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Design and Construction of a French Drain for Groundwater Diversion in Solid Waste Storage Area Six at the Oak Ridge National Laboratory

E. C. Davis
R. G. Stansfield

ENVIRONMENTAL SCIENCES DIVISION
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Environmental Sciences Division

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Design and Construction of a French Drain for Groundwater Diversion in Solid Waste Storage Area Six at the the Oak Ridge National Laboratory[a]

E. C. Davis and R. G. Stansfield[b]

[a]Supported by the Office of Defense Waste and By-Product Management, U.S. Department of Energy under contract DE-AC05-84OR21400 with Martin Marietta Energy Systems, Inc.

[b]Private Geotechnical Consultant.

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ABSTRACT

Engineering modifications or engineered barriers have been suggested as a possible means of improving the performance of low-level waste disposal sites located in the humid eastern United States. This report describes the design and construction of one such barrier, a passive French drain, located in Solid Waste Storage Area No. 6 at the Oak Ridge National Laboratory. The drain was designed to hydrologically isolate a 0.44-ha area that contains a group of 49 low-level waste trenches by separating it from upgradient groundwater recharge areas. The 252-m drain (maximum depth = 9 m) that surrounds the group of trenches on the north and east sides was excavated, lined with filter fabric, backfilled with crushed stone, and covered with a 0.6 m layer of excavated material at an estimated cost of $153,000. Of the 17 days it took to complete the work, about 5 days (29% of the time) were spent excavating sidewall slide material that fell into the drain during excavation. Photography of the drain wall revealed the contorted structure of the weathered shale, which was responsible for many of the slides. Monitoring wells placed at intervals on the drain centerline indicate that groundwater is draining from the surrounding Maryville Formation (Conasauga Group); flows at catch basin No. 2 (serving the eastern portion of the drain) ranged from a base flow of 4–7 L/min to a maximum of 35 L/min, recorded on October 13. In response to groundwater flow in the drain, water levels in several monitoring wells adjacent to the drain have dropped by as much as 2.24 m to an elevation only slightly higher than the bottom of the French drain. In addition to the general lowering of the water table in the vicinity of the drain, water levels in three trenches (T191, T201, and T203) began to subside, indicating that the drain is beginning to have an effect on the water in the trenches as well. Further monitoring of both drain discharge and water levels in monitoring wells across the site is continuing to provide the data necessary for a detailed study of the effectiveness of the drain as a remedial measure.

1. INTRODUCTION

One major concern with the practice of shallow land burial (SLB) of low-level radioactive waste in the eastern United States is the potential for water-waste contact and the resulting radionuclide migration. The problem stems from the fact that intruding water enters waste trenches, either as direct surface infiltration or as lateral subsurface flow, and will often remain in the more porous waste/backfill matrix, where radionuclides can be leached from the waste and transported to unrestricted areas. These water-related concerns have been documented at both Department of Energy (DOE) and commercially managed disposal sites (Carter et al. 1979). In the case of the disposal site located at Maxey Flats, Kentucky, these concerns led to changes in waste disposal operating procedures, the levying of higher taxes on waste materials, and, eventually, closure of the site.

Engineering modifications or use of engineered barriers have been suggested as possible means of improving the performance of low-level waste (LLW) disposal sites, particularly where natural hydrologic conditions are not ideal. Engineered barriers include such structures as surface water-diversion channels, trench covers or seals, trench liners, groundwater-cutoff walls, and trench drainage systems. Regardless of the type of structure being considered, the principal objective is to keep water from coming into contact with the buried waste, or at least to minimize the duration of such contact. A barrier can be constructed as part of the routine site operations, or it may be required as a remedial action after the site, or a particular group of trenches, has been closed. The purpose of this report is to describe and document the construction of one such engineered barrier, a French drain, constructed in Solid Waste Storage Area No. 6 (SWSA 6) at the Oak Ridge National Laboratory (ORNL). The report covers only the design and construction phases of the project. Detailed evaluations of drain effectiveness will be presented in future reports after performance data have been gathered and analyzed.

1.1 SITE DESCRIPTION AND OPERATING HISTORY

The site selected for the drainage project is in SWSA 6 located in Melton Valley, about 3 km to the southwest of ORNL (Fig. 1.1). SWSA 6 contains 28 ha of gently-to-moderately sloping land that is divided into several smaller sections by natural gullies and seasonal streams. The area of concern to this project is a small (0.44-ha) subsection of SWSA 6 containing 49 LLW disposal trenches (49-Trench area), which were filled with waste and closed in 1973–1974.

Two small streams drain the 49-Trench area, one along the western boundry of the site and the other to the south (Fig. 1.1). Both streams drain in a southward direction to White Oak Lake and are usually observed to be dry during July, August, and September. Groundwater recharge to the area is believed to be from higher elevations to the northeast of the site; however, the area immediately east (ORNL grid) of the trenches is probably as important because it lies along the strike of a major fracture in the area (Sledz and Huff 1981), which increases hydraulic conductivity in the east-west direction. In general, groundwater levels in SWSA 6 have been found to follow the surface topography and range from depths of less than 1 m in the lower areas to more than 8 m in the higher elevations (Webster et al. 1980). Figure 1.1 shows the 49-Trench area located in the center of SWSA 6 and the approximate location of the French drain.

1.2 ENVIRONMENTAL CONCERNS OF THE 49-TRENCH AREA

Shortly after closure of the 49-Trench area, it was noted that many trenches were collecting and holding water, particularly during the wet winter and spring months (Arora et al. 1981). In an

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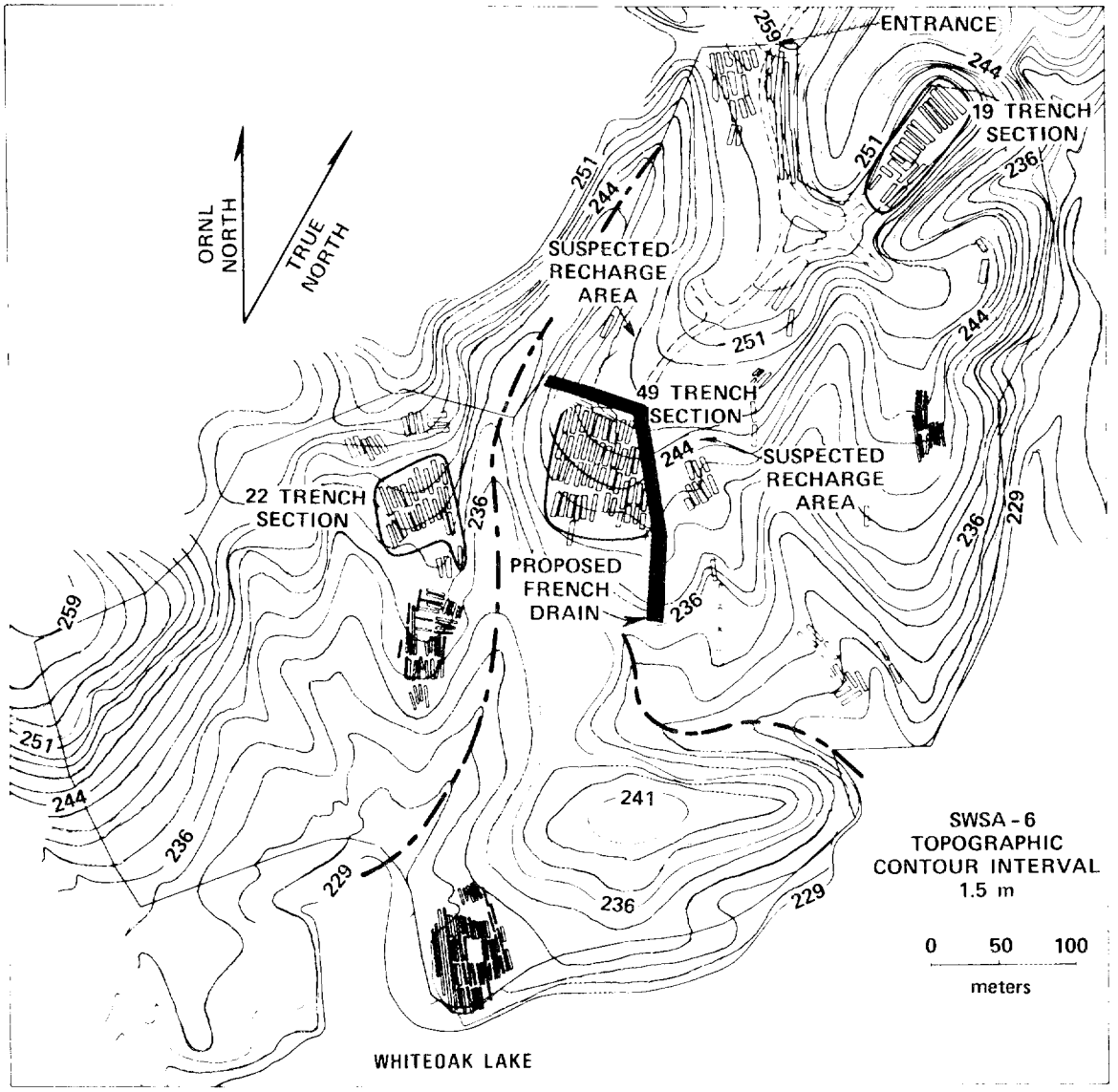

Fig. 1.1. ORNL SWSA 6 showing the location of the 49-Trench area and the passive french drain.

attempt to prevent rainfall from infiltrating the cover material and collecting in the trenches after each rain, the entire 0.44-ha area was sealed by using a bentonite cover in October and November of 1976. This sealing operation was to have led to a dewatering of the underlying trenches.

The sealing procedure consisted of removing the top 0.5 m of cover material, applying bentonite (granular grade Volclay Saline Seal 100, American Colloid Company) at a rate of 19.5 kg/m^2, disking to a depth of 7.5 cm to give an approximate 5%-bentonite-shale mixture, and compacting the seal in place. When completed, the seal was covered to a depth of about 60 cm, using previously removed cover material, and was seeded to prevent erosion. Two major difficulties were encountered in the sealing operation: (1) ensuring that the moisture conditions of the soil were appropriate for the addition of bentonite (18–20% soil water by weight) and (2) sealing around observation wells (done by hand).

Despite the bentonite cover, water was still observed to collect in the underlying trenches as evidenced by Figs. 1.2 and 1.3, which show seasonal water-level variations in four wells in the 49-Trench area. Wells T141 and T150 are located in waste disposal trenches 141 and 150, respectively, whereas wells 401 and 402 are not in trenches but are located in the surrounding soil, which

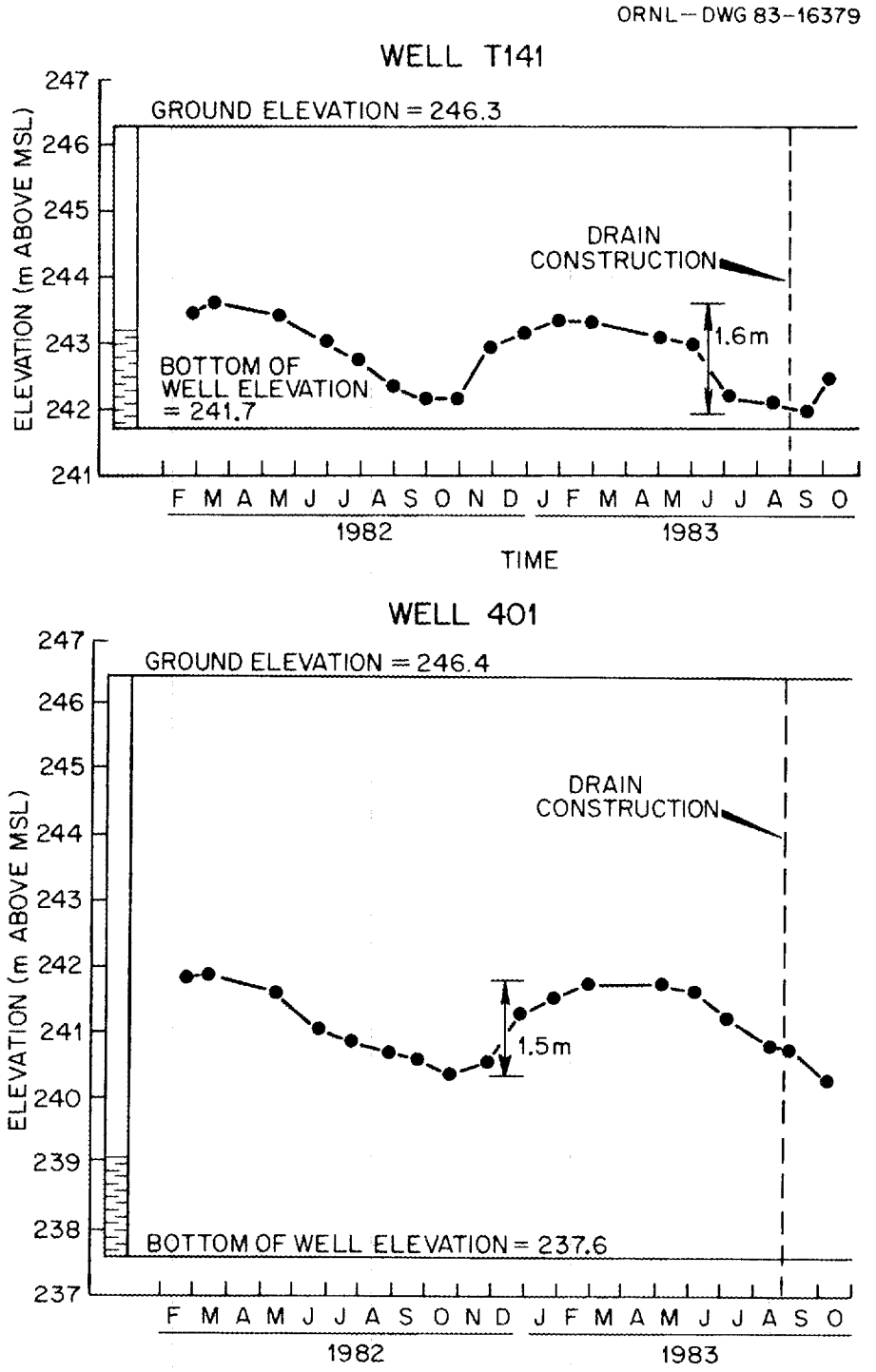

Fig. 1.2. Seasonal water-level fluctuations in wells T141 and 401. The depth of Trench 141 is about 4.6 m.

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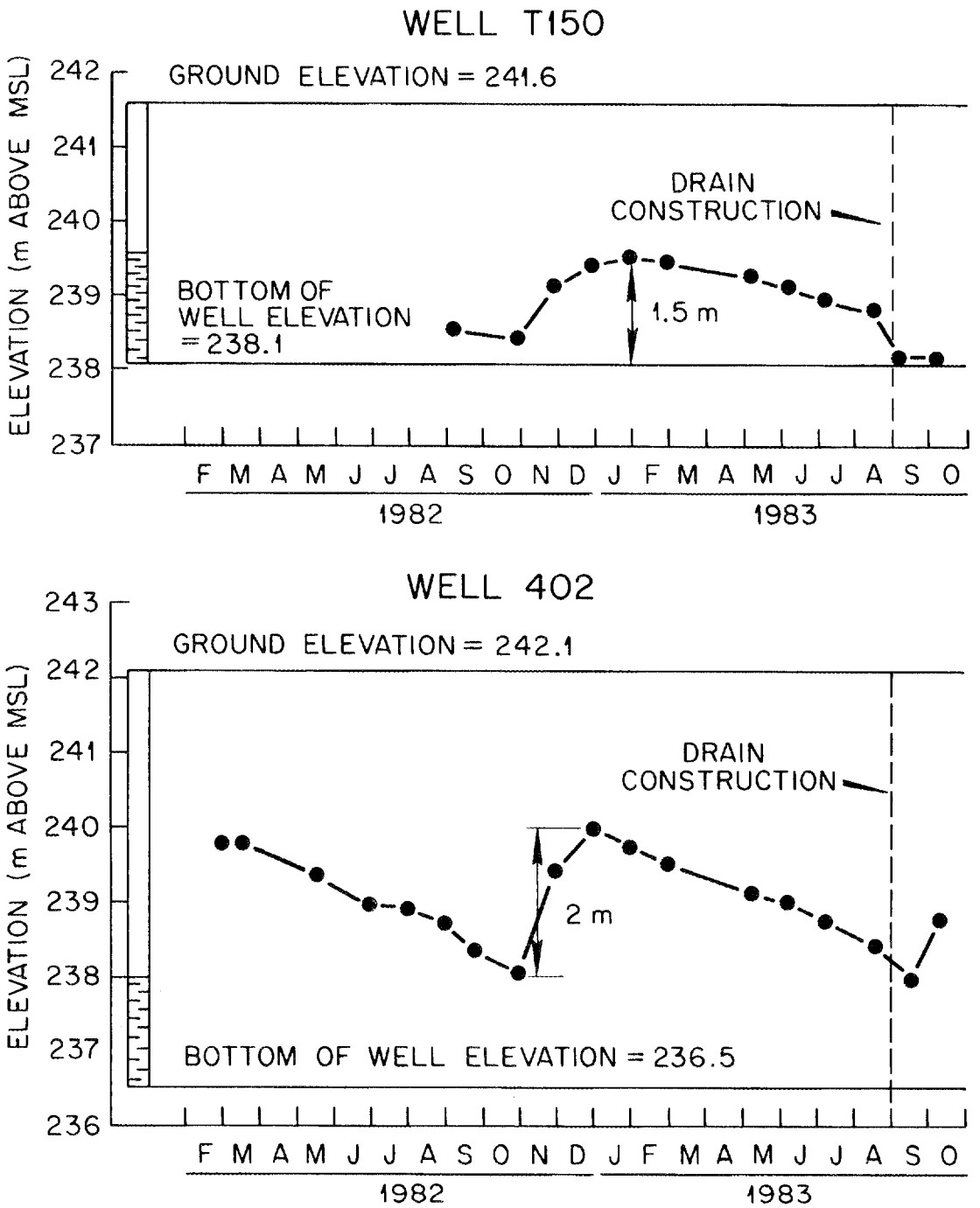

Fig. 1.3. Seasonal water-level fluctuations in wells T150 and 402. The depth of Trench 150 is about 4.6 m.

is of the Conasauga Group. These wells were selected for illustration of the water intrusion problem for two reasons: each pair (T141,401 and T150,402) is within a few meters of the other, one inside a trench and the other outside, and one pair (T141,401) is in the northern group of trenches, whereas the second (T150,402) is in the lower elevation group of trenches to the south. By comparing seasonal fluctuations of water levels in wells T141 and 401, waste disposal trench 141 is

observed to be collecting and holding water about 1.5 m above the water levels measured in adjacent well 401. This is typical of the "bathtub" effect that has been observed in many trenches in the 49-Trench area as well as at other disposal sites at ORNL. In addition, maximum water elevations observed in wells 401 and 402 for the period of record are 241.9 and 240 m, respectively, and are high enough to intersect the bottoms of surrounding waste disposal trenches. The annual water-level fluctuation of 1.5 m in these two trenches is typical of the entire 49-Trench area. The maximum water level occurs in January, February, and March; the minimum occurs in September and October. The seal may be serving to keep out direct infiltration of rainfall; however, now, seven years since construction, water levels in trenches still fluctuate, suggesting that a faulty seal might be responsible or that an alternative mechanism, such as subsurface flow from outside the sealed area, might be contributing to water in the trenches.

To further illustrate the water problem existing in the 49-Trench area, two subsurface profiles have been constructed and are shown in Fig. 1.4. (See Fig.1.5 for location of profiles.) Water elevations measured at two different times of the year are noted on the profiles of wells located both inside and outside of the waste disposal trenches. Based on data presented in Figs. 1.2 and 1.3, these two dates (February and September 1983) should represent high and low values, respectively, of the water table in the area. During February the groundwater intersects the bottoms of the trenches, particularly in the southern portion of the 49-Trench area (see well 399, Fig. 1.4). Although the maximum seasonal water levels are shown in Figs. 1.2 and 1.3, the maximum instantaneous levels would possibly be much greater if measurements were taken during or immediately after a heavy rainfall event.

Because water is entering the trenches, the potential for radionuclide migration to surrounding surface waters exists. Evidence for such migration has been documented in the creek to the west of the 49-Trench area by Cerling and Spalding (1981). From their analysis of streambed gravels within the White Oak Watershed, it was determined that a ^{90}Sr seep exists in this creek and that the source is located within the 49-Trench area. Concentrations of ^{90}Sr associated with gravels downstream of the seep were found to range between 1.7 and 8.3 Bq/g and were higher in the immediate vicinity of the seep (20.5 Bq/g). Upstream of the seep, concentrations of ^{90}Sr in streambed gravels were closer to background levels, ranging between 0 and 0.08 Bq/g.

Trench water samples collected in September 1982 suggest that trenches 150 and 152 contain a high inventory of ^{90}Sr (Table 1.1 and Fig. 1.5) and are possibly responsible for the identified seep. Thus, the environmental concerns associated with the 49-Trench area are related to the presence of water in the trenches and the movement of ^{90}Sr from the buried waste.

1.3 FRENCH DRAIN CONCEPT

Whether or not the bentonite seal has reduced the amount of surface infiltration, water is still collecting in the trenches within the 49-Trench area. For this reason, a second engineered barrier was proposed for the area, this time to prevent the lateral movement of groundwater into the trenches and, essentially, to isolate the 49-Trench area from its upgradient groundwater recharge area. The idea was to construct a French drain to a maximum depth of approximately 9 m (thus ensuring that the bottom of the drain be below the elevations of the bottoms of the trenches) and sloped in such a way that it drains to a lower elevation discharge point. The surface topography of the 49-Trench area is well suited for two such drains (see Figs. 1.1 and 1.5). The northern section of the drain slopes westward (subparallel to geologic strike) and discharges into the creek to the west of the site. The eastern section slopes southward (perpendicular to strike) and discharges into

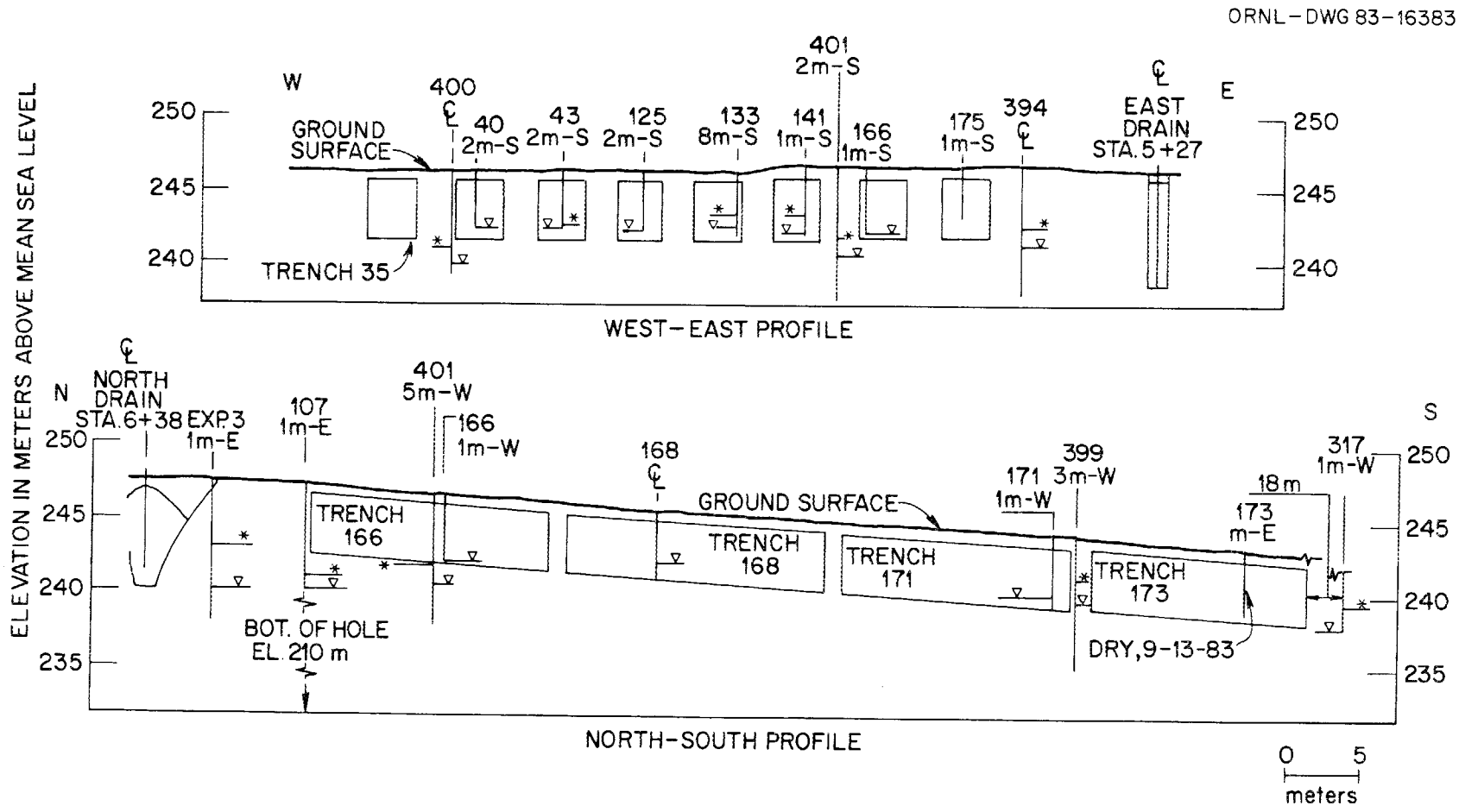

Fig. 1.4. Water-table profile at two cross-sections within the 49-Trench area.

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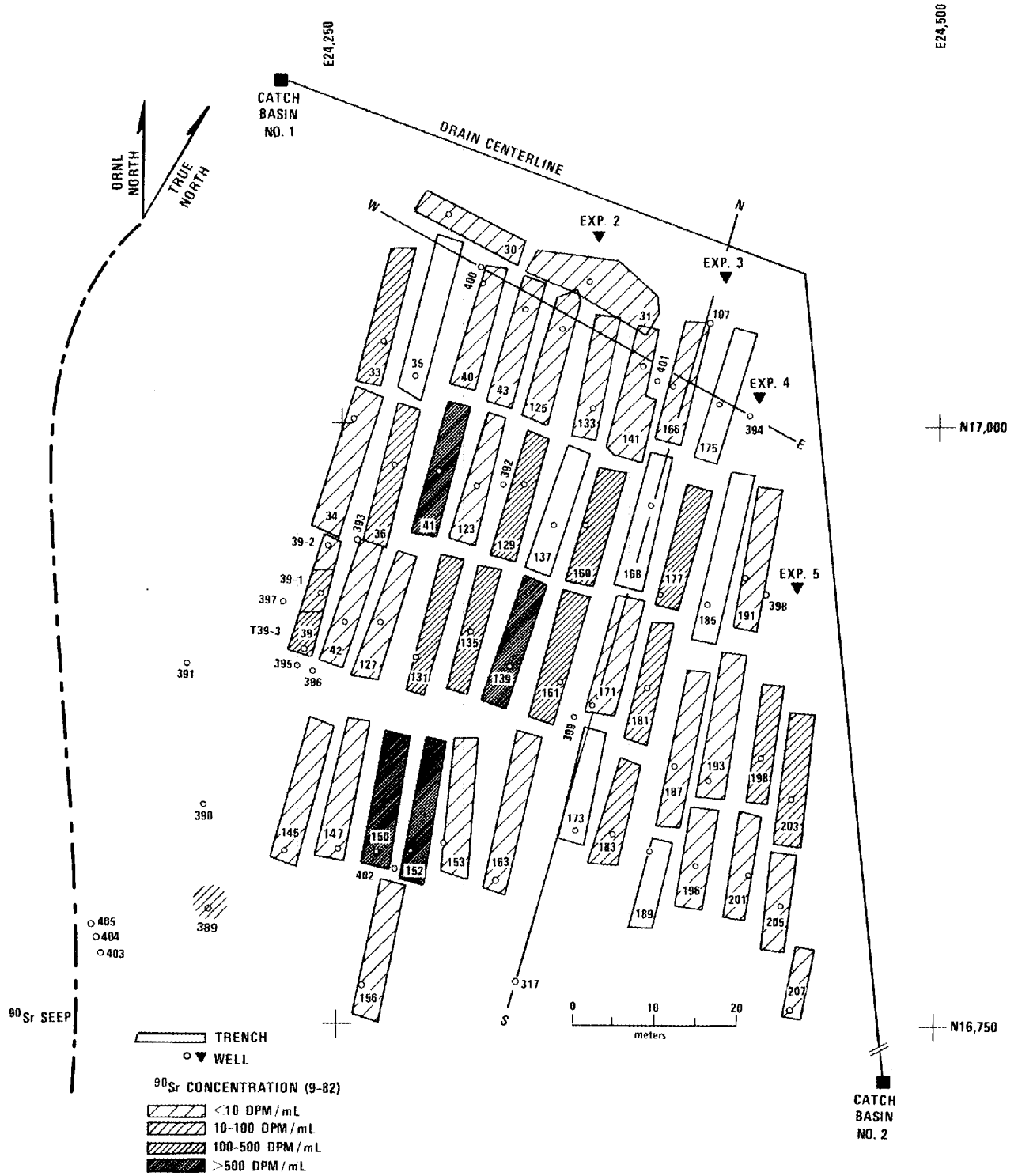

Fig. 1.5. ^{90}Sr concentrations in groundwater samples collected at the 49-Trench area, September 1982.

Table 1.1. Analysis of water samples collected from trenches in the 49-Trench area (September 1982)

Well[a]	pH	EC (μS/cm)	Hardness as $CaCO_3$ (mg/L)	^{90}Sr (Bq/ml)
T30	6.7	1570	300	0.01
T31	6.0	328	90	0.05
T33	5.9	560	102	0.39
T34	6.4	650	280	0.09
T35	6.7	420	*b*	*b*
T36	6.0	420	170	0.36
T39-1	6.6	660	320	0.72
T39-2	6.2	520	250	0.04
T39-3	6.1	240	130	2.15
T40	6.6	800	378	0.10
T41	6.3	998	340	10.69
T42	5.9	450	150	0.06
T43	6.3	500	*b*	0.01
T123	5.6	183	90	0.13
T125	6.0	242	153	<0.01
T127	5.8	110	50	0.05
T129	6.5	1180	490	5.62
T131	6.2	1140	710	6.36
T133	6.7	278	109	<0.01
T135	6.2	680	390	3.94
T139	6.3	1110	670	13.43
T141	6.2	390	150	0.15
T145	5.6	77	40	<0.01
T147	6.2	389	220	0.14
T150	6.6	825	340	61.48
T152	6.4	600	270	23.89
T153	6.4	101	40	<0.01
T156	6.5	365	110	0.02
T160	6.4	900	440	5.24
T161	6.2	910	490	2.35
T163	6.6	130	70	<0.01
T166	6.4	620	330	0.12
T171	5.6	170	80	<0.01
T177	6.6	1000	540	2.15
T181	6.2	1120	480	1.31
T183	6.4	1230	140	1.06
T185	7.4	1000	440	*b*
T187	6.4	313	160	<0.01
T191	5.8	120	80	*c*
T193	5.8	399	180	<0.01
T196	5.9	400	100	<0.01
T198	6.1	478	110	1.05
T201	6.4	420	160	0.12
T203	6.4	800	320	1.42
T205	6.8	121	60	<0.01

Table 1.1 (continued)

Well[a]	pH	EC (μS/cm)	Hardness as $CaCO_3$ (mg/L)	^{90}Sr (Bq/ml)
T207	6.3	550	250	0.14
107	3.8	400	240	<0.01
317	7.3	349	190	<0.01
389	6.1	260	150	0.47
390	5.8	84	60	*c*
391	5.5	70	30	*c*
392	6.3	580	330	0.02
393	6.7	730	450	<0.01
394	6.5	260	132	<0.01
395	7.2	550	310	<0.01
396	6.4	258	100	0.01
397	6.6	730	360	<0.01
398	7.4	200	200	<0.01
399	6.4	210	100	<0.01
400	6.6	255	312	<0.01
401	7.2	600	210	<0.01
402	6.6	109	70	<0.01
403	7.2	181	100	<0.01
404	6.5	1150	420	0.01
405	7.4	450	190	0.02
EXP-4	6.2	197	129	<0.01
EXP-5	6.7	368	200	<0.01
SEEP	7.1	430	260	0.02

[a]For a location of the wells refer to Fig. 1.5.

[b]Measurement not made.

[c]Background—less than or equal to the activity of a distilled water blank.

the creek to the south of the site. Being located on two sides of the 49-Trench area, the drains are designed to lower the flux of water into the trenches during storm events and, also, to suppress the groundwater table, thus preventing its intersection with the trenches.

In addition to possibly solving groundwater recharge problems in the 49-Trench area, the French drain concept has potential application at other disposal locations at ORNL. One area being considered for construction of a similar type of drain is intermediate-level waste (ILW) trench No. 7 (Olsen et al. 1983). It is an example of a liquid waste disposal trench located near a large upslope recharge area and containing a significant inventory of ^{137}Cs, ^{60}Co, and ^{90}Sr. During the wet seasons of the year, groundwater may intersect a portion of the trench or adjacent contaminated soils and solubilize and transport radionuclides, in particular ^{60}Co, to a small seep located downslope (east) of the trench. As in the case of the 49-Trench area, a groundwater interceptor drain could correct this problem and secure trench No. 7 against future groundwater intrusion. Thus, the French drain concept has potential application to a variety of problem areas at ORNL burial sites but lacks a proven demonstration of its usefulness in the hydrologic isolation of radioactive waste disposal areas.

2. DESIGN OF THE DRAINAGE SYSTEM

The drainage system planned for the 49-Trench area was designed to meet two interrelated objectives:

1. to intercept and divert subsurface flow from upgradient recharge areas before it can enter the trenches and
2. to lower the water table and keep it supressed during the wet months of the year.

With these two objectives in mind, several design alternatives were considered before final selection of the French drain.

2.1 DESIGN ALTERNATIVES CONSIDERED

Three approaches were considered for achieving the desired results:

1. installation of a pervious material to intercept groundwater,
2. installation of an impermeable barrier to divert groundwater around the site, and
3. a combination of the first two alternatives.

Table 2.1 summarizes each of these strategies, giving examples of required construction techniques. A more detailed description of each of these alternatives is contained in Appendix A.

From the list of alternatives presented in Table 2.1, the narrow French drain option was selected for detailed study. Advantages of a drain over an impermeable barrier include the fact that a drain does not have to be excavated to bedrock or other impermeable material, as would an effective groundwater barrier. In addition, a drain provides a highly permeable route for water to bypass the 49-Trench area and will not cause a rise in water-level (hydraulic head) upslope of the drain, as would an impermeable groundwater barrier. Therefore, disposal operations upslope of the 49-Trench area should not be affected by a drain but might be affected by the rising water levels resulting from construction of a barrier. Based on these considerations and consideration of cost, ease of construction, and probability of success, the passive French drain was selected and detailed design was initiated.

2.2 LAYOUT OF THE FRENCH DRAIN

Two sections of drain were designed to meet in the northeast corner of the site (Fig. 2.1). The northern leg of the drain begins at station 6+00 (ORNL coordinates N17065.2, E24440.0, approximate surface elevation = 248 m) and drains westward to catch basin No. 1, located at station 8+25.66 (ORNL coordinates N17143.4, E24228.3, approximate surface elevation = 239.4 m). The total length of this section of the drain is 69 m.

The eastern leg of the drain begins at station 6+00 and drains southward to catch basin No. 2, located at station 0+00 (ORNL coordinates N16468.0, E24493.0, approximate surface elevation = 235.4 m). The length of this section of the drain is 183 m, making the total length of the drain 252 m. No drain was needed to the west and south of the 49-Trench area because of the lower elevations in these regions and the resulting lack of substantial groundwater recharge. A detailed site plan that summarizes the two legs of the drain is presented in Fig. 2.1.

Table 2.1. Alternatives considered as 49-Trench area groundwater barriers

	Drains
Scheme Ia	— A French drain in a narrow trench excavated with a backhoe
Scheme Ib	— A French drain with sloped excavation 1.5 : 1 slope
Scheme Ic	— A French drain installed in a braced excavation
Scheme Id	— A free draining rock wall constructed with drilled piers
	Barriers
Scheme IIa	— A narrow slurry trench excavated with a backhoe
Scheme IIb	— A cementatious cutoff wall constructed with drilled piers
Scheme IIc	— A sheet pile cutoff wall
	Combination of Drain and Barrier
Scheme IIIa	— A French drain in series with a slurry trench. Schemes Ia and IIa
Scheme IIIb	— A French drain with an impervious membrane included and excavated with a backhoe
Scheme IIIc	— A cementatious cutoff wall constructed with drilled piers and a free draining rock wall. Schemes Id and IIb

2.3 PLANNING FOR CONSTRUCTION

The French drain was designed as a 0.9-m-(3-ft-)wide trench, having a filter fabric liner and backfilled to within 0.6 m (2 ft) of the surface with crushed stone. The purpose of the filter fabric is to filter out any fine silt or clay particles that could plug flowpaths within the gravel, thus decreasing the life of the drain. After gravel placement, the fabric was folded over the top of the gravel backfill and the remaining 0.6 m of open ditch was backfilled, using previously excavated material, to the original surface elevation. Wells were to be placed in the drain every 7.6 m (25 ft) to serve as water monitoring points. Typical cross sections of the drain are shown in the upper left-hand corner of Fig. 2.2.

No pipe was placed in the bottom of the drain, except for the short sections between stations 0+00 and 0+50 and stations 8+25 and 7+85. These end sections of the drain are fitted with a 15-cm (6-in.) plastic pipe that serves only to move water from station 0+50 to catch basin No. 2 and from station 7+85 to catch basin No. 1 (Fig. 2.2). Gravel was not used as backfill in the portion of the drain carrying the pipe but was backfilled with excavated material and compacted to the original ground surface. Stations 0+50 and 7+85, thus, represent the interface between the French

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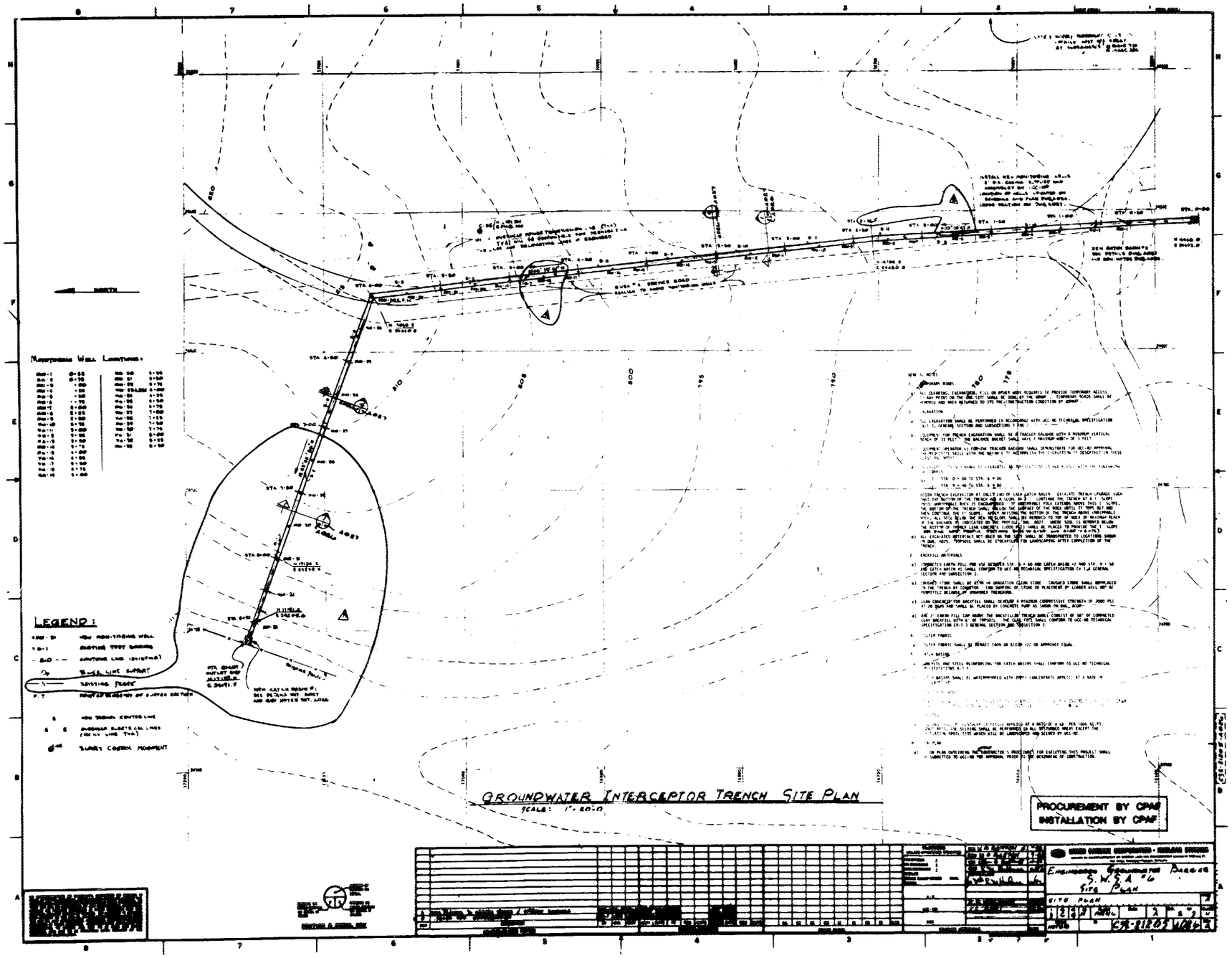

Fig. 2.1. Site plan for the 49-Trench area French drain.

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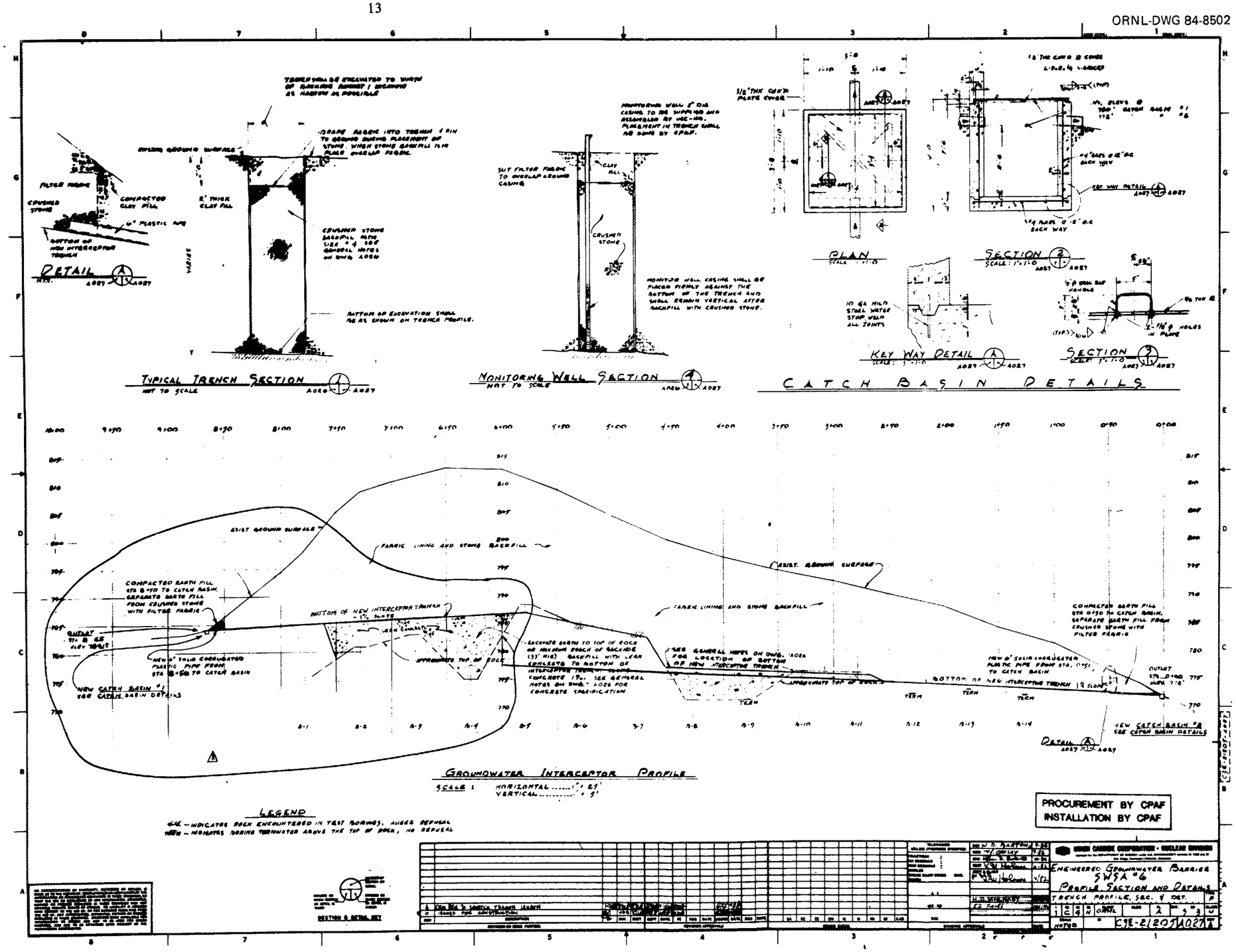

Fig. 2.2. Cross-section of the 49-Trench area French drain.

drain and the pipe leading to the respective catch basins. Fig. 2.2 shows the drain in cross section and details the two sections that contain the 15-cm pipe.

2.3.1 Excavation

To achieve the maximum benefit from the French drain, station 6+00 (the corner where the two drains meet) had to be excavated to the greatest depth possible. However, several limiting factors had to be considered, including the elevations of the catch basins: station 6+00 obviously could not be excavated below the elevation of either catch basin or water would not drain in the required direction. In addition, the limitations on the reach of the backhoe had to be considered along with the possibility of hitting unrippable rock. Under ideal conditions, if one assumes that no unrippable rock will be encountered and the backhoe can dig to any depth, the depth of the drain at station 6+00 can be calculated by maintaining a 1% slope (the slope of the drain bottom) from each of the catch basins to station 6+00. The following results were obtained: a depth at station 6+00 of 10.4 m (34 ft), based on a 1% slope from catch basin No. 2, and 7.9 m (26 ft), based on a 1% slope from catch basin No. 1. Thus, under conditions of no rock, station 6+00 would step down from 7.9 m (northern leg) to 10.4 m (eastern leg).

To answer the question concerning rock, test borings were made every 15.2 m (50 ft) along the proposed centerline of the drain. A Mobile D-53 drill rig having a 15-cm (6-in.) auger was used. At each station the test drilling ceased at auger refusal (the point at which it was concluded that the auger was hitting rock) or when the auger was at a depth several meters below the design 1% grade line (drain bottom). Table 2.2 summarizes the depth to rock that was measured by this technique, along with the depth of the drain in that area (based on a 1% slope from the outlet of the respective catch basin).

From the information presented in Table 2.2, rock above the 1% grade line was expected to be encountered along the eastern leg of the drain between stations 5+75 and 4+75 and stations 3+25 and 2+75. Thus, the depth of the drain at station 6+00 could range anywhere between 7 m (23 ft) and 10.4 m (34 ft), depending on the hardness of the rock. As a result, the excavation depth for this section of the drain could not be precisely planned beforehand but would depend on how much rock was found, how it dips, and how easily it could be removed. For the northern section of drain (stations 6+00 to 8+25) unrippable rock did not appear to be a problem because it was below the planned drain bottom. Figure 2.2 is a profile of the French drain, showing the depth to rock at the various test borings, the design 1% grade, and the speculated drain bottom, based on depth to rock.

Results of the test drillings shown in Fig. 2.2 indicate that several points occur along the eastern leg of the drain where rock extends above the 1% grade line and then dips back down below this line. For maximum benefit in keeping water out of the trenches, it was decided to excavate between these rock summits to either the maximum reach of the backhoe or to unrippable rock and then backfill with lean concrete back to the design drain bottom. The concrete would serve two purposes: (1) it would act as a barrier to deeper water flowing under the French drain and into the 49-Trench area in these regions and (2) it would serve as an impermeable drain floor that would prevent water from being picked up at one point upslope and transported down-gradient to a more permeable region. As discussed subsequently, (Sect. 3.2.1), frequent sidewall slides made the additional excavation and addition of lean concrete impractical. This option was dropped from the project after construction had started.

Table 2.2. Results of drain centerline soil borings at 15.2-m intervals

Hole No.[a]	Station No.	Surface elevation (m)	Design depth of drain (m)	Depth to auger refusal (m)
B1	7+75	244.4	4.0	7.0
B2	7+25	246.3	7.0	8.5
B3	6+75	247.2	7.8	8.7
B4	6+25	247.5	8.0	11.6
B5	5+75	247.5	10.3	7.2[b]
B6	5+25	246.3	9.1	7.0[b]
B7	4+75	245.0	7.8	6.2[b]
B8	4+25	243.5	6.9	8.5
B9	3+75	242.9	6.1	≥7.6[c]
B10	3+25	242.3	5.7	5.5[b]
B11	2+75	241.4	5.1	4.7[b]
B12	2+25	240.8	4.6	≥5.2[c]
B13	1+75	239.9	3.8	≥4.6[c]
B14	1+25	238.3	3.0	≥3.1[c]

[a]For exact location of the soil borings refer to Fig. 2.1.

[b]Indicates that rock was encountered above the design 1% slope (depth of drain) at this station.

[c]Borehole was terminated when drilling was several meters below the anticipated drain bottom.

2.3.2 Equipment and Materials

Excavation of the drain would require a backhoe with an approximate 0.9-m-(3-ft-)wide bucket and a maximum reach of 9 to 10 m. An American Amhoist Power-Hoe Model Number 680 was rented for the duration of the project. A special narrow bucket was fabricated and fitted to the arm of the backhoe so that the drain could be constructed to a width of 1.0 m. A narrower drain would not be possible with this piece of equipment because the width of the arm is a limiting factor.

The filter fabric used to line the walls of the drain was specified as either Mirafi 140N (Celanese), Bidim No. 22 (Monsanto), or an approved equivalent. Each of these fabrics have openings approximately equivalent to a No. 100 U.S. standard sieve (opening = 0.015 cm). The purpose of the fabric is to exclude sand and silt particles from the drain to avoid plugging and drain failure. Table 2.3 illustrates the size distribution of soil in the vicinity of the drain by showing a particle-size analysis of a sample collected at an open waste disposal trench immediately to the east of the 49-Trench area. This sample is considered to be representative of the material to be encountered in excavating the drain and is seen to contain between 82 and 85 wt % of particles greater than 0.012 cm (0.0049 in.). Plugging of the drain as a result of the passage of silt and clay is not considered likely; however, the additional expense of the fabric was minimal and was deemed worthwhile.

The crushed rock used to backfill the drain was delivered via a portable 12-m conveyor system. The conveyor was used to prevent wall collapse under the weight of the gravel trucks. Gravel was specified as ASTM No. 4 gradation (3.81-cm maximum size) clean stone and was purchased from

Table 2.3. Size distribution of soil samples taken from the 49-Trench area

(Filter fabric opening is ~0.015 cm)

Sieve opening		Percentage of sample held		
(cm)	(in)	Rep 1	Rep 2	Rep 3
0.475	0.187	6.86	9.68	9.65
0.335	0.132	6.25	7.28	7.36
0.200	0.0787	11.39	11.85	11.99
0.084	0.0331	18.27	15.13	16.29
0.025	0.0098	11.46	12.76	21.71
0.012	0.0049	30.20	24.60	14.86
≤0.012	≤0.0049	15.57[a]	18.70[a]	18.14[a]

[a]Calculated by difference.

a local quarry. Table 2.4 summarizes the equipment and materials list used to initiate construction of the drainage system.

2.3.3 Estimated Costs

As part of the detailed engineering design for this project, a cost estimate was made by the ORNL Engineering Division. The estimate was based on rental of the backhoe for a two-month

Table 2.4. Estimate of equipment and materials required to construct the SWSA 6 French drain

Item	Estimated quantity
Materials	
Monitoring wells (9-cm-diam PVC screen)	33
Plastic drain pipe (15 cm)	46 m
Grass seed	585 m^2
Straw	585 m^2
Iron	58 m
Concrete (catch basin)	2
Crushed stone backfill	1262 m^3
Filter Fabric	3950 m^2
Equipment	
Rock conveyor (Morgan, 12-m length)	1
Backhoe (American Model 680)	1
Backhoe (Case Model 980B)	1
Rear dump truck	2
Pump (water)	1
Front end loader (Case Model 508C)	1
Front end loader (International 175 series C)	1
Crane (P and H)	1

period and included placement of lean concrete between sections of unrippable rock (this procedure was later dropped from the project). The estimated construction cost totaled $139,200 and was given a range of plus or minus 10%, or between $125,280 and $153,100.

The possibility of encountering contaminated soil or water in the excavation necessitated the use of the on-site construction contractor (RUST Engineering Co.). After several meetings to discuss the scope of the project, the contractor submitted a bid that was within the range estimated by the Engineering Division. Table 2.5 summarizes the cost estimate and gives a breakdown between materials and estimated labor costs.

Table 2.5 Contractor's cost estimate for construction of the French drain in SWSA 6

Item	Materials cost	Labor cost	Total
Temporary facilities	500	2,110	2,610
Temporary services	*a*	2,355	2,355
General cleanup	*a*	1,825	1,825
Drain pipe	200	400	600
Landscaping	300	955	1,255
Excavation	27,000	16,075	43,075
Backfill	27,600	15,505	43,105
Concrete	5,440	3,020	8,460
Miscellaneous iron	190	715	905
Monitoring wells	*b*	5,925	5,925
Subtotal	61,230	48,885	110,115
Taxes and fringes	3,062	10,266	13,328
Direct cost	64,292	59,151	123,443
Indirect cost	*a*	29,575	29,575
Total	$64,292	$88,726	$153,018

[a]Not applicable.

[b]Monitoring wells were supplied from Environmental Sciences Research and Development Projects.

2.3.4 Schedule

Two weather-related factors were considered in scheduling construction of the French drain. First, the excavation should take place when groundwater is at a seasonal low; second, excavation should take place when the probability of intense rainstorms, which might cause sidewall collapse, is minimal. Both of these criteria were likely to be satisfied by scheduling construction to begin in early September 1983.

Another factor, unrelated to weather, that had an effect on the construction schedule was the proximity of the drain to a 161-kV Tennessee Valley Authority (TVA) power line. (Location of TVA power lines are shown in Fig. 2.1.) To operate the large backhoe in the vicinity of these lines, the equipment would have to be grounded at all times and the lines would have to be deenergized at the beginning of each week. Arrangements for deenergizing the lines were coordinated with TVA, and no additional problems were anticipated.

2.3.5 Design Unknowns

The major unknowns in the design of the French drain were the ability of the sidewalls to stand vertically during the backfilling operations and the depth to unrippable rock. The calculation of depth to rock was discussed previously (Sect. 2.3.1) and determined the ultimate depth of the drain at station 6+00. This concern could have been overcome by blasting the rock before excavation, but that option was not considered necessary and was not included in the design. Instead, excavation began and final decisions as to the depth of the drain were to be made as soon as the hardness of the rock could be determined.

The ability of the sidewalls to stand vertically was particularly important from the standpoint of how much excavated material would be removed and how much gravel backfill would be required. Evidence for trench walls standing vertically for extended periods of time is abundant in the waste disposal trenches in SWSA 6; however, these trenches are normally only 4.6 m (15 ft) deep and 15.2 m (50 ft) long and oriented north-south. Even at these dimensions, some instances of sidewall collapse has been noted. To minimize the possibility of trench-wall collapse, the design recommended that the gravel backfill be kept as close as possible to the working face of the drain, thus giving added support to the walls. If collapse could not be prevented, the necessary additional excavation was expected to cause only minimal project delay.

2.4 DESIGN SUMMARY

In summary, the French drain in SWSA 6 is intended to intercept shallow groundwater from upgradient recharge areas before it has a chance to enter the closed disposal trenches of the 49-Trench area. As designed, it consists of a 1.0-m-wide ditch, excavated to a maximum depth of approximately 10 m and backfilled with crushed stone to facilitate drainage. The total length of the drain is 252 m. Water entering the drain is diverted either to the west or south of the site into two small receiving streams that drain southward to White Oak Lake. The estimated cost of the project ranges from $139,200 to $153,100.

In addition to diverting water during rainfall events, the drain will permanently supress the groundwater table in its area of influence (the region around the drain). This will have a beneficial effect because water in the region will not be allowed to rise to a level that will intersect the bottom of the adjacent trenches. It is anticipated that by isolating the 49-Trench area from groundwater recharge, a gradual decline in the area groundwater levels will result and eventually the water trapped in the disposal trenches will dissipate.

3. CONSTRUCTION OF THE DRAINAGE SYSTEM

3.1 CONSTRUCTION SCHEDULE

Construction of the French drain was initiated on August 29, 1983, with the excavation of the first 18 linear meters (59 ft) of drain and mobilization of construction equipment and project personnel at the site. Final excavation and placement of crushed stone and backfill were completed on September 29 and 30, respectively. Construction of the two catch basins at the south and west ends of the drains, seeding and mulching of graded spoil areas, and site cleanup continued intermittently until final completion on October 28.

Daily excavation progress during the 17-d construction of the French drain is shown in Table 3.1. Of the total time, it is estimated that about five days (or 29% of the time) were spent excavating sidewall material that slid into the drain during construction. Essentially all of this five days was spent working on the north section of the drain and in the northern most 18 m of the east section, where slides were most prevalent. Figure 3.1 is a plan of the drain layout showing locations of the sidewall slides.

Table 3.1. Daily excavation progress

Date 1983	Location by station (from—to)	Linear meters excavated	Range of depth (m)	Remarks
8-29	0+00—0+59	18.0	0—1.5	
8-30	0+59—1+90	39.9	1.5—4.3	
8-31	1+90—2+75	25.9	4.3—4.9	Hard rock left 0.45 m above grade at station 2+40
9-6	8+25—7+85	12.2	0—3.9	
9-7[a]	7+85—7+65	7.6	3.9—4.9	
9-13[a]	7+65—7+45	7.6	4.9—5.5	Slide from station 7+77 to station 7+50 south wall.
9-14	7+45—7+35	3.0	5.9—7.6	Started excavating deeper toward hard rock and slides from station 7+52 to 7+40 north wall and station 7+50 to 7+40 south wall.
9-14	2+75—3+10	10.7	4.3—5.8	Also increased depth of excavation between station 2+45 and 2+75 by up to 0.6 m
9-15	3+10			Excavated slide material at station 3+00 west wall
9-15	7+35—7+20	4.5	5.5—6.4	Slides from station 7+40 to 7+20 north wall and station 7+40 to 7+20 south wall
9-16	7+20—6+85	10.7	6.4—7.6	
9-19	6+85—6+35	15.2	7.6	Slide from station 6+77 to 6+60 north wall

Table 3.1 (continued)

Date 83	Location by station (From—to)	Linear meters excavated	Range of depth (m)	Remarks
9-20	6+35—6+05	7.6	7.6—7.9	Slides from station 6+43 to 6+05 north wall and from station 6+50 to 6+05 south wall
9-21	6+05—6+00	1.5	7.9	Slides in same area as on 9-20, but deeper
9-22	3+10—3+72	18.9	5.8—6.4	Slides from station 3+10 to 3+40 west wall
9-23	3+72—4+19	14.3	6.4—7.0	
9-26	4+19—4+80	18.6	7.0—8.4	Slide from station 4+20 to 4+30 west wall
9-27	4+80—5+55	22.9	8.4—7.8	Hard rock left above 1% grade from about station 4+90 to 6+00
9-28	5+55—5+85	9.1	7.8—7.6	Slides from station 5+40 to 5+80 east wall
9-29	5+85—6+00	4.6	7.6	Slides from station 5+60 to 6+00 west wall

[a]Indicates excavation carried on for one-half day only.

3.2 CONSTRUCTION METHODS

3.2.1 Excavation

Excavation of the drain was accomplished using two different backhoes. A Case Model 980B backhoe (Fig. 3.2) was used to excavate from the drain-exit level of the east section (station 0+00) to station 2+75 and from the drain-exit level on the north section (station 8+25.66) to station 7+65. This backhoe is part of the ORNL Operations Division equipment maintained at SWSA 6 and was used before the arrival and assembly of the larger American Model 680 Power-Hoe (Fig. 3.3). The Case 908B has a depth of excavation limit of approximately 4.9 m (16 ft), and its use was discontinued where depths greater than this limit were required (specifically, between stations 2+75 and 6+00, and 7+65 and 6+00). Even with this depth limitation, 102 linear meters (335 linear feet), representing 40% of the total length (252 m) of the drain, were excavated using the Case 908B backhoe.

The American Model 680 backhoe arrived on site in five main pieces on September 7. Assembly was completed by noon September 13, at which time the American took over the excavation at station 7+65. Thereafter, all excavation was done by this larger piece of equipment. Figure 3.4 is a copy of the manufacturer's diagram that shows working depths of the American Model 680 and indicates 10.7 m (35 ft) as its maximum depth limit, using a standard arm.

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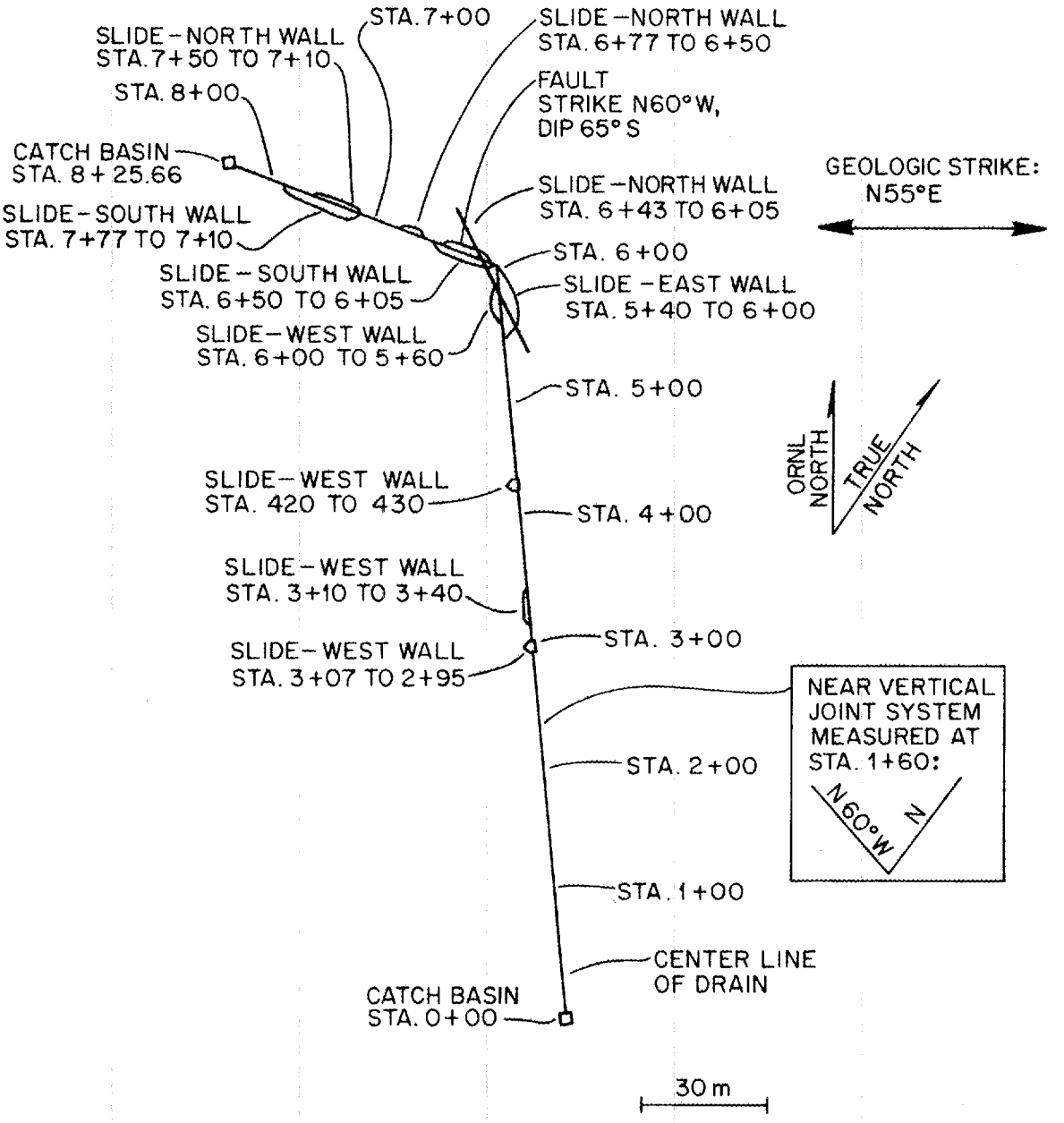

Fig. 3.1. Plan view of the drain layout showing location of the sidewall slides.

Except for the portions of the drain on the steep slope at the west end of the northern section (station 8+25 to 7+65), both backhoes loaded the excavated material directly into rear-dump, over-the-road trucks (Fig. 3.3). Because the haul distance to the three spoil areas used for this project was less than about 150 m from the excavation, only two haul units were required. Material excavated by the Case 980B backhoe from station 8+25 to 7+65 was temporarily stockpiled to the south side of the drain and later carried to and loaded on the trucks by an International Model 175

ORNL-PHOTO 7993-83

Fig. 3.2. Case model 908B backhoe used to excavate the shallow south and west ends of the drain.

ORNL-PHOTO 7998-83

Fig. 3.3. American model 680 Power-Hoe at approximate station 4+10 loading truck at approximate station 4+60.

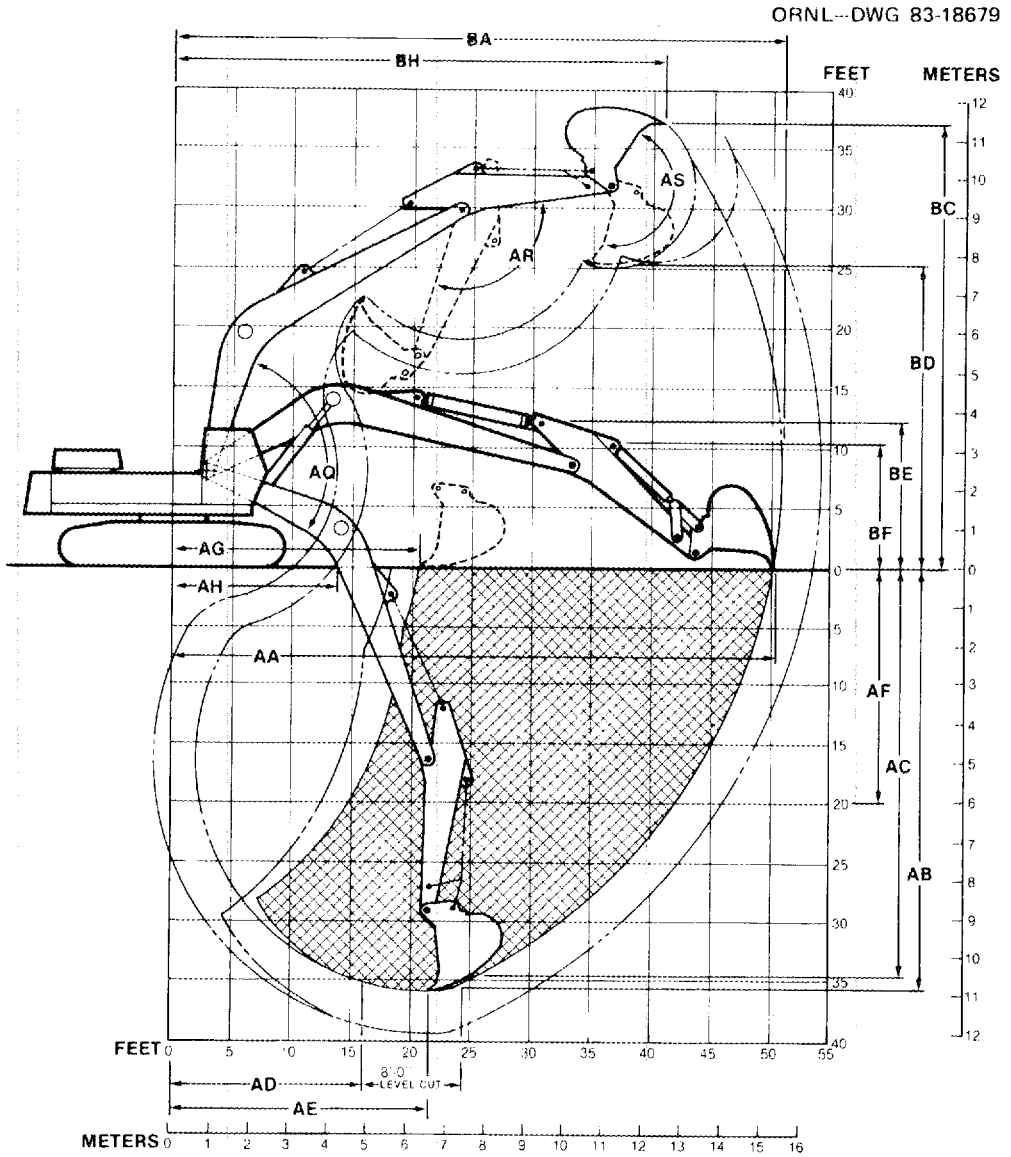

Fig. 3.4. Manufacturer's specifications for the American model 680 Power-Hoe.

front-end loader, as shown in Fig. 3.5. The spoil piles were located in relatively low swales on the hillside and were contoured with the International Model 175 front-end loader to make their locations unnoticable after a grass cover is established.

The basis for grade control of the excavation was a series of cut stakes placed by the surveying crew along the drain centerline at 7.6-m (25-ft) intervals. A grade string was then stretched at a selected height between two iron rods driven at two adjacent grade stakes parallel to the drain centerline. At depths up to 5.5 m, grade was checked using a pole attached at a 90° angle to a second board that was long enough to reach the string line from the center of the drain. The horizontal board was fitted with a carpenter's level to ensure that depth readings were taken from a level datum (Fig. 3.6). In practice, grade was checked at intervals along the drain by placing the lower end of the pole on the bottom of the drain and noting the height of the level board above or below

ORNL-PHOTO 7997-83

Fig. 3.5. International model 175 front-end-loader loading stockpiled excavated material from station 8+25 through 7+80.

the string line. At depths greater than 5.5 m, the apparatus became too difficult to handle and the pole was replaced with a weighted tape set to the grade depth.

At station 2+40, rock that could not be excavated using the Case 980B backhoe was encountered. This rock extended 0.3 to 0.45 m above the 1% grade that originated at station 0+00 (elevation 235.37 m above mean sea level). This estimate of the height of the rock agrees well with an estimate calculated on the basis of photographs taken of the east wall of the drain at station 2+40 and becomes important in the interpretation of water levels observed in this portion of the drain.

When excavation at station 2+75 was resumed using the larger backhoe, experience in the north section of the drain already had shown the high probability that at least some portion of the drain wall would collapse into the excavation when depths exceeded 4.9 m and the drain was left open. Placement of concrete at a 1% grade, starting upslope of the high rock at station 2+40, would require that extended lengths of the drain remain open for the concrete to be poured and harden. Because the probability of a slide was high, excavation below the 1% grade and placement of concrete was dropped from the project and the 0.37-m rock summit at station 2+40 was left undisturbed. When the American backhoe was positioned at station 2+75, it was able to excavate material from station 2+55 through 2+75 that had previously been unrippable using the Case 908B.

Northward from approximately station 4+90, unrippable rock again appeared above the 1% grade line and climbed abruptly until, at station 5+04, the bottom of the drain had risen over 2 m

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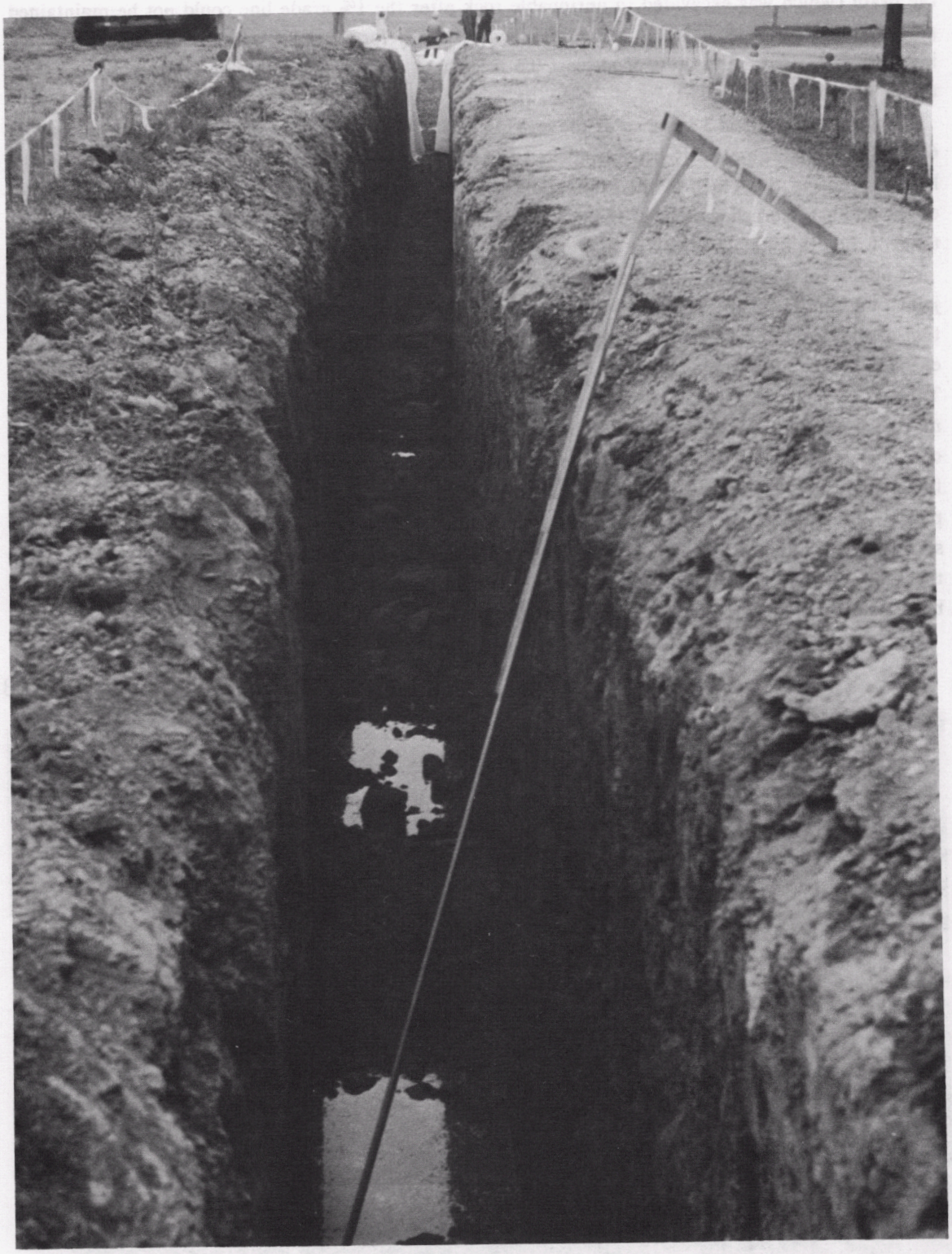

Fig. 3.6. Grade-checking device.

in elevation (2 m vertically for 4.3 m horizontally). At this point, the rate of rise of the bottom of the drain (which was excavated to unrippable rock after the 1% grade line could not be maintained past station 4+90) decreased to a rise of 1.2 m vertically for 22 m horizontally. Within this reach, it is estimated that a thickness of at least 0.6 m of hard, thin-bedded limestone was excavated from the bottom of the drain.

Although the upper 3 m or so of the east section of the drain was excavated right up to station 6+00 (which had previously been excavated to full depth in the installation of the north drain), the lower portion of this reach of the drain could only be excavated to station 5+80. This was because of the sliding of crushed stone that had already been placed in the north drain. Thus, the two sections of the drain do not fully join at station 6+00 except for the top 3 m.

Collapse of the drain walls caused by slides or fall-outs along discontinuities (bedding planes, joints, and faults) in the intensely weathered rock mass were almost continuous along the north section of the drain and in the northern most 18.3 m of the east section. Only occasionally were slides encountered in the remaining reach of the east section (refer to Fig. 3.1 and Remarks column of Table 3.2). However, slides and fallouts only occurred when excavation depths exceeded about 4.9 m. Slides were one of the potential problems mentioned in Sect. 2.3.5 and were not preceded by any cracks or movement of the surface in the area adjacent to the excavation. When slides occurred, the fallen material was removed by the backhoe until the sides stabilized and crushed rock could be placed to help shore up the sidewalls. On two occasions when the American backhoe was sitting at station 6+00, the drain had to be backfilled and a ramp excavated so that the machine could reach the slide material (Fig. 3.7). In these cases, the slide had torn away filter fabric that had already been installed along the side of the drain. Slides are further discussed in Sect. 3.3.3.

ORNL-PHOTO 8010-83

Fig. 3.7. Ramp constructed to excavate slide material between stations 6+50 and 6+00.

3.2.2 Installation of Plastic Pipe Between Crushed Stone Drain and Catch Basins

When excavation reached station 1+90 on August 31, a 15.2-cm-(6-in.-)diam, solid plastic corrugated pipe was placed in the bottom of the drain from beyond station 0+00 to station 0+51 and this portion of the drain was backfilled with clay as shown in Fig. 3.8. Filter fabric from 3.7-m-(12-ft-)wide rolls was draped against both sides of the drain and temporarily staked to the ground surface at the top of the drain. A piece of filter fabric having a circular opening cut at the bottom for the pipe was placed against the clay backfill at station 0+50. Thus, the interface between the drain and pipe was complete.

Placing the end of the plastic pipe through the opening in the filter fabric presented no problem because it was performed by personnel in the bottom of the drain. At this point, the drain was only 1.22 m (4 ft) deep and safety considerations did not prevent personnel from entering. The first of a series of 7.6-cm-(3-in.-)diam PVC monitoring wells was then set vertically on the bottom of the drain at station 0+54 and held in position from the ground surface while crushed stone was dumped around it from the top of the drain, using the Case 508 C front-end loader (Fig.3.9).

The method of drain pipe installation from crushed-stone drain to catch basin as described previously was repeated at the west end of the north section of drain. Here, the plastic pipe was installed from catch basin No. 1 at station 8+25.66 to the interface having the crushed-stone drain at station 7+85. At this point, the depth of the drain was approximately 4.2 m and safety considerations prohibited personnel entering the excavation. As a result, the plastic pipe protruding from the clay backfill at station 7+85 (Fig. 3.10) was manipulated through the hole in the filter fabric by the use of poles (Fig. 3.11).

3.2.3 Placement of the Filter Fabric, Crushed Stone, and Monitoring Wells

In portions of the drain where no slides occurred and the walls of the excavation remained vertical, placement of the filter fabric (Marifi 140N) was easily accomplished (Fig. 3.12). Two persons, one at each end of the 3.7-m-long roll of fabric, would unroll the required length directly into the drain. The fabric would then be cut from the roll and staked at the ground surface, using three to four short steel stakes. The next piece to be installed was placed to overlap the previous piece by about 0.61 m. Initally, the last piece installed overlapped that previously installed; however, with this type of overlap, the crushed stone tended to run between the last installed piece and the wall unless the overlap was held in place by poles (Fig. 3.13). To avoid this, the overlapping procedure was reversed: the last installed piece was placed under the fabric previously installed.

In areas of the drain where slides occurred, the placement of filter fabric was more difficult because of the irregular form and slope of the walls. The fabric would sometimes have to be unrolled and thrown out over the slopes when they were not sufficiently steep or smooth for the fabric to slide down the sides. Even when the fabric was overlapped by more than 1 m at the surface, gaps between adjacent pieces of fabric were sometimes impossible to close in the lower portions of the drain (Fig. 3.14). These gaps probably will not have an adverse effect on the permeability of the drain because the exposed mudstones and shale, although weathered, appear firm enough that the smaller particles probably will not migrate.

Crushed limestone [No. 4 (3.8-cm maximum size)] for the drain material was trucked to the site and stockpiled in rows parallel and adjacent to the drain (Fig. 3.15). Initially, in the shallow

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Fig. 3.8. Clay backfilling over plastic pipe between stations 0+00 and 0+51.

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Fig. 3.9. Case 580 C backfilling crushed stone around the monitoring well at station 0+54.

portion of the drain (less than 3 m deep) where there was little danger of slides, the crushed stone was transported from the stockpiles and dumped directly into the lined drain, using a Case Model 508C front-end loader (Fig. 3.9). For most of the length of the drain in which the danger of sidewall slides existed, the front-end loaders dumped the stone into the hopper of the Morgan Conveyor, which in turn placed the stone in the desired portion of the drain (Fig. 3.16).

The amount of crushed stone needed to fill the drain was greatly increased in the areas where slides occurred (Fig. 3.17). To reduce slides, it is advantageous to keep the placement of the crushed stone as close to the working face of the excavation as possible, but there are several limitations to accomplishing this. The angle of repose of the crushed stone used was 36°. This means, for example, that in a depth of 7.6 m (25 ft), the top of the sloped stone will be about 10.7 m (35 ft) away from the working face. In addition, the filter fabric must be installed ahead of the stone, which means an additional space of 3 m of open excavation is required ahead of the toe of the crushed stone. Further, to excavate a 6- to 7-m trench, the backhoe bucket and arm requires a minimum open distance (ahead of the end of the filter fabric) of about 6 m. These dimensions add up to a horizontal distance of approximately 20 m from the top of the crushed-stone fill to the working face in a 7.6-m-deep trench. Thus, in an excavation of this sort, it was impossible to keep the stone close enough to the working face to prevent slides.

A total of 28 monitoring wells were installed in the drain, and their locations, depths, and elevations are listed in Table 3.2 and shown in profile in Figs. 3.18 and 3.19. The 7.62-cm-diam PVC wells that have a 0.61-m-long section of PVC screen at the bottom were made up on-site; depths were scaled from the engineering plans. An additional 1.5 m was added to each well so the end pro-

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Fig. 3.10. Plastic drainage pipe protruding from clay backfill into drain excavation at station 7+85.

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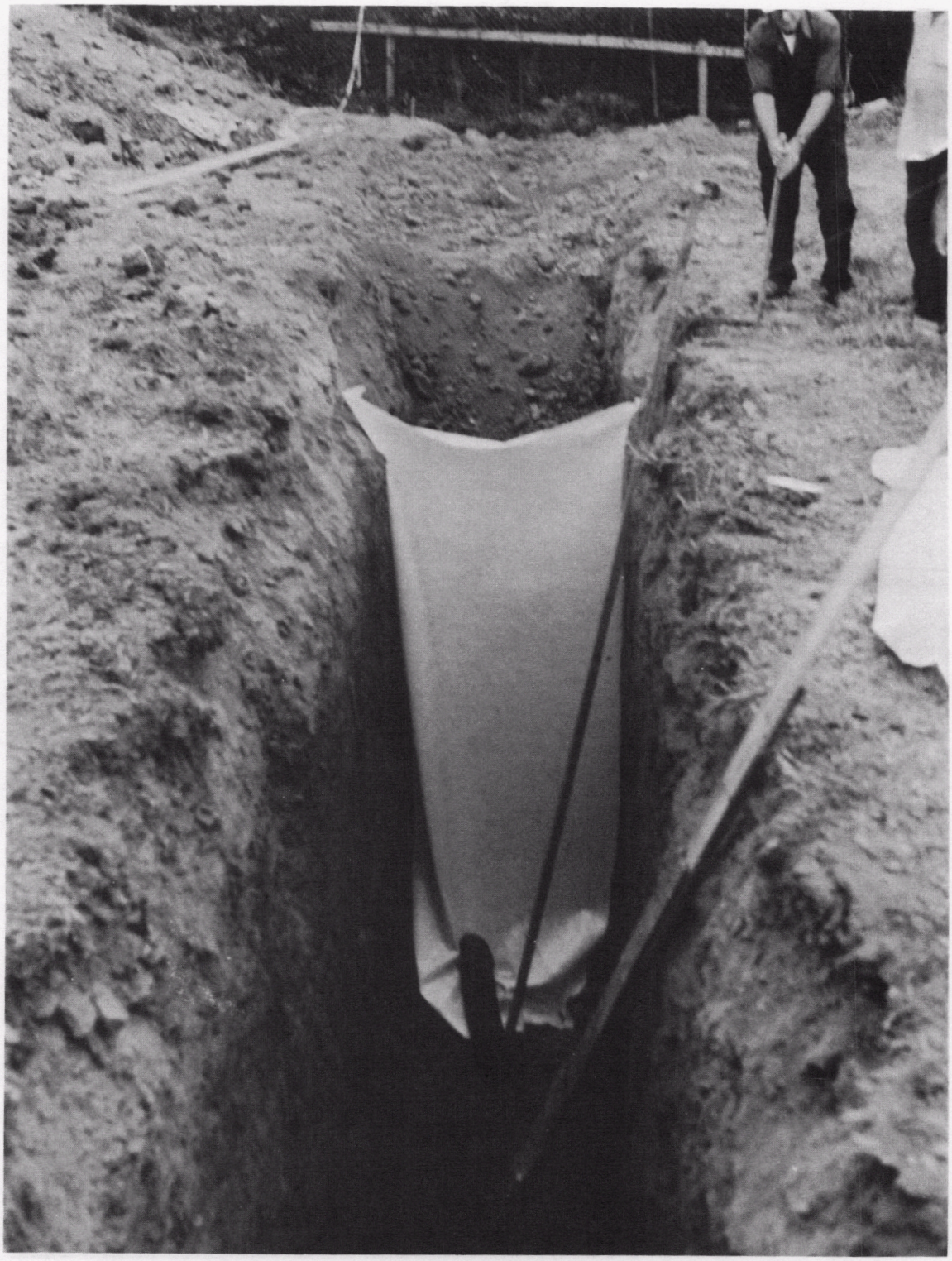

Fig. 3.11. Placing plastic drainage pipe through filter fabric at station 7+85 using poles.

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Fig. 3.12. Installed filter fabric on drain sidewalls up to approximate station 5+30.

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Fig. 3.13. Overlapped joints of filter fabric being held closed with poles during placement of crushed stone at approximate station 1+35.

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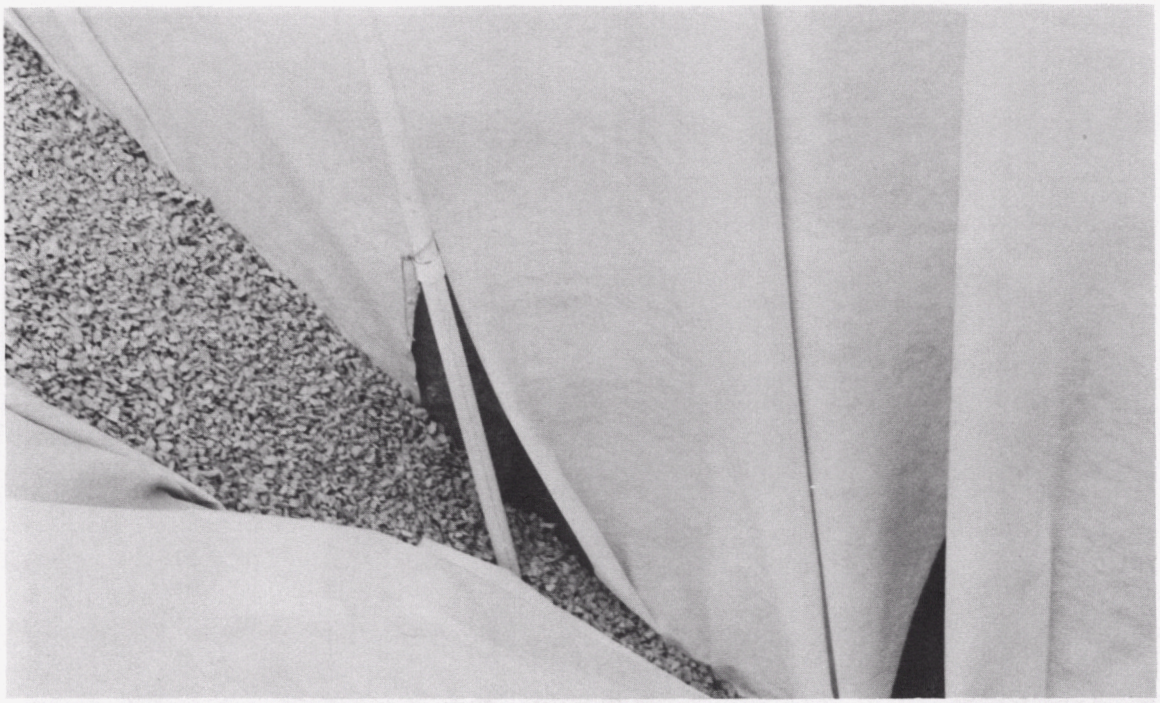

Fig. 3.14. Gaps in filter fabric overlap that occurred in slide areas. Monitoring well seen in center of photo is at approximate station 6+38.

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Fig. 3.15. Stockpile of crushed stone backfill material parallel to drain excavation.

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Fig. 3.16. Morgan conveyor placing crushed stone into the drain.

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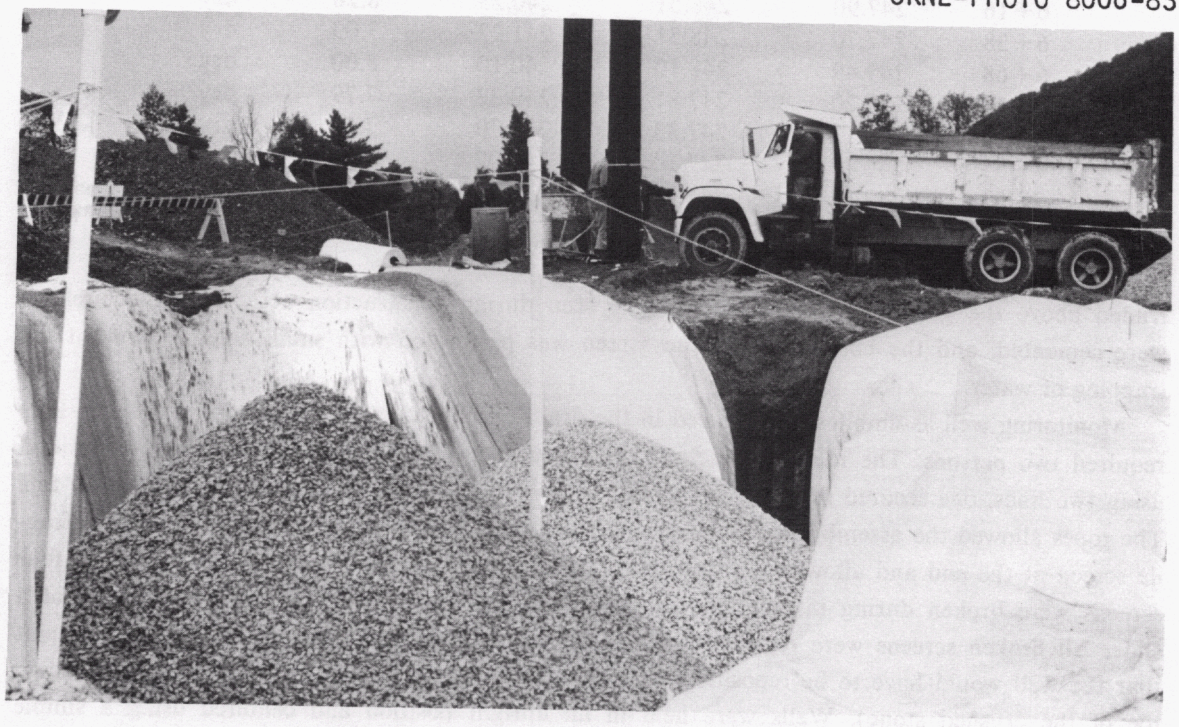

Fig. 3.17. Crushed stone being placed in a slide area (station 6+40 to 6+00).

Table 3.2. Drain monitoring wells-locations, depths, and elevations

Well station	Elevation of ground (m)	Elevation of top of casing (m)	Elevation of bottom of well (m)	Depth of well (m)	Water level 10/18/83 (m)
0+54	236.70	237.31	235.46	1.85	235.48
1+01	238.06	238.67	235.58	3.09	235.70
1+33	238.91	239.52	235.80	3.72	dry
1+53	239.34	239.95	235.65	4.30	235.78
1+85	239.98	240.59	235.80	4.79	235.83
2+02	240.21	240.82	235.73	5.09	235.94
2+26	240.57	241.18	235.91	5.27	236.11
2+51	240.96	241.57	235.87	5.70	236.79
2+85	241.48	242.09	236.13	5.96	236.81
3+03	241.75	242.36	236.16	6.20	236.82
3+25	242.02	242.63	235.57	7.06	236.83
3+52	242.36	242.97	235.70	7.27	236.82
3+76	242.75	243.36	236.26	7.10	236.78
4+01	243.14	243.75	235.79	7.96	236.78
4+27	243.70	244.31	236.56	7.75	236.79
4+58	244.32	244.93	236.35	8.58	236.79
4+77	244.47	245.63	236.62	9.01	236.78
5+04	245.80	246.41	238.56	7.85	dry
5+27	246.48	247.09	238.71	8.38	238.84
5+50	246.98	247.59	239.34	8.25	dry
5+77	247.53	248.14	239.79	8.35	dry
6+16	247.90	248.51	240.23	8.28	dry
6+38	247.70	248.31	241.22	7.09	dry
6+68	247.49	248.10	240.10	8.00	dry
6+94	247.20	247.81	240.32	7.79	dry
7+19	246.72	247.33	239.70	7.63	dry
7+35	246.49	247.10	240.02	7.08	dry
7+81	243.88	244.49	239.47	5.02	dry

truded above the ground surface and could be seen during construction activities. All couplings were cemented, and the bottom cap of the screen was perforated with small holes to prevent the trapping of water.

Monitoring well assemblies were placed in the drain by hand; the placement of wells >1 m long required two persons. The most satisfactory method was to lower the well assembly into position using two lines, one secured in about the middle of the well and the other about 2 m from the top. The ropes allowed the assembly to be lowered to the bottom of the drain without breaking the fragile screen at the end and allowed the well to be raised if necessary to reposition it. A total of four screens were broken during installation, and an additional screen was destroyed as a result of a slide. All broken screens were replaced before final installation. A broken screen generally meant that the well would have to be repositioned a few meters upslope (to be in front of the advancing toe of the crushed stone). Wells were held in an upright position and centered using a simple wooden template laid across the drain (Fig. 3.13) or, in slide areas where the top of the drain was too wide for the template, by attached lines (Fig. 3.17).

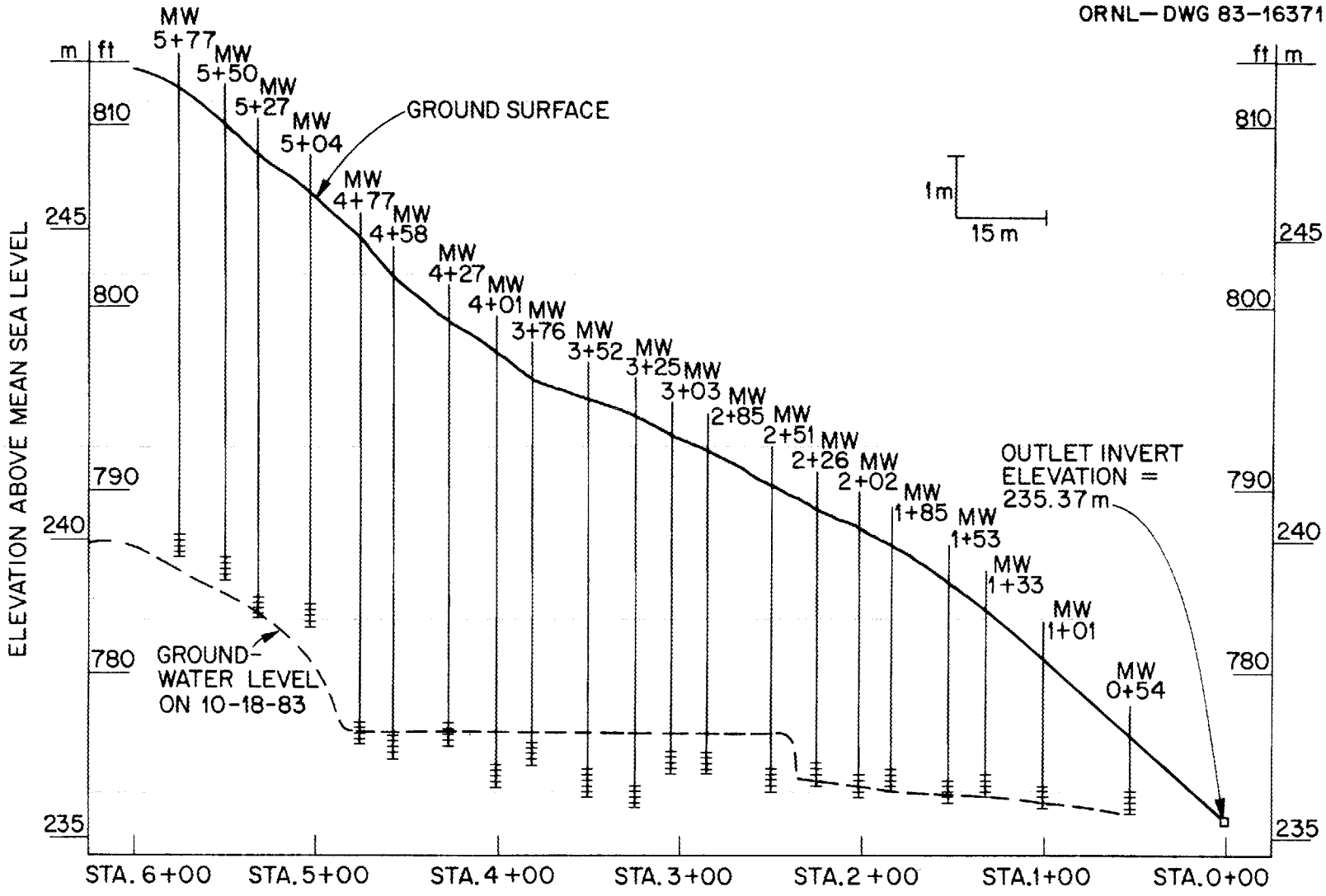

Fig. 3.18. Profile showing depths and locations of monitoring wells installed in the eastern section of the drain. Water levels above station 4+77 are based on information from a single well at station 5+27.

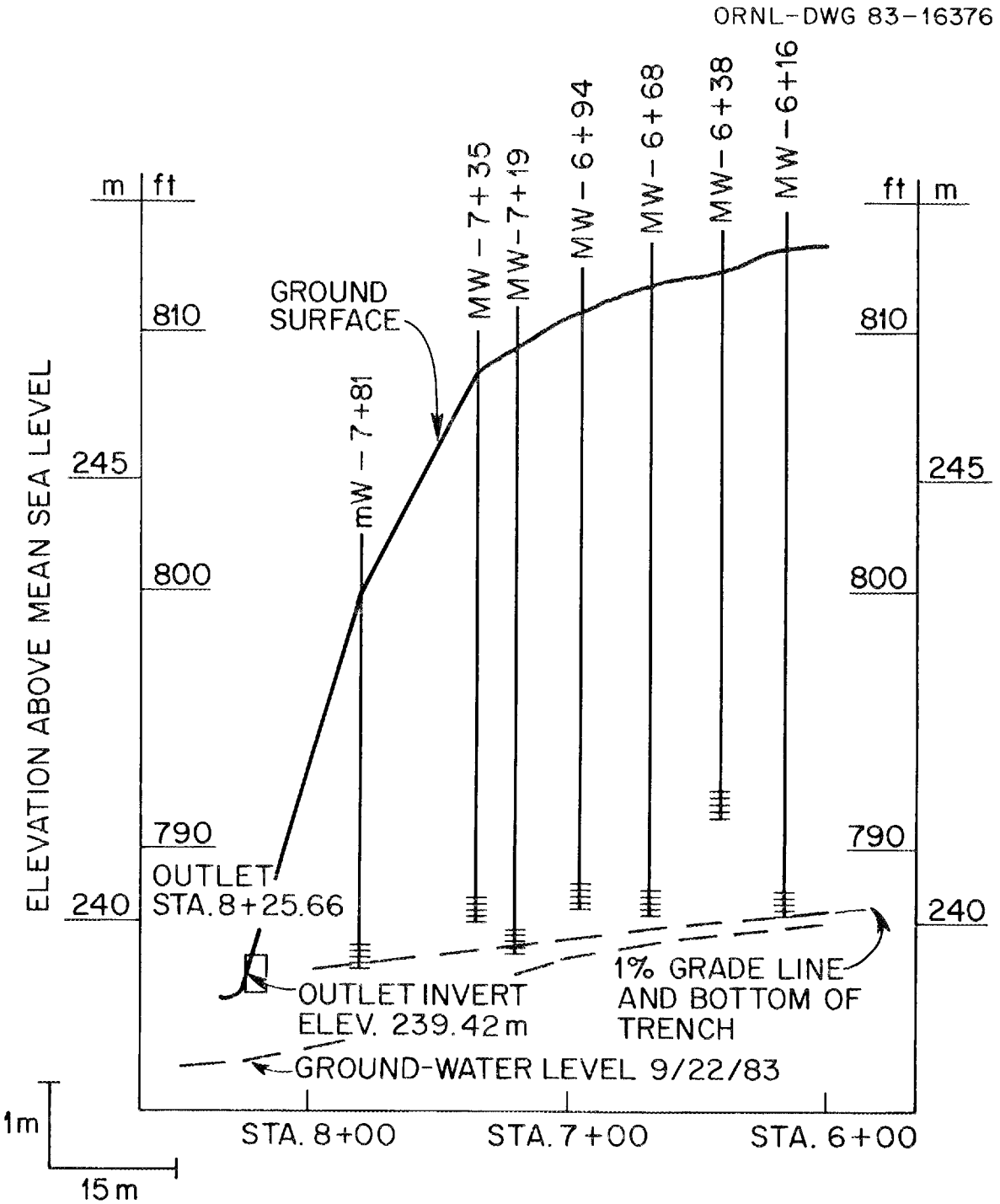

Fig. 3.19. Profile showing depths and locations of monitoring wells installed in the northern section of the drain. The groundwater level measured on September 22, 1983, was taken from existing wells within the 49-Trench area along with observations made during excavation of catch basin number 1.

Where slides occurred, the crushed stone was kept as close to the active face as possible. This procedure prevented some of the monitoring wells from being set on the bottom of the drain as planned because of the proximity to the backhoe. This accounts for the fact that wells at station 7+35, 6+94, and 6+68 were set 0.2 to 0.5 m above the bottom of the drain (Fig. 3.19). The original well at station 6+38 was destroyed by a slide after a depth of 1 m of stone had been placed; as a result, the well at this station is above the bottom of the drain.

A monitoring well was originally planned for the intersection of the two drains (station 6+00), but could not be installed at that position. The trench at station 6+00 was first excavated as the east end of the north section of drain, and the filter fabric and the stone were placed at that time. A well was not installed at this time because the American backhoe would again traverse this station when the northern part of the east section of drain was excavated. As previously discussed, the northern part of the east drain could not be excavated to full depth because of slides and the crushed stone from the northern drain falling into the excavation; therefore, no well was placed at station 6+00.

During well installation, placement of the stone caused the PVC monitoring wells to be deflected downslope of the stone. Although not measured, by visual observation the deflection appeared to reach about 0.9 m horizontally in a 9-m-long well assembly. Such a deflection could cause the vertical span of the well to be slightly less (4.56 cm) than the length (stated depth) of the well. Depending on the amount of deflection, the true elevation of groundwater in these inclined well casings could be slightly higher than that calculated by measuring from the top of the well casing to the water surface. This slight error in measured water elevations is common in nonplumb wells.

As reaches of the drain were filled to within 0.6 m of the ground surface, the ends of the filter fabric were folded over the stone backfill and overlapped. The drain was then covered with excavated material to the original ground elevation (Fig. 3.20).

3.2.4 Project-Specific Safety Requirements

As shown in Fig. 2.1, a 161,000 V (161 kV) overhead electric transmission line runs immediately adjacent to the eastern section of the drain (station 0+00 to 6+00). The height of this line is such that contact with the backhoe equipment was possible. Arrangements were made with TVA to deenergize the line on Monday through Friday of each week, while excavation was in progress. Also during the deenergizing period, TVA grounded the line at the towers at either end of the project. The grounding was discontinued at the end of shift on Friday and was started each Monday morning. As an additional precaution, copper-clad steel grounding rods were driven into the ground at spaced intervals along the east section of the trench and the backhoe and conveyor were attached to one of the grounds at all times.

No personnel were allowed into the drain after it reached a depth of about 2 m. After the drain reached a depth of 4 m, all personnel approaching the edge of the open excavation were required to wear safety belts and tie off to steel stakes. These stakes were driven into the ground a distance from the edge of the drain equal to the depth of the excavation.

Because the drain was close to waste disposal trenches some concern arose over the radionuclide content of the soil and water being excavated. To prevent possible exposure of personnel or contamination of equipment, ORNL health physics personnel monitored all equipment leaving the site. No radioactive contamination was found. As mentioned in Sect. 4.6, even drainage from the eastern

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Fig. 3.20. Covering crushed stone with filter fabric and excavated material.

section of the drain was free from ^{90}Sr, ^{137}Cs, and ^{60}Co, three isotopes that are frequently present at contaminated seeps.

Numerous monitoring wells existed in the 49-Trench area near both the north and east sections of the drain. To prevent damage to these wells resulting from construction traffic, each was marked with colored flags tied on iron rods that protruded 1.5 m above the ground surface.

In addition to the usual safety equipment brought to the site by the contractor, an ORNL security guard was stationed at the entrance to the site to check all personnel entering and leaving. In the event of an accident, the guard could radio for emergency fire or medical assistance from the main plant less than 3 km away.

3.3 GEOLOGIC CONDITIONS ENCOUNTERED

3.3.1 General Geology

The construction site is located in Melton Valley approximately 1.5 km southeast of the southeast dipping Copper Creek Thrust Fault and is within the Valley and Ridge Province. Locally, the strike of the Copper Creek Fault is N55°E, as measured along a lineament of topographic highs on Haw Ridge, the topographic expression of the Rome Formation that lies immediately above the thrust fault. Characterization of a nearby hillock (Davis et al. 1983) indicates that the drain is constructed in the Maryville Limestone Formation of the Conasauga Group. The Conasauga overlies the Rome Formation.

3.3.2 Soils

The soil mantle along the trench alignment was very thin, generally less than 30 cm to nonexistant, except for the areas at both ends of the drain where the two catch basins are located. From visual observations, the soils are classified as lean and sandy clays, according to the Unified Soils Classification. The excavation from station 0+00 to approximately 0+30 and from station 8+25 to approximately 8+10 was entirely within these soil types. For the remainder of the drain the soil thickness, where present, amounted to only a thin mantle over the intensely weathered rock strata. The chemical and physical properties of the soils have been described previously (Davis et al. 1983), and the soils are placed in the Montevello series according to the classification system of the U.S. Soil Conservation Service.

3.3.3 Consolidated Strata

In the 49-Trench area, the Maryville Formation is intensely weathered to depths of up to 7.6 m and, occasionally, to even greater depths. The calcium carbonate originally present in the nonweathered strata has apparently been leached out of the intensely weathered rock encountered at the site. The lithology of most of the material encountered can best be described as very-thin-to-thin bedded mudstones and shales, which now contain very closely spaced (to 1 cm) fractures. However, hard, thin-bedded, very fine-grain limestone (Fig. 3.21) was found at the bottom of the northern end of the eastern drain.

Structural relicts were encountered in the east and north sections of the drain, as shown in profiles of the east and north walls, respectively (Figs. 3.22 through 3.25). These figures show fold

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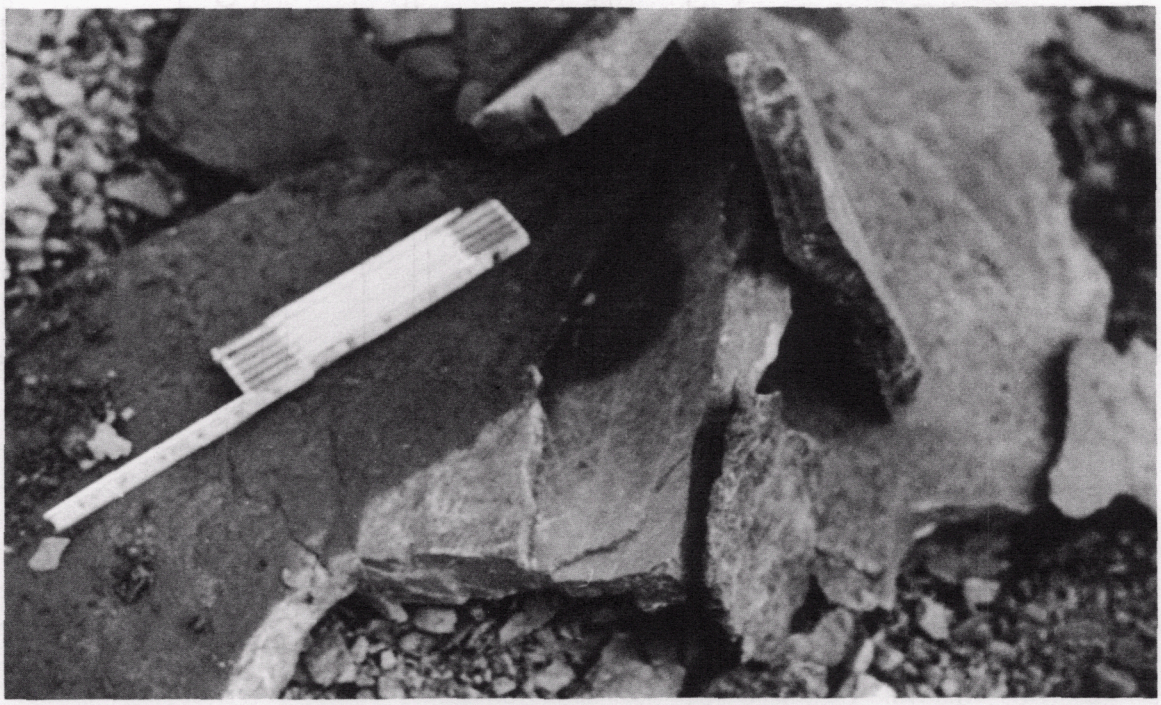

Fig. 3.21. Photo of pieces of limestone excavated from the bottom of the drain at approximate station 5+75.

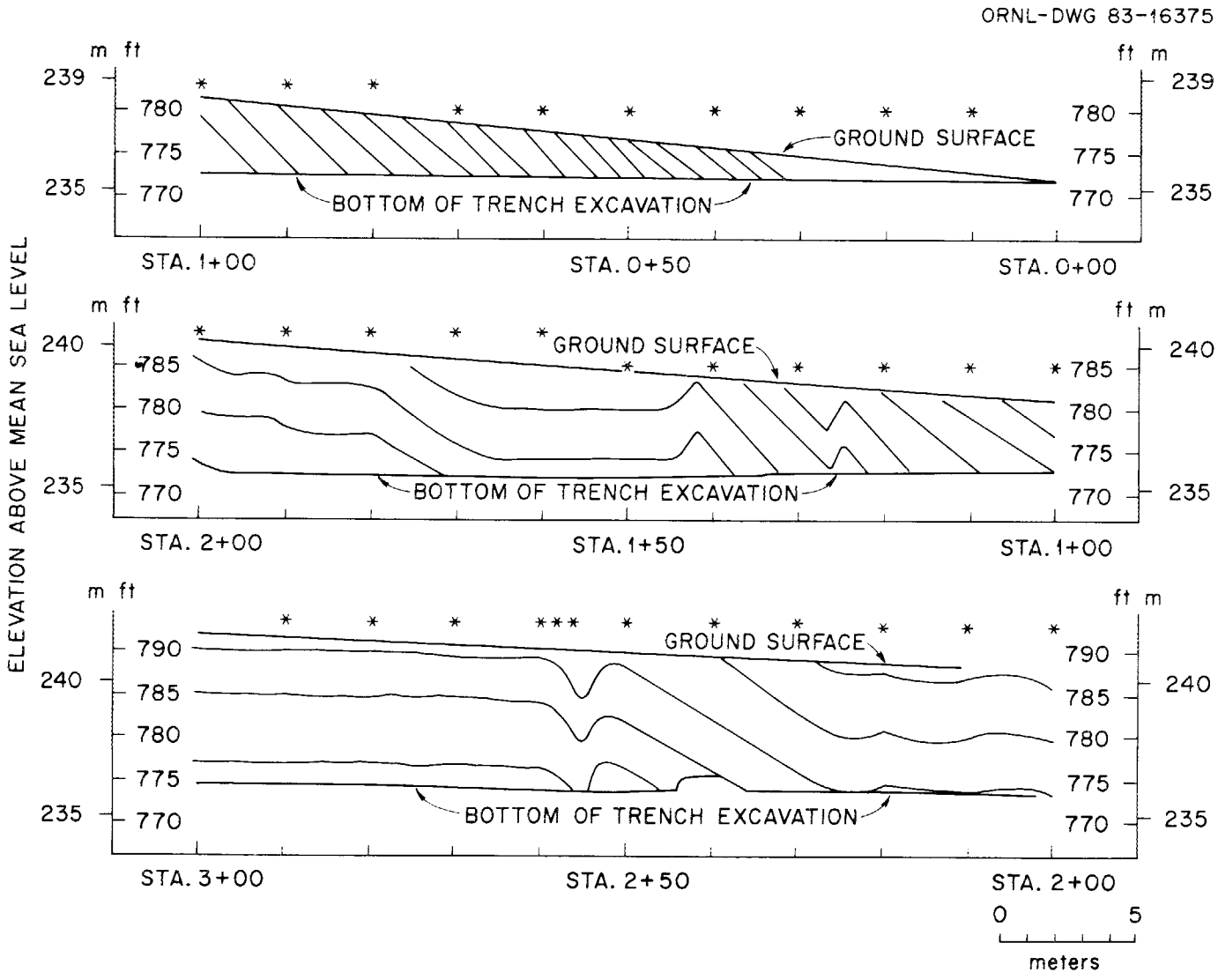

Fig. 3.22. Geologic structure profiles of drain excavation—east wall (station 0+00 to 3+00).

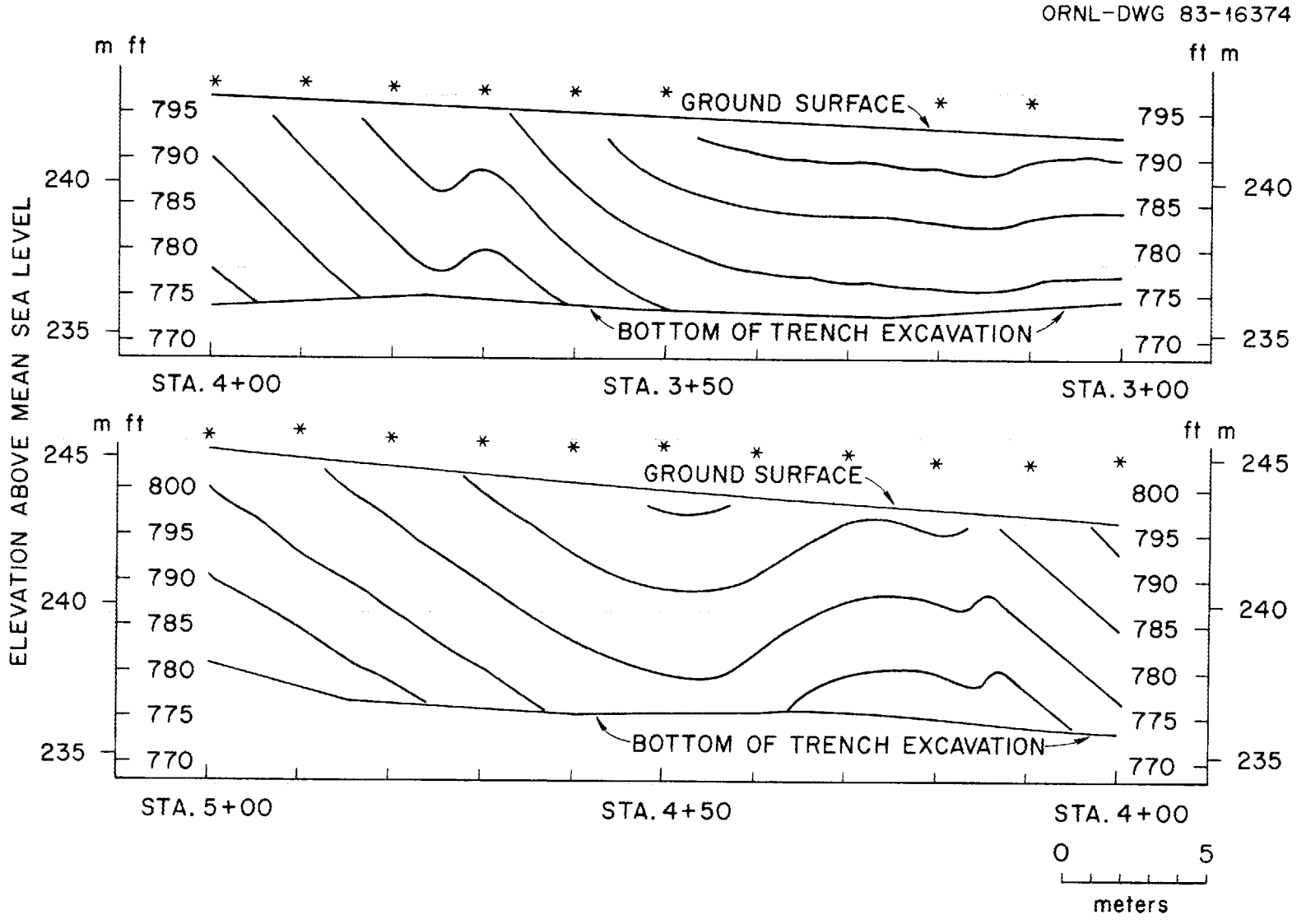

Fig. 3.23. Geologic structure profiles of drain excavation—east wall (station 3+00 to 5+00).

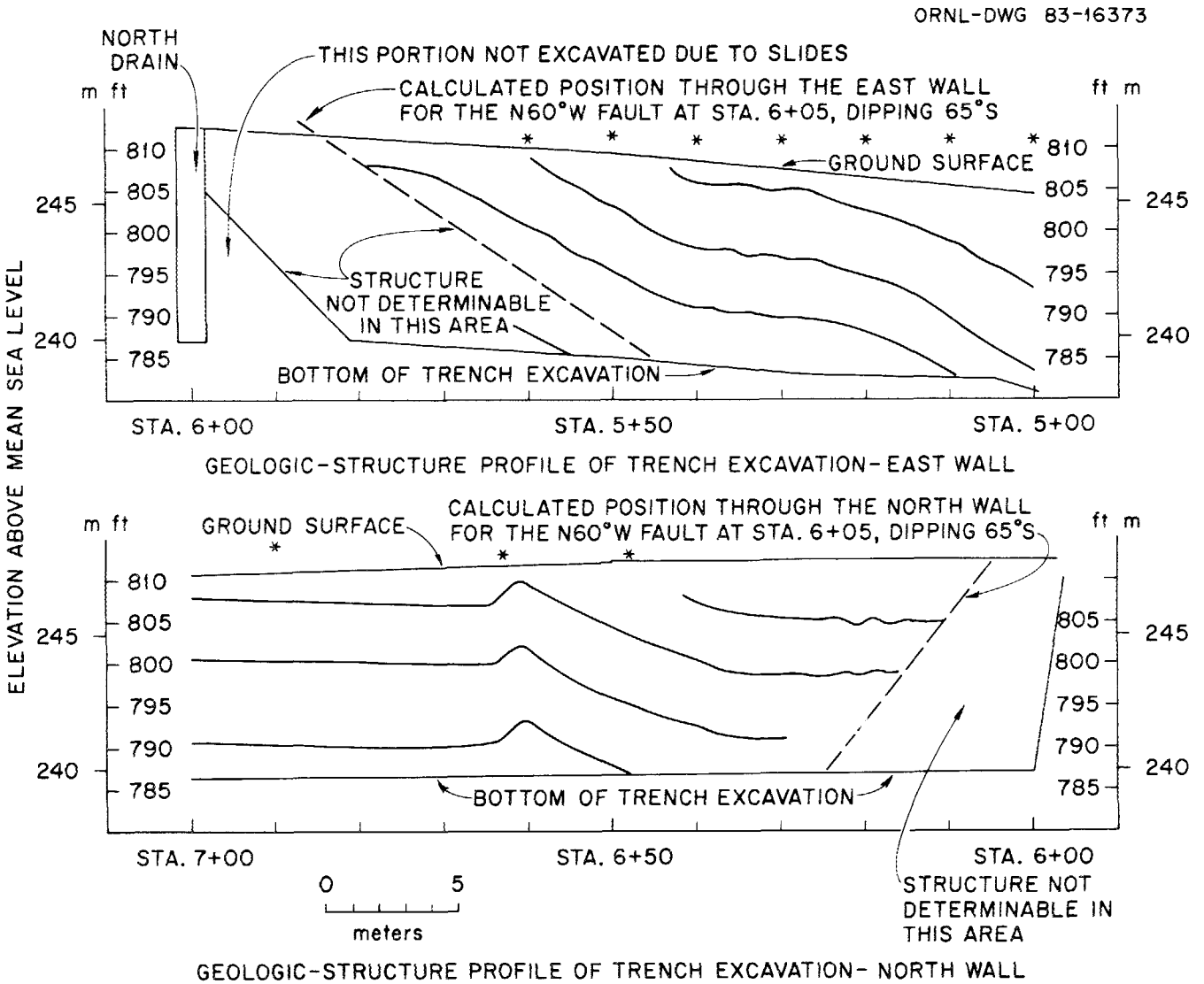

Fig. 3.24. Geologic structure profiles of drain excavation—east wall (station 5+00 to 6+00) and north wall (station 6+00 to 7+00).

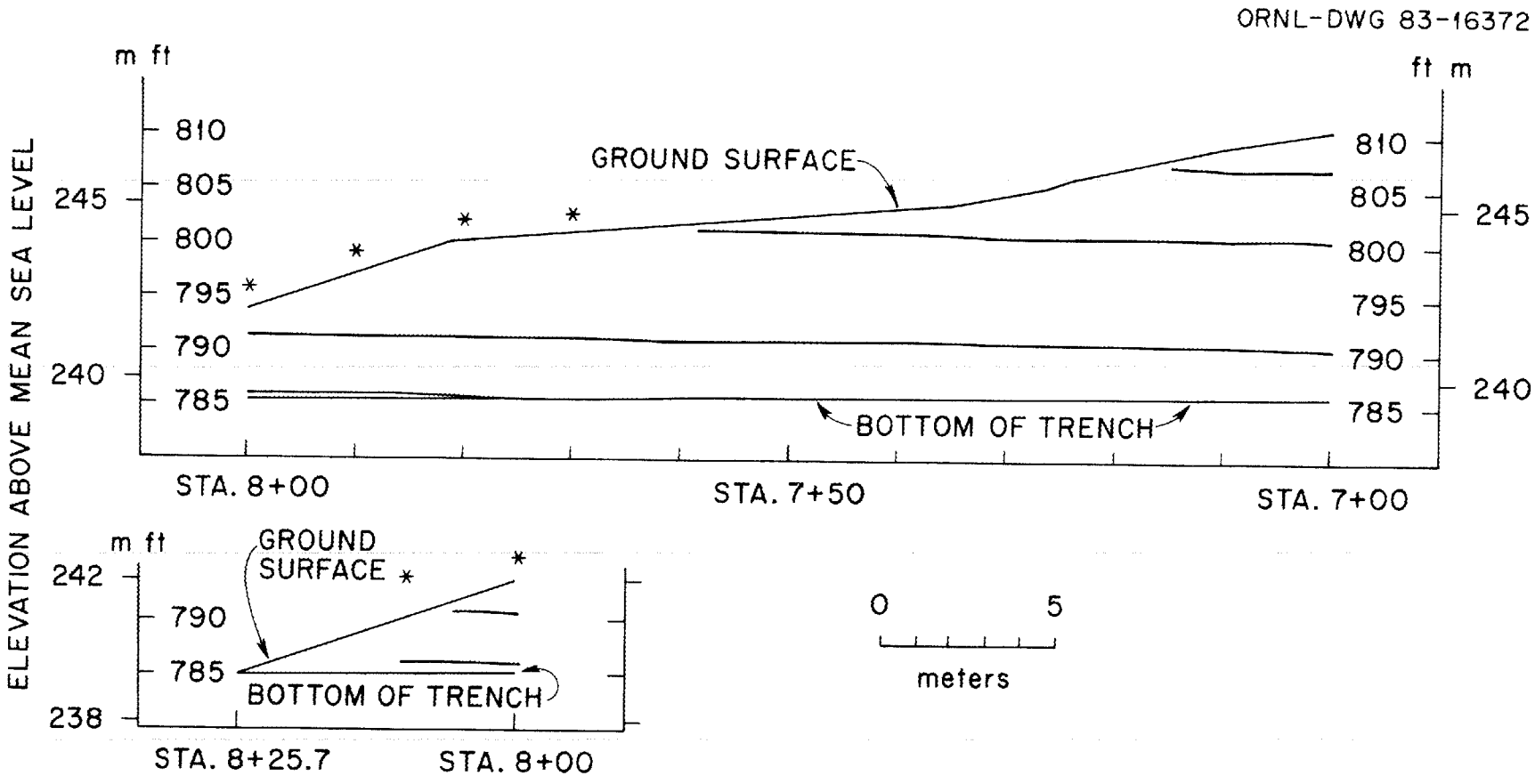

Fig. 3.25. Geologic structure profiles of drain excavation—north wall (station 7+00 to 8+25).

structures and dips of the bedding planes but do not accurately represent the thickness of individual beds. As stated previously, the strike of the nearby (1.5-km) Copper Creek fault is N 55°E, and the general strike of the strata at the site is assumed to be about the same. The characterization study of an adjacent site (Davis et al. 1983) supports this assumption by reporting a strike of N 50°E. Thus, the direction of the east section of the drain is at an approximate angle of only 5° from normal to the strike (Fig. 3.1). Therefore, the angles of the bedding planes (Figs. 3.22 through 3.24) of the east section of the trench are very near true angles of strata dip, which range from near horizontal to as steep as 60°. The north section of the drain is within 20° of parallel to the direction of strike (Fig. 3.1). As expected, much less structural folding and inclinations of bedding planes are apparent from this angle.

The fault shown in Figs. 3.24 through 3.27 is actually a zone containing more than one fault plane and associated contorted strata. Because slides destroyed the structural evidence and the excavation could not be entered, fault type (i.e., whether reverse, normal, or strike-slip) could not be determined. Where it could be measured, the fault plane had a strike of N 60°W and dip of 65°S and was coated with at least 1 cm of clay gouge. At station 1+60, a near vertical joint system of two sets (Fig. 3.28) could be measured and was found to be N 60°W and due north.

Slides of the drain walls were, as anticipated, always along discontinuites in the rock strata (Fig. 3.1). The north section of the drain ran subparallel to a fold limb dipping approximately 45° to the northwest into the drain excavation from the south wall. Slides in this section from the south wall were down weak bedding planes into the excavation (Figs. 3.29 through 3.32). Slides on the north wall were probably associated with near-vertical joints and with the fault zone crossing the drain in the vicinity of station 6+05.

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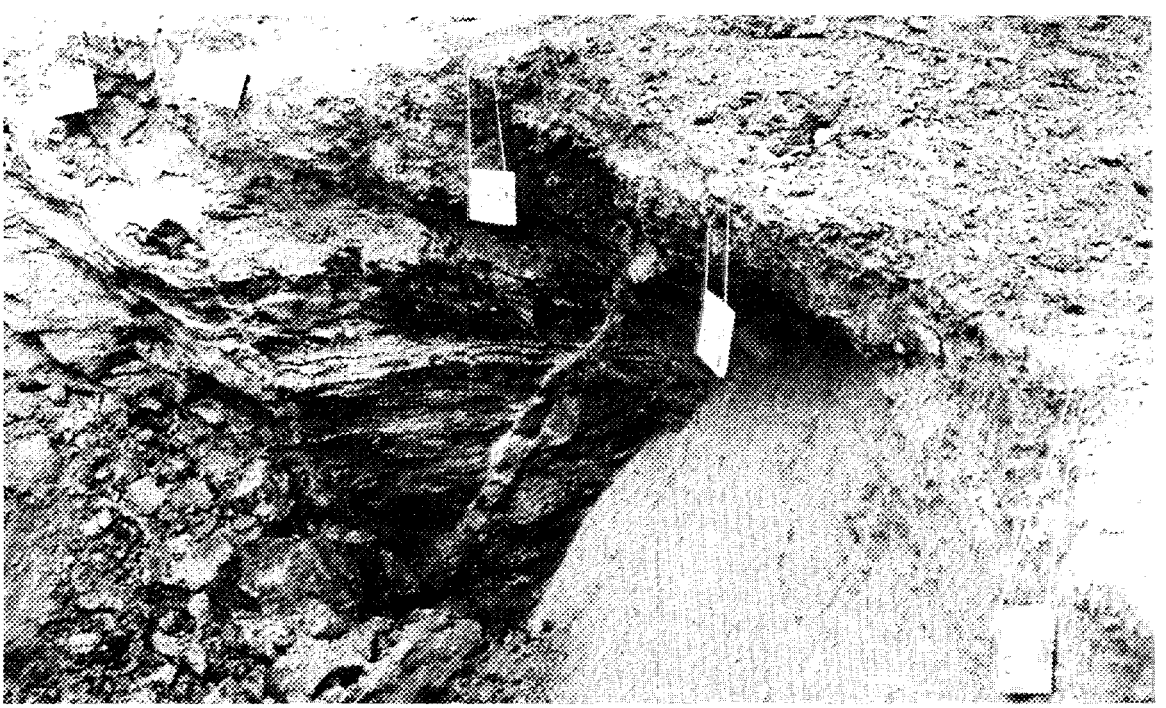

Fig. 3.26. Fault plane (right side of figure) in north wall, visible from approximate station 6+15 to 6+05.

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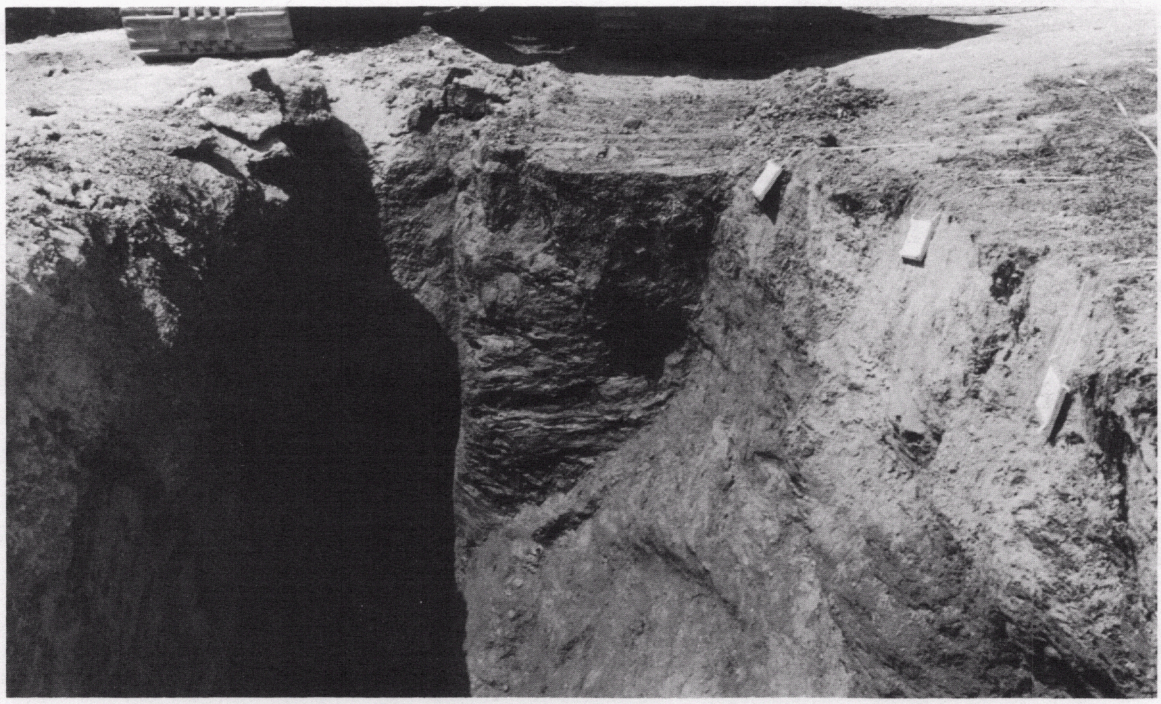

Fig. 3.27. Fault plane (right side of figure) in east wall, visible from approximate station 5+65 to 5+70.

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Fig. 3.28. Near vertical joint system at approximate station 1+60.

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Fig. 3.29. Slide in the south wall of the excavation from station 7+77 to 7+50.

Slides in the east section of the drain were much less frequent and less severe except in the northern-most 18 m, where the fault zone intersects the drain at an acute angle (Figs. 3.27 to 3.32). When excavation of the upper portion of the east drain intersected the previously constructed north drain at station 6+00, sliding of crushed stone from the north into the east drain (Fig. 3.33) occurred. This prevented excavation of the bottom of the east drain all the way to station 6+00; it had to be terminated at approximately station 5+85 (see drain profile at station 5+85, Fig. 3.24).

The geologic structure profiles in Figs. 3.22 through 3.25 are based largely on inclinations of bedding planes measured from photographs taken of the drain walls. Photography was used because saftey considerations prevented personnel from entering the excavation. Color photographs of the drain wall were made at 3.05-m intervals, except at locations of slides. Slides prevented use of the apparatus assembled to lower and raise the camera up the full height of the wall; however, photographs of these areas were taken from the top of the drain. Also, photographs were taken of additional sections of the wall where folded structures could be observed from the top of the drain. In practice, a weighted measuring tape to which was attached a placard noting the station was lowered to the bottom of the drain against the wall. On the opposite wall, a 35-mm SLR camera having remotely controlled flash and film advance was attached to a sliding apparatus and slid down a square wooden pole standing on the drain floor. As the camera was retrieved, a series of overlapping photographs was taken by the operator working from the surface (Fig. 3.34). Because a wide-angle lens was used, each photograph covered an area slightly greater than 0.61 m high by 0.91 m wide. Full sections of the drain wall that were photographed by this method are marked by an asterisk in the profiles in Figs. 3.22 through 3.25. To detect and correct any malfunctioning in the photographic process, film was developed daily.

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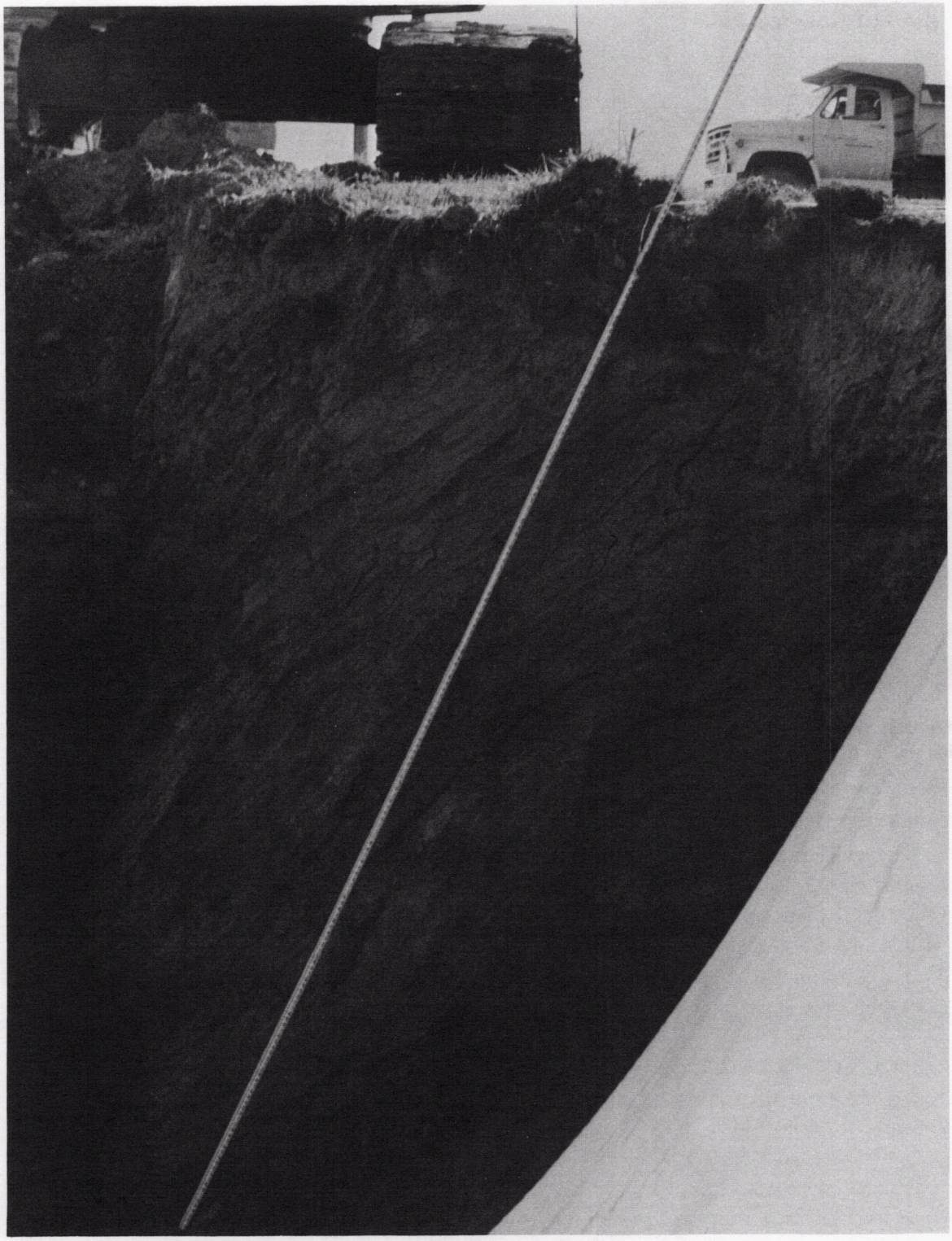

Fig. 3.30. End of slide in south wall at approximate station 7+50 after excavation of slide material. Note dip of strata bedding planes into excavation at approximately 45°. White in lower right of photo is filter fabric.

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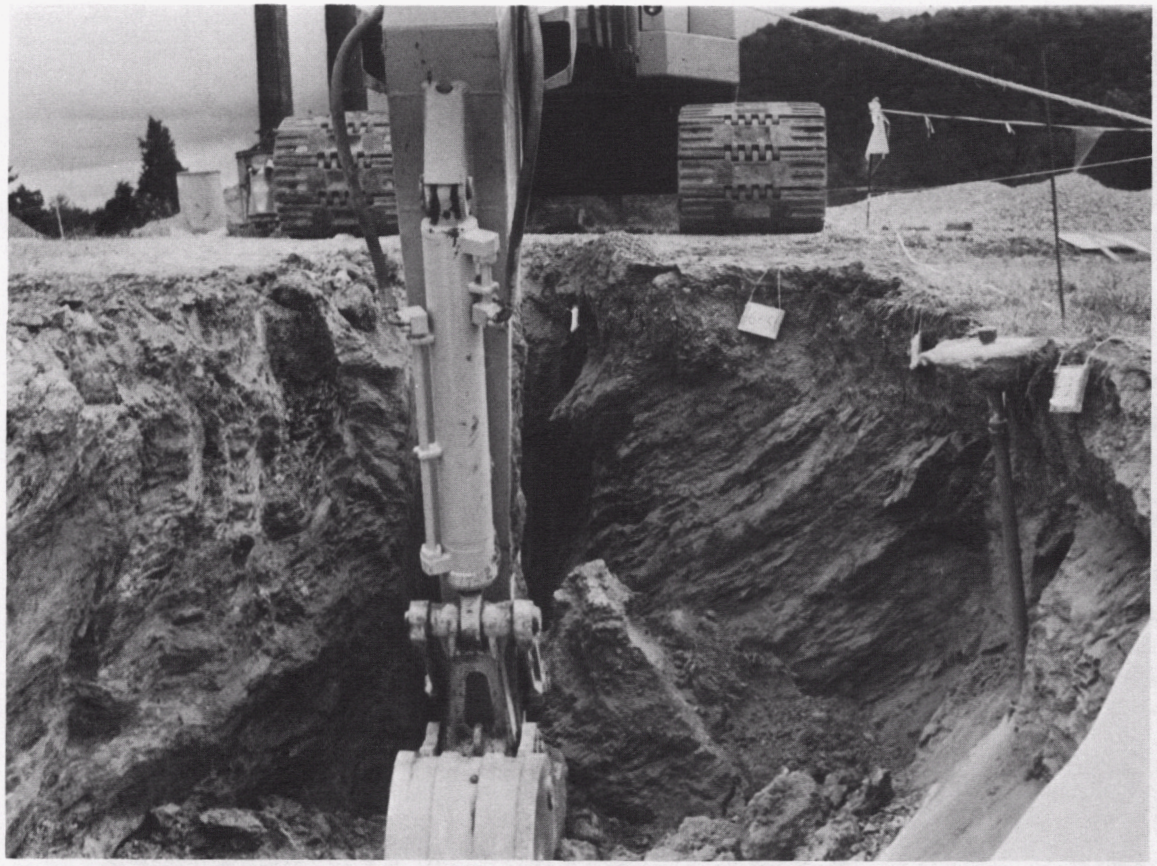

Fig. 3.31. View of slide areas in both north and south walls from stations 6+43 to 6+35. Exposed monitoring well in right of photo is well number Exp 3.

ORNL-PHOTO 7995-83

Fig. 3.32. Excavating slide material from west wall between station 5+60 and 6+00. Filter fabric can be seen in right side of photo.

ORNL-PHOTO 7994-83

Fig. 3.33. Intersection of excavation of east section into completed north section at station 6+00. Note crushed stone from drain installed in north section running into east section.

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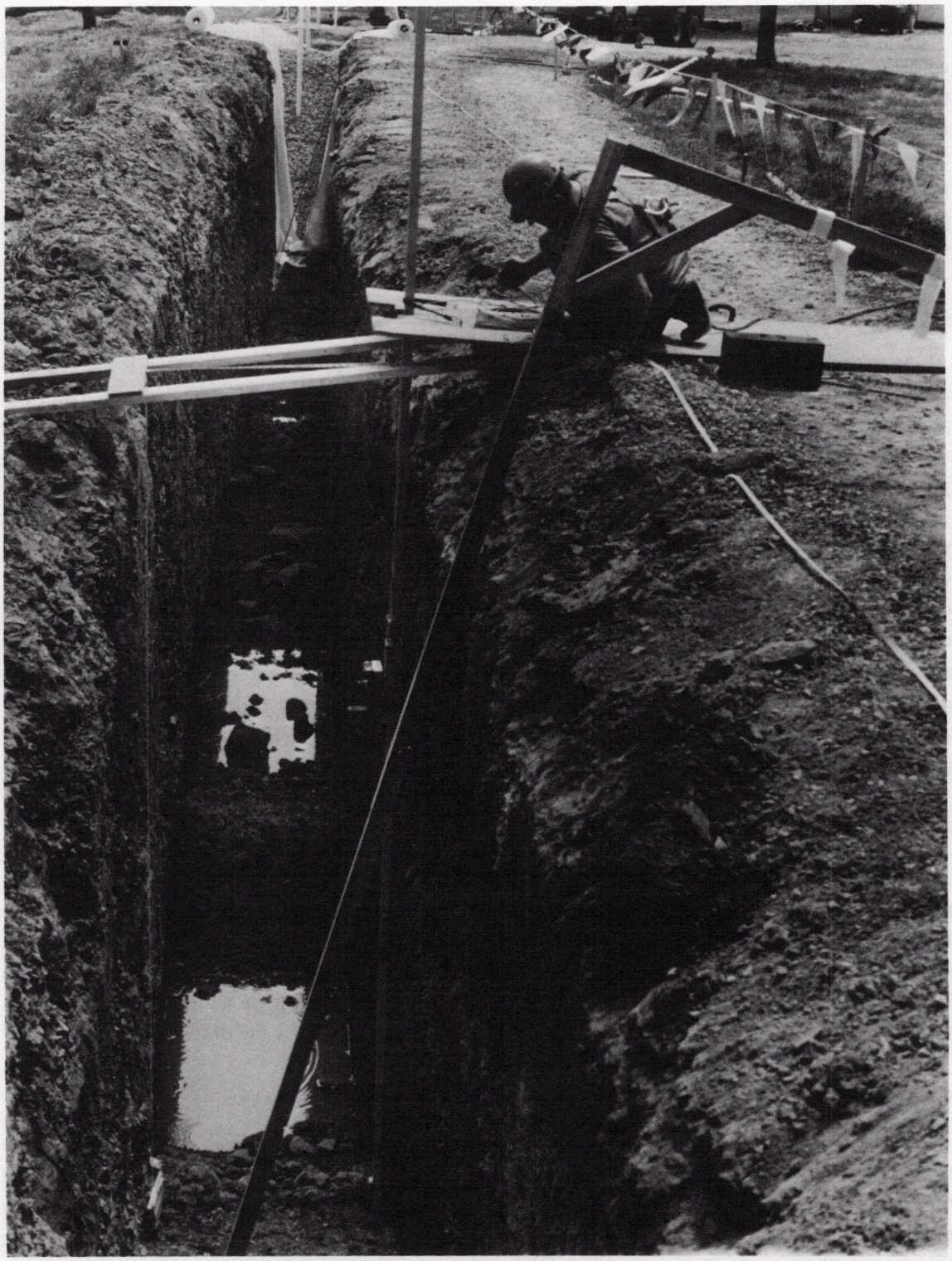

Fig. 3.34. View showing photos being taken of drain wall using apparatus operated from the top of the drain.

4. SUMMARY AND CONCLUSIONS

The 252-m passive French drain described in this report was completed in 17 working days at an estimated cost of $153,000. Of the 17 d, it is estimated that a total of approximately five days (29% of the time) was spent excavating sidewall material that slid into the drain during excavation. Even considering the numerous sidewall slides that would be anticipated, the feasibility of construction of such a groundwater interceptor drain was demonstrated. Observations made will be useful in planning similar construction projects. The conclusions reached as a result of drain construction are presented in this section, as well as preliminary data on the impact the drain is having on the water table.

4.1 GEOLOGIC STRUCTURE RESULTING IN SLIDES

As expected, the sidewall slides can be sudden and frequent in an excavation that is aligned parallel to geologic strike in the weathered rock of the Conasauga Group. This results primarily from planes of weakness along bedding planes or fault zones inclined into the excavation. However, in stretches where the beds are aligned horizontally, an excavation aligned parallel to strike would probably prove no more susceptible to slides than an alignment perpendicular to strike.

Detailed subsurface exploration to determine the geologic structure would be useful in cases in which an excavation alignment is parallel or subparallel to the regional geologic strike. In areas where only a thin soil mantle is present, several inexpensive exploratory trenches (excavated to a depth of only 1.2 to 1.5 m) may well provide adequate information to judge the geologic structure and associated potential problems. In cases in which the thickness of the soil mantle is sufficient to require excavation to a depth unsafe for personnel entrance, oriented cores, down-hole cameras, or geophysical techniques could be considered for exploration. Because slides could be handled by reexcavating the slide material, this project did not include any detailed subsurface exploration prior to construction.

4.2 EXCAVATION TO GRADE

As depth of the excavation increases to over 5 m, the difficulty of accurately maintaining design grade level with the conventional grade-checking methods previously described in Sect. 3.2.1 increases. It is easier to excavate to a maximum allowable grade if over-depth excavation is not a concern. If maintaining a specific grade to close tolerance or a precise profile is necessary, more frequent and accurate methods of recording grade have to be developed and adopted. Such procedures would also yield more accurate records from which to construct the drain profile.

4.3 DELETION OF CONCRETE PLACEMENT IN PERMEABLE ZONES

Experience on this project indicates that placement of concrete in portions of the bottom of the drain between rock summits is not a viable option; to do so, stretches of the drain would have to be left open for the concrete placement operation. Slides occurring in the open drain beyond the reach of the backhoe could not be excavated without considerable loss of time and additional cost. For this reason, the placement of lean concrete in portions of the drain bottom was dropped from this project and would not be recommended in future drain designs.

4.4 PLACEMENT OF CRUSHED STONE AND FILTER FABRIC

As the depth of the drain increases, the length of drain that is left open increases because of the 36° angle of repose of the crushed-stone backfill. In addition, up to 3 m of open space ahead of the toe of the crushed stone slope is required to place the filter fabric to the bottom of the drain. An additional open distance ahead of the filter fabric and monitoring wells (about 6 m) is necessary working space for the backhoe arm and excavating bucket. These distance requirements total approximately 20 m of open trench from the top of the crushed-stone fill to the working face in a 7.6-deep drain. This is a considerably greater distance than the backhoe positioned above the working face could reach if a sidewall slide were to occur. On two occasions, slides did occur because of this open distance and were beyond the reach of the American backhoe. Part of the drain had to be backfilled to provide a ramp for the the backhoe to reach and excavate the slide material. In addition, when slide material falls on the crushed-stone slope, invariably an unknown amount of it becomes mixed with the crushed-stone backfill.

This problem of sidewall slides in open excavations could be slightly lessened if, upon further consideration, the filter fabric was found to be necessary only in the upper portion of the drain to protect against migration of soil particles. In this situation, the open area of the drain could be reduced up to approximately 3 m because the filter fabric could be installed over the slope of the crushed stone rather than in front of the toe of the slope.

Another option worth considering would be placement of a movable brace (bulkhead) in the drain to allow the stone backfill to stand at a >36° angle. If the stone could be maintained at near a 90° angle, the length of the open drain would be dictated only by the backhoe requirements and much more sidewall support would be afforded by the crushed stone.

4.5 DRAIN OBSTRUCTION AT STATION 2+40

Measurements of water elevations in drain monitoring wells were taken shortly after drain construction. Figure 3.18 and Table 3.2 show results of this survey that indicate a sudden drop in the water level within the drain between monitoring wells at stations 2+26 and 2+51. As described previously in Sect. 3.2.1, this drop is believed to result from unrippable rock that protrudes into the drain at station 2+40. This nonweathered rock probably acts as a relatively impermeable barrier and dams water upslope. Currently, there is no indication that this will have any significant effect on the operation of the drain. However, if future monitoring should indicate a benefit to elimination of this obstruction, possible solutions would be to either fracture the unrippable rock by blasting without excavation or reexcavate and cut the rock in place.

4.6 IMPACT ON THE 49-TRENCH AREA WATER TABLE

When excavation reached station 2+40, water was observed to begin to stand in the bottom of the drain. Finally, on September 27, flow was observed from the discharge pipe at catch basin number 2. Between September 28 and November 11, the flow was measured approximately daily, using a container of known volume and a stopwatch (Table 4.1). The flow has been observed to be quite steady during periods of no rainfall, averaging between 4 and 7 L/min. Response to rainfall events has resulted in flows as high as 35 L/min (October 13, 1983), and these high flows gradually decrease to base flow conditions shortly after the rainfall ceases. To date, no flow has been observed at catch basin No. 1, which serves the north drain. Flow in this leg of the drain is not expected to occur until the wet months of November and December.

Table 4.1. Flow measurements from eastern section of French drain and daily precipitation records

Date 1983	24 hour precipitation (mm)	Time	Flow (L/min)
9-28	0.0	2:45 p.m.	7.22
9-29	0.0	8:00 a.m.	7.22
		3:00 p.m.	7.22
9-30	0.0	8:35 a.m.	7.03
		2:30 p.m.	7.03
10-01	0.0		
10-02	0.0		
10-03	0.0	8:45 a.m.	5.55
		4:00 p.m.	5.36
10-04	0.0	9:00 a.m.	5.17
10-05	31.1	9:00 a.m.	11.40
		10:30 a.m.	8.29
		11:45 a.m.	7.03
		2:30 p.m.	6.50
		4:00 p.m.	6.23
10-06	0.0	2:00 p.m.	5.36
10-07	0.0	1:25 p.m.	5.28
10-08	0.0		
10-09	0.0		
10-10	0.0	8:30 a.m.	4.33
10-11	3.2	8:30 a.m.	4.07
		11:00 a.m.	4.14
		3:30 p.m.	4.07
10-12	1.3	11:00 a.m.	3.99
10-13	44.4	9:00 a.m.	35.07
		1:00 p.m.	21.66
10-14	0.0	12:00 a.m.	5.24
		3:30 p.m.	5.24
10-15	0.0		
10-16	0.0		
10-17	0.0	11:45 a.m.	4.07
10-18	0.0	10:30 a.m.	3.95
10-19	0.0	7:40 a.m.	3.88
10-20	0.0	4:00 p.m.	3.65
10-21	3.2	11:30 a.m.	3.53
10-22	5.7		
10-23	30.5		
10-24	0.0	9:30 a.m.	5.85
10-25	0.0	10:20 a.m.	4.83
10-26	0.0	2:00 p.m.	4.33
10-27	0.0	3:30 p.m.	4.14
10-28	0.0	3:15 p.m.	3.80
10-29	0.0		
10-30	0.0		

Table 4.1. Flow measurements from eastern section of French drain and daily precipitation records

Date 1983	24 hour precipitation (mm)	Time	Flow (L/min)
10-31	0.0	12:30 p.m.	3.45
11-01	0.0	3:00 p.m.	3.31
11-02	0.0	12:30 p.m.	3.19
11-03	2.5		
11-04	12.1	11:45 a.m.	3.80

A single grab sample of water collected at catch basin No. 2 was analyzed for ^{90}Sr by Cerenkov radiation counting (Larsen 1981) and for ^{137}Cs and ^{60}Co by low-level gamma-ray spectrometry. No detectable quantities of these radionuclides were observed, indicating that contaminated water from the waste disposal trenches has not yet resulted in a highly contaminated discharge. This result greatly simplified construction of the catch basins. Sampling of the drain effluent for radionuclide analysis will be continued.

As a result of water movement into the eastern drain, the water level in several wells in the 49-Trench area began to decrease. To document this decrease, the monthly water-level monitoring program was accelerated to include weekly observations during the days of active drain construction. Table 4.2 summarizes water-level changes in seven area wells during drain construction. These

Table 4.2. Changes in water level taking place in selected 49-Trench area wells during construction of the French drain (*meters above mean sea level*)

Date	Well[a]						
	394	398	399	401	T191	T201	T203
09-01-83	241.35	241.28	239.76	240.76	242.09	238.58	239.31
09-08-83	241.16	241.25	239.59	240.61	241.32	238.52	239.18
09-13-83	241.28	241.21	239.63	240.59	241.24	238.46	239.21
09-22-83	241.13	241.08	239.51	240.46	241.07	238.25[b]	239.07
09-29-83	240.35	239.82	238.42	240.34	240.83	238.23[b]	238.52[b]
10-03-83	239.91	239.55			240.70	238.23[b]	238.60[b]
10-05-83	240.14	239.50	238.32	240.34	240.67[b]	238.25[b]	238.61[b]
10-10-83	239.74	239.36		239.99	240.64[b]	238.22[b]	238.61[b]
10-13-83	240.62	239.32	238.48	240.78	241.25	238.25[b]	239.32
10-20-83	239.57	239.18	237.86	239.88	240.89	238.24[b]	238.63[b]
10-27-83	239.51	239.11	237.80	239.85	240.67[b]	238.23[b]	238.62[b]
11-03-83	239.46	239.04	237.71	239.69	240.64[b]	238.23[b]	238.62[b]
Change in elevation[c] (m)	-1.89	-2.24	-2.05	-1.07	-1.45	-0.35	-0.69

[a]For exact well location see Fig. 1.5.

[b]Water is at or below the elevation of the well bottom.

[c]Change in water elevation during drain construction (between Sept. 1 and Nov. 3, 1983).

wells are located at different distances from the drain (both inside and outside of waste disposal trenches), and all show a decrease in water elevation ranging from 0.35 to 2.24 m. Some of the observed decrease in elevation resulted from the fact that September and October are the dry months of the year and groundwater levels are naturally dropping at this time. However, the drastic decrease, partiularly in wells 394 and 398, that occurred after the September 22 reading indicate that the drain was responsible. Further groundwater monitoring is in progress to facilitate a more complete analysis of the impact of the drain and to pinpoint the areas of influence. However, initial data indicate that the drain is serving its intended purpose.

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APPENDIX

GROUNDWATER BARRIER ALTERNATIVES CONSIDERED

Scheme Ia. A French drain in a narrow trench excavated using a backhoe.

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GROUNDWATER BARRIER ALTERNATIVES CONSIDERED

Scheme Ia. A French drain in a narrow trench excavated using a backhoe.

A French drain would be constructed by excavating a narrow trench, using a backhoe. The drain would surround the burial site on the north and east sides and be filled with crushed stone.

Scheme Ib. A French drain in a sloped excavation (1.5 to 1 slope).

A French drain would be installed in a sloped excavation. The excavation would surround the burial site on the north and east sides. The effectiveness of the French drain in a sloped excavation would be equal to that of a narrow trench. The advantage of a sloped excavation over a narrow trench is that men and equipment can work safely in the bottom of the trench. The disadvantage is the volume of earth that must be excavated.

Scheme Ic. A French drain installed in a braced excavation.

A French drain would be installed in a trench excavated by equipment located at the surface, except where rock is encountered. In these areas, trench-wall bracing would be used to provide a safe work space for men to complete excavation of the bottom of the narrow trench. The trench would surround the burial site on the north and east sides and would be filled with crushed stone.

Scheme Id. A free-draining rock wall constructed using drilled piers.

A free-draining rock wall would be constructed by using drilled piers filled with clean, crushed stone (ASTM size 57). The bottom of the wall would be sloped to drain the water collected.

Scheme IIa. A narrow slurry trench excavated by using a backhoe.

A slurry wall would be constructed by excavating a narrow trench by the method used in scheme Ia. The trench would be filled with a bentonite slurry and would divert the groundwater around the 49-Trench area. The wall would wrap around the burial site on the north and east sides.

Scheme IIb. A cement cut-off wall constructed by using drilled piers.

A cementatious cut-off wall would be constructed with drilled piers. The wall would be located adjacent to the burial site on the north and east sides to divert groundwater. The intersection of individual piers would require in-section grouting to ensure wall impermeability.

Scheme IIc. A steel sheet pile cut-off wall.

A sheet pile cut-off wall is the least likely to be effective. Steel sheet piles would most likely refuse being driven further before actually penetrating unweathered shale. The reduction in the groundwater flow would probably be minimal.

Scheme IIIa. A French drain in series with a slurry trench. Schemes Ia and IIa.

A slurry wall would surround the burial trenches as in Scheme IIa, and the rock drain would wrap around the slurry wall on the north and east sides where groundwater enters the site. The system combines the approahes of interception and diversion that are included in schemes Ia and IIa. The advantage of this system would be the that groundwater would be eliminated completely.

Scheme IIIb. A French drain including an impervious membrane and excavated using a backhoe.

A French drain would be constructed as in scheme Ia, and a polyethylene membrane would be added on the inside wall of the drain. This system would combine the methods of interception and collection with an impervious barrier without requiring additional trench construction. The polyethylene membrane would function as a backup to the french drain, ensuring the diversion of groundwater.

Scheme IIIc. A cement cut-off wall constructed with drilled piers and a free draining rock wall (schemes Id and IIb).

A combination of schemes Id and IIb would be used in which a rock drain would be constructed outside of the impermeable wall. As with scheme IIIa, the advantage of this scheme is the security provided by the redundant systems.

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