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Evaluation of 1985–1986 Corrective Actions at ORNL Liquid Waste Disposal Trench 7

B. P. Spalding

Environmental Sciences Division
Publication No. 3690

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DEPARTMENT OF ENERGY

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EVALUATION OF 1985-1986 CORRECTIVE ACTIONS AT ORNL LIQUID WASTE DISPOSAL TRENCH 7

B. P. Spalding

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SUMMARY

Several corrective actions were taken in 1985-1986 at the site of ORNL radioactive liquid waste seepage trench 7 in an effort to reduce the discharge of radionuclides, mostly ^{60}Co, from a groundwater seep on the eastern side of the site. First, the size of the asphalt cap over the trench was doubled, and cap runoff was diverted away from the site to the west. Second, the buried waste transfer line to the trench was excavated and plugged and its pipe trench was dammed with clay backfill. These actions were designed to reduce groundwater recharge in the area that might be the source of water to the seep. Third, a series of grout injections was carried out at 5-ft intervals along a perimeter line on the eastern and northern edges of the site.

A total of 65,500 gal of lime-fly-ash grout was injected at 303 locations at depths up to 40 ft in an effort to seal relict contaminated strata with probable hydrologic connection to the seep. However, the grout formulation specified in the contract would not set to a detectable compressive strength nor would the grout samples exhibit a reduction in hydraulic conductivity during over a year of observation. Thus, the material specification for the grout was inappropriate for the desired effect of in situ hydrologic isolation. Core sampling at the site revealed that the grout flowed into the soil formation along discrete thin layers (i.e., along fractures probably enlarged by the grout injection pressures). Only three grout layers, with a maximum thickness of 0.25 in., were found in over 90 ft of core from three locations along the grout injection line. Thus, this grouting action would have little potential to influence containment of radionuclides that leach from contaminated strata. Probably unrelated to the grouting and capping actions, concentrations of ^{60}Co have declined beyond those expected for decay since they were last measured in 1982 in most groundwater monitoring wells and at the seep.

In conclusion, it does not appear that soils, which are formed from the Maryville member of the Conasauga formation that dominates pits and trench waste area grouping, are amenable to establishing in situ grout curtains, particularly with particulate-containing grouts. Any future corrective actions, that use grouting, in situ or otherwise, should specify grout properties, such as permeability and compressive strength, in addition to grout materials. Such specifications should be based on tests that simulate the field conditions under which the grout will be employed.

1. HISTORY OF LIQUID WASTE DISPOSAL TRENCH 7

Liquid waste disposal trench 7 was located and oriented according to geologic advice based on preconstruction water table monitoring and coring data (Fig. 1). To overcome the potential for complete loss of service, as experienced with trench 6, the design of trench 7 delineated three 100-ft-long segments to be operated independently. In case a leak were found in one segment, the remaining segments could still be operated. Successful operation of trench 7, with a planned capacity of 4,000 gpd, would allow the two remaining pits, 2 and 4, to be removed from service. The third or northern-most segment of trench 7 was never constructed because preconstruction groundwater monitoring revealed water table elevation in the area to be near the bottom of the planned segment. Construction of two segments was completed in August 1962. The trench was pretreated with 50,000 gal of 4% sodium hydroxide presumably done to enhance the adsorption of ^{90}Sr should leakage such as that in trench 6 occur. Trench 7 was found to have a seepage capacity about four times that of trench 5 (about 4,000 gpd). Because the combined seepage capacities of trenches 5 and 7 were found to exceed the generation rates of the Oak Ridge National Laboratory (~8000 gpd), seepage trench construction was concluded.

During its operation, trench 7 was found to leak minor amounts of ^{106}Ru, particularly to a seep on its eastern side (Fig. 2). In general, trench 7 performed well in handling the 9.5 million gallons of waste transferred to it. This waste contained about 48000 Ci of ^{90}Sr, 231000 Ci of ^{137}Cs, 3400 Ci of ^{106}Ru, and 1500 Ci of ^{60}Co. These inventories were also divided between the two trench segments, 7A and 7B, about equally. When hydrofracture disposal became operational in 1966, trenches 5 and 7 were used for the last time. Although the hydrofracture operation required that liquid waste be reduced in volume by a new evaporator and stored before injection, it was considered much safer than seepage pit disposal. The continuing problems of groundwater contamination, both major and minor, with all of the pits and trenches compromised the acceptability of this disposal technique. The ground surface over trench 7 was eventually paved with asphalt in 1970. Additional details on the operation and inventory of trench 7 can be found in Olsen et al. (1983).

2. PROBLEMS WITH TRENCH 7 PRIOR TO CORRECTIVE ACTIONS IN 1985

The 1970 asphalt cap over trench 7 was surrounded by a narrow band of crushed stone, which was an extension of the bed for the asphalt pavement. Practically all surface runoff from the asphalt would be caught by a surrounding rim of crushed stone and infiltrate the soil. For every inch of rainfall on the asphalt, about 700 ft^3 (5200 gal) of runoff would result. A significant portion of this quantity could have been finding its way directly into trench 7. Alternatively, this infiltrating water could have been conducted laterally through relict contaminated strata and result in exacerbated leaching of radionuclides.

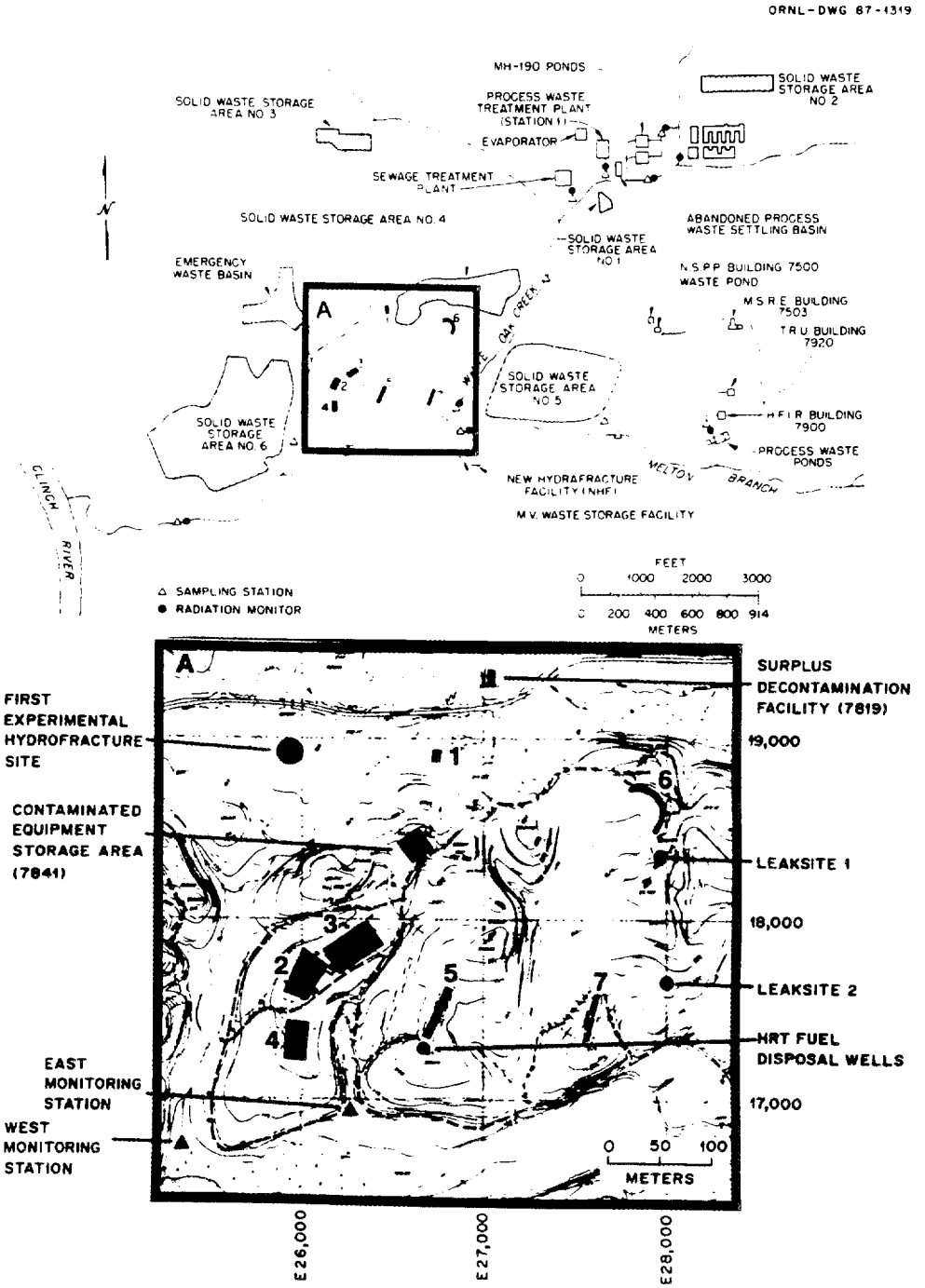
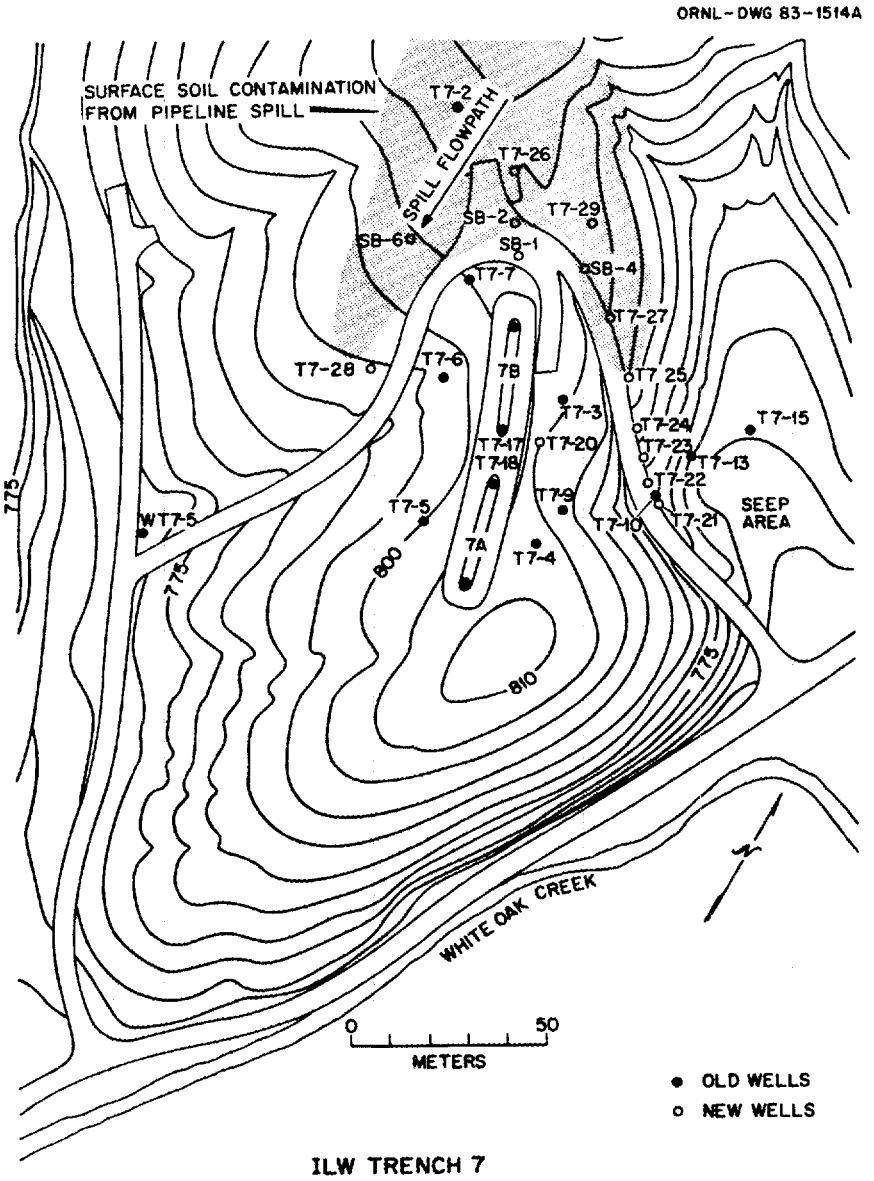


Fig. 1. Location of liquid waste disposal trench 7 in Melton Valley at Oak Ridge National Laboratory.



ILW TRENCH 7

Fig. 2. Location of monitoring wells and seep area east of liquid waste disposal trench 7.

Another feature that may have been conducting water to the trench 7 environs was the abandoned waste transfer pipeline. Although the pipeline itself had been known to leak (Duguid and Sealand 1975) in areas near trench 7, its access trench, constructed for pipeline burial, could have been functioning as a groundwater collector drain. The pipeline and its trench has a gradient sloping 35 ft, vertical within the 400 ft horizontal from the valve box, where it had been disconnected and capped, to the north. Thus, the pipe trench may have been functioning to collect water and channel it to the trench 7 environs.

Another source of groundwater contamination in the trench 7 environs is the relict contaminated strata around the trench. During trench operation, wastewater levels were maintained up to a maximum elevation of 799 ft. Olsen et al. (1983) reported various relict contamination zones or layers to the east of the trench 7 site extending from this elevation to well below the water table at the site (about 775 ft). Infiltrating precipitation or laterally moving groundwater could be passing through these relict contaminated strata and recharging the seep on the eastern side (Means et al. 1978).

To correct these situations that may have led to the radioactive seepage around trench 7, Stansfield (Appendix C) proposed several remedial actions. First, the 1970 asphalt cap over the trench had to be expanded, and its runoff had to be routed away from the trench. This construction was carried out between July 9 and August 23, 1985, and was reported by Mulholland (Appendix D). The cap was enlarged to cover 36% of the site drainage area, and cap runoff was channeled through a conduit to the western side of the site (Fig. 3). To eliminate the possibility of recharge to the site through the waste transfer line installation trench, an excavation was made just north of the trench to a depth about 2 ft below the pipe. A 4-ft section of the pipe was then removed, the ends capped in place, and the excavation filled with an impermeable clay that was to dam any water collected in the pipe trench.

To prevent potential leaching of relict contamination strata by groundwater, Stansfield (Appendix C) proposed injecting grout through a series of points along the eastern and northern site perimeter road. It was anticipated that grout would fill hydrologically active fissures and discontinuities that had been found to be contaminated (Olsen et al. 1983). Conceptually, the grout would form an in situ grout curtain that would also function as a groundwater barrier for any recharge to the seep.

A lime-fly-ash grout formulation was recommended, and because of the anticipated track-mounting of the grout mixing and injecting equipment, a line of grout injection points was specified along the existing access road to the north and east of the trench. This in situ grout curtain was designed to a maximum depth of 40 ft with three rows of injection points spaced every 5 ft (Fig. 4). A grout injection line closer to the trench would have been preferred, but because of the difficulty of constructing a new access road in the contaminated area, the existing access road was recommended. It was assumed that the specified grout curtain would not affect the groundwater table in the area because unimpeded drainage would remain to the western side of site. Thus, no groundwater interception or

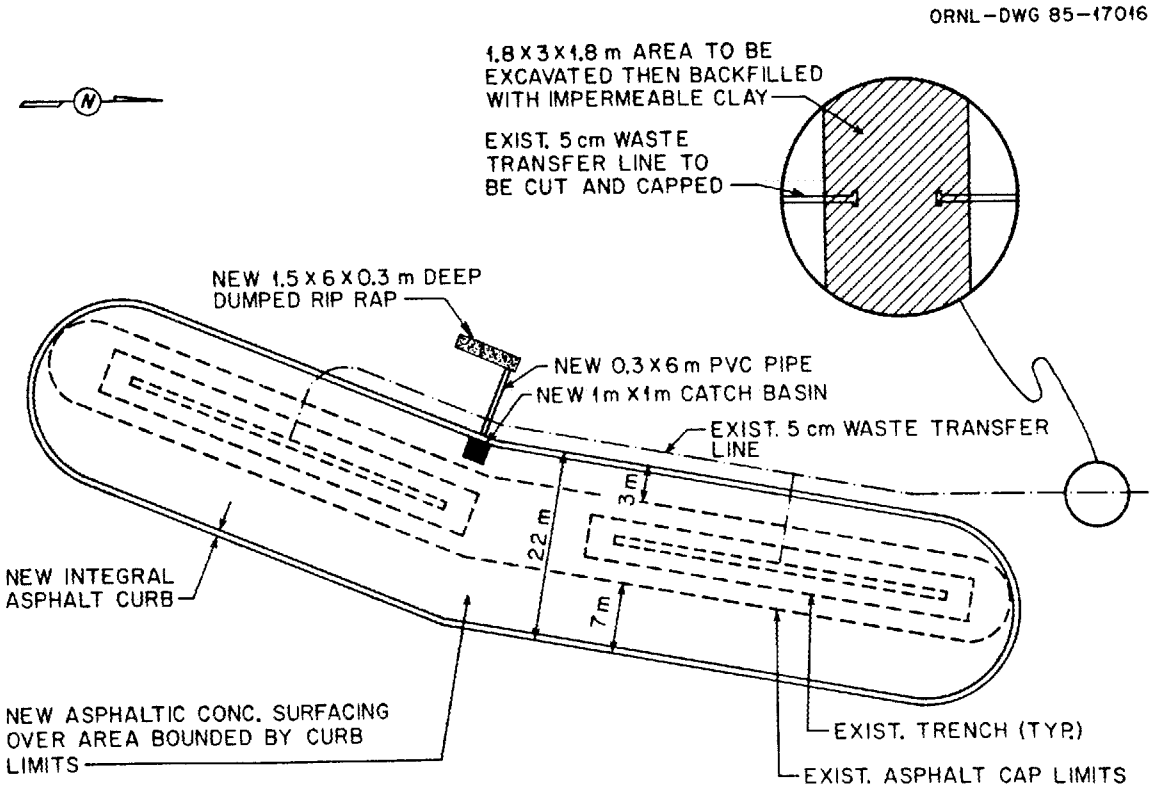


Fig. 3. Plan view of the liquid waste disposal trench 7 new (1985) and previous (1970) asphalt cap.

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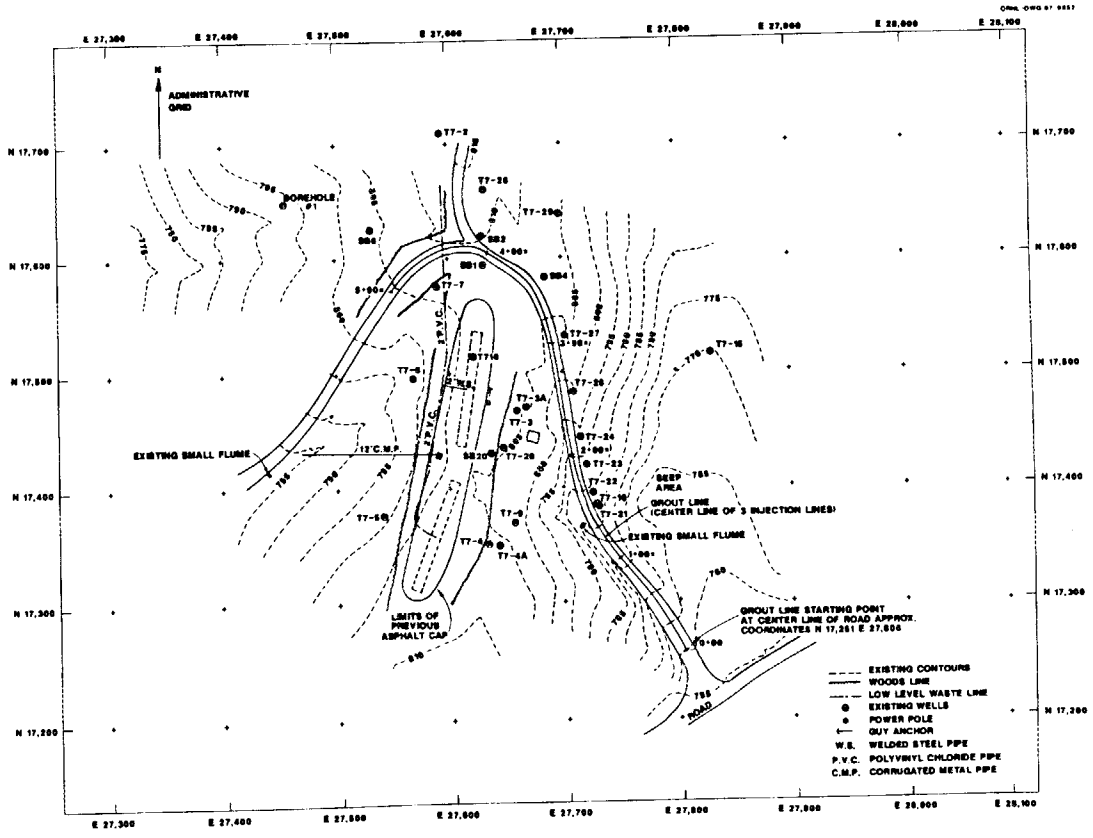

Fig. 4 Location of grout injection centerline for the 500-ft perimeter grout line.

diversion was planned on either side of the grout line. It was anticipated that the filling of secondary or conducting voids in the soil formation would isolate many contaminated strata hydrologically . If this isolation of contaminated strata from further groundwater leaching were achieved, then an eventual decrease in the radioactive discharge at the seep on the eastern side could be expected.

3. FIELD GROUTING OPERATIONS

A contract was awarded to Rembco Engineering Corporation (P. O. Box 23009, Knoxville, TN 37933-1009) to construct the grout curtain. Site activities began with the completion of the first grout injections on December 3, 1985, and continued through April 28, 1986. A total of 65,563 gal of the lime-fly-ash grout formulations was injected into a total of 303 points, as specified in the contract. The grout injections were accomplished in two stages. First, injections were performed on 10-ft centers along the grout injection line. Second, injections were performed between the initial 10-ft spacings, resulting in the specified 5-ft centers. The initial injections on 10-ft centers employed a formulation of 1 lb of lime and 3 lb of fly ash (pozzolanically active) for every 3 gal of water. The second set of injections completing the 5-ft centers, employed a formulation which contained 1 lb of lime, 3 lb of fly ash for every 2 gal of water. Both formulations also contained a 50 lb bag of trisodium phosphate per 1500-gal batch as a surfactant, although the actual contract specifications called for a nonionic surfactant. A copy of the specifications (copied from Engineering Drawing C3E20965A011) is listed in Appendix A.

Most grout preparation and injections were carried out using the equipment depicted in Fig. 5. Grout was prepared in batches of 1500 gal. Fly ash was delivered to the site in bulk and stored under a tarp until needed. It was loaded into the mixing tank with a small front-end loader (Fig. 6). Weights were estimated from the volume of the loader bucket. The fly ash was shovelled into the mixing tank from an access port on top. Lime was delivered in 50 lb bags and the contents of an appropriate number of bags were added to each batch. Trisodium phosphate also was delivered in 50 lb bags. Make-up water was obtained from a fire hydrant near the new hydrofracture facility about 1200 ft to the southeast. During and after the addition of solids to the tank, an air-driven diaphragm pump was used to agitate and resuspend solids within the tank (Fig. 7). Another air-driven diaphragm pump was used to deliver grout to the injection pipe. Hose from this pump was attached to the 1-in.-diam injection pipe via quick-disconnect fittings and was monitored for pressure (Fig. 8). Only one grout hole was injected at a time, but multiple valved connections were often used to save time. Injected grout volumes were estimated from the drop in elevation of the batch tank. Attempts to employ a water meter to measure delivered grout volumes failed because of clogging by the grout's suspended solids. Grout was pumped into an injection pipe up to a maximum pressure of 200 psi but, mostly, not in excess of 50 psi. The initiation of grout acceptance by a given hole was often accompanied by a noticeable drop in pumping pressure. If grout was not accepted by a given hole, the injection pipe was raised about 1 ft, and pumping was reinitiated. Usually only one or two such lifts were possible before grout

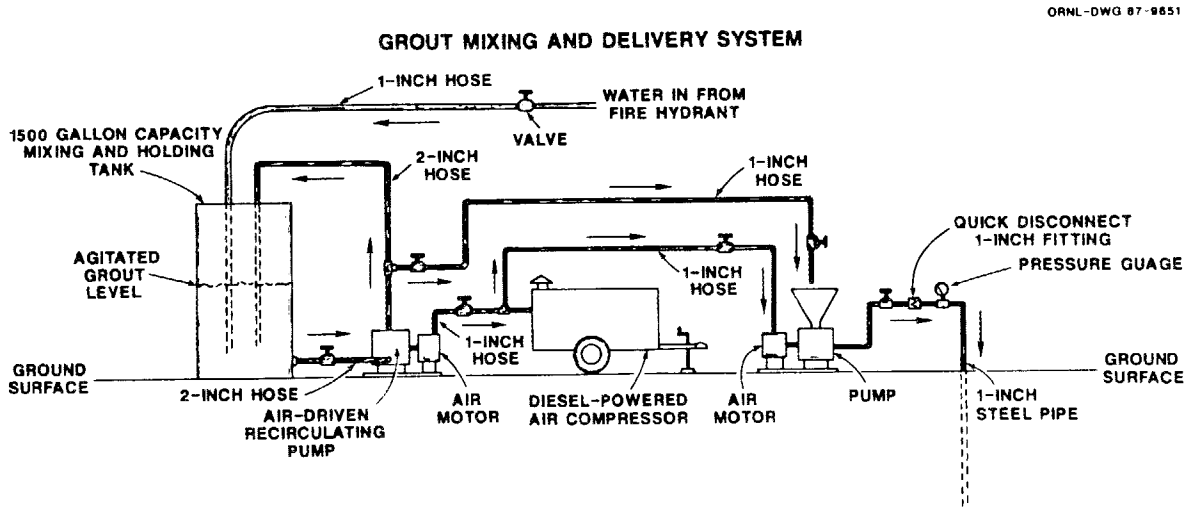

Fig. 5 Schematic layout of the grout mixing and delivery equipment used by Rembco Engineering, Inc., in the perimeter grouting of liquid waste disposal trench 7.

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Fig. 6 Grout preparation and delivery equipment at the trench 7 site on November 20, 1985.

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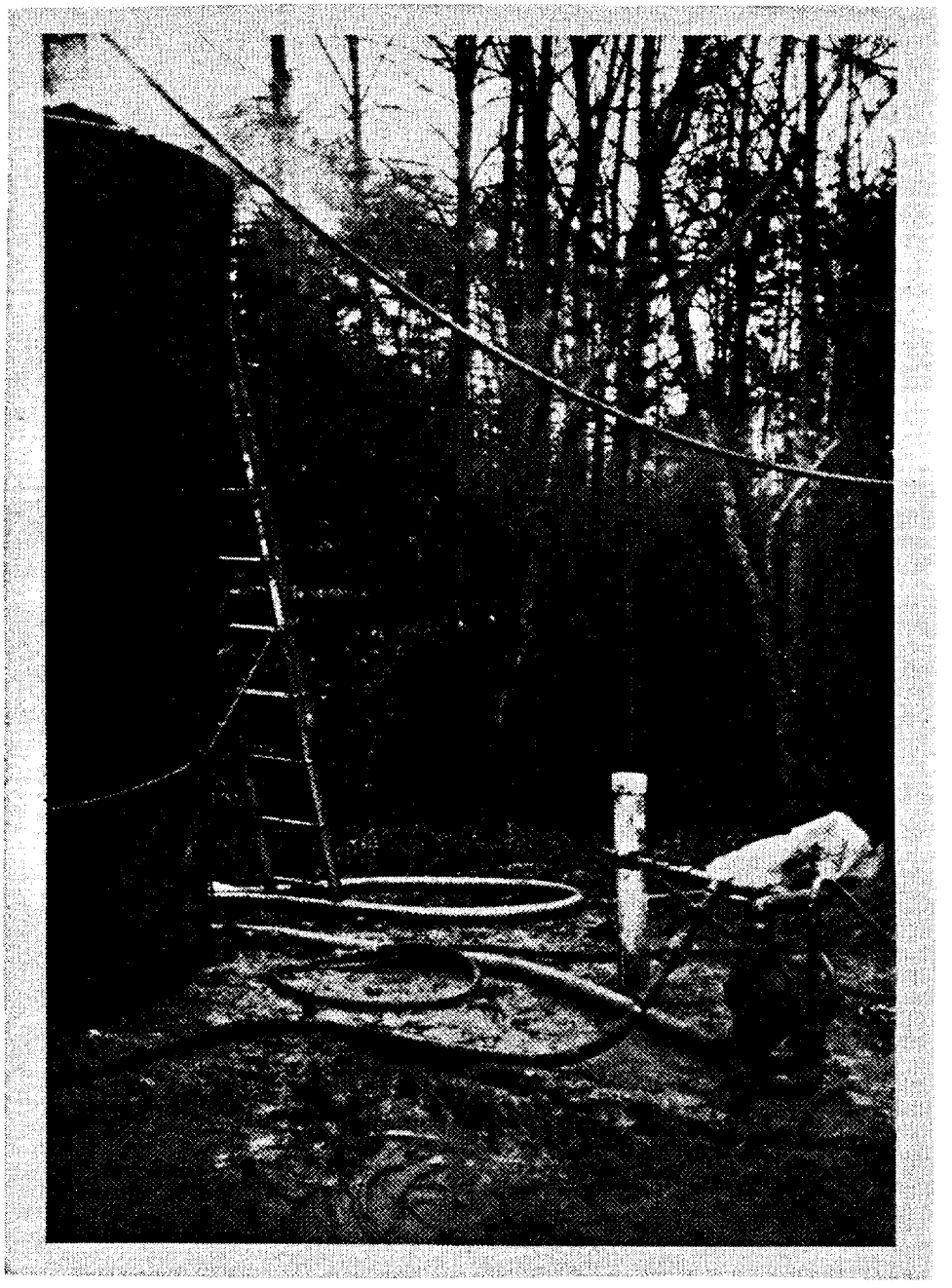

Fig. 7 Grout mixing and holding tank with recirculating pump at the trench 7 site, January 1986.

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Fig. 8 Grout delivery plumbing used on trench 7 site showing valves, pressure gauges, and quick-connect fittings.

return would occur around the pipe. Lifting apparently compromised the frictional seal between pipe and soil formation.

Grout injection was carried out through standard 1-in. carbon steel pipe. This pipe was procured in 10-ft sections which were coupled after driving each 10-ft section until refusal. Initially, the contractor tried to drill the pipe into the ground by means of an air-driven motor. The bottom of each pipe was fitted with a friction-held penetrometer point, a solid steel 60° cone slightly larger in diameter than the pipe with a short section of steel rod to fit into the end of the pipe. When pipe was withdrawn after completing grout injection, the point remained in the ground. A more successful method of pipe insertion used a 140-lb air-driven hammer. An operator would stand on the bucket of the front-end loader and would lift the hammer over the top of a 10-ft pipe section, lowering the bucket as required to maintain penetration. Another pipe section was then added, and the process was repeated until refusal. Refusal was operationally defined as no observed penetration during 1 min of hammering. Typically, an array of 15 to 20 injection pipes were inserted before a batch of grout was injected (Fig. 9). If grout acceptance was low for the whole array of injection pipes, grout was saved until another array was constructed. Grout could be saved for several weeks without setting. Injection pipe was withdrawn by means of a fork-lift attachment to the front-end loader (Fig. 10). A quick-disconnect clamp was used to grasp the pipe with closely spaced forks on the lift. Injection pipe was occasionally reused, but this withdrawal method often bent the pipe to such a degree that it could not be reused. All pipe was disposed of by the contractor after a survey by Oak Ridge National Laboratory Health Physics personnel.

4. FIELD GROUTING RESULTS

As mentioned earlier, the grout injection line covered a 500-ft length on the eastern and northern site access road. These 300 injection points are depicted in Fig. 11. A complete tabulation of these locations and grout injection parameters are given in Appendix B. The depth of injection pipe penetration was extremely variable. Figure 12 depicts the average depth of three grout injection pipe penetrations every 5 ft along the grout injection line. If this depth of refusal has a direct relation to depth to hard rock (i.e., limestone layers), then the site is quite discontinuous. This interpretation is consistent with the geologic exposure along the road cut as studied by Olsen et al. (1983). There was an outcrop of hard rock at or near the road surface between 180 and 210 ft. Hard rock was generally shallower near the beginning of the grout line near White Oak Creek floodplain. This penetration information should prove extremely valuable in characterization of the trench 7 site.

The information on grout "takes" of these various injection holes is depicted in Fig. 13. In this plot, the location of each injection is at the center of a circle, and the area of the circle is proportional to the volume of grout injected at that point. Grout injection volumes ranged from 0 to 3800 gal for the largest circle in Fig. 13. Also depicted in Fig. 13 are the three grout line locations selected for soil coring to verify grout penetration.

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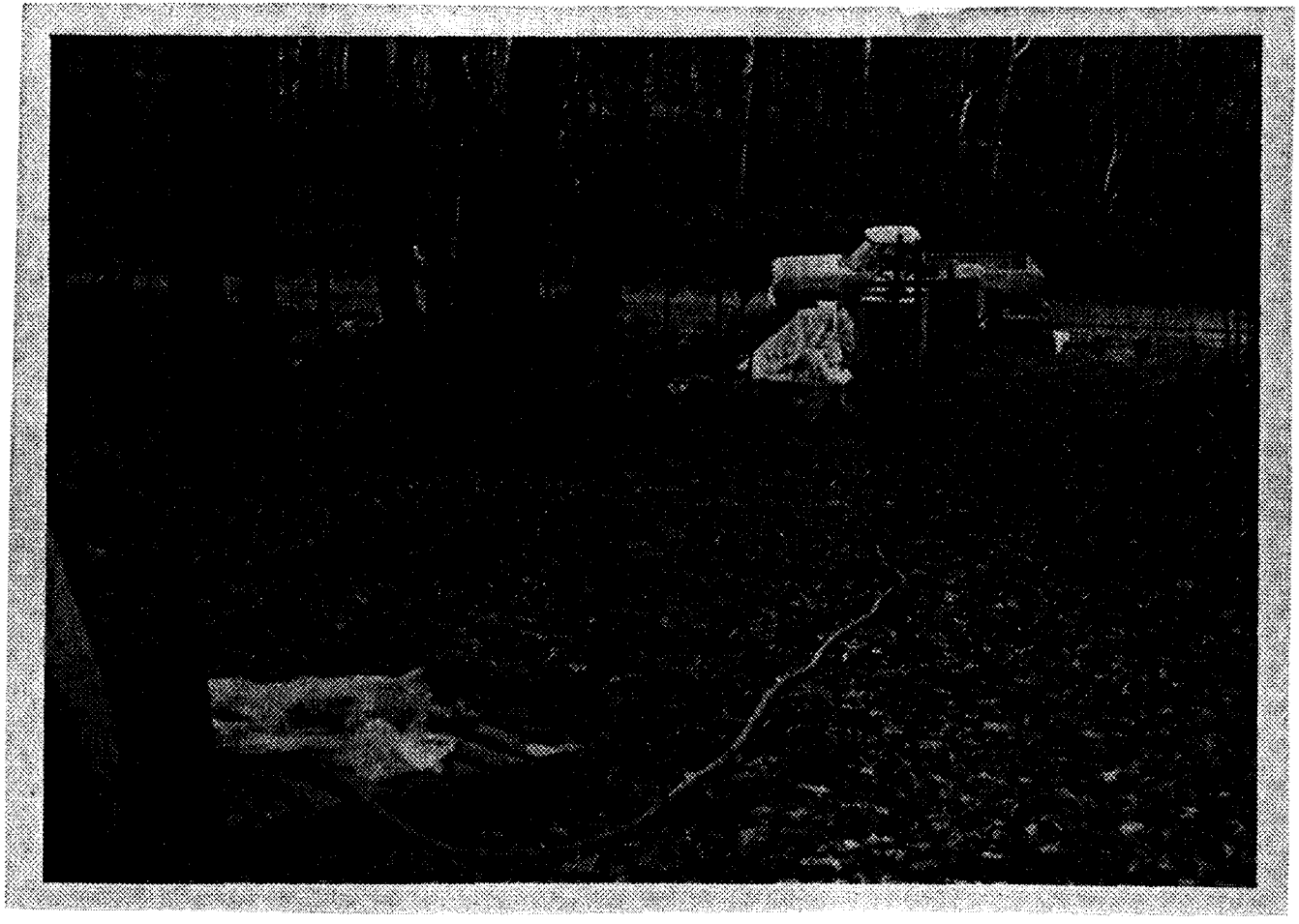

Fig. 9. Array of injection pipes place into ground just north of station 0+00 on November 20, 1985.

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Fig. 10. Grout injection pipe removal method at the trench 7 site, January 1986.

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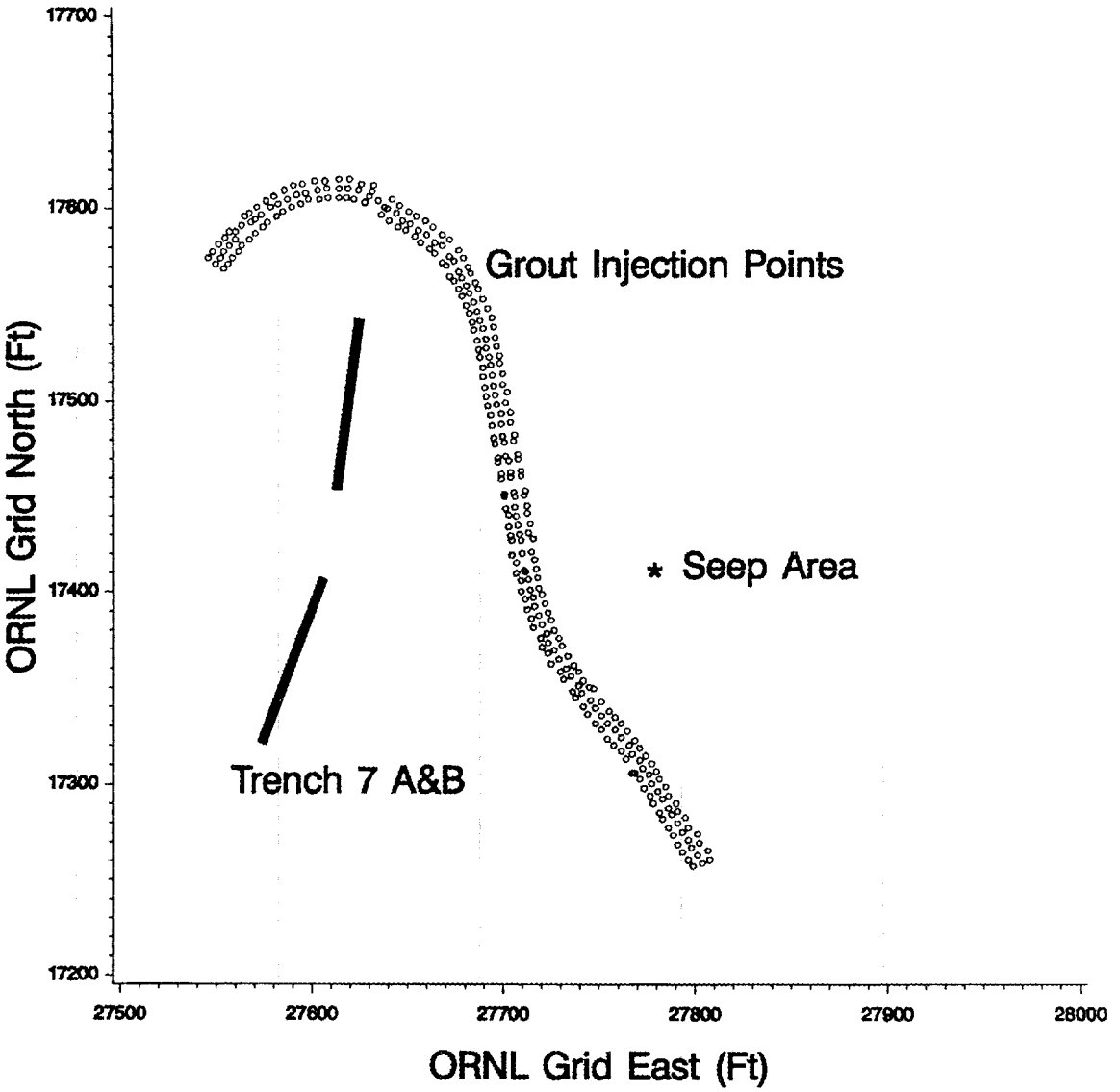

Fig. 11. Grout injection point locations at trench 7 site.

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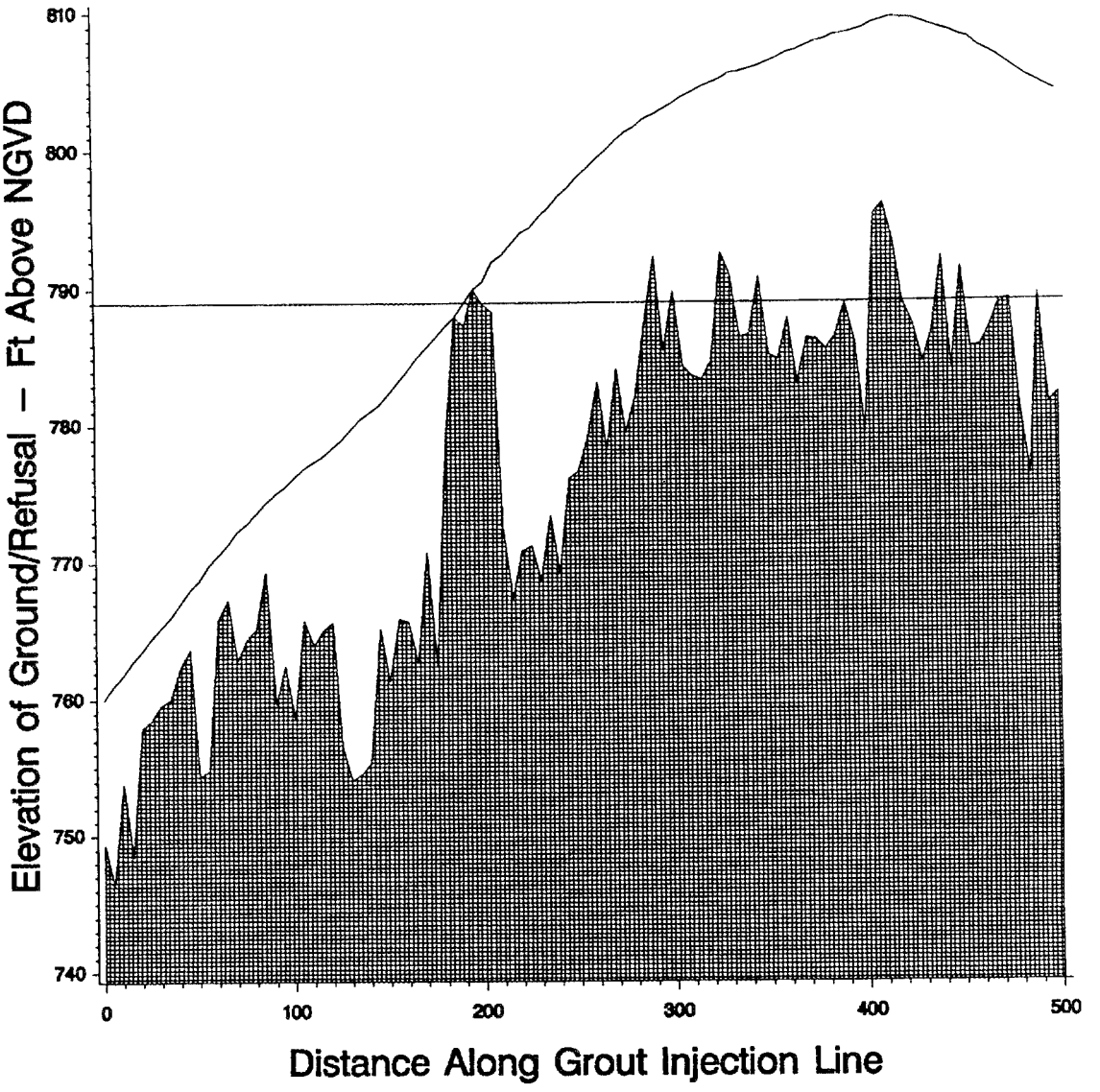

Fig. 12. Average depth of injection pipe penetration at 101 stations on the perimeter line to the east and north of trench 7. Each point represents the average data from three penetrations at each station.

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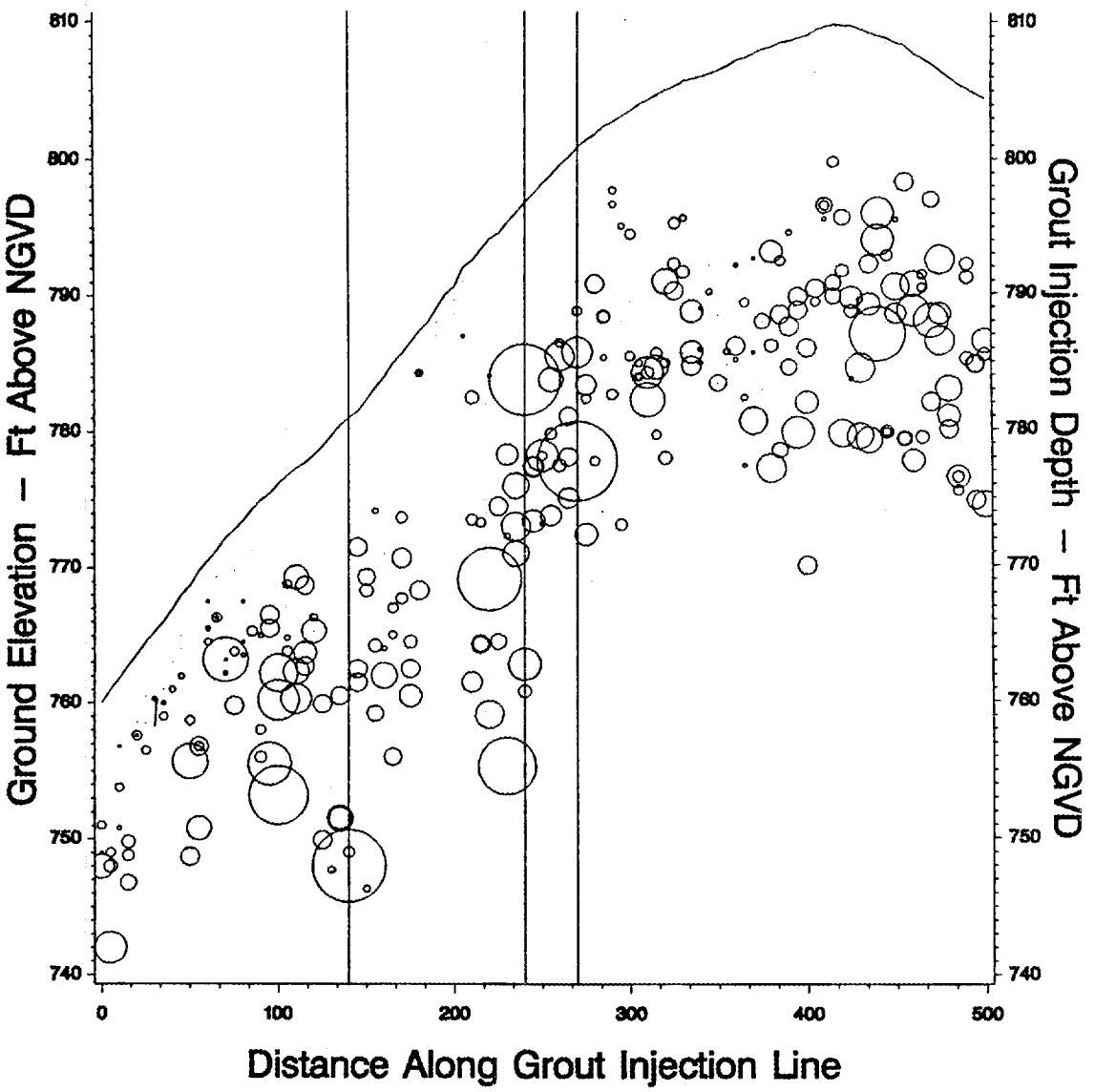

Fig. 13. Elevations of grout injections relative to ground elevations to the east and north of trench 7. The size of each circle is proportional to the volume of grout injected at that point. Vertical lines indicate site for core sampling after grouting.

Because of the randomness of grout "takes" at various points and elevations, a description of an average grout injection would offer some interpretative value in planning future grout injection projects. The average grout injection occurred at 16.8 ft below ground, where 215 gal of grout was injected at an average rate of 9 gal/min and an average maximum pressure of 42 psi. There was no significant correlation of grout volume with any measured parameter, including depth and location. The frequency distribution of grout injection volumes is perhaps the most useful description of the grouting information (Table 1). Because the median grout volume was just under 100 gal, there would be only a 50% probability that an injection hole would accept more than a 100 gal of grout. Similarly, about 20% of the time one could expect little or no take from a grout hole (i.e., 10 gal or less). Likewise, only 3% of the time could one expect to inject more than 1000 gal of grout. Such information should prove valuable in planning other grouting projects, particularly if these results on fluid acceptance are applicable to other types of grout.

Several grout breakthroughs were observed during and after grouting operations at the site. Two possible breakthroughs of grout were noted in the area of the contaminated seep which, if grout had arrived via subsurface contaminated conductive features in the soil, would have been most encouraging. However, both these breakthroughs in the seep area proved to have originated via overland flow just beneath the leaf litter layer in the area. When the leaf litter was removed, a grout-stained path could be traced back to the road cut. The most common form of grout refusal was a return at the injection pipe or a nearby injection pipe. Grout injection was stopped whenever such break through was observed. Occasionally, grout would break through in the drainage ditches along the road. Notes on the location of all breakthroughs are given in Appendix B. Several monitoring wells were equipped with continuous water level recorders during grout injection and the responses to nearby injections were noted (Fig. 14). The bottom elevations of three wells (T7-24, SB-1, and SB-2) actually increased after grouting operations because of presence of grout solids. Chemical indications (mostly high pH) were still present in these wells when sampled over 1 year later. Grout was observed flowing out the top of Well SB-1 during nearby injections. Grout also broke through the soil surface at a nearby soil hydrologic study pit (Luxmoore 1982) just to the east of the road cut. Apparently, grout intercepted a access tube installed within the monolith and flowed up the aluminum pipe and overflowed within the concrete enclosure. This indicated a lateral subsurface grout migration of about 20 ft from the closest injection point.

5. GROUT SPECIFICATION EVALUATION

Several specimens of grout injection batches were taken during the 3 months of grouting operations at the site. In the year and a half during which these grout samples have been stored, none has formed a solid having detectable compressive strength. Samples of the fly-ash were also collected from the bulk pile stored at the site. Fly ash from the site contained 9.8% moisture, and 99.5% passed through

Table 1. Frequency distribution of grout volumes injected at 303 locations on the eastern and northern perimeter of trench 7 site.

Gallons injected	Frequency	Percentage	Cumulative frequency	Cumulative percentage
0	44	14.5	44	14.5
5	12	4.0	56	18.5
6	1	0.3	57	18.8
8	1	0.3	58	19.1
10	14	4.6	72	23.8
12	1	0.3	73	24.1
15	3	1.0	76	25.1
20	10	3.3	86	28.4
25	5	1.7	91	30.0
26	2	0.7	93	30.7
30	5	1.7	98	32.3
40	9	3.0	107	35.3
50	20	6.6	127	41.9
55	1	0.3	128	42.2
60	6	2.0	134	44.2
70	4	1.3	138	45.5
75	8	2.6	146	48.2
80	4	1.3	150	49.5
90	1	0.3	151	49.8
100	21	6.9	172	56.8
130	1	0.3	173	57.1
140	2	0.7	175	57.8
150	10	3.3	185	61.1
170	1	0.3	186	61.4
175	3	1.0	189	62.4
200	35	11.6	224	73.9
210	1	0.3	225	74.3
215	1	0.3	226	74.6
220	2	0.7	228	75.2
225	1	0.3	229	75.6
250	6	2.0	235	77.6
275	1	0.3	236	77.9
285	1	0.3	237	78.2
300	14	4.6	251	82.8
350	6	2.0	257	84.8
360	1	0.3	258	85.1
400	11	3.6	269	88.8
450	3	1.0	272	89.8
475	1	0.3	273	90.1
500	6	2.0	279	92.1
550	1	0.3	280	92.4
555	1	0.3	281	92.7
600	7	2.3	288	95.0
625	1	0.3	289	95.4
700	2	0.7	291	96.0
740	1	0.3	292	96.4
850	1	0.3	293	96.7
1000	1	0.3	294	97.0
1100	1	0.3	295	97.4
1200	1	0.3	296	97.7
1800	1	0.3	297	98.0
1950	1	0.3	298	98.3
2050	1	0.3	299	98.7
2400	1	0.3	300	99.0
3000	1	0.3	301	99.3
3175	1	0.3	302	99.7
3800	1	0.3	303	100.

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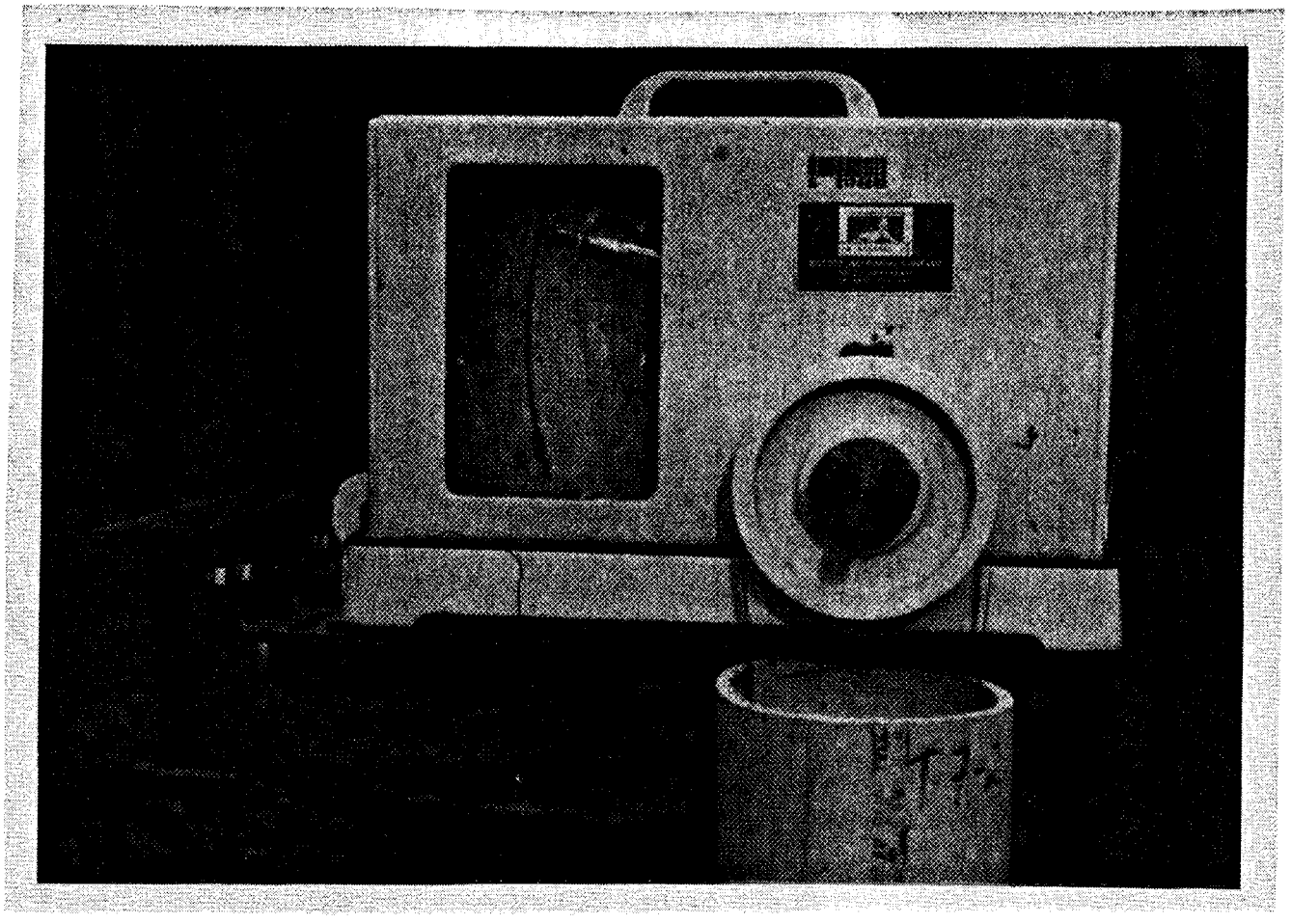

Fig. 14. Continuous water level recorder on well T7-21 showing response to nearby grout injection in January 1986.

a 2-mm sieve. These fly-ash samples were tested for pozzolanic activity via ASTM Method C593: Standard Specification for Fly Ash and Other Pozzolans for Use with Lime. In this procedure, specimens for compressive strength testing are prepared from hydrated lime:fly ash:graded standard sand in the ratio 1:2:8.22 with a minimum of make-up water to a thick consistency.

Unconfined compressive strength of the lime-fly ash grout specimens was determined using a Karol-Warner, Inc., model KW 567, unconfined compression tester equipped with a model KW 2500 R load ring capable of delivering a maximum of 2500 lb to a single specimen. The load to a specimen was increased uniformly over a 30-s interval until the cylinder specimen cracked, indicating that the unconfined compressive strength had been exceeded. The load causing a specimen to fail was divided by the top surface area of the specimen and calculated as a compressive strength in pounds per square inch. Three samples were prepared using the contractor's fly ash; they exhibited a mean compressive strength of 1152 psi with a standard deviation of 187 psi. A second set of three samples was prepared using a Kingston fly-ash sample, used in ORNL hydrofracture grout formulations, and was developed at the same time as the first set. The Kingston fly ash yielded a grout with a mean compressive strength of 1043 psi and a standard deviation of 164 psi. Thus, the fly-ash used for the trench 7 grouting operation was pozzolanically active and within specifications (i.e., 600 psi). Six specimens of grout, sampled from different batches prepared by the contractor, were subjected to the same 54°C curing period as the fly ash samples prepared via ASTM C 593. However, none of these field-collected samples exhibited any detectable compressive strength. In contrast to the ASTM procedure, the field grout was specified to be hydrated lime:fly ash of 1:3 weight ratio with a large excess of water (i.e., 1 to 3 gal of water per 4 lb of solids). The excess of water may have dissolved and/or diluted the slightly soluble hydrated lime (calcium hydroxide) to a concentration that was no longer pozzolanically active.

Several samples of field-collected grout and laboratory-prepared grout (made to the grouting contract specifications) were placed in glass columns in the laboratory and, after various curing periods, subjected to measurements of hydraulic conductivity (Spalding 1985). None exhibited a hydraulic conductivity of less than 10^{-3} cm/s. This indicates that the grout formulation used at the trench 7 site possessed little potential for improving hydrologic isolation of contaminated layers within the soil formation. A typical bulk hydraulic conductivity for the soil formation in Melton Valley would be about 1×10^{-5} cm/s (Spalding 1985). Although it is recognized that secondary porosity (i.e., that as (i.e., that associated with fractures and other geologic structures) probably contributes most to the bulk hydraulic conductivity, the filling of such features with a grout of such inherently large permeability could not be expected to seal up relict contamination layers.

6. GROUT DISTRIBUTION WITHIN THE SOIL FORMATION

As can be seen in Fig. 13, injected volumes of grout were distributed rather randomly with respect to depth and position along the perimeter road. The goal of this in situ grout campaign was to fill relict contaminated layers at the site. The rationale for the soil core sampling effort was to obtain core from strata that contained radioactivity and determine if they also contained grout. As depicted in Fig. 13, three coring sites were selected at the 140-, 240-, and 270-ft marks of the grout injection line. Each of these locations exhibited at least one injection point with a large grout acceptance (i.e., 3175, 3000, and 3800 gal, respectively). In addition, these locations were known to contain significantly contaminated strata; Olsen et al. (1983) reported gamma activity log profiles for monitoring wells T7-21 and T7-25 near these locations with counting rates in the 5000 to 10000 cps range at depths between 19 and 39 ft, respectively. Thus, these locations seemed the most probable to encounter samples containing grout mixed with residual contamination.

Between December 11, 1986, and January 12, 1987, the collection of soil cores was attempted at these three locations. A variety of sampling devices and techniques were required to obtain samples from the extremely variable materials encountered. Initially, a 3-in. diam corer was used with power rotation, but this sampling technique failed to yield an intact core. Subsequently, Shelby or thin-walled tubes (2-in. diam x 30-in. long) were most often used; these were hydraulically pushed into the ground with a minimum of rotation when necessary. Occasional pounding with a 140-lb safety hammer was attempted, but the Shelby tubes tended to collapse under such stress. A standard 2-in. diam split-spoon sampler was used when the formation became too hard for Shelby tubes; this was placed into the formation by hammering. Occasionally, a 3-in. diam x 60-in. split-barrel sampler was employed in combination with hollow-stem auger to penetrate particularly dense or binding material. Sampling was carried out to refusal, which was defined as no penetration with hammering either the split-spoon sampler or Shelby tube. The core hole at the 140-ft distance along the grout line did not exhibit refusal. A free-flowing mud was encountered around the 30-ft depth, and not enough hollow-stem auger was available to hold this hole open at this depth to continue sampling.

The coring logs for the three holes are presented in Tables 2 through 5. Core hole 4 was placed next to core hole 2 to obtain a depth section between 13 and 19 ft which had been lost on core hole 2. A tool broke loose in hole 2 at this depth, and retrieval required this depth interval to be augered; thus, no core was obtained.

The most important observation from these coring results was that grout was found in only three thin-bedded layers in the total length of core obtained. This result was a little surprising because, even if grout was present only along bedding planes in the formation, the large number of injections at varied depths around these coring locations would lead one to expect that grout layers would be encountered much more frequently. In addition, none of these grout ribbons nor

Table 2. Trench 7 grouting investigation, core hole 1 sampling log.

(140-ft station on grout line.
Surface elevation = 781.0 ft)

Section[a]	Date	Depth interval (ft)	Core length (in)	Device	Method
1	12/11/86	0.5-2.0	19.0	3.5-in. corer	Rotated
2	12/11/86	2.0-3.5	19.0	3.5-in. corer	Rotated
3	12/11/86	3.5-5.5	22.5	Shelby tube	Pushed
4	12/11/86	5.5-7.5	24.0	Shelby tube	Pushed
5	12/15/86	7.5-8.6	15.8	Shelby tube	Pushed
6	12/15/86	8.6-9.5	13.3	Shelby tube	Pushed
7	12/15/86	9.6-10.5	11.8	Shelby tube	Pushed
8	12/15/86	10.5-11.5	12.3	Shelby tube	Pushed
9	12/15/86	11.5-12.5	11.8	Shelby tube	Pushed
10	12/15/86	12.5-13.5	11.3	Shelby tube	Pushed
11	12/15/86	13.5-14.5	10.5	Shelby tube	Pushed
12	12/15/86	14.5-15.5	12.3	Shelby tube	Pushed
13	12/15/86	15.5-16.5	12.8	Shelby tube	Pushed
14	12/16/86	16.5-18.5	27.3	Shelby tube	Pushed
15	12/16/86	18.5-20.0	18.5	Shelby tube	Pushed
16	12/16/86	20.0-21.8	27.8	Shelby tube	Pushed
17	12/16/86	21.8-23.4	27.3	Shelby tube	Pushed
18	12/16/86	23.4-25.1	27.0	Shelby tube	Pushed
19	12/16/86	25.1-25.5	27.3	Shelby tube	Pushed
20	12/16/86	25.5-26.3	27.3	Shelby tube	Pushed
21	12/17/86	26.3-26.4	27.3	Shelby tube	Pushed
22	12/17/86	26.4-27.7	12.3	Shelby tube	Pushed
23	12/17/86	27.7-28.3	8.3	Shelby tube	Pushed
24	12/17/86	28.3-28.7	27.3	Shelby tube	Pushed
25	12/17/86	28.7-30.4	27.3	Shelby tube	Pushed
26	12/18/86	30.4-30.4	0.0	Shelby tube	Hammer
27	1/06/87	26.0-30.2	50.0	3-in. split barrel	Hammer

[a]Section Notes: (1/2)-Intact core extrusion impossible, switched to 2-in. Shelby tubes. (4) Shelby tube fell off in hole. Used 3.5-in. sawtooth corer to retrieve, and lost core section 77-83 in. (12) Top 3 in. soft and wet. (13) Standing water on core. (14) On first recovery attempt, very soft core fell out of tube, went deeper and retrieved core. (15) Soft muddy section with standing water. (16) Sandy light colored material. (17) Sandy light colored material. (18) 14 in. of saturated sandy material on top of core. (19) 6-in. section of core with 21 in. of saturated sandy mud on top. Approximately 0.25 in. band of grout at 25.5 ft depth. Augered out hole to 25.5 ft and cased with 3 in. PVC pipe. (20) 8-in. of new core, remainder fall in material. (21) 1 in of new core, remainder fall in material. (22/23) Water in core barrel. (24) Solid core again. (25) 5-in. solid core, remainder was mud. (26) After failing to penetrate by pushing, hammering was tried. However, Shelby tube broke off in the hole and was retrieved. On a second attempt, the Shelby tube crumpled in hole. (27) Free flowing mud, mostly fall in material.

Table 3. Trench 7 grouting investigation, core hole 2 sampling log.

(240 ft station on grout line
Surface elevation = 796.8 ft

Section[a]	Depth Date	Core Interval (ft)	Length (in)	Device	Method
1	12/18/86	1.0-2.4	18.8	Shelby tube	Pushed
2	12/18/86	2.4-4.4	8.8	Shelby tube	Pushed
3	12/18/86	4.4-6.0	17.8	Shelby tube	Pushed
4	12/18/86	6.0-6.8	12.8	Shelby tube	Pushed
5	12/22/86	6.8-8.9	15.0	Split spoon	Hammer
6	12/22/86	8.9-10.9	17.0	Split spoon	Hammer
7	12/22/86	10.9-11.6	15.8	Shelby tube	Pushed
8	12/22/86	11.6-12.9	14.3	Shelby tube	Pushed
9	12/22/86	12.9-13.9	10.0	Shelby tube	Pushed
10	12/29/86	13.9-19.1	19.0	60-in-split auger+barrel	Rotate
11	12/29/86	19.1-19.5	27.0	auger+barrel	Rotate
12	12/30/86	19.5-22.3	31.5	auger+barrel	Rotate
13	12/30/86	22.3-22.5	29.0	auger+barrel	Rotate
14	12/30/86	22.5-23.3	0.0	auger+barrel	Rotate
15	12/31/86	23.3-25.4	25.8	Shelby tube	Pushed
16	12/31/86	25.4-27.4	17.5	Shelby tube	Hammer
17	01/05/87	27.4-29.5	26.3	Shelby tube	Hammer
18	01/05/87	29.5-29.5	0.0	Shelby tube	Hammer

[a]Section Notes: (1) Augered to 12 in. to get through roadbed gravel. (2) Difficult to pull Shelby tube out. (3) Augered with 6-in. hollow stem to depth of 50 in. to hold hole open. (4) Very sticky, clayey material binding Shelby tube in hole. (5) Switched to split spoon sampler to avoid Shelby tube binding problem. (6) Augured to 10-ft depth with Shelby tube in hollow stem to hold hole open. (7) Fall in material on top 6 in. (9) Lost Shelby tube in hole and retrieved with hollow stem auger. (10) Shoe screwed off sampler in the hole and lost much core section due to disturbance by 'fishing' tools down hole. (11) The split barrel shoe was set approximately 1 in. below auger bit, and core was taken while rotating auger. (13) After retrieving section 13, hole was cleaned out by pulling augers. (14) Lost shoe on split barrel sampler and some core during fishing operation in the hole. (15) switched back to Shelby tubes to avoid continuing problem with shoe rotating off (16) Soft material which read 0.1 mrem/h. (17) Shelby tube deformed slightly by hammering so that core was not extrudable. Read 0.2 mrem/h. (18) Shelby tube collapsed to 11 in.

Table 4. Trench 7 grouting investigation, core hole 3 sampling log.

(270-ft station on grout line.
Surface elevation = 800.8 ft)

Section[a]	Date	Depth Interval (ft)	Core Length (in)	Device	Method
1	01/06/87	1.0-3.2	26.8	Shelby tube	Pushed
2	01/06.87	3.2-5.0	21.8	Shelby tube	Pushed
3	01/06/87	5.0-7.0	23.8	Shelby tube	Pushed
4	01/06/87	7.0-9.0	24.0	Shelby tube	Hammer
5	01/07/87	9.0-11.0	17.0	Split spoon	Hammer
6	01/07/87	11.0-13.0	15.3	Split spoon	Hammer
7	01/07/87	13.0-15.0	16.3	Split spoon	Hammer
8	01/07/87	15.0-17.0	17.8	Split spoon	Hammer
9	01/07/87	17.0-19.0	17.3	Split spoon	Hammer
10	01/07/87	19.0-21.0	20.3	Split spoon	Hammer
11	01/07/87	21.0-23.0	21.3	Split spoon	Hammer
12	01/07/87	23.0-25.5	16.8	Split spoon	Hammer
13	01/07/87	25.5-29.8	18.8	Split spoon	Hammer
14	01/07/87	29.8-32.9	26.3	Split spoon	Hammer

[a]Section notes: (1) Augered to 1-ft level before sampling to get through roadbed gravel. (2/3) Rotation required to get into material at end of core. (4) Switched to hammer because sampler would not push in. (5) Switched to split spoon sampler because Shelby tubes tended to collapse when hammering on previous holes. (12) Lower half of this section was soft material. (13) Because the driving was easy, an attempt was made to drive to firm material to seat core in sampler shoe, but after driving 52 in., no firm material was encountered. Soft material must have been pushed to side of hole during driving, since only 18.8 in. of core was recovered but the hole was open to the indicated depth. (14) Hard driving at bottom of section and observed rock in sampler shoe indicating bottom of residuum. Noted 1-ft of water in bottom of hole 15 min after removing last core section. Material read 0.1 mrem/h.

Table 5. Trench 7 grouting investigation, core hole 4 sampling log.

(240-ft Position on grout line
(14 in. east of core hole 2)
Surface elevation = 796.8 ft)

Section[a]	Date	Depth Interval (ft)	Core Length (in)	Device	Method
1	01/08/87	0.9-3.4	16.3	Split spoon	Hammer
2	01/08/87	3.4-5.9	26.3	Split spoon	Hammer
3	01/08/87	5.9-8.1	20.3	Split spoon	Hammer
4	01/07/87	8.1-10.3	24.3	Split spoon	Hammer
5	01/07/87	9.3-11.5	21.8	Split spoon	Hammer
6	01/07/87	11.5-13.7	23.3	Split spoon	Hammer
7	01/07/87	13.7-15.9	20.3	Split spoon	Hammer
8	01/12/87	15.9-18.0	19.3	Split spoon	Hammer
9	01/12/87	18.0-20.0	26.3	Split spoon	Hammer
10	01/12/87	20.0-21.4	22.3	Split spoon	Hammer
11	01/12/87	21.4-21.9	20.3	Split spoon	Hammer
12	01/12/87	21.9-24.0	24.0	Split spoon	Hammer

[a]Section notes: (1) Augered through 11 in. of roadbed gravel before beginning sampling. (3) Hole appeared to be going off plumb back towards core hole 2. Attempted to straighten hole by running cored up and down hole. (4) After sampling, the hole was augered out to 9.3 ft to straighten out. (5) Approximately 11 in. of fill and cave in material was on top of undisturbed core. (6) Hard driving and binding of sampler in hole. On bumping out, core disrupted within sampler. (7) Grout in sampler shoe at 15.8 ft. (8) 0.25 in. thick grout layer about 12 in. below top of core or 16.9 ft depth. (10) Gravel layer 2.5 in. thick at 20.6 ft. (11) Very hard driving, about 30 blows per inch and much gravel and rock in section. (12) Hard driving at first, then easier later. Open hole measured 23.5 ft.

any of the various grout remnants at surface returns around the area has exhibited any firmness. Thus, like the field-collected specimens of grout whose cure was attempted in the laboratory, the grout in the soil cores had not set in the field either.

Where did 65,500 gallons of grout go? First it must be realized that the grout formulation that was injected ranged from 86% water (by weight) for the 3:1:3 formulation to 81% water for the 2:1:3 formulation. By volume, water probably composed somewhere between 75 and 50%, respectively, for these formulations. This volume percent estimate is based on qualitative observations of excess water in grout sample containers after allowing solids to settle under gravity. Thus, the question reduces to "Where did about 20,000 gal of wet solids go after excess water drained away?" A 1/4 in.-thick band would require about 7.5 ft^2 of area per gallon of solids. Thus, a total area of grout band of about 2700 ft^2 (i.e., a 100 x 30 ft area around the grout line) would be required to accommodate this much grout. This sort of a distribution would seem consistent with the frequency of grout observed in the coring operation.

However, the frequency of grout encountered in the cores would not lend much confidence to the assertion that the conducting features of the soil formation had been sealed. It is also notable that the layers of core, which did contain radioactivity, did not contain any discernible grout. Another possible sink for the injected grout would be the frequently encountered free-flowing mud at various depths. It should be emphasized that this mud was not the result of the grouting operation because its occurrence had been reported during logging of various boreholes at the site in 1981 (Olsen et al. 1983). This mud would not be expected to contain the grout in discrete layers and, should any mixing have occurred, grout identification would not be possible by visual inspection. Thus, a potential brown hole for grout exists in the mud beneath the trench 7 site.

With such problems, one is led to ask why was a lime and fly-ash grout selected by ORNL for this project? The evolution of the project probably bears strongly on how this decision was made. On September 8, 1983, Woodbine Corporation, Fort Worth, Texas, performed a demonstration of lime-fly-ash grouting in Solid Waste Storage Area 6 using their patented hydraulic lance grout injection equipment (Blacklock et al. 1982). The facility with which these lances were inserted into waste burial trenches was quite impressive. The technique seemed to offer a safe and economical method for in situ stabilization of buried waste. A resulting infatuation with the potential of this equipment probably led to the specifications for the trench 7 corrective actions project (Appendix A), which had been given a high priority within the waste management program. Several options were considered for corrective actions besides grout injection (Stansfield, Appendix C). However, because Woodbine Corporation specializes almost exclusively in the use of lime and fly ash grout, the contract was prepared to accommodate this limitation. However, Woodbine Corporation failed to bid on the contract, and Rembco Engineering was the only other bidder. In an effort to complete the project, apparently little consideration was given to specifying grout properties such as setting time and compressive strength. Rembco Engineering

was judged to be capable of preparing a grout as specified in the contract, and once a variance was granted for using injection equipment other than the hydraulic lance system, the contract was awarded. It will be important in future grouting contracts to specify desired grout characteristics such as set time, strength, and/or hydrologic properties as well as grout composition and injection machinery.

7. CHEMICAL AND RADIOCHEMICAL CHARACTERISTICS

On May 7, 1987, all observation wells were sampled in the trench 7 environs. Samples were returned to the laboratory and filtered through 0.45-μm filters. Analyses of pH, electrical conductivity, dissolved solids, hardness, and alkalinity were performed as described previously (Spalding 1984a). A 5-mL aliquot of the sample was dried down on a 2-in. ringed steel planchet and counted for gross alpha and beta activities as described elsewhere (Spalding 1984a). A 20-mL aliquot was placed in a plastic scintillation vial and counted for ^{90}Sr via Cerenkov radiation detection in a liquid scintillation counter (Spalding 1984b). Activities of ^{60}Co and ^{137}Cs were determined by gamma scintillation detection as described elsewhere (Cerling and Spalding 1981). Results of these determinations are presented in Table 6.

Several influences of the injected grout's alkalinity could still be seen in the chemical characteristics on this sampling date over a year after the last grout injection. Samples from wells T7-21, T7-24, and SB-1 exhibited pHs above 9.0. The excess water in the collected grout samples exhibited a pH of 12.2, an electrical conductivity of 11,000 dS/m, and an alkalinity of about 3000 mg $CaCO_3$/L; this is typical of a saturated solution of hydrated lime, $Ca(OH)_2$, which was used in the grout. It should be noted that two of these three wells (T7-24 and SB-1) contained residual grout, which has caused the apparent casing bottom elevation to rise. In addition, well T7-21 showed facile water level fluctuations during nearby grout injections (Fig. 14). Thus, the influences of grout alkalinity on these wells was not too surprising. Other well samples did not show considerable difference from chemical characteristics reported by Olsen et al. (1983). Radionuclides in these well water samples have shown a general decline (Table 7) since last investigated in 1981-1982 by Olsen et al. (1983). Samples were collected from all monitoring wells immediately before grouting operations began in December of 1985 but were inadvertently discarded before analysis. Thus, it is not possible to determine more immediate effects of grouting on radionuclide concentration in area groundwaters. However, a significant influence would seem unlikely because of the limited volume of soil formation actually contacted by grout. The decline in ^{60}Co concentrations between 1982 and 1987 in most wells is too large to be accounted for by radioactive decay alone.

It would seem unlikely that the grout could have any effect on the activity of ^{60}Co adsorbed to soil at the seep on the eastern side of the site. Nine soil samples were collected with a hand-operated soil corer to a depth of 12 in. at 3-ft intervals along a semicircle around the seep on May 7, 1987. The activity of ^{60}Co was

Table 6. Groundwater quality of samples from wells in trench 7 area on May 7, 1987.

Sample	Conductivity (DS/M)	pH	Alkalinity (mg $CaCO_3$/L)	Hardness (mg $CaCO_3$/L)	TDS (mg/L)	Gross beta (Bq/L)	Gross alpha (Bq/L)	^{137}Cs (Bq/L)	^{60}Co (Bq/L)	^{90}Sr (Bq/L)
T7-1	320	8.0	139	220	140	47	ND[a]	ND	ND	ND
T7-2	177	7.7	101	130	240	3140	ND	ND	3	1090
T7-3	1050	8.5	338	102	820	666	1.6	1.3	713	ND
T7-4	1520	7.8	87	90	1220	1920	24.9	22.7	1880	22.0
T7-5	665	7.5	71	112	540	1360	22.6	ND	1110	46.6
T7-6	198	7.5	104	138	140	7	ND	ND	6.6	ND
T7-7	440	7.4	225	290	340	6070	ND	13.2	4.8	2410
T7-9	135	6.6	43	56	80	33	ND	12.1	46	ND
T7-10	318	7.9	213	212	200	63	ND	8.2	24	ND
T7-11	715	7.9	200	208	540	664	ND	17.4	316	ND
T7-12	810	8.2	226	194	660	491	ND	32.4	432	ND
T7-13	935	8.6	463	44	800	342	ND	ND	173	ND
T7-15	115	7.7	61	90	60	10	ND	ND	23	2.1
SB-20	835	8.6	356	172	760	432	13.8	21.7	317	ND
T7-21	1190	9.3	38	250	1180	169	ND	0.3	521	ND
T7-22	980	8.0	210	80	840	596	ND	7.4	499	ND
T7-23	645	7.4	60	314	640	19	ND	7.8	ND	ND
T7-24	1060	10.4	311	0	800	231	ND	16.5	184	ND
T7-25	395	7.5	104	84	320	169	ND	7.8	35	ND
T7-26	300	7.8	198	230	200	21	ND	5.8	10	5.4
T7-27	355	7.5	179	182	280	55	ND	5.0	2.5	ND
T7-29	315	7.2	190	160	180	13	ND	2.0	ND	0.2
STRM	665	7.7	174	212	520	459	ND	11.4	227	ND
RS-7	710	7.9	329	202	600	447	ND	15.8	290	ND
SB-1	1830	11.2	615	68	1700	42	ND	3.6	ND	1.1
SB-2	480	8.6	240	46	560	22	ND	19.0	ND	ND
SB-4	415	7.8	247	230	280	32	ND	ND	ND	ND
SB-6	275	7.5	181	162	1800.3	59	0.2	8.0	ND	0.1

[a]ND=not detected. Detection limits were 1, 0.1, 0.1, 0.1, and 0.1 Bq/L for gross beta, gross alpha, ^{137}Cs, ^{60}Cs, and ^{90}Sr, respectively.

Table 7. Comparison of ^{60}Co concentrations in monitoring wells in 1981-1982 reported by Olsen et al. (1983) with those observed in May 1987.

Well	1981/1982[a] (Bq/L)	1987 (Bq/L)
T7-2	6	3
T7-3	1075	713
T7-4	5254	1880
T7-5	2291	1110
T7-6	8	7
T7-9	90	46
T7-10	275	24
T7-13	1080	173
T7-15	20	23
T7-20(SB-20)	790	317
T7-21	2040	521
T7-22	1040	499
T7-23	ND[b]	ND
T7-24	1315	184
T7-25	1030	35
T7-26	14	10
T7-27	ND	2
T7-29	ND	ND
SB-1	ND	ND
SB-2	ND	ND
SB-4	ND	ND
SB-6	ND	ND
RS-7(Seep)	880	290
Stream	953	227

[a]Data from Olsen et al. (1983).
[b] ND = Not detected.

determined after drying and sieving these samples to <2 mm by procedures described elsewhere (Cerling and Spalding 1981). The average level of ^{60}Co was 93±25 Bq/g. Olsen et al. (1983) reported a value of 390 Bq/g for a sample taken in 1980. Means et al.(1975) reported a value of 753 Bq/g for a sample taken in summer of 1975, whereas Duguid (1975) reported a value of 511 Bq/g for samples taken in September 1973. Thus, it does appear that ^{60}Co activities in both the seep water and sediment phases have been declining.

8. CONCLUSIONS

Soils in the seepage pits and trenches area, which are situated in the Maryville member of the Conasauga formation, are not amenable to establishing in situ grout curtains, especially with particulate-containing grouts. What little lime-fly-ash grout was accepted by the formation appeared to result from hydrofracturing of discrete strata rather than permeation of the formation.

Future corrective actions, which involve grouting whether applied in situ or otherwise, should require specification of grout properties in addition to any specification of grout materials. Such specifications should include permeability and compressive strength to ensure that injected material will set. Tests to meet these specifications should be defined to simulate the field pumping and soil environment to which the grout will be subjected.

Detailed site characterization information was obtained from the grouting project providing an excellent picture of depth to hard rock around trench 7. Frequency data on grout volumes accepted by the formation and fracturing pressures have provided information on the soil formation properties. These should be useful in anticipating future grouting and hydraulic projects.

Since last studied in 1982, activities of ^{60}Co have declined in groundwater and at the seep in the trench 7 site. This decline is beyond that attributable to radioactive decay.

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Appendix A
Specifications for Subsurface Grout Injection
(from Engineering Drawing C3E20965A011)

Grout shall be injected into the subsurface to fill existing groundwater pathways and to trap contaminants in place. The work shall be performed in accordance with the following conditions and specifications.

1. Existing Subsurface Conditions

The subsurface conditions are described as follows:
Residual clay soils overlie the site and gradually change to weathered rock. The weathered materials extend to depths of 30 to 40 ft. and generally consist of shales, siltstones, and occasional thin beds of nonweathered limestone at deeper depths. The shales and siltstones are so highly fractured that spacing of fractures in these materials is commonly as close as one centimeter. The average strike of the strata in this area is 55 degrees and dip to the southeast (with dips from near horizontal to high angles) and folds and faults are common.

2. Grouting Procedures

Grout shall be injected in two phases along the line shown on drawing C3E-20965-A011. For the first phase injection holes shall be in three(3) rows, five(5) ft apart, along the line at ten(10) ft centers. The second phase shall be along the same centerline as the first phase and at the same spacing;but these hole locations along the centerline shall split the distance between the previously grouted holes. After the two phases of grouting are completed, the grout injections will have been made at locations spaced five(5) feet apart along the centerline in each of the three lines. A total of 303 injection points shall be grouted. Grouting shall begin at station 0+00 as shown on drawing C3E-20965-A011, and proceed to station 5+00.

Grout shall be placed by inserting injection lances hydraulically to a depth of 40 feet or until the lances cease to penetrate further. Grout injection will then begin at this depth. Injection pressures shall be adjusted to inject the greatest quantity of grout possible within a pressure range of 50-200 psi pump pressure. Maximum depth of 40' at any hole.

Injections shall be continued until the maximum quantity of slurry has been injected into the soil, and the slurry is running free at the surface out of previous injection holes or from areas where the surface soil is fractured. The volume of slurry flowing at the surface shall be approximately equal to the volume of slurry pumped into the soil. If required, injection holes shall be added upon approval by the Company.

The maximum penetration depth of each lance at each injection location shall be recorded. Each injection location shall be identified by stationing along the centerline with offsets as appropriate. The volume of grout injected, measured in cubic feet or gallons at each injection location shall be recorded. If an injection is made through three lances simultaneously, measurement of the injected grout through the three lances will be acceptable. The recorded locations and quantities for each injection shall be provided in writing to the Company.

3. Grout Mix

The grout shall be a slurry consisting of clean water, fly ash, hydrated lime and surfactant, and shall be continually agitated to insure uniformity of the mixture. The hydrated lime shall conform to the applicable parts of ASTM C207 Type N. The fly ash shall be Type C and be pozzolanically reactive. A nonionic surfactant shall be used according to the manufacturer's recommendations, but in no case less than 1 gallon per 3,500 gallons of water.

The designed slurry shall be a lime/fly ash slurry mixed in ratio of one pound of lime, three pounds of fly ash, and from one to three gallons of water. The amount of water shall be determined by the seller and be approved by the Company, and shall generally be of such amount as to obtain the maximum quantity of injected grout solids.

4. Special Conditions

A. Stand-by Time

The seller shall be compensated during periods of inactivity only if such periods are directed and authorized by the Company's representative. During such standby periods, the Sellers personnel and equipment shall, if determined to be necessary by the Company's representative, remain at the site ready to resume work. Compensation shall be a the unit price specified as standby time. In no case shall the Seller be paid for more than eight hours standby time in any working day even though multiple shifts are approved by the Company and are in effect. The Seller shall not be compensated for standby time during weekends and Company holidays.

B. Sellers Qualifications

The lime/fly ash injection seller shall at the request of the Company and prior to award of the contract, furnish documentation of satisfactory completion of similar projects in comparable soil conditions.

C. Water Supply

The nearest point of potable water supply is a fire hydrant located approximately 1200 feet from the work site. The location of the fire hydrant is indicated on the locality map, drawing number C3E-20965-A009.

D. Grouting Options

One option for injecting grout on this project is included and is noted as "Option A" on drawing C3E-20965-A011. "Option A" consists of grouting in accordance with these specifications beginning at station 5+00 and ending at station 6+50, 93 injection points.

Appendix B
Trench 7 Grout Injection Information

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
0	CENTER	03DEC85	17258.7	27804.0
0	EAST	03DEC85	17260.3	27807.9
0	WEST	03DEC85	17257.1	27799.2
5	CENTER	25APR86	17262.7	27801.6
5	EAST	25APR86	17265.1	27807.1
5	WEST	25APR86	17260.3	27796.8
10	CENTER	06DEC85	17266.7	27798.4
10	EAST	06DEC85	17269.0	27802.4
10	WEST	06DEC85	17264.3	27793.7
15	CENTER	25APR86	17270.6	27796.8

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
0	CENTER	760.000	11	5
0	EAST	760.000	12	360
0	WEST	760.000	9	40
5	CENTER	761.000	13	100
5	EAST	761.000	19	600
5	WEST	761.000	12	50
10	CENTER	761.800	8	40
10	EAST	761.800	11	10
10	WEST	761.800	5	5
15	CENTER	762.800	14	75

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
0	CENTER	10	10	7.00000
0	EAST	30	50	7.50000
0	WEST	30	50	7.50000
5	CENTER	25	25	8.00000
5	EAST	25	25	8.00000
5	WEST	40	40	8.00000
10	CENTER	30	30	4.50000
10	EAST	0	10	4.00000
10	WEST	20	40	4.50000
15	CENTER	35	35	8.00000

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
0	CENTER	3:1:3	PIPE
0	EAST	3:1:3	HOLE000C
0	WEST	3:1:3	HOLE000C
5	CENTER	2:1:3	PIPE
5	EAST	2:1:3	15FTEAST
5	WEST	2:1:3	4FTNORTH
10	CENTER	3:1:3	PIPE
10	EAST	3:1:3	PIPE
10	WEST	3:1:3	PIPE
15	CENTER	2:1:3	3FTNORTH

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
15	EAST	25APR86	17273.8	27801.6
15	WEST	25APR86	17268.3	27791.3
20	CENTER	06DEC85	17274.6	27793.7
20	EAST	06DEC85	17277.0	27796.8
20	WEST	06DEC85	17273.0	27788.9
25	CENTER	25APR86	17279.4	27791.3
25	EAST	25APR86	17282.5	27795.2
25	WEST	25APR86	17277.0	27786.5
30	CENTER	06DEC85	17284.1	27788.1
30	EAST	06DEC85	17285.7	27791.3

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
15	EAST	762.800	16	150
15	WEST	762.800	13	100
20	CENTER	763.600	6	50
20	EAST	763.600	5	0
20	WEST	763.600	6	5
25	CENTER	764.500	6	0
25	EAST	764.500	4	0
25	WEST	764.500	8	40
30	CENTER	765.300	5	10
30	EAST	765.300	7	0

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
15	EAST	40	40	8.00000
15	WEST	35	35	8.00000
20	CENTER	30	90	4.50000
20	EAST	0	30	4.50000
20	WEST	20	30	4.50000
25	CENTER	35	35	8.00000
25	EAST	30	30	8.00000
25	WEST	40	40	8.00000
30	CENTER	30	30	4.50000
30	EAST	30	90	4.50000

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
15	EAST	2:1:3	11FTSWEST
15	WEST	2:1:3	10FTNEAST
20	CENTER	3:1:3	HOLE020C
20	EAST	3:1:3	PIPE
20	WEST	3:1:3	PIPE
25	CENTER	2:1:3	PIPE
25	EAST	2:1:3	PIPE
25	WEST	2:1:3	2FTSOUTH
30	CENTER	3:1:3	PIPE
30	EAST	3:1:3	PIPE

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
30	WEST	06DEC85	17281.7	27783.3
35	CENTER	25APR86	17287.3	27786.5
35	EAST	25APR86	17289.7	27790.5
35	WEST	25APR86	17284.9	27781.7
40	CENTER	06DEC85	17292.1	27783.3
40	EAST	06DEC85	17293.7	27786.5
40	WEST	06DEC85	17289.7	27778.6
45	CENTER	25APR86	17296.0	27781.0
45	EAST	25APR86	17298.4	27784.9
45	WEST	25APR86	17293.7	27777.0

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
30	WEST	765.300	5	10
35	CENTER	766.000	5	0
35	EAST	766.000	7	40
35	WEST	766.000	6	10
40	CENTER	767.000	4	0
40	EAST	767.000	6	20
40	WEST	767.000	4	0
45	CENTER	768.000	4	0
45	EAST	768.000	3	0
45	WEST	768.000	6	20

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
30	WEST	30	90	4.50000
35	CENTER	40	40	8.00000
35	EAST	40	40	8.00000
35	WEST	30	30	8.00000
40	CENTER	30	30	4.50000
40	EAST	30	120	4.50000
40	WEST	30	30	4.50000
45	CENTER	40	40	8.00000
45	EAST	10	10	8.00000
45	WEST	30	30	8.00000

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
30	WEST	3:1:3	PIPE
35	CENTER	2:1:3	PIPE
35	EAST	2:1:3	PIPE
35	WEST	2:1:3	PIPE
40	CENTER	3:1:3	PIPE
40	EAST	3:1:3	HOLE040C
40	WEST	3:1:3	PIPE
45	CENTER	2:1:3	PIPE
45	EAST	2:1:3	PIPE
45	WEST	2:1:3	PIPE

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
50	CENTER	09DEC85	17300.0	27777.8
50	EAST	06DEC85	17302.4	27781.7
50	WEST	06DEC85	17297.6	27773.8
55	CENTER	25APR86	17304.8	27776.2
55	EAST	25APR86	17306.3	27780.2
55	WEST	25APR86	17302.4	27771.4
60	CENTER	09DEC85	17307.9	27773.0
60	EAST	09DEC85	17310.3	27777.8
60	WEST	09DEC85	17305.6	27769.0
65	CENTER	24APR86	17311.9	27771.4

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
50	CENTER	768.700	13	740
50	EAST	768.700	10	50
50	WEST	768.700	20	200
55	CENTER	769.800	13	50
55	EAST	769.800	13	200
55	WEST	769.800	19	350
60	CENTER	770.500	6	30
60	EAST	770.500	5	10
60	WEST	770.500	3	5
65	CENTER	771.300	5	50

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
50	CENTER	30	130	4.50000
50	EAST	30	90	4.50000
50	WEST	30	130	4.50000
55	CENTER	50	50	8.00000
55	EAST	40	40	8.00000
55	WEST	35	35	8.00000
60	CENTER	30	120	4.50000
60	EAST	30	90	4.50000
60	WEST	30	30	4.50000
65	CENTER	50	50	8.00000

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
50	CENTER	3:1:3	WESTDITCH
50	EAST	3:1:3	WESTDITCH
50	WEST	3:1:3	WESTDITCH
55	CENTER	2:1:3	PIPE
55	EAST	2:1:3	PIPE
55	WEST	2:1:3	PIPE
60	CENTER	3:1:3	ROAD
60	EAST	3:1:3	HOLE060C
60	WEST	3:1:3	PIPE
65	CENTER	2:1:3	PIPE

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
65	EAST	24APR86	17314.3	27774.6
65	WEST	24APR86	17305.6	27767.5
70	CENTER	11DEC85	17315.1	27767.5
70	EAST	11DEC85	17318.3	27771.4
70	WEST	10DEC85	17312.7	27764.3
75	CENTER	24APR86	17319.8	27765.9
75	EAST	24APR86	17322.2	27769.0
75	WEST	24APR86	17316.7	27761.9
80	CENTER	10DEC85	17323.8	27761.9
80	EAST	11DEC85	17327.0	27765.1

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
65	EAST	771.300	5	5
65	WEST	771.300	2	0
70	CENTER	772.200	9	1200
70	EAST	772.200	10	10
70	WEST	772.200	9	5
75	CENTER	772.800	13	200
75	EAST	772.800	9	50
75	WEST	772.800	3	0
80	CENTER	773.500	6	5
80	EAST	773.500	10	10

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
65	EAST	30	30	8.00000
65	WEST	10	10	8.00000
70	CENTER	30	90	4.50000
70	EAST	30	30	4.50000
70	WEST	30	90	4.50000
75	CENTER	45	45	8.00000
75	EAST	45	45	8.00000
75	WEST	20	20	8.00000
80	CENTER	10	10	4.50000
80	EAST	30	30	4.50000

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
65	EAST	2:1:3	PIPE
65	WEST	2:1:3	PIPE
70	CENTER	3:1:3	ROAD
70	EAST	3:1:3	PIPE
70	WEST	3:1:3	PIPE
75	CENTER	2:1:3	8FTSOUTH
75	EAST	2:1:3	2FTNORTH
75	WEST	2:1:3	PIPE
80	CENTER	3:1:3	PIPE
80	EAST	3:1:3	ROAD

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
80	WEST	10DEC85	17319.8	27757.9
85	CENTER	24APR86	17327.8	27758.7
85	EAST	24APR86	17331.0	27761.9
85	WEST	24APR86	17323.0	27754.8
90	CENTER	11DEC85	17331.0	27754.8
90	EAST	11DEC85	17334.1	27758.7
90	WEST	11DEC85	17327.8	27751.6
95	CENTER	24APR86	17334.9	27751.6
95	EAST	24APR86	17337.3	27755.6
95	WEST	24APR86	17331.0	27748.4

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
80	WEST	773.500	9	5
85	CENTER	774.300	3	0
85	EAST	774.300	9	60
85	WEST	774.300	3	0
90	CENTER	775.000	17	55
90	EAST	775.000	19	80
90	WEST	775.000	10	15
95	CENTER	775.500	20	1100
95	EAST	775.500	10	200
95	WEST	775.500	9	200

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
80	WEST	30	30	4.5000
85	CENTER	35	35	8.0000
85	EAST	40	40	8.0000
85	WEST	30	30	8.0000
90	CENTER	30	110	4.5000
90	EAST	30	90	4.5000
90	WEST	30	30	4.5000
95	CENTER	20	20	8.0000
95	EAST	40	40	8.0000
95	WEST	35	35	8.0000

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
80	WEST	3:1:3	PIPE
85	CENTER	2:1:3	PIPE
85	EAST	2:1:3	4FTNORTH
85	WEST	2:1:3	PIPE
90	CENTER	3:1:3	ROAD
90	EAST	3:1:3	ROAD
90	WEST	3:1:3	PIPE
95	CENTER	2:1:3	15FTSOUTH
95	EAST	2:1:3	20FTSOUTH
95	WEST	2:1:3	2FTSOUTH

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
100	CENTER	30DEC85	17338.9	27748.4
100	EAST	02JAN86	17342.1	27751.6
100	WEST	02JAN86	17335.7	27744.4
105	CENTER	23APR86	17342.9	27746.0
105	EAST	23APR86	17349.2	27747.6
105	WEST	23APR86	17339.7	27742.1
110	CENTER	03JAN86	17346.8	27741.3
110	EAST	02JAN86	17350.0	27745.2
110	WEST	03JAN86	17344.4	27738.1
115	CENTER	23APR86	17350.8	27739.7

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
100	CENTER	776.200	16	1000
100	EAST	776.200	23	2050
100	WEST	776.200	14	850
105	CENTER	776.800	8	40
105	EAST	776.800	13	60
105	WEST	776.800	12	20
110	CENTER	777.300	15	400
110	EAST	777.300	17	555
110	WEST	777.300	8	350
115	CENTER	777.700	9	200

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
100	CENTER	0	30	4.5000
100	EAST	0	30	4.5000
100	WEST	0	60	4.5000
105	CENTER	35	35	8.0000
105	EAST	35	35	8.0000
105	WEST	20	20	8.0000
110	CENTER	0	30	4.5000
110	EAST	0	40	4.5000
110	WEST	0	30	4.5000
115	CENTER	45	45	8.0000

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
100	CENTER	3:1:3	EASTBANK
100	EAST	3:1:3	ROADEAST
100	WEST	3:1:3	EASTBANK
105	CENTER	2:1:3	8FTSEAST
105	EAST	2:1:3	3FTSOUTH
105	WEST	2:1:3	PIPE
110	CENTER	3:1:3	ROADWEST
110	EAST	3:1:3	EASTBANK
110	WEST	3:1:3	PIPE
115	CENTER	2:1:3	PIPE

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
115	EAST	23APR86	17353.2	27742.1
115	WEST	23APR86	17347.6	27736.5
120	CENTER	03JAN86	17355.6	27735.7
120	EAST	03JAN86	17357.9	27739.7
120	WEST	03JAN86	17354.0	27731.7
125	CENTER	21APR86	17359.5	27733.3
125	EAST	21APR86	17361.1	27737.3
125	WEST	21APR86	17357.9	27730.2
130	CENTER	03JAN86	17364.3	27729.4
130	EAST	03JAN86	17365.9	27734.1

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
115	EAST	777.700	15	200
115	WEST	777.700	14	300
120	CENTER	778.300	12	30
120	EAST	778.300	13	350
120	WEST	778.300	13	0
125	CENTER	778.900	19	200
125	EAST	778.900	29	200
125	WEST	778.900	19	200
130	CENTER	779.700	26	0
130	EAST	779.700	32	26

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
115	EAST	40	40	8.0000
115	WEST	20	20	8.0000
120	CENTER	0	30	4.5000
120	EAST	0	30	4.5000
120	WEST	0	20	4.5000
125	CENTER	35	35	8.0000
125	EAST	40	40	8.0000
125	WEST	40	40	8.0000
130	CENTER	0	0	4.5000
130	EAST	0	20	4.5000

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
115	EAST	2:1:3	8FTSWEST
115	WEST	2:1:3	10FTNORTH
120	CENTER	3:1:3	PIPE
120	EAST	3:1:3	PIPE
120	WEST	3:1:3	PIPE
125	CENTER	2:1:3	1FTSOUTH
125	EAST	2:1:3	35FTEAST
125	WEST	2:1:3	7FTNEAST
130	CENTER	3:1:3	PIPE
130	EAST	3:1:3	PIPE

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
130	WEST	03JAN86	17361.9	27725.4
135	CENTER	21APR86	17369.0	27727.0
135	EAST	21APR86	17371.4	27731.0
135	WEST	21APR86	17367.5	27723.8
140	CENTER	18JAN86	17373.0	27723.8
140	EAST	31JAN86	17375.4	27729.4
140	WEST	21JAN86	17370.6	27720.6
145	CENTER	21APR86	17377.8	27723.0
145	EAST	21APR86	17379.4	27727.0
145	WEST	21APR86	17375.4	27719.8

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
130	WEST	779.700	19	0
135	CENTER	780.500	29	400
135	EAST	780.500	20	200
135	WEST	780.500	29	300
140	CENTER	781.000	12	0
140	EAST	781.000	33	3175
140	WEST	781.000	32	70
145	CENTER	781.500	20	200
145	EAST	781.500	19	200
145	WEST	781.500	10	200

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
130	WEST	0	0	4.5000
135	CENTER	35	35	8.0000
135	EAST	30	30	8.0000
135	WEST	40	40	8.0000
140	CENTER	0	0	4.5000
140	EAST	30	30	4.5000
140	WEST	30	200	4.5000
145	CENTER	40	40	8.0000
145	EAST	40	40	8.0000
145	WEST	50	50	8.0000

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
130	WEST	3:1:3	PIPE
135	CENTER	2:1:3	6FTNEAST
135	EAST	2:1:3	4FTNORTH
135	WEST	2:1:3	4FTSOUTH
140	CENTER	3:1:3	PIPE
140	EAST	3:1:3	WESTBANK
140	WEST	3:1:3	ROADCENTER
145	CENTER	2:1:3	2FTNORTH
145	EAST	2:1:3	3FTNORTH
145	WEST	2:1:3	FLUME

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
150	CENTER	18JAN86	17382.5	27720.6
150	EAST	21JAN86	17384.9	27725.4
150	WEST	05FEB86	17381.0	27715.9
155	CENTER	17APR86	17387.3	27719.0
155	EAST	17APR86	17388.9	27723.8
155	WEST	17APR86	17385.7	27715.1
160	CENTER	18JAN86	17392.1	27716.7
160	EAST	04FEB86	17393.7	27722.2
160	WEST	18JAN86	17390.5	27712.7
165	CENTER	17APR86	17396.8	27715.9

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
150	CENTER	782.300	36	26
150	EAST	782.300	14	100
150	WEST	782.300	13	175
155	CENTER	783.200	9	20
155	EAST	783.200	24	150
155	WEST	783.200	19	100
160	CENTER	784.000	13	0
160	EAST	784.000	20	15
160	WEST	784.000	22	400
165	CENTER	785.000	20	40

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
150	CENTER	0	30	4.5000
150	EAST	30	30	4.5000
150	WEST	30	30	4.5000
155	CENTER	45	45	8.0000
155	EAST	35	35	8.0000
155	WEST	35	35	8.0000
160	CENTER	0	0	4.5000
160	EAST	40	40	4.5000
160	WEST	0	30	4.5000
165	CENTER	50	50	12.0000

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
150	CENTER	3:1:3	PIPE
150	EAST	3:1:3	PIPE
150	WEST	3:1:3	PIPE
155	CENTER	2:1:3	PIPE
155	EAST	2:1:3	6FTSEAST
155	WEST	2:1:3	5FTNORTH
160	CENTER	3:1:3	PIPE
160	EAST	3:1:3	ROADCENTER
160	WEST	3:1:3	ROAD
165	CENTER	2:1:3	PIPE

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
165	EAST	17APR86	17398.4	27720.6
165	WEST	17APR86	17396.0	27711.9
170	CENTER	18JAN86	17400.8	27714.3
170	EAST	18JAN86	17401.6	27719.0
170	WEST	18JAN86	17400.0	27709.5
175	CENTER	17APR86	17406.3	27713.5
175	EAST	17APR86	17407.1	27718.3
175	WEST	17APR86	17405.6	27709.5
180	CENTER	18JAN86	17410.3	27711.9
180	EAST	18JAN86	17411.9	27717.5

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
165	EAST	785.000	29	175
165	WEST	785.000	18	60
170	CENTER	785.700	15	225
170	EAST	785.700	18	75
170	WEST	785.700	12	70
175	CENTER	786.500	22	100
175	EAST	786.500	24	200
175	WEST	786.500	26	300
180	CENTER	787.300	3	10
180	EAST	787.300	3	30

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
165	EAST	40	40	12.0000
165	WEST	60	60	12.0000
170	CENTER	0	30	4.5000
170	EAST	0	30	4.5000
170	WEST	0	20	4.5000
175	CENTER	35	35	12.0000
175	EAST	30	30	12.0000
175	WEST	30	30	12.0000
180	CENTER	0	0	4.5000
180	EAST	0	0	4.5000

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
165	EAST	2:1:3	4FTNORTH
165	WEST	2:1:3	HOLE155C
170	CENTER	3:1:3	ROAD
170	EAST	3:1:3	PIPE
170	WEST	3:1:3	PIPE
175	CENTER	2:1:3	2FTSOUTH
175	EAST	2:1:3	8FTNORTH
175	WEST	2:1:3	10FTNEAST
180	CENTER	3:1:3	PIPE
180	EAST	3:1:3	PIPE

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
180	WEST	18JAN86	17409.5	27707.1
185	CENTER	14APR86	17411.1	27711.1
185	EAST	14APR86	17416.7	27716.7
185	WEST	14APR86	17415.1	27707.1
190	CENTER	18JAN86	17419.8	27710.3
190	EAST	18JAN86	17420.6	27715.1
190	WEST	18JAN86	17419.0	27704.8
195	CENTER	14APR86	17427.0	27709.5
195	EAST	14APR86	17427.8	27715.9
195	WEST	14APR86	17427.0	27704.8

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
180	WEST	787.300	19	215
185	CENTER	788.000	0	0
185	EAST	788.000	0	0
185	WEST	788.000	0	0
190	CENTER	789.000	1	0
190	EAST	789.000	2	0
190	WEST	789.000	2	0
195	CENTER	790.000	0	0
195	EAST	790.000	0	0
195	WEST	790.000	0	0

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
180	WEST	0	30	4.5000
185	CENTER	.	.	.
185	EAST	.	.	.
185	WEST	.	.	.
190	CENTER	0	0	4.5000
190	EAST	0	0	4.5000
190	WEST	0	0	4.5000
195	CENTER	.	.	.
195	EAST	.	.	.
195	WEST	.	.	.

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
180	WEST	3:1:3	PIPE
185	CENTER	2:1:3	
185	EAST	2:1:3	
185	WEST	2:1:3	
190	CENTER	3:1:3	PIPE
190	EAST	3:1:3	PIPE
190	WEST	3:1:3	PIPE
195	CENTER	2:1:3	
195	EAST	2:1:3	
195	WEST	2:1:3	

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
200	CENTER	18JAN86	17430.2	27708.7
200	EAST	18JAN86	17431.0	27713.5
200	WEST	18JAN86	17429.4	27704.0
205	CENTER	12APR86	17434.9	27707.9
205	EAST	15APR86	17435.7	27714.3
205	WEST	14APR86	17434.1	27703.2
210	CENTER	05FEB86	17439.7	27707.1
210	EAST	05FEB86	17441.3	27712.7
210	WEST	05FEB86	17440.5	27703.2
215	CENTER	12APR86	17444.4	27706.3

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
200	CENTER	790.600	1	0
200	EAST	790.600	2	0
200	WEST	790.600	2	0
205	CENTER	792.000	1	0
205	EAST	792.000	5	0
205	WEST	792.000	5	5
210	CENTER	792.500	31	250
210	EAST	792.500	19	75
210	WEST	792.500	10	100
215	CENTER	793.300	29	150

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
200	CENTER	0	0	4.5000
200	EAST	0	0	4.5000
200	WEST	0	0	4.5000
205	CENTER	0	0	12.0000
205	EAST	0	0	12.0000
205	WEST	30	50	12.0000
210	CENTER	0	120	4.5000
210	EAST	30	120	4.5000
210	WEST	0	30	4.5000
215	CENTER	50	50	12.0000

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
200	CENTER	3:1:3	PIPE
200	EAST	3:1:3	PIPE
200	WEST	3:1:3	PIPE
205	CENTER	2:1:3	PIPE
205	EAST	2:1:3	PIPE
205	WEST	2:1:3	PIPE
210	CENTER	3:1:3	PIPE
210	EAST	3:1:3	PIPE
210	WEST	3:1:3	PIPE
215	CENTER	2:1:3	8FTNEAST

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
215	EAST	12APR86	17445.2	27712.7
215	WEST	12APR86	17443.7	27701.6
220	CENTER	07FEB86	17450.0	27706.3
220	EAST	05FEB86	17450.8	27711.1
220	WEST	08FEB86	17450.0	27700.8
225	CENTER	12APR86	17452.4	27705.6
225	EAST	12APR86	17453.2	27711.9
225	WEST	12APR86	17451.6	27700.8
230	CENTER	09FEB86	17460.3	27704.0
230	EAST	08FEB86	17461.1	27709.5

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
215	EAST	793.300	20	50
215	WEST	793.300	29	200
220	CENTER	794.100	10	5
220	EAST	794.100	25	2400
220	WEST	794.100	35	475
225	CENTER	794.500	20	200
225	EAST	794.500	20	200
225	WEST	794.500	30	150
230	CENTER	795.300	23	20
230	EAST	795.300	17	275

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
215	EAST	40	40	12.0000
215	WEST	45	45	12.0000
220	CENTER	30	30	4.5000
220	EAST	30	30	4.5000
220	WEST	0	30	4.5000
225	CENTER	40	40	12.0000
225	EAST	35	35	12.0000
225	WEST	40	40	12.0000
230	CENTER	0	30	4.5000
230	EAST	0	30	4.5000

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
215	EAST	2:1:3	4FTNORTH
215	WEST	2:1:3	3FTNORTH
220	CENTER	3:1:3	PIPE
220	EAST	3:1:3	PIPE
220	WEST	3:1:3	PIPE
225	CENTER	2:1:3	10FTNORTH
225	EAST	2:1:3	PIPE
225	WEST	2:1:3	8FTNORTH
230	CENTER	3:1:3	ROADCENTER
230	EAST	3:1:3	EASTDITCH+WEL

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
230	WEST	10FEB86	17459.5	27699.2
235	CENTER	12APR86	17462.7	27704.0
235	EAST	12APR86	17463.5	27709.5
235	WEST	12APR86	17461.9	27699.2
240	CENTER	09FEB86	17469.0	27703.2
240	EAST	04MAR86	17469.8	27707.9
240	WEST	28FEB86	17468.3	27697.6
245	CENTER	12APR86	17471.4	27701.6
245	EAST	12APR86	17472.2	27707.9
245	WEST	12APR86	17470.6	27697.6

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
230	WEST	795.300	40	1950
235	CENTER	796.000	23	500
235	EAST	796.000	20	400
235	WEST	796.000	25	400
240	CENTER	796.800	34	625
240	EAST	796.800	36	100
240	WEST	796.800	13	3000
245	CENTER	797.400	24	300
245	EAST	797.400	20	250
245	WEST	797.400	20	200

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
230	WEST	0	30	4.5000
235	CENTER	35	35	12.0000
235	EAST	45	45	12.0000
235	WEST	35	35	12.0000
240	CENTER	30	30	4.5000
240	EAST	30	30	4.5000
240	WEST	0	30	4.5000
245	CENTER	45	80	12.0000
245	EAST	45	45	12.0000
245	WEST	50	50	12.0000

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
230	WEST	3:1:3	PIPE
235	CENTER	2:1:3	PIPE
235	EAST	2:1:3	PIPE
235	WEST	2:1:3	8FTSOUTH
240	CENTER	3:1:3	HOLE220E
240	EAST	3:1:3	PIPE
240	WEST	3:1:3	ROADCENTER
245	CENTER	2:1:3	8FTNORTH
245	EAST	2:1:3	7FTSEAST
245	WEST	2:1:3	10FTNEAST

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
250	CENTER	04MAR86	17478.6	27700.8
250	EAST	03MAR86	17479.4	27705.6
250	WEST	05MAR86	17477.8	27696.0
255	CENTER	11APR86	17481.7	27700.0
255	EAST	11APR86	17482.5	27706.3
255	WEST	11APR86	17481.0	27695.2
260	CENTER	06MAR86	17488.1	27699.2
260	EAST	06MAR86	17488.9	27704.0
260	WEST	06MAR86	17487.3	27694.4
265	CENTER	10APR86	17493.7	27699.2

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
250	CENTER	798.200	20	50
250	EAST	798.200	25	10
250	WEST	798.200	20	600
255	CENTER	798.800	25	250
255	EAST	798.800	19	75
255	WEST	798.800	15	350
260	CENTER	799.500	22	100
260	EAST	799.500	13	40
260	WEST	799.500	14	500
265	CENTER	800.100	22	200

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
250	CENTER	0	30	4.5000
250	EAST	0	30	4.5000
250	WEST	0	30	4.5000
255	CENTER	35	35	12.0000
255	EAST	45	45	12.0000
255	WEST	45	45	12.0000
260	CENTER	0	30	4.5000
260	EAST	30	30	4.5000
260	WEST	0	30	4.5000
265	CENTER	45	45	12.0000

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
250	CENTER	3:1:3	PIPE
250	EAST	3:1:3	PIPE
250	WEST	3:1:3	PIPE
255	CENTER	2:1:3	13FTEAST
255	EAST	2:1:3	HOLE250E
255	WEST	2:1:3	4FTNORTH
260	CENTER	3:1:3	PIPE
260	EAST	3:1:3	PIPE
260	WEST	3:1:3	PIPE
265	CENTER	2:1:3	HOLE255C

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
265	EAST	10APR86	17494.4	27704.0
265	WEST	10APR86	17492.9	27693.7
270	CENTER	11MAR86	17498.4	27696.8
270	EAST	11MAR86	17499.2	27701.6
270	WEST	14MAR86	17497.6	27692.1
275	CENTER	10APR86	17503.2	27696.0
275	EAST	10APR86	17504.8	27702.4
275	WEST	10APR86	17502.4	27691.3
280	CENTER	14MAR86	17507.9	27695.2
280	EAST	14MAR86	17508.7	27700.0

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
265	EAST	800.100	25	250
265	WEST	800.100	19	200
270	CENTER	800.800	12	50
270	EAST	800.800	15	550
270	WEST	800.800	23	3800
275	CENTER	801.400	19	50
275	EAST	801.400	18	250
275	WEST	801.400	29	300
280	CENTER	801.800	24	50
280	EAST	801.800	24	50

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
265	EAST	35	35	12.0000
265	WEST	40	40	12.0000
270	CENTER	0	30	7.0000
270	EAST	0	30	4.5000
270	WEST	0	30	12.0000
275	CENTER	35	35	12.0000
275	EAST	30	30	12.0000
275	WEST	30	30	12.0000
280	CENTER	20	90	12.0000
280	EAST	20	40	12.0000

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
265	EAST	2:1:3	2FTEAST
265	WEST	2:1:3	6FTSWEST
270	CENTER	3:1:3	PIPE
270	EAST	3:1:3	PIPE
270	WEST	3:1:3	PIPE
275	CENTER	2:1:3	10FTNORTH
275	EAST	2:1:3	6FTNORTH
275	WEST	2:1:3	3FTSOUTH
280	CENTER	3:1:3	PIPE
280	EAST	3:1:3	EASTBANK

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
280	WEST	14MAR86	17507.1	27690.5
285	CENTER	09APR86	17513.5	27694.4
285	EAST	09APR86	17514.3	27700.0
285	WEST	09APR86	17512.7	27689.7
290	CENTER	14MAR86	17519.0	27693.7
290	EAST	15MAR86	17519.8	27698.4
290	WEST	15MAR86	17517.5	27689.7
295	CENTER	09APR86	17523.0	27692.9
295	EAST	09APR86	17523.8	27698.4
295	WEST	09APR86	17523.0	27688.1

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
280	WEST	801.800	11	200
285	CENTER	802.400	17	20
285	EAST	802.400	14	100
285	WEST	802.400	14	80
290	CENTER	802.700	5	30
290	EAST	802.700	6	25
290	WEST	802.700	20	70
295	CENTER	803.100	15	0
295	EAST	803.100	30	75
295	WEST	803.100	8	20

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
280	WEST	30	70	12
285	CENTER	60	120	12
285	EAST	35	35	12
285	WEST	45	45	12
290	CENTER	20	20	12
290	EAST	20	20	12
290	WEST	20	30	12
295	CENTER	130	130	12
295	EAST	40	40	12
295	WEST	30	30	12

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
280	WEST	3:1:3	PIPE
285	CENTER	2:1:3	PIPE
285	EAST	2:1:3	19FTNEAST
285	WEST	2:1:3	6FTNEAST
290	CENTER	3:1:3	PIPE
290	EAST	3:1:3	PIPE
290	WEST	3:1:3	WESTDITCH
295	CENTER	2:1:3	NONE
295	EAST	2:1:3	8FTNEAST
295	WEST	2:1:3	3FTWEST

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
300	CENTER	15MAR86	17527.8	27691.3
300	EAST	15MAR86	17528.6	27696.8
300	WEST	15MAR86	17527.0	27687.3
305	CENTER	09APR86	17532.5	27691.3
305	EAST	09APR86	17533.3	27696.0
305	WEST	09APR86	17531.7	27686.5
310	CENTER	15MAR86	17538.1	27689.7
310	EAST	15MAR86	17538.9	27695.2
310	WEST	15MAR86	17537.3	27684.9
315	CENTER	09APR86	17542.1	27688.1

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
300	CENTER	803.500	9	60
300	EAST	803.500	18	50
300	WEST	803.500	14	0
305	CENTER	804.000	19	30
305	EAST	804.000	20	40
305	WEST	804.000	20	25
310	CENTER	804.300	20	600
310	EAST	804.300	22	700
310	WEST	804.300	20	90
315	CENTER	804.700	19	75

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
300	CENTER	0	90	12
300	EAST	30	100	12
300	WEST	30	90	12
305	CENTER	30	30	12
305	EAST	40	40	12
305	WEST	45	120	12
310	CENTER	20	40	12
310	EAST	30	130	12
310	WEST	20	40	12
315	CENTER	30	30	12

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
300	CENTER	3:1:3	PIPE
300	EAST	3:1:3	PIPE
300	WEST	3:1:3	PIPE
305	CENTER	2:1:3	4FTSOUTH
305	EAST	2:1:3	6FTNORTH
305	WEST	2:1:3	3FTNORTH
310	CENTER	3:1:3	PIPE
310	EAST	3:1:3	PIPE
310	WEST	3:1:3	PIPE
315	CENTER	2:1:3	HOLE295W

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
315	EAST	09APR86	17543.7	27694.4
315	WEST	09APR86	17541.3	27684.1
320	CENTER	15MAR86	17546.8	27686.5
320	EAST	15MAR86	17548.4	27692.1
320	WEST	15MAR86	17546.0	27682.5
325	CENTER	07APR86	17551.6	27684.9
325	EAST	07APR86	17553.2	27689.7
325	WEST	07APR86	17550.0	27681.0
330	CENTER	15MAR86	17556.3	27682.5
330	EAST	15MAR86	17558.7	27687.3

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
315	EAST	804.700	20	350
315	WEST	804.700	25	50
320	CENTER	805.000	27	100
320	EAST	805.000	14	400
320	WEST	805.000	20	50
325	CENTER	805.300	10	75
325	EAST	805.300	13	80
325	WEST	805.300	15	200
330	CENTER	805.700	10	25
330	EAST	805.700	14	100

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
315	EAST	20	20	12
315	WEST	20	20	12
320	CENTER	20	20	12
320	EAST	20	30	12
320	WEST	20	20	12
325	CENTER	25	25	12
325	EAST	30	30	12
325	WEST	30	30	12
330	CENTER	20	20	12
330	EAST	30	40	12

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
315	EAST	2:1:3	5FTWEST
315	WEST	2:1:3	5FTEAST
320	CENTER	3:1:3	ROADNORTH
320	EAST	3:1:3	ROADNORTH
320	WEST	3:1:3	PIPE
325	CENTER	2:1:3	EASTBANK
325	EAST	2:1:3	3FTWEST
325	WEST	2:1:3	4FTEAST
330	CENTER	3:1:3	PIPE
330	EAST	3:1:3	EASTDITCH

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
330	WEST	15MAR86	17554.8	27678.6
335	CENTER	07APR86	17560.3	27681.0
335	EAST	07APR86	17561.9	27685.7
335	WEST	07APR86	17558.7	27677.0
340	CENTER	15MAR86	17564.3	27678.6
340	EAST	17MAR86	17566.7	27683.3
340	WEST	17MAR86	17562.7	27674.6
345	CENTER	05APR86	17567.5	27677.0
345	EAST	05APR86	17569.8	27681.7
345	WEST	05APR86	17565.1	27672.2

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
330	WEST	805.700	20	0
335	CENTER	805.800	17	300
335	EAST	805.800	20	300
335	WEST	805.800	21	220
340	CENTER	806.000	21	10
340	EAST	806.000	17	5
340	WEST	806.000	20	10
345	CENTER	806.200	16	0
345	EAST	806.200	16	20
345	WEST	806.200	14	0

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
330	WEST	30	40	12
335	CENTER	55	120	12
335	EAST	75	75	12
335	WEST	45	45	12
340	CENTER	30	30	12
340	EAST	30	40	12
340	WEST	40	40	12
345	CENTER	25	30	12
345	EAST	30	100	12
345	WEST	30	30	12

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
330	WEST	3:1:3	PIPE
335	CENTER	2:1:3	13FTEAST
335	EAST	2:1:3	8FTNORTH
335	WEST	2:1:3	2FTNORTH
340	CENTER	3:1:3	PIPE
340	EAST	3:1:3	PIPE
340	WEST	3:1:3	HOLE330W
345	CENTER	2:1:3	3FTEAST
345	EAST	2:1:3	HOLE350C
345	WEST	2:1:3	PIPE

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
350	CENTER	15MAR86	17573.0	27674.6
350	EAST	15MAR86	17574.6	27679.4
350	WEST	15MAR86	17570.6	27670.6
355	CENTER	05APR86	17575.4	27672.2
355	EAST	05APR86	17578.6	27677.0
355	WEST	05APR86	17572.2	27668.3
360	CENTER	24MAR86	17581.0	27668.3
360	EAST	24MAR86	17584.1	27672.2
360	WEST	24MAR86	17577.0	27664.3
365	CENTER	05APR86	17582.5	27664.3

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
350	CENTER	806.500	23	170
350	EAST	806.500	23	0
350	WEST	806.500	18	0
355	CENTER	806.800	21	25
355	EAST	806.800	22	0
355	WEST	806.800	23	0
360	CENTER	807.200	22	12
360	EAST	807.200	21	200
360	WEST	807.200	15	8
365	CENTER	807.400	25	25

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
350	CENTER	30	30	12
350	EAST	30	30	12
350	WEST	30	30	12
355	CENTER	30	90	12
355	EAST	30	30	12
355	WEST	25	30	12
360	CENTER	40	50	12
360	EAST	35	50	12
360	WEST	35	60	12
365	CENTER	35	100	12

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
350	CENTER	3:1:3	PIPE
350	EAST	3:1:3	PIPE
350	WEST	3:1:3	PIPE
355	CENTER	2:1:3	PIPE
355	EAST	2:1:3	HOLE350C
355	WEST	2:1:3	HOLE350C
360	CENTER	3:1:3	PIPE
360	EAST	3:1:3	PIPE
360	WEST	3:1:3	PIPE
365	CENTER	2:1:3	HOLE360W

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
365	EAST	05APR86	17586.5	27668.3
365	WEST	05APR86	17579.4	27661.9
370	CENTER	24MAR86	17586.5	27660.3
370	EAST	24MAR86	17590.5	27663.5
370	WEST	24MAR86	17582.5	27657.1
375	CENTER	05APR86	17588.9	27655.6
375	EAST	05APR86	17593.7	27659.5
375	WEST	05APR86	17585.7	27654.0
380	CENTER	15MAR86	17592.1	27651.6
380	EAST	17MAR86	17596.0	27654.8

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
365	EAST	807.400	30	10
365	WEST	807.400	18	50
370	CENTER	807.700	22	6
370	EAST	807.700	27	450
370	WEST	807.700	15	5
375	CENTER	808.000	20	140
375	EAST	808.000	25	0
375	WEST	808.000	20	0
380	CENTER	808.200	22	100
380	EAST	808.200	15	300

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
365	EAST	30	60	12
365	WEST	20	60	12
370	CENTER	.	.	12
370	EAST	35	40	12
370	WEST	40	40	12
375	CENTER	45	90	12
375	EAST	30	90	12
375	WEST	50	95	12
380	CENTER	30	60	12
380	EAST	35	40	12

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
365	EAST	2:1:3	HOLE360W
365	WEST	2:1:3	PIPE
370	CENTER	3:1:3	PIPE
370	EAST	3:1:3	PIPE
370	WEST	3:1:3	PIPE
375	CENTER	2:1:3	3FTSOUTH
375	EAST	2:1:3	PIPE
375	WEST	2:1:3	3FTSOUTH
380	CENTER	3:1:3	PIPE
380	EAST	3:1:3	PIPE

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
380	WEST	17MAR86	17588.9	27649.2
385	CENTER	05APR86	17593.7	27647.6
385	EAST	05APR86	17598.4	27650.8
385	WEST	05APR86	17590.5	27645.2
390	CENTER	18MAR86	17597.6	27644.4
390	EAST	18MAR86	17601.6	27646.0
390	WEST	18MAR86	17593.7	27640.5
395	CENTER	04APR86	17600.0	27639.7
395	EAST	04APR86	17604.8	27642.1
395	WEST	04APR86	17596.8	27636.5

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
380	WEST	808.200	31	500
385	CENTER	808.500	20	210
385	EAST	808.500	30	140
385	WEST	808.500	16	60
390	CENTER	808.600	14	20
390	EAST	808.600	24	150
390	WEST	808.600	21	220
395	CENTER	808.800	20	200
395	EAST	808.800	29	600
395	WEST	808.800	19	200

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
380	WEST	30	40	12
385	CENTER	40	45	12
385	EAST	45	45	12
385	WEST	30	100	12
390	CENTER	30	30	12
390	EAST	20	30	12
390	WEST	20	35	12
395	CENTER	50	50	12
395	EAST	50	50	12
395	WEST	50	50	12

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
380	WEST	3:1:3	PIPE
385	CENTER	2:1:3	HOLE390C
385	EAST	2:1:3	25FTWEST
385	WEST	2:1:3	HOLE380W
390	CENTER	3:1:3	PIPE
390	EAST	3:1:3	PIPE
390	WEST	3:1:3	PIPE
395	CENTER	2:1:3	PIPE
395	EAST	2:1:3	PIPE
395	WEST	2:1:3	PIPE

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
400	CENTER	18MAR86	17604.0	27634.9
400	EAST	18MAR86	17600.8	27638.1
400	WEST	18MAR86	17608.7	27631.7
405	CENTER	04APR86	17606.3	27630.2
405	NORTH	04APR86	17611.9	27632.5
405	SOUTH	04APR86	17603.2	27627.8
410	CENTER	18MAR86	17609.5	27624.6
410	EAST	18MAR86	17612.7	27626.2
410	WEST	18MAR86	17604.8	27622.2
415	CENTER	03APR86	17610.3	27619.0

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
400	CENTER	809.000	39	200
400	EAST	809.000	27	300
400	WEST	809.000	23	200
405	CENTER	809.400	3	0
405	NORTH	809.400	20	50
405	SOUTH	809.400	19	200
410	CENTER	809.600	13	50
410	EAST	809.600	13	150
410	WEST	809.600	14	10
415	CENTER	809.800	10	70

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
400	CENTER	30	40	12
400	EAST	35	40	12
400	WEST	40	50	12
405	CENTER	0	0	12
405	NORTH	35	35	12
405	SOUTH	35	35	12
410	CENTER	30	60	12
410	EAST	30	30	12
410	WEST	30	50	12
415	CENTER	30	30	12

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
400	CENTER	3:1:3	ROADNORTH
400	EAST	3:1:3	PIPE
400	WEST	3:1:3	ROADNORTH
405	CENTER	2:1:3	PIPE
405	NORTH	2:1:3	HOLE395E
405	SOUTH	2:1:3	PIPE
410	CENTER	3:1:3	PIPE
410	EAST	3:1:3	PIPE
410	WEST	3:1:3	PIPE
415	CENTER	2:1:3	PIPE

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
415	NORTH	03APR86	17615.1	27619.8
415	SOUTH	03APR86	17605.6	27618.3
420	CENTER	20MAR86	17610.3	27614.3
420	NORTH	20MAR86	17615.1	27614.3
420	SOUTH	20MAR86	17605.6	27614.3
425	CENTER	03APR86	17610.3	27607.9
425	NORTH	03APR86	17614.3	27607.1
425	SOUTH	03APR86	17605.6	27608.7
430	CENTER	21MAR86	17609.5	27603.2
430	NORTH	21MAR86	17614.3	27601.6

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
415	NORTH	809.800	19	150
415	SOUTH	809.800	20	150
420	CENTER	809.700	18	100
420	NORTH	809.700	14	150
420	SOUTH	809.700	30	450
425	CENTER	809.700	21	100
425	NORTH	809.700	20	300
425	SOUTH	809.700	26	10
430	CENTER	809.500	25	500
430	NORTH	809.500	30	400

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
415	NORTH	35	35	12
415	SOUTH	35	35	12
420	CENTER	30	30	12
420	NORTH	30	30	12
420	SOUTH	30	30	12
425	CENTER	30	30	12
425	NORTH	30	30	12
425	SOUTH	30	30	12
430	CENTER	40	40	12
430	NORTH	30	30	12

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
415	NORTH	2:1:3	PIPE
415	SOUTH	2:1:3	PIPE
420	CENTER	3:1:3	PIPE
420	NORTH	3:1:3	PIPE
420	SOUTH	3:1:3	PIPE
425	CENTER	2:1:3	6FTWEST
425	NORTH	2:1:3	PIPE
425	SOUTH	2:1:3	HOLE435N
430	CENTER	3:1:3	PIPE
430	NORTH	3:1:3	PIPE

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
430	SOUTH	21MAR86	17604.8	27604.0
435	CENTER	02APR86	17607.9	27596.8
435	NORTH	02APR86	17612.7	27595.2
435	SOUTH	02APR86	17604.8	27598.4
440	CENTER	21MAR86	17607.1	27592.1
440	NORTH	25MAR86	17611.9	27590.5
440	SOUTH	25MAR86	17602.4	27594.4
445	CENTER	02APR86	17604.8	27587.3
445	NORTH	02APR86	17609.5	27585.7
445	SOUTH	02APR86	17600.8	27589.7

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
430	SOUTH	809.500	20	20
435	CENTER	809.200	17	200
435	NORTH	809.200	20	300
435	SOUTH	809.200	30	400
440	CENTER	809.000	15	600
440	NORTH	809.000	22	1800
440	SOUTH	809.000	13	600
445	CENTER	808.800	16	75
445	NORTH	808.800	29	40
445	SOUTH	808.800	29	100

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
430	SOUTH	40	40	12
435	CENTER	30	30	12
435	NORTH	30	30	12
435	SOUTH	30	30	12
440	CENTER	40	50	12
440	NORTH	45	45	12
440	SOUTH	40	40	12
445	CENTER	35	35	12
445	NORTH	35	40	12
445	SOUTH	40	40	12

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
430	SOUTH	3:1:3	HOLE420
435	CENTER	2:1:3	PIPE
435	NORTH	2:1:3	PIPE
435	SOUTH	2:1:3	7FTSOUTH
440	CENTER	3:1:3	PIPE
440	NORTH	3:1:3	PIPE
440	SOUTH	3:1:3	PIPE
445	CENTER	2:1:3	PIPE
445	NORTH	2:1:3	PIPE
445	SOUTH	2:1:3	PIPE

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
450	CENTER	26MAR86	17602.4	27582.5
450	NORTH	26MAR86	17606.3	27580.2
450	SOUTH	26MAR86	17598.4	27584.9
455	CENTER	02APR86	17600.8	27578.6
455	NORTH	02APR86	17604.0	27576.2
455	SOUTH	02APR86	17596.0	27581.7
460	CENTER	26MAR86	17596.8	27573.8
460	NORTH	26MAR86	17600.8	27571.4
460	SOUTH	26MAR86	17592.9	27577.0
465	CENTER	02APR86	17594.4	27570.6

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
450	CENTER	808.500	13	15
450	NORTH	808.500	18	450
450	SOUTH	808.500	20	250
455	CENTER	808.300	29	130
455	NORTH	808.300	29	100
455	SOUTH	808.300	10	200
460	CENTER	807.700	17	400
460	NORTH	807.700	30	300
460	SOUTH	807.700	19	600
465	CENTER	807.400	16	50

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
450	CENTER	60	60	12
450	NORTH	60	60	12
450	SOUTH	60	60	12
455	CENTER	35	40	12
455	NORTH	40	40	12
455	SOUTH	40	40	12
460	CENTER	35	35	12
460	NORTH	35	35	12
460	SOUTH	40	40	12
465	CENTER	35	35	12

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
450	CENTER	3:1:3	15FTSOUTH
450	NORTH	3:1:3	PIPE
450	SOUTH	3:1:3	PIPE
455	CENTER	2:1:3	10FTSOUTH
455	NORTH	2:1:3	10FTSOUTH
455	SOUTH	2:1:3	PIPE
460	CENTER	3:1:3	PIPE
460	NORTH	3:1:3	PIPE
460	SOUTH	3:1:3	PIPE
465	CENTER	2:1:3	8FTSOUTH

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
465	NORTH	02APR86	17597.6	27567.5
465	SOUTH	02APR86	17590.5	27574.6
470	CENTER	26MAR86	17592.9	27568.3
470	NORTH	26MAR86	17596.0	27565.1
470	SOUTH	26MAR86	17587.3	27570.6
475	CENTER	01APR86	17588.1	27560.3
475	NORTH	01APR86	17591.3	27563.5
475	SOUTH	01APR86	17584.1	27567.5
480	CENTER	28MAR86	17584.1	27560.3
480	NORTH	27MAR86	17588.1	27557.1

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
465	NORTH	807.400	28	100
465	SOUTH	807.400	17	50
470	CENTER	807.000	10	150
470	NORTH	807.000	25	200
470	SOUTH	807.000	19	700
475	CENTER	806.500	20	500
475	NORTH	806.500	14	500
475	SOUTH	806.500	18	285
480	CENTER	806.000	26	200
480	NORTH	806.000	23	400

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
465	NORTH	40	40	12
465	SOUTH	40	50	12
470	CENTER	35	35	12
470	NORTH	40	40	12
470	SOUTH	30	30	12
475	CENTER	30	30	12
475	NORTH	35	35	12
475	SOUTH	35	35	12
480	CENTER	40	40	12
480	NORTH	35	35	12

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
465	NORTH	2:1:3	PIPE
465	SOUTH	2:1:3	PIPE
470	CENTER	3:1:3	10FTSOUTH
470	NORTH	3:1:3	12FTEAST
470	SOUTH	3:1:3	PIPE
475	CENTER	2:1:3	10FTEAST
475	NORTH	2:1:3	10FTEAST
475	SOUTH	2:1:3	PIPE
480	CENTER	3:1:3	PIPE
480	NORTH	3:1:3	PIPE

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
480	SOUTH	27MAR86	17581.0	27563.5
485	CENTER	31MAR86	17581.0	27557.1
485	NORTH	31MAR86	17584.9	27554.8
485	SOUTH	31MAR86	17577.8	27561.9
490	CENTER	28MAR86	17577.8	27554.0
490	NORTH	28MAR86	17581.7	27551.6
490	SOUTH	28MAR86	17574.6	27558.7
495	CENTER	31MAR86	17574.6	27552.4
495	NORTH	31MAR86	17577.8	27548.4
495	SOUTH	31MAR86	17571.4	27556.3

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
480	SOUTH	806.000	25	300
485	CENTER	805.500	29	300
485	NORTH	805.500	30	60
485	SOUTH	805.500	29	80
490	CENTER	805.200	13	100
490	NORTH	805.200	20	100
490	SOUTH	805.200	14	100
495	CENTER	804.800	30	200
495	NORTH	804.800	20	175
495	SOUTH	804.800	20	200

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
480	SOUTH	35	35	12
485	CENTER	40	40	12
485	NORTH	40	80	12
485	SOUTH	35	35	12
490	CENTER	30	30	12
490	NORTH	40	40	12
490	SOUTH	35	35	12
495	CENTER	35	35	12
495	NORTH	35	35	12
495	SOUTH	35	35	12

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
480	SOUTH	3:1:3	PIPE
485	CENTER	2:1:3	PIPE
485	NORTH	2:1:3	PIPE
485	SOUTH	2:1:3	PIPE
490	CENTER	3:1:3	8FTSOUTH
490	NORTH	3:1:3	PIPE
490	SOUTH	3:1:3	PIPE
495	CENTER	2:1:3	10FTNORTH
495	NORTH	2:1:3	10FTNORTH
495	SOUTH	2:1:3	10FTNORTH

HOLE	POSITION ON CENTER LINE	DATE OF INJECTION COMPLETION	ORNL GRID NORTH	ORNL GRID EAST
500	CENTER	28MAR86	17571.4	27550.0
500	NORTH	28MAR86	17574.6	27546.0
500	SOUTH	28MAR86	17569.0	27554.0

HOLE	POSITION ON CENTER LINE	ELEVATION OF GROUND	DEPTH OF INJECTION	GALLONS OF GROUT
500	CENTER	804.500	19	100
500	NORTH	804.500	30	400
500	SOUTH	804.500	18	350

HOLE	POSITION ON CENTER LINE	MINIMUM PRESSURE (PSI)	MAXIMUM PRESSURE (PSI)	PUMPING RATE (GPM)
500	CENTER	35	35	12
500	NORTH	40	40	12
500	SOUTH	45	45	12

HOLE	POSITION ON CENTER LINE	RATIO OF WATER:LIME:ASH (GAL:LB:LB)	LOCATION OF GROUT RETURN
500	CENTER	3:1:3	PIPE
500	NORTH	3:1:3	PIPE
500	SOUTH	3:1:3	8FTEAST

Appendix C

Proposed Mitigating Action at Low-Level Liquid Waste (LLW) Trench 7*

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March 1984

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*Supported by the Office of Defense Waste and By-Product Management, U.S. Department of Energy under Contract W-7405-eng-26 with Union Carbide Corporation.

**Consultant

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SUMMARY

Adverse environmental effects have been determined to exist at Low-Level Liquid Waste (LLW) Trench 7 and are described in ORNL/TM-8839 (Olsen et al. 1983). They include the migration of radionuclides in groundwater emerging at a surface seep and the possible leaching of alkalinity from Trench 7 such that it may lose its capacity for retention of strontium-90. For the purpose of mitigating these adverse effects, several construction techniques are identified and evaluated for their hydrologic suitability.

Utilizing average hydraulic characteristics for the Conasauga Group developed in other reports (ORNL/TM-7416 and ORNL/NFW-83/13) for nearby sites, together with a groundwater-level contour map and subsurface profiles constructed for Trench 7, groundwater recharge is analyzed as to source distribution. The analysis indicates that the maximum amount of lateral recharge that a gravity drain would intercept uphill of the site would be exceeded by infiltration from precipitation at the site by approximately 9 times. Therefore, a groundwater interceptor drain is not recommended at Trench 7 at this time.

Three mitigating actions recommended at Trench 7 are as follows: 1) an extension of the present asphaltic cap and the construction of a gutter to transport the runoff from this cap to a location down-gradient of the Trench, preliminarily estimated at a cost of $15,000; 2) sever and plug the abandoned waste transfer line and backfilled

ditch leading into the Trench, preliminarily estimated at a cost of $5000; and 3) construct a grout barrier by pressure grouting a line between the seep and the Trench, and extending around the north end of the Trench, preliminarily estimated at a cost of $150,000.

A test grouting program is recommended that could be incorporated into the proposed grout line.

PURPOSE

This report identifies and evaluates construction techniques that, either individually or in concert, are believed most feasible to mitigate the adverse impacts associated with the radioactive ground-water seep surfacing in the gully immediately east of Low- Level Liquid Waste (LLW) Trench 7. As fully discussed in ORNL/TM 8839 (Olsen et al. 1983) these impacts are briefly: 1) migration of radionuclides from Trench 7 and associated relict drainage pathways to the surface environment at the seep, and 2) possible leaching of the alkalinity of Trench 7 to a level such that it would lose its capacity for retention of strontium-90.

INTRODUCTION

History of Seep

It is not known whether the groundwater seep east of the disposal site existed prior to the construction and operation of Trench 7 or if it developed due to the excess hydrostatic pressure created during trench use. Potentially, a maxiumum of approximately 35 feet of head in the area may have been created when the trench was operated at full capacity (maximum elevation of top of waste, approximate elevation 799 ft minus elevation of groundwater adjacent to the seep area, approximately 764 ft). Although a memorandum written in the early winter of 1962 cautions that "springs or seeps may occur during operation of the trench in wet late winter months or early spring months" (Intra-Laboratory correspondence, de Laguna to Struxness, December 5, 1962), the first reference to the subject seep found to date indicates that it was initially noted on December 19, 1962, a maximum of 80 days after waste disposal commenced in Trench 7. Apparently, liquid from the trench was at that time moving rather rapidly along discontinuities (joints, bedding planes, and faults) in the intensely weathered Conasauga strata and eminating at the seep. Utilizing Darcy's Law, an assumed porosity of 0.03 (Davis et al. 1983), and an 80-day travel time, a hydraulic conductivity of 1.25×10^{-4} cm/sec can be calculated for the strata discontinuities between Trench 7 and the seep. This is twice the average hydraulic

conductivity used in the section of this report on hydrogeology of the site. However, as described in that section, this higher hydraulic conductivity is in the direction of geologic strike and faulting which is considered to account for the higher value, i.e., the strata is anisotropic.

Transport Mechanisms

Two mechanisms for transporting radionuclides at Trench 7 are considered in ORNL/TM-8839: "the percolation of perched groundwater along relict contamination layers during drainage and seepage after periods of prolonged rainfall, and a seasonal rise in the groundwater table and its saturation of either relict migration layers or the contaminated alkaline zone at the northern end of the trench." Both mechanisms are driven by precipitation yet would require somewhat different corrective actions. It is, therefore, appropriate to evaluate the relative significance of each mechanism in order that technically sound and cost effective mitigating action can be determined. To do so requires some knowledge of the construction and operation of Trench 7 and its associated transfer line, as well as the geohydrology of the site.

PERTINENT DETAILS OF TRENCH 7 CONSTRUCTION AND OPERATION

Trench 7 Excavation Description

Figure 1 is a topographic map of the Trench 7 site showing the location of its monitoring wells and seep area. Construction drawings of the trench and associated waste transfer pipeline are shown in Figures 2, 3, and 4, and Figures 5 and 6 are photographs taken during trench construction in 1962. As shown in the drawings and photographs, Trench 7 is divided into two north-south trending sections, A and B, with a non-excavated section 30 feet in length dividing the two 100-foot long excavations. Drawings (e.g., Fig. 3) show both excavations to bottom at an elevation of 789 feet; however, normal construction practice is such that this would be the minimum depth and, therefore, the excavation may be slightly deeper. The bottom width of the excavation is 4 feet with walls of the excavation extending to the ground surface on a slope of 4 vertical on 1 horizontal. The excavations are filled with crushed limestone to an approximate elevation of 800 feet; at this level the excavation is 9 feet wide.

Waste Transfer Pipeline

The 2-inch diameter pipeline, which carried the waste to each section of the Trench, parallels the west side of the Trench at a distance of approximately 24 feet (Fig. 4) from the centerline, with an discharge pipeline to the mid-point of each section at an approximate elevation of 799 feet. The flow of waste was controlled by two separate valves located in the 2-inch diameter lines leading to each section.

BASE MAP FROM ORNL-DWG 83-1514A

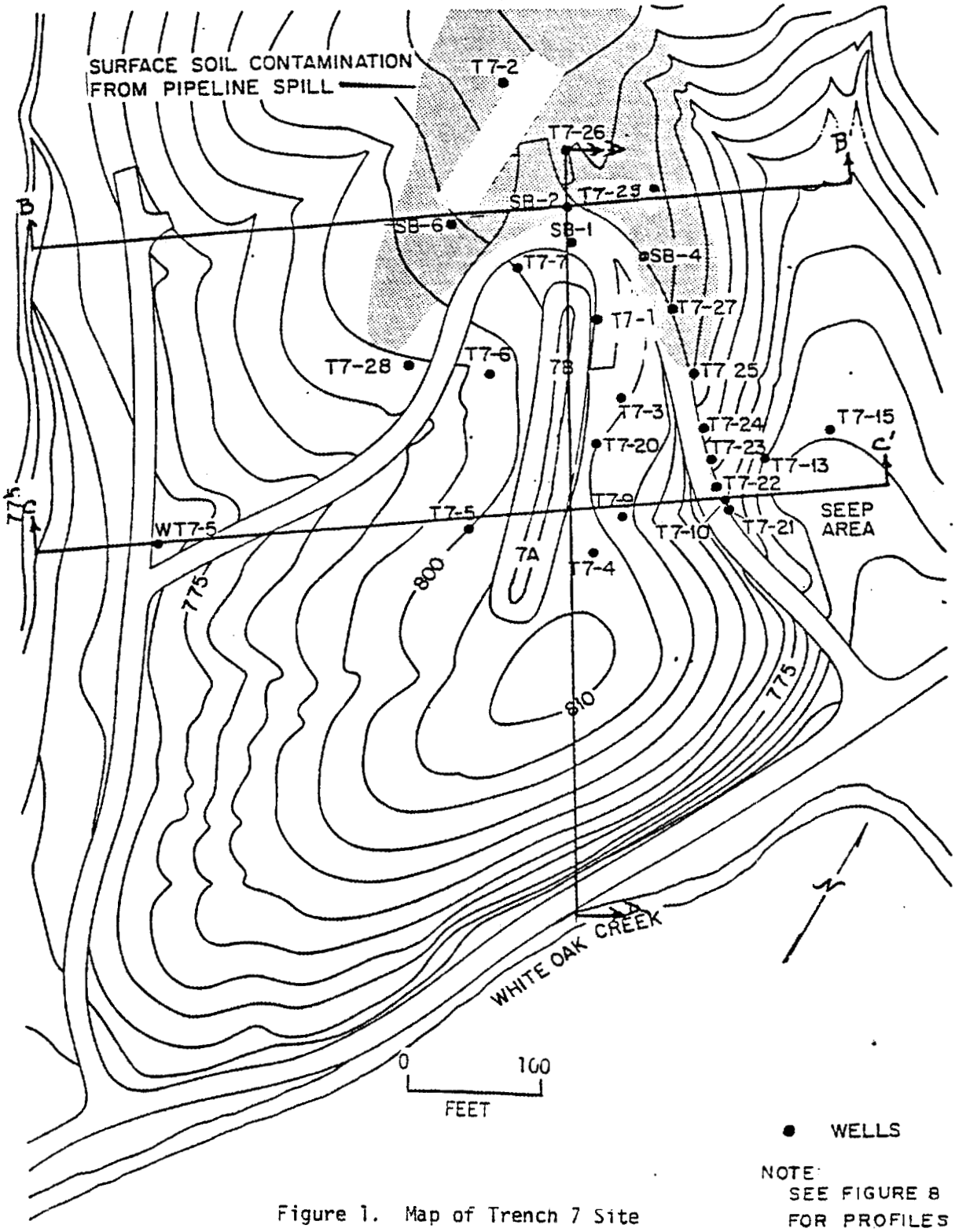

Figure 1. Map of Trench 7 Site

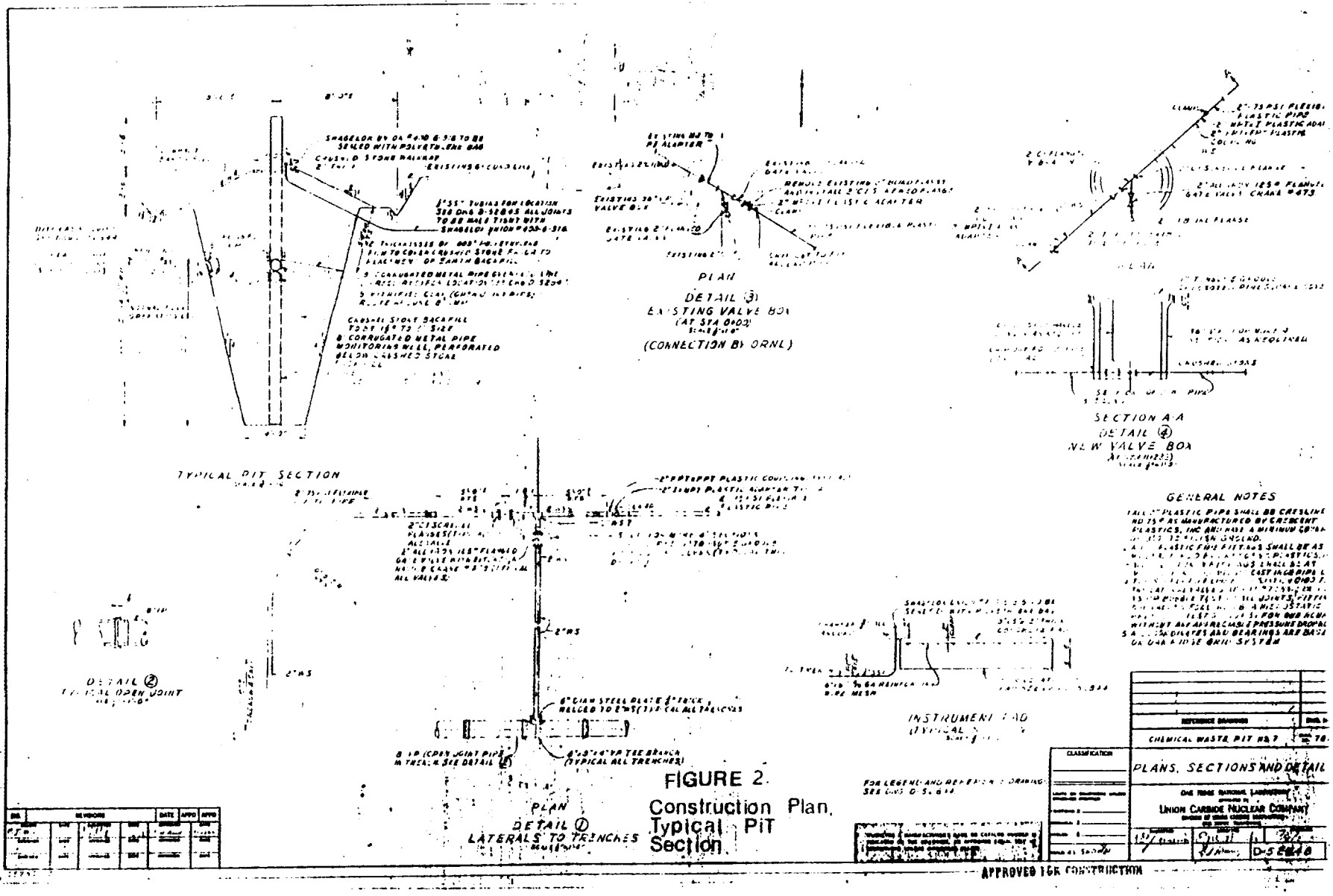

FIGURE 2.
Construction Plan, Typical Pit Section.

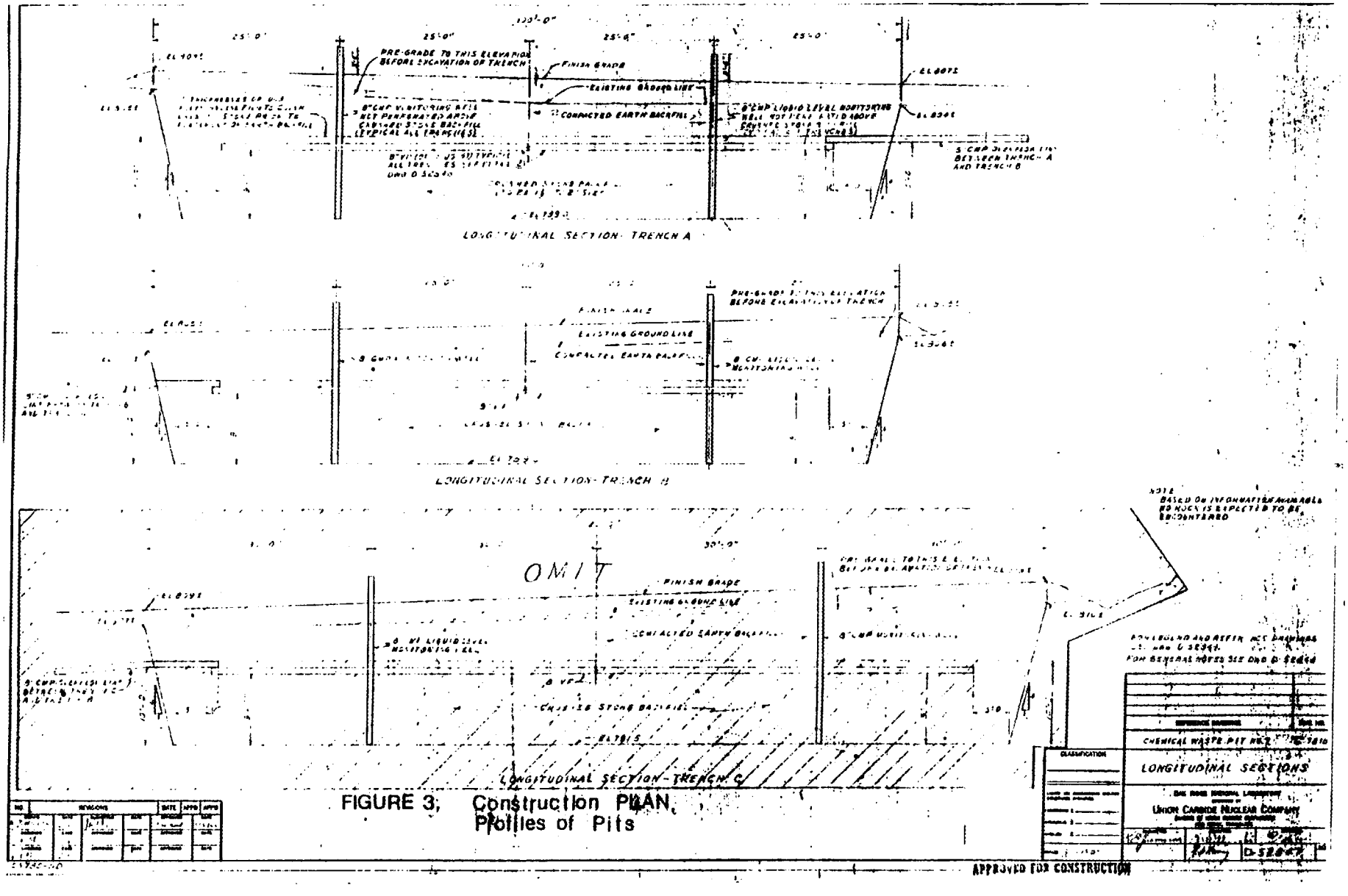

FIGURE 3; Construction PLAN, Profiles of Pits

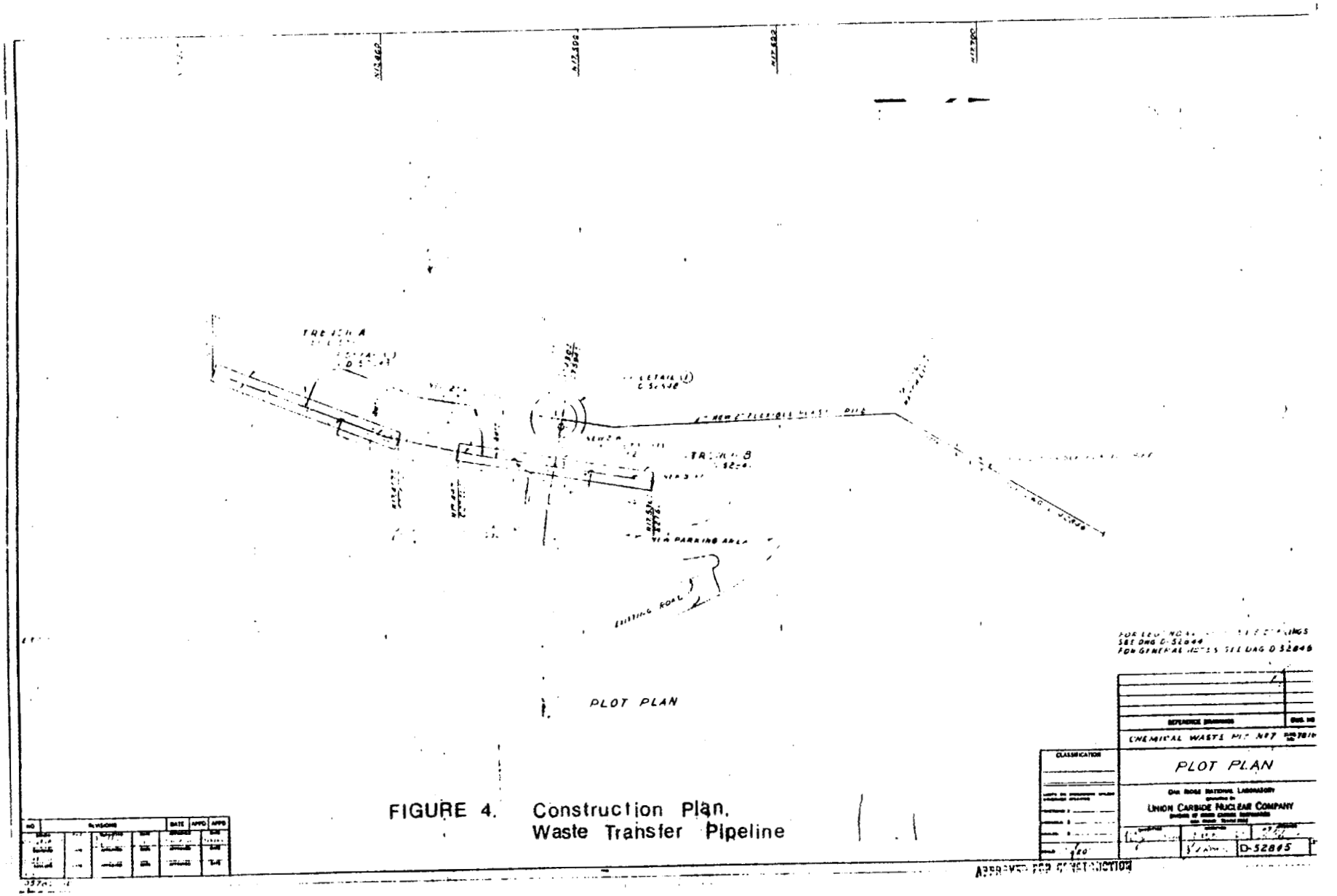

FIGURE 4. Construction Plan, Waste Transfer Pipeline

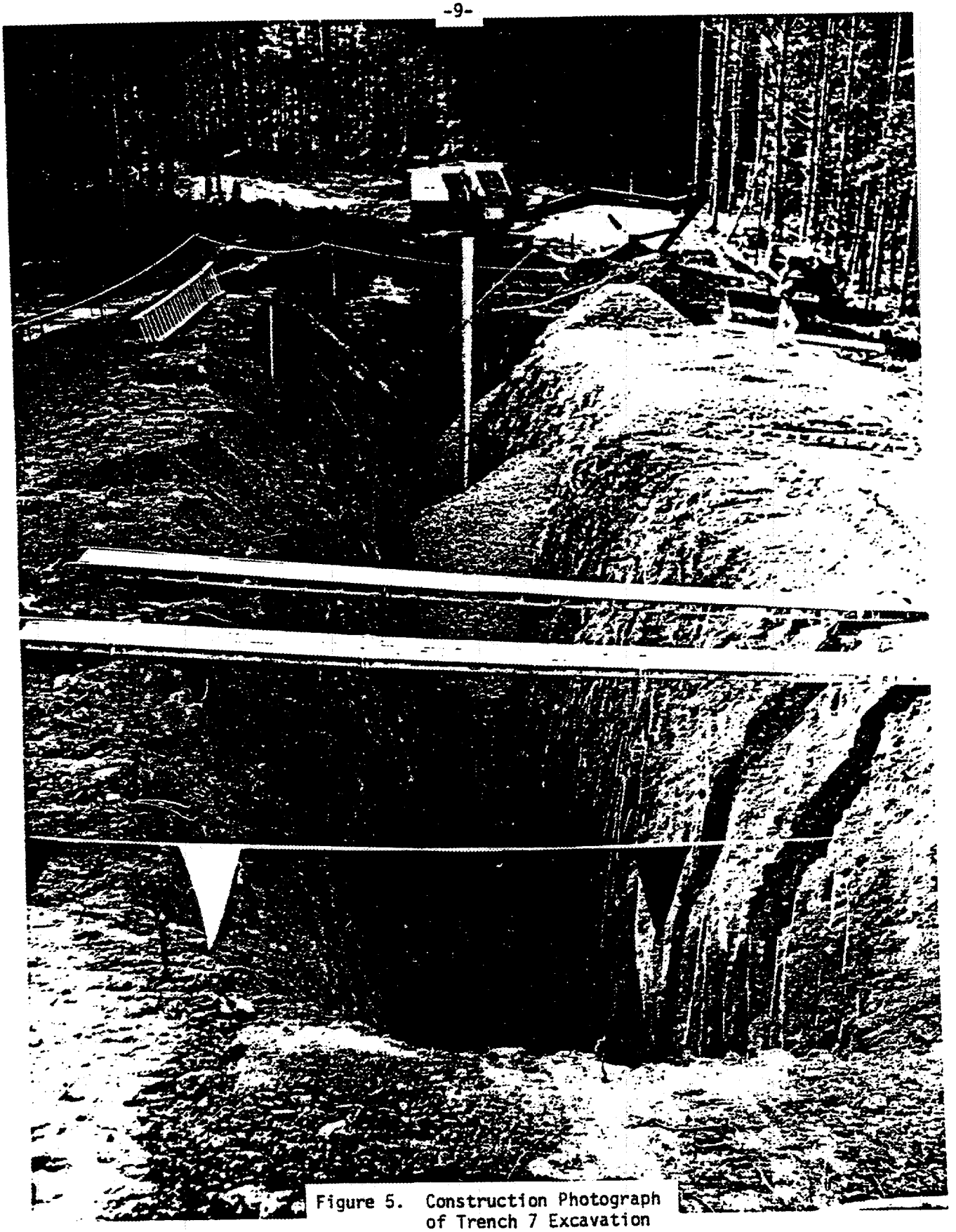

Figure 5. Construction Photograph of Trench 7 Excavation

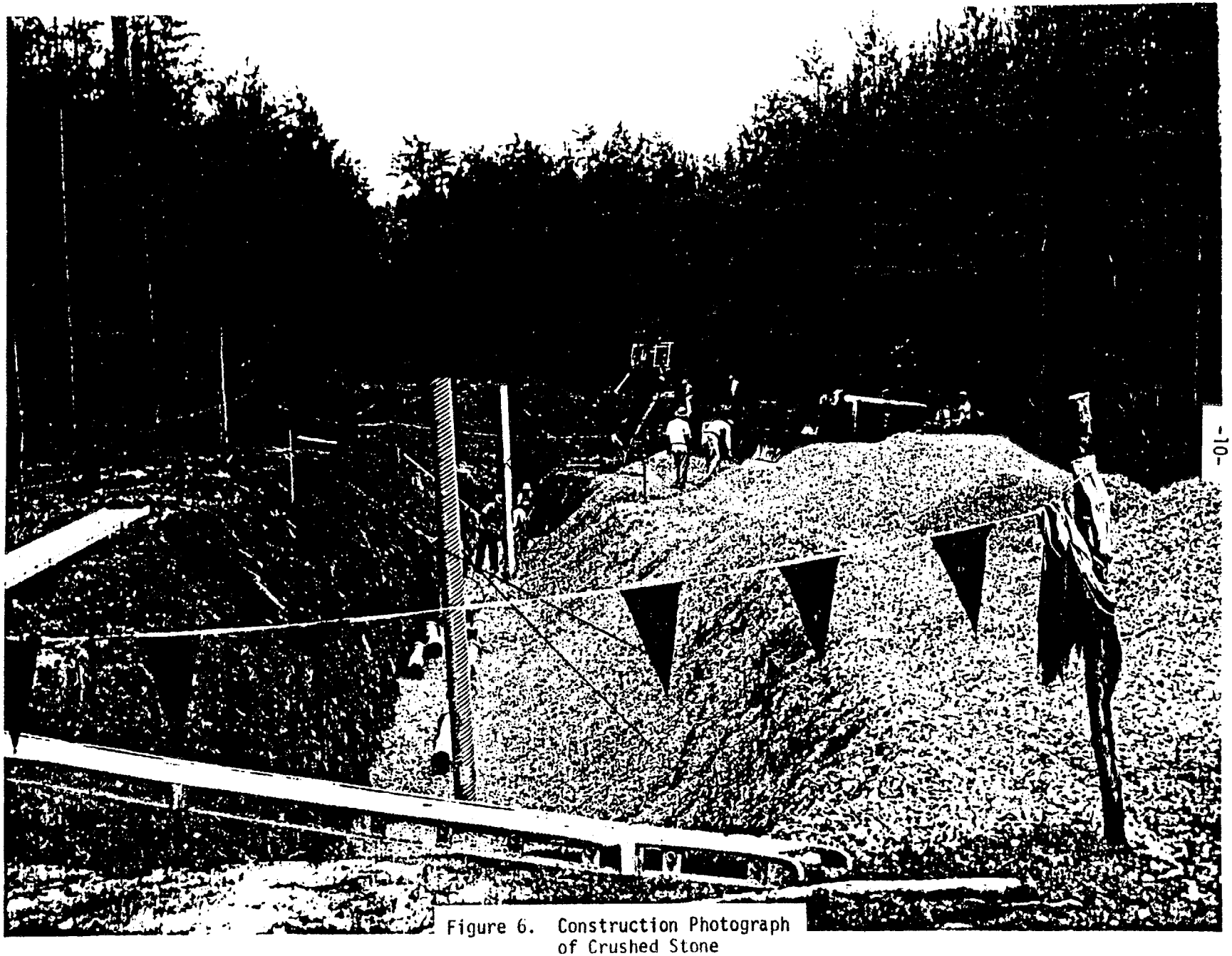

Figure 6. Construction Photograph of Crushed Stone

The valves are located (Figs. 2 and 4) opposite the mid-point of the northern-most trench section (Trench 7B) and are accessible through individual valve boxes open to the ground surface (Fig. 2). The pipelines and valves are approximately 3.5 feet below the ground surface at this point and from there continue northward on an upward grade for a distance of approximately 400 feet to a valve box at an approximate elevation of 835 feet. At the upper valve box, the line was at one time connected to a now abandoned line to the old hydrofracture operation. The line to Trench 7 is now physically separated from the hydrofracture waste line by the removal of the flow-control valve and capping of the Trench 7 line at the upper control box.

Trench 7 Fill Material

An overflow pipe extends across the non-excavated section connecting sections A and B of Trench 7, and a total of four monitoring wells, spaced 25 feet from the ends of each section, are installed to the bottom of the excavations. As seen in the construction photographs (Fig. 6), the crushed limestone placed in the Trench was first stockpiled along the west side of the excavations. It is not known whether the excess stone was entirely removed from this area but normal construction practice did not generally require that this be done in situations similar to this excavation. If not removed, even a thin, continuous layer of crushed stone could provide a permeable pathway for surface drainage to enter Trench 7. The portion of the excavations above and elevation of approximately 800 feet was backfilled with earth

materials and was initially at least 2.5 feet above the ground surface at the centerline, with a 1 vertical on 2 horizontal slope to each side.

Surface Runoff from Trench 7 Asphaltic Cover

Discussion with ORNL personnel familiar with the facility indicates that the trench was rimmed by a gravel-surfaced road during its period of operation. At some later date, the upper surface of the Trench was covered with the present asphaltic concrete that extends from the centerline of the Trench to a distance of approximately 16 and 19 feet on the east and west sides, respectively, making the total width of the cover approximately 35 feet. As the asphalt surface is approximately 240 feet in length, this represents a surface area projected to a horizontal plane of approximately 8400 square feet. Although obscured by forest litter, gravel is presently found rimming the asphaltic cover to a width of up to 10 feet and to depths commonly from 0.5 to 1.0 feet, but may be deeper at some points on the lower elevation west side.

The ground surface along the east side of the facility is higher than that along the west side, and is nearly horizontal,therefore, adequate surface drainage is not promoted. Much of the rainfall that strikes the cover drains from east to west toward the two valve boxes immediately west of Trench B. After a 1.06-inch rainfall on February 13, 1984, four inches of water was measured in the bottom of these two valve boxes. A 1-inch rainfall translates to 700 cubic feet of runoff from the asphaltic cover. A large portion of this runoff could be infiltrating to the groundwater through highly contaminated

materials immediately adjacent to the Trench. Some of the runoff could conceivably be finding its way directly into the Trench through possible permeable pathways such as the pipelines and ditch backfill, and remnant crushed stone fill originally stockpiled on the west side of the Trench.

Abandonment of Waste Transfer Line

Operation of Trench 7 was discontinued in April 1966, when the 2-inch diameter transfer line burst underground at a point approximately 270 feet northwest of the end of Trench 7B, resulting in liquid waste erupting to the ground surface and flowing down the ridge toward Trench 7. The log report of the incident estimates that approximately 3000 gallons of liquid waste escaped under pressure at this point prior to being discovered and pumping stopped. Some of the radioactive liquid waste spill material may have been disposed of by burial in a trench excavated within approximately 100 feet of the north end of Trench 7B, however, records are unclear. The operational log states that contaminated material north of Trench 7B was covered with five feet of earth. It cannot be ascertained that the ruptured point in the line was ever repaired or sealed.

HYDROGEOLOGY OF THE SITE

Site Geology

LLW Trench 7 is located in intensely weathered rock of the Conasauga Group on a northwest-southeast trending ridge just north of White Oak Creek (Fig. 1). The site has a maximum relief of approximately 50 feet and is drained by two unnamed creeks on the east and west sides which empty into White Oak Creek. Work at nearby SWSAs 6 and 7 indicate that the average strike of strata in this area is 55^{0} NE and dip to the SE (with dips ranging from near horizontal to high angle), and that folds and faults are common. Further, weathering commonly extends to depths of 30 to 40 feet with the material generally consisting of shales, siltstones, and occasional thin beds of nonweathered limestone at the deeper depths. The shales and siltstones are so highly fractured that spacing of fractures in these materials is commonly as close as one centimeter, providing an excellent secondary porosity for groundwater movement. Outcrops visible at the site of Trench 7 do not indicate any significant differences from the data collected from nearby sites.

Geologic Structures at Site

ORNL/TM-8839 identifies three high-angle faults visible in the west sideslope of the east-access road going up the side of the Trench 7 ridge. The central fault passes downward through the axis of a small anticlinal structure consisting of limestone. If these structures follow the average geologic strike of the area, which is likely, the

westward projection of the southernmost fault would cross the central area of Trench 7A, and the northernmost fault would cross the central area of Trench 7B. A westward projection of the central fault and anticline would cross the facility at the southern end of the non-excavated plug between Trenches 7A and 7B. It is speculated that the movement of fluids in the subsurface is accentuated along these faults, particularly the southernmost fault, resulting in the radioactive seep area east of Trench 7.

Groundwater Table at Trench 7

The groundwater table (potentiometric) contour map presented in Figure 7 is based on water levels measured in monitoring wells on February 1, 1984, (see Table 1). For comparison purposes, Table 1 also lists water levels measured on December 15, 1983. It can be seen from Table 1 that the increase in water levels that occurred from December to February in wells at the north end of the site is reflected to a much lesser degree in the wells at the southern end of the site. The dampening of well response towards the southern end of the site is also noted in ORNL/TM-8839. Based on the potentiometric surface, it is inferred from Figure 7 that the groundwater divide is somewhat west of the topographic divide at the Trench 7 site. This indicates that some of the recharge to groundwater from infiltration on the west slope of the ridge moves eastward under the trench towards the seep area. Subsurface profiles of the site (Figs 8a through 8c) visually depict this situation. Figures 8b and 8c are drawn parallel to the assumed geologic strike (55° NE) and Figure 8a is approximately

Table 1. Water-level elevations (ft) in monitoring wells at LLW Trench 7.

Well No.	Elevation top of casing	Water level elevation 12/15/83	Water level elevation 2/1/84
T7-1	808 0	777.3	777.0
T7-2	814.3	795.1	798.4
T7-3	809.5	-	768.7
T7-4	807.0	-	766.2
T7-5	800.7	767.4	766.8
T7-6	800.4	770.9	770.7
T7-7	807.6	-	783.5
T7-10	783.0	764.6	764.3
T7-13	771.1	763.6	763.7
T7-20	809.6	767.4	767.7
T7-21	784.3	764.4	764.1
T7-22	786.8	764.9	764.5
T7-23	791.4	765.5	765.0
T7-24	795.7	766.5	766.8
T7-25	801.0	767.7	768.8
T7-26	816.5	780.5	783.8
T7-27	806.9	771.0	770.9
T7-29	809.0	777.2	778.7
WT7-5	768.1	760.8	760.9
SB-2	812.9	776.9	778.3
SB-4	810.1	775.8	776.5
SB-6	806.6	777.1	779.6

BASE MAP FROM ORNL-DWG 83-1514A

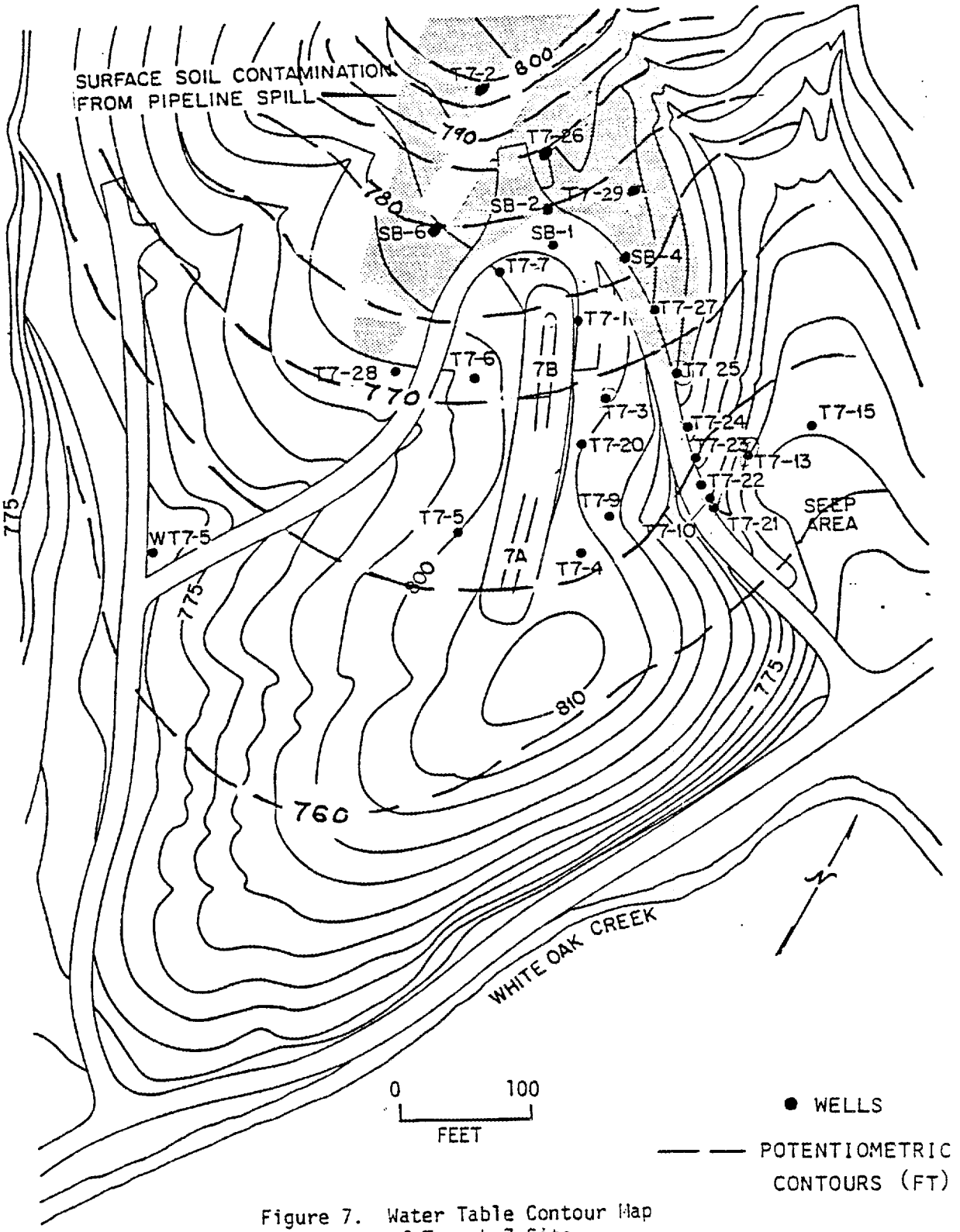

Figure 7. Water Table Contour Map of Trench 7 Site

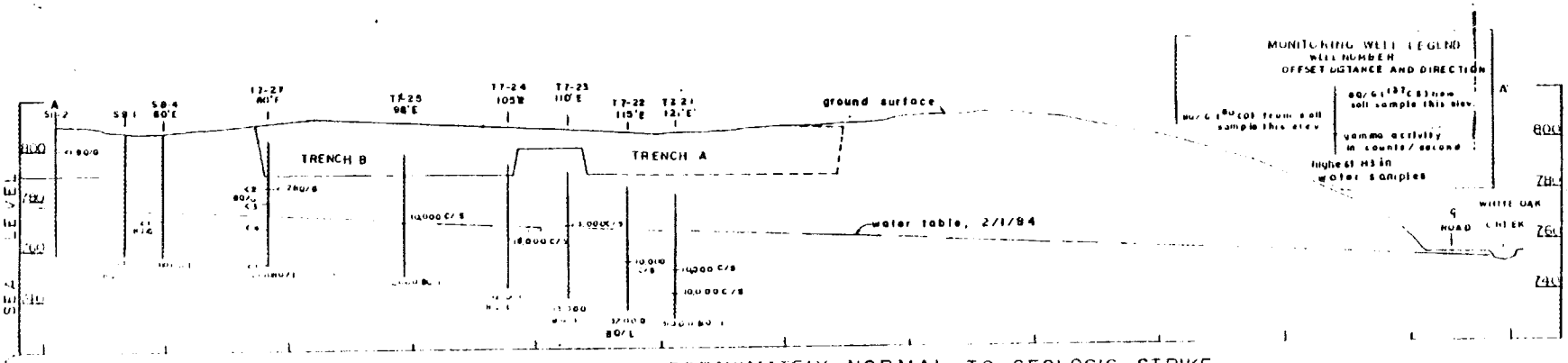

FIGURE 8a. SUBSURFACE PROFILE APPROXIMATELY NORMAL TO GEOLOGIC STRIKE

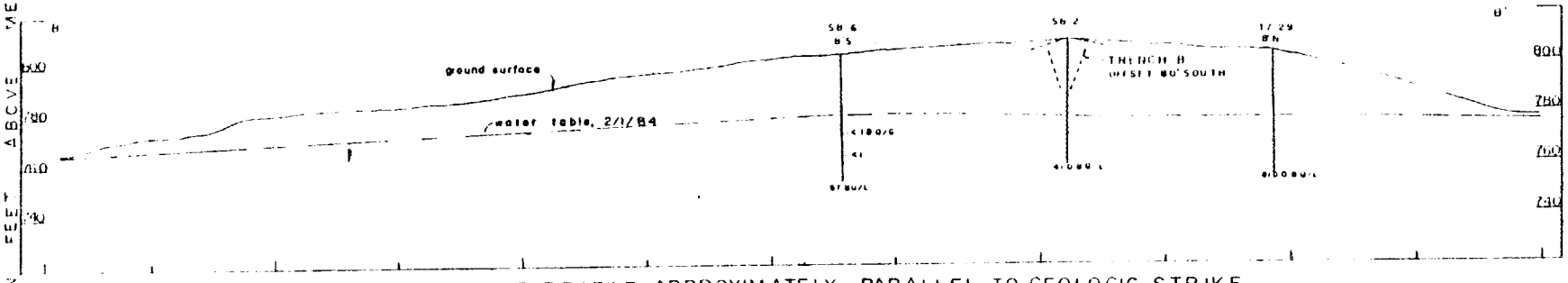

FIGURE 8b SUBSURFACE PROFILE APPROXIMATELY PARALLEL TO GEOLOGIC STRIKE

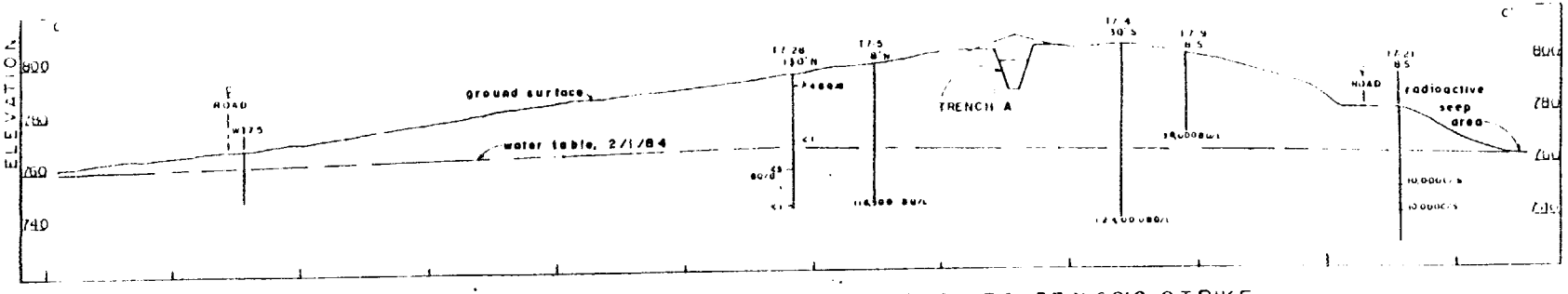

FIGURE 8c SUBSURFACE PROFILE APPROXIMATELY PARALLEL TO GEOLOGIC STRIKE

SCALE 20 FEET

perpendicular to strike. Radioactivity, as summarized from ORNL/TM-8839, is indicated at the appropriate well borings appearing in these profiles.

Groundwater Recharge Source Area Analysis

Potential Recharge Sources. To determine effective and feasible mitigating action at Trench 7, it is necessary to know the relationship between quantities of groundwater moving laterally down-gradient into the site, and that derived from direct infiltration on the site. Only that portion of laterally moving groundwater that could be recovered by a gravity drain system is of interest in this evaluation. Interest is further limited to only that recoverable quantity that would have moved through the area intersected by a drain prior to its construction. This is a lesser quantity than would actually flow to a constructed drain because presence of an efficient drain would induce additional flow during wet periods from upgradient of the drain at this site. If this lateral flow proved significant compared to the infiltration quantity, the construction of an interceptor drain would be an attractive mitigating alternative to be considered.

Selected Model Drain. To perform this evaluation a "best site" was selected for a model of a groundwater interceptor drain such as was constructed at SWSA 6 (Davis and Stansfield 1984). This site is shown in Figures 9a and 9b, and passes just north of Trench 7, extending from the gully on the east of Trench 7 to a point just west of the apparent

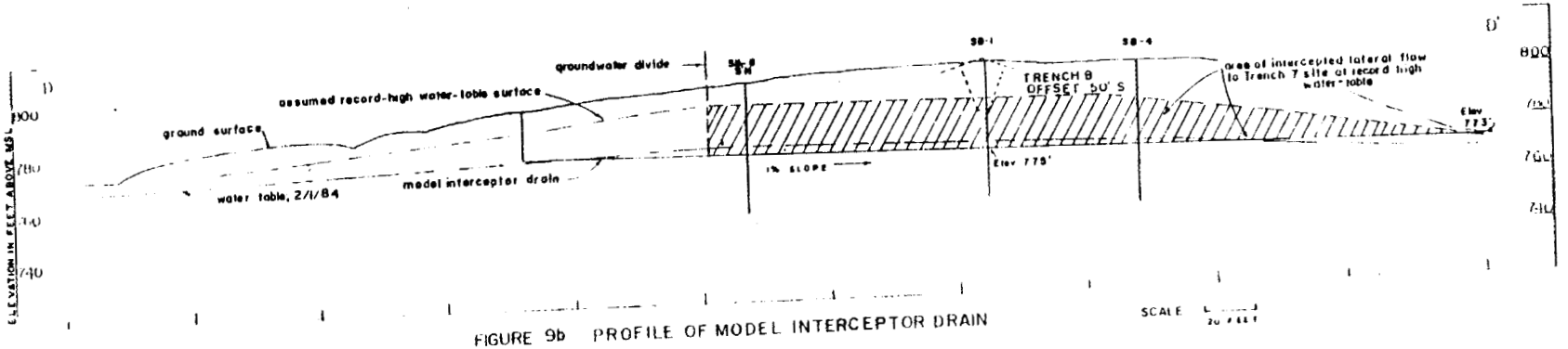

FIGURE 9b PROFILE OF MODEL INTERCEPTOR DRAIN

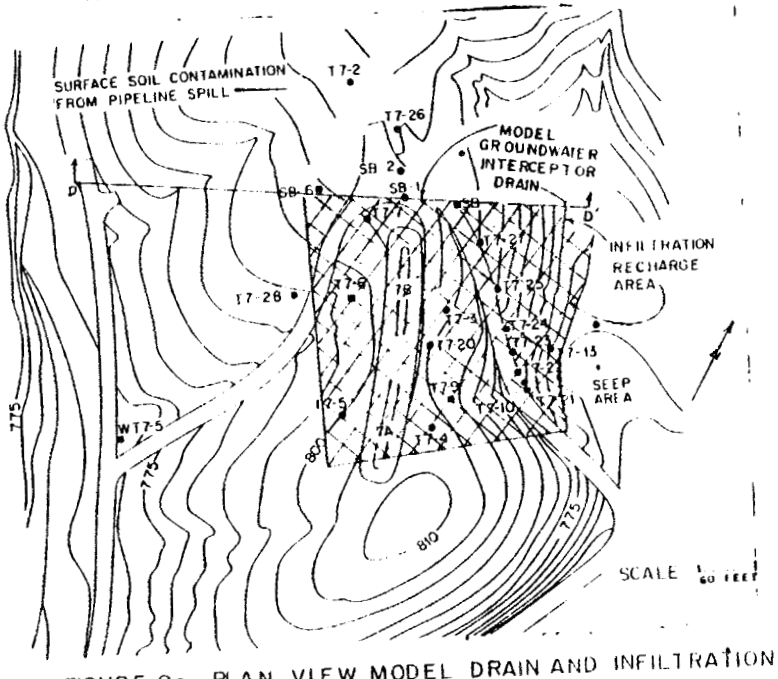

FIGURE 9a PLAN VIEW MODEL DRAIN AND INFILTRATION RECHARGE AREA

groundwater divide. The choice of this location is based on minimizing depth and length of excavation while maximizing the groundwater intercepted, and also to reduce the chances of withdrawing contaminated groundwater to the surface. Further, the discharge would be in the head of the gully already contaminated by the radioactive seep. At this site, the drain would be less than 400 feet in length and reach a maximum depth of approximately 35 feet opposite the north end of Trench 7 where it could intercept groundwater down to elevation 775 feet. Constructed on a 1% grade, the drain would discharge to one catch basin at approximate elevation 773 feet.

Recharge at the Site by Lateral Flow Above Elevation 775 Feet. As reported in ORNL/TM-8839, the highest water level recorded in monitoring wells adjacent to Trench 7 was elevation 789.36 feet measured in well T7-1 on February 26, 1962. This level was slightly higher than the design elevation of the bottom of the then to be constructed Trench 7. In the years since this record high, the groundwater table has not been recorded higher than several feet below the bottom of Trench 7. Data in Table 2 indicate that the period of December 1961 through February 1962 had the most precipitation for that normally wet three-month period of any of the 14 years listed. Therefore, it can be assumed that the frequency of occurrence of any higher water level than that measured in 1962 would be insiginficant for the purposes of this analysis. Referring to the water table

Table 2. Winter season precipitation (inches) at ORNL for the years 1961-1971,* 1982, and 1984.**

Year	January	February	March	December
1961	1.52	7.43	6.32	9.80
1962	5.77	8.21	5.52	3.33
1963	2.96	3.55	9.69	2.45
1964	4.63	3.93	6.29	4.71
1965	4.38	2.89	10.98	0.70
1966	3.13	5.42	2.03	3.52
1967	3.39	3.68	5.43	7.71
1968	4.12	0.85	4.54	4.24
1969	4.03	5.18	2.23	7.89
1970	2.60	2.99	3.67	3.88
1971	4.71	4.69	4.50	6.27
1981				3.6
1982	6.2	4.7	6.1	6.7
1983				6.3
1984	2.7			

*Data from National Oceanographic and Atmospheric Administration, 1972.

**Data from ORNL, Environmental Sciences Division

contour map in Fig. 7, it can be estimated that the maximum groundwater level along the model interceptor drain would have been no more than 5 feet higher than at well T7-1, i.e., maximum elevation 794 feet. Using this elevation and generally following the February 1, 1984 groundwater slope (Figure 9b), the area of aquifer intercepted at highest flow conditions is calculated from Figure 9b to be approximately 4500 square feet. In accordance with Darcy's Law, the quantity of lateral flow through this area can be calculated when the hydraulic conductivity and gradient are known. Assuming that the hydraulic gradient remains the same under high water conditions, a gradient of 0.13 is estimated from the water table map (Fig. 7). ORNL-NFW-83/13 (Davis et al. 1983) reports a log-normalized mean value for hydraulic conductivity derived from tests on 36 wells in Conasauga strata as 6.31×10^{-5}cm/s (0.1789 ft/d), and an effective porosity for that site of 0.03. Using these assumed values, the maximum quantity of water flowing laterally southward towards Trench 7 through the portion of the aquifer that could be intercepted by the north face of a gravity drain is estimated to be in the order of 105 cubic feet/day. This could only occur when the groundwater table is at the maximum recorded level (February 26, 1962).

The December 5, 1961 water level in monitoring well T7-1 is reported as elevation 775.16 feet (Intra-Laboratory correspondence, de Laguna to Morgan, July 1962). Again from the water table contour map (Fig. 7) it can be estimated that the groundwater table at that time would have been at approximate elevation 780 feet or 5 feet above

the bottom of the model drain (Elev. 775 ft.). Therefore, at the beginning of December 1961, the model drain would have been intercepting an aquifer thickness 26% as great as it would at the end of February 1962, when the aquifer thickness would be at its projected recorded maximum of elevation 794 feet at the drain.

To determine how flow through the aquifer thickness intercepted by the model drain may have varied during the winter months of 1961/62 it is necessary to examine past rainfall records. Table 2 lists precipitation data at ORNL during the winter seasons of 1961/62 to 1970/71 and for 1981/82; and also for the months of December 1983 and January 1984. Table 3 lists daily precipitation at ORNL for December 1961 and for January and February 1962. For the three months of December 1961 through February 1962, a total of 23.78 inches of precipitation was recorded, making it the wettest three-month period during the 14 years shown in Table 2. It is determined from Table 3 that by December 18, precipitation totaled 8.6 inches so far that month. For the purposes of this analysis, we are interested in estimating the maximum amount of flow that would have occurred toward Trench 7 through the thickness of aquifer intercepted by the model drain during this extremely wet three-month period. Considering the relative rapid response of wells to rainfall that is exhibited in Conasauga strata (Davis et al. 1983), it is not inappropriate to assume that by December 18, 1961, flow of groundwater through the area intercepted by the model drain could have risen linearly from approximately 27 cubic feet/day (26% of 105 cubic (feet/day) to its

Table 3. Daily precipitation (hundreths of inches) at ORNL for the months of December 1961, January and February 1962.

Day	December 1961	January 1962	February 1962
1	0	11	0
2	4	0	0
3	0	0	0
4	24	0	0
5	18	38	16
6	0	48	0
7	0	0	0
8	0	2	2
9	144	20	74
10	62	05	0
11	152	0	0
12	51	0	0
13	0	0	0
14	0	0	0
15	0	58	0
16	133	0	23
17	198	0	0
18	74	0	36
19	0	14	4
20	0	0	0
21	0	0	101
22	0	43	5
23	12	60	266
24	1	53	0
25	0	37	80
26	0	82	8
27	86	49	57
28	0	57	149
29	0	0	
30	0	0	
31	21	0	
Total	980	577	821

*Data from National Oceanographic and Atmospheric Administration, 1972.

estimated maximum of slightly less than 105 cubic feet/day. Therefore, the total subject flow during the month of December would have been 2550 cubic feet based on an average of approximately 66 cubic feet/day for the first 18 days and 105 cubic feet/day for the last 13 days of the month. Further assuming that a record high flow of 105 cubic feet/day continued through the month of January (even though the recorded precipitation of 5.77 inches for the month of January indicates a probably lessening of flow through the model drain area) and February, an estimated maximum flow quantity towards Trench 7 that would have been intercepted by the model drain is calculated at approximately 8700 cubic feet for the three-month period.

Recharge by Infiltration at the Site. Having an approximation of the quantity of lateral groundwater flow towards Trench 7 that could be intercepted by a gravity drain system during the period of a record-high water table, it is appropriate to compare this with an estimate of the quantity that would enter recharge by direct infiltration of precipitation on the site south of the model drain. An estimate of this recharge area can be made from the water table map (Fig. 7). This recharge area shown in Figure 9a, some of which is certain to be a source of supply to the seep area, amounts to approximately 1.78 acres. A simulated study in Conasauga strata at SWSA 6 (Arora et al., 1981) found that during periods when evapotranspiration is at a minimum, 49% of precipitation infiltrates to groundwater recharge. Using this recharge percentage and a total precipitation of 23.78 inches from Table 2 for the period December 1961

through February 1962 , it is estimated that the 1.78 acre area south of the model drain was directly recharged by approximately 75,300 cubic feet of water through infiltration.

Recharge by Lateral Flow Versus Infiltration. Utilizing the above estimates, which are believed to overestimate the quantity of lateral flow, it appears that the Trench 7 site is recharged by direct infiltration by a factor approximately 9 times times greater than the lateral groundwater flow toward Trench 7 that could be intercepted by a gravity drain. This would be during a period of maximum rainfall and a record high water table at the site. National Oceanographic and Atmospheric Administration (NOAA) records indicate that a total of approximately 16 inches is normal for the December through February period rather than the 23.78 inches used in the foregoing analysis. It is readily apparent that as the water table declines reflecting a lessening rainfall, the ratio and importance of direct recharge at the site by infiltration as compared with interceptable lateral flow will increase. For example, if the water table fell to elevation 775 feet at the model drain as could be expected during periods of low rainfall, no laterally moving groundwater would be intercepted resulting in the ratio becoming an infinitely large number. This statement is supported by the following data: Well SB-01 is located in line of the model drain (Figs. 9a and 9b) and on November 22, 1982, the water level recorded at the well was elevation 771.22 feet (Olsen et al. 1983), or almost 4 feet below the bottom of the model drain. Precipitation

(ORNL, Environmental Sciences Division records) for the previous month of October was 2.25 inches, and 5.09 inches for the first 22 days of November. Again, using the figure of 49% (Arora et al. 1981) of precipitation to recharge, it is calculated that the 1.78 acre recharge area south of the model drain was recharged by approximately 16,000 cubic feet from direct filtration during the first 22 days of November. The drain would not have intercepted any flow in this period. Thus, it appears that the most effective strategy for minimizing recharge to the groundwater in the Trench 7 vicinity is to prevent surface infiltration.

PROPOSED MITIGATING ACTIONS

General

It is indicated by the foregoing analysis that infiltration from the surface is a more important water source to the contaminated seep area and for the possible leaching of alkalinity of Trench 7, than lateral, high water-table flow, from the north. Therefore, mitigating actions that are likely to be most beneficial are those which consider this situation. Three such actions are proposed and discussed in the paragraphs below. They are: 1) extend the present asphaltic cover on Trench 7; and collect the runoff from the impermeable cap with a gutter system, transporting it down-gradient of the recharge area of Trench 7; 2) severing and plugging of the waste transfer pipeline and backfilled ditch at a point north of Trench 7; and 3) grout injection into the subsurface between the contaminated seep and Trench 7, and also at the north end of the Trench.

Control of Surface Runoff

Situation Description. It appears that considerable surface runoff from the asphaltic cover is finding its way into the subsurface immediately adjacent to the edge of the cover. The crushed stone which rims Trench 7 provides an excellent water-holding medium for up to 100% of the runoff from the asphalt surface, the percentage may vary with the intensity of the rainfall. As previously noted, every inch of rainfall on the cap translates into 700 cubic feet of runoff. A portion of this quantity could very well be finding its way directly

into Trench 7 but exiting through the large surface area of the Trench at a rate faster than the inflow, therefore is not measured in any of the four monitoring wells that were installed in the Trench at the time of construction. Even if the runoff does not enter the Trench directly, surface water retained in the crushed stone and soil percolates to the groundwater table through materials highly contaminated by the original waste disposal operation.

Mitigating Action Description. Proposed corrective action for the foregoing situation includes extending the present asphalt surface up to 10 feet in all directions in order to cover the crushed stone rimming the facility. During construction, a proper seal must be formed between the new and old asphalt portions. Clearing probably will not be necessary and required grading will be minimal to accomplish this task. Also, a system for collecting the runoff from the paved area should be provided to carry the surface water to a discharge point down-gradient of the groundwater beneath Trench 7. An open gutter system of asphaltic pavement would suffice. Unless grading was involved to change the grade on the east side, two points of discharge will probably be required, one for each side of the trench.

Preliminary Cost Estimate. For comparison purposes, costs of the above work are initially estimated to be on the order of $15,000. This would include: $4900 for 700 square yards of asphaltic pavement in place at $7.00/square yard; $1000 for preparation for pavement; $5600 for 400 linear yards of asphaltic gutter at $14.00/linear yard; $2000 for clearing and grading for 200 yards of the gutter; and $1500 for miscellaneous expenses.

Securing Abandoned Waste Transfer Pipeline

Situation Description. Historically, abandoned pipelines are frequently found to provide flow pathways for fluids infiltrating from the surface. Such pathways may exist along the outside of the smooth pipe surface in the uncompacted backfill surrounding the pipeline, inside a non-plugged pipeline through openings in the line, or in gravel or sand bedding sometimes placed at the bottom of the pipeline ditch during construction. The waste transfer pipeline and surrounding material were subjected to considerable disturbance at the time of line rupture and clean-up operations. The line has a negative gradient of approximately 35 feet vertical in the 400 feet horizontal from the valve box, where it is disconnected and capped, to Trench 7. Within this distance it is not improbable that during a wet period, surface water may find entrance to one or more of the above listed pathways and discharge into Trench 7 or its immediate area.

Mitigating Action Description. Proposed corrective action includes making a shallow excavation to 2 feet or more below the transfer pipeline at some convenient location north of the north end of Trench 7 but south of well T7-2. A 2-foot or more section of pipe should be cut from the line, and cement grout pumped into the section of line leading to Trench 7. If the old ditch cross-section does not seem to be a major pathway for infiltrating surface water (as evidenced by visually observable low soil moisture content), this single excavation needs only to be filled with compacted clay. If the ditch backfill material does seem to be a major pathway as evidence by wet soil conditions,

consideration should be given to placing either a foundation type drain system discharging on a 1% grade to the closest point on the west side of the ridge, or a second clay plug in the line approximately 50 feet or so north of the first. In order that the judgment as to soil moisture be viable, the first excavation needs to be accomplished and examined during a wet period.

Preliminary Cost Estimate. For comparison purposes, costs of the above work are estimated preliminarily to be between $3000 and $5000, depending upon whether more than the one clay plug is necessary.

Subsurface Grout Injection

Situation Description. Movement of contaminated groundwater above and below the water table at the site is believed to be mostly along discontinuities (secondary porosity) in the intensely weathered Conasauga strata. Further, this water follows some of the same paths that contaminated waste migrated along during and since the waste disposal operation at Trench 7. During operation, the waste level could reach a maximum elevation of 799 feet throughout Trench 7, and as reported in ORNL/TM-8839, various relict zones of contamination are known primarily east and west of the site from that elevation to several feet below the water table (Figs. 2, 3, 4, and 8). Both infiltrating water from the surface and laterally moving groundwater pass through these relict zones of contamination. As previously stated, a maximum of 80 days elapsed between the time that waste was

first pumped to Trench 7 and when radioactivity was first found at the seep. This is rapid movement and indicative of a prominent subsurface pathway or pathways between the Trench and the seep.

Mitigating Action Description. Grout injection into the subsurface will use the same characteristic of the strata that results in the seep and possible leaching of alkalinity but in a manner to alleviate these adverse effects. This characteristic is the secondary porosity of the strata due to its discontinuities. Injection of grout under pressure can be expected to enter and fill these discontinuities in the vicinity of the injection point. Grout enters the wider discontinuities more readily and travels further than in the narrow openings. During the waste disposal operation, the wider discontinuities carried much of the waste and it is reasonable to assume that now these relict wider pathways are a major contributor to the contamination to the seep and leaching of alkalinity. By filling fissures (discontinuities) in the strata, the contamination that resides on the walls of a fissure will be held more tightly in place and the movement of groundwater through it is inhibited or prevented. Grout can be injected above or below the water table and will, therefore, be effective in the zones of percolation and lateral groundwater flow.

Grouts consisting of a mixture of water and solid particles such as Portland cement, fly ash, bentonite, or lime are not generally believed to enter a fissure much less than 0.5 mm in width. However, some narrower fissures can be enlarged by hydraulic pressure during the grouting process. Chemical grouts containing no solid particles and a

viscosity similar to water are obtainable, but some are considerably more expensive than the particulate grouts (Spalding, Hyder, and Munro 1983).

Grout injection in firm rock is accomplished by drilling holes with conventional types of drilling machinery. However, some companies have developed specialized equipment that greatly reduces the time required for grouting in soils and can be used in some intensely weathered rocks. One such company demonstrated its technique in the Conasauga strata at SWSA 6 on September 8, 1983, using a grout slurry of lime and fly ash. The equipment included a unit with multiple injection lances that could be pressed hydraulically to depths of up to 40 feet, and grout injected under pressure up to 100 psi as the lances were withdrawn.

The location of a proposed grouted zone of strata is shown in Figure 10. The grout line will be 15 or more feet in width (a minimum of 3 rows of injection holes wide), and approximately 40 feet in depth. It extends from near the lower part of the east access road, pass the north end of Trench 7, to a point on the west access road that is beyond the groundwater divide. This is a compromise route, partly selected because only a minimum of grading and clearing will have to be performed to accommodate any type of grouting equipment that is selected. Grouting of this route can be expected to inhibit: 1) movement of some of the groundwater through contaminated fissures between the trench and the seep; and 2) movement of groundwater

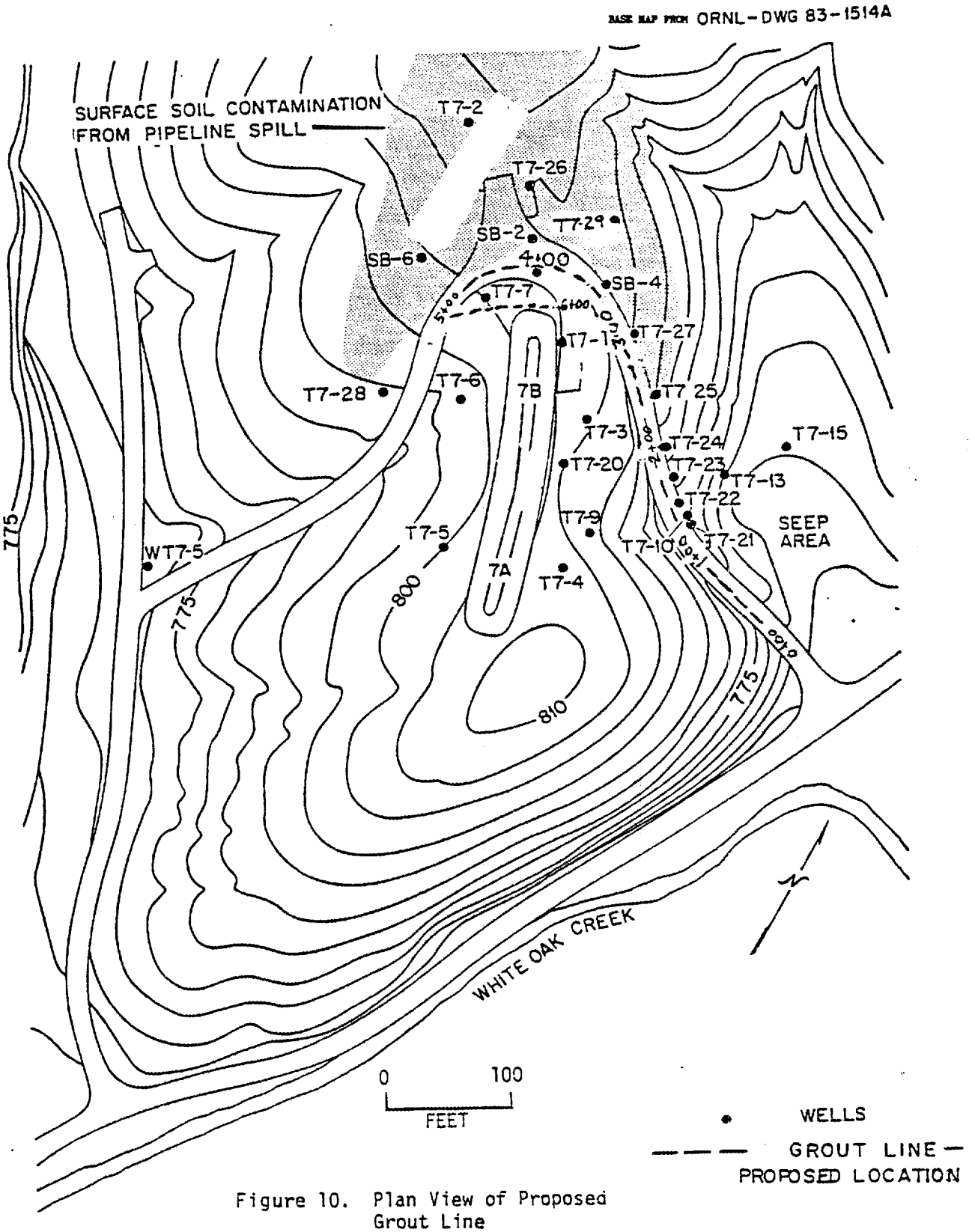

Figure 10. Plan View of Proposed Grout Line

during record high wet seasons through contaminated fissures at high elevations adjacent to the north end of Trench 7. The proposed grouting will physically block some of the major fissures leading to the seep. It is not anticipated that this grouting will cause any significant changes in the water table because there will still be hydraulic connections to both sides of the grout line via routes below the grouted zone and around the ends of the line. A grout line positioned slightly east of the road would have the advantage of separating a larger area, known to contain contaminated fissures, from the seep. However, such a route would require considerable disturbance of the area in the form of clearing and grading in order to provide access for the grouting equipment. The route following the present roadway is therefore proposed as a satisfactory alternative.

Preliminary Cost Estimate. For comparison purposes, costs of grouting by conventional grouting techniques with a particulate grout are initially estimated to be on the order of $150,000. Estimated unit quantities and costs are provided in Table 4. Costs for grouting with a sodium silicate chemical grout, if indicated during grouting or by prior testing, would probably be only slightly more. Based on discussions with personnel of the company which performed the grouting demonstration at SWSA 6, cost by this method would be considerably lower and may be in the range of $100,000.

Table 4. Preliminary cost estimate for construction of grout line at Trench 7; based on assumed line 600' long, 3 rows wide, holes on 5' centers and 40' deep.

Item	Quantity	Unit Cost	Cost
Drill holes	14400 ft.	$4.00/ft.	$57,600
Connections to grout holes	360 ea	$5.00/ea	$ 1,800
Placing grout	9000 cu.ft.	$5.00/cu.ft.	$45,000
Grout	9000 cu.ft.	$4.00/cu.ft.	$36,000
Pipe and fittings	1080 lbs.	$3.00/lb.	$ 3,240
Mobilization and demobilization	job		$ 7,000
Estimated total cost			$150,640

ACTIONS CONSIDERED BUT NOT PROPOSED

Groundwater Interceptor Drain

General. A groundwater interceptor drain at several locations around Trench 7 was considered. The location shown in Figures 9a, and 9b, and used in the recharge source analysis, was deemed to be the best site. At this location only one catch basin is necessary and it would discharge into the drainageway already contaminated by the seep. Further, at this position the drain would have the least chance of withdrawing contaminated groundwater to the catch-basin discharge. Two types of gravity interceptor drains were considered: a surface excavated trench backfilled with crushed stone as was constructed at SWSA 6 (Davis and Stansfield 1984), and a horizontally augered tunnel. The tunnel would have been 2 feet in diameter and lined with a perforated steel liner. No vertical drainage wells would have been necessary with the tunnel drain as it was large enough so that the groundwater cone of depression would not have risen above the elevation of the tunnel. Both types of drains would have been approximately 380 feet in length and 35 feet in depth at their deepest point.

Preliminary Cost Estimate. For comparison purposes, the cost of the surface excavated drain is initially estimated to be $90,000. This estimate is based upon costs for the SWSA 6 project and includes $40,000 for the rental, mobilization, and demobilization of the a backhoe similar to the one used on that project. The cost of boring

the horizontal tunnel drain, including the perforated steel liner is initially estimated to be $40,000. This includes $30,000 for boring the tunnel and setting the steel liner, and $10,000 for site excavation for the boring machine and construction of the catch basin.

The groundwater interceptor drain is not presently proposed for reasons discussed under geohydrology. Briefly, in this particular situation, it is not believed to be as cost effective as the proposed actions.

Sealing the Surface of the Recharge Area

General. Sealing the ground surface to prevent infiltration of groundwater through zones of relict contamination undoubtedly would be helpful. The recharge source analysis indicates this to be a major source of recharge at the site. However, as estimated from the water table contour map and shown in Figure 9a, this would mean sealing a surface area of 1.78 acres. Groundwater recharge at the site would still occur by lateral recharge at depths that could not be intercepted by gravity drains, and contaminated zones presently exist below the water table. Therefore, contaminated water would continue to surface at the seep area, although the level of contamination would be anticipated to be reduced.

Preliminary Cost Estimate. For comparison purposes, the cost of sealing 1.78 acres is initially estimated to be on the order of $184,000. This estimate is based directly upon a 1978 initial

estimate by ORNL Engineering Division of $21.33 per square yard for sealing SWSA 4.

Slurry Trench

A slurry trench was superficially considered as it would provide a barrier of extremely low permeability. However, at an estimated unit cost of between $10 and $20 per square foot, the total cost would be several times that of the grout line that is proposed. Also, pressure grouting can penetrate fissures and immobilize contaminates held therein which is not accomplished by slurry trenches. Further, a slurry trench would have the added disadvantage of having to excavate and dispose of contaminated strata.

MONITORING RESULTS OF MITIGATING ACTIONS

Mitigation of Adverse Effects

It may take a period of several years to determine whether the collective actions taken effectively lessen the level of radionuclide contamination of the seep water and the leaching of alkalinity from Trench 7. This is due to residual contamination at the seep and east of the proposed grout line. However, sampling and analysis of the seep water at the dry and wet seasons can be expected to show a decrease in contamination with time.

Effectiveness of Construction

Controlling Surface Runoff. The effectiveness of the action of diverting surface runoff from the asphalt surfacing to outside the recharge area of Trench 7 could be determined by measuring the runoff under present conditions. After construction, runoff from the gutters can be measured. Any increase in the percent of runoff determined, represents water that prior to the mitigation action, would have been contaminated by infiltration in the immediate area of Trench 7.

Securing Abandoned Waste Transfer Pipeline. The effectiveness of severing and plugging the transfer pipeline and backfilled ditch may not be specifically measurable. However, under the circumstances it would seem to be good practice to be assured of the security of this potential source. In the perhaps unlikely event it was considered

necessary to install a drain from the backfilled ditch, the outlet pipe could be monitored for discharge.

Grout Injection. The effectiveness of grouting programs are generally related to the reduction in hydraulic conductivity obtained. The quantity of grout placed is not a reliable indicator as to this reduction. To determine the reduction, several tests would have to be conducted in the grouted zone, before and after the operation.

CONCLUSIONS

Interdependence and Timing of Mitigating Actions

A summary of proposed mitigating actions with preliminary cost estimates is shown in Table 5a. A summary of actions which were considered but not proposed at this time is shown with cost estimates in Table 5b. The success of each of the proposed actions is independent of the others, and no order of action is necessary. However, of the three proposed mitigating actions, the proposal extending the asphaltic cover, and diverting the collected runoff to outside the recharge area of the Trench has the potential to be the most cost effective. The rate of 700 cubic feet of runoff from the asphalt surface per inch of rainfall amounts to an average of over 36,000 cubic feet of water per year. As previously discussed, a sizable portion of this is suspected to be infiltrating to the groundwater table immediately adjacent to Trench 7.

Only the proposed action to sever and plug the abandoned waste transfer pipeline carries a preference as to the best season for construction. Because of the judgement required as to whether the backfill in the old ditch collects and transports water during wet weather, it would be helpful to accomplish this task in late winter or early spring.

Table 5. Summary of preliminary construction costs of possible mitigating actions

(a) Proposed mitigating actions:

	Action	Cost
1.	Extend asphaltic cover and control surface runoff	$ 15,000
2.	Plug waste transfer pipeline and backfilled ditch	$ 5,000
3.	Construct grout curtain between the Trench and the contaminaed seep, and at the north end of the Trench	$150,000

(b) Actions considered but not proposed:

	Action	Cost
1.	Construction of groundwater interceptor drain by open-cut excavation	$ 90.000
2.	Construction of groundwater interceptor drain by horizontal boring machine	$ 40,000
3.	Sealing the ground surface to prevent infiltration of precipitation	$184,000
4.	Construction of slurry trench cutoff	$10 to $20/ sq.ft.

As indicated above, the three proposed mitigating actions can be performed in any order, and none of the three methods effect the success of the other two. This is not the case for the actions considered but not proposed at this time (Table 5b). For example, if either type of a groundwater interceptor drain was constructed prior to construction of the proposed grout curtain, there is a high probability that the drain would become ineffective due to plugging of some portion of it by the pressure-injected grout. However, if for some reason it became desirable to also install a groundwater interceptor drain after a grout curtin had been constructed, it would be possible to do so without affecting the integrity of the grout curtain. Neither would prior grouting of the subsurface adversely effect the subsequent construction of an interceptor drain.

Potential for Groundwater Level Changes Due to Grouting

As previously stated, changes in the potentiometric surface of the groundwater due to grouting are anticipated to be minimal. This is because hydraulic continuity will be maintained around the ends and beneath the grout curtain; also, there will be some leakage through the curtain. However, the Conasauga Group is known to possess anisotropic hydraulic conductivity in directions of strike and dip (Davis et al. 1983) and may well possess a similar characteristic in the vertical direction. If the hydraulic conductivity in the vertical direction is

lower than in the horizontal direction, there may be a lessening of the equalizing effect of the hydraulic connection beneath the grout curtain on the potentiometric surface of the groundwater. It appears possible, therefore, for the groundwater surface to incur a localized increase in elevation, perhaps on the north or west side of the grout curtain. At some undetermined rise in elevation (it would appear to have to be higher than the record high of February 1962 and for a continued period) new groundwater seeps could possibly develop. If such a seep developed north of the Trench it very well may be free of contamination and no remedial action would be necessary. In the event a contaminated seep did develop in some area, remedial action could perhaps be in the form of additional grouting or drainage, but appropriate action would depend upon the situation at the time.

Test Grouting

Test grouting a short section at the site would be most helpful prior to contracting the main program. From this test it could be established whether the strata will accept particulate grout or if chemical grout will be required. Also, this is the best method to estimate the amount of grout that will be injected during the full program. A section of perhaps no more than 50 feet in length would be adequate for testing purposes and could be incorporated as a portion of the designed line.

Replacing and Modifying Existing Monitoring Wells

During the grouting program, monitoring wells adjacent to the grout line will be blocked with grout. It will be necessary to replace several of the wells but not all, as some have served their purpose. The replacement of monitoring wells has not been included in the preliminary estimate of grouting costs.

The first monitoring wells drilled at Trench 7 in 1961 and 1962 were drilled to depths of up to approximately 70 feet. It may be that there is vertical movement of groundwater in some of these deeper wells. If so, consideration should be given to sealing these wells to prevent cross contamination between various depths, or maintaining groundwater mounds around any wells with possible upward flow. To determine if this condition exists, piezometers could be set in a sample of these wells (T7-1 and T7-2) approximately 30 to 40 feet below the ground surface.

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Appendix D

OAK RIDGE NATIONAL LABORATORY

OPERATED BY MARTIN MARIETTA ENERGY SYSTEMS, INC.
POST OFFICE BOX X, OAK RIDGE, TENNESSEE 37831

DATE: DECEMBER 1985

SUBJECT: PRETREATMENT HYDROLOGIC MONITORING AND CORRECTIVE MEASURES IMPLEMENTATION AT LIQUID WASTE TRENCH 7

TO: DISTRIBUTION

FROM: P. J. MULHOLLAND

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ABSTRACT

To reduce infiltration in the area immediately adjacent to Low Level Waste Trench No. 7 and thereby limit the movement of groundwater from the trench toward a surface seep area to the east of the site, the asphalt cap over the trench was extended and recontoured to reroute drainage away from the perimeter. Before cap extension, the asphalt cap covered approximately 880 m^2 (18% of the site drainage area) and runoff from the asphalt surface could potentially infiltrate the soil at the cap perimeter. After extension and recontouring, the cap covered approximately 1750 m^2 (36% of the site drainage area), and runoff from the asphalt surface was routed via a drain and culvert to an ephemeral stream approximately 30 m down-gradient of the new cap perimeter. Surface runoff monitoring at ephemeral drainages to the east and west of the site was conducted during a 60-d period immediately before (April 24 to July 1, 1985) and for a 41-d period immediately after (August 24 to October 3, 1985) cap extension and recontouring.

Surface runoff from the site increased from 8.4 to 12% of precipitation, and the minimum rainfall necessary to generate surface runoff declined from 9 to 1.8 mm following cap extension and rerouting. Further, following the remedial action, the distribution and intensity of surface runoff changed from a slight dominance via the east drainage to a large dominance via the west drainage. Before cap extension, at least 45% of the precipitation intercepted by the cap infiltrated the soil, presumably at the cap perimeter. After cap extension, recontouring and curbing, 60% of the precipitation intercepted by the cap could not be accounted for as site runoff or as interception storage and evaporative loss from the cap surface, and could be the result of (1) infiltration of drainage water in the unlined ditch into which the culvert discharges, (2) an underestimate of evaporative loss, or (3) an underestimate of site runoff because of the large, sharp pulses of flow after cap extension. Alternatively, a leak may have developed in the drain/culvert system that could allow infiltration in the vicinity of the trench. Monitoring of surface runoff should be continued, and the drain and culvert should be tested for leaks to determine the fate of precipitation intercepted by the expanded asphalt cap.

Groundwater elevation at the site during the period from May to August 1985 declined as would be expected during the spring-summer period. Groundwater elevation at the site on May 7, 1985, was approximately 0.2 to 0.6 m higher than that measured on February 1, 1984, and on December 15, 1983. Measurements of groundwater levels in monitoring wells at the site should be continued to determine if the reduction in infiltration in the vicinity of the trench results in lowering of the water table during the winter period (compared with previous years) when it is at a maximum.

INTRODUCTION

The purpose of this report is to describe the surface hydrologic characteristics of the Trench 7 site before, and for a short period after, the extension of the asphalt cap and the routing of intercepted precipitation away from the trench. Groundwater elevations at the site during the study period are also compared with those measured in 1983 and 1984.

A total of 3.2×10^4 m^3 of low-level liquid wastes generated from routine Oak Ridge National Laboratory operations were disposed of in Trench 7 between 1962 and 1966. The wastes contained about 10^7 GBq of fission nuclides (primarily ^{137}Cs and ^{90}Sr), activation products (primarily ^{60}Co), actinides (primarily ^{232}Th and ^{238}U decay series nuclides), and transuranics (primarily ^{241}Pu and ^{241}Am). Gamma-log profiles of the wells near Trench 7 indicate that the waste liquids seeped along discrete layers parallel to bedding and along the strikes of faults and folds (Olsen et al. 1983). Although most of the radioactivity has been retained by sorption reactions with the trench fill, soils, and weathered bedrock, groundwater in the vicinity of the trench shows elevated levels of radioactivity, primarily ^{3}H, ^{99}Tc, ^{60}Co, and ^{233}U.

Seasonal variation in radionuclide concentrations in groundwaters near the trench indicates that radioactive contamination may be leached from the trench or from the relict migration layers when the groundwater level rises to saturate these zones, or when perched water from precipitation seepage percolates into the trench or along these relict migration layers during drainage. Two suspected transport pathways from the trench to a nearby seep area were identified, one of which appears to result in leaching of alkalinity, thus affecting the retention capacity for ^{90}Sr (Olsen et al. 1983). On the basis of chemical, geological, and hydrological studies of the Trench 7 site, Olsen et al. (1983) and Stansfield (1984) recommended remedial actions to reduce infiltration in the area immediately adjacent to the trench and to prevent the movement of groundwater from the trench toward the seep area. A part of this remedial action involves extension of an existing asphalt trench cap to increase the amount of surface water diverted.

SITE DESCRIPTION

Trench 7 is located southwest of the Oak Ridge National Laboratory (ORNL), south of Solid Waste Storage Area 4 (SWSA-4) and west of SWSA-5 (Fig. 1). Fig. 2 is a topographic map of the Trench 7 site showing the location of its monitoring wells, seep area, and gravel access road. The site has a maximum relief of approximately 15 m and covers approximately 4800 m^2 which is drained by two unnamed, ephemeral drainages on the east and west sides - both of which empty into Whiteoak Creek. Also shown in Fig. 2 are the locations of runoff gaging stations established on ephemeral streams draining the site. The site

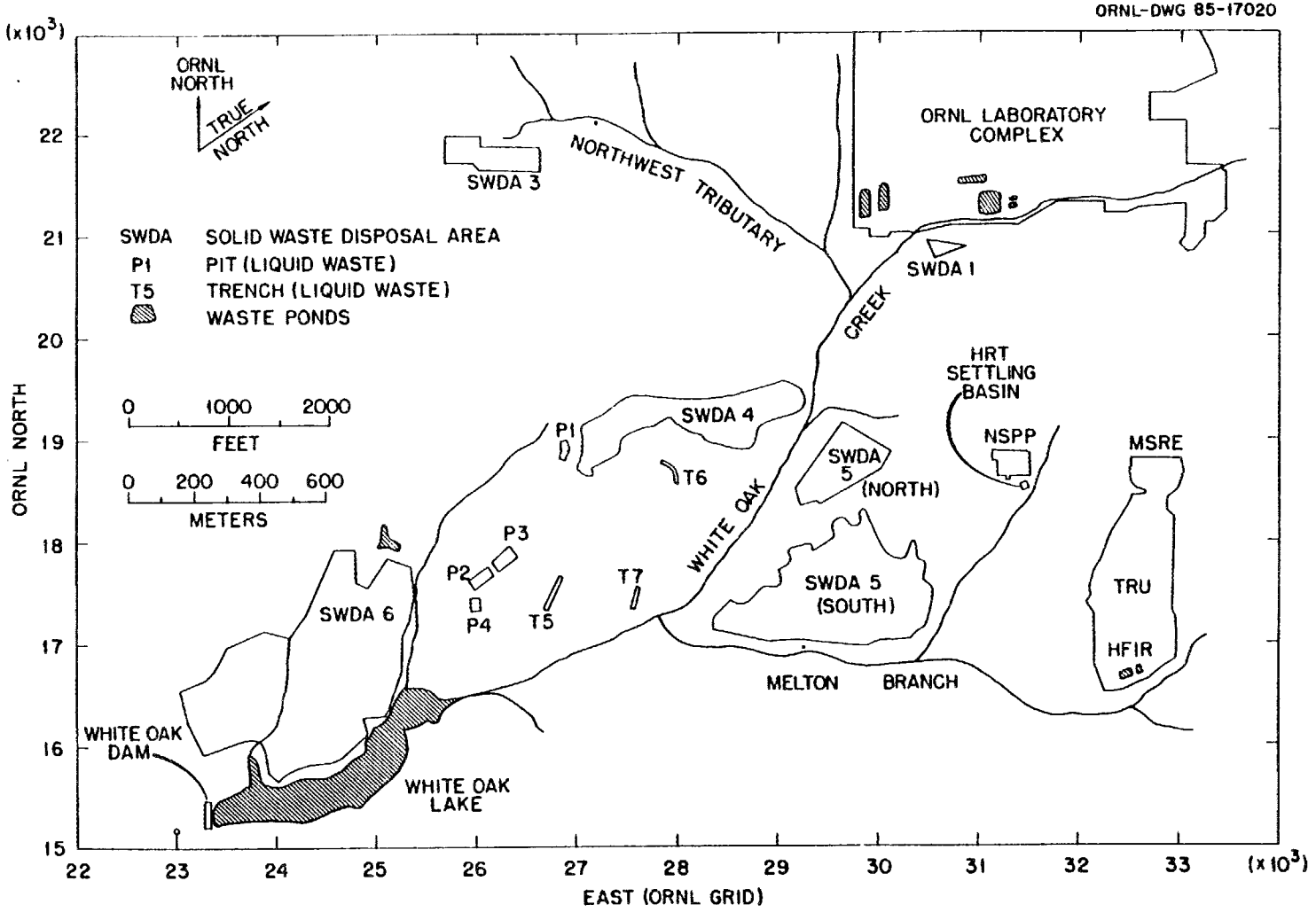

Fig. 1. Map of the ORNL reservation showing location of Trench 7 and other waste disposal areas in the vicinity.

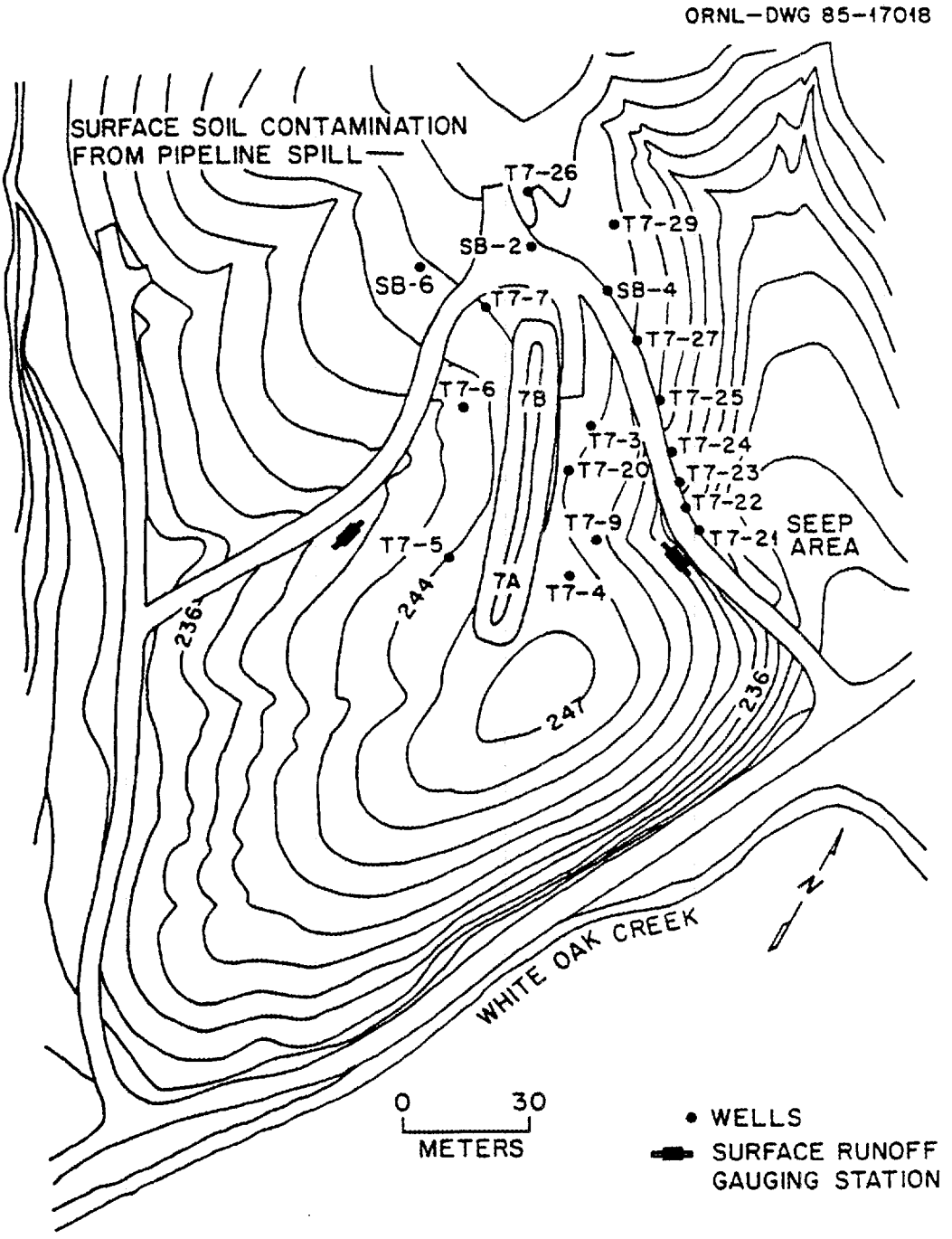

Fig. 2. Topographic map of the Trench 7 site showing locations of monitoring wells and surface runoff gaging stations.

is located in intensely weathered rock of the Conasauga Group on a northwest-southeast oriented ridge just north of Whiteoak Creek. Work at nearby SWSA-6 and 7 indicates that the average strike of strata in this area is 55° NE and dip to the SE, and that folds and faults are common. The shales and siltstones dominating the site are highly fractured and provide excellent secondary porosity for groundwater movement. A more detailed description of site geology is given in ORNL/NFW-84/8 (Stansfield 1984) and ORNL/TM-8839 (Olsen, et al., 1983).

Trench 7 was constructed by excavating two north-south-oriented rectangular pits, each 30.5 m (100 ft) in length and 3.6 m (12 ft) in width at the top, tapering to 1.2 m (4 ft) at the bottom. The pits were excavated to approximately 5 m (16 ft) depth (bottom elevation of approximately 240 m [788 ft] above MSL) and were separated by a nonexcavated section 9.1 m (30 ft) in length. The excavations were filled with crushed limestone to an approximate elevation of 244 m (800 ft) MSL; at this level the pit is approximately 2.7 m (9 ft) wide. The portion of the excavations above this elevation was backfilled with earth materials and was initially at least 0.75 m (2.5 ft) above the ground surface (approximately 245.5 m [805 ft] above MSL) at the centerline, with a 1 vertical on 2 horizontal slope to each side. A 5-cm (2-in.) diam pipeline, which carried the waste to each section of the trench, parallels the west side of the trench at a distance of approximately 7 m (24 ft) from the centerline, with a connecting discharge pipeline to the midpoint of each section at an approximate elevation of 244 m (800 ft) MSL.

Following cessation of low-level liquid waste disposal into the trench, the upper surface of the trench was covered with an asphaltic concrete that extended from centerline of the trench to a distance of approximately 5 and 6 m (16 and 19 ft) on the east and west sides, respectively, making the total width of the cover approximately 12 m (39 ft). As the asphalt surface is approximately 73 m (240 ft) in length, this represents a surface area projected to a horizontal plane of approximately 880 m^2 (18% of the area drained by the gaged ephemeral streams). A layer of gravel, approximately 15 to 30 cm (6-12 in.) deep in most areas, rims the asphalt cover to a width of up to 3 m (10 ft). Further details, drawings, and photographs of trench construction and operation are included in a report by Stansfield (1984).

The ground surface along the east side of Trench 7 is higher than that along the west side and is nearly horizontal. Therefore, much of the rainfall striking the cover drained from east to west (Stansfield 1984). Further, a large portion of the runoff from the asphaltic cover could be infiltrating to the groundwater through radionuclide-contaminated soil immediately adjacent to the trench. Some of the runoff could conceivably be directly entering the trench through potential permeable pathways such as the pipelines, ditch backfill, and remnant crushed stone fill originally stockpiled on the west side of the trench.

A groundwater table (potentiometric) contour map is presented in Fig. 3 and is based on water levels measured in monitoring wells on February 1, 1984 (Stansfield 1984). Based on the potentiometric surface, it is inferred from Fig. 3 that the groundwater divide is

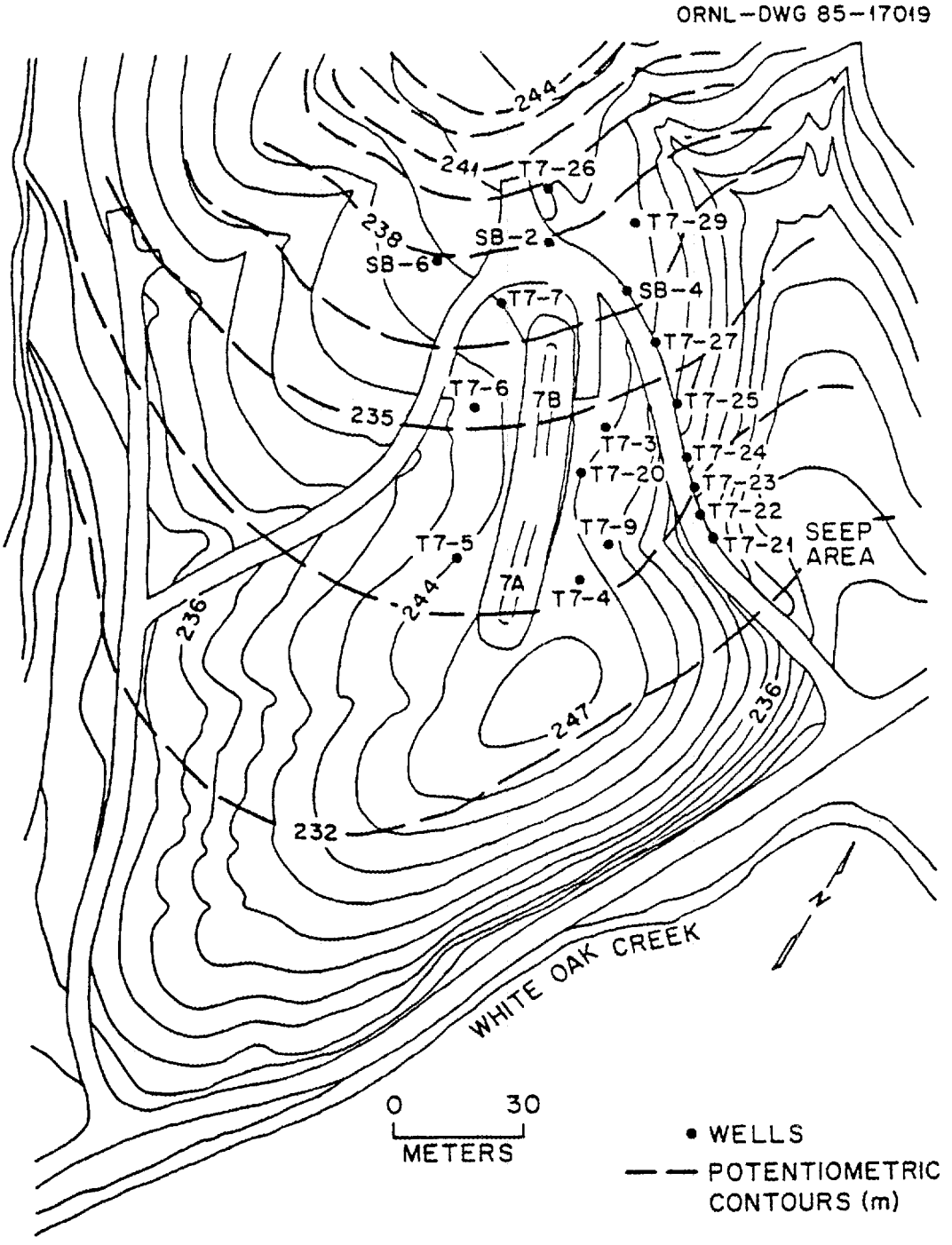

Fig. 3. Groundwater table contour map of the Trench 7 site for February 1984 (from Stansfield 1984).

somewhat west of the topographic divide at the site. This indicates that some of the recharge to groundwater from infiltration on the upper portion of the west slope of the ridge could move eastward under the trench toward the seep area. Further details on subsurface hydrology are given by Stansfield (1984) and Olsen (1983).

From his investigation of construction, operation, closure, and site geology and hydrology of Trench 7 (documented in ORNL/NFW-84/8), Stansfield (1984) concluded that surface infiltration rather than lateral, high water-table flow from the north was a more important source of water to the contaminated seep area. To prevent surface infiltration and thereby minimize recharge to the groundwater in the Trench 7 vicinity, Stansfield proposed that (1) the asphaltic cover on Trench 7 be extended and runoff from the extended impermeable cap be collected with a gutter system and transported down-gradient; (2) the waste transfer pipeline and the backfilled ditch be severed and plugged at a point north of Trench 7; and (3) a grout injection into the subsurface be made between the contaminated seep and Trench 7, and also along the north end of Trench 7.

DESCRIPTION OF CORRECTIVE ACTION

Plans for three separate remedial actions were developed as a result of Stansfield's proposal. These actions are as follows:

(1) Extend the present asphalt surfacing and add a curb to transport surface runoff away from the trench area via a catch basin and drain pipes.

(2) Expose the existing 5-cm (2-in.) waste transfer supply line to the trench, cut and remove a 1.2-m (4-ft) section, and plug the open ends of the line. Backfill the excavation with an impermeable clay to serve as a barrier to the flow of water along the pipeline.

(3) Construct a grout barrier on the east side between the seep and the trench by pressure grouting approximately 12 m (39 ft) deep to seal fissures and discontinuities in the shale and fix the existing contaminants within the fissures. The grout line will consist of holes 1.5 m (5 ft) on center, 3 rows wide, and 183 m (600 ft) long.

Actions 1 and 2 were begun on July 9, 1985, and completed by August 23, 1985. The asphalt cap extension involved vegetation removal and recontouring the site at the perimeter of the existing cap. Soil and gravel were brought in from off-site to build up the adjacent 5-m wide area to the west of the existing cap, which formerly sloped away from the cap. Photographs showing vegetation removal, recontouring adjacent to the trench perimeter, and the final asphalt cover with the collection drain are presented in Fig. 4 to 8. The new asphaltic concrete cap

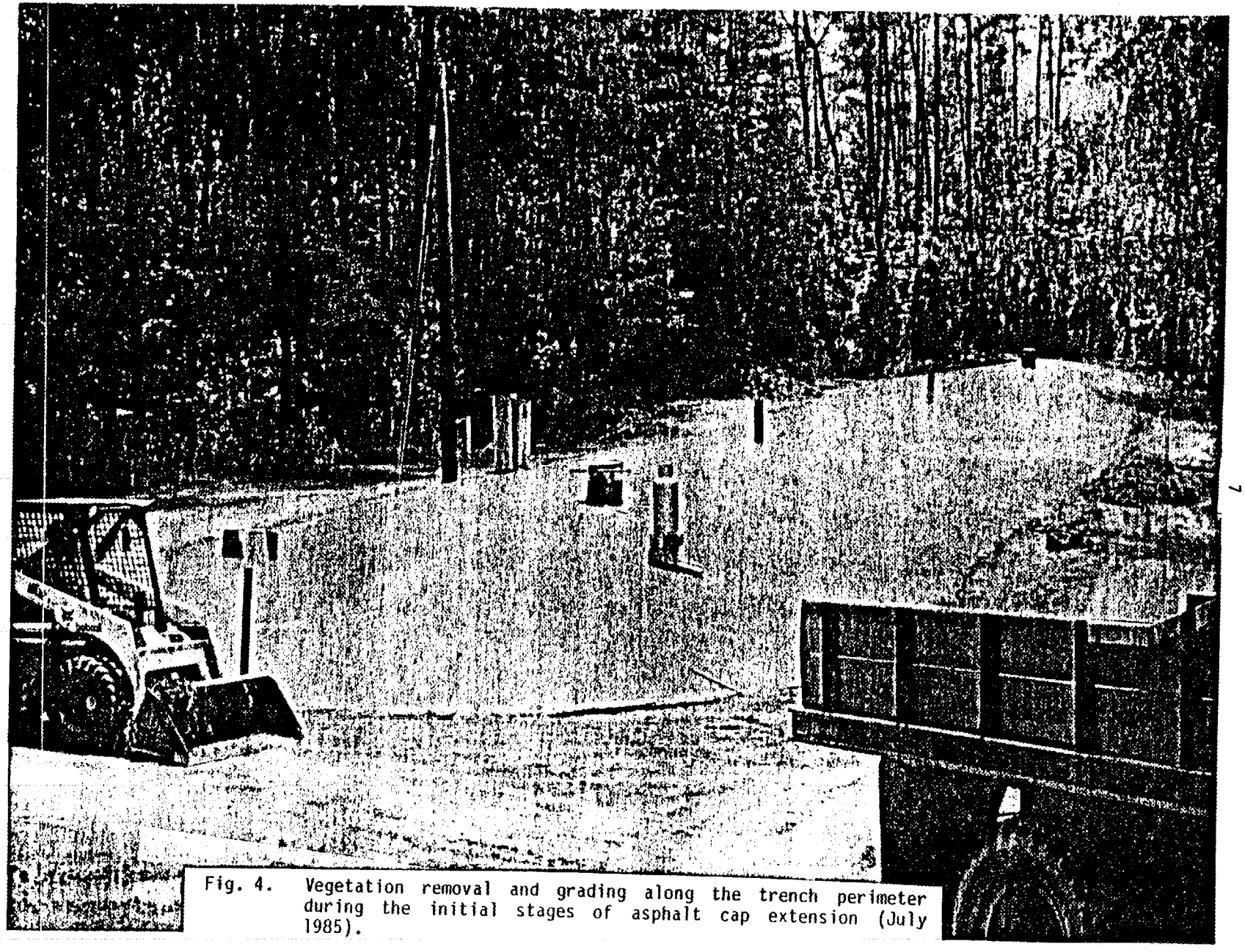

Fig. 4. Vegetation removal and grading along the trench perimeter during the initial stages of asphalt cap extension (July 1985).

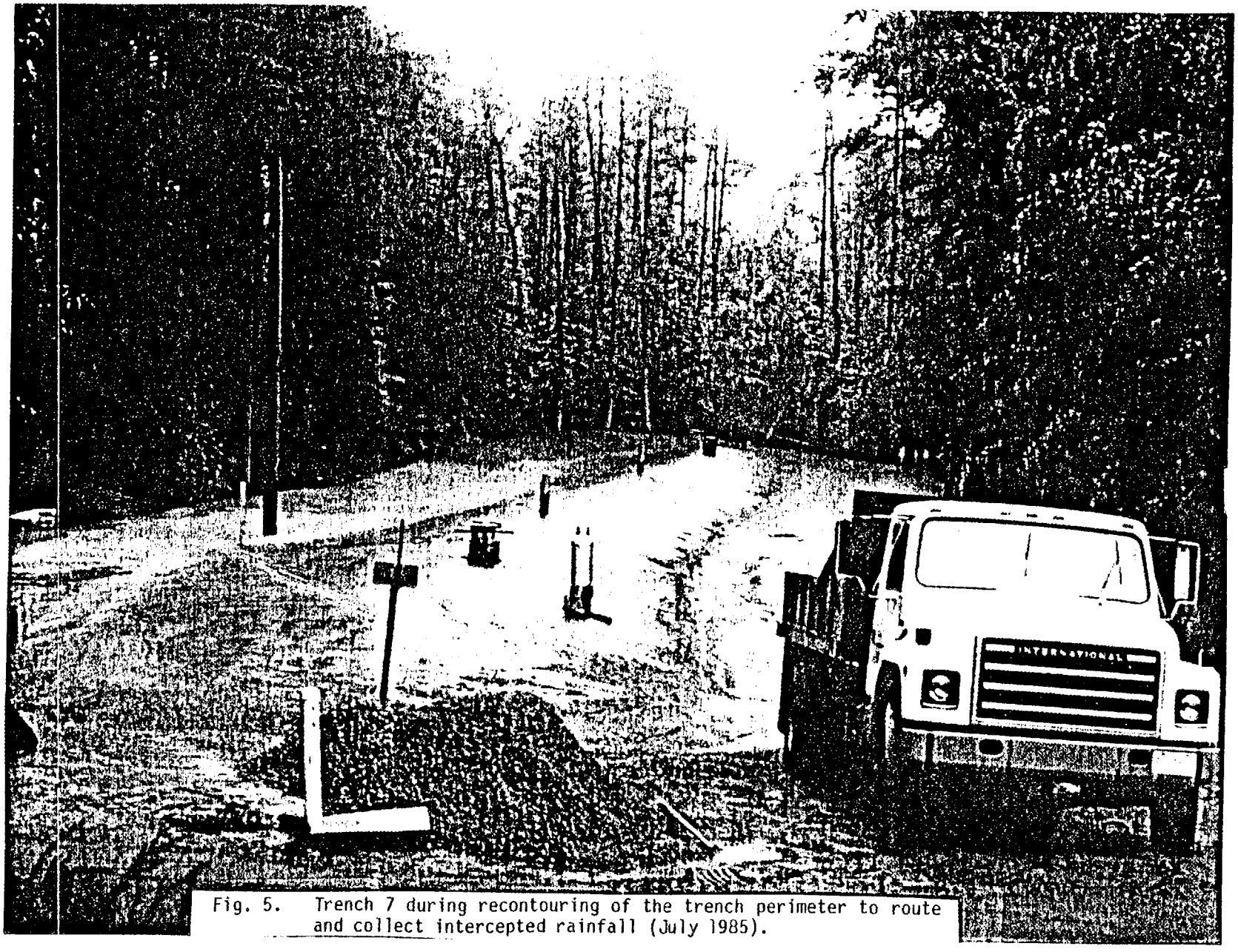

Fig. 5. Trench 7 during recontouring of the trench perimeter to route and collect intercepted rainfall (July 1985).

Fig. 6. Trench 7 at the completion of the asphalt cap extension project (August 1985).

Fig. 7. Runoff collection drain built into the extended asphalt cap (August 1985).

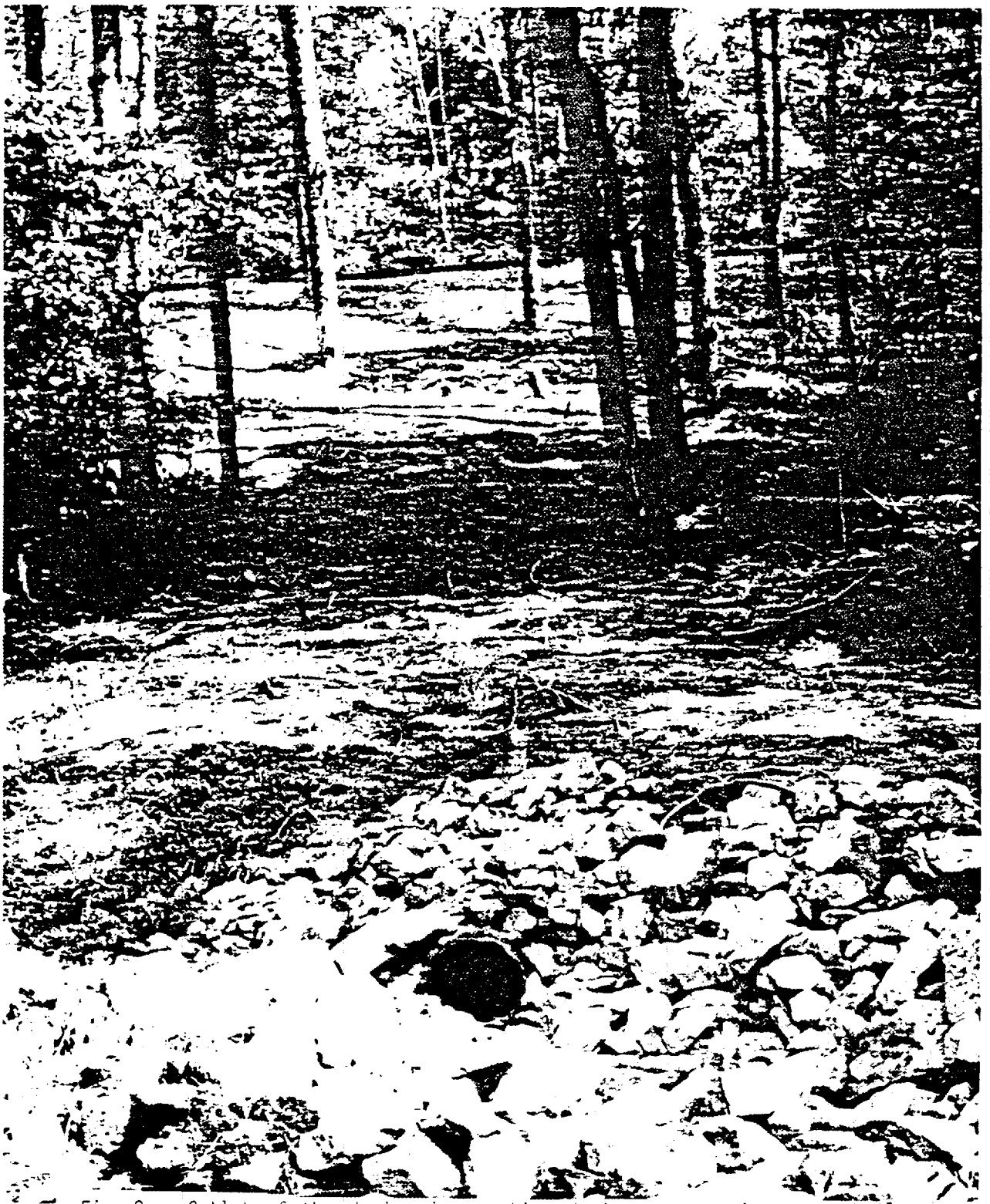

Fig. 8. Outlet of the drain pipe routing drainage water intercepted by the asphalt cap and drain to the ephemeral ditch to the west of the trench (August 1985).

extends the existing cap (approximately 12 m [39 ft] in width) to give a new cap width of approximately 22 m (72 ft) (Fig. 9). The extension was greater on the east side of the old cap (approximately 7 m) than on the west side (approximately 3 m). An integral asphalt curb was formed into the cap at its perimeter. All water intercepted by the cap is now collected by a new catch basin (Fig. 7) located in the cap and routed via a PVC pipe to the west drainage which is approximately 31 m (100 ft) from the cap perimeter.

The grout barrier installation (Action 3) was scheduled to begin in October 1985. Its impact on site hydrology is not included in this report.

SITE HYDROLOGIC MONITORING PRIOR TO CAP EXTENSION

Methods

To measure and continuously monitor surface runoff from the Trench 7 site, streamflow gaging stations were established on ephemeral streams to the east (Fig. 10 and 11) and west of the site (Fig. 12). The stations were constructed by (1) excavating a pool within the existing channel, (2) building a dam at the downstream end of the pool, (3) implanting a corrogated steel pipe in the dam to drain the pool, and (4) covering the pool bottom, the ditch leading into the pool, and the dam face with a heavy plastic liner. A combination V-notch/rectangular weir plate was mounted in the pipe at the downstream outlet. A small standpipe was mounted in the pool, and water level was continuously recorded by a Manning Portable Flowmeter (Series F-3000A) driven by a 12-V battery. Fig. 10 is a photograph of the east gaging station.

A theoretical rating curve for the combination V-notch/rectangular weir was developed (Fig. 13). For flow constrained entirely within the V-notch (less than 0.14 L/s), flow (Q, L/s) was computed according to the equation for a 90-degree triangular, sharp-crested weir:

$$Q = 1327\ H^{5/2},$$

where H = head above base of V (m).

For flows that exceeded the capacity of the V-notch, flow was computed as the sum of the maximum V-notch flow (12 L/s) and the flow computed according to the equation for a rectangular, sharp-crested weir:

$$Q = 1837\ BH^{3/2},$$

where H = head above rectangular base (m),

B = weir width (m).

Field measurement of water level and flow closely agreed with the theoretical curve for flows less than 1 L/s. Measurement of water level and

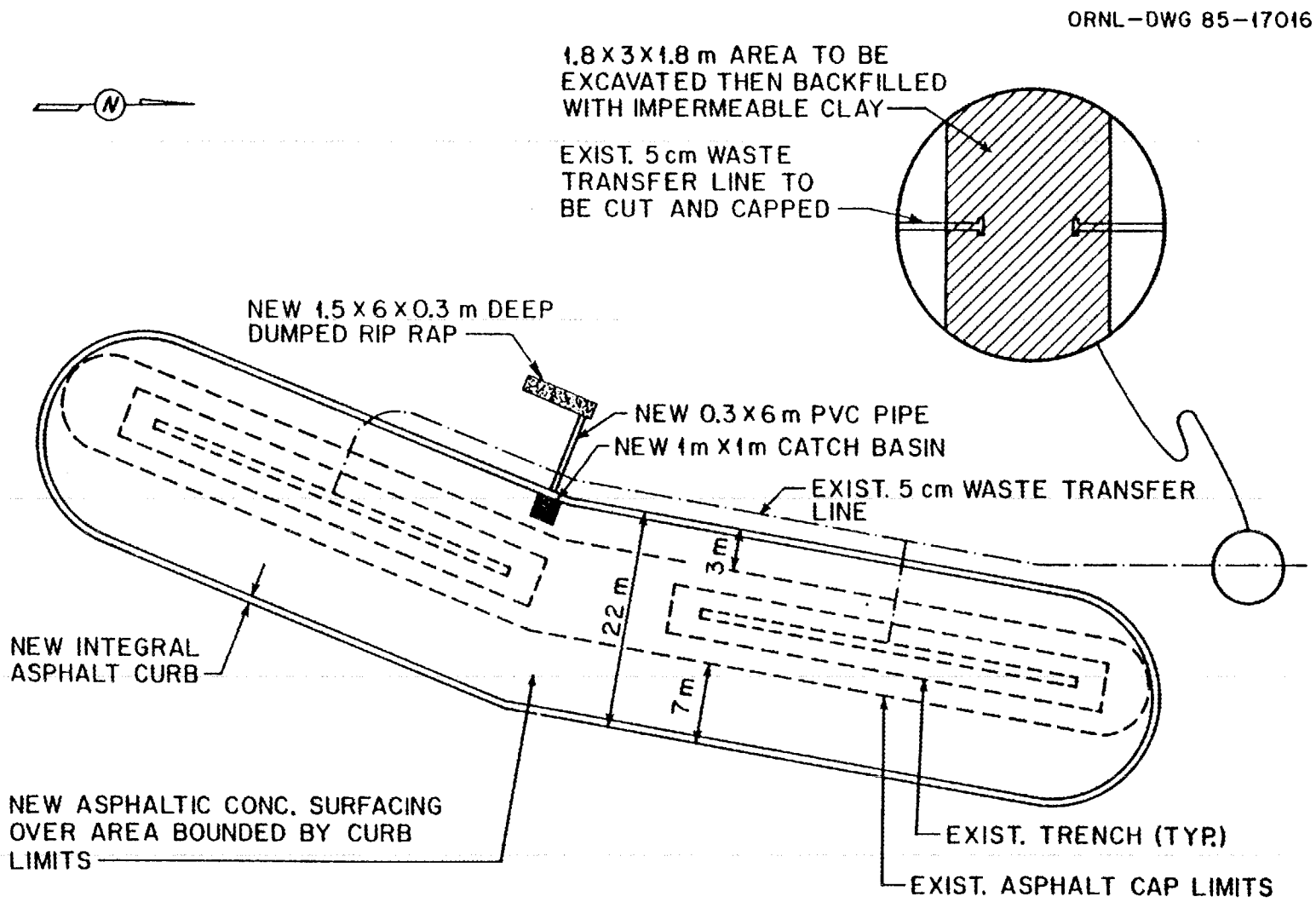

Fig. 9. Plan view of the Trench 7 site indicating cap extension project plans.

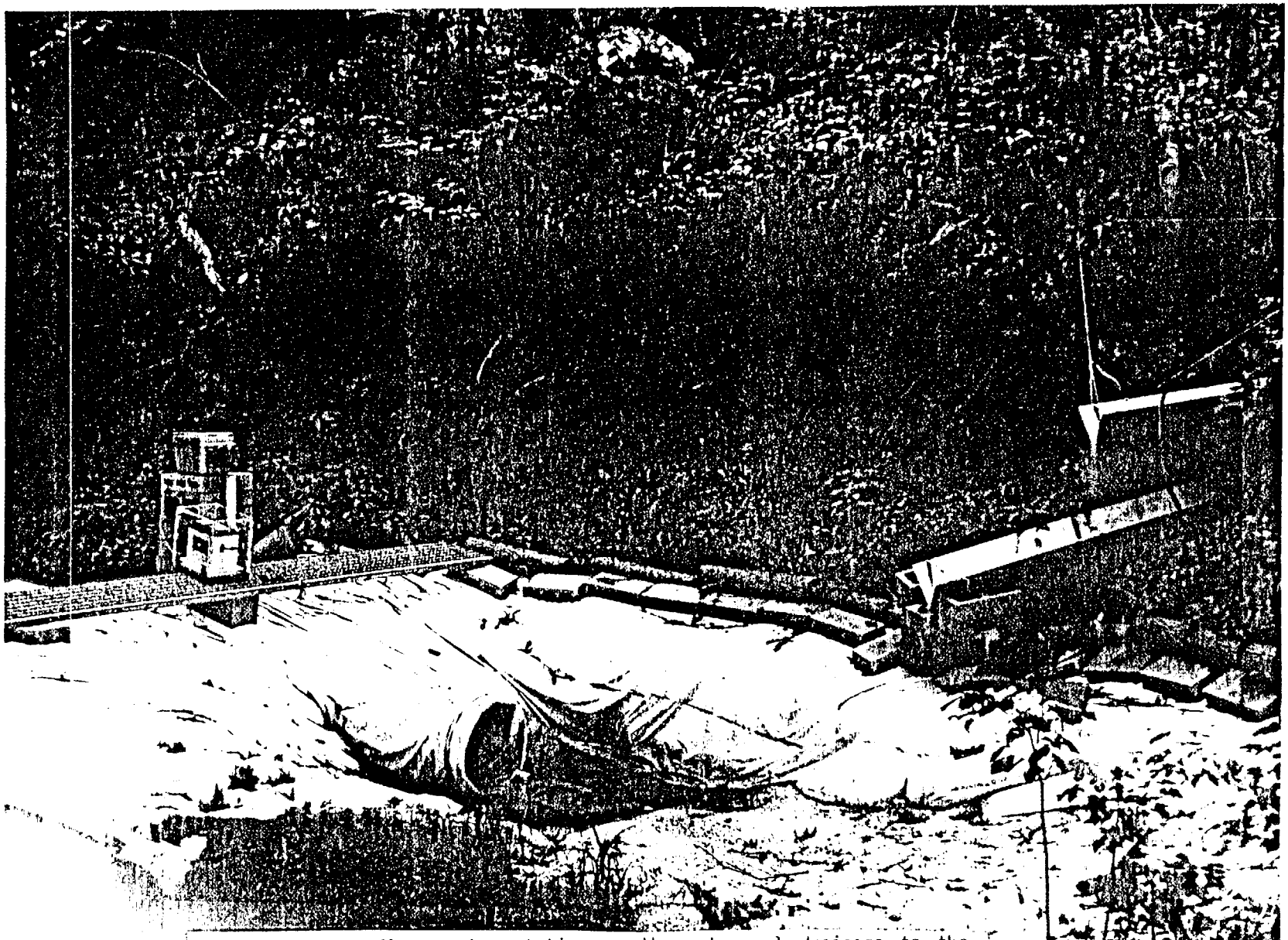

Fig. 10. Streamflow gaging station on the ephemeral drainage to the east of Trench 7, upstream view (May 1985).

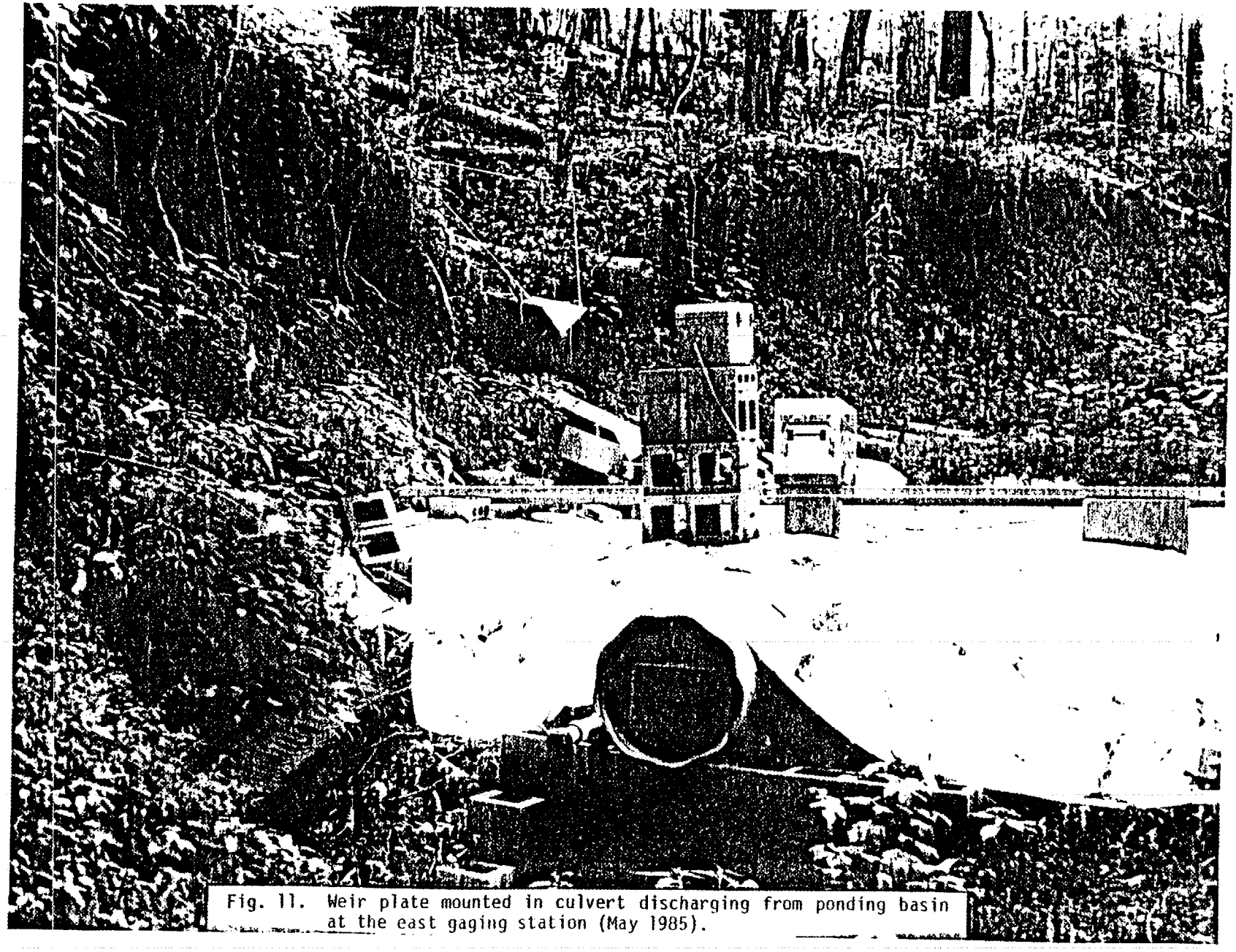

Fig. 11. Weir plate mounted in culvert discharging from ponding basin at the east gaging station (May 1985).

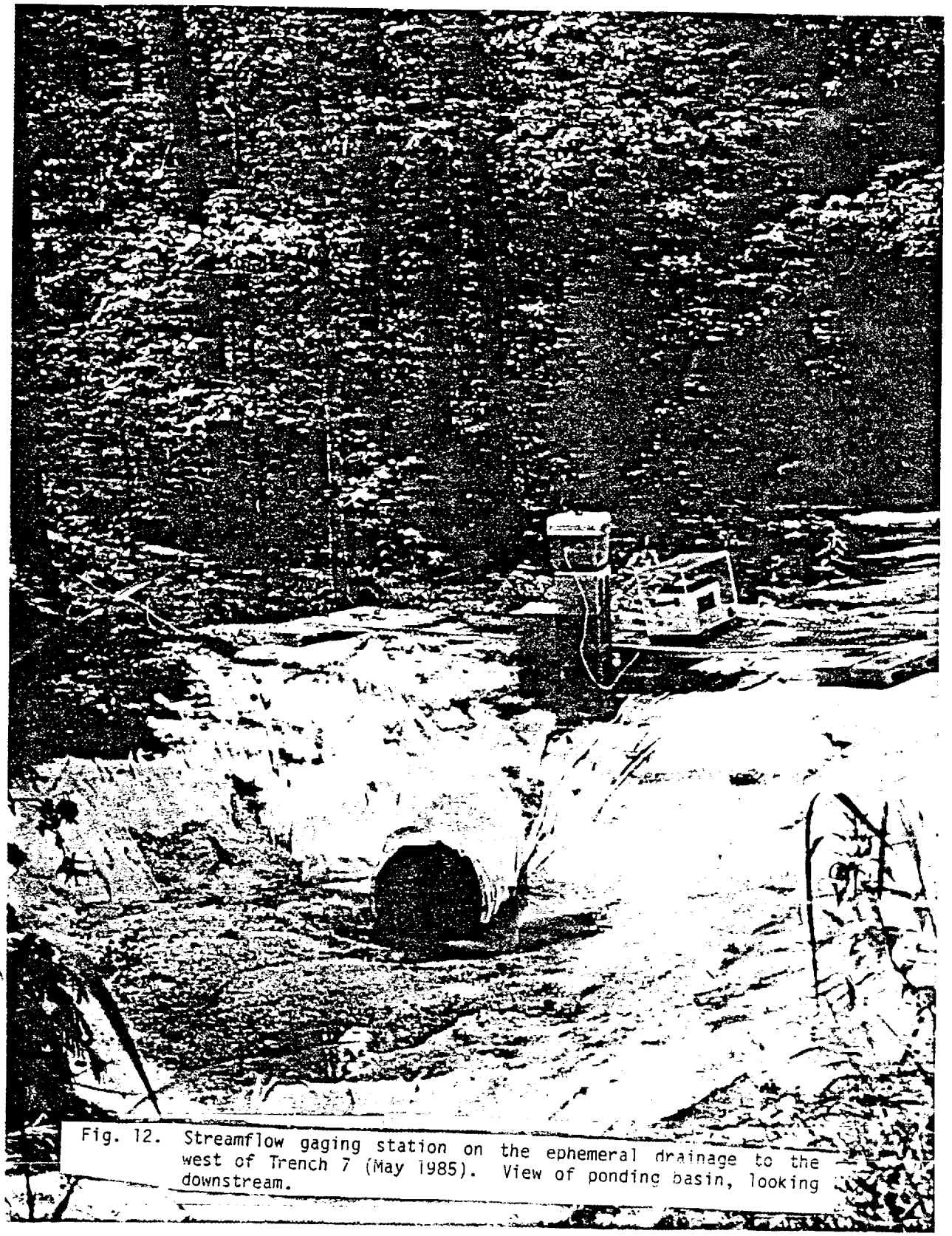

Fig. 12. Streamflow gaging station on the ephemeral drainage to the west of Trench 7 (May 1985). View of ponding basin, looking downstream.

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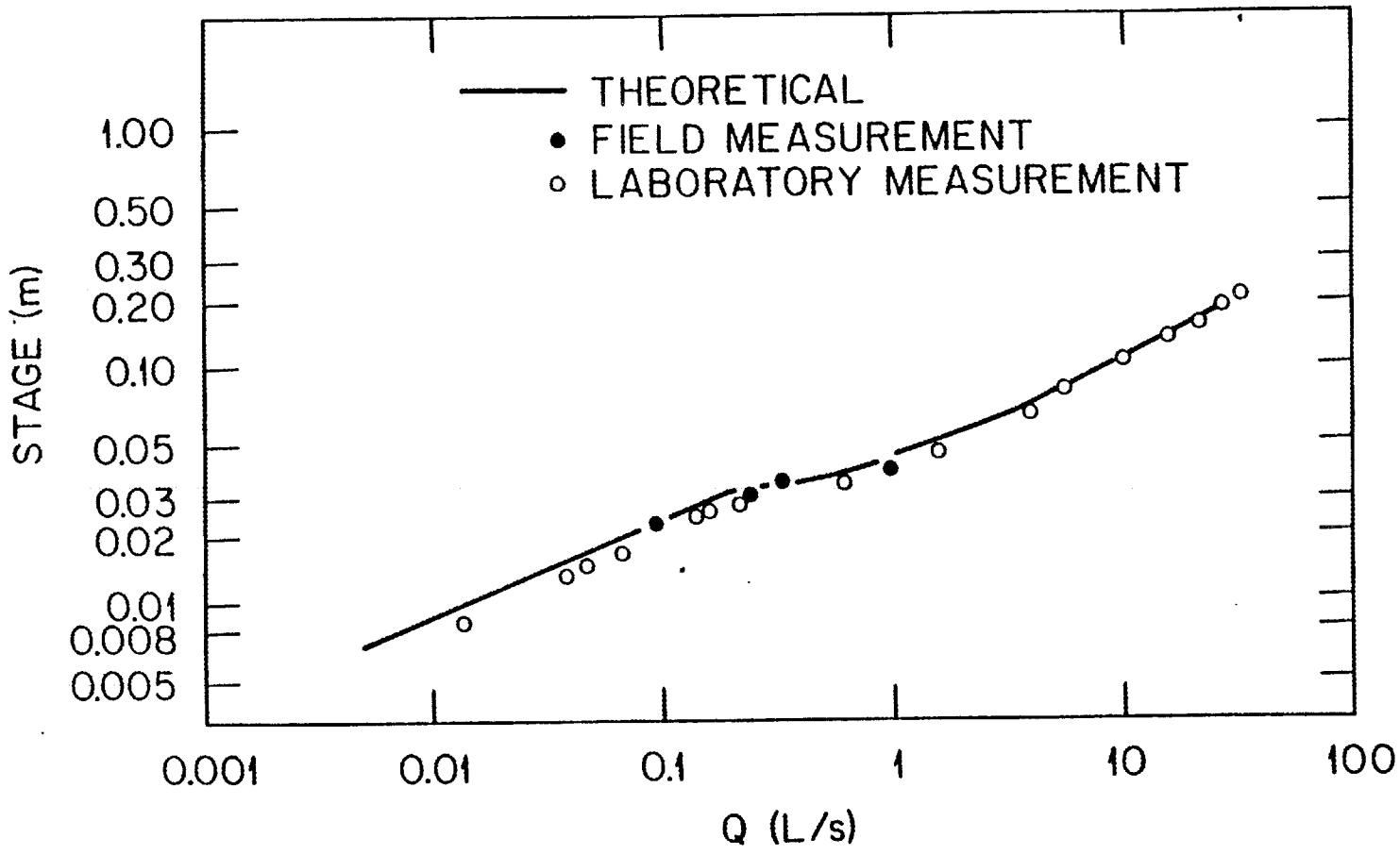

Fig. 13. Theoretical rating curve for the combination V-notch/rectangular weir used at east and west gaging stations at Trench 7. Field and laboratory measurements are also indicated on figure.

flow in a laboratory flume into which the weir had been mounted also closely agreed with the theoretical curve. The theoretical rating curve was used to compute flow from stage height at both weirs.

To determine changes in the water table at the site, groundwater level was monitored during the period from May through August 1985 by measuring depth to water in 17 of the monitoring wells (see Fig. 2). Measurements were taken at 2- to 4-week intervals during the period.

Rainfall at the site was collected and continuously recorded by a Belfort 30.5 cm (12-in.) reversing rain gage (Mfg. No. 8130). The rain gage was positioned at the center of the trench during the period April 24 to July 1, 1985, and during the period August 26 to October 3, 1985. During the period of construction, July 9 to August 25, 1985, the rain gage was positioned at the west runoff monitoring station. Rainfall data during this period may be influenced by a partial tree canopy at this location.

Results

During the precap extension period from April 24 to July 1, 1985, there were 29 dates on which measurable (greater than 0.1 mm) rainfall was recorded. Surface runoff was recorded on seven of those dates, with a minimum of approximately 9 mm of rainfall needed to initiate measurable runoff (0.1 mm) (see Table 1). For the six dates on which runoff was measured at both stations, totals varied from 3.2 to 16.9% of precipitation, with the larger values generally occurring for larger rainfall events.

Total precipitation at the Trench 7 site during the 69-d period from April 24 to July 1, 1985, was 197.8 mm with a maximum 1-d value of 34.5 mm. Total runoff recorded at the east and west gaging stations was 15.6 mm, although this value does not include flow at the east station on June 18, 1985, when the water-level recorder malfunctioned (Table 1). On the basis of the distribution of runoff on June 12 and June 17, 1985, for storms of similar intensity and duration to that on June 18, runoff at the east station was estimated to be approximately 1 mm on June 18, 1985. Therefore, total surface runoff from the Trench 7 site during the precap extension period from April 24 to July 1, 1985, was approximately 16.6 mm, which was 8.4% of precipitation. The distribution of this runoff was skewed toward the east side of the site. Approximately 59% of the total surface runoff during the period occurred via the east drainage.

Using the asphalt cap area (880 m^2) only as a basis, measured runoff from the site during the April 24 to July 1 period totaled 90.7 mm, which was 46% of precipitation. In addition to site runoff, interception storage (wetting the surface) by the asphalt cap and evaporation from the cap during rainfall could account for some of the precipitation intercepted by the cap. Interception storage probably accounts for up to 0.2 to 0.5 mm of incident precipitation during each event. Assuming the larger value, cap interception storage could account for approximately 13 mm of precipitation during the period April 24 to July 1 (26 precipitation events greater than 0.5 mm, Table 1). Rates of evaporation from the cap during precipitation are probably on the order of 0.1 mm/h. Assuming an average event time of 2 h, evaporative losses during rainfall from April 24 to July 1 were approximately 5.2 mm. Therefore, interception storage and evaporation from the cap probably accounted for an additional 18.2 mm (9%) of precipitation

Table 1. Daily precipitation (mm) and runoff (mm) recorded at the Trench 7 site (4800 m^2) during the precap extension period - April 24 to July 1, 1985. No entry under runoff indicates no measurable runoff (less than 0.1 mm) on that date

Date	Precipitation	Runoff East	Runoff West	Runoff Total	Fractional Runoff (%)
April 24	0.5				
April 27	1.6				
May 1	2.4				
May 2	30.9	1.0	0.4	1.4	4.5
May 7	2.8				
May 8	4.0				
May 9	0.5				
May 11	0.8				
May 12	1.0				
May 16	0.8				
May 17	5.3				
May 21	6.9				
May 22	0.1				
May 23	1.2				
May 24	2.3				
May 28	6.5				
June 2	3.8				
June 6	1.7				
June 7	33.8	3.7	2.0	5.7	16.9
June 10	0.2				
June 11	0.6				
June 12	15.7	1.0	0.7	1.7	10.8
June 14	0.3				
June 17	13.0	0.4	0.5	0.9	6.9
June 18	12.6	[a]	0.8	[a]	
June 24	0.9				
June 26	9.1	0.1	0.2	0.3	3.3
June 30	34.5	2.6	2.2	4.8	13.9
July 1	4.1				
Total	197.9	8.8+	6.8	14.8+	

[a] Water-level recorder malfunctioned on June 18, 1985. See text for runoff estimate.

to the cap. Summing runoff from the site with interception storage and evaporative loss from the cap could account for up to 55% of precipitation. Therefore, at least 45% of precipitation intercepted by the cap during the April 24 to July 1 period must infiltrate the soil after leaving the cap surface. Stansfield (1984) suggested that the area immediately adjacent to the cap perimeter may have a large infiltration capacity caused by disturbance during trench excavation and backfilling and by the gravel placed on the soil surface. The data presented here support this hypothesis.

Groundwater elevations during the this study and those from previous studies in 1983 and 1984 are presented in Table 2. Groundwater elevations on May 7, 1985, were generally 0.2 to 0.6 m higher than those on February 1, 1984, and on December 15, 1983. Groundwater level declined during the period from May 7 until August 15, 1985, with the greatest decline (1 m on the average) occurring in the area to the north-northwest of the trench at the groundwater elevation maximum (wells T7-26, T7-29, SB-2, SB-6; see Fig. 2). Groundwater elevation declined least during this period (average decline of 0.25 m) in the area to the east of the trench where groundwater level was at a minimum (wells T7-21, T7-22, T7-23). Precipitation at the site during the period May 7 to August 15, 1985, was 587 mm. By August 15, 1985, groundwater elevations at the site were very similar to those on December 15, 1983, and generally lower than those on February 1, 1984.

Based on potentiometric contours developed from the February 1, 1984, groundwater level survey, Stansfield (1984) concluded that the groundwater divide was west of the topographic divide, and therefore, slightly west of the trench at the site (Fig. 3). Consequently, Stansfield hypothesized that some of the recharge to groundwater from infiltration on the west slope of the ridge (west of the trench) moved to the east under the trench toward the seep area. The groundwater-level data of 1985 continue to support this hypothesis. For example, groundwater elevations along an east-west axis near the middle of the trench (T7-6, T7-20, and T7-22) indicate a consistent decline from west to east.

One of the primary objectives of expanding and recontouring the asphalt cap was to reduce infiltration of runoff from the cap in the area immediately adjacent to the cap perimeter. Reducing infiltration at the cap perimeter would in turn reduce groundwater recharge in the vicinity of the trench and subsurface transport of radioactive contamination downgradient from the trench (toward the east). Groundwater elevation in the immediate vicinity of the trench and to the east of the trench should continue to decline if the cap expansion is to successfully meet this objective. Measurement of water level in the site monitoring wells, particularly those to the east of the trench (T7-3, T7-4, T7-20, T7-21, T7-22, T7-23, T7-24, and T7-25), should be continued, at least bimonthly, to determine if groundwater elevation has indeed been lowered by this action.

Table 2. Groundwater elevations (m) in monitoring wells at Trench 7 on dates in 1983, 1984, and 1985 (this study)

Well No.	12/15/83[a]	02/01/84[a]	05/07/85	06/13/85	07/10/85	08/15/85
T7-3	---	234.3	235.2	235.0	234.9	234.8
T7-4	---	233.5	234.5	234.3	234.1	234.0
T7-5	233.9	233.7	234.3	234.1	234.0	233.9
T7-6	235.0	234.9	235.1	234.7	234.7	234.5
T7-7	---	238.8	238.1	237.8	237.5	237.3
T7-20	233.9	234.0	234.5	234.3	234.0	233.8
T7-21	233.0	232.9	233.1	233.0	232.9	232.9
T7-22	233.1	233.0	233.2	233.2	233.0	233.0
T7-23	233.3	233.2	233.3	233.1	233.0	233.0
T7-24	233.6	233.7	233.9	233.8	233.7	233.6
T7-25	234.0	234.3	234.6	234.4	234.3	234.1
T7-26	237.9	238.9	239.2	238.7	238.4	238.0
T7-27	235.0	235.0	235.1	235.1	235.0	234.9
T7-29	236.9	237.3	237.9	237.6	237.3	237.1
SB-2	236.8	237.2	237.7	237.5	237.3	237.0
SB-4	236.5	236.7	237.2	236.9	236.7	236.5
SB-6	236.9	237.6	238.1	237.6	237.3	237.0

[a] Groundwater elevations from Stansfield (1984).

SURFACE RUNOFF AFTER CAP EXTENSION

The asphalt cap extension at Trench 7 was completed by August 23, 1985. The new cap represents an increase in impermeable area over that of the old cap by approximately a factor of 2, from 880 m^2 to approximately 1750 m^2. In addition, all precipitation intercepted by the new cap is collected and routed away from the site via a drain system (see Figs. 7 and 8). Formerly, precipitation intercepted by the trench cap was shed to the perimeter of the cap where it could infiltrate the soil.

During the period August 24 to October 3, 1985, measurable rainfall occurred on 13 dates, ranging from 1.3 to 31.2 mm (Table 3). Measurable runoff (greater than 0.1 mm) was recorded at the west gaging station on all of these dates, with the exception of October 3 when less than 0.1 mm (200 L) was recorded. Measurable runoff was recorded at the east gaging station on only two dates (August 24 and September 24, 1985), although no data are available for five dates due to water-level recorder malfunction (see Table 3). Occasional visits to the site on some of these dates indicated that it is doubtful that measurable runoff occurred at the east weir on most of these dates. Total surface runoff recorded for the period August 24 to October 3, 1985, was 17 mm (Table 3). Omitting the period August 26 to September 10, when the east gaging station was inoperable, 75.2% of the total runoff (12.5 mm) from the site occurred via the west drainage. The runoff data following the extension of the asphalt cap at Trench 7 clearly indicate a dramatic shift in the runoff patterns at the site from a distribution skewed toward the east drainage (59% of total runoff from April 24 to July 1, 1985) to a distribution highly skewed toward the west drainage.

Total runoff recorded from the site during the period August 24 to October 3, 1985, (17 mm) amounted to 12.0% of the 141.3 mm of rainfall recorded during this period (assuming negligible runoff via the east drainage on the five dates the station was inoperable). On individual dates, runoff varied from a high of 28% of rainfall on August 24 to 3.8% of rainfall on October 2, 1985 (Table 3).

On the basis of runoff in the expanded cap area only (1750 m^2, 36% of the site), total runoff from the site accounted for 47 mm of the 141.3 mm of precipitation intercepted by the cap surface. Assuming an additional 0.5 mm of interception storage and 0.2 mm of evaporative loss during each precipitation event, approximately 56.1 mm of the 141.3 mm of precipitation intercepted by the cap was accounted for. This leaves approximately 60% of the precipitation intercepted by the expanded asphalt cap unaccounted for. The fate of this water is uncertain, although four possibilities exist:

(1) Runoff from the cap routed to the west drainage via the drain and pipe could partially infiltrate the soil in the 10 m of unlined streambed upstream from the gaging station;

Table 3. Daily precipitation (mm) and runoff (mm) recorded at the Trench 7 site (4800 m^2) during the postcap extension period August 24 to October 3, 1985. No entry under runoff indicates no measurable runoff (less than 0.1 mm) on that date

Date	Precipitation	Runoff East	Runoff West	Runoff Total	Fractional Runoff (%)
August 24	24.6	2.9	4.0	6.9	28.0
August 25	8.9		0.9	0.9	10.1
August 26	9.1	[a]	1.7	1.7[b]	
August 30	12.1	[a]	1.7	1.7[b]	
September 5	1.8	[a]	0.2	0.2[b]	
September 6	9.1	[a]	0.3	0.3[b]	
September 10	8.3	[a]	0.6	0.6[b]	
September 23	11.5		0.6	0.6	5.2
September 24	12.6	0.2	1.2	1.4	11.1
September 26	5.5		0.3	0.3	5.5
October 1	31.2		2.2	2.2	7.1
October 2	5.3		0.2	0.2	3.8
October 3	1.3				
Total	141.3	3.1	13.9	17.0	

[a] Water-level recorder malfunctioned.
[b] Assuming negligible runoff via the east drainage (see text).

(2) Evaporative loss from the cap surface may be much greater than assumed (0.2 mm per event) due to ponding and high heat of absorption of asphalt;

(3) Systematic underestimation of runoff measured at the west gaging station following cap extension due to the large and sharp water-level changes at the west gaging station (occurring within minutes of the onset of runoff) which may have been underestimated if the Manning recorder responded too slowly;

(4) A leak may have developed in the drain-culvert system built into the expanded cap, allowing infiltration of runoff from the cap.

Of the four factors listed above, the first two (infiltration in the unlined streambed and higher than estimated evaporative loss probably occurred to some extent. In particular, evaporative loss from the asphalt surface may have been greatly underestimated by the assumption of the 0.2 mm value for each storm. The perimeter curb anbd contouring of the expanded asphalt cap resulted in much longer contact time between intercepted rainfall and the asphalt surface. High heat of absorption of the asphalt during the warm months of August and September combined with longer contact time likely resulted in much greater evaporative loss of intercepted rainfall following cap extension compared to before extension. However, it is doubtful that underestimation of cap evaporative loss could account for most of the 85.2 cm of intercepted rainfall which was not accounted for. Finally, only the last factor listed above (a leak in the drain-culvert system) could result in infiltration of cap runoff in the vicinity of the trench, and consequently, contribute to groundwater recharge and subsurface transport of radioactive contamination away from the trench.

Despite our inability to account for runoff from the expanded asphalt cap, surface runoff from the site as a whole, as a fraction of precipitation, was greater after than before cap extension, as indicated by the total runoff values and the range of values for individual dates (Table 4). In addition, the amount of precipitation needed to generate measurable runoff via the east or west drainage declined after cap extension. Before cap extension, 6.9 mm of rainfall on May 21, 1985, resulted in no runoff (Table 1), whereas 1.8 mm of rainfall on September 5, 1985, resulted in about 0.2 mm of runoff (Table 3). Finally the timing of surface runoff at the site has also been greatly altered by cap extension, with runoff occurring much more rapidly at the onset of heavy rainfall and subsiding more quickly than before extension. For example, rainfall totalling 12.1 mm over a 30-min period on August 30, 1985, resulted in 1.7 mm of runoff at the west drainage, with more than 90% of it occurring over a 45-min period. However, before cap extension on June 12, rainfall of 14 mm over a 20-min period resulted in 0.7 mm of runoff at the west drainage and 1.0 mm at the east drainage, with less than 50% of the total runoff occurring in the first 45 min (an additional 1.7 mm of rainfall occurred about 5 h before the 14-mm event but resulted in no runoff).

Table 4. Comparison of site runoff characteristics for a period before cap extension (April 24 to July 1, 1985) with a period following cap extension (August 24 to October 3, 1985)

Period (d)	Total Precipitation	Lowest Runoff Generating Precipitation (mm)	Total Runoff (mm)	Fractional Runoff (%)		Runoff Distribution	
				Total	Range	East	West
Pre-extension (68 d)	197.8	9.1	16.6	8.4	3.2-16.9	59	41
Post-extension (41 d)	141.3	1.8	17.0	12	3.8-28.0	25	75

CONCLUSIONS AND RECOMMENDATIONS

Extension of the asphalt cap at Trench 7 increased the impermeable area over contaminated liquid wastes from approximately 880 m^2 to 1750 m^2. In addition, the perimeter of the new cap was contoured to route all water intercepting the cap to the site's west drainage via a collecting drain and pipe.

This corrective action resulted in an increase in the fraction of rainfall that leaves the site via surface drainage and a reduction in the time required for runoff to respond to precipitation. In addition, the corrective action greatly altered the distribution of runoff at the site, from slight dominance of the east drainage (59% of total runoff) to large dominance of the west drainage (75% of total runoff). Finally, the corrective action increased surface runoff from small rainfall events. In the 2-month period before cap extension, a 6.9-mm rainfall event produced no measurable runoff from the site, but after cap extension, a 1.8-mm rainfall produced about 0.2 mm of runoff.

The extension of the asphalt cap at Trench 7 increased the impermeable area from 18 to about 36% of the total area drained by the east and west drainages. During the precap extension period April 24 to July 1, 1985, measured surface runoff from the site, interception storage, and evaporation during rainfall accounted for approximately 55% of precipitation intercepted by the cap surface. Because this may also include runoff generated from areas other than the cap surface, more than 45% of the cap runoff infiltrated the soil at the site, presumably with most occurring at the cap perimeter where it could recharge groundwater in the vicinity of the trench. Expanding, recontouring and curbing the cap perimeter and collecting rainfall intercepted by the new cap increased total runoff at the site by about 50%. However, of the precipitation intercepted by the new cap surface approximately 60% (185.2 cm) was not accounted for as site runoff, interception storage, and evaporative loss during the study period after cap extension (August 24 to October 3, 1985). Some infiltration of drainage water may occur in the west ditch in the 10 m between the outlet of the pipe discharging water collected from cap interception of rainfall. It is also likely that evaporative loss from the recontoured cap was underestimated. Therefore, the increase in runoff and evaporation resulting from cap extension and recontouring may be greater than indicated by the data presented here. Finally, it is possible that a relatively large leak has developed in the drain culvert system diverting drainage from the cap surface to the west gaging station. If so, groundwater in the vicinity of the trench may still be recharged to some degree by runoff from the cap surface.

In this study, surface runoff at the Trench 7 site was monitored for a relatively short period after completion of the asphalt cap extension. Monitoring of runoff should be continued, preferably using a 24-h chart interval rather than the 1-week chart interval used for this study, to provide a better record of the relationship between precipitation and runoff. A test of the drain and culvert system should

be made to determine if a leak has developed. Groundwater level in wells at the site should also continue to be monitored, approximately bimonthly, to determine if the reduction in infiltration from cap extension and rerouting of intercepted rainfall results in permanent reduction in groundwater elevations.

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