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In Situ Grouting of Low-Level Burial Trenches with a Cement-Based Grout

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ABSTRACT

A restoration technology being evaluated for use in the closure of ~~one of the low-level~~ radwaste burial grounds at Oak Ridge National Laboratory (ORNL) is trench stabilization using a cement-based grout. To demonstrate the applicability and effectiveness of this technology, two interconnecting trenches in Solid Waste Storage Area 6 (SWSA 6) were selected as candidates for in situ grouting with a particulate grout. The primary objective was to demonstrate the increased trench stability and decreased potential for leachate migration following in situ injection of a particulate grout into the waste trenches. Stability against trench subsidence is a critical issue. For example, construction of impermeable covers over the trenches will be ineffective unless subsequent trench subsidence is permanently prevented.

After grouting, soil-penetration tests disclosed that stability had been improved greatly. For example, refusal (defined as >100 blows to penetrate 1 ft) was encountered in 17 of the 22 tests conducted within the trench area. Mean refusal depths for the two trenches were 3.5 and 2.6 m. Stability of the trench was significantly better than pregrout conditions, and at depths >2.4 m, the stability was very near that observed in the native soil formation outside the trench. The major differences between postgrout tests and tests conducted in native soil formations outside the trench were at soil depths of 1.8, 2.1, and 2.4 m. Tests within the trench showed lower stability within this range probably because of the presence of intermediate-sized soil voids (formed during backfilling) that were too small to be penetrated and filled by the conventional cement grout formulation. Hydraulic conductivity within the trench remained very high (>0.1 cm/s) and significantly greater than outside the trench. Postgrout air pressurization tests also revealed a large degree of intervoid linkage within and between the two trenches. To effectively reduce hydraulic conductivity and to develop stability within the upper level of the trench, injection of a clay/microfine cement grout into the upper level of the grouted trench is planned.

INTRODUCTION

Oak Ridge National Laboratory (ORNL) has recently finished placing an interim covering over ~4 ha Solid Waste Storage Area 6 (SWSA 6), where low-level radioactive wastes (LLW) have been buried in shallow trenches. The final closure of SWSA 6 awaits the completion of a remedial investigation into the nature and extent of contamination as well as the development of effective and safe techniques to stabilize the burial trenches. To select trench stabilization and closure alternatives, a group of 19 burial trenches in SWSA 6 was identified as a demonstration and test area to (a) identify promising trench stabilization and closure techniques applicable to the ORNL setting, (b) carry out these techniques on a field scale in actual LLW trenches, and (c) collect the necessary data to evaluate each technique relative to its feasibility, effectiveness, and cost. Several demonstrations involving dynamic compaction and grouting with a chemical grout have been conducted in this area (1).

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This demonstration was conducted on a small hillock in the northeastern corner of SWSA 6. The site selection was based primarily on two criteria: (a) the site is away from daily waste management activities and (b) most important it is located entirely on high ground and is isolated hydrologically from any peripheral recharge areas that would complicate the formulation of a site water budget and the conducting of performance monitoring as part of the stabilization/closure evaluations. The water table at the site is at least 6 m below the bottoms of the trenches, and the trenches in the area are, therefore, unsaturated throughout the majority of the year. Two trenches, Nos. 151 and 170 were selected as candidates for in situ grouting with a particulate grout. Each of the trenches is ~4.6 m deep. Trench 151 measures 11.5 by 4.3 m, and trench 170 measures ~14.3 m by 4.3 m. The trenches are interconnected, such that the south end of trench 170 overlaps into the north side of trench 151 (Fig. 1).

The primary objective of this demonstration was to determine if in situ grouting with a particulate based grout is a technology that will provide long-term burial trench stability and decreased potential for leachate migration. Stability against trench subsidence is a critical issue. For example, construction of impermeable covers over the trenches will be ineffective unless subsequent trench subsidence is permanently prevented. Trench stability (characterized by soil-penetration tests) and decreased potential for leachate migration (characterized by hydraulic conductivity tests) were measured before and after grouting.

METHODS AND MATERIALS

Soil-Penetration Tests

A nonstandard penetration test has been developed for use over trenches to avoid augering contaminated waste to the surface, as would be expected with the standard ASTM (D 1586-84) soil-penetration test. The nonstandard test uses a 64-kg drill-rig-mounted drop hammer to drive a 5-cm-diam 60° cone point attached to a 4.5-cm-diam drill rod into the ground. The drill rod is marked at 0.3-m lengths, and penetration is measured by the number of blows required to move the device 0.3 m (1 ft) into the ground. In January 1990, 12 penetration tests were conducted in trench 170, and 10 tests were conducted in trench 151. Holes from the penetration tests were used to insert slotted casings, which in turn were used to conduct hydraulic conductivity tests and as injection wells for pumping grout into the trench. Because of the large pore volumes and high permeability within the trenches, boundaries for trench hydraulic conductivity could only be estimated by recording the time required for a known volume of water to permeate into the trench via the soil-penetration holes. In addition to the penetration tests within trenches 170 and 151, penetration tests were conducted outside the perimeter of the two trenches.

Installation of Injection Wells

A previous particulate grouting demonstration in SWSA 6 (2) used lances made of 5-cm-diam Schedule 80 steel pipe as injection wells. These were placed at ~1.5-m centers over the surface of the trench (total of 36). The lances were driven by a 54-kg portable air-driven hammer either to the bottom of the trench or until an obstruction was encountered. Once in place, the lance was pulled up 15 cm and the drive point knocked out of the end for grout injection. When pressures >20 psi or flow rates <23 L/min were encountered, the lance was then pulled up another 15 cm for additional grout injection.

It was observed that five injection wells accounted for ~80% of the grout injected, and it was recommended that future demonstrations use fewer injection wells (e.g., injection wells be based on 6- to 7.6-m centers rather than the 1.5-m center that was used). It was also recommended that smaller-diameter pipe be used because of the difficultly in reaching the bottom of the trench. For example, in most instances the lances could not be driven as much as 2.1 m. The trench bottom was reached in only 4 out of the 36 injection wells installed. Several other operational problems were encountered. For example, because of the closeness of the 1.5-m centers, grout was often pumped directly into one well and out an adjacent well. The poor seal between the lance and surface soil (as a result of moving the lance upward in 15-cm increments during grout injection) often resulted in breakthrough of grout around the injection wells, even at a relatively low grout injection pressure of 15 to 20 psi.

Several changes were made in the installation of injection wells for this demonstration. The major changes included the use of (a) slotted pipe as injection wells and (b) considerably fewer injection wells [10 to 12 per trench compared with the 36 in the 1987 grouting study (2)]. Slotted polyvinylchloride (PVC) pipe (threaded and flush-jointed, Schedule 80, 3.2-cm diam) constructed with three rows of 0.25-cm-wide slots (138 slots/m-row, for an effective filter area of 243 cm^2/m) were placed into the holes made during the penetration tests. Solid sections of similar pipe were installed to at least 0.6 m below the soil surface. To ensure that injected grout did not return to the surface along the pipe/soil interface, a 0.3-m section of 5-cm-diam pipe was placed over the 3.2-cm-diam pipe and driven into the ground to ~8 cm below the soil surface. A cement-based grout collar was then molded around the surface of the pipe/soil interface.

Void-Volume Measurements

The combined void volume of trenches 151 and 170 had previously been determined by filling with water and taking into account losses resulting from seepage into the surrounding soil. This practice may result in serious leaching of radionuclides and/or hazardous wastes into groundwater and may induce premature settling of soil overburden into the trench. It also leaves soil within the trench near saturation levels for a significant period of time, which could have an effect on solidification of grout introduced into the trench. Consequently, flooding trenches with water is no longer an acceptable practice for determining trench void volume. Thus, an alternative nondestructive method was needed for determining trench void volume.

A potential nondestructive method for determining trench void volume is based on pressurization of the trench (treating it as a closed but leaky vessel) and calculation of void volume from the ideal gas law. The trench is obviously not a closed vessel, but experience during grouting exercises has revealed that pressures on the order of 10 to 15 psi can be achieved. After taking into consideration leak rates (similar to those used in determining trench void volume by flooding with water), void volume can be calculated from the ideal gas law. Validation of such a simple technique to determine trench void volume would be a significant accomplishment. For example, one of the most important criterion in determining the effectiveness of in situ grouting is the extent to which the available void space is filled with grout. To do this, it is important that the total void volume of the trench be known. Trenches 151 and 170 were selected for grouting because trench volumes had previously been determined by flooding with water, a technique that may no longer be practiced. Thus, these trenches offered an opportunity to compare the use of alternative methods to determine trench void volume.

This technique is based on the principle that small pressure differentials (<0.5 psi) can be monitored within the trench as a known rate of gas is pumped into the trench. The technique assumes that the void volume of the trench remains constant and that air leaking from the trench occurs across a constant resistance to air flow. The measurement is based on the ideal gas law, where

$$V_0=\frac{n_0RT}{P_0} \tag{1}$$

describes conditions in the trench before pressurization,

$$V_0=\frac{n_xRT}{P_c} \tag{2}$$

describes conditions in the trench after pressurization to a constant pressure, and

V_0	=	void volume of trench,
n_0	=	moles of gases initially present in trench,
R	=	ideal gas constant,
T	=	ambient absolute temperature in trench,
P_0	=	pressure initially present in trench (assumed to be 1 atm),
P_c	=	pressure in trench after pressurization to constant pressure,
n_x	=	$n_0 + n_1$, or moles of gases after pressurization, where n_o is equal to the moles of gas initially present and n_1 is equal to the net moles of gases added to cause increased pressure to P_c.

Thus, the volume of gas added V_1 to result in P_c is equal to

$$V_1=\frac{n_1RT}{P_c}. \tag{3}$$

Combining Eqs. (1) and (2) and assuming no difference in temperature on pressurization,

$$n_0=\frac{n_1P_0}{P_c-P_0}, \tag{4}$$

and substituting Eqs. (1) and (3) respectively for n_o and n_1 results in

$$V_0=\frac{V_1P_c}{P_c-P_0}. \tag{5}$$

To implement this technique, air was pumped into each of the trenches using at least three injection wells for each trench. Pressure as a function of time was also monitored in each of the trenches by using pressure gauges installed in at least three monitoring wells for each trench. These data provided estimates for V_I from which V_o can be calculated. A detail description of the procedure is being prepared (3).

Grout Formulation and Testing

The performance criteria established for grout formulation was the same as that used by Tallent et al. (4); namely,

Apparent viscosity	<50 cP
10-min gel strength	<5 kg_f/m^2
28-d phase separation	0 volume %
28-d compressive strength	>60 psi

Other tests included penetration resistance and consistency.

The dry solid materials used to develop the grout formulation included the following: two cements (a Type 1 Portland cement obtained from Dixie Cement Company, Knoxville, Tennessee, and a microfine grade of cement produced from finely ground blast-furnace slag which is merchandized as MC-100 by Geochemical Corporation, Ridgewood, New Jersey), two coal-fired utility fly ashes (an ASTM Class F ash from Owensboro, Kentucky, and a ASTM Class C fly ash from Rockport, Illinois), and a cement-grade Wyoming bentonite. Three blends were tested: one for each of the fly ashes mixed at 39 wt % Type I Portland cement, 55 wt % fly ash, and 5.5 wt % Wyoming bentonite and one using $100\%_w$ MC-100 cement. Blends 1 and 2 contained class C and F, respectively, and blend 3 contained the MC-100 cement. The weight of dry solids added to a given volume of liquid (the mix ratio) for grouts containing the fly ashes was 1.4 to 1.5 kg/L, and a mix ratio of 0.90 to 0.96 kg/L was used for the grouts made with the MC-100 microfine cement. The test procedures (phase separation, compressive strength, penetration resistance, apparent viscosity, gel strength, and consistency) were all performed in triplicate.

Field Grout Mixing and Injection

Grout solids were dry-blended and then mixed with water in a concrete mixing truck. Each truckload contained 5754 kg of dry blend (2245, 3200, and 319 kg of cement, fly ash, and clay, respectively) and 3780 L of water containing 454 g of delta gluconolactone (used as a dispersant and set retarder for the grout). The density of the grout was 1.6 kg/L; thus, each truckload contained ~5.9 m^3. Grout from the concrete-mixing truck was transferred to the grouting module using a grout pump. The grouting module was designed and constructed at ORNL in 1982. The module contains two 1890-L mixing tanks (each with single-propeller lighting mixers) and two progressive cavity pumps capable of pumping 113 L/min of grout at pressures up to 200 psi. One of the two pumps was used to pump grout from the grouting module via a 2.5-cm-diam high-pressure hose to a single injection well. Pressure at the grouting module was maintained manually at levels ranging from 50 to 100 psi. Grouting pressure at the injection well was usually <10 psi; however, as the available void space became full of grout, pressures up to 50 and 75 psi were observed, making it necessary to move to another injection well.

RESULTS AND DISCUSSION

Selection of Grout Formulation

Grouts made from each of the three blends met or exceeded established criteria (Table 1). Along with the criteria listed in Table 1, two additional criteria, penetration resistance and fluid consistency, were tested. If one assumes that penetration resistance readings in excess of 4000 psi reflect complete setting of cements, then all three blends set in <3 d. Results from the consistometer tests revealed that grouts made from the Class F fly ash possessed a considerably longer field time before setting began. For example, the data indicated that grouts made from Class F fly ash could be pumped for periods up to 7 to 8 h whereas grouts made of the Class C fly ash may display considerable thickening that would inhibit pumping at periods longer than 5 to 6 h. Grouts made from the microfine cement displayed substantially less consistency (bearden units <5) during the first 6 h compared with grouts made of fly ash (bearden units >10). The addition of 0.02% δ gluconolactone to the water used to make the microfine cement grouts extended pumping times from 4 to 10 h. Because of the longer pumping times of grouts made from Class F fly ash compared with the Class C fly ash, it was recommended that the grout formulation made from the Class F fly ash be used for grouting trenches 151 and 170. The grout made from the microfine cement appeared to be technically acceptable; however, there was concern that it may be difficult to develop sufficient shear to adequately mix this grout formulation in the field using a cement-mixer truck. This factor, plus the fact that material costs for the microfine cement grout were on the order of ten times higher than conventional cement/fly-ash grouts, made the choice of microfine cement less attractive.

Soil-Penetration Tests

Pregrouting soil-penetration tests within the trench area revealed that as the depth of penetration increased an increasing number of blows was required to drive the cone further into the ground. Mean blows per foot inside the trench area were significantly less than mean blows per foot into soils outside the 151 and 170 trenches (Fig. 2). Generally speaking, at soil depths >1.5 and <4.6 m, mean blows per foot within the trench averaged on the order of ten blows per foot below that outside the trench, implying considerable uncompacted soil or void space within the trench area.

In January 1991, ~6 months following grouting, soil-penetration tests were conducted within and to adjacent to trenches 151 and 170. These tests, to soil depths of 4.6 m, included 10 tests within trench 170 and 12 tests within trench 151 (4 were also conducted outside of but adjacent to the trenches). Refusal (i.e., >100 blows to penetrate one foot) was encountered in 17 of the 22 tests conducted within the trench area, resulting in mean refusal depths of 10.4 and 2.6 m in trenches 151 and 170, respectively. Results from the postgrouting soil-penetration tests revealed that the stability of the trench after grouting was significantly better than before grouting (Fig. 2), and at depths >2.4 m, the stability (measured in terms of soil penetration) was very near that observed outside the trench. For example, soil penetration in mean blows per foot at depths >2.4 m within the trench was very similar to that measured outside the trench. Pregrouting tests ranged from 10 to 20 mean blows per foot, whereas postgrouting tests ranged from 20 to 40 mean blows per foot. Only at soil depths of 1.8, 2.1, and 2.4 m were there major differences between postgrout tests and tests conducted outside the trench. The lower stability exhibited in this range (1.8 to 2.4 m) probably reflects the presence of intermediate-size soil voids (formed during backfilling) that were not penetrated by the conventional cement grout formulation.

Hydraulic Conductivity Measurements

Hydraulic conductivity within the trench area before as well as after grouting was greater than 0.1 cm/s and appreciably greater than that measured in undisturbed soil outside the trench. For example, mean hydraulic conductivity of undisturbed soil formation in "driven" wells using the above soil-penetration procedure was 1.2×10^{-5} cm/s. Previous measurements in SWSA 6 have resulted in mean hydraulic conductivities of 3.5×10^{-5} and 2.0×10^{-5} cm/s, Luxmoore et al. (5) and Davis et al. (6), respectively. Thus, these data indicate that in situ grouting with a conventional cement grout made of Portland Type 1 cement does not offer a great deal of protection against subsurface leaching of contaminants if periodical inundation by groundwater occurs.

Void Volume Measurements

As an alternative to water pump-in tests, an air-pressurization technique was used to compare accuracy and operational features of the two techniques. Preliminary pressurization tests included a series of tests to determine the extent of intervoid connection between trenches 151 and 170. For example, air was injected at one end of trench 151 to determine if pressure differences could be detected in wells at the further end of trenches 151 and 170. Other tests included air injection in a single well in trench 170 and multiple injections at six wells within the two trenches. Three flow rates were used in each test to confirm that pressure measured at the monitoring wells was a function of airflow into the trenches. The pressurization data measured at the other wells verified that a direct linkage occurred between the underground void volumes of the two trenches and that changes in flow rates in the range of 1 to 1.4 m^3/s would be adequate to detect significant changes in pressure at the well head.

To determine trench void volume using the pressurization technique, it was necessary to determine the pressure change with time as a constant flow of air was pumped into the trenches. To do this, air was pumped into six injection wells, three in each of the two trenches. Pressure was monitored at eight wells, four in each of the two trenches. The gage pressures in trenches 151 and 170 for pressurization at 1.1, 2.0, 2.8, and 3.7 m^3/min were recorded with respect to time (Fig. 3). These data were fitted using a nonlinear least-squares regression technique (7). The calculated trench void volumes determined by this procedure ranged from 175 to 228 m^3 and were all considerably higher than the 88 m^3 determined by the water pump-in tests. Air pressurization at 2.8 and 3.7 m^3/min gave very similar void volumes, 176 and 175 m^3, respectively. In both of these data sets (at 2.8- and 3.7-m^3/min flow rates), the data appeared to fit better with the regression equation than it did for 1.1- and 2.0-m^3/min tests. Also, the coefficients of variance (%) for the estimates of the regression coefficient in the 1.1- and 2.8-m^3/min pressurization tests were 23 and 14%, respectively, compared with 9.4 and 8.8% for the 2.8- and 3.7-m^3/min tests. The simple fact that the range in pressure readings at the 2.8- and 3.7-m^3/min tests (0.22 to 0.27 psi) were so much larger than the 1.1- to 2.8-m^3/min tests (<0.12 psi) made it easier to record the pressures as a function to time for the higher flow rates. Thus, the 2.8- and 3.7-m^3/min tests probably represent better estimates of the trenches' void volume than do the 1.1- and 2.8-m^3/min tests.

It is not clear why the void volume for trenches 151 and 170 determined by the pressurization technique is on the order of 100% higher than that determined by water pump-in tests. However, if one keeps in mind that both methods involve considerable experimental error and that both are subject to a variety of boundary conditions governed by the conditions of the soil and distribution of pore volumes, the agreement between the two methods may not be so poor.

Trench Grouting

Grouting of trench 170 was started on June 26, 1990, and finished on June 29, 1990. Grouting of trench 151 occurred ~1 week later, July 7 through July 12, 1990. Grout solids were dry-blended, mixed with water in a redi-mix-concrete truck, and transported to SWSA 6. Each truckload contained 5.9 m^3. A total volume of 61 m^3 of grout (37 and 24 m^3 in trenches 151 and 170, respectively) was injected. If one assumes the void volume to be 176 m^3 (as determined by air pressurization tests), 34% of the available void volume was filled with grout. (If volume of 88 m^3 as determined from the water-pump test is taken as available void volume, then 89% of the void volume was filled with grout.) Trench 170 is the larger of the two trenches, but more grout was injected into trench 151. Assuming the trenches to be 4.6 m deep and covered with 0.9 m of soil, the volumes of the trenches are 223 and 180 m^3, respectively. In this respect, ~15% of the total volume was filled with grout (20% for trench 151 and 11% for trench 170). In the 1987 grouting study (2) 31 m^3 of the same type of grout was injected into a much smaller trench (trench 150, measuring 17 x 2.8 x 3.6 m) whose volume, assuming a 0.9 m soil cover, was 116 m^3. Based on these measurements, the volume of grout injected would constitute 26% of the total trench volume. However, the final disposition of some of this grout was outside the trench because breakthrough of grout to the surface was recorded at 6 to 8 areas outside the trench. Also, the quantity of injected grout represented 109% of the measured void volume, indicating the estimated volume of grout injected to be high. Most important to recognize is that the void volume available for grouting is dependent on the waste disposed of in the trench, which is quite variable from trench to trench within SWSA 6.

Groundwater Monitoring

Sampling of groundwater was very limited because of the site's elevated position above groundwater. The only wells found to contain water were wells 165-NW and 165-SW; however, a routine monitoring program (i.e., weekly or monthly sampling) was not instigated. Water sampled (February 1991) from 165-NW and 165-SW wells outside the trench area contained higher levels of radioactivity (gross alpha, gross beta, and ^{90}Sr) after grouting than did that sampled before grouting (June 1990). The greatest difference was in the concentrations of gross beta activity, levels between 1 and 2 Bq/L before grouting compared with 7 and 10 Bq/L after grouting. Strontium-90 concentrations were also higher in water sampled after grouting (1.2 Bq/L compared with 0.6 Bq/L), but levels of gross alpha remained unchanged. If the character of groundwater were to be significantly changed as a result of grouting with a cement-based grout, levels of sodium would expected to be elevated as the bleed water or leachate from the alkali-based grout came in contact with the groundwater. However, concentrations of sodium in water sampled from these wells after grouting were similar to those found in water before grouting. Also, unlike ^{90}Sr concentrations, the levels of Ca, Mg, and Sr were similar in water sampled before vs after grouting. The major difference in water quality before vs after grouting appears to be in levels of Al, Fe, Ni, and Zn. Concentrations of all of these metals were elevated in water sampled after grouting. Elevated concentrations of these metals are probably the result of increased reducing conditions in the groundwater during February (cool and wet) compared with conditions in the groundwater during June rather than any influence of grouting per se.

SUMMARY AND CONCLUSIONS

Trench stabilization using a cement-based grout is being evaluated for use in the closure of one of the low-level radwaste burial grounds at ORNL. To demonstrate the applicability and effectiveness of this technology, two interconnecting trenches in SWSA 6 were selected as candidates for in situ grouting with a particulate grout. The trenches are ~4.5 m deep and measure 11.5 by 4.3 m and 14.3 by 4.3 m. The primary objective was to demonstrate the increased trench stability (characterized by soil penetration tests) and the decreased potential for leachate migration (characterized by hydraulic conductivity tests) following in situ injection of a particulate grout into the waste trenches. Stability against trench subsidence is a critical issue. For example, construction of impermeable covers to seal the trenches will be ineffectual unless subsequent trench subsidence is permanently suspended.

A grout composed of 39% Type 1 Portland cement, 55.5% Class F fly ash, and 5.5% bentonite mixed at 1.5 kg/L of water was selected after laboratory testing of several grout formulations. Results of laboratory studies revealed that this formulation exhibited no liquid-to-solid phase separation and a compressive strength of >900 psi after 28 d. Penetration resistance after 2 d was >8000 psi. Most important, the viscosity of the freshly made grout was <50 cP, allowing it to penetrate and fill intermediate-size trench voids, and the suspension was stable for pumping and injection for up to 8 h.

Before the trenches were grouted, the primary characteristics relating to physical stability, hydraulic conductivity, and void volume of the trenches were determined. Their physical stability was evaluated using soil-penetration tests. Pregrout soil-penetration tests revealed that at depths >1.5 and <4.6 m mean blows per foot within the trench averaged approximately ten blows per foot fewer than that outside the trench. These data imply that considerable uncompacted soil and/or void space exists within the trench area which over time will lead to significant subsidence of the upper soil layers. Hydraulic conductivity tests within the trench area also revealed a high potential for the infiltration of water (hydraulic conductivities >0.1 cm/s).

Void volume within the trenches was determined by two techniques: (1) water-pump tests and (2) a newly developed air pressurization technique. Estimates of void volume using the air pressurization technique were ~100% higher than that determined by the water-pump test (176 m^3 compared with 88 m^3). It is not clear which of the two techniques is the most accurate because each has theoretical and operational constraints. The development of the air pressurization technique was pursued because the water-pump test could result in serious leaching of radionuclides and/or hazardous wastes into groundwater and may induce premature settling of soil overburden into the trench. The water-pump test also leaves soil within the trench near saturation levels for a significant time, which could have an influence on solidification of grout introduced into the trench. Consequently, the flooding of trenches with water is no longer an acceptable practice for determining trench void volume. The air pressurization method is fast and can be repeated under a variety of pressures and air flows. It is environmentally superior to the water-pump test in that it does not promote the leaching of contaminants to groundwater. The major concern in the use of the air-pressurization technique is that its accuracy has not been validated. More research is needed to demonstrate its accuracy and precision, as well as its theoretical and operating limitations under controlled conditions.

Soil-penetration tests before grouting revealed considerable uncompacted soil or void space within the trench area. For example, at depths >1.5 and <4.6 m, mean blows per foot within the trench averaged on the order of ten blows per foot fewer than that outside the trench. After grouting, soil-penetration tests disclosed that stability had been improved greatly. For example, refusal (defined as >100 blows to penetrate 1 ft) was encountered in 17 of the 22 tests conducted within the trench area. Mean refusal depths for trenches 151 and 170 were 3.2 and 2.6 m, respectively. The postgrouting soil-penetration tests revealed that the stability of the trench after grouting was significantly better than before grouting, and at depths >2.4 m, the stability (as measured in terms of soil penetration) was very near that observed in the native soil formation outside the trench. The major differences in results between postgrouting tests and tests conducted in native soil formations outside the trench were found at soil depths of 1.8, 2.1, and 2.4 m. Tests within the trench showed lower stability within this range (1.8 to 2.4 m), probably because of the presence of intermediate-size soil voids (formed during backfilling) that were too small to be penetrated and filled by the conventional cement grout formulation. Hydraulic conductivity within the trench remained very high (>0.1 cm/s) and significantly greater than that outside the trench. Postgrouting air pressurization tests also revealed a large degree of intervoid linkage within and between the two trenches. Even after grouting, the combined void volume of the two trenches by this test procedure was 61 m^3 (compared with 176 m^3 for the ungrouted trenches). Only 60 m^3 of grout were injected, leaving 55 m^3 unaccounted for [i.e., total void volume measured (176 m^3) minus the sum of the grout injected (60 m^3) plus the void volume measured after grouting (61 m^3)]. Thus, it appears that to effectively reduce hydraulic conductivity and to develop stability within the upper level of the trench, it may be necessary to implement additional stabilization steps. One possibility is the injection into the upper level of the grouted trench a chemical or microfine cement grout which may fill voids too small for the conventional grout.

FUTURE NEEDS

Grouting of trenches with conventional particulate grouts typically fills only 30 to 60% of the available void volume, allowing the trenches to remain vulnerable to a certain degree of leaching. On the other hand, chemical grouts typically fill all of the void volume but generally tend to be considerably more costly. Certain chemical grouting materials (e.g., acrylamide) are also toxic and present a risk to those who prepare and inject it and are potential contaminants to drinking water supplies. For these reasons, considerable testing is required to demonstrate that the chemical's use will not present unacceptable risks. Recently, microfine-size cements have become readily available. These cements, most often made from finely ground blast-furnace slag, can be used to form slurries capable of attaining permeabilities on the order of 10^{-5} to 10^{-9} cm/s when injected into fine sands. These highly permeable microfine grout formulations are attractive from the standpoint of being superior to conventional Portland cement grouts in providing hydrologic isolation. However, their costs are approximately ten times that of conventional Portland cements, and in certain instances, their costs are similar to chemical grouts.

Recent experience with microfine cement indicated that low ratios (1:1) of water to cement would have to be used unless special equipment were purchased to suspend suspensions containing high (3:1) water:cement ratios. For example, formulations based on high (3:1) water:cement ratios would require high-energy mixers (those that generate sufficient shear to fully disperse the microfine cement) to avoid significant liquid/solid phase separation. An alternative grout formulation was investigated by using blends of bentonite clay and microfine cement with varying ratios of water. One formulation that was found to be stable and pumpable through a 30-cm column of coarse sand was made using a 8:2:1 ratio of water, bentonite, and microfine cement. The formulation was much less expensive than chemical grouts or microfine cement grouts formulated on a 1:1 basis. Because of its lower permeability, such a formulation should offer significantly better hydrologic isolation of waste trenches than conventional particulate grouts. One approach is to use a combination of conventional particulate grout to fill the large void spaces often present at the bottom of the trench and a microfine grout to fill the intermediate and small void spaces in the upper levels of the trench. This would be economical and probably feasible technically. To demonstrate the utility of this approach, it is proposed that a clay/microfine cement grout be injected into the upper sections of trenches 151 and 170 via the remaining slotted injection wells. After injection of the clay/microfine cement grout, additional soil-penetration tests will also be conducted to examine if the stability in the 1.8- to 2.4-m depth range has improved as a consequence of grouting with the clay/microfine cement grout. Hydraulic conductivity tests will also be conducted to evaluate the performance of the clay/microfine cement grout in reducing leaching of radionuclides and other potential contaminants into groundwater.

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DISCLAIMER

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Table 1. Results of laboratory grout studies

Parameter	Criteria	Blend[a]		
		1	2	3
Apparent viscosity	<50 cP	35±1	37±1	10±1
10-min gel strength	<5 kg_f/m^2	1.5±0.1	1.4±0.3	0.35±0.05
28-d phase separation	0 vol%	0	0	0
28-d compressive strength	>60 psi	1601±17	908±28	2000±253

[a]Blend 1 contained 39% Type 1 Portland cement, 39% Class C fly ash, and 5.5% bentonite mixed at 1.5 kg/L of water. Blend 2 contained 39% Type 1 Portland cement, 39% Class F fly ash, and 5.5% bentonite mixed at 1.5 kg/L of water. Blend 3 contained 100% MC-100 microfine cement mixed at 0.9 kg/L of water.

FIGURE LEGENDS

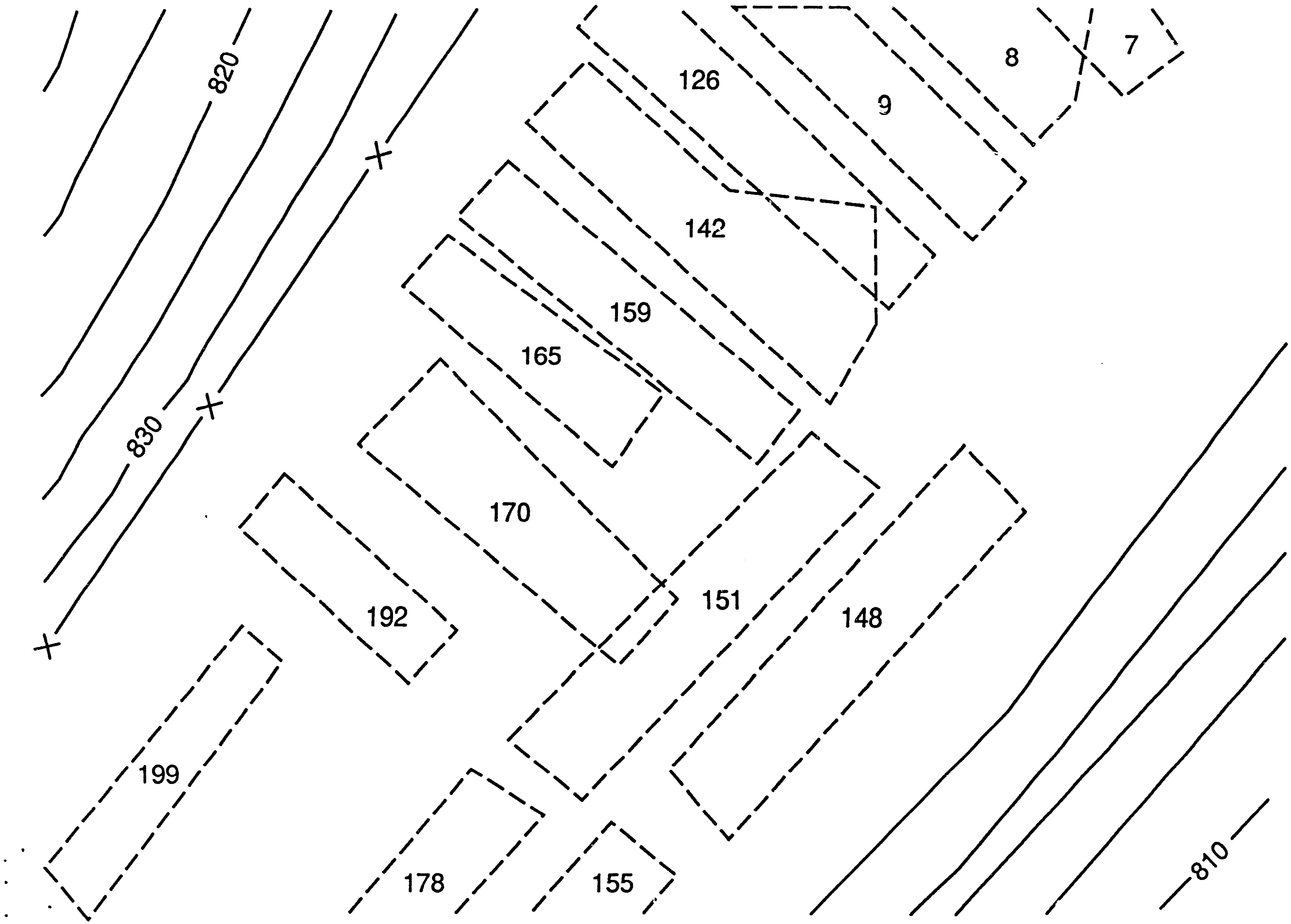
Figure 1. Trench 151 and 170 boundaries.

Figure 2. Summary of soil penetration tests.

Figure 3. Pressurization tests on trenches 151 and 170 at 3.7 m^3/m.

Fig. 1

ORNL DWG 91M-7203



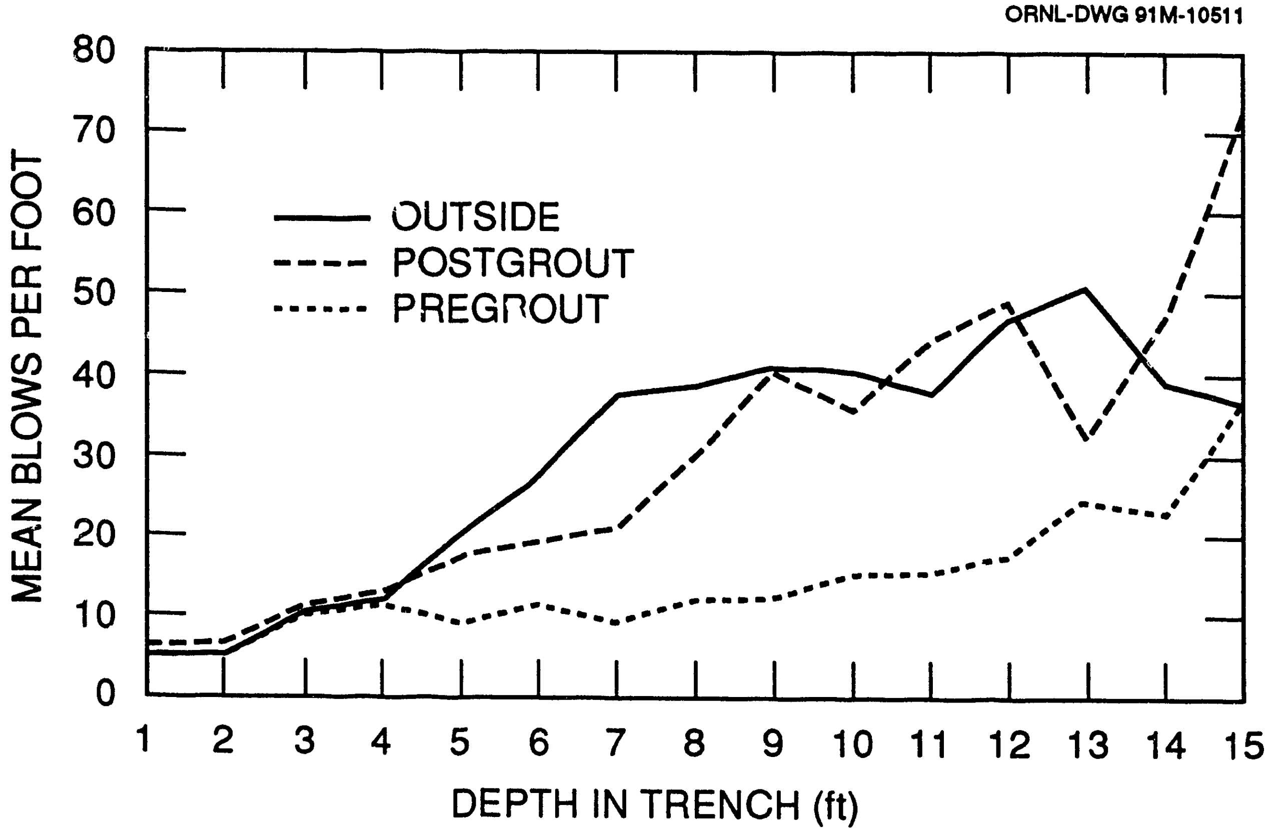

ORNL-DWG 91M-10511
MEAN BLOWS PER FOOT
0
10
20
30
40
50
60
70
80
OUTSIDE
POSTGROUT
PREGROUT
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
DEPTH IN TRENCH (ft)

Fig. 2

ORNL-DWG 91M-10510

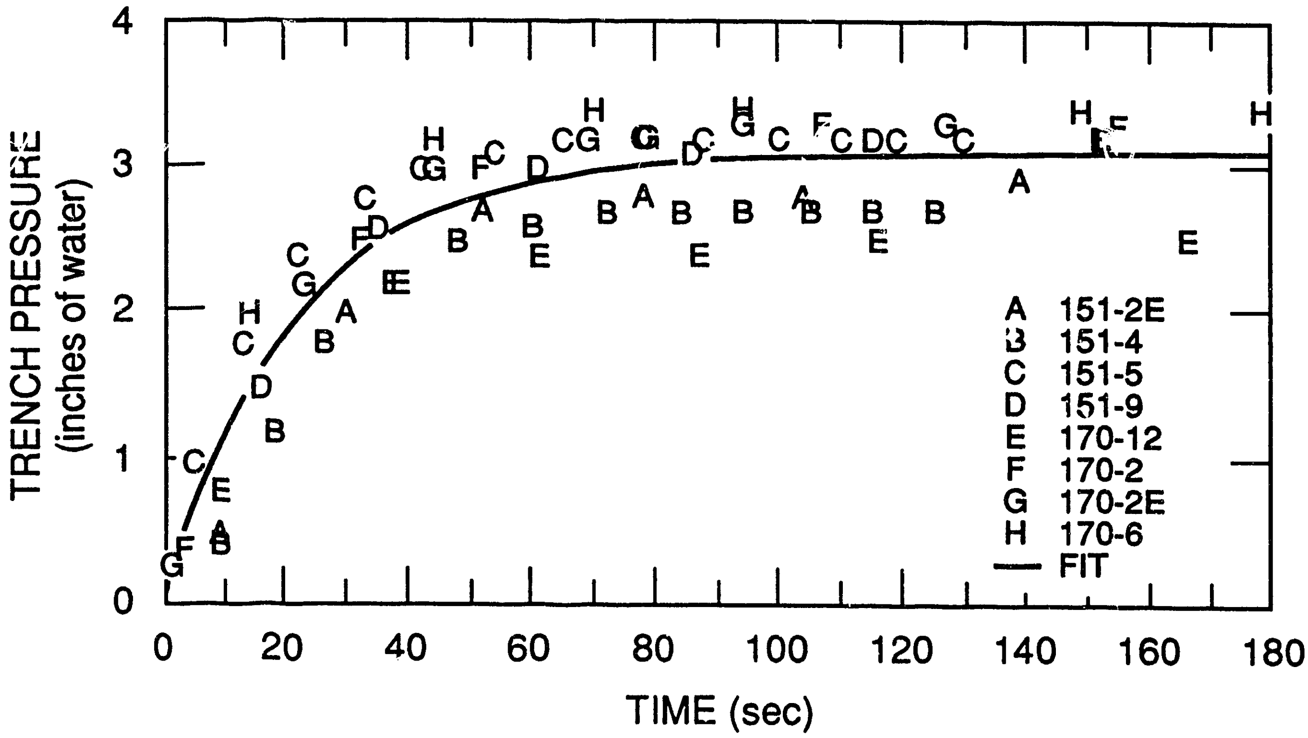


Fig. 3

END

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