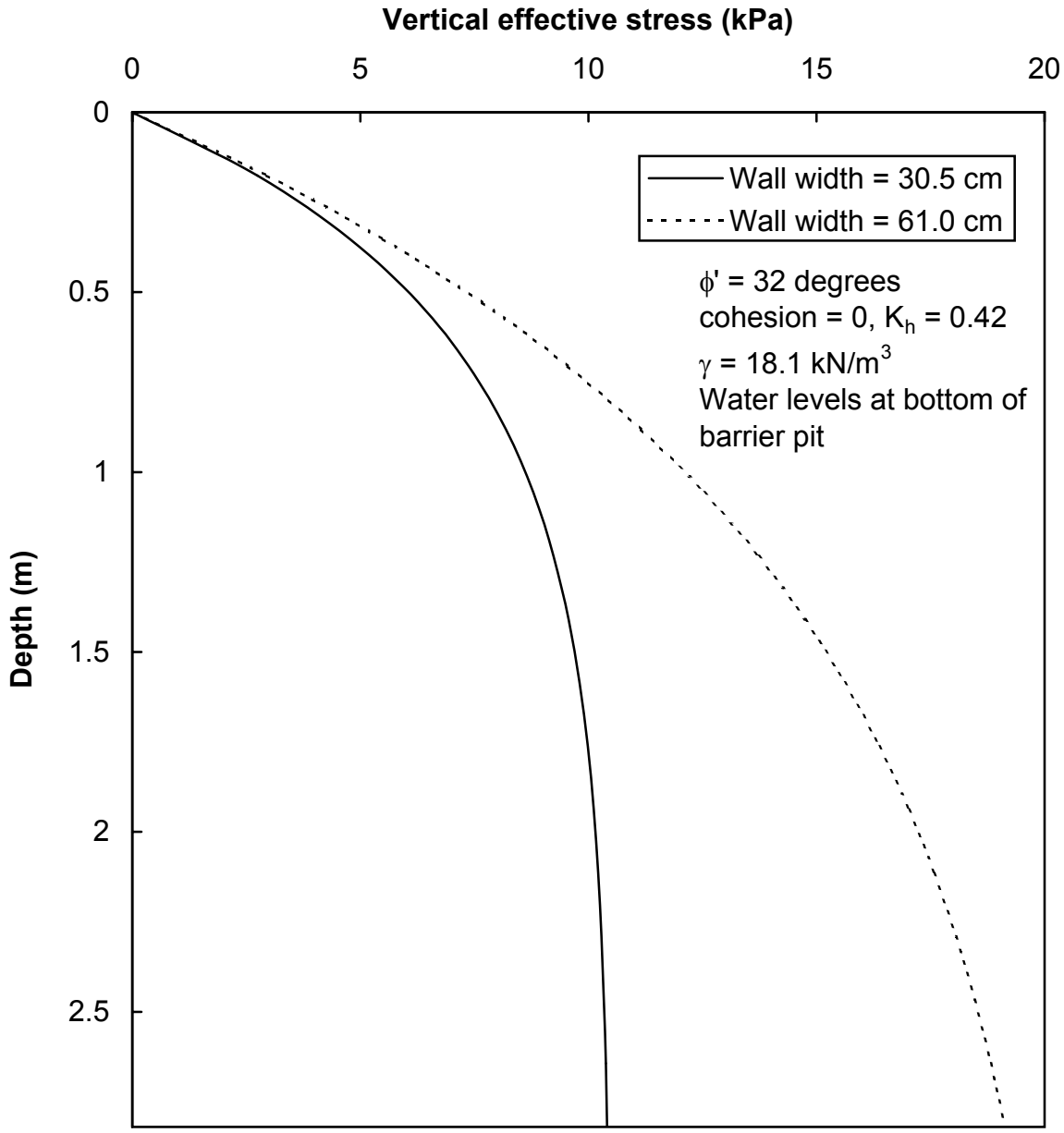

Figure 4-13. Predicted stresses in pilot-scale cutoff walls from arching theory

4.9.1 General Procedure

At the end of each pilot-scale experiment, the barrier pit was completely excavated using the Bobcat loader. Prior to this excavation, the water levels in the LCS were lowered to close to the top of the CCL. Destructive evaluation of the cutoff walls included geometry measurements, observations, and sampling, followed by lab tests. The measuring, observing, and sampling time period was approximately one week per wall. In the following sections, geometry measurements, observations, and sampling locations and test results for each cutoff wall are described.

4.9.1.1 Destructive Evaluation of W1

The width of W1 was measured to range from 28.0 to 33.0 cm.

Roughly the top one-half meter of the soil-bentonite wall appeared desiccated. Cracks were not observed, but the soil-bentonite felt stiffer than it did at greater depths.

There was a 2.5-cm-thick layer of black soil-bentonite at the cutoff wall/LCS interface. The black color is assumed to be from degradation of bioslurry. No cakes were observed on the trench walls.

There was a defect consisting of LCS at the bottom of the wall. The height of the defect is shown in Figure 4-14. The sand at the soil-bentonite/CCL interface, shown with top dimension *t* in Figure 4-14, was very permeable. During excavation of the barrier pit, water seeped through this sand very quickly into the open excavation. The water came from the small height of water sitting above the CCL. This seepage indicated a highly permeable, connected flow path through the key, beneath the soil-bentonite, where the distance *t* in Figure 4-14 was greater than zero on each side of the wall. In addition, it was clear that water could easily seep from one side of the wall to the other through the sand whose height was greater than that of the CCL. The influence of this defect on the flow rate through the wall was extremely significant. The hydraulic conductivity of the defect was so high that it was impossible to sustain a working hydraulic gradient across the wall.

The following mechanism for creation of the defect was hypothesized:

> The shear stresses along the trench walls were high enough to keep the soil-bentonite from flowing all the way to the bottom of the narrow, 30-cm-wide trench. When the bioslurry at the bottom of the trench degraded, the sand trench-walls caved in and formed the defect.

Based on this hypothesis, W2 and W3 were made 61-cm-wide.

Figure 4-14 also shows the locations of 11 water content samples, including the water content values, and 2 undisturbed samples. The hydraulic conductivity tests performed on the undisturbed samples are described in Chapter 5.

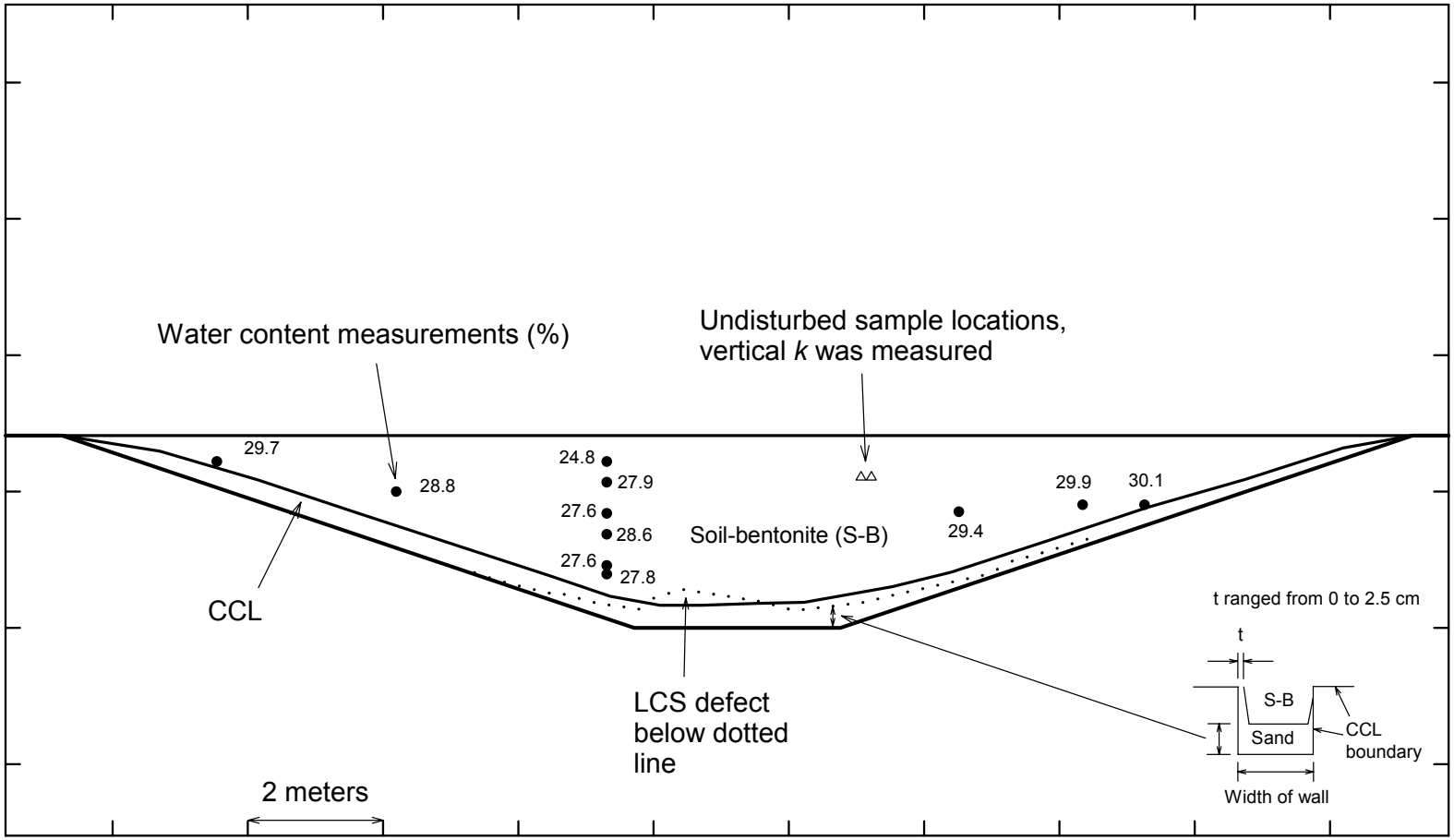

Figure 4-14. Destructive evaluation of W1

4.9.1.2 Destructive Evaluation of W2

The width of W2 was measured to range from 58.4 to 66.0 cm.

As with W1, roughly the top one-half meter of the soil-bentonite wall appeared desiccated. Again, cracks were not observed, but the soil-bentonite felt stiffer than it did at greater depths. In addition, notice was made of the unsaturated appearance of the soil-bentonite in the desiccated zone, compared to the saturated appearance below this zone.

The black soil-bentonite was again observed at the cutoff wall/LCS interface, and also in the key. The black soil-bentonite in the key suggests that bioslurry got mixed into this soil-bentonite. No cakes were observed on the trench walls.

The geometry of the soil-bentonite cutoff wall key was measured and is shown in Figure 4-15. The key was close to vertical in the vertical bulkhead. There was more deviation from the 1:1 slope in the 1:1 bulkhead, but the soil-bentonite was still keyed into the CCL sufficiently. The deviation was due to the damage to the excavation guide pipes discussed in Section 4.6.1.2.

Defects were observed at the bottom of the wall and in the part of the wall keyed into the vertical CCL bulkhead. As with W1, the defects consisted of LCS. The defects in the bottom key and the vertical key are described below.

Bottom key: The height of sand was measured and is shown in Figure 4-15. To the left of Line *A* in Figure 4-15a, sand was found at the soil-bentonite/CCL interface above the height of sand shown by the dotted line in the figure. The thickness of this interface sand (represented by the dimension *t* in Figure 4-14 for W1) trended from thin (trace amounts) at Line *A* to thick (1.3 cm) next to the vertical key. For all thicknesses of this interface sand, it always made a permeable connection between the defect sand at the bottom of the wall and the compacted LCS. This was evident from the water quickly seeping into the open excavation through the defect sand from the small height of water above the CCL.

Vertical key: From the bottom to the top of the vertical key, there were pockets of sand in the middle of the soil-bentonite. Also, in plan view of the vertical key, there was sand along all three soil-bentonite/CCL interfaces, with up to 7.6 cm at the interface parallel to the width of the wall. This created permeable pathways through the entire height of the vertical key.

The defects in W2 described above were less influential compared to the defect in W1. It was possible to apply a hydraulic gradient across the wall and measure its average hydraulic conductivity.

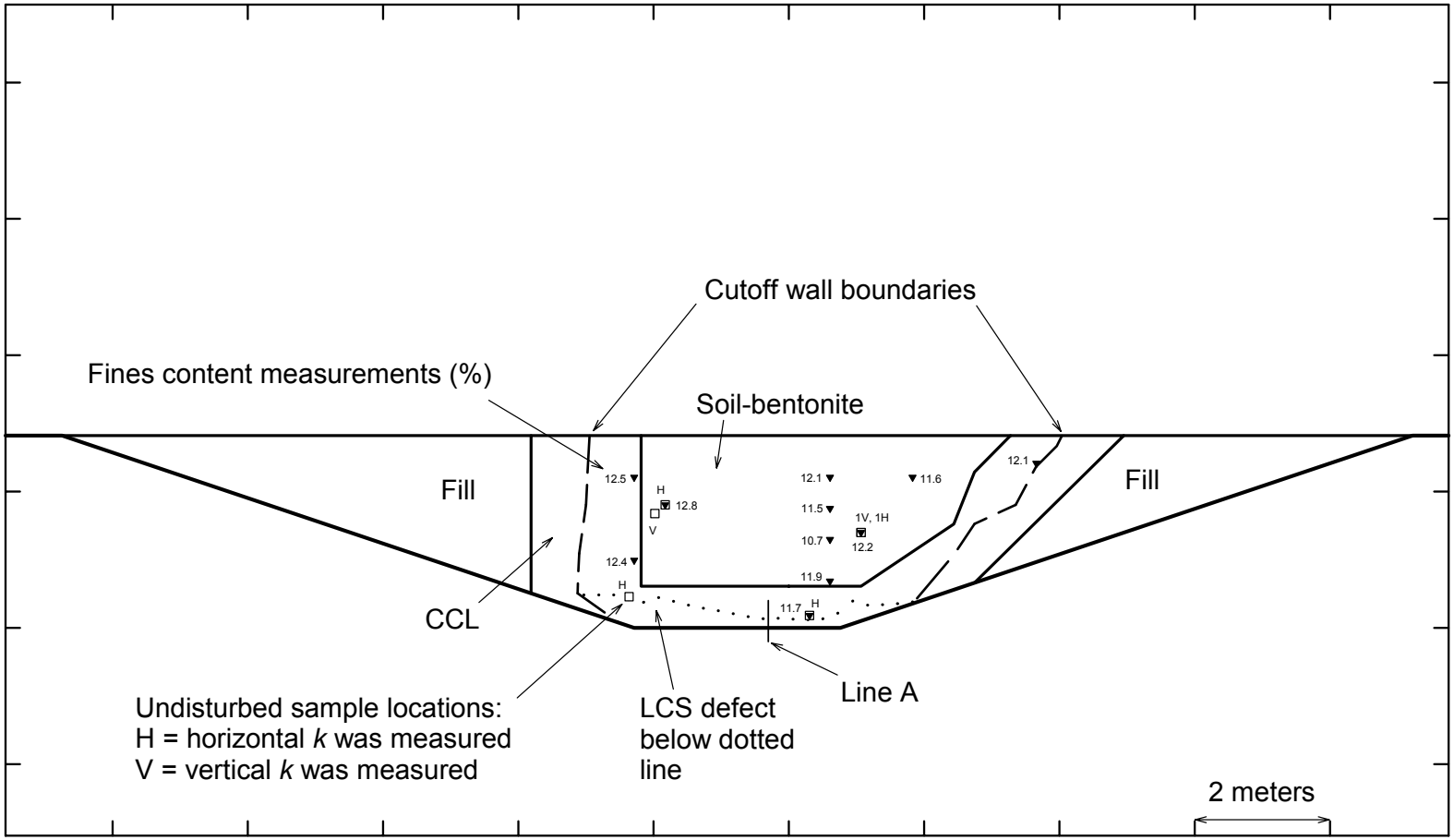

Figure 4-15a. Destructive evaluation of W2

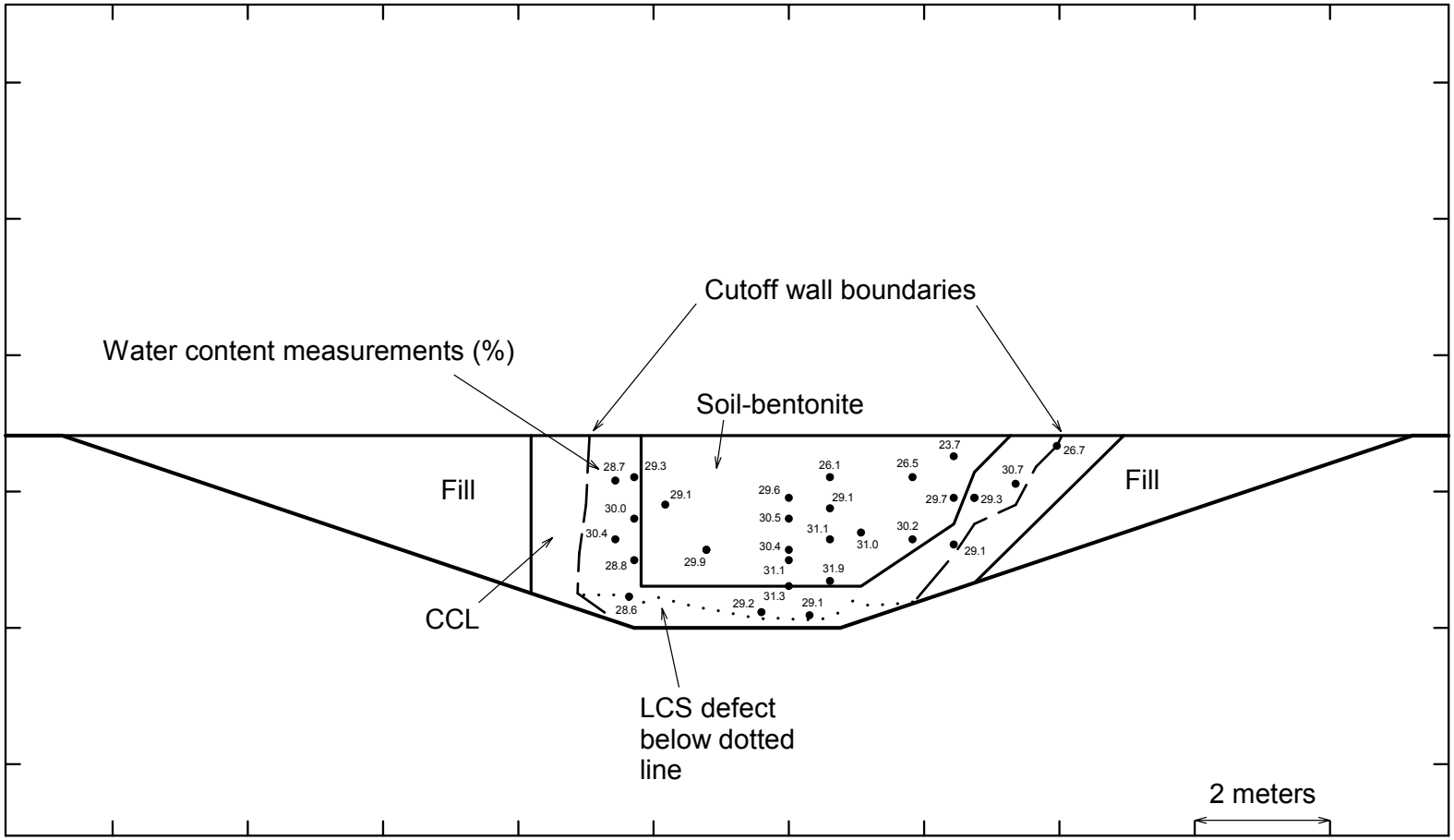

Figure 4-15b. Destructive evaluation of W2

Because the width of the trench was twice that of the trench for W1, and the soil-bentonite slid into the trench for W2 more easily, the hypothesized mechanism described above for creation of the defect in W1 seemed less likely. Two new mechanisms were hypothesized:

1. The viscosity of the bioslurry degraded so that it no longer suspended[8] sand particles. During backfilling, sand was not displaced from the trench with the bioslurry, but was instead left in significant accumulations around the edges of the wall.
2. As the soil-bentonite slid into the trench through the bioslurry (this occurred during initial placement when soil-bentonite was sliding down the 1:1 slope - before new loads of soil-bentonite could be placed directly on top of soil-bentonite already in the trench), the bioslurry flushed through the soil-bentonite to some degree and washed out some of the bentonite. This mechanism would seem to affect the soil-bentonite at the bottom of the trench the most, as this soil-bentonite slid the farthest through the bioslurry.

Water content samples, fines content samples, and undisturbed samples for hydraulic conductivity tests were taken from W2. The locations of these samples are shown in Figure 4-15, including the water content and fines content values. The fines content measurements were made to test the second hypothesis for defect formation.

The fines content measurements in Figure 4-15a do not show a big difference between values at the bottom of the wall versus the top of the wall (where soil-bentonite was less likely to be flushed by bioslurry), although there is only one fines content measurement in the soil-bentonite in the bottom key, where the fines content was hypothesized to be decreased the most according to the second hypothesis above for defect formation. The black coloration observed in the key is evidence that the bioslurry mixed with the soil-bentonite. There were three locations in the wall where both fines content and hydraulic conductivity from undisturbed samples were measured. In these locations, hydraulic conductivity increased as fines content decreased (the hydraulic conductivity values are presented in Chapter 6). This finding confirms the expected relationship between fines content and hydraulic conductivity, but there is not enough data to conclusively confirm the second hypothesis for defect formation.

There is substantial evidence supporting the first hypothesis given above for defect formation. The bioslurry used in this research has no gel strength[9]. It relies on its high viscosity to keep sand particles from rapidly falling to the bottom of the trench. Sand particles fall very slowly through fresh bioslurry, but as the bioslurry and its viscosity degrade, sand particles fall faster. Laboratory-measured relationships between bioslurry viscosity and time are shown in Figure 4-16. The bioslurry with LCS (Batch *B*)

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[8] With no gel strength, the bioslurry cannot suspend sand particles, strictly speaking, even when it is fresh. The word "suspend" is meant to describe a very small settling velocity controlled by the high viscosity of the bioslurry.

[9] Rantec has products with guar in combination with gel-forming polymers, but in this research, only guar was used.

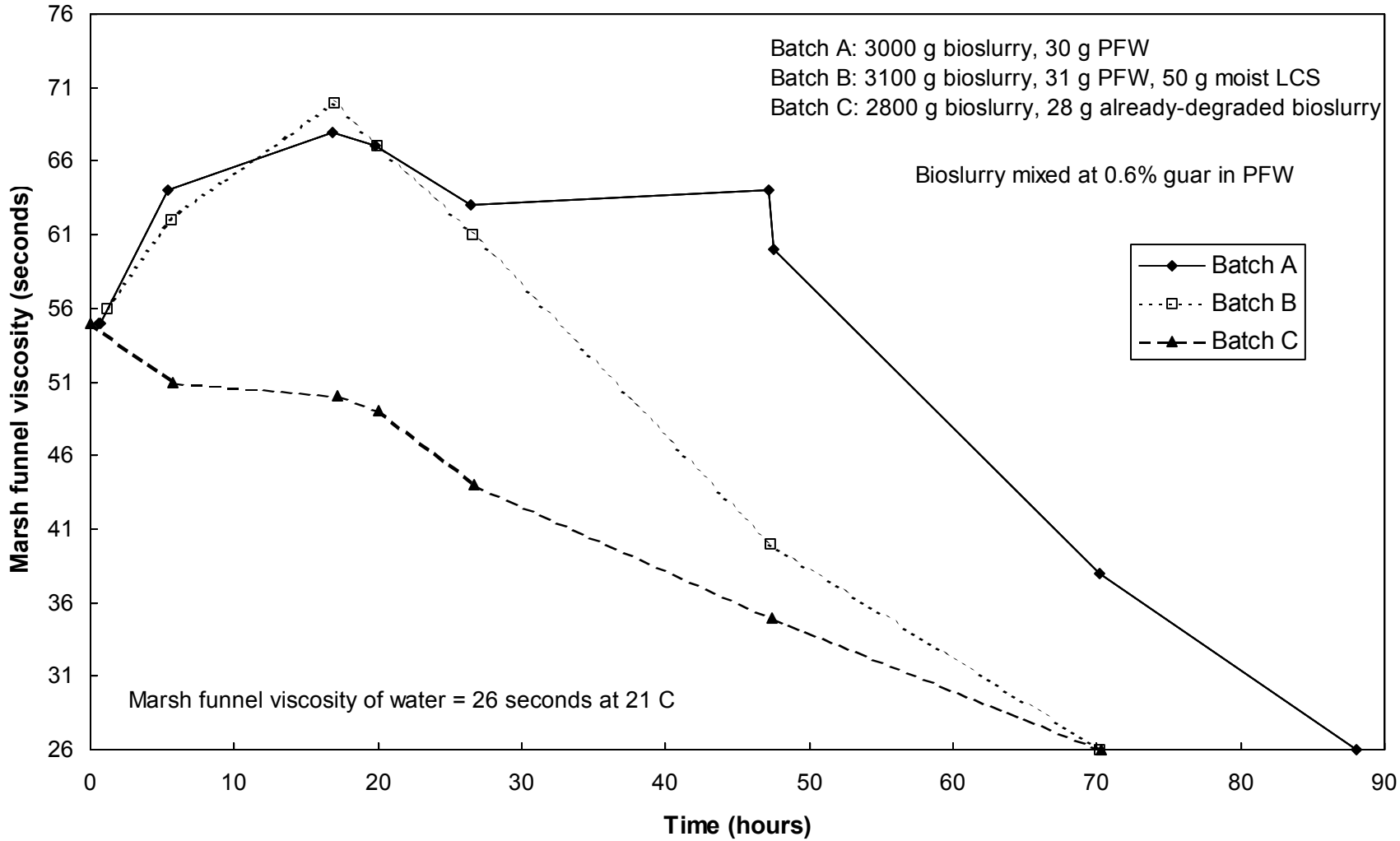

Figure 4-16. Degradation of bioslurry viscosity

degraded faster than the plain bioslurry (Batch *A*), and the bioslurry with a small component of already-degraded bioslurry, and therefore a pre-existing micro-organism population (Batch *C*), degraded the fastest. The results of these batch experiments indicate that the bioslurry degradation rate is accelerated in the trench, a point that is also mentioned by the producers of the guar (Rantec). In summary, the viscosity of the bioslurry breaks down with time in the trench; as it does, the ability of the bioslurry to keep sand from settling is diminished. The sand defects in W2 were probably accumulations of sand that did not stay suspended in the bioslurry and did not get removed when the bioslurry was removed from the trench.

4.9.1.3 Destructive Evaluation of W3

The width of W3 was measured to range from 58.5 to 62.5 cm.

The depth of desiccation was less for W3 compared to W1 and W2. Roughly the top 0.3 m of soil-bentonite appeared desiccated.

No defects were observed in W3. There was up to 2.5 cm of sand at the very bottom of the wall in the middle of the barrier pit, no sand at the bottom of the wall on the concrete slope the soil-bentonite was initially backfilled on, and from zero to a thin layer of sand at the bottom of the wall on the opposite concrete slope.

Bentonite filter cakes were observed at the trench walls. Figure 4-17 shows the filter cake between the LCS and the soil-bentonite at a depth of approximately 193 cm, just above the CCL, in the trench excavated in Stage 2. The sand at the bottom of the picture is loose LCS from the destructive evaluation; there was no LCS in the CCL or the soil-bentonite.

A filter cake of varying thickness was always observed between the LCS and the soil-bentonite. Figure 4-18 shows measurements of the filter cake thickness versus depth in the wall. Recall that the part of the trench excavated in Stage 1 was open for approximately 312 hours before backfilling and the part of the trench excavated in Stage 2 was open for approximately 8 hours. In general, the cakes were thicker in the part of the trench excavated in Stage 1. For comparison, the cake thickness predicted using Eq. 4-1 (Nash, 1974) is also shown in the figure.

$$L_{fc} = [\ (2\ k_{fc}\ h_{fc}\ (1 - n_s)\ t) / (n_s - n_{fc})\ ]^{0.5} \qquad (4\text{-}1)$$

where L_{fc} = filter cake thickness, k_{fc} = filter cake hydraulic conductivity, h_{fc} = filter cake formation head (assumed equal to depth in trench for W3), n_s = porosity of the slurry, n_{fc} = porosity of the filter cake[10], and t = length of time the trench was slurry-supported. The

[10] The porosity of the cake may change as the horizontal effective stress in the trench changes, but this complication is ignored here.

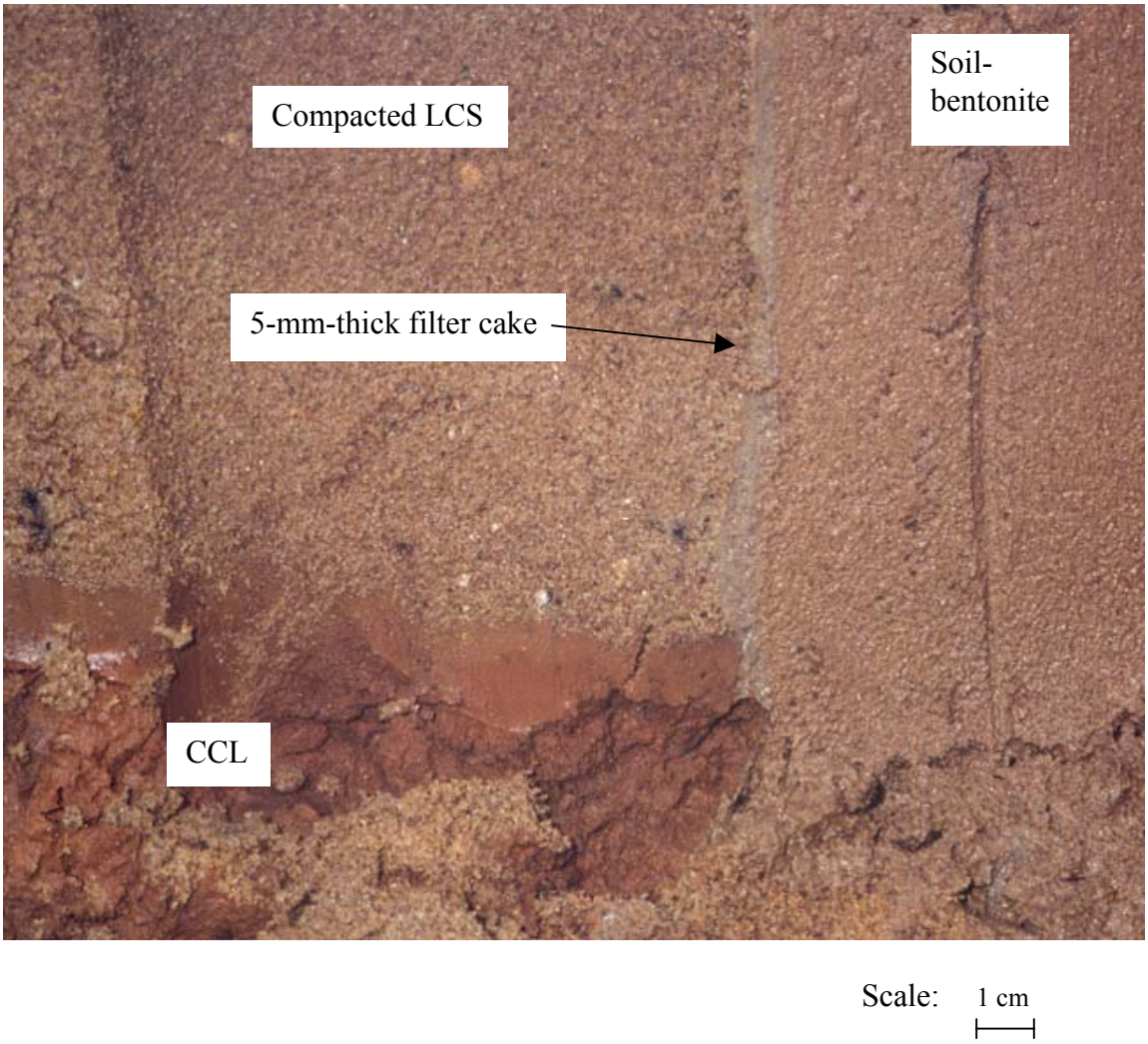

Figure 4-17. Filter cake in part of W3 excavated in Stage 2

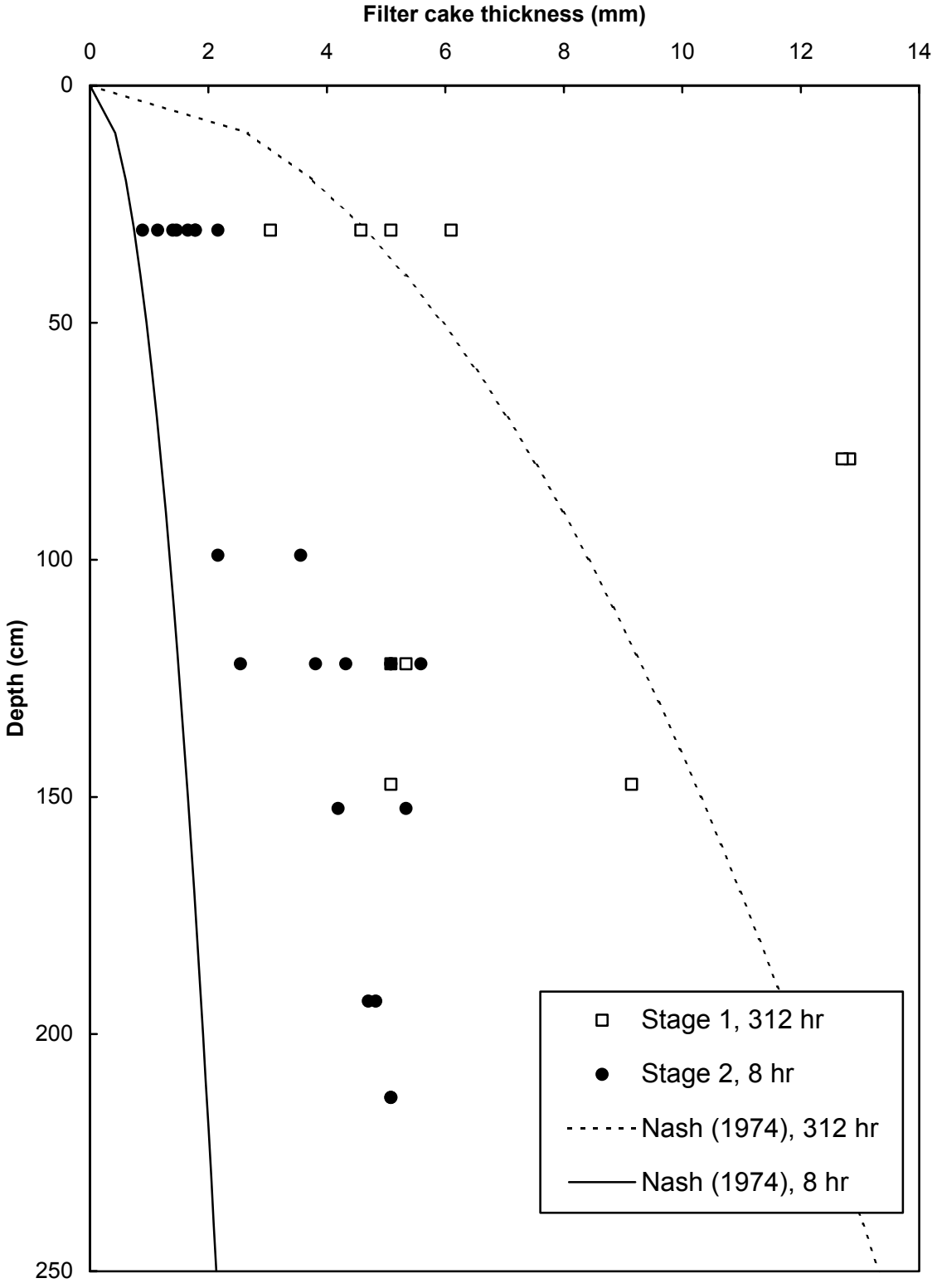

Figure 4-18. Filter cake thickness between soil-bentonite and LCS - W3

filter cake hydraulic conductivity and porosity were taken from charts in Henry et al. (1998) that relate these parameters to formation head and amount of silt and sand in the slurry. From these charts, k_{fc} is approximately 1×10^{-8} cm/s and n_{fc} is approximately 0.9 for the range of formation heads for W3 and the measured in-trench slurry unit weight of 10.5 kN/m^3. The porosity of a 6% slurry is 0.98. As seen in Figure 4-18, the measured cake thicknesses in the Stage 2 part of the wall were significantly greater than those predicted by Eq. 4-1. With the water table at the bottom of the pit during excavation, the compacted LCS was partially saturated. Capillary pressures in the LCS would cause the filter cake formation head to be greater than simply the depth in the trench. This may account for the difference between the measured and predicted thicknesses in Stage 2. There is better agreement between measured and predicted thicknesses in Stage 1, although the measured values are quite scattered.

The filter cake properties will be needed in Chapter 6 for interpretation of the in situ hydraulic conductivity test results. A value of $k_{fc} = 1 \times 10^{-8}$ cm/s and, to simplify analyses, a single representative filter cake thickness, in the depth range of interest, of 5 mm will be used.

Figure 4-19 shows the locations of water content samples and undisturbed samples for hydraulic conductivity tests. The water content values from the samples are shown in the figure.

4.10 Summary and Conclusions

The three main components of each pilot-scale experiment were 1) a low-hydraulic conductivity compacted clay liner simulating an aquiclude, 2) a high-hydraulic conductivity formation of compacted sand simulating an aquifer, and 3) a soil-bentonite cutoff wall constructed through the aquifer and keyed into the aquiclude. The hydraulic conductivity of the CCLs was on the order of 1×10^{-7} cm/s and the hydraulic conductivity of the compacted LCS formations was on the order of 1×10^{-2} cm/s. Soil-bentonite mixtures with 1% bentonite and 3% bentonite were used for the cutoff walls, having hydraulic conductivities (from API mix design tests) of approximately 2×10^{-6} cm/s and 1×10^{-7} cm/s, respectively. In these three components, head losses during hydraulic conductivity testing are assumed to be negligible in the compacted LCS compared to the head losses in the CCL and soil-bentonite. The influence of a potential flow component through the CCL when measuring the global average hydraulic conductivity of a cutoff wall is addressed in Chapter 5.

In addition to different bentonite contents in the soil-bentonite from W1 and W2 to W3, all of the bentonite was added to the Bedding Sand in slurry form for W1 and W2, while half was added in slurry form and half was added dry for W3.

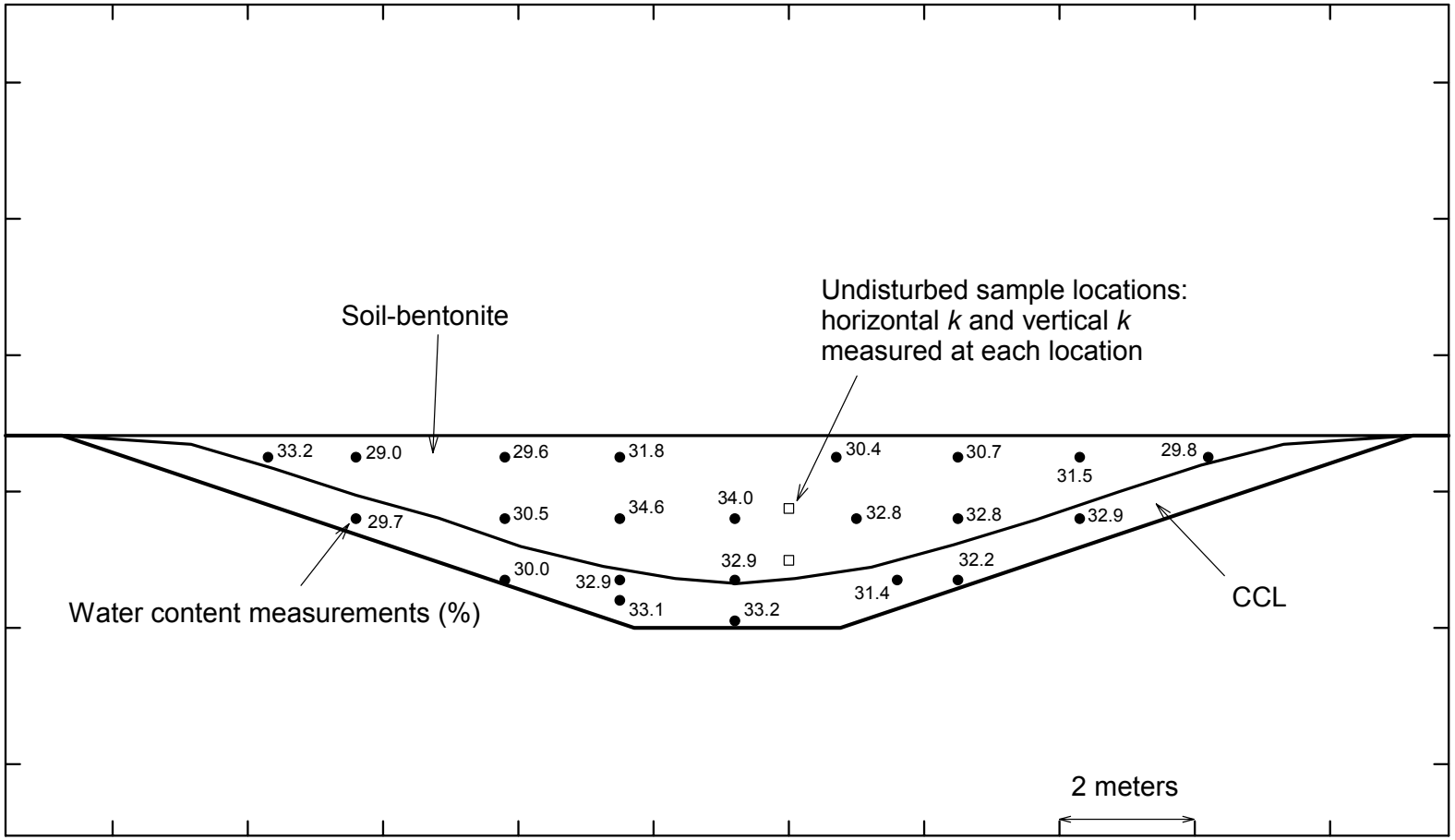

Figure 4-19. Destructive evaluation of W3

When using a relatively high soil-bentonite hydraulic conductivity of 2×10^{-6} cm/s, the existence of low-permeability bentonite filter cakes on the trench walls would significantly affect the hydraulic behavior of the cutoff wall. To avoid difficult interpretations of hydraulic conductivity tests due to low-permeability filter cakes, a biodegradable slurry was used to excavate the trenches for W1 and W2. Because use of the bioslurry leads to the formation of defects in the walls, a bentonite-water slurry was chosen for W3. In order to reduce the influence of the bentonite filter cakes on the hydraulic behavior of W3, a soil-bentonite mix with a hydraulic conductivity on the order of 1×10^{-7} cm/s was used.

During trench backfilling, six grab samples of soil-bentonite were taken for each cutoff wall. The hydraulic conductivity tests performed on these samples are described in Chapter 5.

The cutoff walls were consolidated with the water levels in the compacted LCS at the bottom of the barrier pit. The resulting effective stresses in the walls were estimated using arching theory. These effective stresses, shown in Figure 4-13, are preconsolidation pressures in the walls; in situ hydraulic conductivity tests were performed with higher water levels in the LCS, and therefore lower effective stresses[11]. Because the recompression ratio of the soil-bentonite is low, a reasonable approach is to relate hydraulic conductivity to preconsolidation pressure. For example, when the global average hydraulic conductivity of a wall is measured, the hydraulic conductivity may be related to the consolidation pressures shown in Figure 4-13 evaluated with the water levels at the bottom of the barrier pit, even though the water levels during the global measurement are higher than the bottom of the pit.

It was found through the experiences of W1 and W2 that use of bioslurry (composed solely of PFW and guar gum) for trench excavation support may increase the chances of defects forming in a cutoff wall. Defects existed in both W1 and W2. The defects were attributed primarily[12] to breakdown of the bioslurry, and its viscosity, in the trench, causing an increased settling rate of sand through the bioslurry and an inability of the bioslurry to remove sand from the trench as it was displaced during backfilling. The use of additives to prevent degradation of the biopolymer was not studied in this research.

There is sometimes uncertainty regarding the state of bentonite filter cakes after backfilling. Do the cakes remain intact, or do they get scraped off to some degree by the moving backfill? In the case of W3, the filter cakes did remain intact. This case supports the claim that bentonite filter cakes remain intact during backfilling and may lower the equivalent hydraulic conductivity of the cutoff wall.

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[11] The exception is the piezometer test, because insertion of the piezometer results in a higher major principal effective stress in the wall compared to the stresses in Figure 4-13.

[12] The other hypothesized mechanism was flushing of the soil-bentonite by the bioslurry, possibly washing fines from the soil-bentonite.