TABLE 4.3.1-1. (Cont'd)

Group	Type	Composition
Clay Grouts	Bentonite Suspension	water, bentonite (10 percent)
	Bentonite-silicate	water, bentonite, sodium silicate, sodium phosphate
	Bentonite-diesel	bentonite, diesel oil, water
Silicate Grouts	Joosten	sodium silicate solution, calcium chloride solution
	Guttman	sodium silicate, calcium chloride, sodium carbonate solutions
	Silicate-Bicarbonate	sodium silicate, sodium bicarbonate solutions
	Silicate-Ethylacetate	sodium silicate solution, ethylacetate
	Silicate-Aluminate	sodium silicate solution + sodium aluminate
Organic Polymers	Epoxy Resin	non-aqueous resin
	Polyester Resin	non-aqueous resin
	Chrome-lignin	calcium lignosulphonate and sodium dichromate
	Urea-Formaldehyde	urea and formaldehyde in acid solution
	Polythixon	polyurethane
	Resorcinol-Formaldehyde	resorcinol and formaldehyde in aqueous solution

4.3.1.3 Design Considerations for Particulate (Cement and Clay) Grouts

The physical properties of the host geologic medium play a major role in the design and construction of cement grout cut-offs. Houlsby (1982a) lists eight effects of geology on cement grouting:

1. Spacing of Open Joints. Widely spaced joints make grouting easier while close spacing can lead to surface leaks and patchy grouting.

2. Size of Open Joints. Joints wider than 2 mm assist grout penetration. Joints wider than 6 mm inhibit proper tightening to grout refusal. Conversely, joints tighter than 0.5 mm make penetration difficult.

3. Direction of Open Joints. An average dip between 30° and 60° is easiest to intercept by vertical grout holes and is less likely to permit rock movement than a more vertical or horizontal dip.

4. Rock Strength. Rock should be massive, tough and well-anchored to bedrock. Weak rock or weakly imbedded slabs may tend to shift under grouting pressure.

5. Rock Soundness. Rock soundness is important in keeping grout injection drill holes from collapsing.

6. Tectonic Stress. Strain energy release resulting from tectonic stress can cause open joints between the detached rock and bedrock on the order of several centimeters.

7. Uniformity. Irregular host material (i.e., jointing, variable rock types, intrusions, faults, etc.) can greatly complicate the grout layout procedure. Weak fractures may require intensive localized grouting. The more uniform the geologic medium the easier it is to layout the grout holes.

8. Proneness to Piping. The seepage removal of material in joints is referred to as piping. If piping is possible more intensive grouting may be required than would otherwise be necessary.

The constituent materials used in cement-based grouts are: 1) water, 2) cement, and 3) various fillers primarily used to lower the overall grout cost without significantly effecting the flow properties and strength. The principal variable which effects the properties of cement grouts is the water/cement ratio (ω) by weight. Excessive water increases bleeding, causes shrinkage, decreases durability and lowers the grout strength (Littlejohn 1982). Fillers consist mainly of clays, pozzolans, fine sands, and other admixtures. Admixtures are materials other than water, aggregates, or cementitious materials, used as a grout ingredient for cement-based grouts.

Clay/cement grouts have an ability to form gel structures due to the absorptive capacity of the clay (usually sodium montmorillonite). Sodium montmorillonite is generally referred to as bentonite. The setting time for bentonite/cement grouts is not well-defined and strength development is slow (Littlejohn 1982). Normally within 24 hours the clay/cement grout sets up to a strength of soft to firm clay (Harris et al. 1982a). In clay/cement grouts where high proportions of clay are used (e.g., 50 percent clay content) the clay filler increases the volume yield per unit weight of material thus reducing the cost in relation to low clay/cement ratio grouts with lower volume yields per unit weight.

Pozzolans such as naturally occurring finely ground shale, pumicite, and diatomite or artificially produced flyash and ground blast furnace slag are not cementitious but react with free lime cement (in the presence of water) to form a cementitious compound (Littlejohn 1982). Pozzolans are primarily used as cheap bulk fillers for large cavity grouting where strength may not be of great concern.

Sand fillers are used in grouts requiring high frictional shear strength. These grouts typically have a low water content. Sand/cement ratios are usually limited to a maximum of three parts sand to one part cement to maintain particle suspension (Littlejohn 1982). Occasionally admixtures are combined with grouts to alter their flow, or set properties. They should not be used indiscriminately however, and are not a substitute for good grout practices (Littlejohn 1982).

4.3.1.4 Design Considerations for Non-Particulate Grouts

Non-particulate grouts (i.e., chemical grouts) normally consist of solutions of two or more chemicals which react to form a gel. The reaction causes a decrease in fluidity and facilitates solidification and subsequent formation of occulusions in fill voids of the host material (U.S. Army Office of the Chief of Engineers 1973). The viscosities of chemical grouts tend to be very low and generally (except for fillers that are sometimes added) contain no particulate matter. Chemical grouts can therefore be injected into materials with voids small enough to limit penetration of cement-based grouts. Chemical grouts have been used primarily in fine granular material and to seal fine fissures in fractured rock (U.S. Army Office of the Chief of Engineers 1973).

Most chemical grouts belong to one of the following four groups:

1. Sodium silicate grouts
2. Acrylamide grouts
3. Lignin grouts
4. Epoxy and polyester resins grouts

Each grout exhibits certain characteristics that make it suitable for certain applications. For several chemical grouts the speed of the chemical or physicochemical reaction limits the radius of grout penetration. Other factors affecting grout penetration include: 1) concentrations of constituent chemicals, 2) permeability of the material being grouted, 3) grouting pressure, and 4) continuity of injection technique (U.S. Army Office of the Chief of Engineers 1973).

Sodium Silicate Grouts

Sodium silicate is the chemical basis of a variety of silicate grouting processes. Sodium silicate forms a gel in the presence of specific reactants. The gel fills voids and binds particles together when injected into granular material. Several reactants can be used and the choice is based on desired gel time, strength and permanence requirements, and cost.

The chemical reaction occurs when sodium silicate (an alkaline) is mixed with an acidic material. A gel is formed if the silica concentration in the silicate solution is greater than one or two percent by volume. Acidic mixers commonly used are: chlorine, ammonium salts, bisulfates, bicarbonates, sulfur dioxide, and sodium silicofluoride (U.S. Army Office of the Chief of Engineers 1973).

Sodium silicate is injected in either a two-solution process or as a single solution. The two-solution (termed "two-shot method") process consists of the injection of a solution of sodium silicate followed by a second separate injection of the reactant chemical(s). The reaction between the silicate and the reactant solution is almost instantaneous thus allowing the sealing of water bearing strata with moderate ground-water velocities and pressure heads. Disadvantages of the two solution process include limited grout radii due to the speed of the reaction and uncontrolled mixing of the solutions in the host material (U.S. Army Office of the Chief of Engineers 1973).

The one solution process consists of injection of the sodium silicate with the reactant in a single solution. Prior to mixing, the reactant(s) are diluted with water and introduced into the aggitated sodium silicate solution. The one-solution process allows more complete grout penetration and better control of the grout radius. However, sodium silicate grouts placed with the two-shot technique tend to have greater strength (U.S. Army Office of the Chief of Engineers 1973).

Acrylamide Grouts

The most widely used acrylamide grout has been composed of acrylamide and methylene bisacrylamide mixed in proportions the produce stiff gels from dilute water solutions (U.S. Army Office of the Chief of Engineers 1973). Gel time can be controlled within the range of a few seconds to several hours by varying the proportions of the constituent materials. The viscosity of acrylamide grouts approaches that of water and they maintain a low viscosity for roughly 95 percent of their fluid life. If allowed to dry the acrylamide gel will lose water and shrink. However, if allowed to continue drying the gel will slowly re-swell to its original volume. Excessive drying will destroy the gel (U.S. Army Office of the Chief of Engineers 1973).

Acrylamide grouts have been used to construct grout curtains and to grout jointed and fissured rock to control water seepage. The principal use has been to stop saturated and partially saturated ground-water flow. Acrylamide grouts can penetrate materials with a grain size of 0.01 mm (silt size range) and have been used in fissured rock with fissures up to 10 to 15 cm in width (U.S. Army Office of the Chief of Engineers 1973).

Prior to early 1978 three acrylamide grouts and one acrylamide-based grout were commercially available in the U.S. However, with the recognition of acrylamide as a neurotoxin and subsequent cases of acrylamide poisoning, U.S. manufacturing of AM-9® (trade name of acrylamide) was discontinued. The Japanese and the French marketed acrylamide-based grouts in the U.S. in 1979 but they also withdrew from the U.S. market. Terragel®, Q-Seal®, and PWG® are all distributor trade names for AM-9® and are no longer marketed in the U.S. (Karol 1982a).

Lignin Grouts

Lignin is a by-product of the paper making sulfite process that forms an insoluble gel when combined with a chromium compound. Viscosities of various lignin solutions vary over a wide range making lignin grouts suitable for injection into voids of fine sand to coarse silts (U.S. Army Office of the Chief of Engineers 1973).

Lignin grouts consist of materials that are rapidly soluble in water. The gel in normal grout concentrations (i.e., the weight ratio of water to lignosulfonate of 4:1 to 5:1) has a rubbery consistency and is practically impermeable to water. If protected against drying and freezing the grout ordinarily does not deteriorate (U.S. Army Office of the Chief of Engineers 1973).

Lignin grouts are used primarily in fine granular material. They have also been successfully used to grout fine fissures in fractured rock. The U.S Army Office of the Chief of Engineers (1973) does not recommend lignin grouts for use in soils containing an appreciable amount of material finer than the No. 200 (0.0029 in.) sieve. A dilute solution of lignosulfonate can be used to grout fine, nonargillaceous sands for permeability reduction.

Epoxy and Polyester Resin Grouts

Epoxy and polyester resins are organic compounds comprising two-component systems made of a resin base and a hardener. Epoxy resins are resistant to acids, alkalies, and other organic chemicals and they cure without volatile by-products thus preventing formation of bubbles or voids. Epoxy resins are also compatible with various thickening agents (e.g., bentonite). Epoxy resins are thermosetting (i.e., they will not liquify once they have hardened even when heated) (U.S Army Office of the Chief of Engineers 1973). This property may be important for applications in close proximity to a core melt accident where ground-water temperatures may be significantly higher than surrounding ambient temperatures.

Polyester resins are also two-component systems that have been used to stabilize or strengthen fractured rock. Polyester resins are low viscosity, thermosetting liquid plastics that chemically cure to a solid. Polyester resins do not bond as well to moist rock as due epoxy resins. Also, they are more brittle and exibit greater shrinkage than epoxy resins (U.S. Army Office of the Chief of Engineers 1973).

The four categories of non-particulate grouts discussed above are not all-encompassing of chemical grouts, although they encompass the most frequently used non-particulate grouts. Other chemical grouts include (U.S. Army Office of the Chief of Engineers 1973):

1. Cationic organic - emulsions utilizing diesel oil as a carrier
2. Resorcinol - formaldehyde
3. Epoxy - bitumen
4. Calcium acrylate
5. Aniline - furfural
6. Aluminum octoate compounds
7. Urea - formaldehyde
8. Polyphenolic polymers

These additional grouting compounds and systems are all classified as Newtonian low viscosity grouts. A complete listing of chemical grouts and their trade name and manufacturer is included in Table 4.3.1-2.

TABLE 4.3.1-2. Chemical Grouts and Manufacturers (Source: U.S. Army Office of the Chief of Engineers 1973)

Type	Trade Name	Manufacturer, Producer, or Distributor
Acrylate	AC-400[a]	--
Acrylamide Pre-polymer	Injectite 80[a]	--
Resin	Cyanaloc	Cynamid International
Resin	Herculox	Halliburton Oil Well Cementing Co.
Silicate	Injectrol-G	Halliburton Oil Well Cementing Co.
Silicate	Siroc	Raymond International Inc.
Silicate	Geloc-4[b]	--
Silicate	Terraset[b]	--
Silicate	Hardener 600[b]	--
Lignin	Blox-All	Halliburton Oil Well Cementing Co.
Lignosulfonate	Terra Firma	Concrete Chemicals Co.
Epoxy resin	--	George W. Whitesides Co.
Polyester resin	--	Cyanamid International
Polyphenolic polymer	Terranier	Rayonier, Inc.
Resorcinol-formaldehyde	CR-726	Catalin Corp. of America
Phenoplast or resorcin-formal	--	Soletanche
Aluminum octoate	Firmgel	Byron Jackson, Inc.
Cationic organic-emulsion	SS-13	Brown Mud Co.
Aminoplasts or urea-formols	--	--
Epoxy-bitumen	--	--
Calcium acrylate	--	--
Aniline-furfural	--	--
Polyurethane	TACSS	--
Polyurethane	CR250*	--

(a) Identified by Karol 1982a.
(b) Identified by Baker 1982.

4.3.1.5 Choice of Grouts

The choice of grout is made so as to allow effective penetration of the host material and provide the necessary reduction in permeability with acceptable duration. The effect of ground water on the various grouts also influences the choice of the grout. Ground water can dissolve some soluble elements of a grout and can cause certain chemical and physicochemical changes in the grouts. According to Caron (1982) these two effects may result in a reduction of the imperviousness of a grout that is variable among the differing types of grouts.

Soil penetration by a grout is primarily by impregnation and occasionally by fracturing-impregnation for most applications to control ground water and contaminant movement via grouted cut-off walls. Impregnation grouting (i.e., permeation grouting) requires grouts that are adopted to the size of the voids of the host material in order to penetrate the soil voids. Thus a wide range of grouts, both particulate and non-particulate, are potentially applicable (Caron 1982).

In most cases cement-based grouts are suitable for fissured coherent soils. In granular soils there is a filtration of cement-based grouts as grainsize decreases. Chemical grouts may be more suitable because of lower viscosities and lack of particulate matter as the host material becomes more fine. The criteria of grainsize distribution (d_{10}), permeability (K), and specific surface (S) can be used to recommend types of grouts appropriate to each type of granular soil (Caron 1982). Table 4.3.1-3 lists three soil types and the recommended grout.

Caron (1982) suggests that the limit between chemical grouts and cement-based grouts is fairly well-defined. When cement-based grouts can only proceed by fracturing because of the fineness of the host material, chemical grouts become more suitable. The limit between gels and resins, however, is not as well-defined because the only significant difference in preset properties is viscosity.

The permeation characteristics of both particulate grouts and chemical grouts are limited by increasing shear resistance of the interface between the grout and the host soil (Attewell and Farmer 1976). Figure 4.3.1-1 shows the soil size limitations on grout permeation.

TABLE 4.3.1-3. Grout Recommendations Based on Soil Type (Source: Caron 1982)

Soil Type	Grain Size Distribution (mm)	Permeability (cm/sec)	Specific Surface (1/cm)	Recommended Grout
1. Coarse-grained	$d_{10} > 0.05$	$K > 10^{-1}$	$S > 1/10$	Cement and clay/element
2. Medium-grained	$0.02 < d_{10} < 0.05$	$10^{-3} < K < 10^{-1}$	1/1000 < S 1/1000	Sodium silicate, lignochrome gels, colloidal solution, and prepolymer grouts.
3. Fine-grained	$d_{10} < 0.02$ mm	$K < 10^{-3}$	$S < 1/1000$	Acrylamide-based grouts and other pore solution grouts

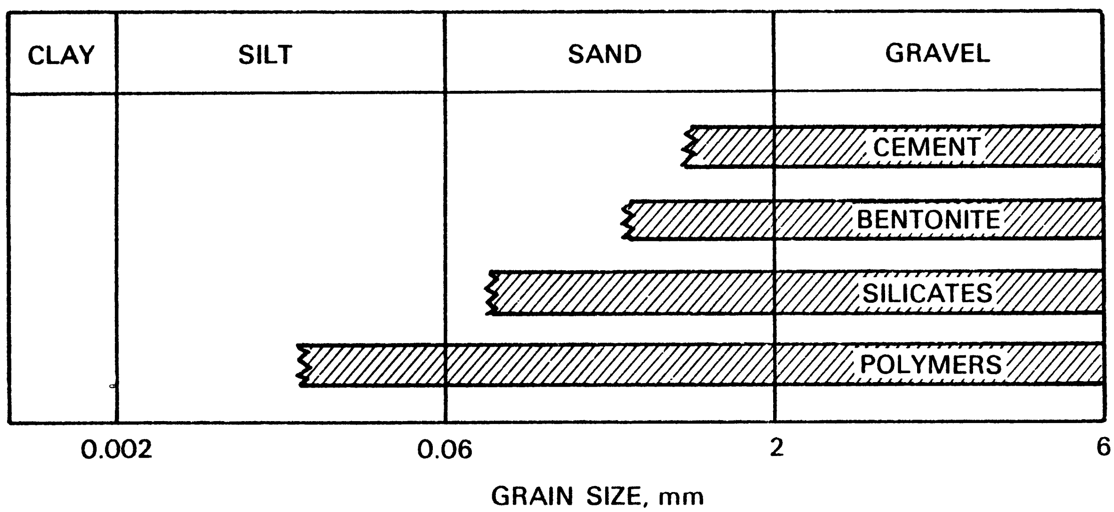

FIGURE 4.3.1-1. Permeation of Various Grouts in Relation to Soil Grain Size (Source: Attewell and Farmer 1976).

Baker (1982) states that the initial permeability of the host soil material is the overall key to determining the groutability of the soil mass. Host materials with permeabilities ranging from 10^{-1} cm/sec to 10^{-3} cm/sec are "easily groutable". Permeabilities in the range 10^{-3} cm/sec to 10^{-4} cm/sec are "moderately groutable", while material permeabilities between 10^{-4} cm/sec and 10^{-5} cm/sec are marginal with respect to practical ability. Host materials with permeabilities less than 10^{-5} cm/sec are considered ungroutable.

4.3.1.6 Grout Curtain Construction Considerations

The process of constructing a grout curtain to function as a barrier to ground-water flow involves several steps. These steps include:

1. Geohydrologic investigations,
2. Layout,
3. Drilling grout holes,
4. Grout mixing, and
5. Grout injection.

Site investigations for grouting may involve geological or geotechnical methods normally used for any geohydrological site characterization. These studies should be conducted sufficiently to avoid any unsuspected major surprises as to host material properties, ground-water flow characteristics, etc. When feasible, test grouting is recommended to determine rates of grout takes, suitable pumping pressures, and estimates of the volume of grout that may be required for a particular grouting operation (Albritton 1982). Normally, volumetric grout requirements are estimated from the porosity of the host material. Typical groutable material has an effective porosity between 0.25 percent and 0.45 percent (Baker 1982).

Layout

Penetration grout curtains are constructed in a series of primary, secondary, tertiary, etc. grout applications. This construction process, termed "split-spaced injection staging", refers to multiple grouting episodes in the same zone or area. Primary grouting is the initial grouting of a previously ungrouted area. Individual grout cylinders or "bulbs" are not in contact or overlap only slightly. Secondary and tertiary grouting successively fills the ungrouted areas remaining in the same zone. Grout pipe spacing is designed to locate primary and secondary grout injection points. The secondary locations are usually at the midpoint between primary injection points (Baker 1982).

Project costs are highly sensitive to grout pipe spacing. Pipes spaced too close to each other (i.e., less than 0.5 m) will result in excessive costs for drilling. Pipes spaced too far apart (i.e., more than 2.5 m) will result in long pumping times and loss of control of the grouting process due to uncertainty about the location of the grout front. Most grouting operations have a pipe spacing between 0.8 m and 1.5 m (Baker 1982). The U.S. Army Corps of Engineers suggests that the proper spacing for the "split-spaced method" primary grout holes should be rarely less than 3.0 m (Albritton 1982).

There are four basic stage grouting methods (Houlsby 1982a):

1. Downstage without packer,
2. Downstage with packer,
3. Upstage, and
4. Circuit grouting downstage.

For high standard grouting (which would be required to assure control of radionuclide migration following a severe power plant accident) Houlsby (1982a) recommends downstage grouting without packer (Figure 4.3.1-2). The steps involved in downstage grouting require repeated drilling/grouting operations at successively greater depths in the same grout hole. The advantages of this procedure are proving of upper grouted stages and automatic handling of material weaknesses as they exist (Houlsby 1982a).

Grout Hole Drilling

Minimum diameters as small as 38 mm have been successfully used for grout hole specifications. However, for deep or inclined grout holes larger diameters are recommended because the stiffness of the drill rods will result in straighter boring. Also, distribution of grouting pressures in the host medium is affected by the diameter of the boreholes. Smaller holes require greater pressures to achieve the same relative results compared to larger holes with less pressure (Albritton 1982).

Both rotary drilling and percussion drilling with water are acceptable means for grout hole boring. Caution must be exercised to insure that no premature plugging of fine fissures by dry rock flour, drilling mud, or clay

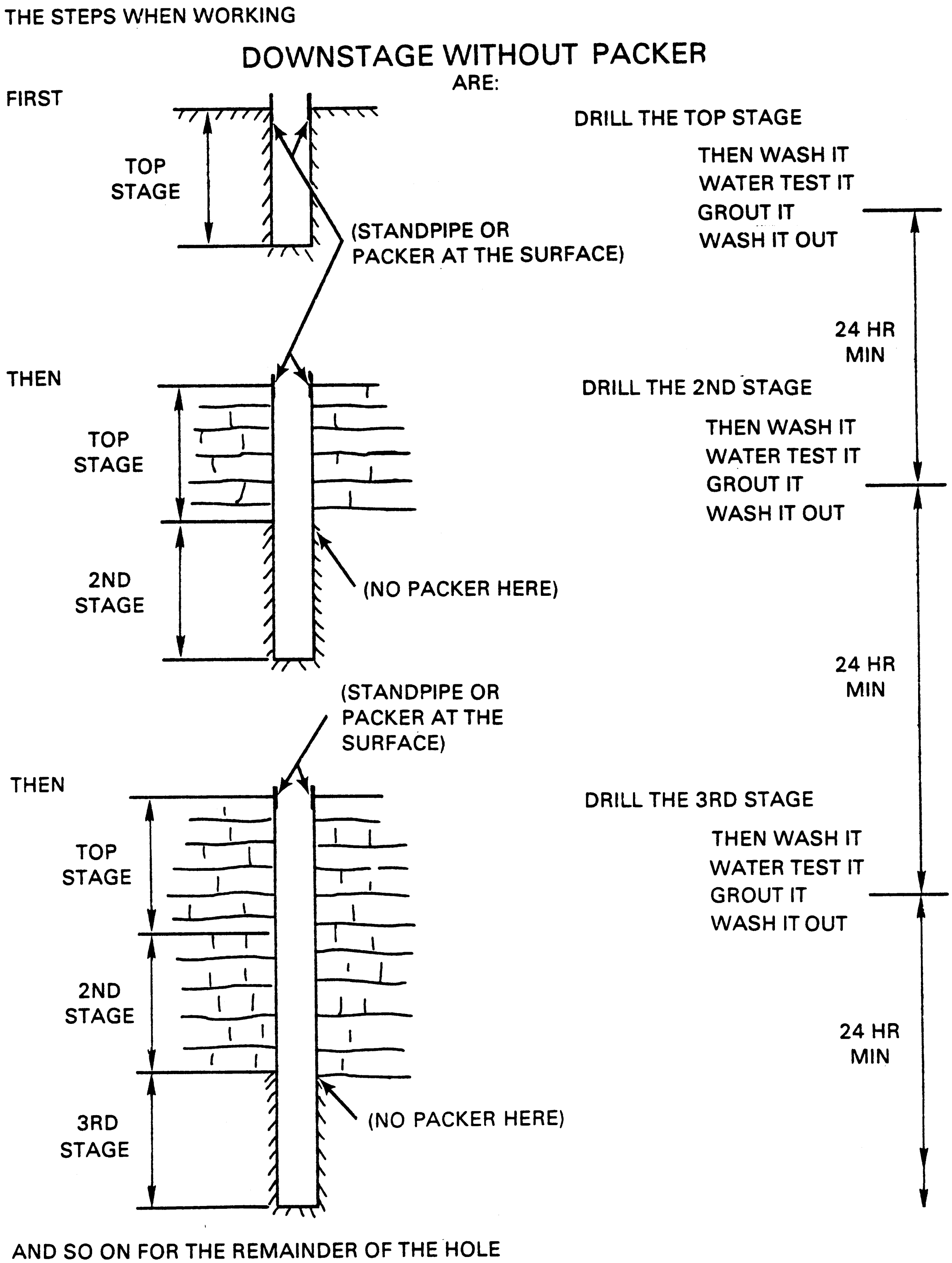

FIGURE 4.3.1-2. Diagram of Downstage Grouting without Packer (Source: Houlsby 1982a)

slurry occurs. The U.S. Army Corps of Engineers normally requires water, as opposed to air, for a drilling fluid. However, water may occasionally cause caving, erosion, and/or bit binding. Grout holes should be flushed of all drill cuttings and turbidity prior to grout injection (Albritton 1982).

Grout Mixing

For penetration grouting, high speed mixing of cement grouts is essential. Cement grouts mixed at high speeds usually penetrate well due to the absence of conglomerations of cement grains. High speed mixing facilitates complete and thorough wetting of cement grains thus allowing thorough hydration. High speed mixers operate at speeds in the range 1500 to 2000 revolutions per minute. High speed mixing may require as little as 15 seconds per mixing cycle enabling rapid feed to a "fast" hole (Houlsby 1982a).

The U.S. Army Corps of Engineers indicates that starting with a thin water/cement ratio mix (i.e., 5:1 or 6:1) may be preferable to a thicker starting grout, even if eventually a thick grout is required. By starting with a thin grout fine fractures may be successfully grouted that otherwise would have been plugged by a thicker grout (Albritton 1982). However, sedimentation of cement grains increases with increasing water/cement ratios. A 6:1 water/cement mix may experience up to 60 percent sedimentation in two hours. Sedimentation may be lowered by adding bentonite in small amounts (Deere 1982).

Mixing of chemical-based grouts is specific to and dependent on the grout employed. For some two-shot processes mixing (of multiple constituents) is not necessary; mixing essentially takes place in the grouted medium.

Grout Injection

Penetration grouting requires a moderate injection pressure that does not cause excessive disturbance of the host medium. Normal penetration grout pressures do not exceed 0.4 bar per meter of depth. Allowable pressures increase as the depth of stage increases with rule-of-thumb injection pressures of 0.23 bars per meter of depth for average to weak host materials. The rule-of-thumb injection pressure can be doubled for sound material. Regardless of the sustained injection pressure the build-up of pressure should be gradual (Houlsby 1982a).

Grout injection should continue until absolute grout take refusal. Once refusal has been reached it is advised to hold the pressure for approximately 15 minutes. For grouting wide cracks (i.e., 0.3 cm to 0.6 cm) second injections after a one to two day delay may be advisable (Houlsby 1982a).

4.3.1.7 Grout Performance and Durability

The two most important issues related to the suitability of grout curtains as barriers to ground-water contaminant migration resulting from a severe power plant accident are the long-term permeability of the grout barrier and the durability of the barrier. Quality control should be maintained throughout the grouting operations. Variations (i.e., non-uniformity of final grout curtain properties) result from three causes (Littlejohn 1982):

1. Inadequate/improper mixing of grout,
2. Variations in grout material (both quality and quantity), and
3. Apparent variations from the testing procedure.

The assurance of acceptable quality requires rigid engineering supervision of all grouting operations.

Grout Curtain Permeability

For cement-based grouts the cured permeability is a function of the original water/cement ratio (ω). For fresh or aging cement grouts the permeability is related to the age of the grout. Figure 4.3.1-3 shows the relationship between water/cement ratios and 28 day permeability for a typical cement-based grout. Table 4.3.1-4 shows the permeability increase with age for a Type I (ordinary porland cement) grout.

Chemically grouted cut-off walls can achieve the same relative permeability reduction as cement-based grout curtains. Permeability testing in fine, medium, and coarse grained sands indicate that permeabilities as low as 5×10^{-9} cm/sec can be achieved with acrylate grouts such as AC-400® (Clarke 1982). The U.S. Army Corps of Engineers states that sands with ungrouted permeabilities in the range of 1 cm/sec to 1×10^{-3} cm/sec were grouted with a 10% acrylamide solution to permeabilities of 2×10^{-10} cm/sec (U.S. Army Office of the Chief of Engineers 1973).

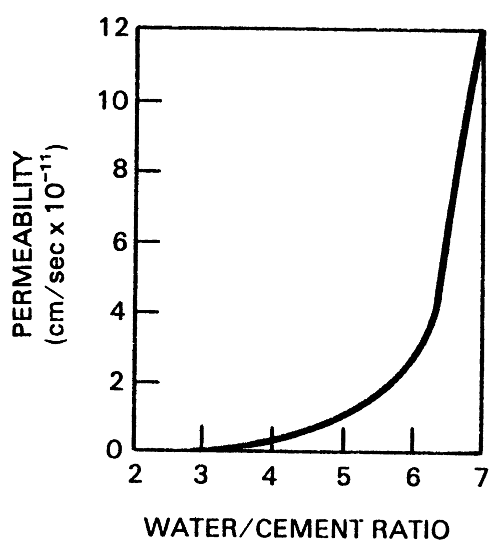

FIGURE 4.3.1-3. Twenty-Eight Day Permeability of a Typical Cement Grout (Source: Littlejohn 1982)

TABLE 4.3.1-4. Permeability Versus Age of a Portland Cement Grout with $\omega = 0.7$ (Source: Littlejohn 1982)

Age (days)	Permeability (cm/sec)
Fresh	2×10^{-4}
5	4×10^{-8}
6	1×10^{-8}
8	4×10^{-9}
13	5×10^{-10}
24	1×10^{-10}
Ultimate (Estimated)	6×10^{-11}

Test grouting of alluvial sands with silicate-based grouts indicate that under field conditions permeability reductions of one to two orders of magnitude lower than the untreated permeability can be realized. Laboratory tests with the same grout material achieved permeability values averaging 4.8×10^{-7} cm/sec or two to three orders of magnitude lower than the field-grouted sand (Davidson and Perez 1982).

In general, chemically grouted sands exhibit permeability reductions of approximately three to six orders of magnitude lower than the original ungrouted sand (Baker 1982).

Grout Curtain Durability

Grout durability and grout compatibility with the surrounding environment are closely related. The durability (i.e., maintenance of permeability reduction) of cement-based grouts is effected primarily by the chemistry of the ground water in contact with the set grout. Deterioration of cement grouts can be caused by high concentrations of dissolved sulphates or acids in ground water, large-scale temperature fluctuations causing freeze/thaw cycles, and prolonged exposure to sea water (Littlejohn 1982).

Littlejohn (1982) recommends a water/cement ratio of 0.4 for grouts subjected to freeze/thaw cycles. Increased resistance to chemical breakdown of cement grouts can be achieved by using higher cost aluminous cements.

Silicate-based chemical grouts with silicate concentrations of 35% or more by volume tend to resist deterioration by freeze/thaw and by episodes of wetting and drying. Silicate grouts containing less than 30% silicate by volume should be used only for temporary applications. Repeated freezing and thawing will cause deterioration of acrylamide-type grouts because of the rupture of gel particle bonds. Laboratory tests indicate however, that for host materials grouted below the water table no significant deterioration of acrylamide-type grouts occurred in 15 years (U.S. Army Office of the Chief of Engineers 1973).

4.3.1.8 Effects of Ground Water on Grouting

The effect of ground water on grouting operations and the resulting grouted barrier is twofold. First, there are mechanical impacts associated with the ground-water pressure head and the velocity of the moving ground-water chemistry. Second, there are influences on the durability of the in-place grout caused by variations in ground-water chemistry.

As grouting closes drainage pathways, up-gradient ground-water elevations will rise thus causing increased pressure head on the grout itself. These higher pressures, caused by the placement of the grout, may cause the grout to be pushed out of the seepage zones and result in reopening of the flow pathways. Karol (1982b) recommends injecting grout near the end of treatment at pressures higher than the maximum anticipated future ground-water head.

Because ground water will be flowing in contact with the grout curtain there is a continuous potential for weakening of the grout. If grouts with high water/cement ratios are used, loss of most of the effectiveness of the barrier can occur as early as one year after construction (Houlsby 1982b). Uneven grouting may be caused by instability and mixing at the grout/ground-water interface. If the grout viscosity is greater than the viscosity of the water being displaced mixing will be reduced (Attewell and Farmer 1976).

For chemical grouts, the flow of the grout will be reversed due to their low viscosity as soon as the pumping injection pressures are eliminated. To avoid reverse flow pumping pressures should be maintained until the grout has developed some set strength (Attewell and Farmer 1976).

As long as the mechanical and chemical effects of ground water are considered in the design and installation of the barrier grouting can be successfully performed above and below the water table. Some precautions need to be exercised, however. For instance, ground water with a high pH can lead to premature deterioration of silicate-based grouts by inhibiting initial gellation. Conversely, low pH ground water may accelerate gellation of silicate grouts while preventing the setting of acrylate grouts. It is also necessary to determine if perched water exists in the grout zone and to establish the presence of any artesian pressures (Baker 1982).

4.3.1.9 Grouting Implementation Considerations

The five key issues related to implementation of grout barriers to mitigate ground-water contamination resulting from a severe power plant accident are:

1. Construction time,
2. Cost,
3. Toxicity of grout material,
4. Equipment mobilization, and
5. Worker safety.

Construction Time

The time required to construct a grout barrier to ground-water flow is highly variable and site specific. Construction time is a function of:

1. Size and orientation of barrier,
2. Grout hole spacing,
3. Grouting method,
4. Lithology of host material
5. Drilling method, and
6. Grout take rates and setting time.

Since each job is unique or customized there are no adequate unit timings for the various procedures comprising a grouting operation. Compared with other techniques, however, most grouting operations are significantly slower (Harris et al. 1982a). Several months may be required for a complete grouting operation.

Cost

The total cost of a grouting operation is a function of (U.S. Army Office of the Chief of Engineers 1973):

1. Initial cost of materials,
2. Location of job site,
3. Quantities and types of grout to be used,
4. Volume of material to be placed,
5. Labor,
6. Overhead,
7. Equipment rental, and
8. Drilling cost.

Of the total cost, direct contractor costs for supply of labor and plant may typically range from 40% to 55%. Site preparation, maintenance, and supplies costs may be expected in the range from 25% to 30% of the total cost. Finally, design and engineering costs may typically range from 20% to 30% of the total project cost (Fox and Jones 1982). The actual cost is highly variable from job to job and may not breakdown into the above ranges in every case.

Chemical grouts are commercially available at prices ranging from $0.13 to $2.64 per liter. Sample grout material cost data are presented in Table 4.3.1-5. While grouting material costs may vary 20 to 1 the overall in-place costs typically vary from 3 to 1 because the cost of grout placement is a major cost factor in the overall cost of the job (Karol 1982a).

Additional costs related to radiation protection should also be considered.

TABLE 4.3.1-5. Grouting Material Costs (1980$) (Source: EPA 1982)

Grout Material	Unit Cost ($/liter)
Portland Cement	0.25
Bentonite	0.33
Silicate	
20%	0.46
30%	0.55
40%	
Lignochrome	0.41
Acrylamide	1.76
Urea Formaldehyde	1.51

Toxicity

Most cement-based and clay/cement-based grouts are considered non-toxic although they are skin irritants. However, certain types of chemical grouts are highly toxic and manufacture of some otherwise very useful grouts, has been stopped due to their toxic behavior.

Sodium silicate grouts are considered non-toxic and non-corrosive and consequently do not pose any health threats. However, some of the reactant compounds used with silicate gels may be toxic and thus require a certain measure of care when handling (Karol 1982a).

Acrylamide grouts have been found to be neurotoxins and their manufacture has been discontinued in the U.S. Only the powders and solutions are toxic, however. The gel does not exhibit toxic behavior. An acrylate polymer grout (AC-400)® was made commercially available in 1980 as a replacement to acrylamide grout. AC-400® possesses much of the same properties as the discontinued AM-9® grout with approximately 1/100 the toxicity of AM-9® (Clarke 1982).

Lignosulfonate grouts containing a hexavalent chromium compound are extremely toxic. The resulting gels formed by these grouts may leach toxic materials into the ground water (Karol 1982a).

Grouts containing phenol or formaldehyde and an alkaline base represent potential health hazards. Grouts using urea solutions are also toxic and corrosive because of formaldehyde concentrations (Karol 1982a).

Equipment Mobilization

Many different pieces of equipment are necessary to complete a grouting operation. Drilling equipment is required for grout hole development and selection of a clear, continuous drilling right-of-way of several meters width should be part of the layout process.

Based on the size of the job (i.e., amount of grout to be placed) a variety of equipment component types may be suitable. Basic equipment requirements include mixer, agitator, pump, circulation line, and control fittings. These items can be individual components or in some cases combined into single units. For very large jobs the machinery can be installed in a central mixing and pumping station with circulation lines to particular site locations (Houlsby 1982a). This approach may be particularly conducive to grout curtain construction after a severe accident since the majority of heavy equipment could be placed in relatively "safe" areas on the site. Circulation lines up to two miles return have been successfully used. For most power plant sites grouting equipment mobilization would not exclusively preclude a grouting operation.

Worker Safety

Worker safety and protection from radiation exposure would be a serious implementation issue. The grouting should be automated and streamlined as much as practically possible in order to minimize the size of work crews. Consolidation of equipment in relatively safe zones should also be considered. The layout of the grout curtain should consider opportunities for placement of the curtain upwind from the prevailing wind direction if possible.

4.3.2 Slurry Trenches

Slurry trenches or cut-off walls are engineered/constructed barriers that may be appropriate for use in protecting local water supplies from contaminated ground water resulting from a severe power plant accident.

A slurry trench is a ground-water barrier that penetrates vertically through pervious layers of soil. It is keyed (i.e., built) into an underlying soil layer that is impervious to local ground-water flow. A gel like slurry mixture of bentonite clay and water is normally used to support a trench excavated for development of the slurry wall. The slurry supports the trench sidewalls and prevents collapsing of the excavation.

The slurry is either replaced with a backfill material, or with the direct addition of cement the slurry itself will harden to form the cut-off wall. Slurry walls are designed to specifications that are made on a site-specific basis. The wall must be sufficiently impervious to ground-water flow, resistant to degradation by the ground-water contaminants, and relatively permanent.

Slurry wall construction originated in Europe, but is now used extensively in the U.S. Over seventy slurry walls were built in the U.S. during the two year period before 1980 (D'Appolonia 1980). They have been used in subways, mine shafts, and building construction for structural stability, in dams to control seepage, at waste disposal sites for isolating contaminated ground water, and at construction sites for dewatering.

Slurry cut-off walls have several advantages over other types of ground-water barriers. They generally cost less than other methods. Also slurry walls can key into the underlying impervious layers without interlocks that are necessary for steel sheet piling. There is also no need to estimate overlap as in grouting. Finally, homogeneity and continuity can be tested by sampling excavation cuttings. Sampling insures that the wall will be placed to the appropriate depth (Miller 1979).

4.3.2.1 Design and Construction Considerations for Slurry Trenches

There are four general types of slurry walls. Each has its own set of advantages and disadvantages that require consideration on a site-specific basis.

Soil-Bentonite (S-B) Slurry Wall

The soil-bentonite slurry wall, sometimes referred to as the American method, is the most deformable and plastic slurry wall. Construction starts with marking and leveling the area where the trench is to be excavated. A backhoe usually begins the excavation by digging several feet along the planned trench alignment. A slurry mixture is then continuously added to the trench to prevent the sidewalls from caving during excavation. The slurry is a viscous mixture of bentonite clay and water. Bentonite is a high sodium montmorillonite clay that expands when wetted. It is prepared using a mixing technique best suited for the time and space restrictions at the project site.

Backfilling begins when the maximum trench depth has been reached over a portion of the wall length. The sides and bottom of the trench should be cleared of sediments by scraping them with excavation tools. Soundings of the trench depth, and cuttings or samples from the trench bottom are sometimes made to insure that the entire trench bottom is open and cleared (Miller 1979). If the sediments encountered are less permeable than the backfill their removal is usually not required and will only add cost and time to the project (D'Appolonia 1980).

Some construction companies use mechanical desanders or sedimentation methods to clean the slurry before backfilling. D'Appolonia (1980) states that these methods are useless and do not increase the performance of a slurry wall.

The backfill is usually mixed at the side of the trench. Either excavated soil or soil imported to the site is sluiced with the slurry, and then mixed by tracking and blading with a bulldozer. It is recommended (D'Appolonia 1980) that the slurry, to be used in the backfill, be taken directly from the trench. This slurry is thicker than freshly mixed slurry and contains a higher level of suspended particles. These two properties of the slurry that was used during the excavation process tend to decrease the permeability of the completed wall when the slurry is used as a constituent in the backfill.

Mechanical batchers and pugmills have been used to mix the backfill at sites that do not have enough room to use a bulldozer or where backfill materials are costly (D'Appolonia 1980). The backfill is placed initially in the bottom corner of the trench with clamshells, or pumped there through pipes that extend to the trench bottom. Placement continues until the backfill reaches ground level. Additional backfill is bulldozed into the trench causing downward slough over the initially placed backfill. Excavation, cleaning of the trench bottom, and backfilling occur simultaneously as pictured in Figure 4.3.2-1 until the wall is complete.

To finish construction a compacted clay cap, two to three feet thick, is usually placed over the trench.

Cement-Bentonite (C-B) Slurry Wall

In the construction of a cement-bentonite slurry wall, there is no need for backfilling. Cement is added to the bentonite-water slurry right before it is placed in the trench. The slurry itself hardens and forms the ground-water barrier. The wall has a relatively high strength and is not deformable like the S-B wall.

The alternate-slot method is usually used for deep trenches or for trenches passing through unstable soils. As shown in Figure 4.3.2-2, trench sections between 3 and 6 meters (10 and 20 ft) long are dug with the same length of unexcavated ground between them. The primary panels are formed when the C-B slurry hardens in the initially excavated trenches. The slots between panels are then excavated and filled with slurry to form the secondary panels (Harris et al. 1982b).

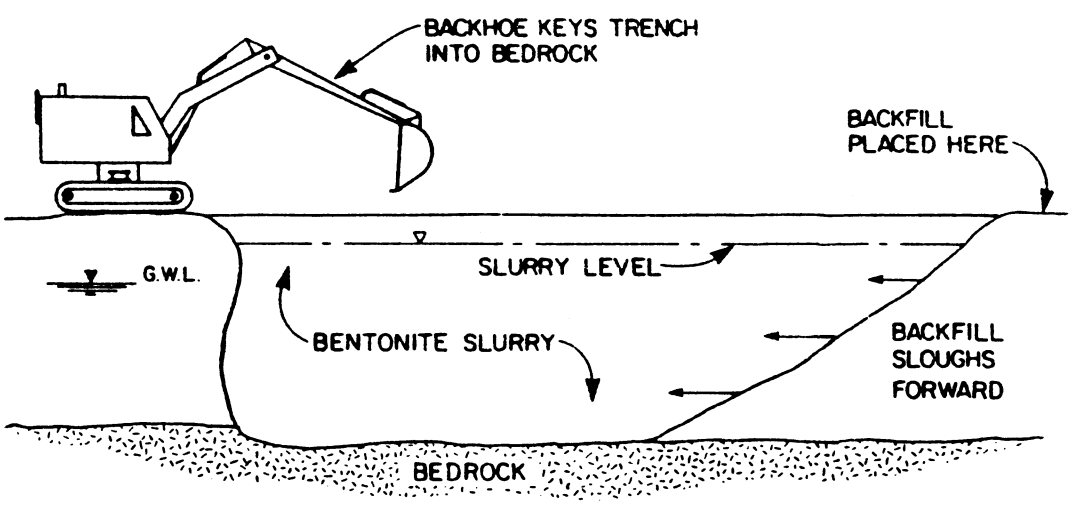

FIGURE 4.3.2-1. Schematic Section of Slurry Wall Construction (Source: Ayres et al. 1983)

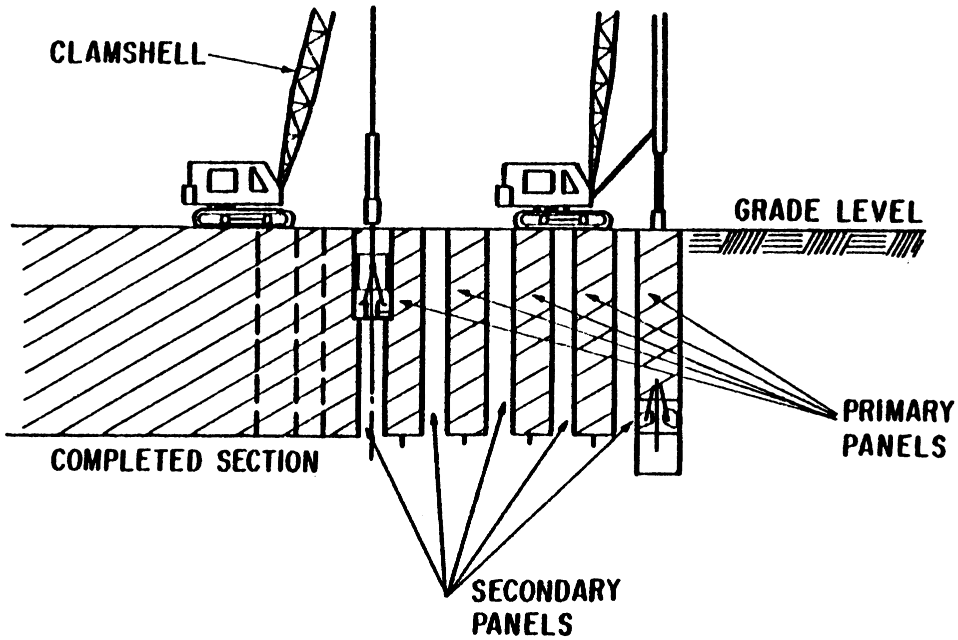

FIGURE 4.3.2-2. The Alternate-Slot Method
(Source: Harris et al. 1982b)

Cement-bentonite slurry walls have several advantages over S-B slurry walls. There is less length of open trench during construction of a C-B wall. The shorter trench length coupled with a faster slurry hardening time, stabilizes the ground and leaves less chance for trench failure. In the event of a failure, repairs are easier because the slurry mixture hardens relatively quickly. Trenches can be cut through a C-B wall without causing sloughing of the backfill. Traffic can cross the trench in a few days. Finally, C-B wall construction is not dependent on the availability or quality of soil for backfill (Ryan undated).

Lean-Concrete (L-C) Slurry Wall

Lean concrete (34-68 kg/m^2 unconfined strength) slurry walls are best suited for deep trenches or when highly pervious zones are encountered (Harris et al. 1982b). In this method concrete is pumped through tremie pipes that extend to the bottom of a trench filled with bentonite slurry. The slurry is displaced by the concrete and removed at the top of the trench. The tremie pipes should remain at least 1.5 meters (5 ft) below the level of concrete in the trench. By keeping the same concrete in horizontal contact with the bentonite slurry, the concrete can be cleaned at the top of the trench of impurities transferred from the bentonite slurry[a].

(a) Information from advertising brochure of Bencor Corporation of America, Dallas, Texas.

To prevent discontinuities in the completed wall, some contractors will desand before concreting begins. One desanding method requires sucking the slurry out of the trench through a pipe. The slurry is then sent through a vibrating screen to sift out large particles and subsequently through a fine grain desander. Sand can also be removed from the slurry by reducing the power of suspension of the slurry by adding sodium tripolyphoshpate. The sand that falls to the bottom can be removed with a clamshell.

The alternate slot method is used during construction. One tremie pipe per 4.6 meters (15 feet) is standard procedure (Millet and Perez 1981). To ensure the continuity of the wall, the ends of the primary panels are shaped by using end-pipes or wide-flanges. These ends are filled in when the secondary panels are tremied in.

Vibrating-Beam Thin (VBT) Slurry Wall

This technique was brought to the U.S. from Europe in 1975[a]. There exists some controversy vibrating-beam slurry wall effectiveness as a groundwater barrier because they are very thin; usually no more than 10 cm (4 in.) wide.

A special crawler crane equiped with vibrator, leads, and injection beam repeatedly injects a C-B slurry into the soil forming a continuous impervious wall (Figure 4.3.2-3). The slurry is made in a mixer, pumped to the injector and forced into the soil; no backfilling is necessary. Slurry Systems states that the VBT method accurately keys into the bottom impervious layer, increases the homogeniety of the wall, and uses slurry that is less contaminated than backfilled trenches[b].

Slurry Systems follows specific procedures when mixing C-B slurry (Schmednecht undated). Bentonite is augered into a stream of water and pumped through a centrifugal pump for approximately six minutes. Cement is added and a centrifugal pump mixes it for roughly 3 additional minutes. The slurry is then stored for a limited time or is pumped directly to the vibrating beam injection rig.

The mixed slurry is jetted into the ground with the aid of an injection beam driven by a vibrating pile-hammer (Figure 4.3.2-4). The injection beam is a standard wide flange section. Wear tips are welded to the end of the injection beam to adjust the width of the slurry wall. The lead on the crane can be adjusted laterally and vertically to assure plumbness on uneven or loose ground. There is a vertical hydraulic support ram with a bearing pad on the bottom of the lead for stability.

(a) Information from advertising brochure of Slurry Systems, a division of Thatcher Engineering Corporation, Gary, Indiana.
(b) Letter from Frank Zlamal, Slurry Systems Division of Thatcher Engineering Corporation, to John Shafer, PNL.

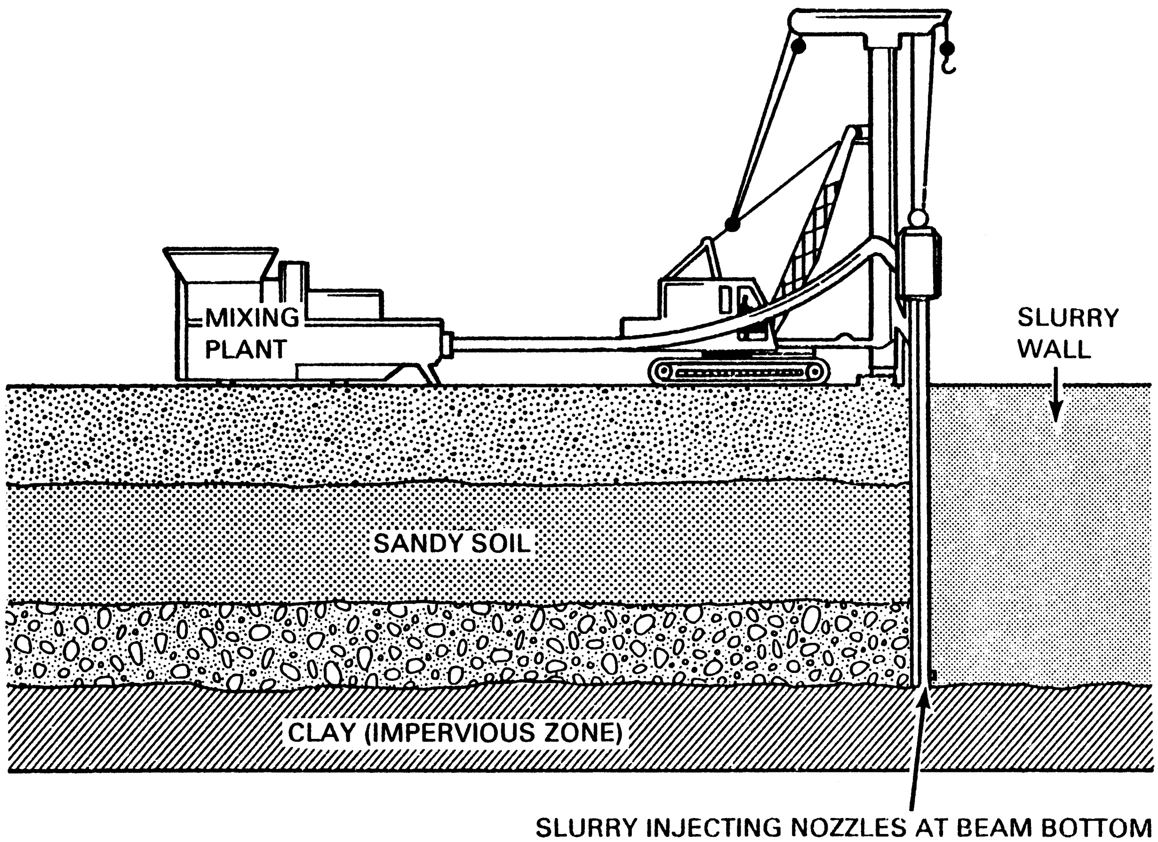

FIGURE 4.3.2-3. Vibrating-Beam Slurry Wall Construction
(Source: Slurry Systems Brochure)

While the injection beam is removed from the ground, the C-B slurry is jetted under pressure into the depression. The slurry wall is constructed by successive injections of slurry into the ground.

The main problem with the VBT method is assuring continuity at depth between adjacent passes. Calculations must be done at each site to make sure the vibrator is powerful enough to force the beam into the soil. The VBT technique works best in sandy type soils which are easy to penetrate. Keying-in to consolidated underlying layers is not possible (Schmednecht undated).

There are several advantages to the VBT method. Construction is not dependent on the quality of on-site soil for backfill as in the S-B method, and mixing can be done at a distance from the trench. The cement in the backfill makes for a quick set. The VBT method uses less materials and time than other slurry wall methods.

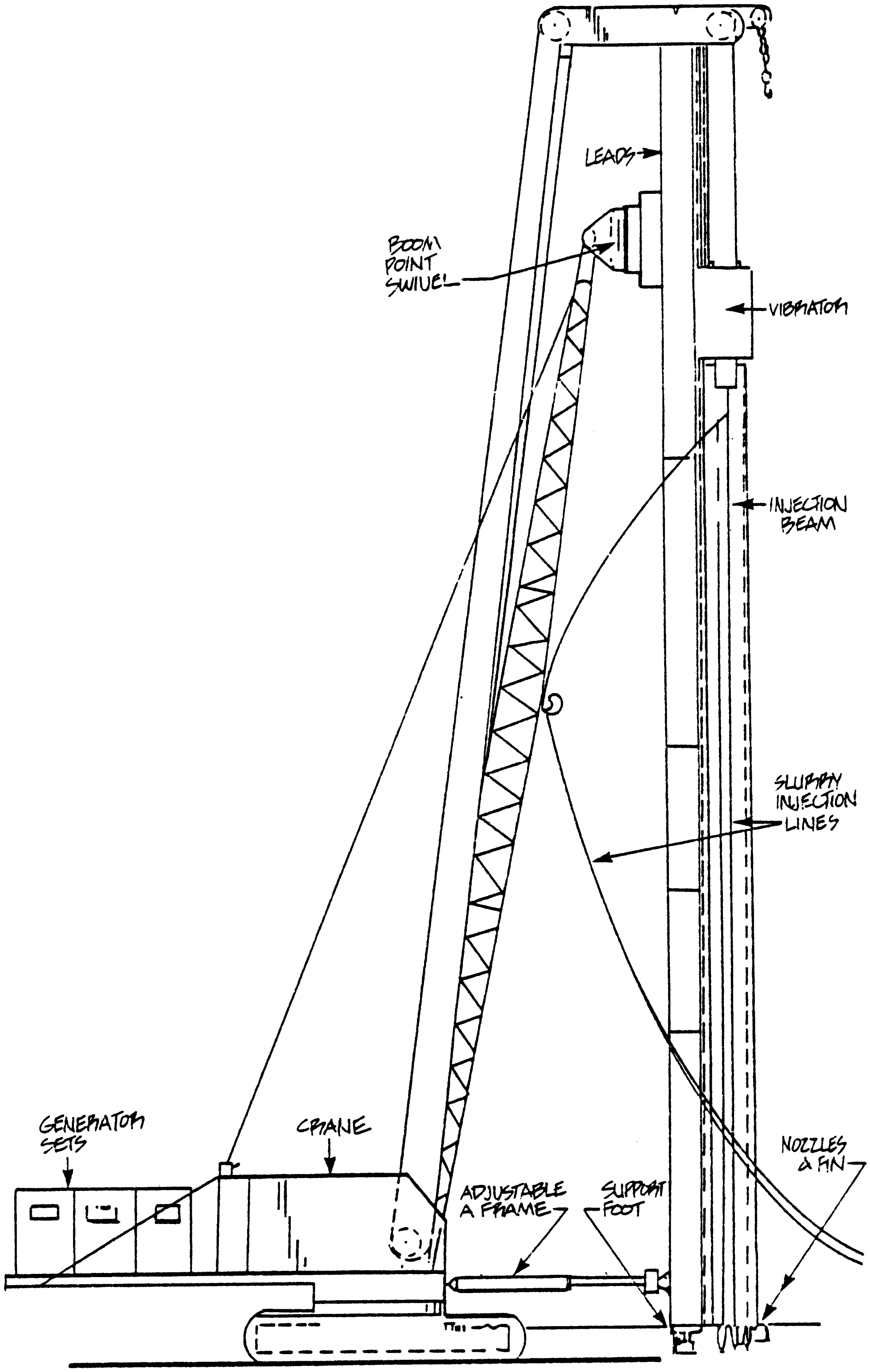

FIGURE 4.3.2-4. Typical Vibrated-Beam (VBT) Injection Set-Up (Source: Schmednecht undated)

Size

The depth of a slurry wall is dependent on both the geologic formations into which the slurry wall is to be keyed, and the strategy used to control radionuclide migration (Millet and Perez 1981). Backhoes can dig up to 17 meters (55 ft) into the ground; draglines to about 30 meters (100 ft). Clamshells can be used up to 85 meters (280 ft), which is the practical limit for excavating unconsolidated layers. Long wall drills (i.e., multiple head drills) can be lowered by a steel cable and block to break up bedrock[a]. Percussion drills (i.e., compressed air driven pistons which transmit hammer blows to the drill rods) will penetrate down to 60 meters (200 ft) which is therefore the consolidated layer limit (Harris et al. 1982b).

The choice of width of a slurry wall depends on the required permeability, the material make-up of the wall, the hydraulic head across the wall, and the size of the available excavation equipment (Millet and Perez 1981). An S-B trench width can range from 0.6 to 2.5 meters (2 to 8 ft) in width. Most are built 2 to 2.5 meters (6 to 8 ft) wide. The average width of a C-B trench is narrower: 0.6 meters (2 ft). A wider trench is needed during construction of a S-B slurry wall in order for the backfill to overcome the frictional forces of the trench sidewalls and slough downward.

At depths up to 15 meters (49 ft), continuous excavating and backfilling can be done in order to form an entire C-B wall. At depths beyond this lower limit, and up to 75 meters (250 ft), the alternate-slot method is most often used.

Widths of slurry walls constructed using the VBT method range from a minimum of 8 cm (3 in.) (Schmednecht undated) to a maximum of 16 cm (6 in.) (Harris et al. 1982b).

The average width of a VBT slurry wall is about 10cm (4 in.) (Harr et al. undated). The wall should be thicker for more permeable soil. To adjust the width of the wall, wear plates are welded to the tip of the injection beam (Schmednecht undated). Depths of 30 meters (100 feet) can be reached in permeable soils (Harr et al. undated).

Location and Orientation

The location of the slurry wall depends on the direction and the gradient of ground-water flow. If the aquifer has a well-defined unidirectional hydraulic gradient and is laterally confined, then either an up-gradient or down-gradient slurry wall could be constructed. The barrier must divert ground water around the contaminated site, stop the movement of contaminants, or sufficiently slow their migration to the point where they decay to acceptable levels. Various shapes such as L-shaped walls should be considered.

(a) Information from advertising brochure of Bencor Corporation of America, Dallas, Texas.

A wall placed up-gradient from the power plant may effectively divert local ground water around the contaminated area. The wall may even divert ground water below the contaminant source without being keyed into a low permeability soil layer. A slurry wall placed down-gradient may sufficiently retard or stop the contaminated leachate (EPA 1982). Contaminated ground water that reaches the wall could be pumped to temporary storage for treatment and subsequent recharge.

Another strategy to prevent contaminated ground water from migrating, is to completely surround the plant with the barrier. This type of barrier is best suited to areas where the direction of ground water may reverse, such as tidal areas and near major rivers (EPA 1982). Surrounding the site with a barrier may have two advantages: 1) uncontaminated ground water is effectively diverted around the contaminant source, and 2) the barrier will isolate the site from the regional hydraulic gradient which would considerably reduce contaminant transport.

As discussed in the section on key-in integrity, there is a possibility that contaminants may vertically leak through irregularities in the keying layer and out of the slurry wall confinement area. Pumping water out of the slurry wall confine can be used to mitigate this downward leakage. As shown in Figure 4.3.2-5, when the fluid level in the slurry wall confinement area is kept at a lower level than the surrounding ground water, flow will be into the confined area, and no contaminants will escape (D'Appolonia undated).

Water should be pumped until a balance in the hydrostatic pressures inside and outside of the contained area directs flow inward. Additional pumping may be needed to maintain this balance, although the slurry wall will greatly reduce the required pumping volume by slowing down the movement of ground water into the confinement site. Pumping to create an inward flux of ground water might not be economical at sites where leakage is great (Harris et al. 1982b).

Care must be taken to avoid hydrofracturing of the slurry wall when pumping. This occurs when the ground-water pressure exceeds the gel strength of the slurry. Blowout tests on slurry samples can be done to determine the hydraulic gradient at which a failure may occur (Harris et al. 1982).

4.3.2.2 Performance Considerations for Slurry Trenches

The effectiveness of slurry walls in restricting ground-water flow depends on several factors. These factors include ground-water conditions, soil limitations, and keying layer restrictions (Harris et al. 1982b). Low permeability is the most important performance criterion that must be met. However, deformability, strength, and durability of the wall should also be considered.

Permeability

Typical values for the coefficient of permeability (K) for several soil types are presented in Table 4.3.2-1.

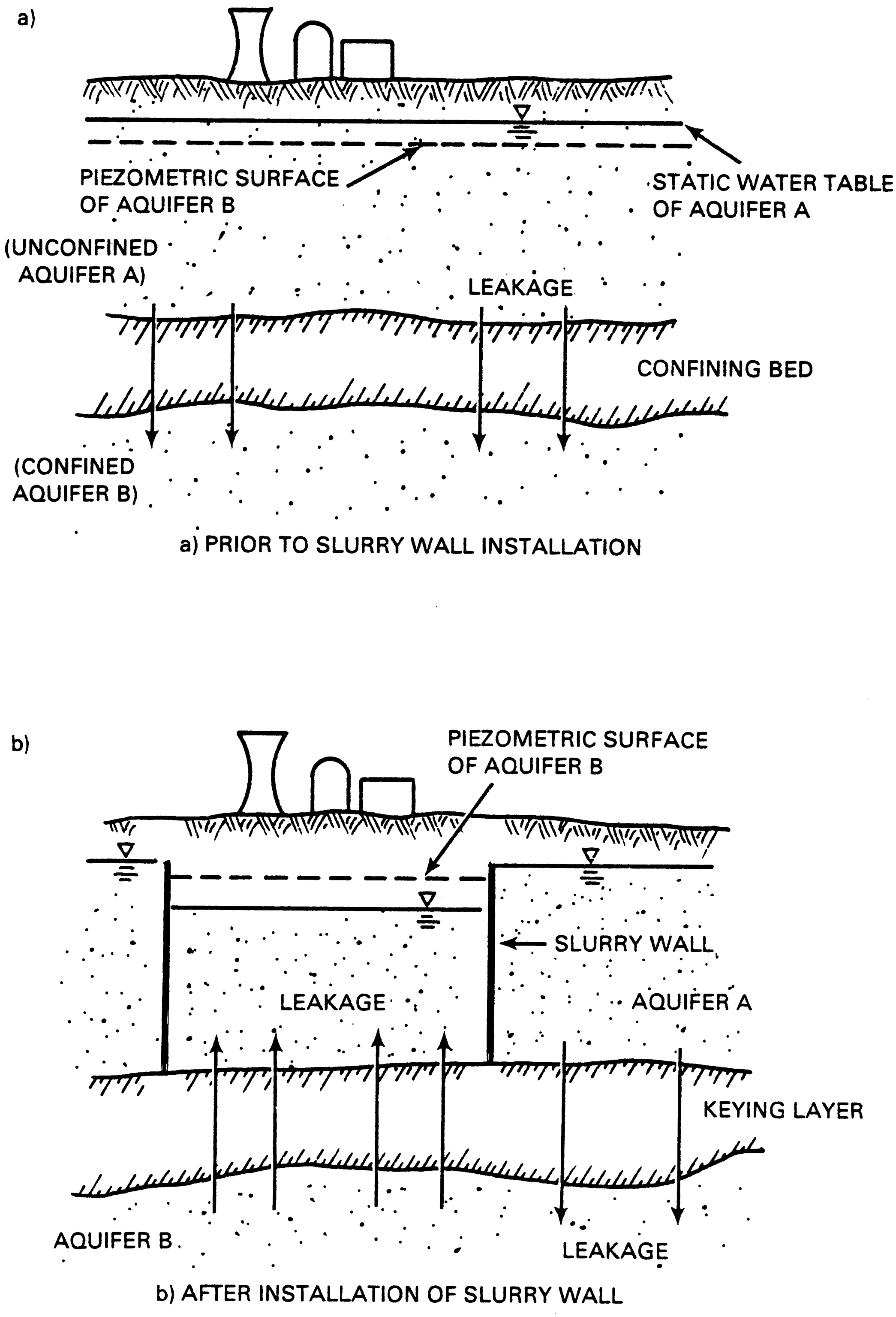

FIGURE 4.3.2-5. Reversal of Leakage Direction Through Aquitard/Keying Layer as a Result of Pumping Ground Water Within Slurry Wall Confines to a Level Below the Piezometric Surface for the Underlying Aquifer (Source: Harris et al. 1982b)

TABLE 4.3.2-1. K Values Ranked According to Soil Particle Size. (Source: Attewell and Farmer 1976)

Clay: $K < 10^{-7}$ (cm/s)

Silts: $10^{-7} < K < 10^{-5}$ (cm/s)

Fine sands: $10^{-5} < K < 10^{-3}$ (cm/s)

Coarse sands: $10^{-3} < K < 10^{-1}$ (cm/s)

Gravels: $10^{-1} < K$ (cm/s)

Variance of Permeability With Particle Size

As is shown in Table 4.3.2-1, there is a relationship between particle size and the coefficient of permeability. This is important when considering the proportions of fine and coarse grade particles to be used in the S-B backfill mixture.

Figure 4.3.2-6 plots the permeability of S-B backfill as a function of soil gradation (the bentonite content is held constant at 1%). The gradation is classified by the percentage of material that passes through a standard No. 200 mesh sieve. For both plastic and non-plastic fines the smaller the soil particle, the less permeable the backfill. Therefore soil types near the top of the table in Table 4.3.2-1 (clay, silt, and fine particles) will decrease the permeability of the backfill. By using mixes that contain over 30% plastic fines, a low permeability wall can be made (D'Appolonia undated).

The amount of bentonite clay used, also has an effect on permeability. In Figure 4.3.2-7 the amount of bentonite is plotted against the permeabilities of backfills containing various grades of soils. Permeability is shown to decrease as increasing percentages of bentonite are used. However, a mix containing a high bentonite concentration is seldom used for the reasons pointed out in the following example.

Performance criteria specifying the limits on the gradation of the backfill mix are often required of the contractor building the slurry wall. At a hazardous waste disposal site located in Nashua, New Hampshire (Ayres et al. 1983), the backfill material was required to contain over 5% bentonite and over 30% fines. The desired permeability of 10^{-7} cm/s for the completed slurry wall could have been achieved with the addition of bentonite alone. The silt size fines were used instead because they were less expensive, improved the consistency of the backfill, and were found to degrade less in the presence of leachate from the dumpsite.

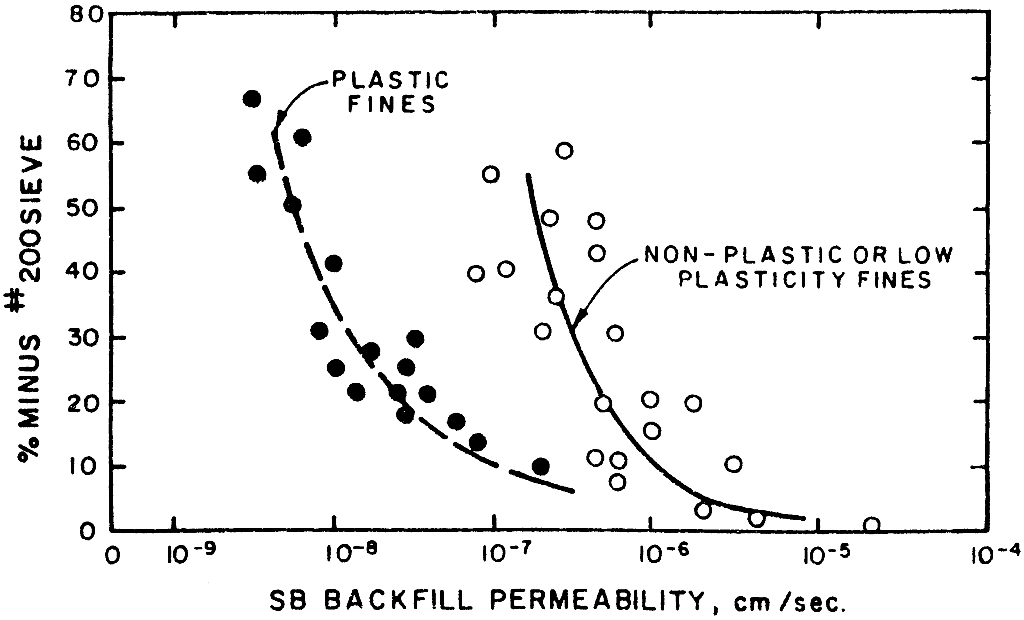

FIGURE 4.3.2-6. Permeability of Soil-Bentonite Backfill Related to Fines Content (Source: D'Appolonia undated)

The grade of the bentonite used has an effect on the permeability of a slurry wall. Bentonite of higher grade has a greater swelling potential and will be less pervious (Ayres et al. 1983).

Filtercake Versus Backfill Permeability

As shown in Figure 4.3.2-8 there are two phases for a S-B slurry wall. First there is an outer filtercake layer formed by the seeping of the slurry through the walls of the trench. The bentonite penetrates through the sidewalls to a distance dependent on the surrounding soil permeability and on the viscosity and gel strength of the bentonite suspension (Harris et al. 1982b). This thickness ranges from less than a meter to a meter.[a] Seepage stops when the filtercake thickness limits any more flow (Ryan undated).

The inner phase of the cut-off wall is made up of the backfilled material (S-B method), concrete (L-C method), or the hardened slurry (C-B and VBT methods). The down-gradient side of the filtercake is often ruptured by seepage forces and extruded into the trench sidewall (D'Appolonia 1980). The up-gradient side usually stays intact. Separate permeability tests on the filtercake and the backfill of C-B slurry walls have been carried out (Harr et al. undated).

(a) Information from advertising brochure of Bencor Corporation of America, Dallas, Texas.

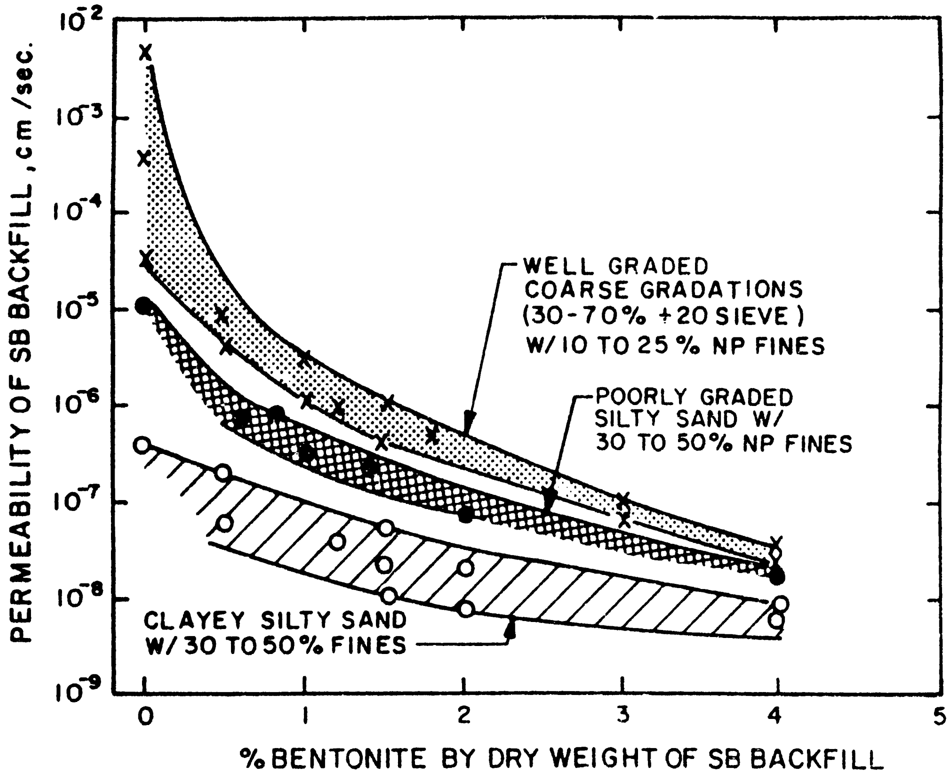

FIGURE 4.3.2-7. Relationship Between the Permeability and Quantity of Bentonite Added to S-B Backfill (Source: D'Appolonia undated)

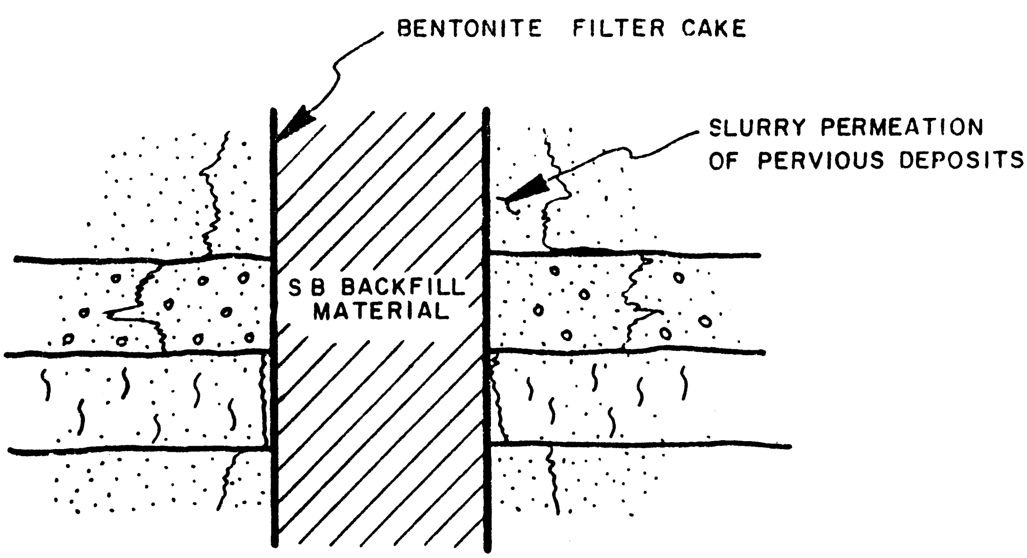

FIGURE 4.3.2-8. Schematic Cross Section of S-B Slurry Trench Cut-Off Wall (Source: D'Appolonia undated)

To measure filtercake permeability with these tests, water is suctioned through a sand bed covered thinly with C-B slurry. After a layer from 3 to 4 mm forms, it is covered with a layer of water. A head of about 9 meters is applied to the sample by suction from below. The resulting permeability values range from 3 X 10^{-8} to 5 X 10^{-8} cm/s.

When measuring backfill permeability the slurry is put in a container in contact with a Millipore filter held by a fritted glass support. A head of about 30 cm is applied by capillary column on the 3 cm thick specimen. A typical coefficient of permeability determined with this test is 3 X 10^{-6} cm/s.

From the experimental results it is postulated (Harr et al. undated) that the filtercake is less permeable than the backfill because the bentonite particles have more time to orient themselves. In situ tests using drawdown methods in test wells yield values of 10^{-6} to 10^{-7} cm/s, which fall between the laboratory test results for the two phases. There exists some debate as to the effect of the filtercake on the overall permeability of C-B slurry walls.

A thorough study of the relative permeabilities of the filtercake and backfill layers was done by D'Appolonia (1980) for S-B slurry walls. The average permeability (K) of the wall is presented in Figure 4.3.2-9.

The average permeability of the wall is calculated using Darcy's Law and making the assumption that the thickness of the backfill is very much greater than the thickness of the filtercake. The study (D'Appolonia 1980) concluded that the overall permeability of the slurry wall is controlled by the backfill when the backfill permeability is low. However, when the backfill permeablity is high the filtercake is the controlling factor. Furthermore, due to the low permeability of the filtercake, the upper limit of the wall permeability is on the order of 10^{-6} cm/s. This figure is accurate assuming that the up-gradient filtercake does not rupture under the hydraulic pressure of the ground water.

The permeability and thickness of the filtercake depend on several criteria. One criterion is the bentonite-water ratio of the slurry. The greater the permeability of the soil the slurry wall penetrates, the higher the concentration of bentonite needed. For soil permeabilities between 10^{-1} and 10^{-2} cm/s, a bentonite concentration of 4 to 6% will suffice. For highly permeable soils, concentrations up to 12% may be needed, although flocculants can be added to reduce the fitrate loss and to save on the added expense of the bentonite. Lightweight aggregate or plastics are other additives used to plug fissured soils and rock formations (Harris et al. 1982b).

The American Petroleum Institute filter press test was used to compare filtercake permeability and filtrate loss for several different types of bentonite. In the investigation the slurries that form thin filtercakes were less pervious than the thick filtercakes. The permeability to thickness (K/t) ratio, therefore, remains unchanged as thickness is varied. This cancelling effect makes filtrate loss irrelevant for use as a quality control criteria.

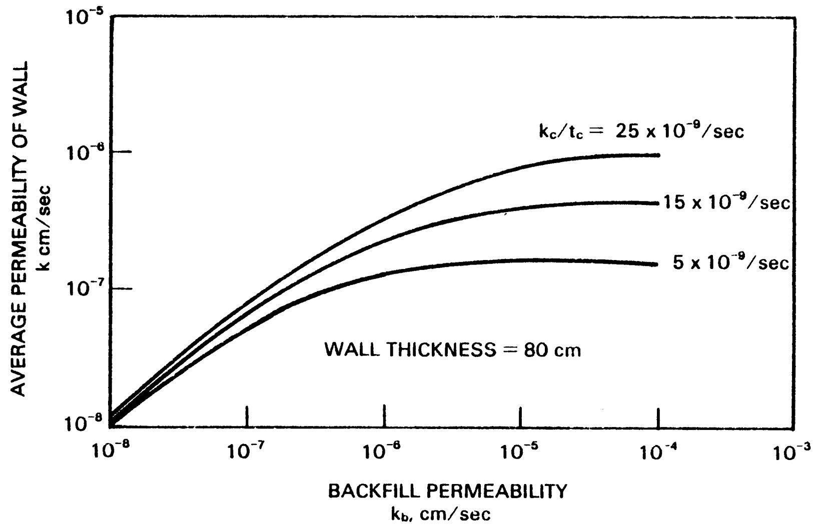

FIGURE 4.3.2-9. Theoretical Relationship Between Wall Permeability and Permeability of Filtercake and Backfill (Source: D'Appolonia 1980) b = backfill, c = filtercake, t_c = thickness of filtercake)

The Marsh Funnel Test is commonly used to determine viscosity. The test involves simply recording the time it takes for a slurry to run through a funnel of standard size. This amount of time is referred to in "Marsh-seconds". D'Appolonia (1980) found the viscosity of the slurry to have little effect on filtercake permeability if it is measured greater than about 38 Marsh seconds.

The time it takes for the filtercake to form and the difference in head between the slurry and the pore fluid in the soil also have an effect on the filtercake (D'Appolonia 1980). In a filter press test, using the apparatus shown in Figure 4.3.2-10, permeability (K/t) is measured by the flow rate of water through the filtercake layer, divided by the head and area of the sample.

In an experiment done by D'Appoplonia (1980) cake permeability is plotted against the head applied to the sample for four varying lengths of time (Figure 4.3.2-11). The four seperate curves plotted from the data imply that the filtercake permeability is more dependent on the formation time than on the pressure applied to it. Lower permeabilities were found for filtercakes formed over greater periods of time. Letting a slurry with a viscosity of over 40 Marsh seconds sit in the trench for 24 hours before backfilling allows a filtercake of a sufficiently low permeability to form.

Soil Permeability Limits for Slurry Wall Use

A typical soil-bentonite slurry wall containing 10-20% fine particles (No. 200 standard sieve) and 2-4% bentonite clay by weight will have a

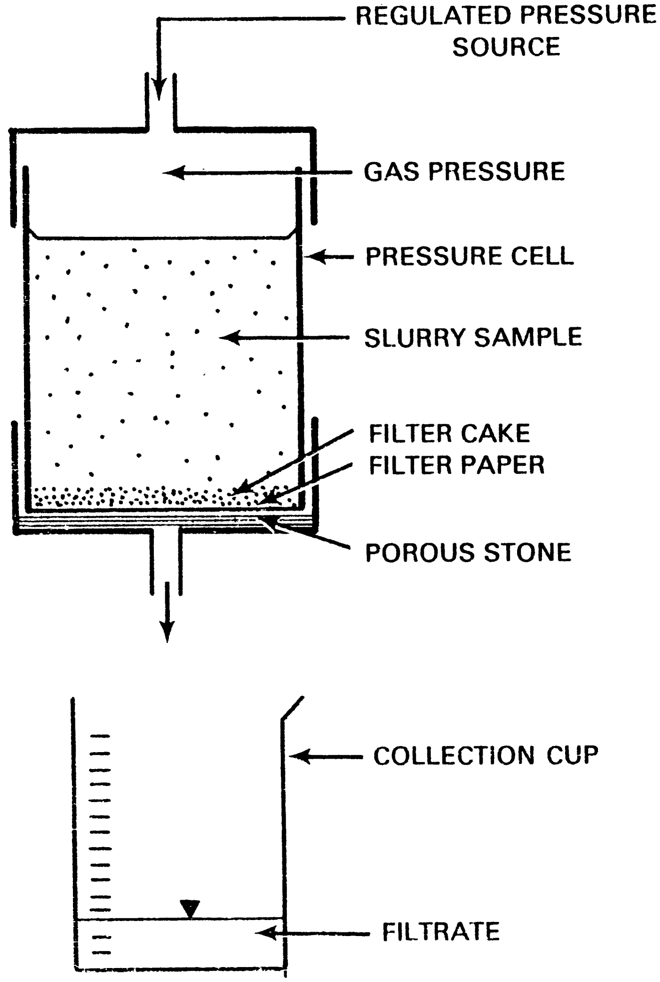

FIGURE 4.3.2-10. Schematic of Filter-Press Test Apparatus (Source: D'Appolonia 1980)

permeability of about 10^{-7} cm/sec (Millet and Perez 1981). Darcy K factors between 10^{-6} and 10^{-7} cm/sec are found with field measurements for cement-bentonite slurry walls (Harr et al. undated). This range in permeability values encompasses that of clay (shown in Table 4.3.2-1). A slurry wall, therefore, placed in a clay medium would not decrease the flow velocity of contaminated ground water through that area; the wall would be a redundant feature. The wall would be helpful if there were highly fissured zones within the relatively impervious soil, or if a less permeable slurry wall was used. Newly developed C-B slurry mixes are reaching Darcy K values between 10^{-9} and 10^{-10} cm/s (Harr et al. undated).

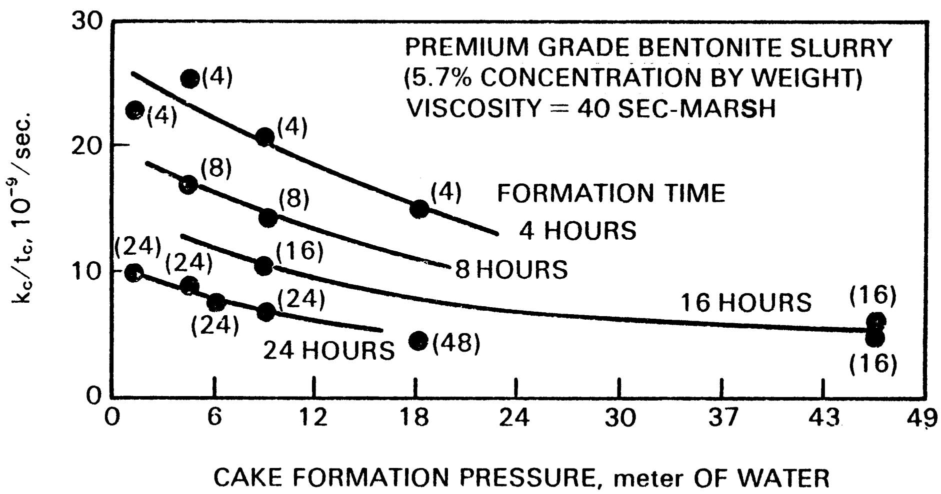

FIGURE 4.3.2-11. Relationship Between the Filtercake Permeability and Cake Formation Pressure and Time (Source: D'Appolonia 1980)

When seepage velocities are above 5 cm/s the bentonite particles in the soil-bentonite slurry cannot layer themselves to form filtercake on the trench sidewalls. The slurry will seep through highly permeable soils and will not fulfill its function of stabilizing the trench. Plastics or light-weight aggregate can be used to slow slurry losses through rock, gravel, and other very permeable layers (Harris et al. 1982b).

In summary, the utilization of slurry walls is limited to soils having Darcy K values greater than 10^{-6} cm/s and ground-water velocities less than 5 cm/s. This range is suitable for most soils, since their permeability range averages from 10^{-2} to 10^{-3} cm/s.

Continuity

A slurry wall must be continuous and have good integrity to maintain low permeablility. Often times C-B slurry walls and L-C slurry walls are constructed in sections. The connections between the sections must be good to avoid cold joints or windows through which ground water could leak. Retarders can be used to slow the hardening process and permit better contact between panels (Millet and Perez 1981).

A high level of plasticity will permit the healing of cracks caused by the ground shifting or other pressures put on the wall. An S-B slurry wall is similar to a slowly thickening gelatin. The slurry migrates toward the point of higher liquid flow. This expansion tends to repair seal deformations and fissures in the wall.

C-B slurry walls are not considered infinitely plastic. Although they can withstand compressive strains of several percent under in situ conditions without cracking. A new slurry wall construction method having elastic properties and an extended urethane base is being developed by Slurry Systems, a division of Thatcher Engineering Corporation.

Sand material and other large soil particles left in the slurry suspension after excavation will make the wall more brittle. These particles increase the strength and the permeability of the wall; two deleterious qualities for a ground-water barrier.

Slurry walls are non-structural. They are not built to support bending moments of significant shear stress. Concentrated loads put on top of the wall are mitigated by placing a cap over the completed wall.

Key-in Integrity

To insure low permeability, contaminated ground water must be stopped from flowing under the slurry wall. The slurry wall should penetrate through all pervious zones, such as desiccation cracks, and make a good connection (i.e., have high keying-in integrity) with a highly impervious layer beneath it.

If the underlying stratum is clay, the wall should be keyed from 0.5 m to 1.0 m (2 ft to 3 ft) into the layer. When bedrock is the keying layer, the slurry wall is usually built directly over it (EPA 1982). Percussion drilling to excavate the trench through the bedrock is expensive and can cause the rock to fracture. In this case the bottom of the trench is cleaned thoroughly to insure that unwanted sediments (i.e., those of higher permeability than the surrounding soil) that could cause pervious voids are not caught between the bedrock and the slurry wall.

Discontinuities in bedrock or gaps in soil can be filled by grouting. The grouting method consists of pumping a C-B slurry into the ground under pressure to seal any voids beneath the slurry wall.

Contaminants can travel vertically through a leaky keying layer in several ways (Harris et al. 1982b). Figure 4.3.2-12a shows a slurry wall which has been keyed into a layer of low permeability. The contaminated ground water seeps through this layer to the main aquifer below. In Figure 4.3.2-12b the slurry wall is keyed into sufficiently impervious geologic strata with good integrity. The contaminants, though, leak through a permeable window inside the contained area. A similar problem exists in Figure 4.3.2-12d where contaminants leak through an undetected permeable zone. Grouting has been used in Figure 4.3.2-12c to seal voids around the bottom of the slurry wall. Leakage still occurs through cracks in the soil strata. Pumping can mitigate the downward flow of contaminated ground water, although it is more economical to avoid this situation by carrying out proper tests during design.

4.3.2.3 Appropriate Geologic Media

Typical ground sequences where slurry cut-off walls have been succesfully used are shown in Figure 4.3.2-13. It is important to consider variances in soil types, and the depth at which they occur in the implemenation area for

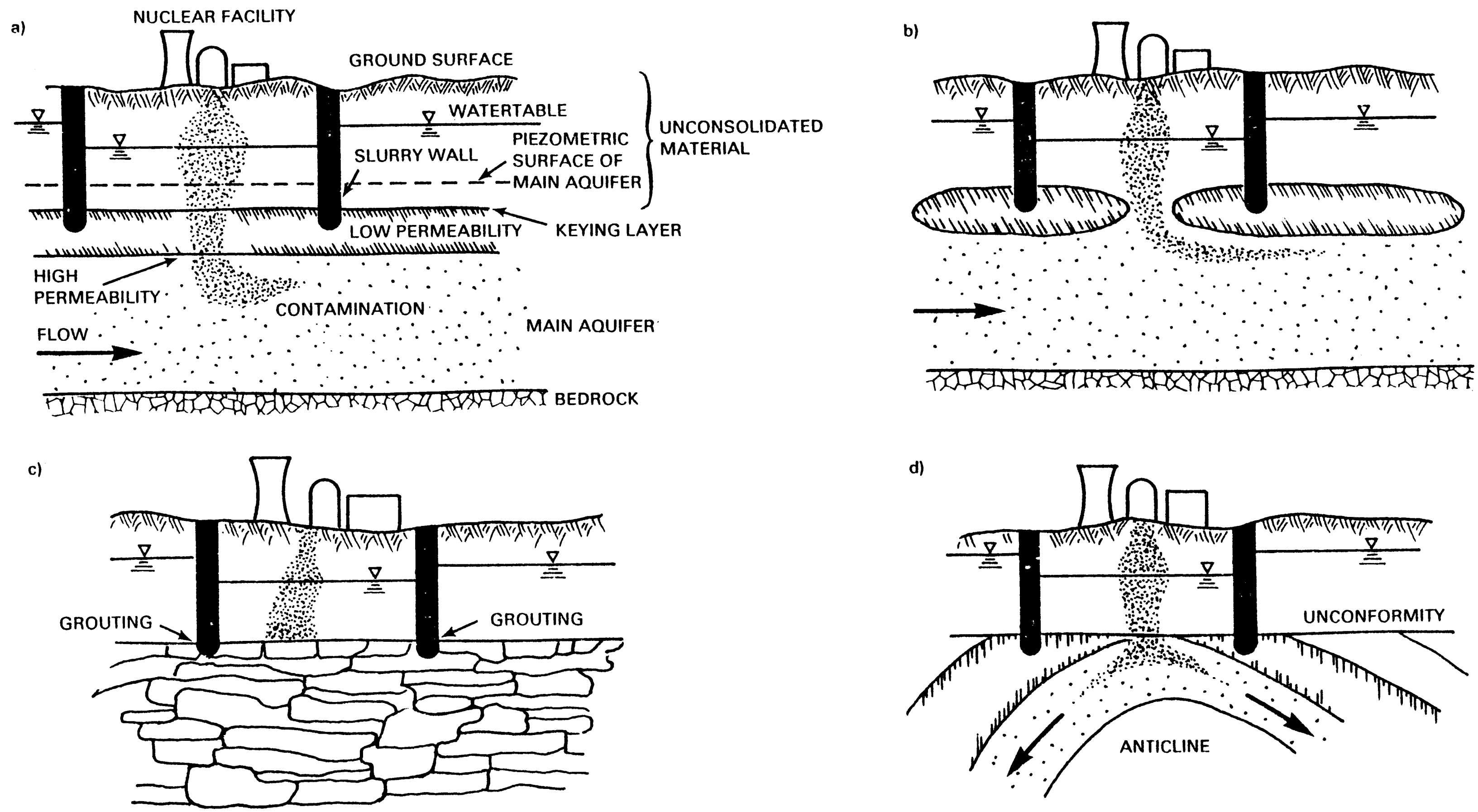

FIGURE 4.3.2-12. Vertical Leakage of Contaminants [through: (a) area of high permeability in aquitard layer, (b) opening in discontinuous clay lenses, (c) secondary openings in limestone or dolomite, (d) undetected geological structures (Source: Harris et al. 1982b)]

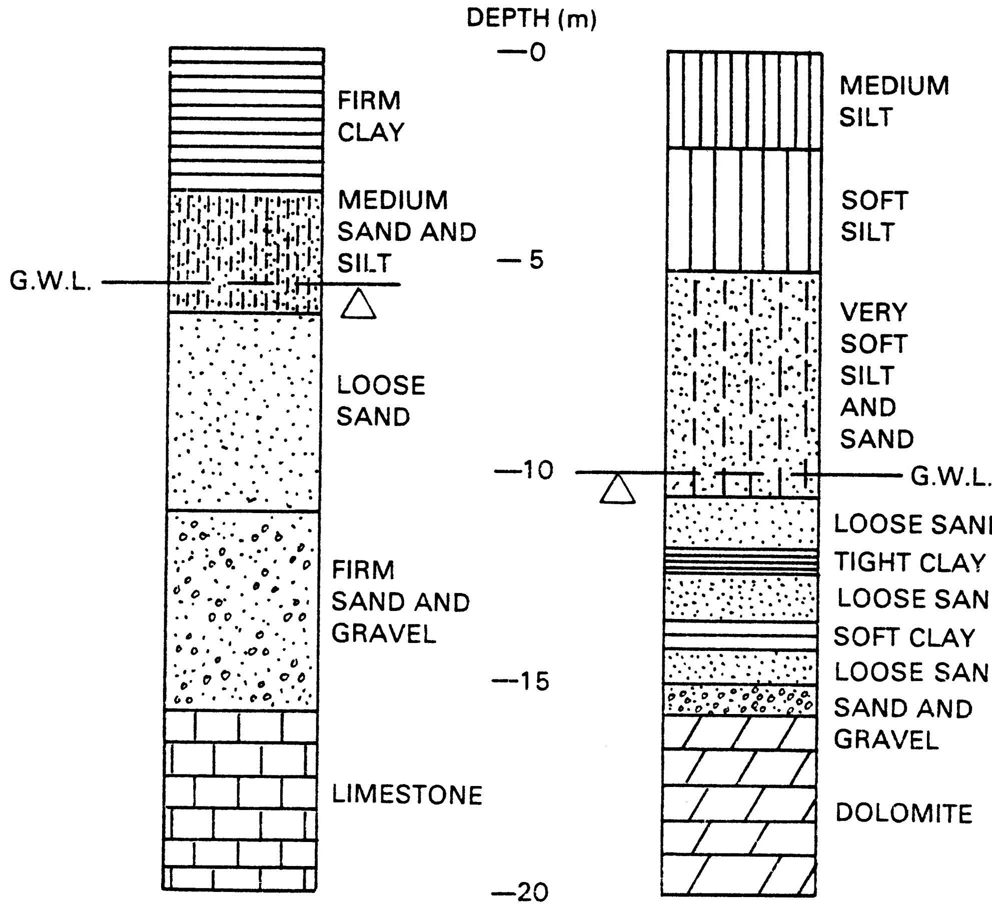

FIGURE 4.3.2-13. Typical Ground Sequences Where Bentonite Support Has Proved Effective (Source: Attewell and Farmer 1976)

several reasons. First of all, the shear strength (i.e., internal friction) of each soil layer must be greater than the hydrostatic pressure exerted by the slurry on the trench sidewalls. Some loose soils such as marine clays, alluvium, and fresh hydraulic fill, do not meet this requirement, and may cause the trench to cave in (Harris et al. 1982b).

Seepage of slurry out of the trench through a highly permeable layer such as gravel, combined with ground-water inflow from a saturated layer may also cause trench failure (Figure 4.3.2-14). To prevent such an event, tests for permeability, density, water content, hydrologic pressures, and porosity for each soil type should be conducted (Harris et al. 1982b).

If loose soils are an expected problem, the wall should be designed as straight as possible. Draining with well-point systems is also recommended in some cases to decrease the soil void ratio and increase the shearing strength.

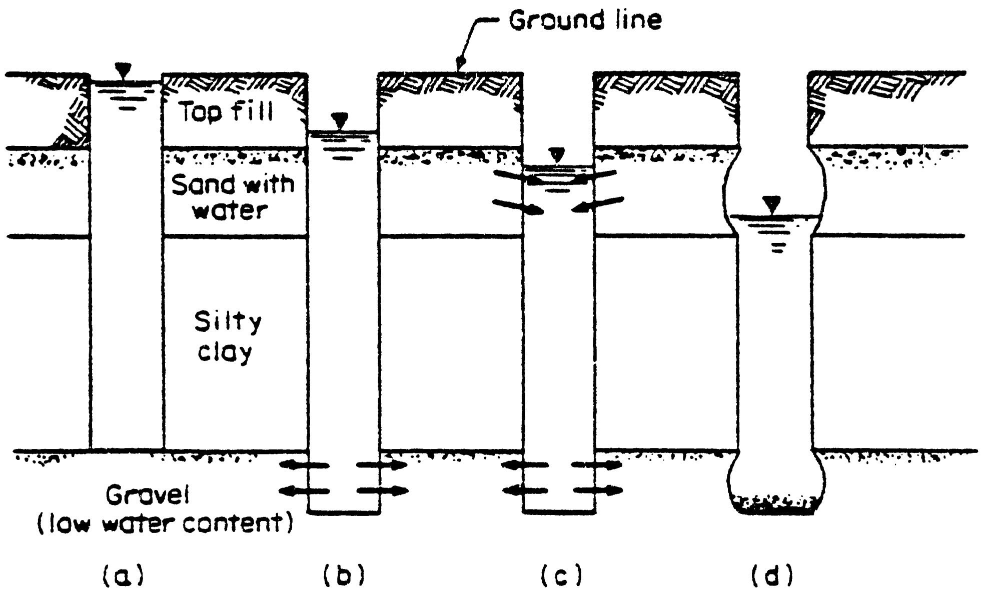

FIGURE 4.3.2-14. Loss of Slurry Water to Unsaturated, Highly Permeable Gravel Zone Leads to Loss of Slurry (Source: Harris et al. 1982b)

A dense array of well-points may be needed to draw water from soft soils which have low permeabilities. The likelihood of trench collapse can also be lessened during construction by excavating smaller panels and backfilling them appropriately.

The type of geological media encountered is a factor in choosing the type of slurry wall. Compact soils have greater shear strengths and exert less pressure against a slurry, whereas loose and soft soils would tend to collapse in the trench. A C-B slurry may be the best choice in soils prone to trench failure. A C-B slurry wall is more viscous than the S-B slurry, provides more physical strenghth when hardened, and is easier to repair if a failure does occur (Harris et al. 1982b).

A C-B slurry may be the best choice in very permeable soils such as sands. Due to its high viscosity and density, there would be less slurry loss by seepage through the trench walls.

The saturated loose type soils are best for the VBT method because less force is needed to penetrate these soils (Schmednecht undated). The beneficial qualities of a C-B slurry (i.e., viscous and self-hardening properties) also exist for a VBT slurry in loose soils, because they are basically the same mixtures.

The L-C wall is most appropriate in deeper trenches that pass through coarse gravel and boulder zones (Harris et al. 1982b). A C-B slurry wall might ordinarily be effective in highly permeable zones except it might set before the excavating depth is reached.

In mid-range permeable soils the S-B method is the cheapest if the excavated soil can be used to backfill the trench. S-B walls are more elastic and could prove to hold up better in areas with more ground shifting (Miller 1979).

The slurry wall at the Bonneville Dam in Washington is a good example of a slurry wall that was successfully built through many different layers of soils. The wall was constructed to control seepage through a bottom pervious alluvium layer while a powerhouse was built next to the dam. Excavation of the trench was made through the toe of an ancient Cascade slide mass. Various layers consisted of preslide alluvium, slide blocks, slide debris, recent alluvium, and deep sand deposits. To make matters even more difficult, the ground-water level in the topmost alluvial deposit fluctuates with rainfall. Despite the challenging combination of soil layers and varying ground-water levels, a L-C wall was successfully tremied into the trench[a].

Slurry walls can be effective ground-water barriers even when built through soils made up of many different layers. The combined effects these layers have on trench stability, seepage through the sidewalls, and the soil needed for backfill (S-B wall), should be thoroughly studied while designing and choosing the type of slurry wall to be used. The geological history of the area and boring tests are good sources of this information.

Preconstruction Testing

Soil characteristics important in designing a slurry trench cut-off wall for a specific site are: permeability, the amount of soil stratification, and the depth and nature of the impervious layer.

Many techniques are used to gather this information. At the Rocky Mountain Arsenal (Miller 1979), located in Denver, Colorado, test boring holes were made around a contaminated basin (Figure 4.3.2-15). The results were used to determine the depth to bedrock and the orientation of subsurface materials to be penetrated by the slurry wall.

In Figure 4.3.2-16, four borings depict a two-dimensional vertical cut into the ground. Table 4.3.2-2 shows the Unified Soil Classification used in the boring profile. The horizontal line that approximately bisects the profiles divides the top most clay soils from the sandy soils below. The shale key-in layer is shown by the lower line. Rock quality designation (RQD) indices and qualitative hardness are sometimes used to characterize the bedrock to determine ripability. It was decided that the slurry wall was to be keyed into the shale layer by at least 0.6 meters.

The borings were made by a hollow stem auger. Split spoon samples were taken at 1.5 m increments of change in stratum. Tests results from these samples for boring profile #461 can be seen in Figure 4.3.2-17. The table

(a) Information from advertising brochure of Bencor Corporation of America, Dallas, Texas.

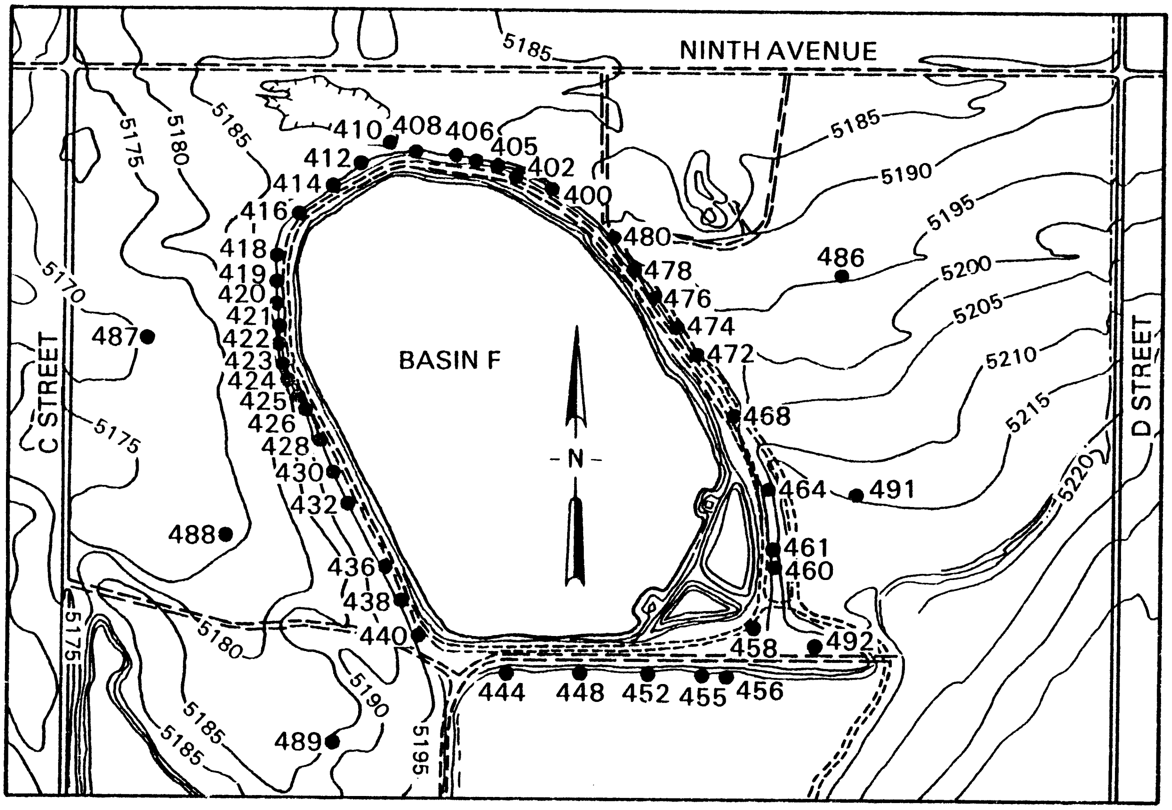

FIGURE 4.3.2-15. Soil Boring and Observing Well Locations in the Basin F Area at the Rocky Mountain Arsenal (Source: Warner 1979)

directly next to the boring shows the blows per foot, or N value, a measure of the relative density of the soil. These values were estimated from Standard Penetration Test results (Figure 4.3.2-18). This test is fairly reliable for sands, but is a crude measurement for clays (Peck 1953).

The water content naturally occuring in the soil types is indicated by an unfilled circle in Figure 4.3.2-17. The dark dots mark the Atterberg plastic state limits which define the effect of exchangeable ion composition and are sketched for only the clay minerals. The minimum moisture content at which a clay exibits plasticity is the plastic limit. The liquid limit is the point at which the clay begins to flow (Attewell and Farmer 1976). Specific gravities, and estimated values of density, strengths, and porosities are also tabulated.

At the Nashua, New Hampshire site (Ayres et al. 1983), gradation tests were done using on-site soils and soils brought in from elsewhere. Hydraulic conductivity tests on S-B mixtures using both short term, high gradient and

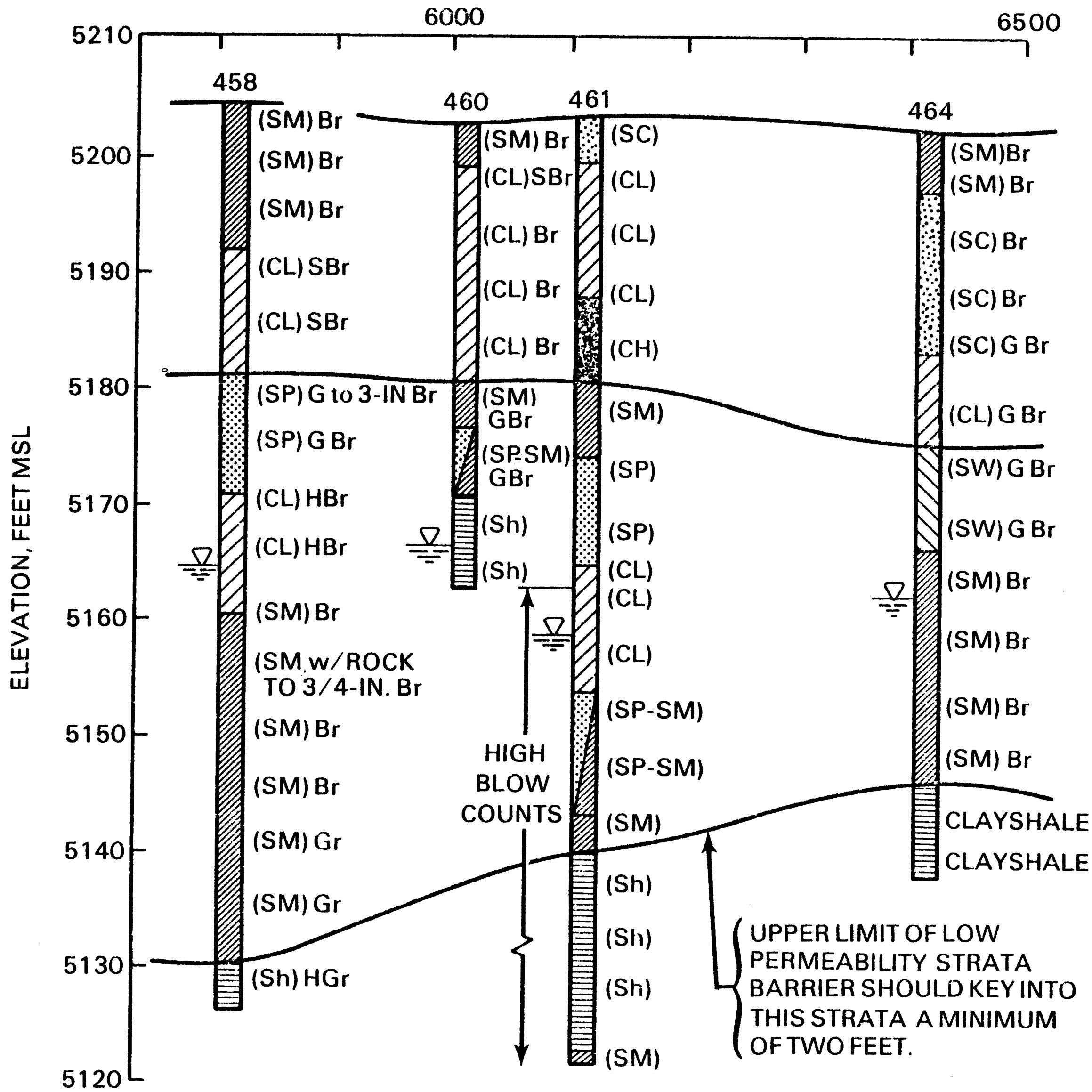

FIGURE 4.3.2-16. Partial Soil Profile Depicted by Borings Around Basin F at Rocky Mountain Arsenal (Source: Warner 1979)

long term, low gradient methods along with x-ray diffraction tests, of samples permeated with the contaminated leachate, were used to set limits on the amount of fine grained borrow and bentonite additives to be used.

Other preconstruction testing includes hydrogeologic investigations to determine the depth, flow rate, direction, and chemical characteristics of ground water. These characteristics need to be known to prevent caving in of

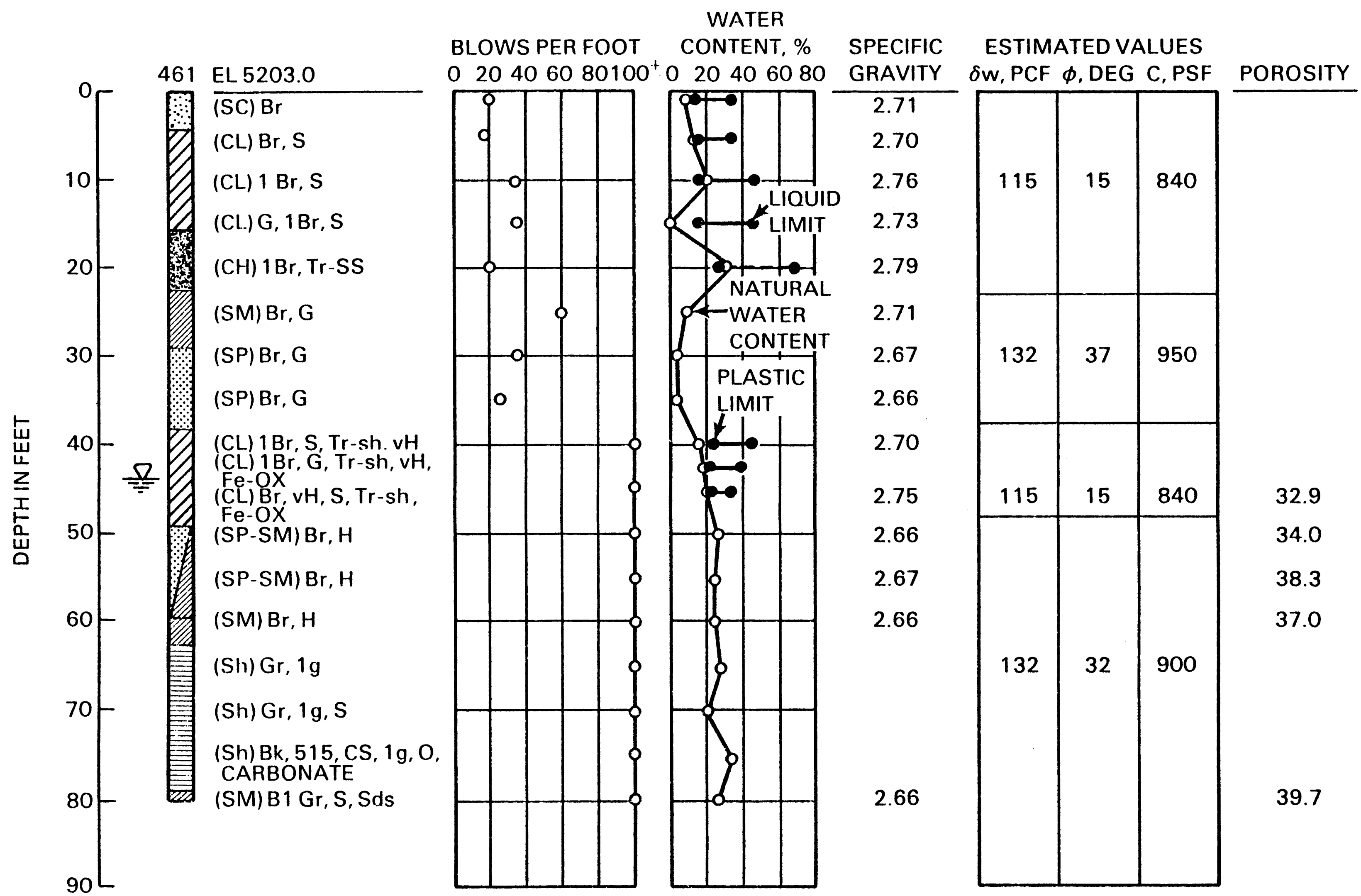

FIGURE 4.3.2-17. Boring Log for Hole No. 461 (Source: Warner 1979)

TABLE 4.3.2-2. Unified Soil Classification System (Source: U.S. Dept. of Interior, Bureau of Reclamation 1974)

Group Symbols	Typical Names
GW	Well graded gravels, gravel-sand mixtures, little or no fines.
GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.
GM	Silty gravels, poorly graded gravel-sand-silt mixtures.
GC	Clayey gravels, poorly graded gravel-sand-silt-clay mixtures.
SW	Well graded sands, gravelly sands; little or no fines.
SP	Poorly graded sands, gravelly sands; little or no fines.
SM	Silty sands, poorly graded sand-silt mixtures.
SC	Clayey sands, poorly graded sand-clay mixtures.
ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity.
CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
OL	Organic silts and organic silt-clays of low plasticity.
MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
CH	Inorganic clays of high plasticity, fat clays.
OH	Organic clays of medium to high plasticity.
P	Peat and other highly organic soils.

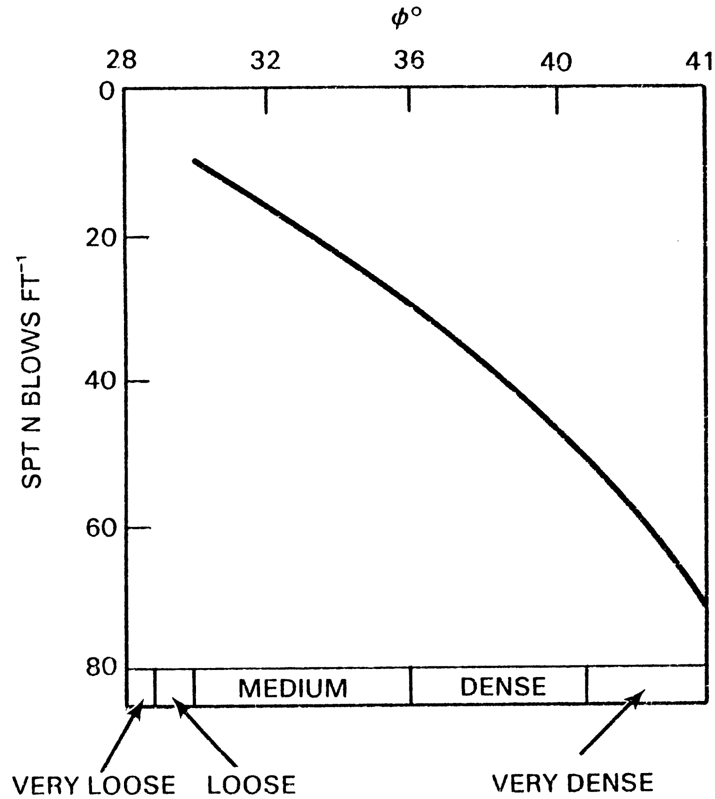

FIGURE 4.3.2-18. O-N Relationship for Standard Penetration Test (Source: Attewell and Farmer 1976)

the trench due to high hydrostatic pressures, and to predict the direction and velocity of contaminants reaching the wall.

Water quality information from past monitoring programs and test results from observation wells give information on artesian pressures, ground-water levels, springs, and seasonal variations in ground-water conditions. It should also be determined whether man-made recharge areas (e.g., cooling resevoir) exist at the site (Millet and Perez 1981).

The tests carried out to aid in the design of a cut-off wall can be divided into three groups: surface and subsurface reconnaissance, in situ tests, and laboratory tests. These tests and their objectives are outlined in Table 4.3.2-3.

4.3.2.4 Quality Control Considerations

Clay Mineralogy

Clay minerals can be divided into three major groups: montmorillonites, illites, and kaolinites. Montmorillonites that have their surface charge

Table 4.3.2-3. Determination of Site Characteristics (Source: Harris et al. 1982b)

	Techniques and Information Sources	Objective
Surface and Subsurface Reconnaissance	Use of available information provided in: - Topographic maps - Aerial photographs - Soil surveys - Surficial and bedrock geologic maps - Regional and local geological reports - Existing site investigation reports - Hydrologic investigation maps and reports - Geophysical investigation maps and reports	- Topographic/physiographic analysis - Hydrologic/geohydrologic reconnaissance - Geologic reconnaissance - Geologic reconnaissance - Soils reconnaissance
In situ Site Characterization	Surface geophysics - Electric resistivity surveys - Seismic surveys	Identification of water-bearing formations; lithologic units; bedrock contacts; physical, rock properties, such as porosity, fluid content, and elastic constants; faults; fractures; cavities; and geologic structures.
	Borehole geophysics - Electric resistivity - neutron probe - Acoustic velocity - flow meter - Spontaneous potential - fluid conductivity - Gamma-gamma probe	Supplementary information to coring and hydrologic sampling. Can be used to analyze perched water and annual water table fluctuations; stratigraphy and structure of aquifers; hydraulic conductivity; effective porosity; mineralogy of consolidated and unconsolidated material; rate of groundwater movement; dipersivity coefficients; groundwater chemistry; transmissivity, and storage coefficients.
	Well logging - Auger boring - Well drilling - Split-spoon sampling	Systematic description of soil or rock material (degree of weathering, microstructure, color grain-size alteration); core fracturing; discontinuity; spacing, horizontal distribution and extent of the different soil strata; porosity.
	Soil surveys - Standard penetration test - Core penetration test - Field sampling and description - Auger boring, corings test pits, etc.	Description of soil-grain size, plasticity, relative density, undrained shear strength, location, and extent of soil units.
	Geologic surveys - Site survey	Detailed site survey to map topographic, geomorphic, geologic, and hydrologic features such as slumping soils, swampy soils, springs, steep slopes, subsidence, faults, etc., that may have been overlooked during preliminary site investigation or that were noted during site reconnaissance.
	Hydrologic surveys - In-situ permeability tests - Field pumping tests - Piezometric level surveys	Groundwater levels, seasonal fluctuations, rate of flow of confined and unconfined ground water, regional and local groundwater flow patterns, leakage of confined aquifers, identification of aquifers, permeabilities.
Laboratory Investigations	Grain-size analysis - Sieve - Hydrometer	Slurry/groundwater compatibilities, permeabilities, porosities, dispersivity coefficients; suitability of borrow and backfill material.
	Laboratory permeability tests	
	Chemical analysis--soil and water - Soluble salts - pH - Organic matter - Chemical contaminants - Clay analysis	

balanced by the sodium ion, are of particular interest for their use in slurry mixtures. These clays are referred to as sodium bentonite, sodium montmorillonite, or because they are found and mined principally in Wyoming, they are called Wyoming bentonite.

A combination of tetrahedral and octahedral sheets form a lattice structure for sodium bentonite. The tetrahedral layer is composed of units of one silicon atom surrounded by four oxygen atoms. The octahedral layer has units of one multivalent cation and several hydroxyl ions (OH-). These layers form clay particles that are flat and flake-shaped (Jepsen undated).

Compared to illite clay, there is much less replacement of Si^{+4} by Al^{+3} in the tetrahedral layers of bentonite clay. This causes the total cationic charge between the structural units to decrease and, subsequently, expansive tendencies are not held back. A net charge imbalance is set up in bentonite by about 20% of the Al^{+3} ions being replaced by Mg^{+2} ions in the octohedral layer. In sodium bentonite clays this imbalance is satisfied by a sodium ion at the surface of the unit. These sodium ions are held loosely and readily exchange for other cations, especially those of higher valence (Jepsen undated).

Sodium bentonite has a large surface area of up to 800 m^2/gram when fully hydrated (Jepsen undated). This characteristic coupled with its high ionic exchange tendency gives it a cationic exchange capacity between 80 and 150 meq/100 grams. Compared with the cationic exchange capacities of kaolinite and illite, which range from 3 to 15 meq/100 grams and 20 to 40 meq/100 grams, respectively, the range of values for sodium bentonite clay is very high. This high ionic exchange capacity causes bentonite to expand 10 to 15 times its dry size upon hydration. When water is added to bentonite attractive forces set up between the water and the clay. The flake-shaped clay particles separate as a thin film of water forms between them, which acts like a lubricant to disaggregate them further. This phenomenom accounts for bentonites high swelling property and its use as a soil sealant (Jepsen undated).

When bentonite is mixed with water it becomes a thixotropic gel (i.e., it becomes fluid when agitated and rethickens when left stationary). The viscosity of hydrated bentonite increases over time as the clay particles orient themselves. When bentonite is mixed with the right quantity and sizes of particulate matter it forms an effectively impervious barrier.

Slurry Properties

The primary function of a slurry in the construction of cut-off walls is to support the sides of the trench during excavation. In order to fulfill this requirement, the slurry must be sufficiently dense. Although if it is too thick it will impede the excavation, backfilling, and trench cleaning operations. A low filtrate loss is also needed to ensure that the slurry remains in the trench and does not seep through the sidewalls. Another variable slurry property is viscosity. A thick slurry is needed to suspend loose soils and prevent them from accumulating at the base of the trench. Conversely, the slurry must be thin enough to facilitate pumping and circulating during construction operations (Harris et al. 1982b).

By controlling the viscosity, specific gravity, and filtrate loss, an optimal slurry can be made. The pH of a slurry should fall between 6.5 and 10. A deflocculating agent may be required for a pH greater that 10.5 (Millet and Perez 1981).

Repeated testing during construction will insure that quality controls are met. Table 4.3.2-4 outlines the quality control testing done at a hazardous waste disposal site located in Nashua, New Hampshire. The types, frequency, and desired values from the tests are tabulated. Many of these procedures are defined in the API specification RP-13B.

Viscosity

Viscosity is the primary property tested to determine the usability of a slurry. It is a measure of the ability of a fluid to resist shearing and depends largely on the extent of hydration of the bentonite clay (Ryan undated).

The Marsh Funnel test is commonly used to determine viscosity. Acceptable values for both S-B and C-B slurry range from 30 to 80 Marsh-secs at 20°C, with an optimum value of about 40 Marsh-secs. Changes in the slurry viscosity during excavation may be due to differences in batches of slurries mixed and added to the trench, changes in the underground environment, and the time the slurry is left sitting in the trench. In order to return the slurry to optimum consistency, new slurry may be added to the trench (Ryan undated).

The development of gel strength may be more important in making an efficient slurry wall than the apparent viscosity. Many problems are encountered, though, when measuring gel strength. Ultrasonic pulse velocity techniques have been used to circumvent some of these problems. These tests give shear modulus data, G, by measuring the pulse velocity for a shear wave moving through the gel.

Filtrate loss

Filtrate loss is the loss of water from a slurry when put under pressure. A high filtrate loss wastes slurry and causes a concentration of slurry in the filtercake. Filtrate loss is irrelevant as a quality control criterion. Despite this finding, many contractors measure the extent of hydration of the bentonite slurry by this property.

A filter press test (such as API Test PP131B) simulates the formation of filter cake on the trench walls. Filtercake is the slurry and soil combination resulting from electrokinetic and seepage forces that push the slurry through the sides of the trench. The test predicts how much slurry will be lost during

TABLE 4.3.2-4. Quality Control Testing Program During S-B Slurry Wall Construction at Gilson Road Hazardous Site (Source: Ayres et al. 1983)

ITEM		STANDARD	TYPE OF TEST	MINIMUM FREQUENCY	SPECIFIED VALUES
Materials	Water	--	-- pH -- Total Hardness	Per water source or as changes occur	As required by bentonite supplier to properly hydrate bentonite with approved additives
	Additives	--	Manufacturer certificate of compliance with stated characteristics		As approved by Engineer
	Bentonite	API Std 13A	Manufacturer certificate of compliance		Premium grade sodium cation montmorillonite
Slurry	Prepared for placement into the trench	API Std 13B	- Unit Weight - Viscosity - Filtrate Loss	1 set per shift and per batch (pond)	Unit Weight = 1.03-1.30 gm/cc V $\geq$ 15 centipose or 40 sec-Marsh 20°C Loss $\leq$ 30 cm in 30 min 690 kilopascal
	In Trench	API Std 13B 1	- Unit Weight	1 set per shift	Unit weight - 1.03-1.30 gm/cc at point of backfilling
			- Sand Content	1 set per shift	
Backfill Mix*	At Trench	ASTM C 141	- Slump	1 set per 375 M'	Slump 10 to 15 cm
		ASTM D 422-61	- Gradation	1 test per 375 M'	Consistent with design mix ($\geq$ 30 passing 200 sieve; $\geq$ 5 bentonite)
		ASTM C 138	- Density	1 test per 375 M'	$\geq$ 1.6 gm/cc
		EM1110-2-1906 Appendix VII	- Triaxial hydraulic conductivity test	1 test per 2000 M'	$\leq 1 \times 10^{--}$ cm/sec

* Note: Hydrometer testing of off-site borrow shall be required if said borrow contains greater 0.5 - 2μ material; for use in computing bentonite in backfill mix.

excavation and, also, how fast the cake will form or reform if disturbed. For a C-B slurry the normal range of filtrate loss is from 100 to 180 ml in 30 minutes (Millet and Perez 1981). For a S-B slurry the filtrate loss should measure less than 30 ml in 30 minutes (Ryan undated).

Slurry additives such as slag and fly ash can reduce fluid loss by as much as 20%. However, testing of the long term effects of these additives on the durability of the slurry wall should be made because many of these additives are biodegradable (Jefferis 1982).

Specific Gravity

The specific gravity of the slurry in the trench must be slightly greater than that of the surrounding ground water in order to deter an inward flow of water. The Mud Balance is a standard device used to measure specific gravity. A S-B slurry should exhibit specific gravity range between 1.03 and 1.30 gm/cc (Ryan undated). A slightly higher range of 1.03 to 1.40 gm/cc is permissible for C-B slurries. The upper limit is greater for C-B slurry where backfilling is not necessary. A lesser value is necessary for S-B slurries due to the difficulty of backfilling a trench containing a very dense slurry. The backfill may fold over some slurry material and trap it causing discontinuities or areas of high permeability in the wall.

In some situations, such as where a large differential hydraulic head will be in contact with the slurry, a highly dense slurry can supply needed support to the trench. Adding excavated materials of sand to the slurry may increase its density (Ryan undated). This increases the chances of losing colloidal stability of the bentonite gel which can lead to hydrofracturing of weak soil layers. Trapping the sand during backfilling (S-B method) or tremie concreting operations (L-C method) is another danger that presents itself when sand concentrations are high in the slurry.

Mixing Trench Support Slurry (S-B and L-C Methods)

There are several ways to adequetely mix bentonite and water. Two of the fastest mixing machines are the centrifugal digester and the colloidal mixer. It takes only a few minutes to mix a batch of slurry if peptizing agents are used with a continuous high speed colloidal mixer. It takes several hours to mix a slurry in a venturi flash-mixer. After mixing, the slurry can either be used right away or held in a slowly-circulating pond (Ryan undated).

The circulating time and the mixing time greatly effect the gel strength and viscosity of a slurry. The viscosity of a slurry increases over time and does not tend toward an equilibrium value. Tests (Jefferis 1982) conducted on four different types of bentonite found positive evidence for this hypothesis.

An increase in the gel strength of slurry over time was tested. The data points are plotted in Figure 4.3.2-19. The rate of gel strength increase is controlled by the time from original mixing. Gel strength values should not be determined by extrapolation before a few days time has elapsed. The sharp curve in the graph would cause higher than true values to be calculated.

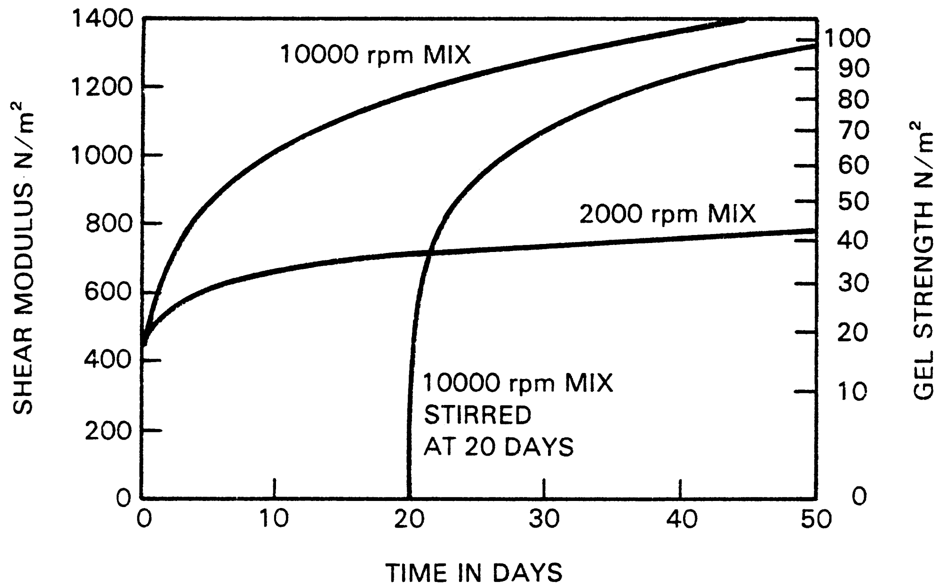

FIGURE 4.3.2-19. The Cell Strength and Shear Modulus of Slurry as a Function of Time (Source: Jefferis 1982)

There are several reasons why viscosity and gel strength increase over time. One reason is that delamination, the process of swelling and separation of the clay sheets that form the clay particles, continues with time. Water seeps between the clay layers and new smaller clay particles are formed. Viscosity and gel strength both increase as the quantity of these particles rises. Another reason may be that the clay particles change their orientation with respect to one another over time and form stronger interparticle bonds (Jefferis 1982).

Assuming that delamination and reorientation of the clay particles increases the gel strength and viscosity of a slurry, decreased particle size should lower the permeability of the slurry wall. Secondly, time and mixing will not necessarily decrease filtrate loss. If a decrease in filtrate loss is desired, a thicker filtercake can be make by adding dispersing agents to the slurry (Jefferis 1982).

Treatment of Slurry Problems

During construction quality control testing might show the need to alter the composition of the slurry. Table 4.3.2-5 lists problems encountered in the field and gives techniques used to solve them.

Water quality parameters that may prevent slurry problems include: pH of 7 (+ or - 1), hardness less than 500 ppm, and oil, organics, or other potentially harmful substances limited to 50 ppm each (Miller 1979). The groundwater chemical composition also has effects on slurries. The calcium content

TABLE 4.3.2-5. Summary of Slurry Problems and Treatment (Source: Harris et al. 1982b)

Problem	Control and Treatment
To increase viscosity and gel in fresh water	Add bentonite, CMC[(a)], or both
To reduce viscosity and gel when adequate colloid material present	Add water slowly or treat with thinners
To reduce viscosity and gel due to high noncolloid solid content	If solids are not completely dispersed, use mechanical separation; add water slowly and thinners
To reduce viscosity and gel when dilution is inadvisable because of inadequate colloid material or weight reduction	Add thinners; if viscosity drops appreciably and overtreatment occurs, adjust using CMC
To increase viscosity and gel due to high colloid solid content (sand)	Remove solids by mechanical separation; add bentonite or CMC
To decrease density	Recirculate fluid to remove solids by mechanical separation or by allowing them to settle; do not add water, but adjust flow properties if required after the density is decreased
To reduce filtration rate and thickness, i.e., reduce fluid loss	Add bentonite and CMC; if viscosity becomes too high, treat with FCL[(b)] or other thinners
To handle large volumes of entrained sand and cuttings	Use mechanical dispersion; avoid adding water and chemicals
To control salt flocculation in offshore drilling and excavation in salt formations	Stabilize solution through the protective action of CMC or use thinners
To permit trench excavation in sand and gravel (sand will increase density, decrease viscosity, and aggravate tendency toward lost circulation	Provide adequate initial gel strength to keep sand in suspension; build good filter cake and film to keep fluid loss low; use higher bentonite concentrations and add CMC

TABLE 4.3.2-5. (cont'd)

Problem	Control and Treatment
To permit trench excavation in clay	Keep viscosity and gel low; use thinnest suspension colloidally stable; use thinners
To permit trench excavation in shale	Reduce filtration rate to prevent hydrous disintegration or sloughing of formation; add bentonite and CMC; monitor slurry level to control sudden loss of fluid
To permit excavation in erratic formulations	Base selection of slurry on most critical formation; make periodic adjustments
To reduce lost circulation	Use lost-circulation materials; maintain minimum safe slurry weight
To control contamination with cement	Add FCL or other thinning agents; if restoration is not achieved, reject slurry; use pretreated bentonites
To control contamination with organic matter and sewage	Avoid peptized brands; use natural bentonite and monitor slurry closely

(a) Sodium carboxymethyl cellulose
(b) Ferrochrome lignosulfonate

of hard waters has a flocculating effect on the slurry. High concentrations of sodium and alkali salts will decrease the swelling of bentonite. Thorough testing of the slurry in the presence of the on-site ground-water chemistry should be conducted to guard against any decrease in swelling which could potentially increase the permeability of the wall.

Slurry Additives and Processes

The complication and cost involved in the use of slurry additives has restricted their use. Additives such as fluid-loss control agents, contamination resistant agents, and peptizers of polyelectrolytes to improve colloidal stability have been used in the past (Harris et al. 1982b). A more extensive list is outlined in Table 4.3.2-6.

Slurry wall contractors have developed a number of processed (and patented) slurry mixes. ASPEMIX® is a cold asphalt emulsion developed by Slurry Systems to withstand chemicals that bentonite-based slurry can not due to cation-exchange and clay degration. Slurry Systems is also presently developing a slurry having elastic properties[(a)].

The Environmental Products Division of the American Colloid Company has developed two chemical treatment processes that produce slurries that are more contaminant resitant. Saline Seal® is used to resist contamination in excess of 100,000 ppm TDS. Ultra Gel® is for use where high viscosity slurries are required. It contains peptized bentonite and restricts flocculation in strong ionic solutions[(b)]. The American Colloid Company has patented a process that increases the swell potential of sodium bentonite. In this process water soluble polymers disaggregate the clay particles and increase the clay surface area available for hydration. A contaminant resistant sodium montmorillonite was also developed. This slurry was initially made to resist environments of high salt concentrations. It was later discovered to hold up against other leachate constituents that decrease the swelling ability of the slurry by both cationic exchange and water of hydration crowding (Jepsen, undated).

Consistency

Slump Tests are used to determine the consistency and fluidity of the S-B backfill, the C-B slurry, and the concrete used in a L-C wall. A conical mold is filled with material and inverted on a flat surface. The drop in height below the mold height is measured and labeled the slump of the material[(b)].

The S-B backfill slides down into the trench at a slope determined by the slump of the backfill and the gradation of the material in the mixture. The higher the slump and more uniform the gradation, the flatter the slope will be. The lower the slump and the coarser the grade material, the steeper the

(a) Information from advertising brochure of Slurry Systems, a division of Thatcher Engineering Corporation, Gary, Indiana.
(b) Lapedes, Daniel N., ed. 1974. McGraw-Hill Dictionary of Scientific and Technical Term 2nd ed. McGraw-Hill Book Co. New York.

TABLE 4.3.2-6. Common Slurry Materials and Additives (Source: Harris et al. 1982b)

Purpose	Type
Weight materials	- Barite (barium sulfate) or soil (sand)
Colloid materials	- Bentonite (Wyoming, Fulbent, Aquagel, Algerian, Japanese, etc.), basic freshwater slurry constituent - Attapulgite, for saltwater slurries - Organic polymers and pretreated brands
Thinners and dispersing agents	- Quebracho, organic dispersant mixture (tannin) - Lignite, mineral lignin - Sodium tetraphosphate - Sodium humate (sodium humic acid) - Ferrochrome lignosulfonate (FCL) - Nitrophemin acid chloride - Calcium lignosulfonate - Reacted caustic, tannin (dry) - Reacted caustic, lignite (dry) - Sodium acid pyrophosphate - Sodium hexametaphosphate
Intermediate-sized particles	- Clay, silt, and sand
Flocculants and polyelectrolytes	- Sodium carboxymethyl cellulose (CMC) - Salts - Starches - Potassium aluminate - Aluminum chloride - Calcium
Fluid loss-control agents	- CMC or other flocculants - Pregelatinized starch - Sand in small proportions
Lost-circulation materials	- Graded fibrous or flake materials; shredded cellophane flakes, shredded tree bark, plant fibers, glass, rayon, graded mica, ground walnut shells, rubber tires, perlite, time-setting cement, and many others

slope. Slopes average between 5:1 and 10:1. With steeper slopes (low slump) there is a greater possibility of trapping unwanted materials such as sediment, partially excavated material, or fluid slurry in the wall. With flatter slopes (high slump) excavating problems may arise. The results of a slump test should range between 10 and 15 cm (4 and 6 in.) for S-B backfill (Millet and Perez 1981).

A slump range of 18 to 23 cm (7 to 9 in.) is appropriate for concrete that is to be tremied into a slurry trench to form a L-C wall. Too stiff of a mix may lead to voids and open honeycombs in the panels (Millet and Perez 1981).

Deformability and Strength

Slurry walls are non-structural; they are not built to support bending moments or significant shear stress. Concentrated loads put on top of the wall are mitigated by placing a cap over the completed wall. S-B walls are generally assumed to be infinately plastic for construction purposes (Ryan undated), although functionally they need to achieve the strength of the surrounding host material. The actual plasticity of an S-B slurry wall is a function of the amount of fines in the backfill. The strength is a function of internal friction which is controlled mainly by consolidation stress (D'Appolonia 1980).

A soil-bentonite wall will be sufficiently plastic and resistant to cracking if its slump is between 10 and 15 cm (4 and 6 in.), and there is reasonable gradation of coarse to fine material. A coarse gradation and a low slump makes for a rigid wall (Millet and Perez 1981).

The higher the cement-water ratio in a C-B slurry or a concrete mix (L-C method) the stiffer and less deformable the slurry wall will be. Strain at failure increases when the cement-water ratio is increased. Conversely, the higher the bentonite-water ratio, the more flexible the wall becomes (Millet and Perez 1981).

The brittleness and low deformability resulting from high cement concentrations can be detrimental to the performance of a slurry wall. A high cement concentration can be beneficial by protecting the wall from erosion by ground water and reducing seepage of slurry through permeable soils (Schmednecht undated). Processes combining the benefits of a high cement ratio and eliminating the brittleness associated with this type of mix are being developed. For example, Slurry Systems is developing a slurry wall construction method having elastic properties with an extended urethane base.

To specify the required structural strength of the concrete used to form a L-C wall, American Concrete Institute specifications can be used. Although all slurry walls should be designed as if they had zero strength (Harr et al. undated).

Vane Shear and Swedish Fall Cone measurements of shear strength of usual slurry mixes used for VBT slurry walls produced shear strength values of about 171 kg/sq. meter (35 psf) within a week of preparation. The strength of the slurry was found to double after a month. Very high strengths on the order of

4882 kg/sq. meter (1000 psf) can be made by doubling the usual cement content. The increased strength is gained at the expense of permeability, which rises significantly (Harr et al. undated).

Mixing C-B Slurry

There are essentially three different ways (Jefferis 1982) to mix cement, bentonite, and water to form a material that is suitably impervious to ground water. In the dry mixing method bentonite and cement are mixed together and then hydrated in a mixer. The bentonite particles do not disperse because of reactions between the cement and water, so a low water to cement ratio is obtained. With this type of slurry mix bentonite is not present for its swelling capabilities. It is added to enhance the flow and cohesiveness of the slurry. This mix achieves high strength upon hardening, and it is used mainly for injection grouts.

In another mixing method the bentonite alone is hydrated and then the cement is added. The mix stiffens suddenly when the cement is first added and then becomes fluid again shortly after. This is due to negatively charged bentonite particles flocculating with positively charged particles in the solution. When the bentonite particles surround the cement particles, the floc structure breaks and the slurry becomes more fluid. The slurry becomes more viscous as calcium and other ions are released by the cement and flocculate the bentonite.

Finally, a slurry can be made by wetting the cement first. Cement grains often bunch together and become encapsulated by the bentonite when added to the slurry. By wetting the cement before the addition of bentonite, a more homogeneous mix is produced. These cement nodules can also be broken apart by high shear mixing or by mixing for longer periods of time.

Cement nodules incorporated into a slurry wall act as high stress points which decrease its strength and act as pervious material which increases its permeability. A low shear mixing rate produces a less homogeneous mix that tends to be more weak and plastic. High shear mixing, on the other hand, produces a homogeneous mix that is stronger, more brittle, and less pervious (Jefferis 1982).

When very low permeability is a quality control criteria, pre-wet cement and high shear mixing can prove to be effective. For high strain capabilities (and less expense), dry cement and low shear mixing are the best choices (Jefferis 1982).

Filtrate loss of slurry through the trench sidewalls increases markedly with the addition of cement (Ryan undated). Adding fine material such as ground granulated blast furnace slag and fly ash to C-B slurries can control bleeding (Jefferis 1982). Unlike cement they do not react with bentonite by releasing ions into solution which causes the bentonite gel to bleed. They work by making the slurry more viscous and less likely to seep through the sidewalls (Figure 4.3.2-20).

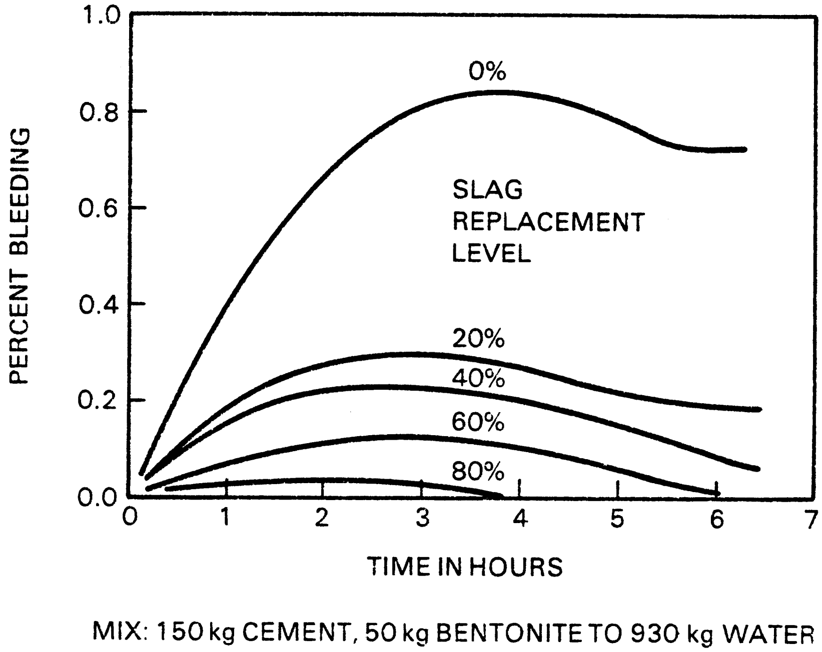

FIGURE 4.3.2-20. The Effect of Slag Replacement on Bleeding (Source: Jefferis 1982)

Up to 95% of the cement can be replaced by slag. A slurry with this concentration of slag will be very fluid and easy to mix. Consequently, the mix will be homogeneous and will form into a strong, brittle, and highly impervious barrier. Fly ash produces a less brittle wall than slag. No more than 50% of the cement can be replaced by fly ash or the slurry takes too long to set (Jefferis 1982).

4.3.2.5 Contaminant Compatibilty of Slurry Trenches

Cut-off walls used to contain contaminated ground water following a severe power plant accident must be designed so as not to increase in permeability when in contact with the contaminated ground water. There are two ways (D'Appolonia undated) in which a contaminated liquid can increase the permeability of a slurry wall.

First, the soil minerals in the wall may be soluble in the permeant. A loss of solids in the wall increases its pore volume allowing more liquid to pass. Second, pore fluid substitution may reduce the double layer of partially bound water surrounding the hydrated bentonite or other clay particles in the wall. This would lower the effective size of the clay particles that clog the pore space between the soil grains, again allowing more room for liquid to pass through the barrier. Organics adsorped onto the bentonite surface decrease the swelling potential of the slurry by lowering the area available for water to react (Harris et al. 1982b).

The sodium ions in bentonite clay are easily and rapidly replaced by multivalent cations (e.g., calcium, magnesium, and heavy metals) transported in the ground water. This leads to the smaller double layer described above and decreases the swelling of the hydrated bentonite. When sodium ions of bentonite exchange with an equivalent number of ions in the contaminant, steady state conditions prevail, and the permeability reaches a constant but higher level (D'Appolonia undated). The ion exchange capacity may offset the effects of increased permeability through adsorption of the radionuclide onto the clay minerals.

The magnitude of the permeability increase of a slurry wall depends on the difference in chemistry between the initial and final pore fluid and the sensitivity of the clay to the pore fluid chemistry change. Sodium bentonite is the most sensitive of the montmorillonite clays. Alluvial or lucustrine clays are less sensitive and undergo less change when leached with a contaminant. The particular type of sodium bentonite does not seem to change the permeability level increase due to leaching with contaminants (D'Appolonia undated).

Backfill that is contaminated with the leachate will increase in permeability less than if uncontaminated soil were used. Therefore, contaminated soil excavated from the trench can be used to increase the durability of the slurry wall (Millet and Perez 1981).

Table 4.3.2-7 shows the effects of various pollutants on a soil-bentonite slurry wall.

4.3.2.6 Implementation Considerations for Slurry Trenches

Cost

S-B and C-B walls range from about $43 to $75/sq. meter ($4 to $7/sq. ft) of vertical cut-off wall (Warner 1979). The price differs somewhat between contractors and increases with soil depth. The usual trend is that S-B walls are the least expensive, followed closely by C-B walls, and L-C walls being the most expensive. The ICOS Corporation of America prices S-B walls at $32 to $65/sq. meter ($3 to 6/sq. ft), C-B walls at $54 to $108/sq. meter ($5 to 10/sq. ft), and L-C walls at $161 to $323/sq. meter ($15 to $30/sq. ft) (Harris et al. 1982b).

The cost of a C-B wall depends largely on the type of soil available for backfill. Cost increases with the transportation costs of soils brought to the site. Much slurry is wasted during excavation when constructing a C-B wall. The entire trench must be filled with slurry, and much of it leaks through the sides of the trench. If on-site soil can be used in the backfill of a S-B wall, holding all other variables constant, it would be less expensive than the C-B method (Harris et al. 1982b).

The cost of a slurry wall increases with the difficulty of excavating the trench. The maximum depth penetrated by a S-B wall at a site in Nashua, New Hampshire is 33 meters (108 ft). The estimated cost through overburden was

TABLE 4.3.2-7. S-B Permeability Increase Due to Leaching with Various Pollutants (Source: D'Appolonia 1980)

Pollutant	S-B Backfill (silty or clayey sand) 30 to 40% fines
CA^{++} or MG^{++} @ 1000 PPM	N
CA^{++} or MG^{++} @ 10,000 PPM	M
NH_4NO_3 @ 10,000 PPM	M
Acid (pH>1)	N
Strong Acid (pH<1)	M/H*
Base (pH<11)	N/M
Strong Base (pH>11)	M/H*
Benzene	N
Phenol Solution	N
Sea Water	N/M
Brine (SG = 1.2)	M
Acid Mine Drainage ($FeSO_4$ pH≈3)	N
Lignin (in Ca^{++} solution)	N
Organic residues from pesticide manufacture	N
Alcohol	M/H

N - No significant effect; permeability increase by about a factor of 2 or less at steady state.

M - Moderate effect; permeability increase by factor of 2 to 5 at steady state.

H - Permeability increase by factor of 5 to 10.

* - Significant dissolution likely.

$50/sq. meter (4.50/sq. ft) and increased to $1500/sq. meter ($150/sq. ft) through bedrock (Ayres et al. 1983).

Spencer, White, and Prentis, Inc. claims that the VBT method is inherently quicker, therefore there is economy of labor and equipment. VBT walls are also thin which keeps material costs low[a].

Construction Time

Excavation time is a function of equipment used, soil conditions, and the depth of excavation. A C-B wall takes either the same amount of time or less to construct than a S-B wall, and a L-C wall takes the most amount of time to complete. The ICOS Corporation of America approximates S-B and C-B construction time at 280 sq. meters/day; and for L-C walls, 30 to 70 sq. meters/day. Similar quotes are given by Geo-Con Inc. with average construction times, for S-B and C-B walls, between 280 and 370 sq. meters/day (Harris et al. 1982b).

Construction time varies with depth. Engineered Construction International, Inc. estimates an average time of 370 sq. meters/day for a 12 meter depth. Raymond International, Co. estimates an average time of 140 sq. meters/day for a 9 meter depth. These figures are for 8 hour shifts. Construction time could be decreased by employing 3 eight hour shifts per day (Harris et al. 1982b).

Time could also be saved by using the proper equipment. Clamshell excavation takes 2 to 3 times longer than with a backhoe. A backhoe is faster than either a dragline or a clamshell. Also the type of mixer used can greatly hasten operations. For example, no time is needed for bentonite hydration if an 1800 rpm mixer is used (Harris et al. 1982b).

The Timing of Several Case Studies

At a site in Nashua, New Hampshire, the excavation, transport, and compaction of 190,000 cubic meters (250,000 cu. yds) of soil, and backfilling of the trench, were completed in 8 weeks. The cap or cover system was then placed over the S-B slurry wall bringing the total construction time to 3 months (Ayres et al. 1983).

A C-B wall encompasses the Tilden Tailings Disposal System in Gribben Basin, Michigan. The wall was built through soil strata at about 1200 sq. meters/day. When bedrock was reached, the production rate dropped to less than 750 sq. meters/day. The maximum depth was 25 meters (80 ft) (Harris et al. 1982b).

An un-reinforced lean concrete cut-off wall was used for dewatering at Bonneville Dam in Washington. Excavation averaged 16 sq. meters/day (170 sq. ft/day) up to a depth of 46 meters (150 ft) (Harris et al. 1982b).

(a) Information from advertising brochure of Spencer, White, and Prentis, Inc.

In Warsaw, Missouri, the VBT method was used to construct a slurry wall for excavation dewatering. In 40 working days 9290 sq. meters (100,000 sq. ft) of slurry wall was placed to a depth of 14 meters (45 ft) (Harr et al. undated).

Equipment Mobilization

Following a severe power plant accident, atmospheric releases may limit work near the site. Additional time may be needed to mobilize special equipment to be used under these conditions (Harris et al. 1982b). Construction workers must be informed of and protected from radiation exposure, and difficulty may be experienced in organizing work crews.

Bentonite Supply

Sufficient quantities of bentonite would be available for slurry wall construction in the event of an accident. Although bentonite may have to be acquired from several suppliers in order to obtain a sufficient amount on short notice. Quality control would be made more difficult by mixing different types of bentonite.

Site Restrictions

Aboveground obstacles such as man-made structures or vegetation must be removed where the wall is to be constructed. Underground obstacles such as subsurface piping (e.g., utility service) must be located to insure avoidance.

The amount of space available for construction at the site should be considered. The choice of a S-B wall may be eliminated if the site is small. S-B walls are relatively wide and space is needed next to the trench to mix the backfill, unless it is specially prepared elsewhere and transported to the trench (adding expense to the project).

For locations where ground-water levels are high, a difference in hydrostatic pressure in and outside the trench can be induced by pumping. In order for the filtercake layer to form, the hydrostatic pressure in the trench must be greater than the external hydrostatic pressures. For this condition to exist, the slurry level in the trench must be about 1.3 meters (4 ft) above the ground-water level. A berm built along the trench alignment can be used to raise the ground level and increase the amount of slurry above the ground-water level (Harris et al. 1982b).

Near coastal regions, the tide may increase the hydrostatic pressure on one side of the trench. In this case hydrostatic pressure of the slurry on the trench sidewalls can be increased by either increasing the height of the berm or the density of the slurry (Harris et al. 1982b).

Weather Constraints

Rain or snow can stop slurry wall construction temporarily. Although, the mixer used in the construction of a C-B wall makes it easier to continue construction during the rain. A mixing plant could be used during S-B wall construction to mitigate changes in the bentonite-water ratio due to rain water infiltration (Harris et al. 1982b).

Freezing temperatures can halt slurry wall construction. The delivery of materials to the site may be hampered by poor road conditions. Slurry freezes at about -1°C (28°F) and regains its original properties when thawed. Slurry walls may fracture at the 1.2 meter frost line after installation (Harris et al. 1982b).

Environmental Effects

The slurry has minor impacts on the environment. The migration of slurry into the soil surrounding the trench is relatively low. It is not probable that the small amounts of chemical additives sometimes used in slurries will leach from the wall into the local ground water (Harris et al. 1982b).

The greatest environmental effect will be on the local ground-water flow. An increase in the up-gradient hydraulic head, due to ground-water flow impeded by the slurry wall, can have effects on the rate of vertically moving water. The local water table may rise creating a "bathtub effect" (EPA 1982). Areal ground water modeling can be used to analyze the effect of a slurry wall on ground-water flow and changes in water table elevations (Miller 1979).

4.3.3 Steel Sheet Piling

Sheet piling, along with grout curtains and slurry trenches, can be used to form static barriers to ground-water movement. Various materials can be used to construct sheet piles (e.g., wood, precast concrete, and steel) but steel is the most effective and widely used as a ground-water cut-off. A steel sheet piling ground-water barrier consists of interlocking steel piles driven into the ground via a pile driver. The piles are typically driven from ground surface or from pre-dug trenches (EPA 1982).

4.3.3.1 Sheet Piling Design Considerations

Steel sheet pilings are typically hot rolled steel sections 1.25 m to 12.2 m in length (i.e., vertical) and 0.4 m to 0.8 m in width (i.e., horizontal direction of cut-off wall). The shapes of individual piling sections are higly varied and manufacturer dependent. Many manufacturers have also developed their own piling interlocking designs and all manufacturers of steel sheet piling make special corner sections and "T" connections (EPA 1982).

The effectiveness of steel sheet piling as a ground-water barrier is a function of the integrity of the interlocking system (Harris et al. 1982a). The cut-off effectiveness can be lost if sections of the wall become unlocked allowing seepage through the resulting gap. If out of interlock, an individual sheet can stray more than a meter out of position without detection.

Steel sheet piling is applicable only to unconsolidated host materials, except in very rare situations where the piles may be hard driven through consolidated material. Steel sheet piling is usually not even considered suitable for use in very rocky soils because of the difficulty in driving the piles through cobbles and boulders and the resulting damage to the piles themselves (EPA 1982).

Steel sheeting is most effective as a ground-water flow barrier when anchored (i.e., driven into) in a low permeability bed of firm clay (Harris et al. 1982a). When first placed in the ground the permeability of steel sheet piling cut-offs is quite high even with an impermeable key because the interlocks, which are loose to facilitate placement, allow significant seepage. However, as time passes, the permeability is reduced due to the siltation of fine soil particles in the interlock seams. The rate of sealing depends of ground-water flow rates and the adjacent soil properties (EPA 1982).

The three most important design considerations for steel sheet piling cut-offs are the (Harris et al. 1982a):

1. Interlock,
2. Shape of pile cross-section, and
3. Material.

Examples of various types of interlocking systems are presented in Figure 4.3.3-1. The pilings are assembled before being driven to facilitate a positive lock. Good interlocks are relatively soil tight, however none are completely water tight.

The cross-sectional shape of pilings is designed to facilitate resistance to bending of the resultant wall. In addition to bending strength the pilings must be suitable for driving into soil to appropriate depths. The shapes of cross-sections are also designed to provide required stiffness (Harris et al. 1982a). Figure 4.3.3-2 shows examples of typical sheet piling cross-sectional shapes. Generally, steel sheet piling shapes are divided into two main types: 1) U-type sections, and 2) Z-type sections. The U-type sections are commonly referred to as arch web types (see Figure 4.3.3-2). Steel sheet piling typically weighs between 73 kg/m^2 and 78 kg/m^2 of wall area with 0.6 cm to 2.25 cm of thickness (Harris et al. 1982a).

For purposes of mitigating ground-water contamination from severe power plant accidents only steel sheet piling is being considered. Pilings made of other materials are not as effective in controlling ground-water seepage (EPA 1982; Harris et al. 1982a; Lee 1949). Steel can withstand the force of driving during installation and thus can be used in more resistant strata than other materials; timber in particular). The steel piling design specifications, particularly the amount of steel in the cross-section and the quality of the steel, are determined primarily by the soil resistance to be overcome in driving the pilings (Harris et al. 1982a).

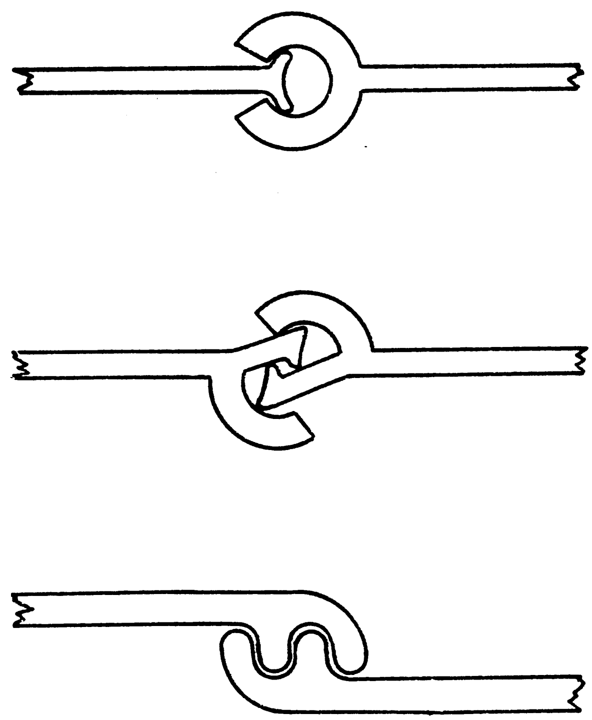

FIGURE 4.3.3-1. Steel Sheet Piling Interlock Designs (Adapted from: Merriman and Wiggin 1947)

4.3.3.2 Sheet Piling Construction Considerations

Sheet piling is forced into place in pairs, with a pneumatic or steam driven pile driver or a drop hammer. Steel sheet piles usually drive outward and tend to creep in the direction in which the cut-off wall is being driven. In order to prevent outward movement of the pilings they are pitched and driven to part penetration. The piles are driven a meter or so at a time over the entire length of the wall until all the piles have been driven to the desired depth (Figure 4.3.3-3) (EPA 1982, Lee 1949).

For driving steel sheeting in cohesive soils the recommended ratio of the weight of the hammer to the weight of the sheeting being driven is 2.0. For less cohesive, granular, soils a double acting steam hammer is recommended because the rapidity of hammering results in vibration of the subsoil which greatly facilitates penetration (Lee 1949).

Heavy equipment is usually preferable to lighter weight equipment for faster driving and prevention of damage to the piles. Often a cap block or driving head is used to prevent damage to the top edge of the sheeting (EPA 1982). Hammering should be temporarily suspended when an obstruction or sudden resistance is encountered in order to save damaging the toe of the piles and

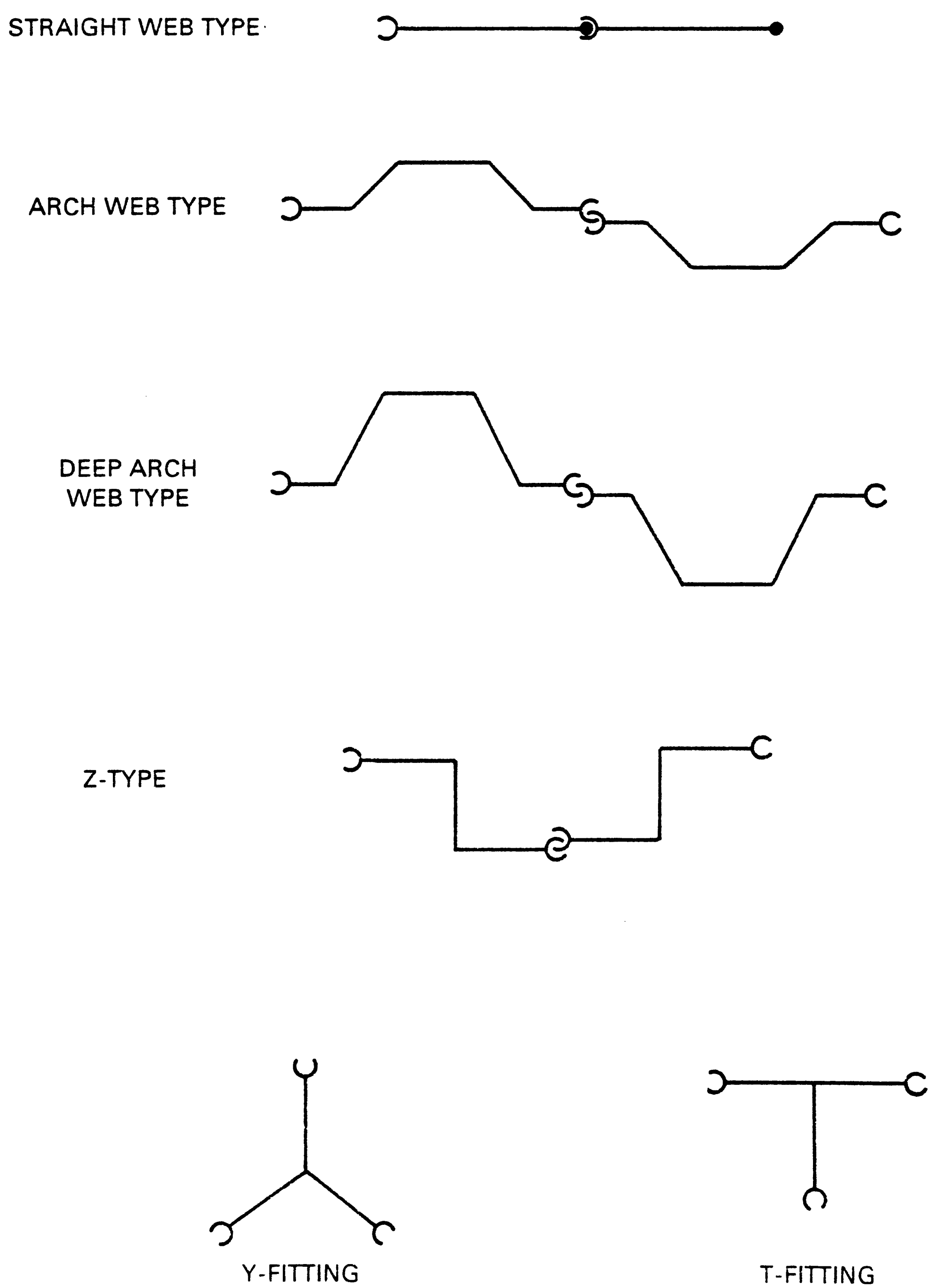

FIGURE 4.3.3-2. Typical Steel Sheet Piling Shapes
(Source: EPA 1982)

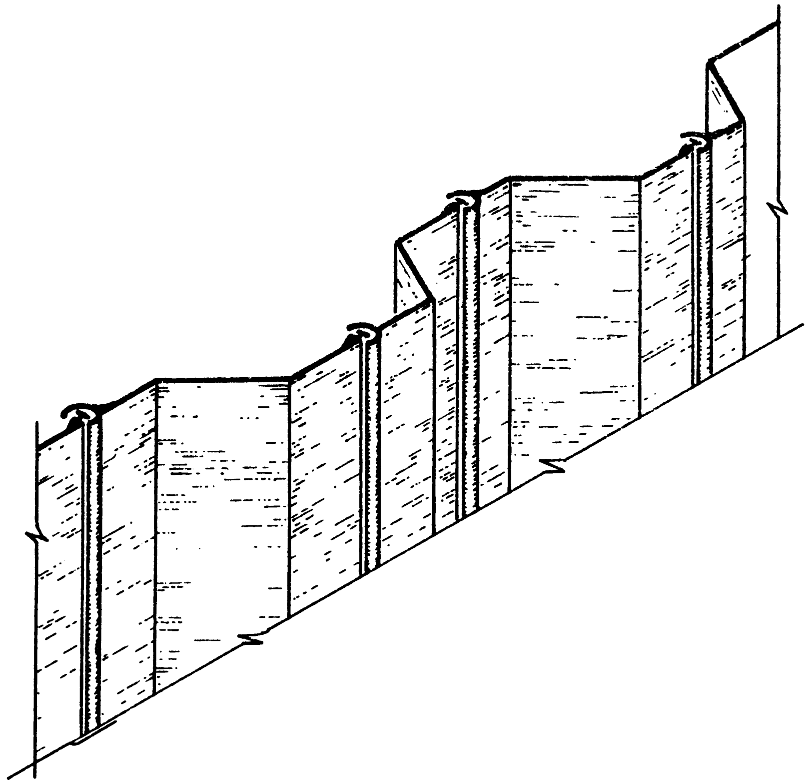

FIGURE 4.3.3-3. In-Place Steel Sheet Piling Cut-Off Wall
(Source: Miller 1979)

possibly opening the interlock. Hammer bounce indicates when an obstruction or stiff resistance has been encountered. If only stiff resistance is considered the cause of the bounce a heavier hammer may be required for continued penetration. Conversely, if an obstacle has been met hammering of adjacent piles may free the snagged sheet (Lee 1949).

A pile frame is often used with a pile hammer since the frame helps alignment of the sheets. When steel sheeting is driven without a pile frame timber walings are used to support the sheeting during installation (Harris et al. 1982a; Lee 1949). Some types of steel sheet piling are delivered interlocked in pairs offering a significant time savings. Also, for installation in stiff cohesive soil the bottom edge of the sheets are occasionally reinforced by bolting or welding steel strips thereby reducing the skin friction higher on the pile (Lee 1949).

The play in most interlock systems allows for a significant degree of curvature in the constructed cut-off wall. However, the strength of the interlock and its watertightness are inversely related to the swing properties of the interlock (Lee 1949).

4.3.3.3 Performance Considerations for Steel Sheet Pilings

Performance considerations for steel sheet piling ground-water barriers consist primarily of the ability to control ground-water seepage, the durability of the barrier, and the methods of failure.

Ground-Water Seepage Control

According to Miller (1979) several studies of the effectiveness of sheet piling to control ground-water seepage, including instrumented studies, have shown that the seepage cut-off efficiency of sheet piling is low. The principal reason for the low efficiency is inadequate or improperly sealed interlocks. Two methods of sealing the interlocks have been used with varying degrees of success. One method consists of coating (prior to driving) the interlocks with bentonite. The clay then swells when exposed to water after the piles have been installed. However, the bentonite tends to scrape off during installation thus reducing the overall effectiveness of the coating in controlling seepage.

A second method for sealing sheet piling interlock systems involves grouting. Grouting of interlocks can be performed two ways. One way consists of grouting after installation through tubes welded to the pile interlock area installed before driving. The second approach requires driving or jetting the injection pipe along the interlock of the in-place pile to the bottom of the pile and injecting grout as the pipe is withdrawn. The grouting methods are costly and success has been questionable (Miller 1979).

Another reason for the possible reduction in performance of sheet piling as a ground-water cut-off is broken interlocks caused by hard or improper driving. Related to this problem is also the concern over adequate sealing of the piles in an impervious key-in strata or foundation. Both of these problems are difficult to detect for in-place pilings (Miller 1979). If the pilings have to be driven hard to obtain sufficient penetration in an impervious layer the sheets may buckle causing damage to the interlock. Because of the damaged interlock the succeeding sheet may be forced out of the lock causing potentially significant leakage through the gap (Lee 1949).

Where effective seals (i.e., little or no leakage) can be maintained both for interlocks and the key-in, sheet piling provides an essentially continuous impermeable barrier. In practice sheet piling cut-off walls have been constructed in soils ranging from well-drained sand to impervious clay (EPA 1982).

Steel Sheet Piling Durability

Steel corrodes under typical ground conditions at a maximum rate of 2 to 5 mils/year for the first few years and then the rate declines (Miller 1979). Steel also corrodes faster in a sea-water environment than a fresh-water environment (Lee 1949). Depending on the conditions of the soil and the ground-water chemistry the performance life of a steel sheet piling wall may be between 7 and 40 years (EPA 1982).

Typically, a sheet piling cut-off wall is in contact with three subsurface environments (Miller 1979):

1. Relatively dry soils above the water table,

2. Alternatively wet and dry soils in the transition zone from the vadose zone to the saturated zone, and

3. Continuously wet soil in the saturated zone below the water table.

The transition environment (#2 above) is the most corrosive and could potentially have a corrosion rate in excess of 10 mils/year (Miller 1979). Some measure of protection against corrosion can be obtained by using hot-dip galvanized or polymer-coated steel pilings. Cathodic protection has also been proposed for corrosion protection of submerged pilings (EPA 1982). However, corrosion protection will not indefinately extend the life of a steel sheet piling cut-off wall.

Because of the relatively short life of a sheet piling cut-off compared to grouts and slurry trenches this ground-water interdiction techniques should most probably be considered as a temporary or short-term corrective measure while more long-term or permanent solutions are studied and/or implemented.

Causes of Sheet Piling Wall Failure

Failure of steel sheet piling walls can usually be attributed to: 1) insufficient penetration of the piling toe causing the wall to tilt and then possibly slide forward, and 2) ineffective anchorage also resulting in tilting of the wall and subsequent sliding forward. Ineffective anchorage may be in the form of inadequate or improper bolting of walings to pilings, the walings themselves, the tie rod and end fixings, or the anchors themselves (Harris et al. 1982a; Lee 1949). Also anchorages may fail because they are placed too near the wall. Failure of sheet piling walls by overstressing the sheeting in bending are rare (Lee 1949).

4.3.3.4 Steel Sheet Piling Implementation Considerations

There are certain advantages and disadvantages of steel sheet piling walls in comparison with other engineered (i.e., constructed) barriers. The advantages of steel sheet piling ground-water cut-offs are (EPA 1982):

1. Materials and construction expertise are readily available,
2. Relatively easy to install,
3. Relatively inexpensive, and
4. Low maintenance requirements.

There are major disadvantages using steel sheet piling include (EPA 1982):

1. Cannot be used in consolidated medium,

2. Cannot be used effectively in rocky unconsolidated medium,

3. Limitations on the maximum depth of installation,

4. Initially not waterproof and without secondary sealing of joints may not achieve required levels of watertightness, and

5. Relatively short-lived.

Considerations for implementing or installing a steel sheet piling cut-off wall are: 1) installation time, 2) cost, 3) equipment mobilization, and 4) differential water levels, and 5) worker safety.

Installation Time

As with grout curtains and slurry trenches the time required to install a steel sheet piling cut-off is both a function of the specifications of the wall and the location of construction. Consequently, the construction time is highly job-specific. In general, the installation time per lineal meter of wall is greater for sheet piling than a similar depth slurry wall and less time per lineal meter than for a grouted curtain cut-off assuming no grouting of the sheet piling interlocks.

Cost

As is the case with construction time, installed sheet piling costs are job specific. Unit steel pricing can be used to estimate the materials cost, however the cost of driving the piles depends on the size, length, and type of section used, the nature of the soil, the amount of piling used, local labor conditions, and the method of driving. The EPA (1982) suggests guidelines to estimate the unit cost of a steel sheet piling wall. Table 4.3.3-1 contains estimates for materials and installation.

Once the total area of the wall to be constructed has been determined an adjustment factor of 1.6 is normally used to account for the area of the interlocking device. The adjusted area (i.e., the required area multiplied by 1.6)

TABLE 4.3.3-1. Unit Costs of Steel Sheet Piling (Source: EPA 1982)

Commodity	Cost/Unit
Black Steel	\$1,139/metric ton (assumed 1980 dollars)
Hot-dipped Galvanized Steel	\$1,296/metric ton (assumed 1980 dollars)
Installation	\$231/metric ton (assumed 1979 dollars)

can then be multiplied by the weight per area of the type of steel to be used which results in the total weight of the wall. The unit costs in Table 4.3.3-1 can then be used to estimate the total cost of the wall (EPA 1982).

There are several manufacturers of steel sheet piling in the U.S. as well as in Japan, West Germany; France; Britain; Luxenborg; the USSR and some eastern European countries (Harris et al. 1982a). U.S manufacturers of steel sheet piling include Bethlehem, U.S. Steel, and Jones and Laughlin Company.

Equipment Mobilization

The majority of the heavy equipment necessary to install a steel sheet piling cut-off consists of the pile driving apparatus. A derrick or a pile frame is sometimes used to support the hammer and, in the case of the frame, to align the pilings. A drop hammer is preferred for clay or marl and a double-acting hammer for non-cohesive soils (Lee 1949). Cranes are also used to suspend leaders and raise and lower hammers. Sufficient clearance must be provided to maneuver the crane or pile frame along the course of the wall; and because the sheeting is typically delivered to the job site via flatbed tractor-trailer rigs there is also a limit to the one piece length of sheeting that can practically be delivered.

Differential Water Levels

Because the sheet piling acts as a barrier and redirects the ground-water flow there is potential for different water levels on each side of the wall, hence differential hydrostatic pressure on the sheet piling wall (Harris et al. 1982a). Where this pressure gradient exists across the wall the potential for seepage under the wall increases. If seepage occurs (due to inadequate key-in of the toe of the piling) the seepage flow will increase the effective unit weight of the soil on the up-gradient side of the wall and decrease the effective unit weight of the soil on the down-gradient side of the wall. The likelihood of increased pressure differentials due to differential water levels across the wall should be factored into the design of the wall. If such differentials are expected to be high, as might be the case with a heavy rainstorm over a local up-gradient recharge area which could cause a rapid rise in the water table, the design of the wall and anchorage system should accomodate the additional load (Harris et al. 1982a).

Worker Safety

The same worker safety issues arising for grout curtain construction and slurry trench development apply to steel sheet piling placement. However, steel sheet piling does not require opening of a trench as is the case with slurry wall construction. Thus the potential radiation exposure associated with trenching below the water table is avoided by sheet piling.

4.4 ANALYSIS OF DYNAMIC GROUND-WATER CONTAMINANT MITIGATION STRATEGIES

Dynamic and quasi-dynamic mitigation alternatives influence the state of the contaminated ground-water in an active manner. These alternatives are better characterized as "mitigation strategies" rather than "interdictive techniques" which refers more to passive, engineered/constructed barriers. Dynamic mitigation strategies offer a wide range of potentially feasible (depending on the accident scenario and geohydrologic conditions of the particular site) approaches to containing, diverting, and/or treating ground-water contamination.

Dynamic strategies tend to be energy intensive and require some level of maintenance as opposed to the static barrier techniques (e.g., slurry trenches). For this reason, and others discussed under each strategy, dynamic strategies tend to be temporary corrective actions. However, even though most of these strategies are temporary they are not necessarily short-lived. Also, the dynamic strategies are more conceptual in design than the constructed barriers and often a complete mitigative course-of-action will comprise several of the individual strategies presented herein.

The dynamic mitigation methods are primarily concerned with active means to manipulate the ground-water flow regime in order to intercept the contaminant plume to remove it, treat it in-place, remove and treat it, or divert it. Many of the methods are only feasible in shallow aquifers in unconsolidated media. Other techniques are theoretically feasible in any geologic setting. However, certain practicalities prohibit application in some circumstances. The underlying philosophy of the dynamic mitigation schemes is one of concentration reduction to acceptable levels, not total contaminant removal. Several of the strategies require handling of the contaminated ground water. Safety issues are important since the potential radionuclide concentrations are substantial.

Many of the dynamic mitigation strategies are combinations of pumping and injection or re-injection schemes designed to lower the water table or through gradient control contain the contaminant plume. Two barrier construction techniques are also included in the dynamic strategies category because their maintenance is energy intensive. Table 4.4-1 contains a list of the dynamic

TABLE 4.4-1. Dynamic Mitigation Strategies

1. Ground-water withdrawal for potentiomnetric surface adjustment.

2. Ground-water withdrawal and/or re-injection for contaminant plume control.

3. Subsurface collection with recovery drains.

4. Selective filtration via permeable treatment beds.

5. Ground freezing.

6. Air injection.

mitigation strategies considered for mitigation of ground-water contamination resulting from a severe power plant accident.

4.4.1 Ground-Water Withdrawal for Potentiometric Surface Adjustment (Aquifer Dewatering)

Ground-water withdrawal to lower the water table in a predefined region can be an effective means of mitigating ground-water contamination resulting from a severe power plant accident. Lowering the water table via ground-water pumping can produce three consequences that are favorable in reducing the concentration of radionuclides in ground-water or their flux to an accessible environment. The three consequences of ground-water withdrawal as applied to ground-water contaminant mitigation are (EPA 1982):

1. Lowering the water table sufficiently to prevent the contaminated ground-water from discharging to a receiving stream that is in hydraulic contact with the unconfined aquifer,

2. Lowering the water table so that it is not in direct contact with the solidified core mass, and

3. Lowering the water table to preclude leaky aquifers from contaminating other aquifers.

Figures 4.4.1-1, 4.4.1-2 and 4.4.1-3 pictorially represent these three schemes.

Figure 4.4.1-1a shows the possible pre-drawdown condition of contaminated ground water being discharged to a down-gradient surface stream. Pumping can be implemented to create a cone of depression that extends to the stream-aquifer interface and, if sufficient, reduces or eliminates ground-water discharge to the stream (Figure 4.4.1-1b). Figure 4.4.1-2a shows a possible situation wherein the containment basemat is below the water table elevation. A breach of the basemat would allow direct contact of the sumpwater with the saturated ground-water flow system thus allowing immediate down-gradient transport of the radionuclide concentration in the sumpwater. However, with ground-water dewatering below the basemat (Figure 4.4.1-2b) a partially-saturated zone is created between the basemat and the water table. Radionuclide transport in this partially-saturated region would be slowed and oriented vertically downward especially in the case of a leaching solidified core mass as opposed to a sumpwater release. The third application of ground-water pumping to mitigate the effect of a severe power plant accident is shown in Figure 4.4.1-3. A situation might exist where the aquifer in which ground-water contamination arises is in leaky contact with another uncontaminated aquifer. Creation of a drawdown area in the overlying aquifer (Figure 4.4.1-3b) near the contaminant source may reduce or preclude contamination of previously uncontaminated aquifers.

The first two applications of ground-water pumping to lower water table elevations are best suited for shallow unconfined aquifers. However, piezometric heads in a confined aquifer can also be lowered by pumping until water

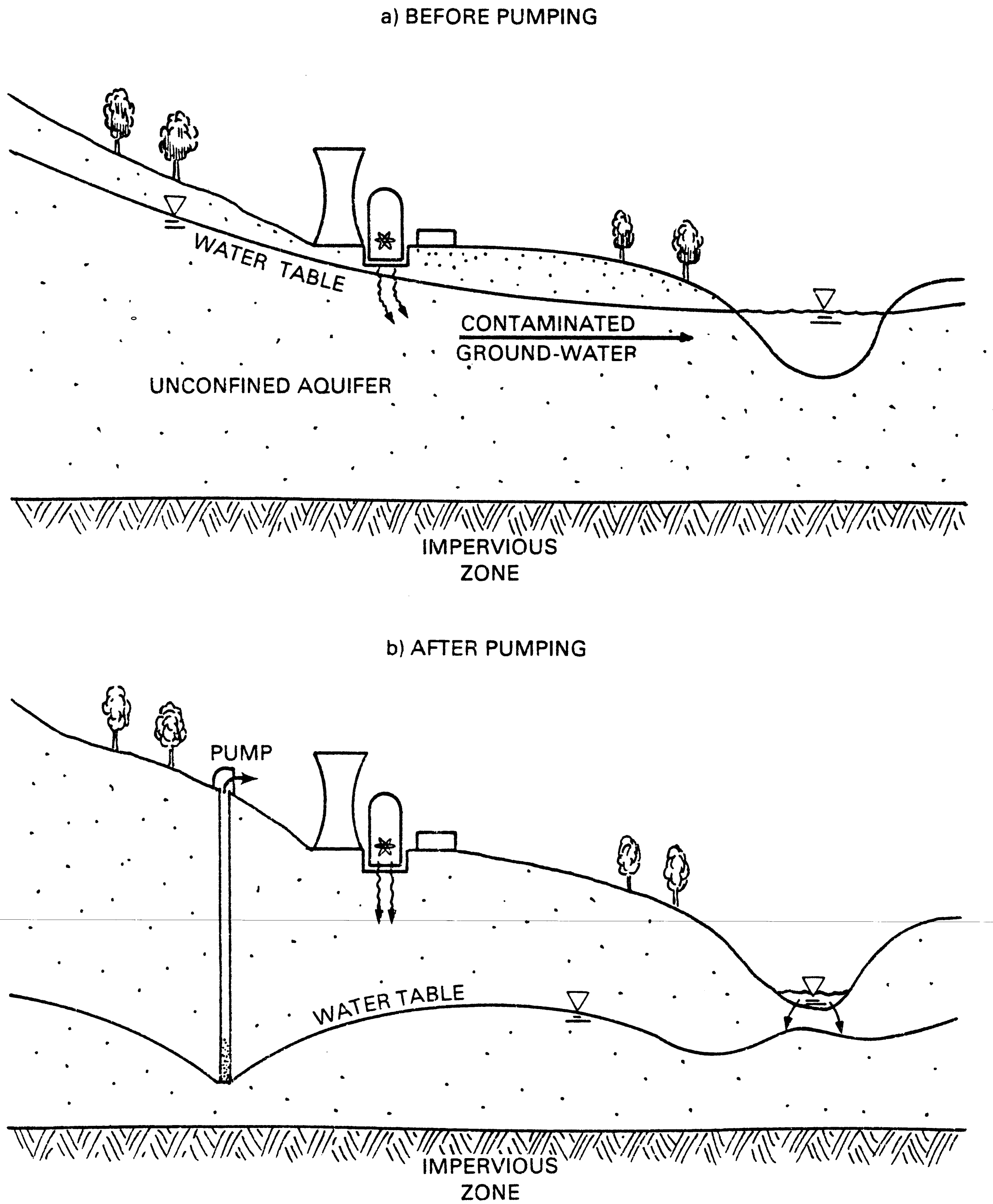

FIGURE 4.4.1-1. Pictorial Representation of Pumping to Lower Water Table Below Receiving Stream (After: EPA 1982).

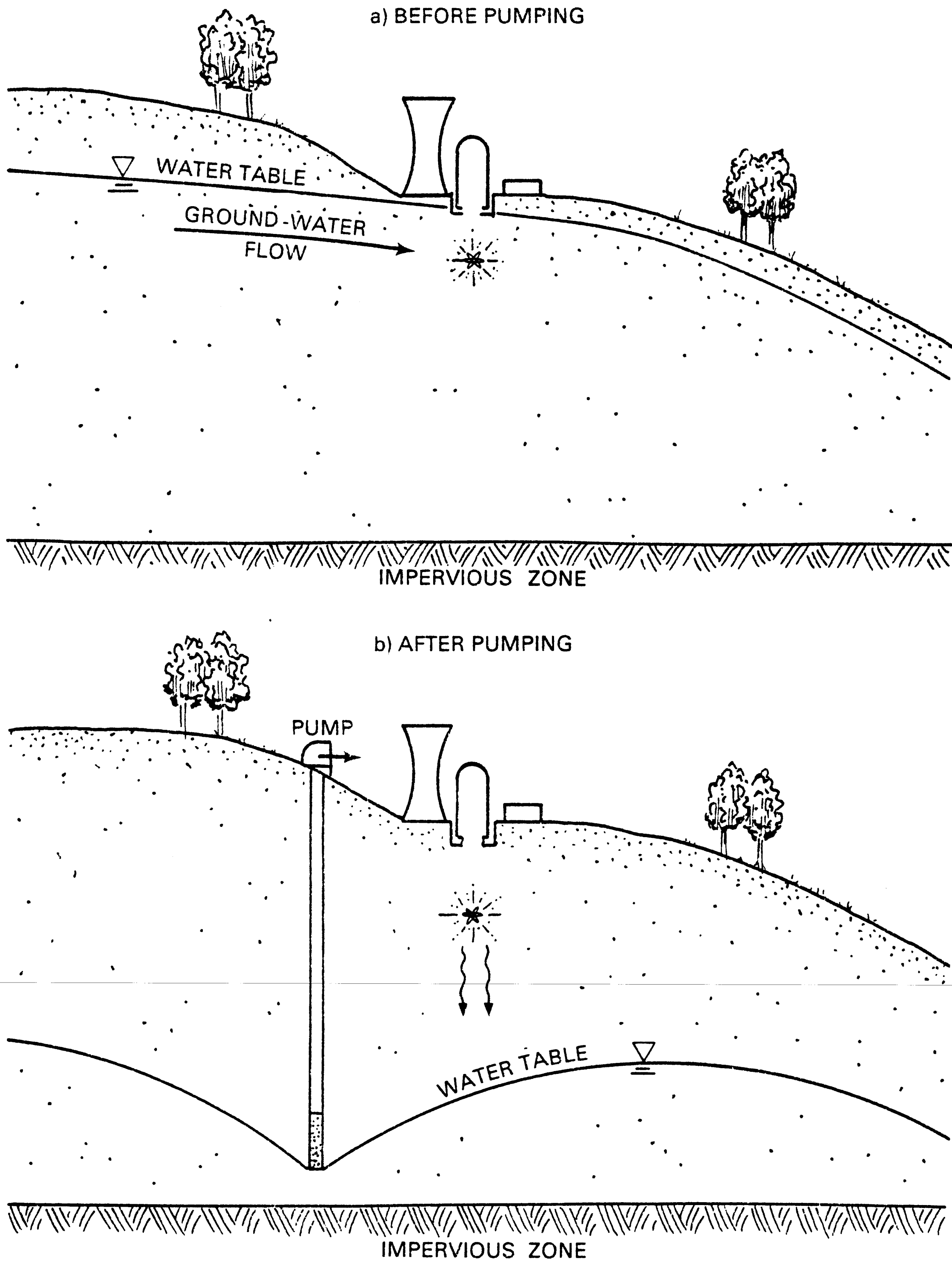

FIGURE 4.4.1-2. Pictorial Representation of Pumping to Lower Water Table Below Containment (After: EPA 1982)

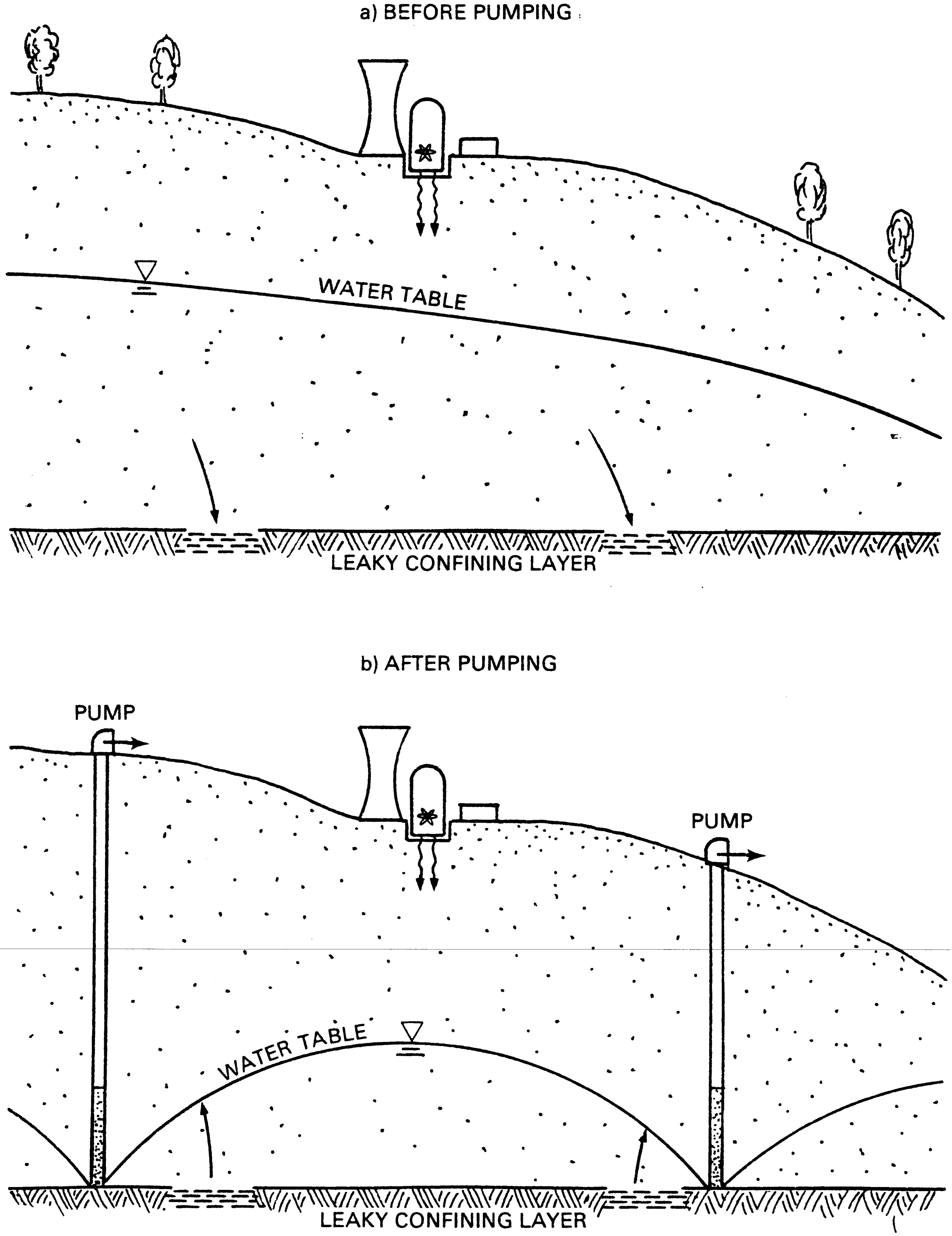

FIGURE 4.4.1-3. Pictorial Representation of Pumping to Prevent Contamination Through a Leaky Confining Layer. (After: EPA 1982).

table conditions exists. The third application is feasible for both confined and unconfined ground-water systems. All three applications may result in contaminated water being pumped to the surface. This situation may necessitate expensive and/or difficult handling procedures for the well discharge.

4.4.1.1 Design Considerations for Ground-Water Withdrawal for Water Table Adjustment

The following discussion is provided in order to present certain considerations for the design of dewatering systms. Within the context of this study, design refers to a conceptual development of dewatering strategy not the engineering design of a specific application. Detailed site analyses are necessary for the engineering design of dewatering systems.

Lowering of the water table necessarily requires the pumping of ground water by either using a well point dewatering system or a deep well system. A well point system is typically composed of a header pipe connecting a series of closely spaced wells which are pumped by suction centrifugal pumps, submersible pumps, or jet ejector pumps depending on the volume pumped and the depth of pumping. A separate pump may be used for each well point or a central pump may be used for several well points (EPA 1982).

Deep well systems are required for consolidated geologic formations where the water table is too deep for suction lifts. Each well is equipped with its own pump and can dewater at greater depths than a well point. The cost of a deep well system is generally higher than a well point system (Harris et al. 1982a).

Regardless of which well system (i.e., well point or deep well) is used the proper design of a ground-water dewatering scheme requires an understanding of well hydraulics and the hydrogeologic properties of the aquifer. The radius of influence of a well system must be determined for various pumping rates in order to estimate the extent of the cone of depression. The drawdown and total radius of influence for a well is a function of (EPA 1982):

1. Pumping rate,
2. Hydraulic-conductivity and saturated thickness of aquifer,
3. Ground-water recharge,
4. Regional and local flow boundary conditions, and
5. Length of time pumping continues.

The drawdown at various distances from the well field can be estimated using the above information. These estimates are best produced by representing the aquifer and well field in a mathematical ground-water flow model and solving for the drawdown at various distances. Well point spacing is based on the composite radii of influence required to achieve the necessary drawdown.

According to the EPA (1982) designs for well point dewatering systems are highly variable and depend on the depth to which drawdown is required, the transmissivity and storativity of the aquifer, the depth at which the contaminant arises, and the depth of the aquifer. Well points are normally installed by jetting down, driving in-place, or in open holes (Harris et al. 1982a; EPA 1982).

There are special situations which may require design modifications. Fine-grained soils (e.g., silts) with low permeabilities cannot be easily drained by well point systems. These soils can be partially drained with well points that are gravel-packed from the bottom of the hole to within a meter of the surface with the remainder sealed with bentonite or similar sealant. This system requires closely spaced well points, and pumping capacity is reduced. For stratified soils, vertical sand drains may be used along with well points to facilitate drainage. The drains are installed in the reduced permeability layers that require dewatering and extended to underlying higher permeability layers where the well points are located. Separate well point systems may be required for dewatering multiple permeable layers separated by impervious zones (EPA 1982). Similar analyses of aquifer response (i.e., drawdown) due to pumping are required to design a deep well dewatering system.

4.4.1.2 Construction Considerations for Ground-Water Withdrawal for Water Table Adjustment

Depths for well points are a function of the depth to which the water table must be lowered. The depth for the well points subsequently determines the type of pump that would be most efficient and the size (i.e., diameter) of the well points (EPA 1982).

For situations where the water table is relatively near the ground surface and maximum drawdown of 5 or 6 meters is required a well point system with a centrifugal suction pump may be adequate (Harris et al. 1982a; EPA 1982). Because in practice (primarily due to friction losses) suction pumping is limited to about 5 meters a deep well system, a multistage well point system, or a combination of deep wells and well points is required for pumping lifts greater than 5 or 6 meters.

Well point pipe sizes are normally determined from experience and site conditions. Recommended sizes vary depending the properties of the host material. For fine-grained material (e.g., silts) well points with a 1 1/2 inch (3.8 cm) diameter are suitable. For material with higher permeabilities well point diameters as large as 6 inches (15.2 cm) may be required. Riser pipe sizes usually vary between 1 inch and 3 1/2 inches in diameter depending on the well point diameter (EPA 1982).

Well point spacing depends on the radius of influence of each well and the required composite radii of influence. The normal range for well point spacing is 1 to 3 meters depending on ground-water velocities, host material properties, and the time available for dewatering (Harris et al. 1982a). Narrower spacing (i.e., 1 to 2 meters) may be required for stratified or fine-grained soils.

A typical well point system will yield between 40 liters per minute and 100 liters per minute per well point (Harris et al. 1982a). Greater yields can sometimes be achieved by using larger diameter well points. A hydrogeologic study of the aquifer characteristics in the site area is required to accurately estimate the yields and subsequent drawdown from a well point system or a deep well system.

Deep well construction involves the selection of the size of pump to be used which dictates the minimum diameter of well casing and screen (Harris et al. 1982a, EPA 1982). A 4-inch submersible pump can be used for well discharge rates less than 375 liters per minute. A 6-inch pump can be used for discharges between 500 and 1500 liters per minute (EPA 1982). Drawdowns in excess of 12 meters can be achieved by deep well systems (Harris et al. 1982a).

There are certain geologic conditions that favor filter packing of deep wells. These conditions are (EPA 1982):

1. Fine uniform soils where filter packing would allow larger slot openings in the well screen,

2. Thick confined aquifers where filter packing would allow screening the entire thickness,

3. Loosely cemented sandstone where filter packing would allow larger slot openings in the well screen, and

4. Thinly bedded formations where the thickness of each strata is not known.

Depending on the geologic conditions of the host material, the spacing for wells in a deep well system can be on the order of 15 meters (Harris et al. 1982a).

4.4.1.3 Performance Considerations for Ground-Water Withdrawal for Water Table Adjustment

Ground-water dewatering schemes are temporary measures for mitigating the effects of a severe power plant accident. However, depending on the accident scenario and resulting magnitude of the potential ground-water contamination, dewatering of the permeable geologic units may be an efficient and cost effective means of minimizing the impact of the accident. Specific advantages of aquifer dewatering schemes are listed in Table 4.4.1-1.

There are also certain disadvantages to water table adjustment schemes. One of the more serious is the problem of safely handling, processing, and/or disposing of contaminated ground water discharged from the wells. Additional disadvantages of this mitigation alternative are presented in Table 4.4.1-2.

TABLE 4.4.1-1. Advantages of Aquifer Dewatering Schemes (Source: Harris et al. 1982a, EPA 1982)

- Construction methods are relatively simple and there is a high degree of design flexibility.
- Construction costs are typically lower than for engineered barriers and construction times are less than for a grout curtain or sheet piling cut-off.
- Highly site adaptable and responsive to changes in contaminant plume migration - system can be easily disassembled.
- For well point systems, many wells can be discharged with a single pump.
- Systems are reliable if properly monitored.

TABLE 4.4.1-2. Disadvantages of Aquifer Dewatering Schemes (Source: Harris et al. 1982a, EPA 1982)

- Inadequate performance of well point systems in fine silty soils. Design flexibility is significantly reduced in this manner.
- Ongoing maintenance and operational costs escalate with time.
- Continous need for utility service.
- For well point systems, supervision is required to detect any breaks in the vacuum throughout the system.
- Consolidation and subsidence may cause problems in the vicinity of the drawdown.

4.4.1.4 Implementation Considerations for Ground-Water Pumping for Water Table Adjustment

As is the case with constructed barriers the implementation considerations for dewatering of a geologic unit are construction time requirements, cost, equipment mobilization and worker safety.

Installation Time

The construction time for installation of a well point system or a deep well system is dependent on the hydrogeologic site conditions, the size of the area to be dewatered, the depth of dewatering, and the work load. For development of a well point system four wells per day can be installed with one drilling rig working one shift. Two deep wells (i.e., 15 to 18 meters) per day can be drilled with one rig working one shift. These estimates assume average drilling conditions and extensive geologic-contaminant sampling was omitted during construction (Harris et al. 1982a). These estimates are very optimistic. If more drilling rigs are used and/or extra shifts are worked construction time can be reduced.

The well installation time is only one component of the overall time required to achieve a dewatering objective. Engineering design and equipment mobilization increase front-end time. Once dewatering (i.e., pumping) begins significant drawdown is not instantaneous. Depending on pumping rates and aquifer hydrogeologic conditions it may take several weeks or more to obtain satisfactory potentiometric adjustment.

Cost

The cost of a dewatering scheme is highly site specific. Cost are a function of (Harris et al. 1982a):

1. design and layout,
2. mobilization of equipment,
3. well pretesting and pump test analyses,
4. system installation,
5. operation and maintenance costs (i.e., labor, materials, energy), and
6. monitoring costs.

The EPA (1982) estimated the cost of a hypothetical well point dewatering system at $240,000.[a] The cost is for a system of 2 inch diameter well points placed approximately every 2 meters with the total number of well points equalling 416. The total length of header pipe is 762 meters with one centrifugal pump with a 5 meter lift. Included in the design are two high capacity wells with 4-inch submersible pumps. The water table is expected to be drawn down about 4 meters. Three monitoring wells with centrifugal pumps are also included in the cost.

Equipment Mobilization

Equipment mobilization should not pose any significant restrictions on ground-water dewatering. Standard drilling techniques are used and no specialty equipment is normally required. The contractor would require a certain amount of time to move equipment onsite, however. Unobstructed access for drilling equipment would be a necessity and drilling must be clear of overhead and subsurface utility services. A reliable power source for pump operation must be reachable from the site.

Worker Safety

An issue of critical concern is the safe handling of any contaminated water that may be pumped from the aquifer. Depending on retardation of radio-

(a) Assumed 1980 dollars.

nuclides by the host material, initial source concentrations, the pumping rate, and the distance the wells are from the contamination source variable concentrations of radionuclides may be in the pump discharge. Such contaminated discharge would require safe handling and disposal. Also, pumping contaminated ground-water could cause secondary contamination of well system equipment thus posing an additional safety problem.

4.4.2 Ground-Water Withdrawal and/or Injection for Plume Control

Ground-water withdrawal, with or without injection, for contaminant plume containment is a dynamic mitigation technique that has been used successfully to control saltwater intrusion in coastal areas. Plume containment may also be appropriate in certain instances for controlling radionuclide contamination from severe power plant accidents. Like ground-water dewatering, plume control via ground-water withdrawal is a conceptual approach to ground-water contaminant mitigation. There are four general pumping schemes that can be used for contaminant plume containment. These are (Harris et al. 1982a; EPA 1982).

1. A series of withdrawal and injection wells (often in pairs) that extract contaminated ground water for surface treatment and subsequent re-injection,

2. Ground-water withdrawal without recharge,

3. Withdrawal and surface treatment of contaminated ground water with recharge through recharge basins, and

4. Injection to reverse the hydraulic gradient.

Figure 4.4.2-1 shows the basic extraction/injection scheme for contaminant plume containment. The effect of withdrawal without recharge and recharge via recharge basins can also be visualized from Figure 4.4.2-1. Withdrawal without recharge may be feasible only for cases where small quantities of ground-water are being pumped because pumping large volumes may alter the potentiometric surface and direction of flow within a confined aquifer. Recharge is necessary when the withdrawal would adversely impact the regional ground-water flow regime (EPA 1982). Finally, water injection can be used to create a gradient barrier to force contaminated ground water to flow away from a given area of concern (Harris et al. 1982a).

4.4.2.1 Design Considerations for Contaminant Plume Control

Well design considerations for contaminant plume containment are the same as those for ground-water dewatering schemes. Both well point systems and deep well systems (depending of contaminant concentrations and hydrogeologic conditions) are applicable to plume containment. The approach is based on locating wells and establishing pumping rates that incorporate the plume within the radius of influence of an extraction well. Therefore, the effect of injection and withdrawal on ground-water flow and contaminant transport must be well

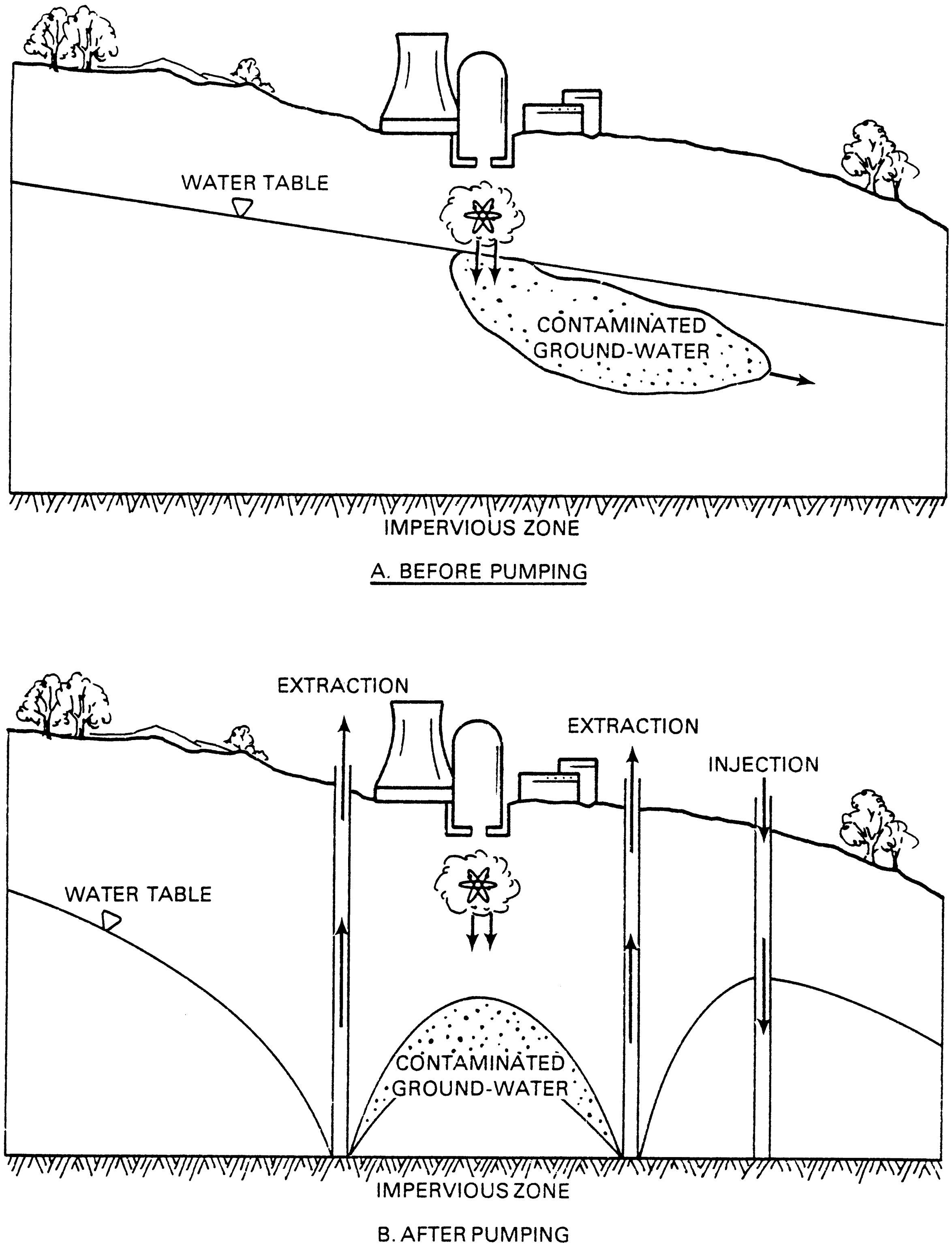

FIGURE 4.4.2-1. Pictorial Representation of Extraction/Injection Contaminant Plume Control (After: EPA 1982)

understood in order to design an effective plume containment system. The purpose of the injection well is to shorten the radius of influence of the extraction well and decrease the drawdown at greater distances from the extraction well (EPA 1982).

The EPA (1982) recommends that the design of an extraction/injection system be such that the radii of influence of withdrawal and recharge do not excessively overlap. This requires a reasonable separation of injection wells from extraction wells so that the overall effect of the scheme can be adaptive to changes in contaminant plume migration. However, site limitations may prohibit adequate separation of extraction and injection wells resulting in the overlapping of radii of influence. Contamination of the recharge water may be avoided by constructing an impermeable barrier between the extraction and injection wells.

A plume control system that consists entirely of extraction (at low withdrawal rates so as not to appreciably effect the regional ground-water flow regime) may be feasible in some situations. The design of an extraction system is less complex than a containment system that includes injection. The design must still be flexible enough to respond to changes in the contaminant plume velocity and direction of travel (EPA 1982).

For cases that require recharge to replenish local ground-water supplies but do not necessarily require injection for plume control, recharge basins may be cost effective means of recharging ground water where permitted by geologic conditions. Recharge basins should still be located, if possible, beyond the radii of influence of the extraction wells. Recharge basins require significant maintenance to insure that the porosity is not reduced. The side walls of the basin should also be pervious to facilitate recharge. The dimensions of recharge basins vary according to site conditions and the volume of recharge (EPA 1982).

Water injection, without extraction, can also be employed to control the movement of a contaminant plume. This approach can be used to redirect the movement of leachate or to increase the hydrostatic pressure head in a certain region creating a barrier to ground-water flow. Deep well injection could also be used in combination with extraction to remove the contaminant plume from a shallow aquifer and inject it into a deep unused (because of water quality limitations, etc.) aquifer (Harris et al. 1982a). A distinct advantage of injection systems, without extraction, is that contaminated ground water is not brought to the surface resulting in safety problems related to handling.

However, there are many problems associated with injection systems. Among these are clogging, maintenance, and cost. Also, injection increases ground-water mixing.

4.4.2.2 Construction Considerations for Contaminant Plume Control

Well construction for plume control and containment is similar to well construction for ground-water dewatering schemes (Section 4.4.1). Well point systems will be adequate for shallow aquifers and are recommended because of their flexibility if sufficient capacities can be achieved. Otherwise, high capacity wells must be installed (EPA 1982).

Water injection is more difficult than extraction. Injection wells are susceptible to clogging and recharge water containing suspended solids and other matter can reduce the efficiency of water injection. Filtration of injection water is usually required to insure effective recharge.

4.4.2.3 Performance Considerations for Contaminant Plume Control

Theoretically, contaminant plume containment via pumping and injection is feasible in any water bearing medium. Practical limitations related to the amount of water that can be pumped and/or injected, the ground-water velocity, and the physical dimensions of the plume determine on a site by site basis the actual effectiveness of plume containment as a mitigation technique. Two-dimensional or preferably three dimensional ground-water flow and contaminant transport simulations of the site should be conducted to determine the feasibility of this approach and estimate well locations and pumping rates.

Like most dynamic mitigation schemes plume containment is a temporary alternative that might preceed or be implemented in conjunction with a permanent barrier. Because of the ability to control withdrawal and injection rates this approach is highly flexible and adaptive to changes in plume velocity and size. For this reason plume control may be very important in the early stages of ground-water contamination because of the limited ability to "steer" the contaminant plume away from potentially dangerous locations.

The main advantage of ground-water withdrawal and injection for plume control is that the depth of the contaminant plume does not deter successful implementation of this mitigation scheme. Plume control via withdrawal and/or injection provides a positive means of reducing the velocity or changing the direction of the spread of a contaminant plume. It may be most appropriate in situations where a constructed barrier is not feasible or would result in excessive costs as would be the case in a crystalline bedrock aquifer (Harris et al. 1982a). Other advantages of plume containment are low cost compared to constructed barriers, design flexibility, and operational flexibility which facilitates adaptation to changes in contaminant plume migration (EPA 1982).

There are also several disadvantages to plume containment via pumping and injection. These include (Harris et al. 1982a; EPA 1982):

1. Operation and maintenance costs that significantly exceed O & M costs for other mitigation alternatives,

2. Injection water availability,

3. Suitability of host medium for injection,

4. Plume volume and characteristics are time dependent and may vary with climate conditions and site conditions,

5. Extensive monitoring is required to detect excursions beyond the control boundaries, and

6. In areas where the water table is near ground surface a high differential head may not be achievable.

4.4.2.3 Implementation Considerations for Contaminant Plume Control

The implementation considerations for contaminant plume control are the same as those for aquifer dewatering schemes (Section 4.4.1.4) and other mitigation alternatives. These considerations are time of installation, cost, equipment mobilization and worker safety. An additional implementation consideration is monitoring of the performance of a containment scheme.

Installation Time

As with strict dewatering schemes, the time required to install a plume containment system is site specific. It is dependent on the hydrogeologic properties of the site and the size and velocity of the contaminant plume. Roughly two deep wells (i.e., 15 to 18 meters) can be drilled per day with one drilling rig working one shift (Harris et al. 1982a). If deeper wells are to be installed the installation time would be increased accordingly.

Cost

The EPA (1982) has estimated the cost of a hypothetical plume containment system (excluding operation and maintenance costs) at $272,400.[a] This cost represents a system of 18 extraction and injection wells and four monitoring wells. The wells are 6 in. diameter wells approximately 10.5 meters deep. The plume dimensions are roughly 610 m long, 230 m wide, and 10 m deep. Water is extracted by seven pumping wells and injected, after surface treatment, through seven injection wells. Four injection wells are held in reserve in case of clogging in an active injection well. Each well is designed for a 4 in. submersible pump. The extraction wells and injection wells are approximately 300 m apart to avoid overlap of radii of influence. This system also requires over 1500 m of 8 in. steel pipe to connect the extraction system with the treatment system and subsequently to the injection system.

The total cost (i.e., $272,400) was computed using the following unit costs (EPA 1982):

1. Construction and installation of uncased well = $8.20 per $\frac{\text{inch diameter well}}{\text{meter of depth}}$
2. 6 in. PVC casing = $21.32/meter
3. 4 in. submersible pump = $1,175.00
4. 8 in. steel pipe = $150.92/meter

(a) Assumed 1980 dollars.

By far the most expensive component of the system is the network of steel piping linking the extraction wells with the treatment system and injection wells. This component accounts for $230,000 of the total cost. The cost does not include the development of the treatment system or energy costs related to system operation.

Equipment Mobilization

No specialty equipment is required to implement a plume containment system. Standard well drilling equipment can be used for installation. The system layout should consider any buried or overhead obstacles such as utility services which may obstruct drilling. In some cases temporary roads may have to be constructed or drill rigs pulled to the site via bulldozers. On site access of drilling equipment must also be considered. A reliable power source must also be assured along with an injection water supply of acceptable quality.

Safety in handling contaminated ground water that is brought to the surface is also important. If surface treatment is to be used safety precautions must be invoked to insure that uncontaminated areas do not become contaminated thus compounding the problem.

Monitoring

Careful monitoring of the extent of the contaminant plume and any changes in plume configuration is a necessity if a plume containment scheme is to be successful. Because pumping and injection schemes are highly adaptive, containment system operation can be adjusted to respond to changes in plume characteristics that if undetected would result in loss of control of the plume.

4.4.2.4 Examples of Existing Plume Containment Systems

Rocky Mountain Arsenal, Denver, Colorado

To control the migration of contaminants leaching from a surface storage basin a combination plume containment/impermeable barrier system has been installed at the Rocky Mountain Arsenal. The system consists of a series of extraction wells up-gradient from an impermeable barrier and a series of injection wells down-gradient from the barrier. Approximately thirty-three 8 in. extraction wells remove contaminated ground water which is treated and injected through forty 16 to 18 in. injection wells. The total system is 5200 ft long and handles a flow of 443 gpm (EPA 1982, Miller 1979).

Palo Alto, California

A series of extraction/injection wells is being used to create a barrier to further salt-water intrusion in a multiple aquifer ground-water system in the bayfront area of Palo Alto, California. Nine extraction/injection well doublets with a total capacity of 7.6 million liters per day comprise the system. In addition, three types of monitoring wells (i.e., shallow, mid-depth, and deep) were designed and installed to serve as both monitoring points and test holes for hydrogeologic site characterization. The bid price for the system was $400,000 (assuming 1975 dollars) (Sheahan 1977).

4.4.3 Interceptor Trenches (Subsurface Drains)

Interceptor trench or recovery drain systems are quasi-dynamic ground-water contaminant mitigation techniques that may be appropriate for radionuclide contamination of a shallow, unconfined ground-water aquifer (Harris et al. 1982a). However, handling and disposal of the recovered contaminated ground water may pose significant safety problems. Subsurface drains are gravel or sand-filled trenches with plastic or ceramic drain title. The trenches can be placed either up-gradient or down-gradient from the contaminant source and intercept (in the up-gradient case) uncontaminated water that was destined to become contaminated or (in the down-gradient case) contaminated water for treatment depending on location.

Subsurface drains are most suitable for application in clay or silty clay soil where the permeability of the drain can be made significantly greater than that of the host material (EPA 1982). Recovery drains may not be feasible where deep frost zones exist.

4.4.3.1 Design Considerations for Interceptor Trenches

The design of a subsurface collection system is dependent on the volume of contaminated ground water (for down-gradient systems), or the volume of uncontaminated ground water (for up-gradient systems), to be intercepted. The quantity of ground water to be drained can be used to estimate the performance requirements for the drains (EPA 1982). The design of the collection system is based on the estimate of the quantity of intercepted ground water.

To effectively convey ground water, the drain must be more permeable than the soil being drained. The envelope material (i.e., backfill material) should be roughly twenty-five times more permeable than the host material (i.e., material being drained) (Harris et al. 1982a). Also, the drain should be below the water table to a depth that is adequate to intercept the contaminant plume. Consequently, a limiting factor in the design of a subsurface drainage system is the operational limits of trenching equipment.

According to the EPA (1982) subsurface drains should have a slight slope. Grades provide velocities sufficient to keep the drains clean during discharge and increase speed of drain emptying when discharge has stopped. Slopes accurately excavated of 0.1 percent are feasible with current trench digging equipment.

An important design consideration for subsurface drains is the resistance to flow in the drain. Because of the small area of inflow for most drains significant resistance to flow is sometimes encountered. The resistance depends on (EPA 1982):

1. The hydraulic conductivity of the material surrounding the drain pipe,

2. The geometric flow characteristics, and

3. The distribution and orientation of openings in the wall of the conveyance pipe.

The type of drain pipe is usually less critical to performance than the resistance of approach properties of the envelope material (EPA 1982).

Once the drain has been designed, the removal system can be designed. The removal system usually consists of one or more sump basins or wetwells. The entire system should be located as close to the contaminant source as practically possible in order to maximize the collection of contaminants while minimizing the collection of uncontaminated ground water (Harris et al. 1982a; EPA 1982).

4.4.3.2 Construction Considerations for Interceptor Trenches

Typically, subsurface drainage systems are constructed by excavating a trench and placing plastic or ceramic drain tile end to end along the bottom. The trench is backfilled with a suitable envelope material (e.g., gravel, sand, etc.) to a certain thickness above the drain pipes and then capped with soil or clay (EPA 1982). Slit trenches excavated by backhoe may be suitable where seasonal fluctuations in the water table are minimal and the site soil is relatively cohesive. When overburden material is less cohesive and water table elevations are deep, trenching becomes more complex and expensive (Harris et al. 1982a).

In some cases a synthetic, impermeable liner can be placed at the downgradient side of the interceptor trench in order to prohibit contaminated ground water from flowing through the trench. The liner may be necessary if the envelope material has a relatively high permeability (EPA 1982). Also, after the trench is backfilled with the envelope material it may be necessary to wrap the material with a pervious fabric to prevent clogging of the gravel and drain with soil particles. The EPA (1982) suggests a strongly woven fabric called Typlar® which allows water to pass but prevents soil from entering the granular envelope.

The construction of interceptor trenches is limited by encounter with impermeable soil layers and the operational limits of trenching machinery. While theoretically trenches can be excavated to considerable depths, the practical economic constraints become prohibitive. Hydraulic backhoes can excavate to depths on the order of 17 meters. For greater depth excavations a crane and clamshell apparatus can be used (EPA 1982).

4.4.3.3 Performance Considerations for Interceptor Trenches

The performance of subsurface drains is a function of design, accident scenario, and local climate conditions, all of which are site specific. The primary advantages of subsurface drains as a ground-water contaminant mitigative technique are (Harris et al. 1982a; EPA 1982):

1. Active removal of contaminated ground water,

2. Considerable design flexibility and adaptation to dynamics of ground-water contamination,

3. High reliability because of extensive monitoring,

4. Relatively low maintenance requirements.

As is the case with other mitigative techniques that actively remove contaminated ground water (e.g., recovery wells, permeable treatment beds, etc.) a serious problem may exist for the safe handling and disposal of contaminated ground water recovered via subsurface drains. Other disadvantages to subsurface drains are (Harris et al. 1982a; EPA 1982):

1. Poorly suited for low permeability soils,
2. Limited to areas of shallow ground water in unconfined aquifers,
3. Location in close proximity to contaminant source, and
4. Continuous monitoring required.

Monitoring is extremely important in assessing the performance of subsurface drains since the opportunity exists for the contaminated ground water to breach the drainage system. An auxilliary or back-up system, perhaps a dewatering scheme, should be available in the event of a failure of the drainage system.

4.4.3.4 Implementation Considerations for Interceptor Trenches

Time of constructioin, cost, equipment mobilization, and safety of workers are the most important implementation issues for interceptor trenches.

Time of Construction

Construction time is dependent on the length and depth of the trench and the properties of the overburden. If shoring of the trench walls is not required, subsurface drains can be developed fairly quickly, especially in relation to other constructed mitigative techniques. However, if extensive shoring and/or dewatering requirements are associated with the trench excavation the

Cost

The cost of interceptor trenches is site specific and dependent on the final design. Unit costs for such items as excavation, drainage tile, crushed stone, etc. can be used to approximate the total cost of a subsurface drainage system. The EPA (1982) gathered data pertaining to the unit costs of several items required for the developed of a drainage system. These costs are presented in Table 4.4.3-1. Maintenance costs are estimated by Harris et al. (1982) to be approximately $1,600[a] per year.

(a) Assumed 1978 dollars.

TABLE 4.4.3-1 Unit Costs for Subsurface Drainage Collection Systems (Source: EPA 1982)

Item	Unit Cost[a]
Excavation; 6 m deep, 1.2 m wide hydraulic backhoe	$1.30 m^3$
Crushed stone; 3/4 inch Cost to buy, load, haul 5 Km, place, and spread	$10.85 m^3$
Tile Drainage	
Vitrified clay (Standard bell and spigot)	
4" perforated	$7.00 m installed
6" perforated	$8.60 m installed
8" perforated	$14.20 m installed
Percast concrete manholes	
48" x 3'	$180.59
48" x 4'	$215.73
Concrete wetwells	$6,500.00
Sewer piping;	
Concrete; nonreinforced; extra strength	
6" diameter	$12.90 m
8" diameter	$14.10 m
Bituminous fiber	
4" diameter	$6.70 ms
Sewer piping, PVC	
4"	$5.70 m
6"	$9.50 m
8"	$15.10 m
Backfilling:	
Spread dumped material by dozer	$0.86 m^3$
4" Submersible pumps installed; to 55 m	
2 HP; 840 - 1440GPH	$1,700
5 HP; 1302 - 1494 GPH;	$2,375
Holding tank;	
Horizontal cylindrical glass fiber reinforcement phthalic resin tanks	
37,850 ℓ (10,000 gal)	$6,354 installed
75,700 ℓ (20,000 gal)	$14,164.50 installed

(a) Assumed 1978 dollars.

The design considerations associated with permeable treatment beds are (EPA 1982):

1. Selection of suitable filtration material,

2. Location of the treatment bed in relation to the regional and local ground-water flow regime,

3. Length (perpendicular to general direction of flow),

4. Width (in the direction of the flow), and

5. Depth, must be keyed to impervious strata or barrier.

In most cases it would be advisable to place the treatment bed reasonably close to the contaminant source. Dispersion of the contaminant plume decreases closer to the source thereby reducing the overall dimensions of the treatment bed and consequently its cost. However, close proximity to the contaminant source may not be feasible because of inability to adequately anchor the filter in an impervious layer or the contaminant plume has migrated a considerable distance prior to design and construction of the treatment bed. The length of the treatment bed may be reduced with a fractional increase in width or thickness if highly permeable converging ground-water flow channels (e.g., gravel drains, etc.) are used to intercept contaminated ground water and divert it to the filter. Down-gradient from the treatment bed a similar arrangement for dispersing treated ground water via diverging drains, can be used.

Selective filtering using permeable treatment beds is potentially a very versatile and adaptive technique. For example, this technique may be feasible in deeper aquifers if combined with a grouting operation to form a cut-off below the permeable treatment bed. Once the trench has been opened, grouting can be performed beginning at the trench bottom and continuing down to an impervious key. The trench can be opened a limited depth on either side of the grout curtain to form a sill or a key for the cut-off wall into the treatment bed. The grout curtain would then prevent the flow of untreated contaminated ground water under the permeable treatment bed. The permeability of the treatment bed could then be adjusted to minimize the "bathtub" effect of the grout curtain cut-off. Figure 4.4.4-2 demonstrates this approach.

The width of the trench and consequently the width of the filtration material is a function of the ground-water velocity, the permeability of the filtration material, and the required contact time of the contaminated ground water with the filtration material to achieve effective treatment (EPA 1982). A rough approximation of the bed width can be calculated by multiplying the highest local (i.e., in the vicinity of the proposed location of the treatment bed) ground-water velocity by the required contact time. The determination of the effective contact time requires knowledge of both the radionuclide concentration in the ground water, and sorptive properties of the radionuclide species, and the filtration material being considered. Disturbance to the

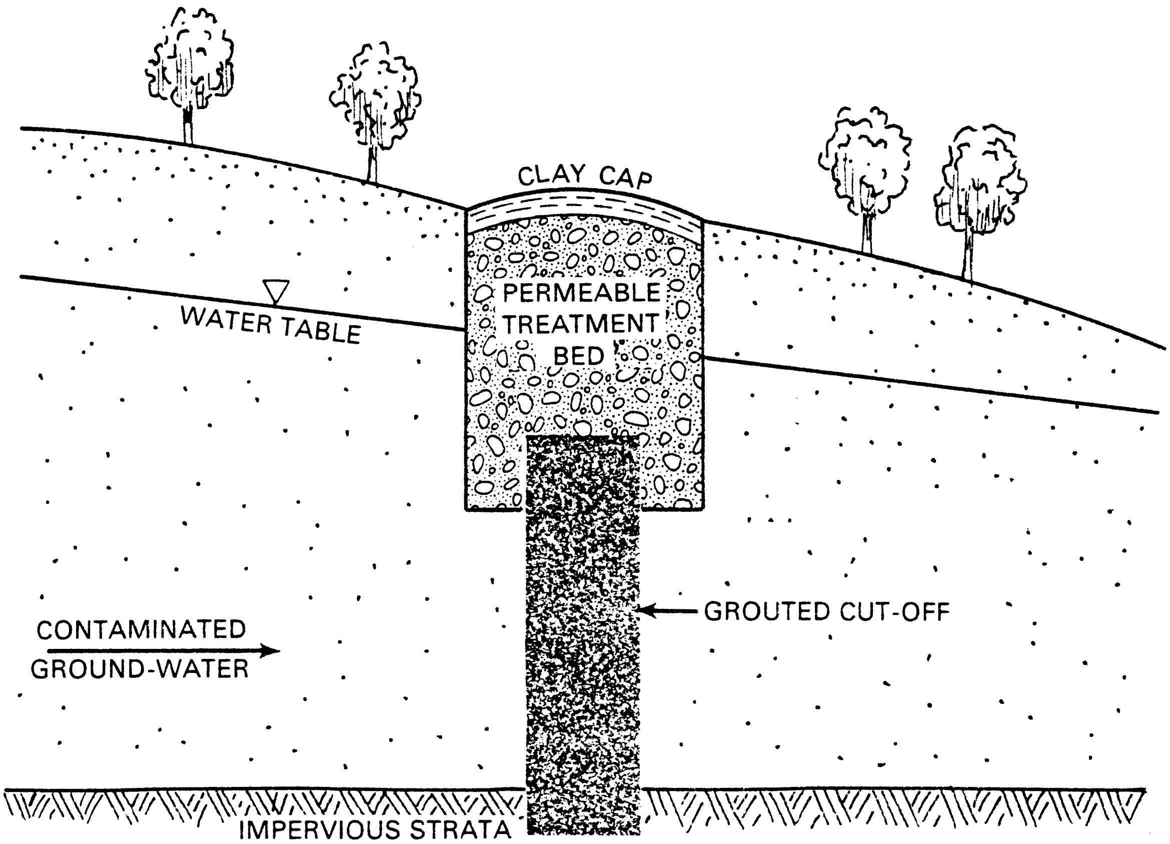

FIGURE 4.4.4-2. Permeable Treatment Bed in Combination with a Grouted Cut-Off

general ground-water, and flow regime can be minimized by adjusting the permeability of the treatment bed to approximate the permeability of the surrounding aquifer (EPA 1982).

4.4.4.2 Construction Considerations for Permeable Treatment Beds

Permeable treatment beds are only practically applicable to unconsolidated host media because of excavation requirements. Consequently, excavation normally requires shoring (e.g., sheet piling). The trench will intersect the water table thus requiring dewatering of potentialy contaminated ground water. Trenches are dug in a manner similar to the construction of interceptor trenches. Backhoes or clamshells would ordinarily be used for the excavation. The trench borrow material can be used for a compacted cap or, if inadequate, a suitable cap of compacted clay can be placed over the trench once the filter material is in-place.

Natural materials or synthetic resins with high ion exchange capacities must be used for the filter material. The radionuclides sorb onto the skeletal framework of the material thereby reducing their concentration in the ground water. Glauconitic greensand has good adsorption properties for heavy metals and may be applicable to mitigation of radionuclide contaminants. Laboratory experimentation is required to determine representative contact times for various removal efficiency levels. Deposits of accessible glauconitic greensands are found almost exclusively in New Jersey, Delaware, and Maryland in the U.S. Because of transporation costs for the material, their use would practically be limited to permeable treatment bed construction in the Mid-Atlantic region. Synthetic ion exchange resins also have high ion exchange capacities but are short-lived and very costly (EPA 1982).

4.4.4.3 Performance Considerations for Permeable Treatment Beds

Selective filtration employing permeable treatment beds is a dynamic and, for most applications, a temporary mitigation technique for ground-water contaminants. This technique is dynamic in the sense that its performance is time-dependent. Radionuclide removal efficiency will decrease with time. The filter may gradually become plugged and the ion exchange capacity of the filter material will decrease with time. The acutal time-history performance of a permeable treatment bed is highly specific to individual applications. The technique remains principally conceptual and field data are rare or nonexistent.

The selective filtration technique, when feasible, has a distinct advantage over ground-water barriers and other passive techniques. It is an aquifer restoration or remedial action strategy. Permeable treatment beds actively remove the contaminant from the ground water instead of diverting the plume away from sensitive areas with reliance on decay and naturally occurring sorption to mitigate the hazard.

4.4.4.4 Implementation Considerations for Permeable Treatment Beds

There are important implementation considerations for permeable treatment beds. Among these are construction time, cost, equipment mobilization, disposal of spent filtering material and worker safety. Permeable treatment beds can be constructed very rapidly assuming adequate site access and no additional site work (e.g., grouting or drainage construction) is required. No sophisticated equipment is necessary, consequently little lead-time is required for equipment mobilization. Backhoes can be used for trench excavation and common sheet piling can be used for shoring of open trenches. The most time consuming aspect of permeable treatment bed construction may be the quarrying and hauling of the filter material.

If glauconitic greensands are to be used as the filter material, the client's contractor may have to negotiate the purchase of land overlying an accessible deposit because there are few or no commercial producers of glauconitic greensands. The contractor would have to excavate the filter material and haul it to the site thus greatly increasing the total cost of the permeable treatment bed (EPA 1982).

The EPA (1982) has estimated the cost[(a)] of a permeable treatment bed based on approximations for unit costs of various activities. The costs are broken into trench excavation costs, materials costs, and installation costs. Trench excavation costs include:

1. Excavation ≅ \$0.76/cubic meter (\$1/cubic yard)

2. Spreading of borrow ≅ \$0.50/cubic meter (\$0.66/cubic yard)

3. Well point dewatering ≅ \$23.00/lineal meter (\$75.00/lineal foot)

4. Sheet piling:
   Sheeting ≈ \$0.53/square meter (\$5.70/square foot)
   Walers and struts ≈ \$95.25/metric ton

Materials and installation costs are difficult to estimate, especially for glauconite bearing deposits, because possible land purchase and transportion are significant cost items that cannot be unit priced. Synthetic ion exchange resins are very expensive with costs estimated by the EPA (1982) at more than \$10.00 per kilogram (\$5.00/lb). For comparison of the effect of the type of filter material on total project costs, the EPA (1982) estimated the costs for a permeable treatment bed approximately 300 m long, 1.2 m wide, and 6 m deep. If the filter material is crushed limestone, the total cost is \$485,000. If activated carbon is used for the filter material, the cost is \$4,531,000. The difference in cost is nearly an order of magnitude.

In summary, there are particular advantages and disadvantages of glauconitic permeable treatment beds (EPA 1982). The advantages include potentially high removal efficiencies, good residence time properties thus reducing the volume of material needed, and good permeability. Disadvantages of glauconitic treatment beds are unknown saturation characteristics, application practically limited to Mid-Atlantic Region, bed plugging may occur, may reduce ground-water pH, and removal efficiencies of constituents at high concentrations is unknown. The technique is feasible however, and additional experimentation and analysis will define limits of performance and ranges of feasible application.

4.4.5 Ground Freezing

Ground freezing is a technique whereby, depending on soil particle size and ground-water velocity, ground-water flow can be significantly reduced. A frozen subsurface wall a few meters thick is created in much the same manner as a grout cut-off. Ground freezing is an energy intensive dynamic interdiction technique that may provide temporary mitigation while more permanent measures are implemented.

(a) Assumed 1982 dollars.

4.4.5.1 Design Considerations for Ground Freezing

The mechanical properties of frozen soil are such that in many cases they have relatively higher strengths than similar soils treated with other geotechnical procedures. Because of the increase in strength of frozen soils groundwater freezing can, in some cases, prevent ground-water flow (Attewell and Farmer 1976). Although full saturation of the host material pore space is not required for this method to be effective, a minimum moisture content is necessary. The minimum required moisture content is a function of the material grain size and distribution (Harris et al. 1982a).

According to Sanger (1968) frozen ground engineering requires a great deal of field experience and engineering judgement. There are only a few commercial contracting companies and much of their information on design and construction of frozen barriers (especially soil properties) is proprietary. However, the design of a frozen barrier must necessarily include considerations of the structural stength and deformation of the frozen soil and the thermal properties of the soil.

The strength that frozen soil exhibits is derived from the bonding of soil particles and ice. Clay soils which adsorb a large amount of water develop less ice and weaker bonds than quartz-type soils which adsorb very little water. This phenomenon is due to the lower freezing point of the water adsorbed onto the surface of clay minerals than the surrounding free water. Conversely, quartz-based soil particles adsorb very little water thus allowing more free water ice to form.

A continuous ground-water barrier is achieved in much the same manner as a grouted cut-off is developed. Cylinders of frozen soil are formed around freeze pipes through which a suitable coolant is circulated. The radii of the frozen cylinders increase until adjacent cylinders intersect. Continued cooling then increases the frozen wall thickness until the design thickness is reached (Attewell and Farmer 1976). The process involves two stages (Attewell and Farmer 1976; Sanger 1968):

1. Stage I - A solid (frozen) soil cylinder is forming around the freeze pipe, and

2. Stage II - The cylinders have merged and the wall thickens.

During Stage I (the transient stage) the rate of advancement of the ice front is a function of the thermal diffusivity and the moisture content. The rate of advancement of the freeze zone decreases with increasing radius from the freeze pipe. Once Stage II (steady state) has been reached, the heat outflow is a function of the thermal conductivity of the soil (Attewell and Farmer 1976).

In theory, ground freezing can be applied to almost any geologic medium in which freeze pipes can be installed and suitable moisture exists. However, both Stage I and Stage II freezing are affected by ground-water flow. Consequently, ground-water velocities are important considerations in determining the feasibility and subsequent design of a frozen barrier. The method is not feasible for ground-water velocities that are not relatively slow. Sanger (1968) states that most experienced contractors consider 0.9 m/day to 1.2 m/day the maximum ground-water velocity that can be successfully tolerated. At higher ground-water velocities the resulting frozen wall may be practically (99+%) impervious over solid portions but contain small windows that will not close regardless of the amount of refrigeration.

Small windows may also be present in frozen soil that was unsaturated. Consequently, freezing should not be considered for host material with <10% saturation (Harris et al. 1982a). Also, because of lower freezing points, ground freezing is not normally feasible in strata bearing heavily contaminated ground water.

Harris et al. (1982a) polled four ground freezing contractors (Frontier-Kemper Constructors; More Trench; Geofreeze Corp.; and ECI). They state that there are no depth limitations for a frozen wall although thermal erosion under warm ambient conditions must be considered. Recommended temperatures for suitable strength of frozen walls are -7°C for sand and -29°C for soft clay (Harris et al. 1982a).

4.4.5.2 Construction Considerations for Ground Freezing

Construction of a frozen wall requires the vertical installation in the host medium of a series of steel refrigeration or freeze pipes. Once the freeze pipes are in-place to the desired depth, a refrigerant is circulated through the pipes which causes heat removal from the host material. Continued heat removal causes an expanding frozen cylinder to form around each freeze pipe. As the cylinders intercept each other a continuous frozen wall develops (Harris et al. 1982a).

A wide variety of equipment can be used to construct a subsurface frozen wall. However, basic equipment requirements include a freeze plant for the refrigerant, a system of surface pipes and pumps to distribute the refrigerant to the freeze pipes, the freeze pipes, and instrumentation pipes which are also inserted in the ground to monitor soil temperature.

The freezing plants for cooling of the refrigerant are normally composed of one or more mobile refrigeration machines. Circulating cooling systems are used except in rare emergencies when, because of strict time limitations, expendable coolants (i.e., liquid nitrogen or liquid carbon dioxide) are used in a non-circulating open system. Expendable refrigerant systems are not recommended because of difficulty in field control and expense. Normal ground freezing operations employ a brine (e.g., calcium chloride) which is cooled by the freezing plant and circulated to the freeze pipe. Brine circulating systems require greater time in comparison to cryogenic liquid systems but are easier to control and less expensive. Circulating systems can also be run for longer periods of time (Harris et al. 1982a). For permanent refrigeration systems, an alcohol solution (e.g., ethylene glycol) is used for a coolant (Sanger 1968).

Refrigeration procedures are important considerations in ground freezing. The refrigeration sequence is comprised of an initial period with peak refrigeration load followed by the maintenance period for temperature and consequently wall thickness. During the Stage I period, 24 hour monitoring is recommended for quality assurance (Harris et al. 1982a). The lowest feasible brine temperature will require the least amount of time to complete the wall but will result in the heaviest refrigeration load. Refrigeration loads are computed as tons of refrigeration per foot of pipe with 1 ton representing 200 Btu per minute. Both refrigeration load and temperature may require adjustment during the freezing process to accomodate changing needs (Sanger 1968). The two primary considerations in constructing a frozen subsurface barrier are the insulation of the above-ground system to minimize heat leakage and the specifications and placement of the freeze pipes. Sanger (1968) states that proper insulation of above-ground piping by closed-cell foamed plastic can significantly reduce the heat loss of the above-ground system. Such insulation should be protected from weather and construction damage and painted with white or aluminum paint. To combat open air convection and solar radiation, any exposed frozen surfaces should be covered. Natural ground cover should not be disturbed, if possible. Plastic membranes can sometimes be used to protect the surface area directly above the wall.

Freeze pipes are typically composed of an open-ended inner feed pipe, through which the brine is injected, inside a closed-end freeze pipe for brine recovery. The coolant (brine) is pumped down the open-ended inner feed pipe. As the brine rises in the annular space between the feed pipe and the freeze pipe it absorbs heat from the surrounding host material. The brine eventually returns to the refrigeration plant for recooling and recirculation. The feed pipe usually has a diameter of 1-1/2 in. (4 cm) to 2 in. (5 cm) and is approximately half a meter shorter than the closed-end freeze pipe. These pipes are usually ordinary steel, however, plastic feed pipes have also been used (Sanger 1968).

The initial cost of the piping can be expensive creating a tendency to use small pipe with large spacing. However, the radial expansion of the frozen cylinders around the freeze pipes slows down quickly enough to cause excessive time requirements if piping and spacing are not properly designed. Freeze pipes are usually between 4 in. (10 cm) and 6 in. (15 cm) in diameter and are placed 1.0 m to 2.0 m apart. The larger the diameter the better the alignment control during placement (Sanger 1968).

Much like grouting, multiple rows of freeze pipes can be installed to construct a thick frozen wall. Also, for a frozen ground-water cut-off it is important to insure penetration of the freeze pipes into an impervious soil or rock strata. The host material will ordinarily freeze below the pipe to a depth of approximately 0.4 times the average wall thickness. Sanger (1968) recommends a 3 m penetration of the freeze pipe into the impervious layer for a satisfactory waterstop.

4.4.5.3 Ground Freezing Performance Considerations

All characteristics of a subsurface frozen cut-off are strongly dependent on temperature. When complete freezing is achieved and no windows exist, a frozen ground-water barrier is essentially impervious. However, problems of windowing have occurred with single-row walls, thus requiring a double row wall. Ground-water freezing in fissured and fractured rock can be especially difficult because of the lack of adequate means to estimate the amount of water in the fissures. While the overall rock material may be frozen, a window at a large fissure may remain open (Sanger 1968).

The determination of initial wall closure (i.e., end of Stage I) is sometimes difficult. Thorough monitoring is necessary to track the freezing process. A surface heave pattern may indicate the progress of Stage I but temperature sensors provide more reliable information in most cases, especially in sands. However, temperature measurements may be misleading in fine-grained material such as clays and silts because the effective freeze temperature of water is below 0°C. Also, dissolved salts can lower the freeze point, thus a brine leak posses a potentially serious problem (Sanger 1968). Ground freezing within the influence of a salt-water/fresh-water interface would very seldom be practical because of lower freeze temperatures of the salt water.

Ongoing maintenance is required for frozen ground-water flow barriers. The Stage II temperature of the cut-off must be held at a suitable temperature if the wall is to maintain its integrity. Because of the necessity to hold a particular temperature, the maintenance of a frozen wall is energy intensive. The freeze plant may be only operated periodically during Stage II but it must be serviced and available.

4.4.5.4 Ground Freezing Implementation Considerations

Like most of the potentially feasible mitigative techniques for ground-water contamination resulting from a severe power plant accident, the key implementation considerations for ground freezing are: 1) installation time, 2) cost, 3) equipment mobilization; and 4) worker safety. Ground-water freezing must also consider the location of the thermal plume and its heat content. A frozen ground-water barrier may be placed under severe thermal stress when encountered by ground water with temperatures significantly above ambient temperatures. As is also the case with other mitigative techniques ground freezing implementation considerations are highly site specific and site sensitive.

Installation Time

Data are not readily available concerning the actual times required to close a frozen wall. However, once Stage I begins (i.e., freeze pipes are installed and refrigeration commences) the time for closure is roughly exponentially proportional to the relative spacing of the freeze pipes. The closer the pipes the shorter the time required to achieve closure (Harris et al. 1982a).

Sanger (1968) presents two figures (Figure 4.4.5-1 and Figure 4.4.5-2) showing the results of the development of two straight frozen walls; one in a fine-grained soil and the other in a coarse-grained soil. The time for closure (i.e., Stage I) in the fine-grained soil was 38 days while it only took 21 days for closure in the coarse-grained soil. The time requirements for Stage II are also presented in the figures. The specifications of the freezing operation were (Sanger 1968):

1. ambient temperature = 15.5°C (60°F),
2. freeze pipe temperature = -23.3°C (-10°F),
3. freeze pipe diameter = 6 in.,
4. pipe spacing = 1.2 m (4 ft), and
5. design wall thickness = 2.1 m (7 ft).

Prior to initiating freezing, however, the freeze pipes must be installed in the host material. They are placed in pre-drilled holes with strict vertical alignment tolerances. Fractured and fissured rock and unconsolidated material containing large cobbles or boulders decrease drilling accuracy and require longer drilling time. Equipment mobilization may take a minimum of one week or longer while the drilling of holes and subsequent installation of freeze pipes may require six weeks or more (Harris et al. 1982a).

Equipment Mobilization

Surface access must be provided for drilling rigs and there must not be any subsurface drilling obstructions (e.g., gas pipelines, etc.). Standard industry drilling equipment is readily available and would pose no constraint to the overall constuction of a frozen cut-off.

The freeze plants are combinations of portable refrigeration machines and do not require a foundation. However, substantial electrical service is necessary because the refrigeration plants are normally powered by electricity. According to Harris et al. (1982a) electric motors offer greater reliability and less maintenance than gasoline or diesel powered engines. Without the availability of sufficient commercial power (300 kVA to 1000 kVA) freezing is not feasible.

Cost

Because of high costs, ground freezing is not competitive with other barrier-type methods of interdicting ground-water flow and contaminant transport. Although costs cannot be generalized, some rule-of-thumb figures[a] suggest: $35 to $45 per cubic meter for freezing costs for cut-off walls; and $.90 to $3.80 per square meter of wall area per week for refrigeration maintenance costs (Harris et al. 1982a). Drilling costs represent a substantial addition to the project cost, and potentially can cost as high as 35% to 50% of total construction cost (Harris et al. 1982a).

(a) Assumed 1982 dollars.

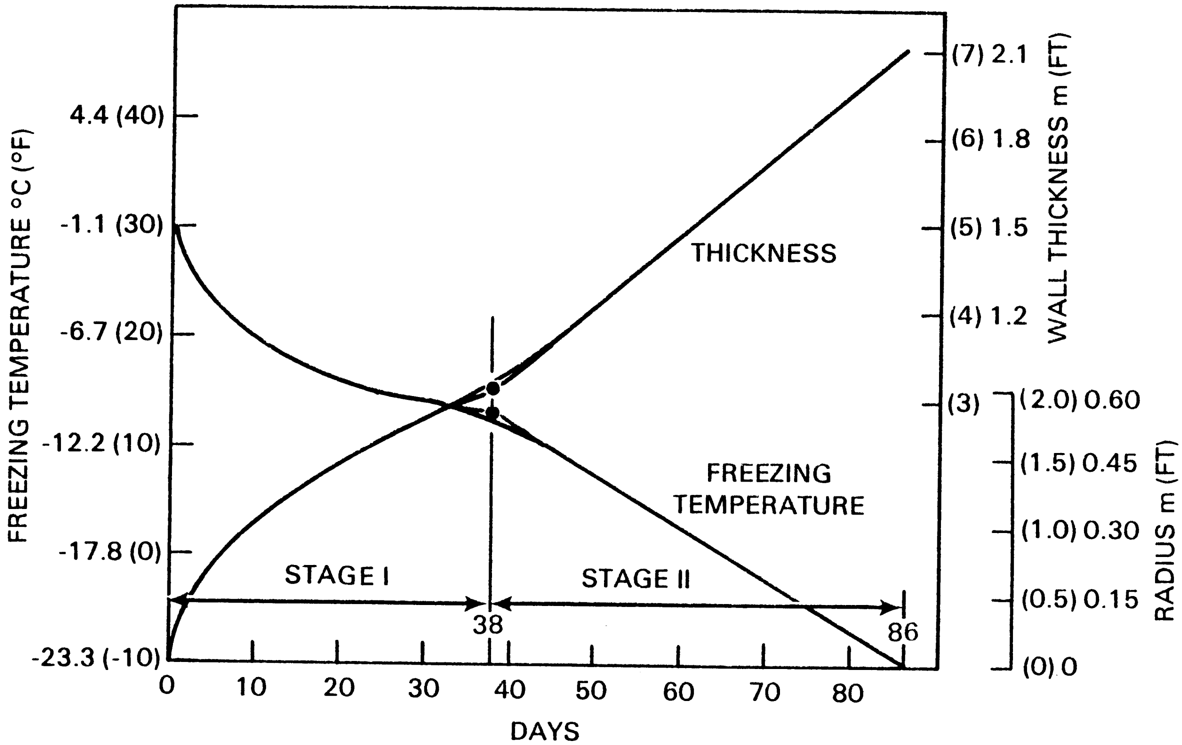

FIGURE 4.4.5-1. Frozen Straight Wall Development in Fine-Grained Soil (Source: Sanger 1968)

Artificial ground freezing is expensive and is not feasible for long-term closure. Also, experience is limited to a few contracting companies and is highly dependent on qualified and experienced personnel. Sanger (1968) states that ground freezing is usually considered as a last resort.

Worker Safety

The same considerations for worker safety requirements for other mitigation alternatives are also required for ground water freezing operations.

4.4.6 Air Injection

Air injection below the water table has been studied as a possible mechanism for the retardation of the movement of fluid borne contaminants (Nelson 1966). Although few data exist on the practical engineering and installation aspects of air injection, this interdictive technique is, in theory, feasible in certain situations. Air injection into the very permeable strata below the water table is capable of retarding flow or expanding flow into longer, lower permeability, flow paths. Air injection may be suitable as an emergency control measure in porous unconsolidiated and some porous consolidated saturated media.

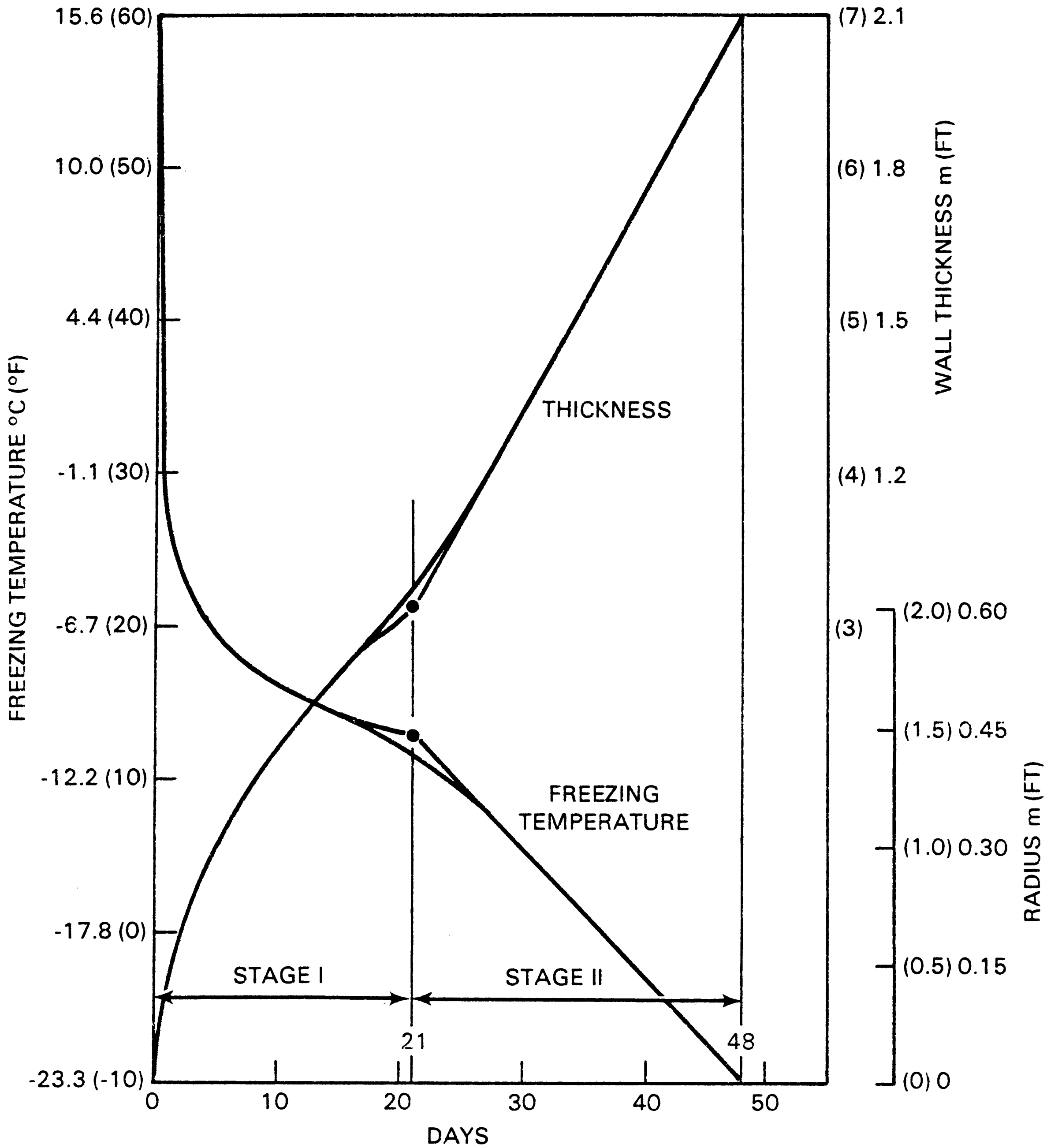

FIGURE 4.4.5-2. Frozen Straight Wall Development in Coarse-Grained Soil (Source: Sanger 1968)

The physical phenomenon of air retarded ground-water flow in a porous media relates to the energetics of adsorption and surface tension which constrain the water to the smaller (and sometimes less conductive) pore spaces. More particularly, through air injection a multiphase or partially saturated flow system is induced. The significant reduction in hydraulic conductivity with greater capillary pressure brings about the slowing and spreading of the flow system. The larger and more conductive parts of the pore channels are reserved for air flow. As the air pressure is increased, the capillary pressure increases causing the air-water interface to retreat to smaller and smaller pores. By significantly increasing the air pressure, the effective water premeability of the material can be reduced to near zero (Nelson 1966).

4.4.6.1 Design and Construction Considerations for Air Injection Systems

The most important issues concerning air injection feasibility as a ground-water interdiction strategy are (Nelson 1966):

1. Required air injection pressures,
2. Quantities of air flow,
3. Types of injection configurations, and
4. Effects of variations in host material properties.

These four items can be determined using traditional multiphase fluid flow models.

Nelson (1966) reports that greater than a thousand fold reduction in the velocity of water borne contaminants can be achieved with air injection pressures of 1.4 kg/cm^2 to 1.8 kg/cm^2 and air flow rates between 0.14 and 0.23 m^3/min/per approximately 9.0m^2 of area. It is suggested that to be most effective in retarding or diverting ground-water flow air should be injected into the most permeable material in the saturated zone within which the radionuclides are traveling. Although air injection in less permeable strata may reduce air flow rates it is not as effective in reducing ground-water flow.

The installation of an air injection system requires a layout similar to that required for grouting or ground freezing. Holes must be drilled to the required depth of injection and appropriate injection apparatus installed. Readily available construction grade air compressors can be used thus eliminating expensive mobilization of special purpose equipment.

4.4.6.2 Implementation Considerations for Air Injection Systems

There is little field experience in utilizing air injection to retard the movement of ground water through a porous media and the technique is energy intensive. Air injection pressures must be maintained if the reduction in water permeability is to be maintained. As is the case with other ground-water interdictive techniques, implementation considerations include time to install a system, its corresponding cost and worker safety. This technique is very competitive with other mitigative approaches because of the lack of need for special and/or deliverable materials (e.g., grouting compounds). Also, no special equipment is required for air injection systems (Nelson 1966). Long-term maintenance of the reduced water permeability does require a moderate energy consumption for compressor operation for continuous air injection. In comparison with the time necessary to develop a frozen cut-off air injection provides an almost immediate influence on the movement of ground water in the region affected by the air flow. Air injection may only be suitable as a temporary emergency strategy to quickly divert ground-water flow away from a sensitive area or permanent barrier construction site. There is a safety consideration associated with air in contact with the contaminant. The air may capture some of the radionuclides. This portion of the contaminants may then be returned to the surface as the aquifer de-aerates along the fringe of the air barrier and/or when the system is terminated.

4.5 U.S. GEOTECHNICAL ENGINEERING CAPABILITY

The following table (Table 4.5-1) of U.S. geotechnical engineering capability was developed from a review of geotechnical engineering literature and a letter survey of geotechnical design and construction firms identified in trade journals. The list of firms is not complete but it is representative of the experience and capability of U.S. firms related to implementation of the various ground-water contaminant mitigation techniques.

TABLE 4.5-1. U.S. Geotechnical Engineering Capability

I. Hydrogeologic Site Investigation[a]

1. Dames and Moore, Inc.
Los Angeles, California
2. Donohue and Associates, Inc.
Sheboygan, Wisconsin
3. Geraghty and Miller, Inc.
Syosset, New York
4. Grout Water Associates, Inc.
Westerville, Ohio
5. Hayward Baker Company
Odenton, Maryland
6. James M. Montgomery, Inc.
Pasedena, California
7. JRB Associates, Inc.
McLean, Virginia
8. SCS Engineers, Inc.
Long Beach, California
9. STS Consultants, Ltd.
Northbrook, Illinois
10. Sverdrup and Parcel, Inc.
St. Louis, Missouri
11. TAMS
New York, New York
12. Woodward-Clyde Consultants
San Francisco, California
13. D'Appolonia
Pittsburgh, Pennsylvania
14. Metcalf and Eddy, Inc.
Boston, Massachusetts
15. CH2M-Hill
Portland, Oregon

II. Grout Curtains

1. American Cyanamid Company
Princeton, New Jersey
2. Burgess and Niple, Inc.
Columbus, Ohio
3. Cementation Company of America, Inc.
Tucson, Arizona
4. Diamond Chemicals
Cleveland, Ohio
5. Halliburton Company
Duncan, Oklahoma
6. Hayward Baker Company
Odenton, Maryland
7. Layne New York Company
Pittsburgh, Pennsylvania
8. Mitsubishi International Corporation
Chicago, Illinois
9. Mueser, Rutledge, Johnston and Desimone
New York, New York
10. Pressure Grout Company
Doly City, California
11. Raymond International Builders, Inc.
Pennsauken, New Jersey
12. Stang - Cofor, Inc.
Crange, California
13. STS Consultants, Ltd.
Northbrook, Illinois
14. W. G. Jaques Co.
Des Moines, Iowa

III. Slurry Trenches

1. American Colloid Company
   Skokie, Illinois

2. Bencor Corporation of America
   Dallas, Texas

3. Case International Company
   Chicago, Illinois

4. Engineered Construction International, Inc.
   Pittsburgh, Pennsylvania

5. GEO-CON, Inc.
   Pittsburgh, Pennsylvania

6. ICOS Corporation of America
   New York, New York

7. International Minerals and Chemical Corp.
   Mundelein, Illinois

8. Moretrench American Corp.
   Rockaway, New Jersey

9. Mueser, Rutledge, Johnston, and Desimone
   New York, New York

10. Raymond International Builders, Inc.
    Pennsauken, New Jersey

11. Soletanche and Radio, Inc.
    McLean, Virginia

12. STS Consultants, Ltd.
    Northbrook, Illinois

13. Thatcher Engineering Corp.
    Gary, Indiana

14. White, Morrison-Knudson, Mergantine
    Newton, Massachusetts

(a) There are many competent geotechnical consulting firms throughout the U.S. that can perform hydrogeologic site investigations. This list presents several representative firms.

IV. Steel Sheet Piling

Most heavy construction companies throughout the U.S. are experienced with the design and construction of steel sheet piling cut-offs.

V. Well Design/Drilling

The 1983 National Water Water Well Association Membership Directory lists several hundred well drilling contractors and equipment suppliers.

VI. Ground Freezing

1. Cementation Company of America, Inc.
   Tucson, Arizona

2. Engineered Construction International, Inc.
   Pittsburgh, Pennsylvania

3. Frontier-Kemper Constr.

4. Geofreeze Corp.

5. Moretrench American Corp.
   Rockaway, New Jersey

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5.0 MITIGATIVE TECHNIQUES FOR GENERIC SITES

5.1 ANALYSIS OF PRE-MITIGATIVE CONTAMINANT DISCHARGE

The release of radionuclides and subsequent transport to an accessible environment without mitigative action are discussed in this section. The release of strontium-90, cesium-137 and ruthenium-106 following a severe accident are used as indicators of the severity of contamination via the ground-water pathway. Existing and proposed nuclear power plant sites are individually characterized by one dimensional transport analysis and the results are presented in composite by generic hydrogeological classification. The nomenclature adopted in this report uses the term "release" in reference to leach or sump water release of radionuclides from the core melt debris, the term "discharge" is used in reference to radionuclides discharged from the ground-water flow system to an accessible environment.

The analysis of pre-mitigative discharge fluxes for the generic classifications is intended to provide a generic representation of the data trends and extremes that are anticipated in the event of a severe accident. Individual sites are discussed only as examples of generic liquid pathway responses to a core melt accident. The reader is cautioned that this generic analysis is not intended to provide precise transport results of any specific site. The radionuclide activities and first contaminant discharge times at specific sties may be in error by an order of magnitude or more due to simplifying leach rate assumptions, geohydrologic data base estimates, and one-dimensional contaminant transport analysis limitations.

The purpose and value of the pre-mitigative analysis of radionuclide discharges is to demonstrate in a generic fashion the general time constraints and activity magnitudes of contaminanted ground-water discharges that are possible following a core melt accident. Knowledge of the generic range of contaminant arrival times and activities at the nearest downstream surface water body provides a screening to determine, in general, the necessity of contaminant interdiction for one hydrogeoloic classification versus another. Analysis of a specific site for the evaluation of the need of mitigative measures can only be properly addressed in a case study format. A description of the pre-mitigative discharge fluxes is provided for each generic site classification.

The generic characteristics of a hydrologic nature (i.e., effective porosity, hydraulic gradient, etc.) determine the average linear ground-water velocity for each site. The indicator radionuclides (i.e., strontium-90, cesium-137 and ruthenium-106) have individual decay rates and individual degrees of retardation by sorption. Hence, the amounts and respective ratios of the indicator radionuclides and radionuclides discharged to accessible environments via the ground-water pathway demonstrate the generic nature of a contaminant release for each hydrogeologic classification. If generic groupings or central tendencies are not observed in the transport results, the analysis indicates that site specific conditions (i.e., travel distance) are more important than generic hydrogeology.

Important to the pre-mitigative analysis are the questions: 1) does the contaminant decay to insignificant levels prior to discharge? 2) when does the first contaminant arrive at an accessible environment? 3) how long does contaminant continue to be discharged? and 4) when does the discharge attenuate to low levels? The first question in the analysis focuses on what constitutes a significant radionuclide release. A release into a ground-water system will always be a part of the hydrologic cycle and eventually reach a surface water body or other accessible environment. The time necessary to transport the contaminant can be as short as a few weeks to time periods measured in hundreds of thousands of years. Long transport times allow decay processes to reduce the radionuclide activity to virtually background levels. Therefore, a time limit is useful to discriminate between immediate releases of appreciable quantities of radionuclides and late arrivals of insignificant quantities.

In this study, a particular radionuclide discharge is considered significant if it occurs prior to 40 half-lives of decay. The time period of 40 half-lives allows radioactive decay to reduce the initial activity by a factor of 9.09×10^{13}. The 40 half-life time period of indicator radionuclides is also a long enough time span in which a detailed hydrologic investigation could be completed before first discharge to surface water. For the radionuclides of interest, the 40 half-life time periods are: 1,128 years for strontium-90, 1,209 years for cesium-137, and 40.4 years for ruthenium-106. For example, if the entire core inventory of strontium-90 (3.71×10^{18} pCi) is instantaneously released into the ground water, the strontium-90 activity after 40 half-lives would be 3.37×10^{6} pCi. If the entire remaining amount of the initial inventory were discharged instantaneously to a surface water body, (assuming no mechanical dispersion or molecular diffusion) the resulting activity would be within the limits of Title 10 of the Code of Federal Regulations, Part 20 of 300 pCi/ℓ if the volume of the water body was greater than 1.12×10^{4} ℓ (3.97×10^{2} ft^3). This minimum volume of water needed to dilute the strontium-90 activity to permissible levels is comparable to a small pond even under very conservative assumptions of release and transport. The purpose of setting the 40 half-lives limit for a significant release is to screen sites from consideration that do not require activities to mitigate imminent environmental consequences of a core melt accident. The indicator radionuclides were selected for mobility and long half-lives, therefore they would be the first to be discharged at the highest activities. After 40 half-lives of decay, the necessity of mitigative action is dependent on site specific characteristics of the accessible environment and associated human factors. The percentage of sites for each generic classification where over 40 half-lives occur prior to surface water discharge is presented as the first level of analysis in Sections 5.2 through 5.7.

The next key characteristic of the generic examination is the first arrival time of contaminant. This time is important as it represents the maximum time available for the implementation of a mitigative technique. If the first arrival of contaminant is shortly after release, there may be insufficient time to implement adequate and/or cost effective mitigative measures. Conversely, a long delay for arrival of the first contaminant provides greater time for site studies of plume migration and local hydrogeologic conditions before initiating mitigation activities. The spread time or time over which contaminant continues to arrive at the accessible environment is also a factor in mitigation. Long spread times of hundreds of years require mitigative measures that possess durability and performance characteristics sufficient to continue the mitigative technique for as long as release rates are above acceptable limits. Long spread times are associated with leaching of core melt masses. Short spread times, as might be the case for sump water releases, will concentrate the contaminant in space and time. Mitigative techniques for this type of release would be spatially dependent on current plume location in contrast to a core melt leachate plume which would extend continuously from the source to the accessible environment.

Another key characteristic of this generic study is the radionuclide discharge rate to accessible environments, particularly surface water. The capacity for mixing and dilution of a surface water body is an important consideration when evaluating the severity of an accident. The generic discharges to surface water are given in flux units (i.e., pCi/yr) so that the magnitude of a release can be examined in relation to the size and uses of the receiving water body.

The radionuclide fluxes are given for both leaching of the solid core melt mass and liquid sump water releases. A core melt mass may be expected to release radionuclides by leaching for both boiling water reactors (BWR) and pressurized water reactors (PWR). Liquid release of sump water used in emergency cooling is expected to be important primarily for PWR's. Therefore, accidents at BWR sites are described, in general, by release from the core melt mass. Although there may be liquid releases from BWR's, PWR accidents may have both a sump water and core melt mass component. This study examines the effects of a sump water and core debris leach release at each site regardless of reactor type. A discussion of leach and sump water releases to ground water is presented in Section 2.0.

5.2 GENERIC SITE: FRACTURED CONSOLIDATED SILICATES - CRYSTALLINE

5.2.1 Pre-Mitigative Contaminant Discharge

5.2.1.1 Significant Radionuclide Discharge

The first level of analysis of the pre-mitigative contaminant discharge involves determination of the percentage of fractured consolidated silicates sites where a discharge to surface water is calculated to occur prior to a 40 half-life time limit. The three radionuclides used as indicators of contaminant discharge (i.e., strontium-90, cesium-137 and ruthenium-106) travel in the ground-water flow system at different rates and have individual rates of decay. Therefore, not all of the radionuclides would discharge to surface water prior to 40 half-lives of decay. The percentage of individual sites with a calculated significant release is presented in Figure 5.2.1-1. In the fractured consolidated silicate-crystalline classification, 94% of the sites have a

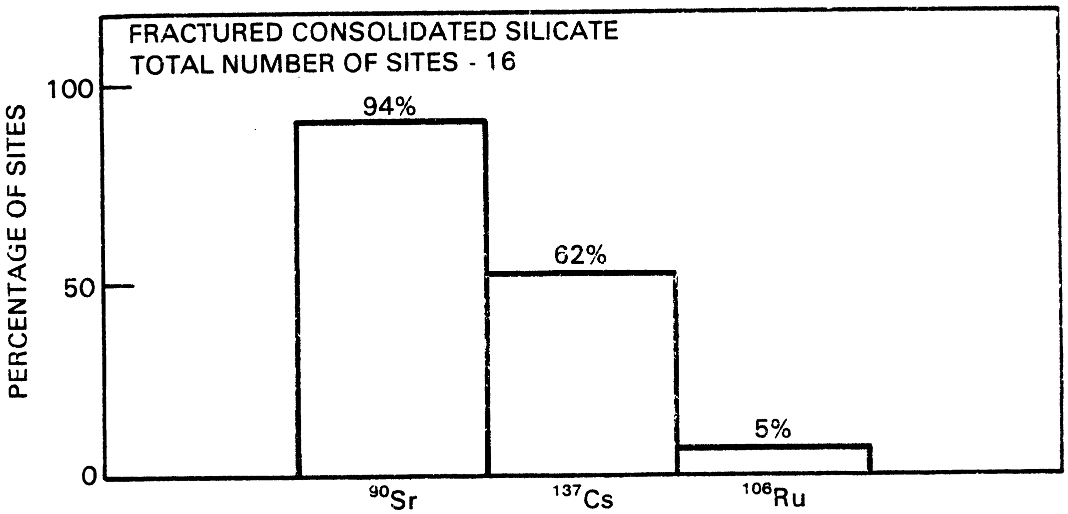

FIGURE 5.2.1-1. Percentage of Fractured Consolidated Silicates-Crystalline Sites That Would Discharge Each of the Indicator Radionuclides Prior to 40 Half-Lives of Decay

significant strontium-90 discharge while only 5% experience a significant ruthenium-106 discharge. The difference in these two percentages is, in part, due to the longer half-life of strontium-90 (28.1 years) as compared to a half-life of 1 year for ruthenium-106. For ruthenium-106 to be discharged at so few sites before 40 half-lives have occurred indicates short contaminant travel times are feasible at only a limited percentage of sites in this generic classification.

Although cesium-137 has a somewhat longer half-life than strontium-90, sorption delays cesium migration about 20 times more effectively in this generic classification. Cesium-137 is discharged before 40 half-lives at 62% of the sites. As compared to all other generic hydrogeologic classifications, fractured consolidated crystalline silicates exhibit the second highest percentage of calculated significant radionuclide discharges.

5.2.1.2 Core Melt Leachate Discharge to Surface Water

The leaching of the core melt mass would release radinouclides into the ground-water flow system where they would be transported toward a surface water body. The flux of radionuclide activity at the point of contact with a surface water body is given in Figure 5.2.1-2. The time scale in years represents the time after the beginning of core melt leaching. The heat generated in the core melt mass is expected to delay ground-water contact with the core melt for up to one year as discussed in Section 2.1.1. Strontium-90 has a slower rate of decay than ruthenium-106 and hence, retains its activity over a longer span of time. The first arrival of contaminant is indicated by the left most line perpendicular to flux/year curve. The flux/year curve is not extended past a lower limit of 1 pCi/yr as a plotting convenience.

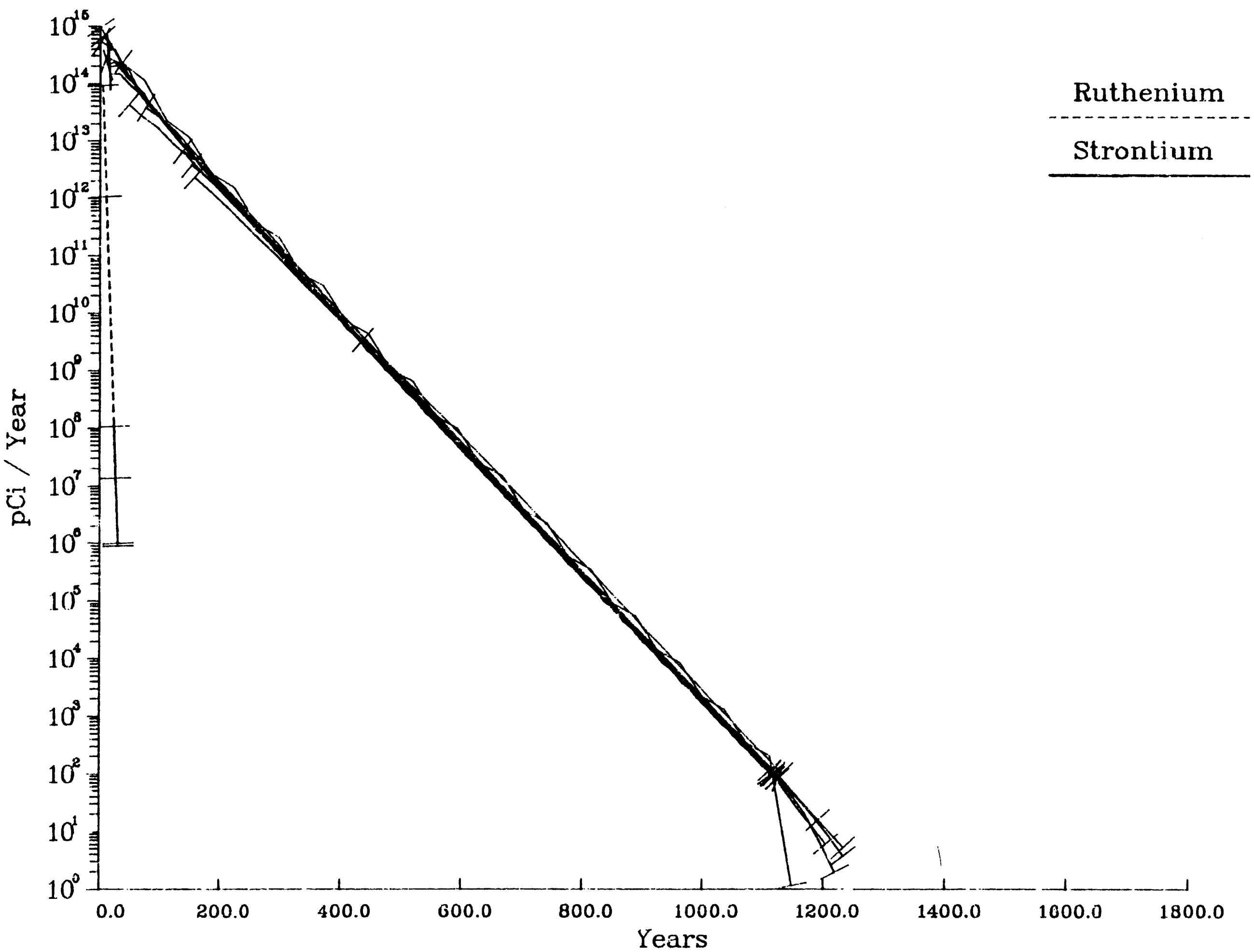

FIGURE 5.2.1-2. Discharge Flux of Core Melt Leachate from Fractured Consolidated Silicates-Crystalline Sites to Surface Water for Strontium-90 and Ruthenium-106

Strontium-90

The first calculated arrival times for strontium-90 are spread between 0.9 years and 150 years. One site has an extreme first arrival of contaminant at the nearest surface water at 430 years. The fractured consolidated silicate-crystalline classification exhibits a major grouping of strontium-90 arrival times at about 10 years. This result is not unexpected in a fractured flow system where sorption is hindered and hydraulic conductivities and effective porosities favor contaminant transport. The remaining sites not contained in the major grouping have calculated first arrival times scattered between 20 and 150 years.

The strontium-90 flux for the major grouping of first arrivals ranges between 1×10^{15} and 1×10^{14} pCi/yr. All remaining sites, except one, have strontium-90 fluxes to a surface water body of more than 1×10^{12} pCi/yr.

Ruthenium-106

Ruthenium-106 leaching from a silicic core melt has calcualted first arrival times somewhat later than strontium-90 due to the greater retardation of ruthenium-106 by sorption. The dashed line in Figure 5.2.1-2 represents ruthenium-106 arrivals. Ruthenium decays to insignificant levels after 40 years of transport. The first contaminant arrival is at 4.6 years with a flux of 1×10^{14} pCi/yr for the most rapid site. Later first contaminant arrivals of ruthenium-106 occur at 10 and 23 years. The remaining sites in this classification do not discharge ruthenium prior to 40 half-lives of decay. Such discharges would result in fluxes below 1×10^{3} pCi/yr.

5.2.1.3 Sump Water Discharge to Surface Water

Liquid release of sump water used in reactor cooling is feasible in a core melt accident involving pressurized water reactors. The amount of liquid released would be power plant and accident-sequence specific. In any case, the release of liquid into the ground-water system would be many orders of magnitude faster than the leaching of the core melt debris. Cesium is noteworthy because sump water releases would contain roughly 4.67×10^{18} pCi of cesium-137 which has a half-life of 30.2 years. The radionuclide fluxes for the indicator contaminant are prsented in Figure 5.2.1.-3. Only the initial peak radionuclide flux is given for sump water releases because the duration of these releases is relatively short. The permeability in and around the core melt mass and the hydraulic pressure head would control the liquid release rate of sump water.

Strontium-90

The discharge fluxes calculated for strontium-90 show a cluster of arrival times at 0.9 to 40 years which includes 60% of the sites. The remaining sites have first arrival times spread out over 400 years after release. The fluxes are greater than anticipated for the core melt mass which would be a characteristic of sump water releases. The maximum flux of the sites is 6×10^{16} pCi/yr. The lowest value in the clustered data is 2.5×10^{15} pCi/yr. The remaining strontium discharges range from 1×10^{10} to 3×10^{15} pCi/yr. Other than the clustering of points at times less than 40 years there is no trend or generalization evident in the data except for the decreasing flux with time due to radioactive decay.

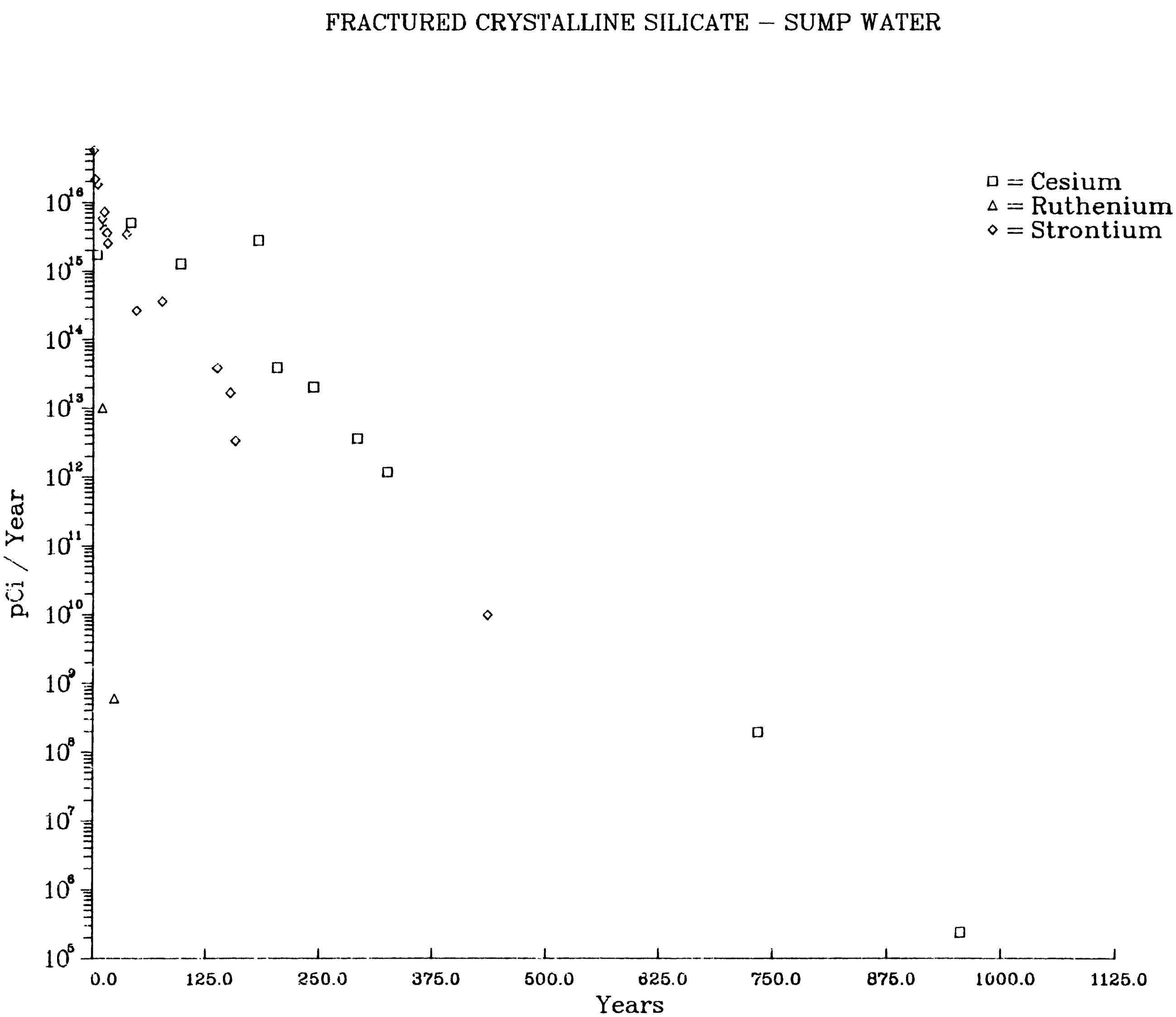

FIGURE 5.2.1-3. Discharge Flux of Reactor Sump Water from Fractured Consolidated Silicates-Crystalline Sites to Surface Water for Strontium-90, Cesium-137 and Ruthenium-106

Cecium-137

Cesium is more strongly retarded than the other indicator radionuclides and arrives at the surface water body last in the contaminant stream. The arrival times do not group but follow a linear trend from 18.4 to 950 years. Peak flux is 1 x 10^{17} pCi/yr at 18.4 years. Generally, the cesium fluxes are greater than those of strontium when travel times to the surface water body are greater than 50 years. The sump water discharge of cesium-137 exhibits the greatest flux value in the fractured consolidated silicates classification.

Ruthenium-106

Ruthenium-106 decays to low levels before discharge into a surface water body at most of the sites in this generic classification. The peak flux is 1 x 10^{13} pCi/yr or about 3.5 orders of magnitude less than strontium-90.

5.2.2 Mitigative Techniques for Fractured Consolidated Silicates

Table 5.2.2-1 presents a matrix of ground-water contaminant mitigative techniques versus feasibility of implementation at fractured consolidated silicates sites. Constraints on feasibility as they relate to this generic site are also briefly summarized in the table.

5.3 GENERIC SITE: FRACTURED AND SOLUTIONED CONSOLIDATED CARBONATES

5.3.1 Pre-Mitigative Contaminant Discharge

5.3.1.1 Significant Radionuclide Release

The analysis of pre-mitigative contaminant discharge involves determining the percentage of sites where a calculated discharge to surface water occurs prior to 40 half-lives of decay. Any discharge of the indicator radionuclides before 40 half-lives of decay is considered significant. Radionuclide discharges calculated for the fractured and solutioned consolidated carbonate classification demonstrate the high transport rates that are feasible in this type of flow system. The percentage of sites with an anticipated discharge to surface water before the 40 half-life time limit is shown in Figure 5.3.1-1. Significant amounts of strontium-90, cesium-137, and ruthenium-106 remain to be discharged from 83, 58 and 33% of the sites, respectively. This is the highest average percentage of sites of all generic classifications. The factors which produce these significant discharges are the shortest average distance to surface water and the highest average hydraulic conductivity of all the generic classifications.

It is also noteworthy that despite hydrogeologic conditions which produce short contaminant transport times, a long-lived radionuclide such as strontium-90 does not discharge prior to 40 half-lives at 17% of the sites.

This demonstrates that within generic classifications that are favorable to rapid contaminant discharge, there are individual sites that do not have the potential for concentrated radionuclide discharges to surface water.

5.3.1.2 Core Melt Leachate Discharge to Surface Water

The contact of ground water with the core melt debris would initiate leaching and release of contaminant to the ground-water flow system. Carbonate rock when melted forms a calcine material that leaches at a faster rate than a silica melt. Therefore, this generic classificaiton (i.e., fractured and solutioned carbonates) has the chemical and hydraulic potential for the largest radionuclide fluxes from core melt leaching. The flux of the indicator radionuclides is given in Figure 5.3.1-2. As previously described, the initial

TABLE 5.2.2-1. Mitigative Techniques for Fractured Consolidated Silicates

Mitigative Technique	Feasibility	Constraints on Feasibility
1. Grouting: 1a) Particulate/ Cement-based 1b) Non-particulate/ Chemical	Fissure grouting Fracture grouting	• Joint gaps between 0.5 mm and 6.0 mm for cement-based grouts. • Fissure width up to 10 cm - 15 cm can be grouted w/acrylamide-based grouts.
2. Slurry Trenches: 2a) Soil bentonite (S-B) 2b) Cement bentonite (C-B) 2c) Lean concrete (L-C) 2d) Vibrating beam (VBT)	Infeasible	• Excavation prohibited by competent rock.
3. Steel Sheet Piling	Infeasible	• Pilings cannot be hard-driven through consolidated media.
4. Ground-Water Withdrawal for Potentiometric Surface Adjustment: 4a) Prevent discharge to receiving stream 4b) Prevent water table contact w/core melt mass. 4c) Prevent contamination of leaky aquifer	Marginally feasible	• Shallow aquifers preferable. • Ground-water system response may be prohibitively slow due to relatively low hydraulic conductivity. • Definition of fracture system is necessary. • Drilling costs may be high. • Detailed hydrogeologic studies of in complex flow system required to determine feasibility (i.e., difficulty arises in determining radii of influence of wells in fractured media). • Proper handling required for contaminated water brought to the surface.
5. Ground-Water Withdrawal and/or Injection for Contaminant Plume Control: 5a) Withdrawal and injection 5b) Withdrawal without injection 5c) Withdrawal and recharge 5d) Injection	Feasible	• Definition of fracture system required. • Drilling costs may be high. • Ground-water system response may be prohibitively slow due to relatively low hydraulic conductivity. • Detailed hydrogeologic studies of complex flow system required to determine feasibility. • Fracture system may enhance performance by concentrating contaminants.

TABLE 5.2.2-1. (contd)

		• Proper handling required for contaminated water brought to the surface.
6. Interceptor Trenches	Infeasible	• Excavation prohibited by competent rock.
7. Permeable Treatment Beds	Infeasible	• Excavation prohibited by competent rock.
8. Ground Freezing	Marginally feasible	• Drilling costs may be high. • Very expensive and energy intensive. • Thermal erosion may preclude implementation. • 0.9-1.2 m/day maximum groundwater velocity. • Surface piping insulation required. • Host material saturation >10%.
9. Air Injection	Marginally feasible	• Little engineering expertise or implementation experience. • Saturated conditions required. • Drilling costs may be high. • Energy intensive. • Air bleeding of contaminants.

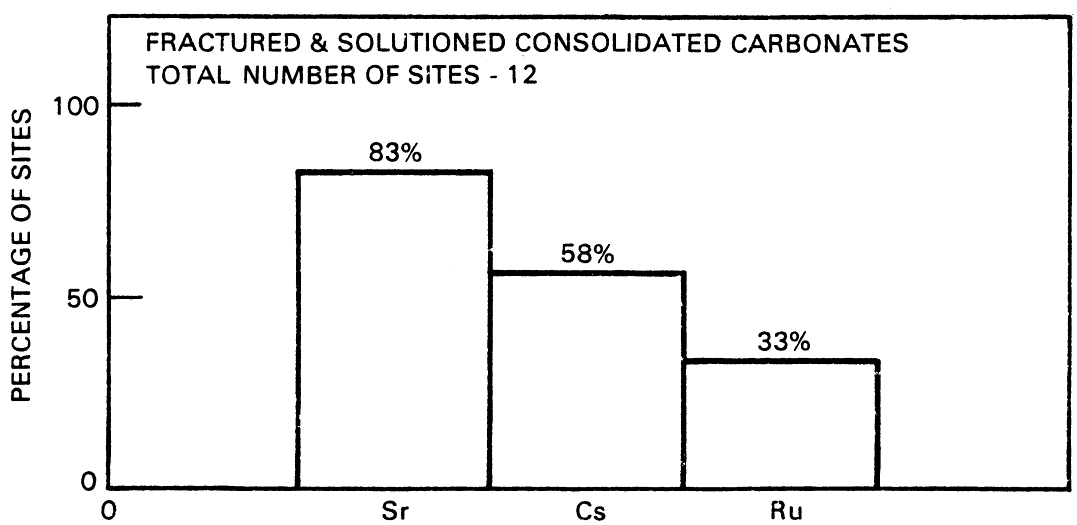

FIGURE 5.3.1-1. Percentage of Fractured and Solutioned Consolidated Carbonate Sites That Would Discharge Each of the Indicator Radionuclides Prior to 40 Half-Lives of Decay

FRACTURED CONSOLIDATED CARBONATES – CORE MELT

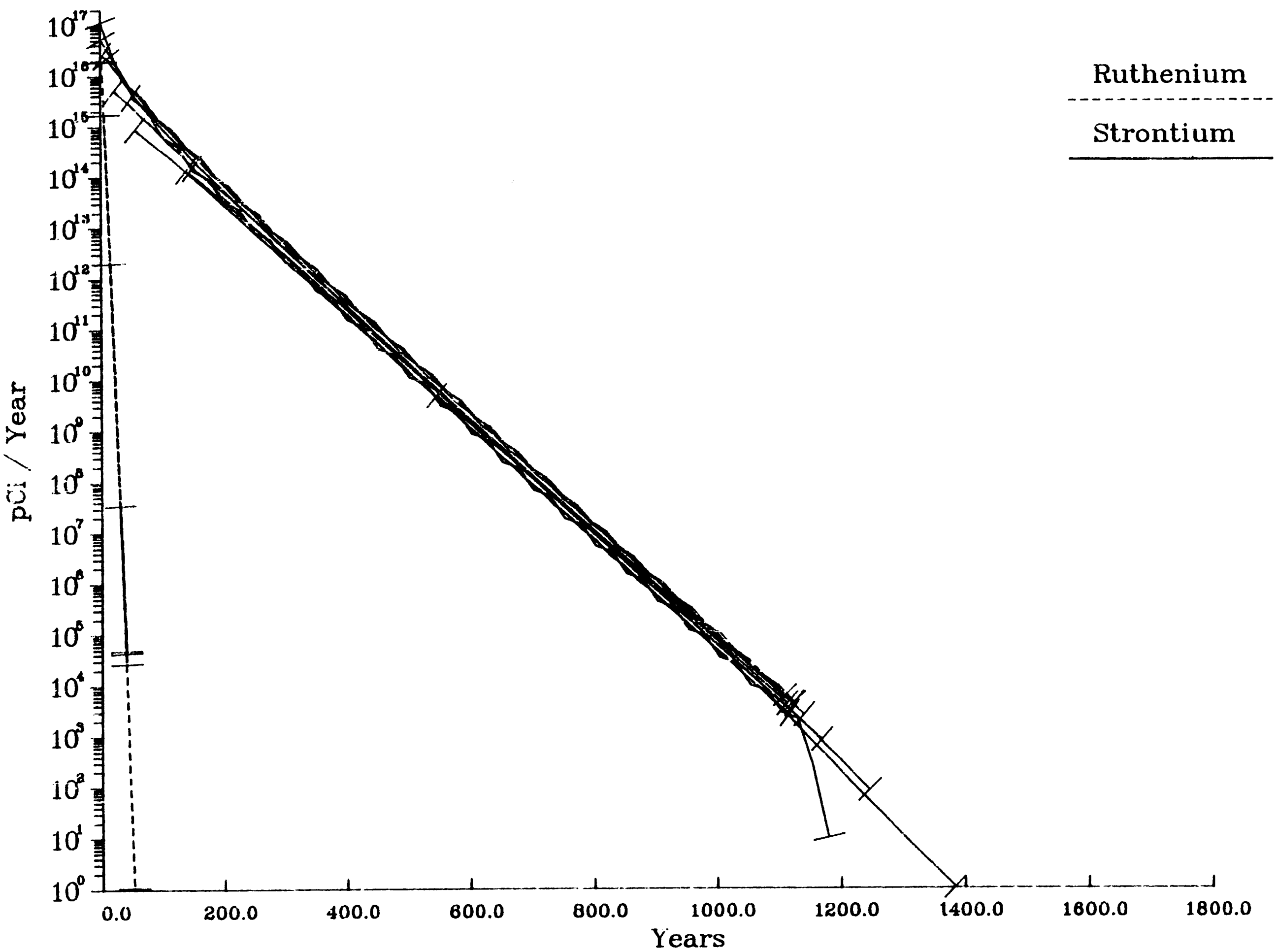

FIGURE 5.3.1-2. Discharge Flux of Core Melt Leachate from Fractured and Solutioned Consolidated Carbonate Sites to Surface Water for Strontium-90 and Ruthenium-106

contact of contaminant with surface water is indicated by a perpendicular line at the start of the flux/year curve. The flux rates are plotted to a lower limit of about 1 pCi/yr. Amounts below this level are considered insignificant.

Strontium-90

The calculated first arrival times of this contaminant are nearly all between 0.6 and 140 years. One site has a first contaminant arrival at

540 years. Of interest is the number of sites that have arrival times that are very short. Four sites have short arrival times and also have flux rates of 1×10^{17} to 1×10^{16} pCi/yr. There are three sites where contaminant is expected to reach the accessible environment prior to 60 years after the accident. The flux rate for strontium-90 is relatively high at about 1×10^{15} pCi/yr for these sites. The remaining sites have arrival times greater than 160 years and the flux rate is reduced to less than 1×10^{14} pCi/yr. Strontium-90 has a greater discharge flux rate for a core leaching release than ruthenium-106. The discharge flux for strontium-90 for this classification is the largest core melt release rate of all the generic classificatons.

Ruthenium-106

Ruthenium-106 is more strongly sorbed than strontium-90 and exits the ground-water system at later times. This feature coupled with the 1 year half-life of ruthenium prevents most sites from having a significant ruthenium release. The shortest first arrival time calculated for ruthenium is 2.2 years with a flux rate of 2×10^{16} pCi/yr. Three sites have first arrival times greater than 20 years and at greatly reduced radionuclide flux rates.

5.3.1.3 Sump Water Discharge to Surface Water

Liquid release of radionuclides included in the sump water is possible in a pressurized water reactor. The release of contaminant is not affected by the chemical rock type as it is for core debris leaching as described above. For sump water release the peak flux is the only value plotted due to the short time span of the release. The sump water radionuclide flux is presented in Figure 5.3.1-3.

Stronium-90

The sump water discharges for strontium show three sites where very short travel times can be expected. These sites have associated radionuclide fluxes of over 2×10^{17} pCi/yr which are the second highest values calculated. There are four sites that have first arrival times less than 60 years and have discharge fluxes between 4×10^{16} and 2×10^{15} pCi/yr. There is a lack of data clustering for all radionuclides in this classification which precludes generalization.

Cesium-137

Radionuclide discharge flux reaches the overall highest calculated value for a cesium-137 sump water release in fractured and solutioned carbonates. The peak flux occurs at 2.6 years with a value of 2.5×10^{17} pCi/yr. The next arrival time of cesium is at 7.1 years at a flux rate of 1.7×10^{17} pCi/yr. The remaining sites lie along a generalized decay curve without a clustering of data values.

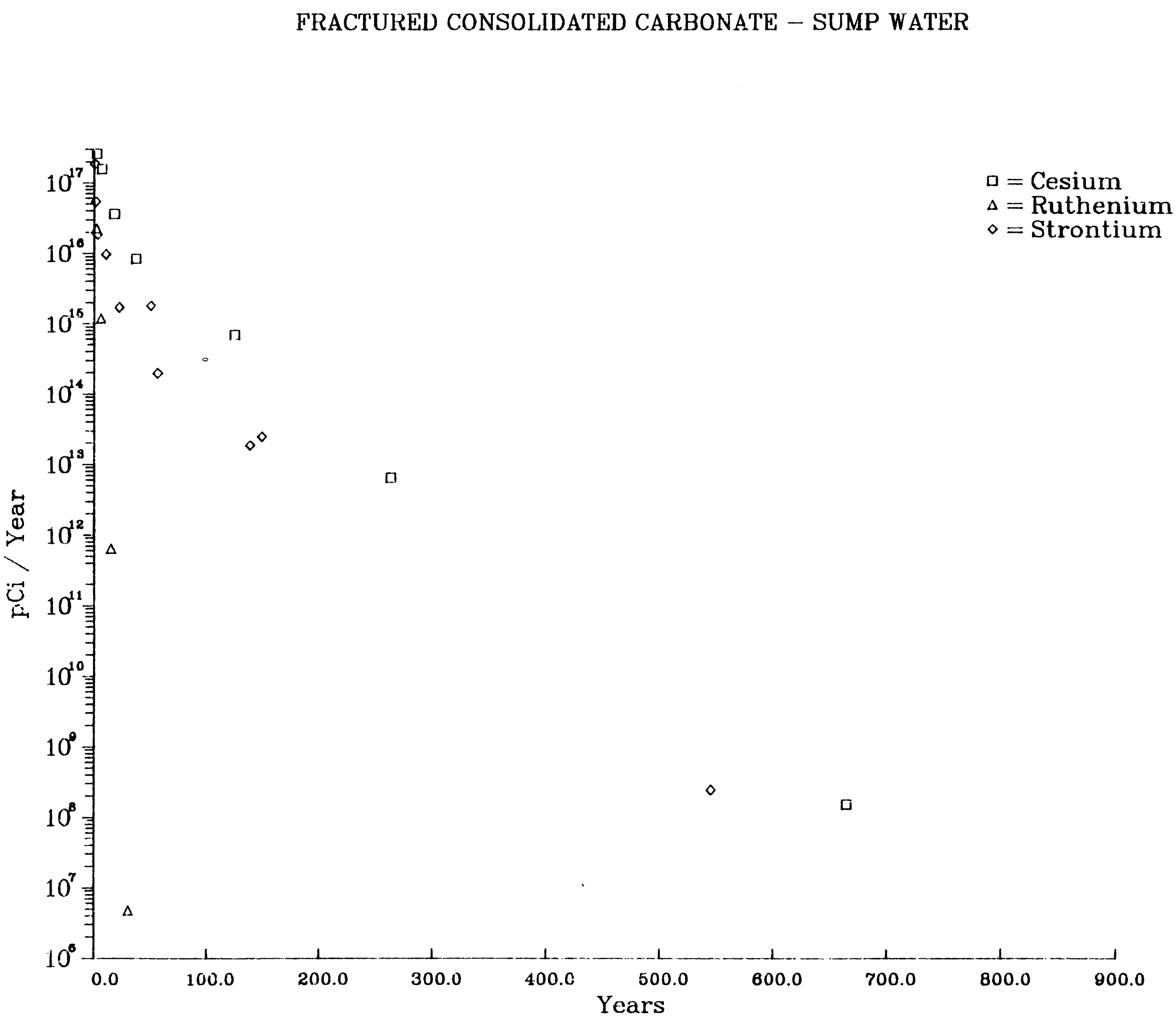

FIGURE 5.3.1-3. Discharge Flux of Reactor Sump Water from Fractured and Solutioned Consolidated Carbonate Sites to Surface Water for Strontium-90, Cesium-137, and Ruthenium-106

Ruthenium-106

The ruthenium-106 flux levels are also greater than average for this generic classification. The peak value is 2 x 10^{16} pCi/yr occurring at 2.2 years. Ruthenium-106 flux levels are below 1 x 10^{15} pCi/yr for most sites.

5.3.2 Mitigative Techniques for Fractured and Solutioned Consolidated Carbonates

Table 5.3.2-1 presents a matrix of ground-water contaminant mitigative techniques versus feasibility of implementation at fractured and solutioned consolidated carbonates sites. Constraints on feasibility as they relate to this generic site are also briefly summarized in the table.

5.4 GENERIC SITE: POROUS CONSOLIDATED CARBONATE

5.4.1 Pre-Mitigative Contaminant Discharge

5.4.1.1 Significant Radionuclide Discharges

This generic classification has the smallest percentage of individual sites with calculated surface water discharges prior to 40 half-lives of decay. As shown in Figure 5.4.1-1 all of the cesium-137 and ruthenium-106 decay to insignificant levels prior to initial discharge. Strontium-90 is less retarded by sorption and consequently discharges before 40 half-lives at 20% of the site. The major factors which cause significant decay prior to discharge are: 1) a long average travel distance of 650 meters, 2) the lowest average hydraulic gradient, 3) a relative high effective porosity, and 4) a slightly lower than average hydraulic conductivity as compared to the other generic classifications. Radionuclide releases at most power plants in the porous consolidated carbonates classification would be decayed to insignificant levels while contained within the ground-water system.

5.4.1.2 Core Melt Leachate Discharge to Surface Water

There are two sites that demonstrate a significant core melt debris leachate discharge but only for strontium-90. The discharges are plotted versus time in Figure 5.4.1-2. The discharged flux of strontium-90 indicates that at one of these sites discharge fluxes are elevated with a value of 4 x 10^{15} pCi/yr arriving at 44 years. The other site has a calculated first arrival at time of 204 years at 2 x 10^{13} pCi/yr. There are an insufficient number of calcualted significant core melt releases to form any trends.

5.4.1.3 Sump Water Discharge to Surface Water

Sump water release in this generic classification is similar to core melt leachate because the only radionuclide to reach the surface water environment prior to 40 half-lives of decay is strontium-90. The peak sumpwater discharge

TABLE 5.3.2-1. Mitigative Techniques for Fractured and Solutioned Consolidated Carbonates

Mitigative Technique	Feasibility	Constraints on Feasibility
1. Grouting: 1a) Particulate/ Cement-based 2a) Non-particulate/ Chemical	Permeation grouting Fissure grouting Fracture grouting Bulk grouting	• Rapid contaminant travel time. • Joint gaps between 0.5 mm and 6.0 mm for cement-based grouts. • Major cavities may be encountered requiring massive bulk grouting. • Fissure width up to 10 cm - 15 cm can be grouted w/acrylamide-based grouts. • High ground-water velocities may prohibit grouting or require bulk fill material.
2. Slurry Trenches: 2a) Soil bentonite (S-B) 2b) Cement bentonite (C-B) 2c) Lean concrete (L-C) 2d) Vibrating beam (VBT)	Infeasible	• Excavation prohibited by competent rock.
3. Steel Sheet Piling	Infeasible	• Pilings cannot be hard-driven through consolidated media.
4. Ground-Water Withdrawal for Potentiometric Surface Adjustment: 4a) Prevent discharge to receiving stream 4b) Prevent water table contact w/core melt mass. 4c) Prevent contamination of leaky aquifer	Marginally feasible	• Because of relatively high hydraulic conductivity sufficient drawdown may not be achieved; and large withdrawal volumes may be required. • Definition of possibly complex fracture system is necessary • Detailed hydrogeologic studies required to determine feasibility (i.e., difficulty arises in determining radii of influence of wells in fractured media). • Proper handling required for contaminated water brought to the surface.

TABLE 5.3.2-1. (contd)

5. Ground-Water Withdrawal and/or Injection for Contaminant Plume Control: 5a) Withdrawal and injection 5b) Withdrawal without injection 5c) Withdrawal and recharge 5d) Injection	Feasible	• Shallow aquifers preferable. • Definition of possibly complex fracture system required. • Detailed hydrogeologic studies required to determine feasibility. • High hydraulic conductivity may require pumping of large quantities of water. • Fracture system may enhance performance by concentrating contaminants. • Proper handling required for contaminated water brought to the surface.
6. Interceptor Trenches	Infeasible	• Excavation prohibited by competent rock.
7. Permeable Treatment Beds	Infeasible	• Excavation prohibited by competent rock.
8. Ground Freezing	Marginally feasible	• Very expensive and energy intensive. • Thermal erosion may preclude implementation. • 0.9-1.2 m/day maximum ground-water velocity. • Surface piping insulation required. • Host material saturation >10%.
9. Air Injection	Marginally feasible	• Little engineering expertise or implementation experience. • Saturated conditions required. • Energy intensive. • Air bleeding of contaminants.

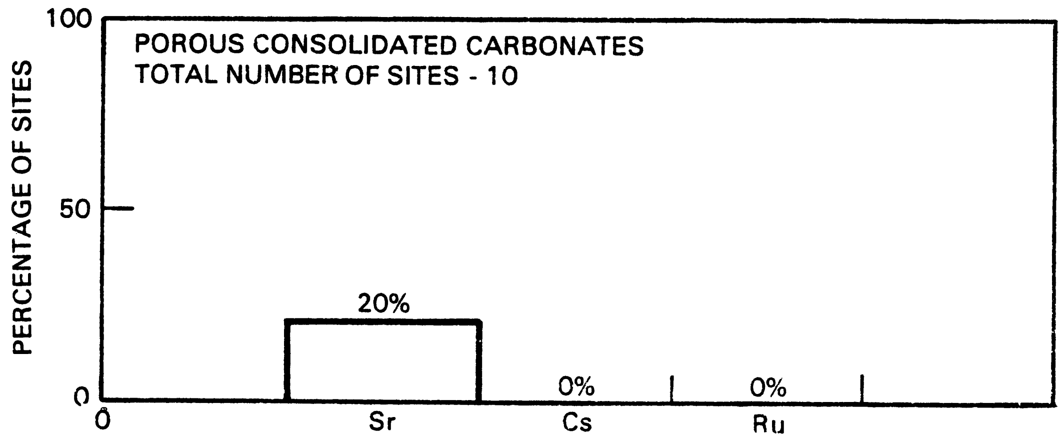

FIGURE 5.4.1-1. Percentage of Porous Consolidated Carbonate Sites That Would Discharge Each of the Indicator Radionuclides Prior to 40 Half-Lives of Decay

flux is given in Figure 5.4.1-3. The first arrival times are similar to arrivals from the core melt lechate. Flux levels are 1.2 x 10^{15} pCi/yr for the shortest arrival time of 44 years.

5.4.2 Mitigative Techniques for Porous Consolidated Carbonates

Table 5.4.2-1 presents a matrix of ground-water contaminant mitigative techniques versus feasibility of implementation at porous consolidated carbonates sites. Constraints on feasibility as they relate to this generic site are also briefly summarized in the table.

5.5 GENERIC SITE: POROUS CONSOLIDATED SILICATE

5.5.1 Pre-Mitigative Contaminant Discharge

5.5.1.1 Significant Radionuclide Discharges

The percentage of sites in this generic hydrogeologic classification where radionuclides are calculated to discharged to a surface water body before 40 half-lives of decay is shown in Figure 5.5.1-1. Strontium-90 and cesium-137 are discharged at 38% and 31% of the sites, respectively. Ruthenium-106 decays to insignificant levels for nearly all sites indicating a moderately long average ground-water travel time. The amounts of sorption for the indicator radionuclides are less in a silicate aquifer versus a carbonate aquifer. Therefore, strontium-90 and cesium-137 are not strongly sorbed and are discharged at significant amounts at nearly the same percentage of individual sites.

POROUS CONSOLIDATED CARBONATE – CORE MELT

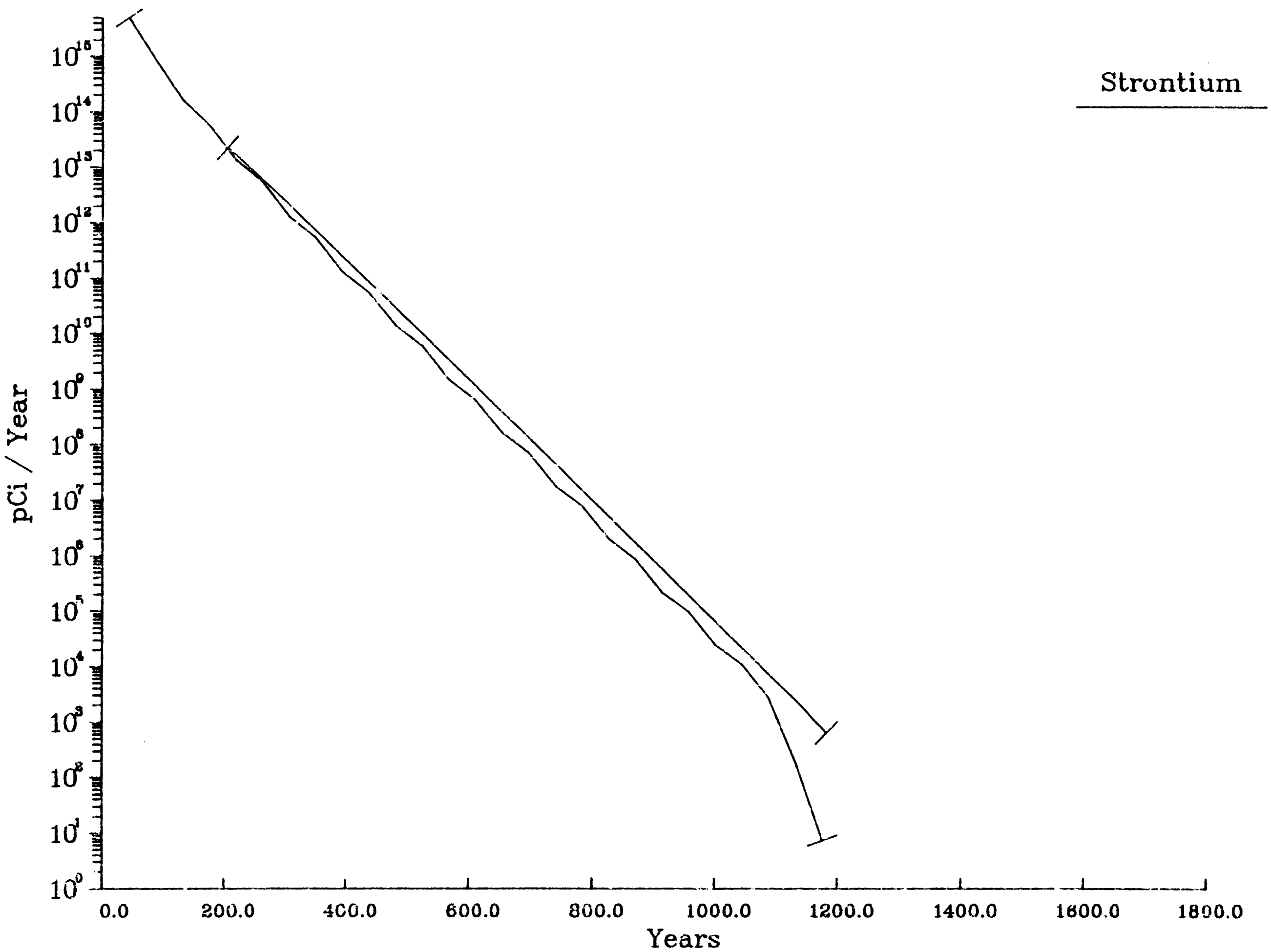

FIGURE 5.4.1-2. Discharge Flux of Core Melt Leachate from Porous Consolidated Carbonate Sites to Surface Water for Strontium-90 and Ruthenium-106

POROUS CONSOLIDATED CARBONATES

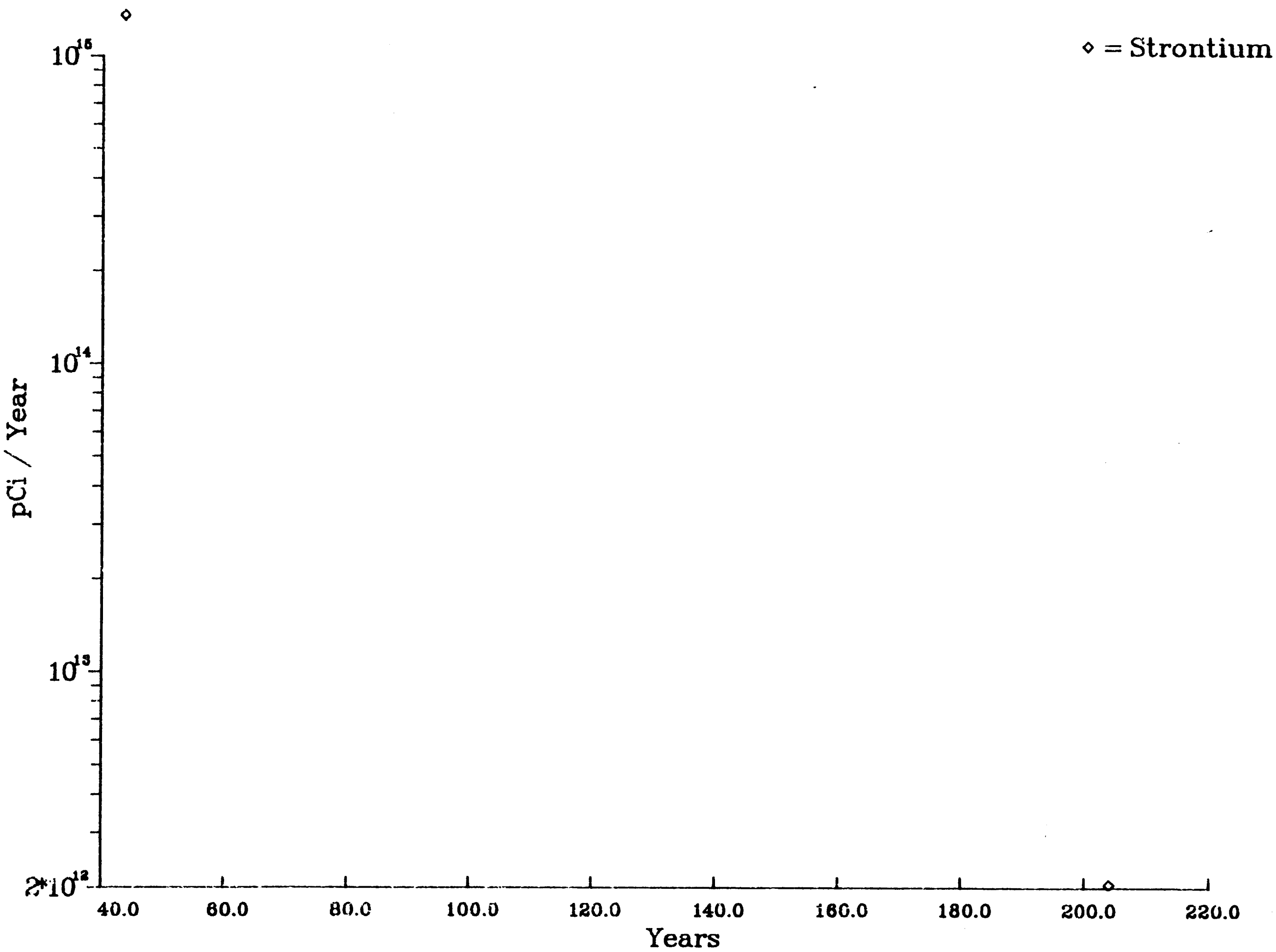

FIGURE 5.4.1-3. Discharge Flux of Reactor Sump Water from Porous Consolidated Carbonate Sites to Surface Water for Strontium-90, Cesium-137, and Ruthenium-106

5.5.1.2 Core Melt Leachate Discharge to Surface Water

When the core melt debris cools to temperatures below the boiling point of water the ground-water flow system will begin to flow through the melt mass and initiate leaching. As radionuclides leave the core melt mass they are transported via the ground-water pathway. Silicic melts, as would form in this generic classification, leach more slowly than those formed from carbonates. This generic classification is also a porous medium which has a higher effective porosity and slower transport rate than for fractured media. The core melt leachate for these sites can therefore be expected to produce attenuated fluxes to surface water. Figure 5.5.1-2 presents the calculated values for the indicator radionuclides.

TABLE 5.4.2-1. Mitigative Techniques for Porous Consolidated Carbonates

Mitigative Technique	Feasibility	Constraints on Feasibility
1. Grouting: 1a) Particulate/ Cement-based 1b) Non-particulate/ Chemical	Permeation grouting	• Host medium permeabilities: 10^{-1}-10^{-3} cm/sec* = easy 10^{-3}-10^{-4} cm/sec = moderate 10^{-4}-10^{-5} cm/sec = marginal. * Mean values for this classification. • Polyester and epoxy resins for grain sizes below 0.06 mm. • Minor solution cavities may be encountered.
2. Slurry Trenches: 2a) Soil bentonite (S-B) 2b) Cement bentonite (C-B) 2c) Lean concrete (L-C) 2d) Vibrating beam (VBT)	Infeasible	• Excavation prohibited by competent rock.
3. Steel Sheet Piling	Infeasible	• Pilings cannot be hard-driven through consolidated media.
4. Ground-Water Withdrawal for Potentiometric Surface Adjustment: 4a) Prevent discharge to receiving stream 4b) Prevent water table contact w/core melt mass. 4c) Prevent contamination of leaky aquifer.	Feasible	• Proper handling required for contaminated water brought to the surface. • Detailed hydrogeologic studies required to determine performance. • Shallow aquifers preferable.
5. Ground-Water Withdrawal and/or Injection for Contaminant Plume Control: 5a) Withdrawal and injection 5b) Withdrawal without injection 5c) Withdrawal and recharge 5d) Injection	Feasible	• Significant contaminant plume dispersion, prior to implementation, may limit performance. • Detailed ground-water flow system simulation required to determine effectiveness. • Surface handling and treatment of contaminated water must be considered.
6. Interceptor Trenches	Infeasible	• Excavation prohibited by competent rock.

TABLE 5.4.2-1. (contd)

7. Permeable Treatment Beds	Infeasible	• Excavation prohibited by competent rock.
8. Ground Freezing	Feasible	• Very expensive and energy intensive. • Thermal erosion may preclude implementation. • 0.9-1.2 m/day maximum ground-water velocity. • Surface piping insulation required. • Host material saturation >10%.
9. Air Injection	Marginally feasible	• Little engineering expertise or implementation experience. • Saturated conditions required. • Energy intensive. • Air bleeding of contaminants.

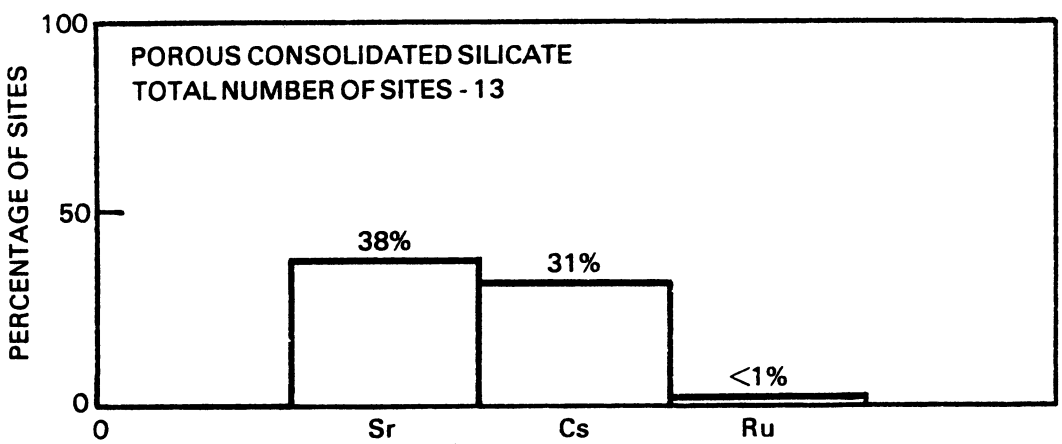

FIGURE 5.5.1-1. Percentage of Porous Consolidated Silicate Sites That Would Discharge Each of the Indicator Radionuclides Prior to 40 Half-Lives of Decay

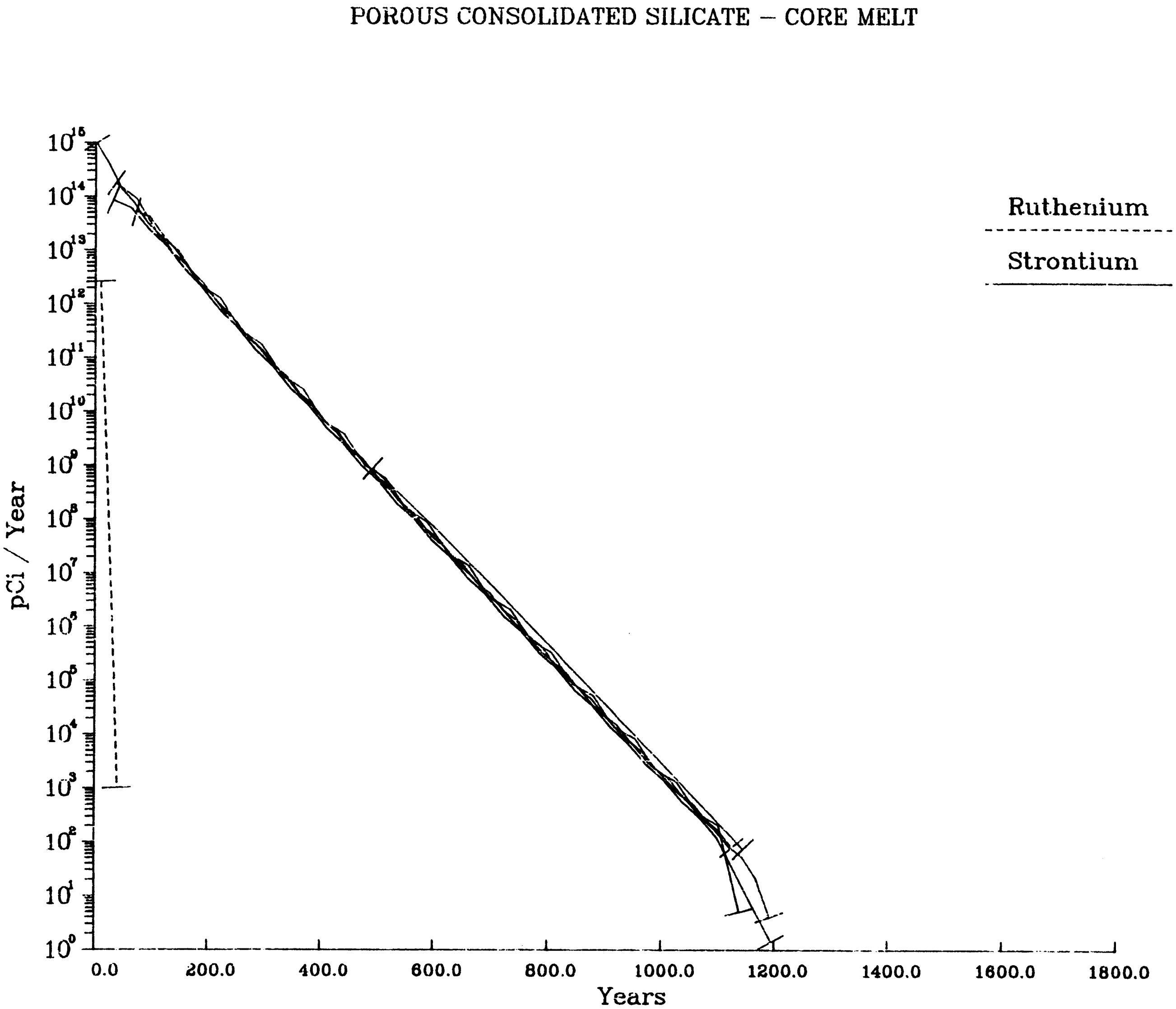

FIGURE 5.5.1-2. Discharge Flux of Core Melt Leachate from Porous Consolidated Silicate Sites to Surface Water for Strontium-90 and Ruthenium-106

Strontium-90 and Ruthenium-106

Calculated first discharge of core melt strontium-90 arrives at 2 years after release at a flux level of 1 x 10^{15} ρCi/yr. Three sites are clustered about a first contaminant arrival time of about 50 years at a flux of 8 x 10^{13} ρCi/yr. A single site has a first arrival at 475 years with a corresponding flux of 5 x 10^{8} ρCi/yr.

Ruthenium-106 arrives in significant amounts for only one site. This site is for 9.8 years at a flux of 2 x 10^{12} ρCi/yr.

5.5.1.3 Sump Water Discharge to Surface Water

Liquid release of sump water is possible for pressurized water reactors. The sump water would contain dissolved portions of the melt mass that would enter the ground-water system over a short time span. Flux values for the sump release of the indicator radionuclides are given in Figure 5.5.1-3. Sump water release rates are independent of the chemical composition of the core melt mass.

Strontium-90

Strontium-90 is the first radionuclide to discharge with an arrival time of 2 years at a flux level of 4 10^{16} ρCi/yr. There is a cluster of first contaminant arrivals at about 50 years. The sump water flux for these arrivals is approximately 1 x 10^{15} ρCi/yr which is one order of magnitude greater than the associated core melt discharge.

Cesium-137

The highest flux rate in this generic classification is for a cesium-137 sump water discharge. The peak level is 8 x 10^{16} ρCi/yr with a first arrival time of 10 years. The remaining cesium discharges arrive at times greater than 150 years and flux levels less than 2 x 10^{14} ρCi/yr.

Ruthenium-106

Ruthenium arrives in significant activities at a surface water body for a single site which is also the case for core melt leachate. The first arrival time is 9.8 years at a flux rate of 3.5 10^{13} ρCi/yr.

5.5.2 Mitigative Techniques for Porous Consolidated Silicates

Table 5.5.2-1 presents a matrix of ground-water contaminant mitigative techniques versus feasibility of implementation at porous consolidated silicates sites. Constraints on feasibility as they relate to this generic site are also briefly summarized in the table.

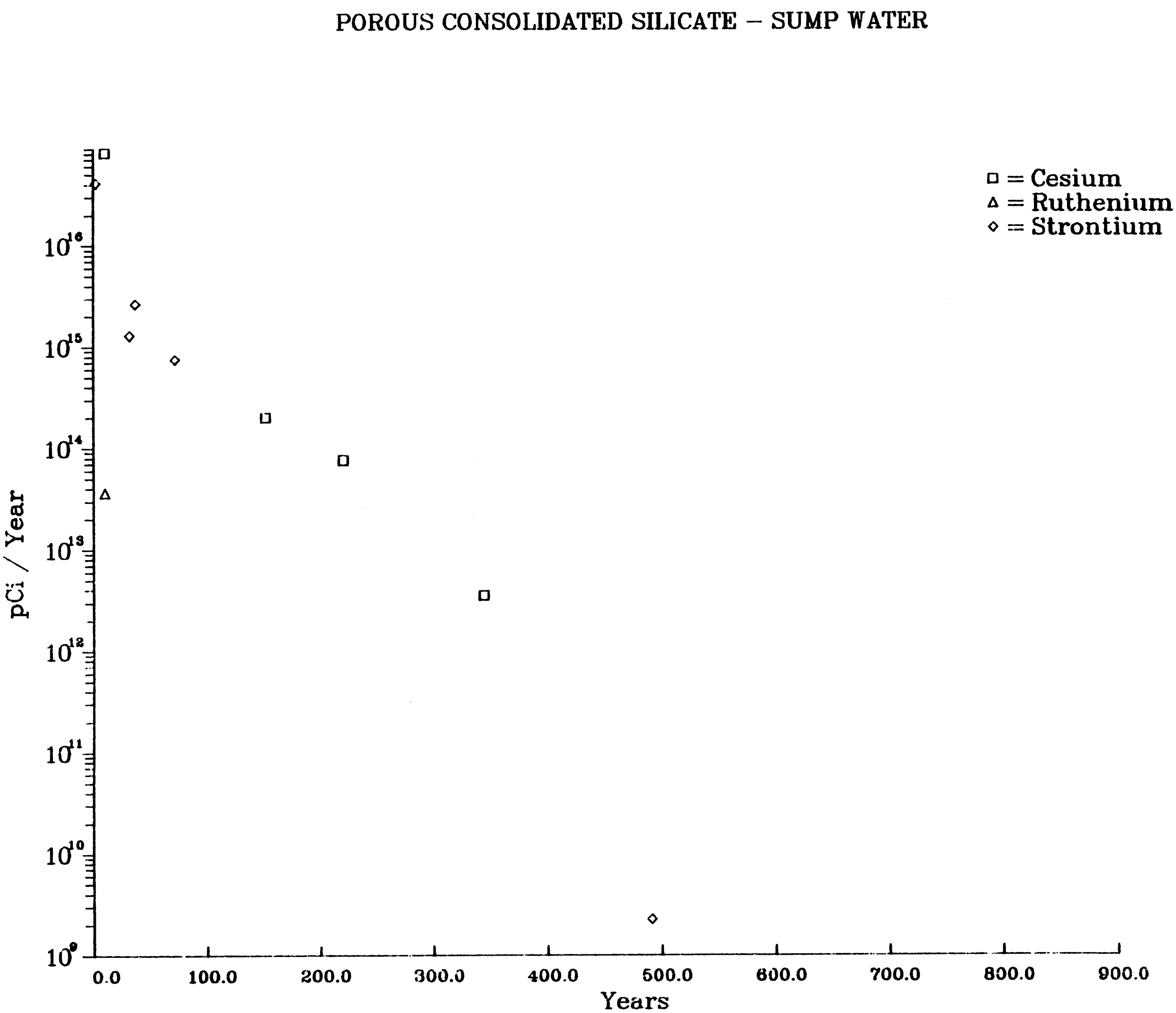

FIGURE 5.5.1-3. Discharge Flux of Reactor Sump Water from Porous Consolidated Silicate Sites to Surface Water for Strontium-90, Cesium-137, and Ruthenium-106

TABLE 5.5.2-1. Mitigative Techniques for Porous Consolidated Silicates

Mitigative Technique	Feasibility	Constraints on Feasibility
1. Grouting: 1a) Particulate/ Cement-based 1b) Non-Particulate/ Chemical	Permeation grouting	• Host medium permeabilities: 10^{-1} - 10^{-3} cm/sec = easy, 10^{-3} - 10^{-4} cm/sec* = moderate, 10^{-4} - 10^{-5} cm/sec = marginal, * Mean values for this classification • Grain size as low as 0.01 mm (silt size) can be grouted w/ acrylamide-based grouts. • Fine granular material as low as 0.10 mm (fine sand - coarse silt) can be grouted w/lignin grouts.
2. Slurry Trenches: 2a) Soil bentonite (S-B) 2b) Cement bentonite (C-B) 2c) Lean concrete (L-C) 2d) Vibrating beam (VBT)	Feasibility limited to soft, rippable competent rock. VBT technique is infeasible.	• Multiple head drills and/or percussion drills can be used to excavate trench in soft, consolidated media. • Practical limit in consolidated media is 60 m. • Ground-water velocities less than 5 cm/sec. • Host medium permeability greater than 10^{-6} cm/sec.
3. Steel Sheet Piling	Infeasible	• Pilings cannot be hard-driven through consolidated media.
4. Ground-Water Withdrawal for Potentiometric Surface Adjustment: 4a) Prevent discharge to receiving stream. 4b) Prevent water table contact w/core melt mass. 4c) Prevent contamination of leaky aquifer.	Feasible	• Proper handling required for contaminated water brought to the surface. • Filter packing of deep wells may be required. • Detail hydrogeologic studies required to determine performance. • Shallow aquifers preferable.
5. Ground-Water Withdrawal and/or Injection for Contaminant Plume Control: 5a) Withdrawal and injection 5b) Withdrawal without injection 5c) Withdrawal and recharge 5d) Injection	Feasible	• Significant contaminant plume dispersion, prior to implementation, may limit performance. • Detailed ground-water flow system simulation required to determine effectiveness. • Surface handling and treatment of contaminated water must be considered.

TABLE 5.5.2-1. (contd)

6. Interceptor Trenches	Feasibility limited to soft, rippable competent rock.	• Multiple head drills and/or percussion drills can be used to excavate trench in soft consolidated material. • Significant contaminant plume dispersion, prior to implementation, may limit performance. • Shallow, water table aquifer required. • Surface handling and treatment of contaminated water must be considered.
7. Permeable Treatment Beds	Feasibility limited to soft, rippable competent rock.	• Multiple head drills and/or percussion drills can be used to excavate trench in soft consolidated material. • Significant contaminant plume dispersion, prior to implementation may limit performance. • Shallow, water table aquifer preferable. • Availability of suitable filtration material. • Proper disposal of spent filtration material.
8. Ground Freezing	Feasible	• Very expensive and energy intensive. • Thermal erosion may preclude implementation. • 0.9-1.2 m/day maximum groundwater velocity. • Surface piping insulation required. • Host material saturation >10%.
9. Air Injection	Marginally feasible	• Little engineering expertise or implementation experience. • Saturated conditions required. • Drilling costs may be high. • Energy intensive. • Air bleeding of contaminants.

5.6 GENERIC SITE: POROUS UNCONSOLIDATED SILICATES

5.6.1 Pre-Mitigative Contaminant Discharge

5.6.1.1 Significant Radionuclide Discharges

The percentage of sites where indicator radionuclides are calculated to enter a surface water body prior to 40 half-lives of decay is shown in Figure 5.6.1-1. Strontium-90, cesium-137 and ruthenium-106 would be in the ground-water flow system in significant amounts at 49%, 27% and 5% of the sites, respectively. About half as many sites have significant cesium-137 discharges as compared to strontium-90 which has a similar half-life. This result is expected since the equilibrium distribution coefficient for cesium-137 is six times greater than strontium-90 in this generic classification. The discharge of ruthenium-106, which has a half-life of 367 days, indicates that 5% of the sites in this generic classification have short contaminant transport times.

5.6.1.2 Core Melt Leachate Discharged to Surface Water

The core melt debris will release radionuclides from a silicic melt at slow rates over long periods of time. The peak value of radionuclide flux is indicated by a line perpendicular to the flux/year curve at the left terminus. The porous unconsolidated silicates classification has the largest number of individual sites and generic trends are readily observable. Figure 5.6.1-2 presents the flux rate with time for the core melt leachate reaching a surface water body at significant levels.

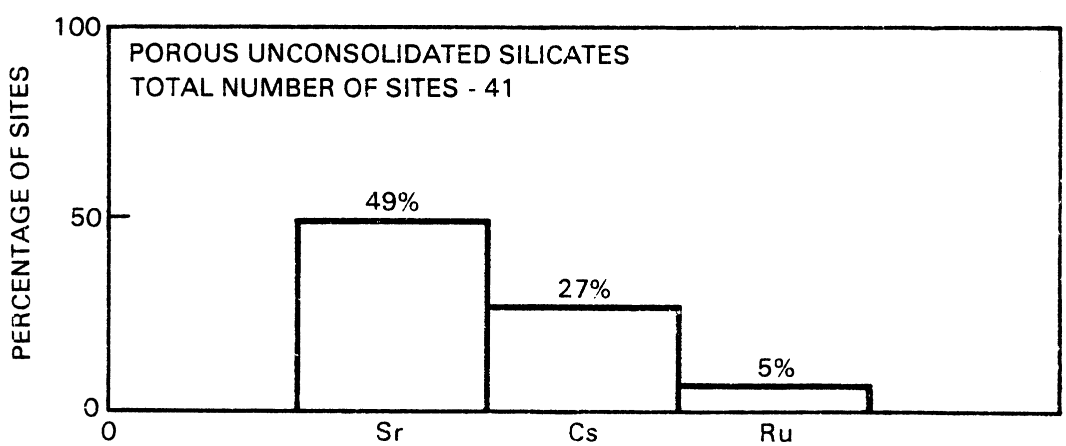

FIGURE 5.6.1-1. Percentage of Porous Unconsolidated Silicate Sites That Would Discharge Each of the Indicator Radionuclides Prior to 40 Half-Lives of Decay

POROUS UNCONSOLIDATED SILICATE – CORE MELT

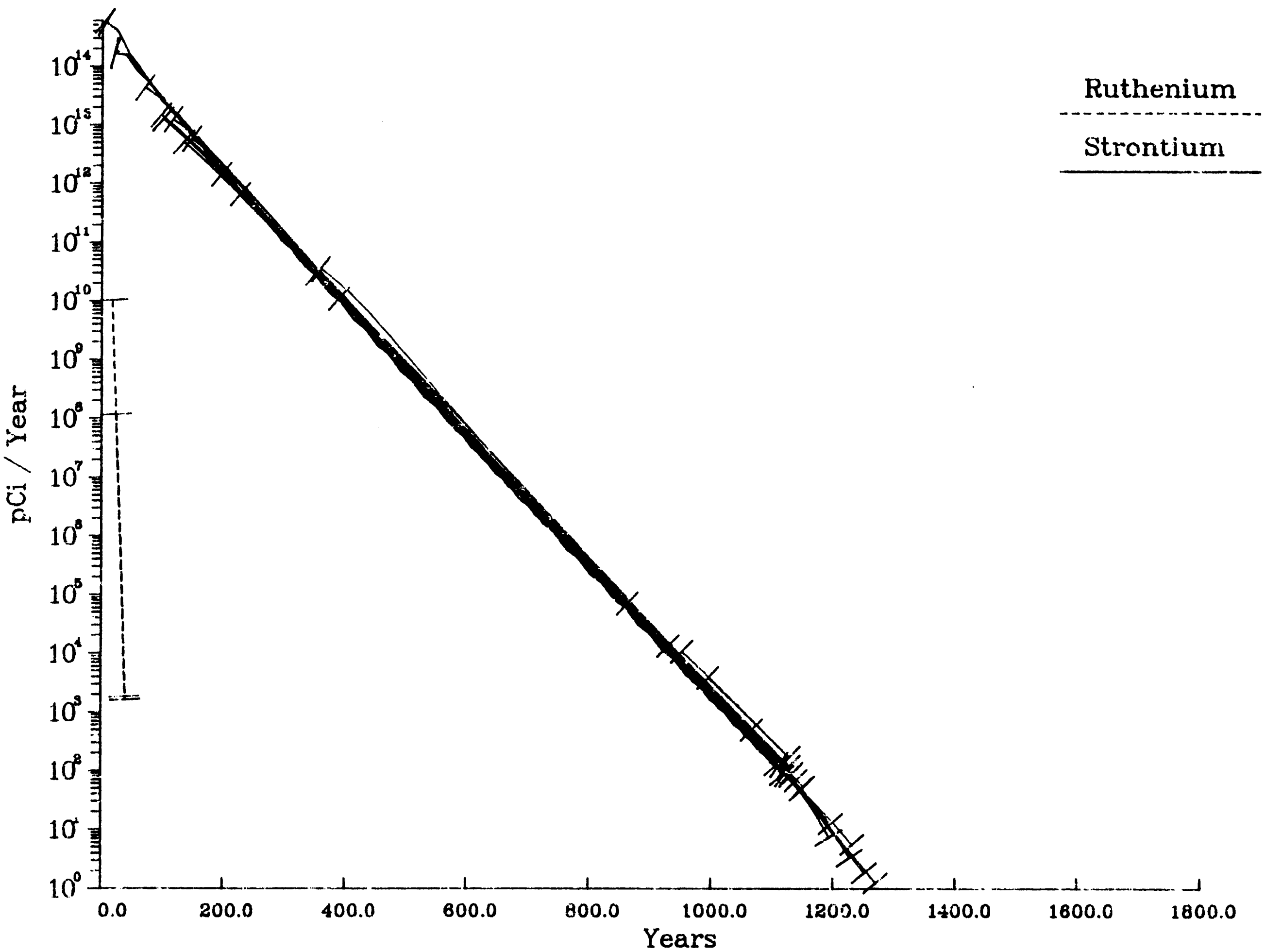

FIGURE 5.6.1-2. Discharge Flux of Core Melt Leachate from Porous for Unconsolidated Silicate Sites to Surface Water Strontium-90 and Ruthenium-106

Strontium-90

The calculated first contaminant arrival time is 4.4 years for two sites. Flux of strontium-90 at 6 x 10^{14} ρCi/yr for these two sites. Additional discharges are calculated at 25 years with a loose grouping of arrivals at times between 75 years and 250 years. The flux rates of the loose grouping are between 7 x 10^{12} and 1 x 10^{13} ρCi/yr. First arrival of contaminant occurs at about 375 years for three sites and at extreme times of about 950 years for four remaining sites. The flux rates of the radionuclide discharges at times over 375 years are at flux levels less than 1 x 10^{10} ρCi/yr.

Ruthenium-106

The travel times to surface water are sufficiently long to decay ruthenium-106 to insignificant levels for all but two sites. The first arrival of ruthenium contaminant is at 17.6 years with a flux of 1 x 10^{10} ρCi/yr and the next first arrival time is 25 years at 1 x 10^{8} ρCi/yr.

5.6.1.3 Sump Water Discharge to Surface Water

Sump water can collect in the containment structure during a severe nuclear power plant accident involving a pressurized water reactor. The liquid would be released when the top of the core melt debris cooled below the boiling point of water. Radionuclides contained in the sump water would be released at a rapid rate with high radionuclide concentrations as compared to a leach release. High radionuclide concentrations would result. Figure 5.6.1-3 presents the peak sump water discharge fluxes at the surface water body.

Strontium-90

Strontium-90 is the first radionuclide to be discharged with a calculated first arrival time of 4.4 years and a flux of 2 x 10^{16} ρCi/yr. Later strontium-90 discharges are loosely grouped between 75 and 250 years with an average flux of 3 x 10^{13} ρCi/yr. The remaining sites exhibit much longer first arrival times and have flux rates below 1 x 10^{11} ρCi/yr.

Cesium-137

The highest rate for cesium-137 discharges in this generic classification is 2 x 10^{16} ρCi/yr occurring at about 24 years. The cesium-137 data do not cluster around a specific time or flux rate.

Ruthenium-106

The calculated first arrival of ruthenium for this generic classification takes place after that of strontium-90. The contaminant transport time is long enough to decay most of the ruthenium before surface water discharge. The peak ruthenium flux is 1 x 10^{11} ρCi/yr or about five orders of magnitude less than cesium-137 and strontium-90.

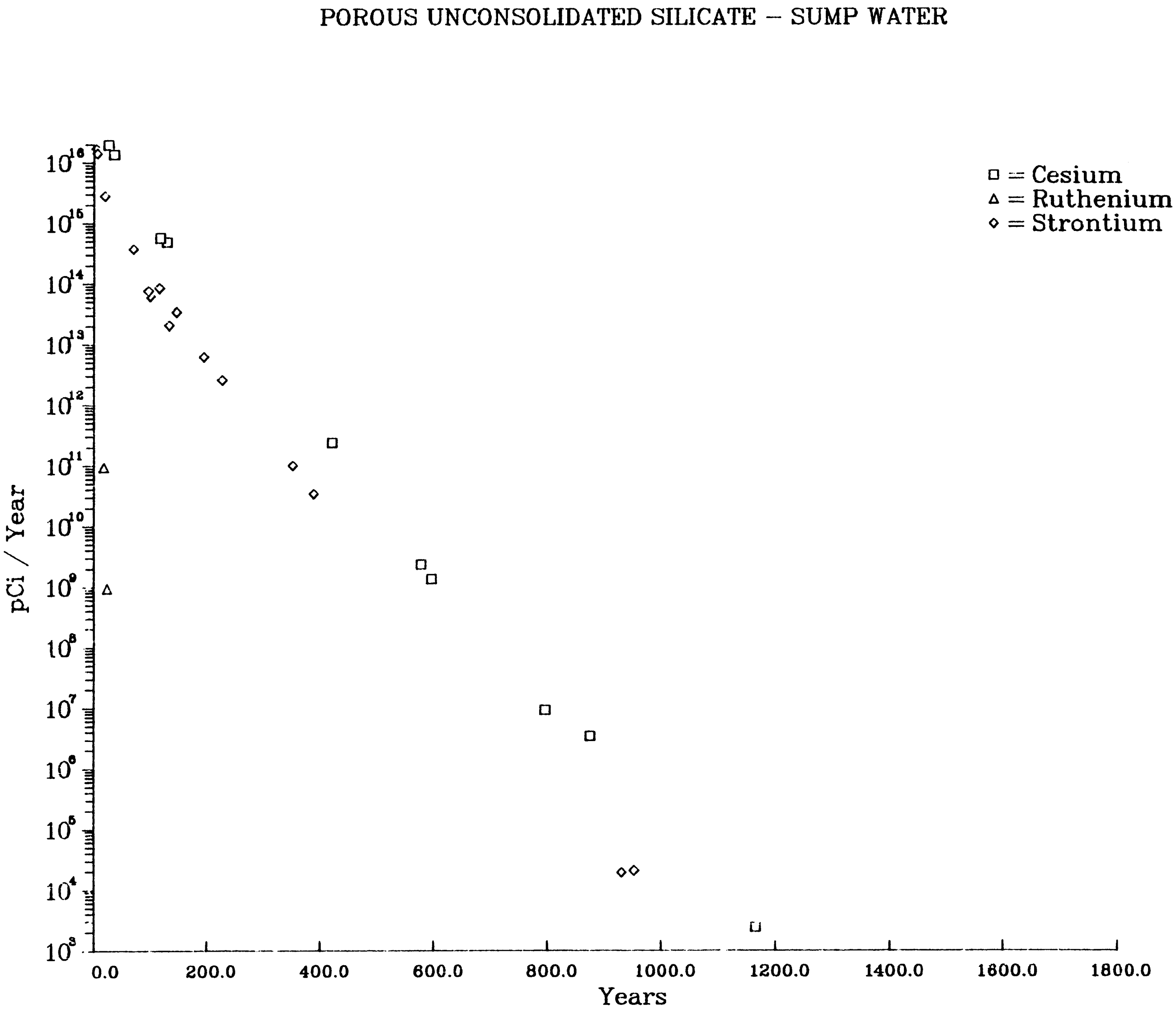

FIGURE 5.6.1-3. Discharge Flux of Reactor Sump Water from Porous Unconsolidated Silicate Sites to Surface Water Strontium-90, Cesium-137, and Ruthenium-106

5.6.2 Mitigative Techniques for Porous Unconsolidated Silicates

Table 5.6.2-1 presents a matrix of ground-water contaminant mitigative techniques versus feasibility of implementation at porous unconsolidated silicates sites. Constraints on feasibility as they relate to this generic site are also briefly summarized in the table.

TABLE 5.6.2-1. Mitigative Techniques for Porous Unconsolidated Silicates

Mitigative Technique	Feasibility	Constraints on Feasibility
1. Grouting: 1a) Particulate/ Cement-base 2b) Non-particulate/ Chemical	Permeation grouting Compaction grouting	• Host medium permeabilities: 10^{-1} - 10^{-3} cm/sec* = easy, 10^{-3} - 10^{-4} cm/sec = moderate, 10^{-4} - 10^{-5} cm/sec = marginal *Mean range of values for this classification. • Grain size as low as 0.01 mm (silt size) can be grouted w/acrylamide-based grouts. • Fine granular material as low as 0.10 mm (fine sand - coarse silt) can be grouted w/lignin grouts.
2. Slurry Trenches: 2a) Soil bentonite (S-B) 2b) Cement bentonite (C-B) 2c) Lean concrete (L-C) 2d) Vibrating beam (VBT)	Feasible within depth limitations and with proper key-in integrity.	• Backhoes for excavation to 17m. • Draglines for excavation to 30m. • Clamshells for excavation to 85m. • Ground-water velocities less than 5 cm/sec. • Soil permeability greater than 10^{-6} cm/sec. • VTB method depth limitation is roughly 30 m. Boulders and cobbles may cause limited penetration and/or sealing.
3. Steel Sheet Piling	Feasible in loose soils without appreciable cobbles or boulders.	• Difficult to moisture seal piling interlocking systems. • Must be hard-driven into impervious key-in layer. • Relatively short (7-40 years) effective life. • Effect of differential hydrostatic head must be considered.

TABLE 5.6.2-1. (contd)

4. Ground-Water Withdrawal for Potentiometric Surface Adjustment: 4a) Prevent discharge to receiving stream 4b) Prevent water table contact w/core melt mass. 4c) Prevent contamination of leaky aquifer	Feasible	• Fine-grained soils with low permeabilities may be prohibitive. • Proper handling required for contaminated water brought to the surface. • Detailed hydrogeologic studies may be complicated by heterogeneous nature of unconsolidated materials. • Shallow aquifers preferable.
5. Ground-Water Withdrawal and/or Injection for Contaminant Plume Control: 5a) Withdrawal and injection 5b) Withdrawal without injection 5c) Withdrawal and recharge 5d) Injection	Feasible	• Fine-grained soils with low permeabilities may be prohibitive. • Proper handling required for contaminated water brought to surface. • Detailed hydrogeologic studies of complex flow system required to determine effectiveness. • Significant contaminant plume dispersion, prior to implementation, limit performance.
6. Interceptor Trenches	Feasible	• Shallow, water table aquifer required. • Significant contaminant plume dispersion, prior to implementation, may limit performance. • Surface handling and treatment of contaminated water must be considered.
7. Permeable Treatment Beds	Feasible	• Shallow, water table aquifer preferred. • Significant contaminant plume dispersion, prior to implementation, may limit performance. • Availability of suitable filtration material. • Proper disposal of spent filtration material.

TABLE 5.6.2-1. (contd)

8. Ground Freezing	Feasible	• Very expensive and energy intensive. • Thermal erosion may preclude implementation. • Soil heave may occur in saturated materials. • 0.9-1.2 m/day maximum ground-water velocity. • Surface piping insulation required. • Host material saturation >10%.
9. Air Injection	Marginally feasible	• Little engineering expertise or implementation experience. • Saturated conditions required. • Energy intensive. • Air bleeding of contaminants.

5.7 GENERIC SITE: FRACTURED CONSOLIDATED SILICATES - SHALE

5.7.1 Pre-Mitigative Contaminant Discharge

5.7.1.1 Significant Radionuclide Discharge

The percentage of individual sites that have contaminant travel times to a surface water body of less than 40 half-lives represent the significant radionuclide releases. These values for the indicator radionuclides (i.e., strontium-90, cesium-137 and ruthenium-106) are presented in Figure 5.7.1-1. Strontium-90 and cesium-137 have similar decay rates yet reach the surface water body in significant amounts at 60% and 20% percent of the sites, respectively. This is because the equilibrium distribution coefficient of cesium-137 is 33 times greater than that of strontium-90. Consequently, cesium-137 is more strongly retarded. Ruthenium-106 decays to insignificant amounts prior to surface water discharge at all shale sites.

The average travel distance (i.e., 700 meters) is the longest for this generic classification and has a strong influence on the discharge quantities of short-lived radionuclides such as ruthenium-106.

5.7.1.2 Core Melt Leachate Discharged to Surface Water

The core melt debris will slowly release radionuclides to the ground-water flow system as the silicic matrix leaches. The arrival of strontium-90 at the nearest surface water body is presented in Figure 5.7.1-2. The calculated peak flux is 2×10^{14} ρCi/yr at the first arrival time of 32 years. Two other sites which discharge to surface water bodies prior to 40 half-lives of decay have arrival times of 450 and 625 years after initial release.

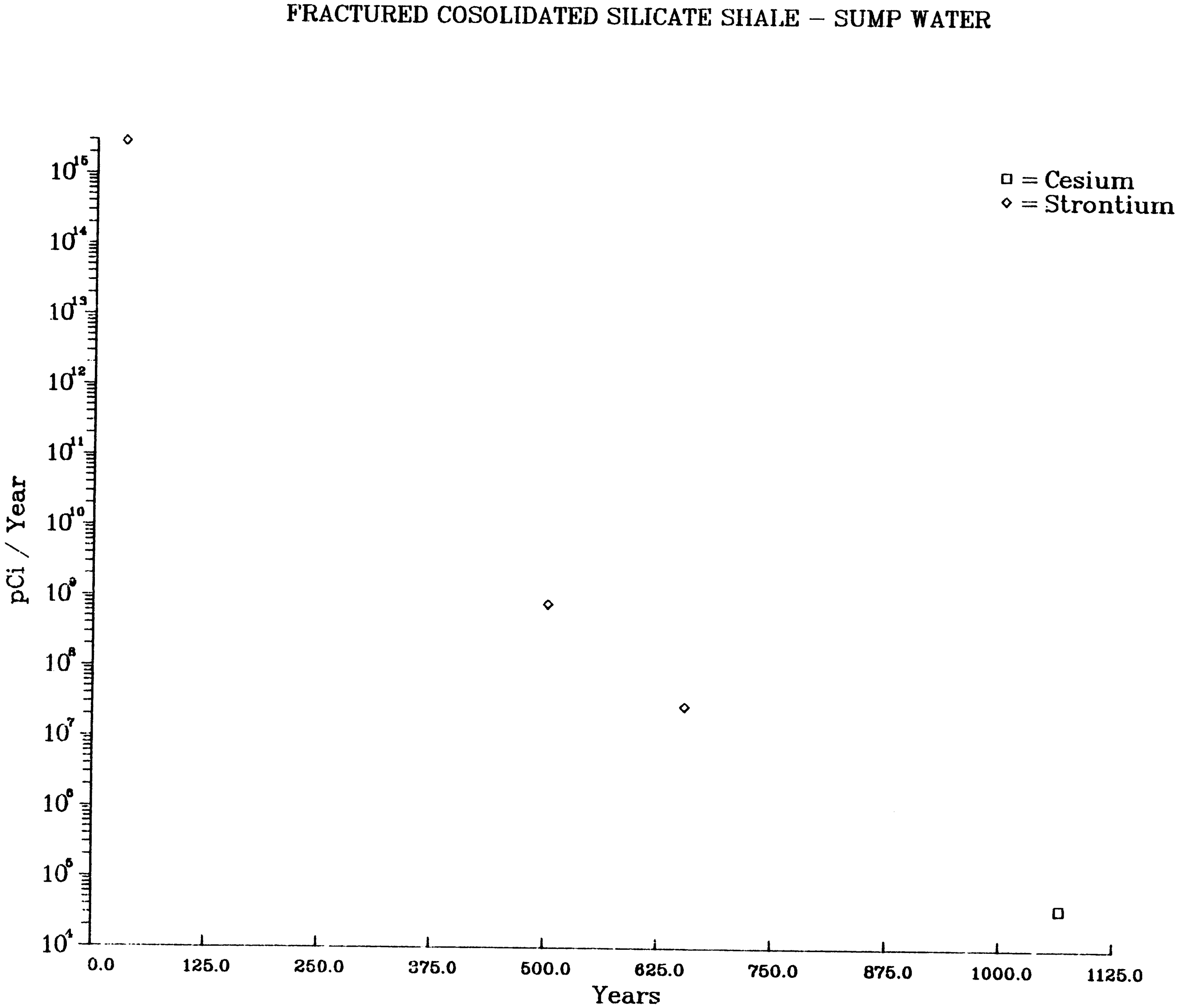

FIGURE 5.7.1-3. Discharge Flux of Reactor Sump Water from Fractured Consolidated Silicate-Shale Sites to Surface Water for Strontium-90, Cesium-137 and Ruthenium-106

TABLE 5.7.2-1. Mitigative Techniques for Fractured Shale

Mitigative Technique	Feasibility	Constraints on Feasibility
1. Grouting: 1a) Particulate/ Cement-based 1b) Non-particulate/ Chemical	Fissure grouting Fracture grouting	• Joint gaps between 0.5 mm and 6.0 mm for cement-based grouts. • Fissure width up to 10 cm - 15 cm can be grouted w/acrylamide-based grouts.
2. Slurry Trenches: 2a) Soil bentonite (S-B) 2b) Cement bentonite (C-B) 2c) Lean concrete (L-C) 2d) Vibrating beam (VBT)	Infeasible	• Excavation prohibited by competent rock.
3. Steel Sheet Piling	Infeasible	• Pilings cannot be hard-driven through consolidated media.
4. Ground-Water Withdrawal for Potentiometric Surface Adjustment: 4a) Prevent discharge to receiving stream 4b) Prevent water table contact w/core melt mass. 4c) Prevent contamination of leaky aquifer	Marginally feasible	• Shallow aquifers preferable. • Definition of fracture system is necessary. • Detailed hydrogeologic studies required to determine feasibility (i.e., difficulty arises in determining radii of influence of wells in fractured media). • Ground-water system response may be prohibitively slow due to relatively low hydraulic conductivity. • Proper handling required for contaminated water brought to the surface.

TABLE 5.7.2-1. (contd)

5. Ground-Water Withdrawal and/or Injection for Contaminant Plume Control: 5a) Withdrawal and injection 5b) Withdrawal without injection 5c) Withdrawal and recharge 5d) Injection	Feasible	• Definition of fracture system required. • Ground-water system response may be prohibitively slow due to relatively low hydraulic conductivity. • Detailed hydrogeologic studies required to determine feasibility. • Fracture system may enhance performance by concentrating contaminants. • Proper handling required for contaminated water brought to the surface.
6. Interceptor Trenches	Infeasible	• Excavation prohibited by competent rock.
7. Permeable Treatment Beds	Infeasible	• Excavation prohibited by competent rock.
8. Ground Freezing	Feasible	• Very expensive and energy intensive. • Thermal erosion may preclude implementation. • 0.9-1.2 m/day maximum ground-water velocity. • Surface piping insulation required. • Host material saturation >10%.
9. Air Injection	Marginally feasible	• Little engineering expertise or implementation experience. • Saturated conditions required. • Energy intensive. • Air bleeding of contaminants.

5.8 COMPARISON OF PRE-MITIGATIVE CONTAMINANT DISCHARGES

5.8.1 Significant Discharges to Surface Water Bodies

Some of the indicator radionuclides are calculated to arrive at the nearest surface water body at insignificant flux rates. The discrimination of these sites is to develop guidelines to which hydrogeologic classifications are sensitive to radioactive discharges and consequently may be of immediate environmental concern. A 40 half-life limit is used to delineate significant radionuclide discharges from those discharges that exhibit very late arrivals with low levels of radioactivity. In all actual severe accidents the site specific pre-mitigative discharge would be determined before deciding upon the necessity and type of mitigation to be implemented. The three radionuclides used as indicators of potential environmental consequences are discussed separately below.

The 28.2 year half-life and low rate of sorbtion of strontium-90 makes it a good environmental indicator since it would not necessarily undergo 40 half-lives of decay prior to discharge into a surface water body. The percentage of sites within each generic classification that would exhibit a significant strontium-90 discharge is shown in Figure 5.8.1-1. The hydrogeologic classification numbers are used as a convenience in the figure and are defined in Table 5.8.1-1. Strontium-90 is discharged to surface water prior to 40 half-lives at some sites in all generic classifications. Discharges are most likely

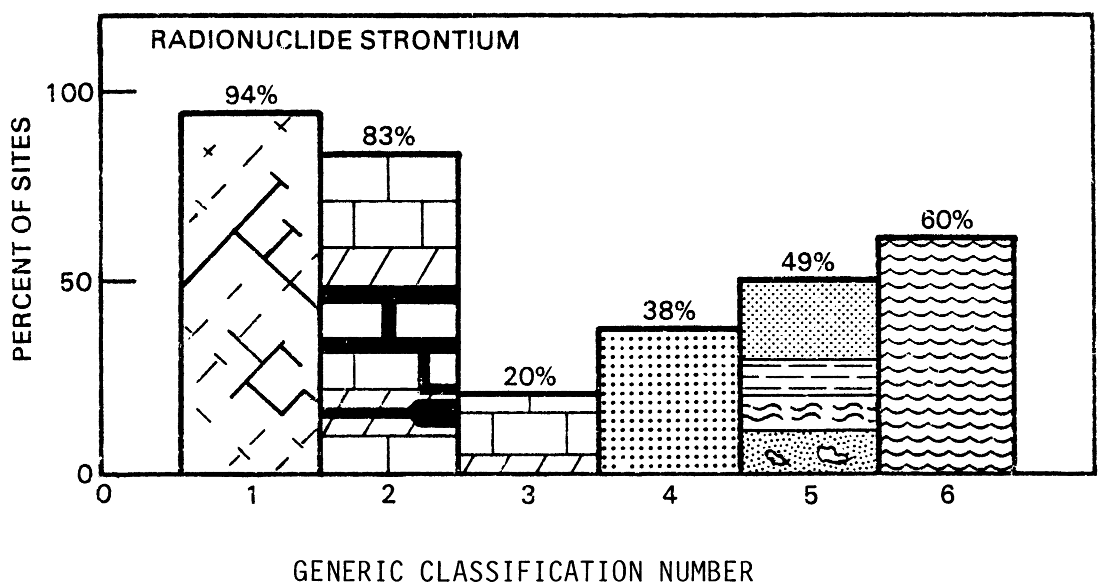

FIGURE 5.8.1-1. Percentage of Nuclear Power Plant Sites in Each Generic Hydrogeologic Classification That Would Discharge Strontium-90 Prior to 40 Half-Lives of Decay

TABLE 5.8.1-1. Generic Classification Numbering Index

Generic Classification Number As Used in Figures 5.8.1-1,2,3	Generic Classification As Developed in This Report
1	Fractured Consolidated Silicates-Crystalline
2	Fractured & Solutioned Consolidated Carbonates
3	Porous Consolidated Carbonates
4	Porous Consolidated Silicates
5	Porous Unconsolidated Silicates
6	Fractured Consolidated Silicates - Shale

in fractured consolidated silicates-crystalline (at 94% of the sites) and fractured and solutioned consolidated carbonates (at 83% of the sites). Porous geologic materials demonstrate a smaller percentage of significant discharges reaching a minimum of 20% of the sites in the porous consolidated carbonate classification. The large span of percentages of significant strontium-90 discharges demonstrates that it is a good indicator of the potential for adverse environmental consequences. In three of the six generic classifications significant discharges would occur at an average of less than 35% of the individual sites. These generic classifications are relatively insensitive to a strontium-90 core melt release. The probability of a significant strontium-90 discharge is less than 50% when all generic classifications are considered.

The percentage of sites that would discharge cesium-137 prior to 40 half-lives of decay for all generic classifications is presented in Figure 5.8.1-2. Again, Table 5.8.1-1 serves as a key to the generic classification numbers. As compared to strontium-90, there are fewer cesium-137 significant discharges to a surface water body. Although cesium-137 has a half-life similar to strontium-90, retardation is stronger for this radionuclide. Four out of the six generic classifications have less than a 50% probability of a significant cesium discharge. The porous consolidated carbonate classification has a minimum of 1209 years (40 half lives) of decay prior to cesium-137 discharge at all individual sites.

Ruthenium-106 discharges in significant amounts at few sites. Figure 5.8.1-3 and accompanying Table 5.8.1-1 show that the greatest probability of a significant ruthenium-106 discharge is 33% and is associated with the fractured and solutioned consolidated carbonate classification. The five remaining generic classifications have significant discharges at 5% or less of the sites. Clearly, ruthenium-106 is decayed to low levels at most sites while still in the ground-water system. Consequently, ruthenium is not the primary radionuclide of concern at the point of discharge in any generic classification.

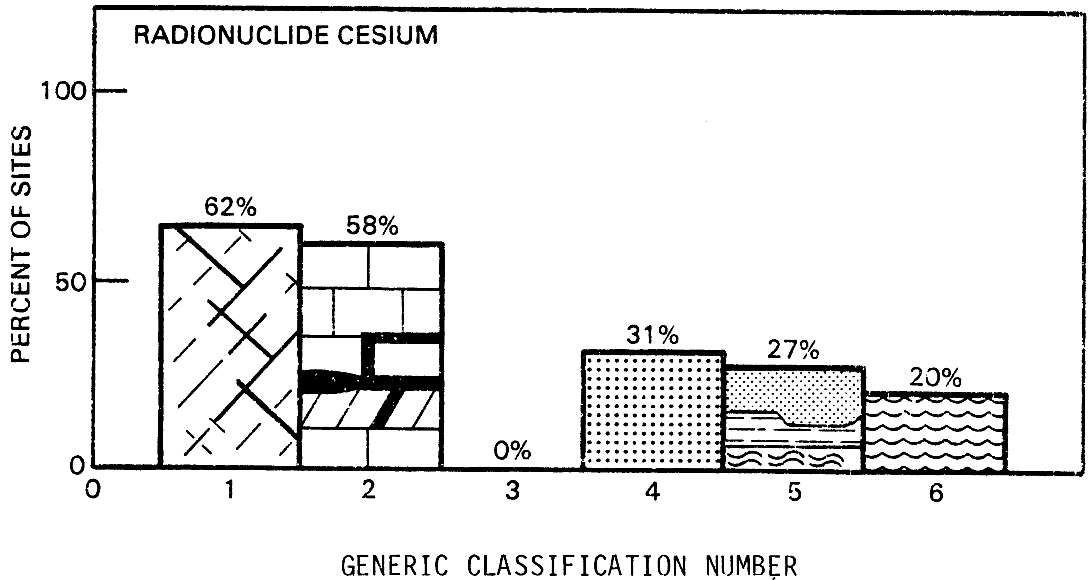

FIGURE 5.8.1-2. Percentage of Nuclear Power Plant Sites in Each Generic Hydrogeologic Classification That Would Discharge Cesium-137 Prior to 40 Half-Lives of Decay

The percentage of significant discharges can also be used to rank the generic sites according to the probability that a severe nuclear accident will require mitigative action. For this analysis it was assumed that if any of the indicator radionuclides reached a surface water body in significant amounts, then mitigation would be required. In actuality some sites having a calculated significant discharge would not need mitigation due to site specific characteristics. The dilution factor of the receiving water body, precise contaminant outflow flux and location, or contaminant (i.e., strontium-90) chemically replacing calcite could be important factors in determining the need for mitigation. Therefore, assuming mitigation is required for any discharge above a conservatively defined level of significance is pessimistic. The sensitivity of a generic classification to a core melt accident was determined in this manner. The results are presented in Table 5.8.1-2.

The ranking of generic classifications based on the percent of sites with significant surface water discharges conforms to the basic concepts of contaminant ground-water hydrology. Fractured geologic materials are about twice as likely to have significant radionuclide discharges as their porous counterparts.

These percentages indicate that even under a conservative definition of a significant discharge, 43% of all nuclear power plant sites fail to produce prominent radionuclide fluxes after a simulated core melt accident. Fractured ground-water systems are more sensitive than porous flow systems. Fractured rock can be expected to produce significant discharges to surface water at 85%

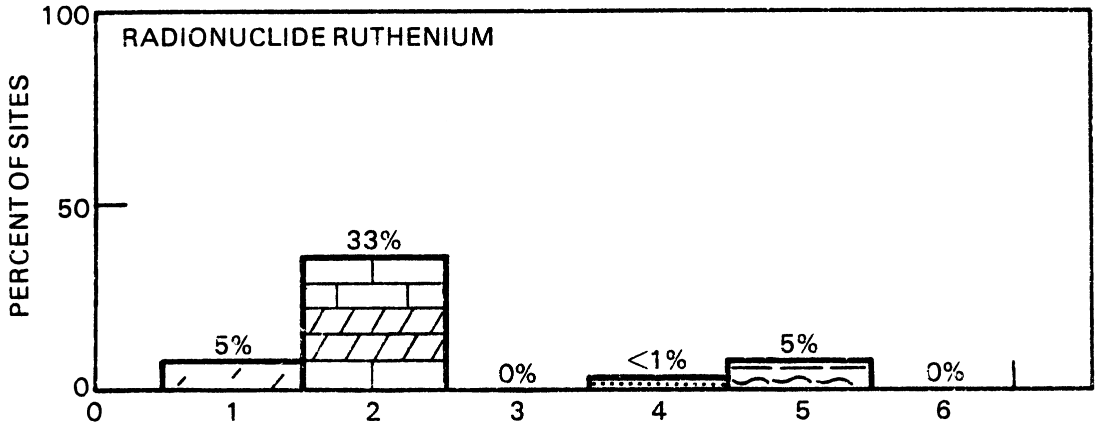

FIGURE 5.8.1-3. Percentage of Nuclear Power Plant Sites in Each Generic Hydrogeologic Classification That Would Discharge Ruthenium-106 Prior to 40 Half-Lives of Decay

TABLE 5.8.1-2. Generic Sensitivity to a Severe Nuclear Accident

Rank	Generic Classification	Percent of Sites with Significant Surface Water Discharges*	Number of Sites in Generic Classification
1	Fractured Consolidated Silicates-Crystalline	94%	16
2	Fractured & Solutioned Consolidated Carbonates	83%	12
3	Fractured Consolidated Silicate-Shale	60%	5
4	Porous Unconsolidated Silicates	49%	41
5	Porous Consolidated Silicates	38%	13
6	Porous Consolidated Carbonates	20%	10

* All three indicator radionuclides considered.

of the sites. Porous ground-water systems can be expected to produce significant radionuclide discharges at only 42% of the nuclear power plant sites comprising the porous classifications.

5.8.2 Core Melt Leachate Discharges to Surface Water

Variations in the hydrogeologic conditions upon which the generic classifications are based result in large scale differences in contaminant discharges at individual sites. There is a wide range of possible first arrival times and radionuclide activity fluxes even within a single generic classification. Comparison of representative results of extreme values and data trends gives a basis of evaluation of the environmental sensitivity of the generic classifications.

Table 5.8.2-1 presents a summary of core melt leachate entering surface water from each generic classification. The shortest time of first contaminant arrival within each classification is shown in Column 2. The time of first contaminant arrival at the surface water body under the assumptions of the transport analysis is the same for sump water and core debris leachate. The arrival times are in relation to the initiation of contaminant transport, and not the accident. The first contaminant arrival times are the data extremes and hence reflect the worst case in each generic classification. The first arrival times have a constrained range with regard to the wide variations of hydraulic parameters. Generally, the fractured flow systems have the shortest first arrival time of contaminant except for fractured shale media. Porous ground-water flow systems have a first arrival time of about a half an order of magnitude longer than the fractured generic classifications except for shale media as noted above. The data extremes for first contaminant arrivals indicate that this parameter is not strongly generically correlated and may be more of a site specific characteristic. The first arrival time of contaminant by generic classification does show that a minimum contaminant travel time from the core melt to a surface water body is on the order of several months for all power plant sites.

The radionuclide activities flux associated with the first arrival times is shown in Column 3 of Table 5.8.2-1. The flux values show a relationship to generic classification. Strontium-90 is, of the two core melt radionuclides of interest, a better indicator than ruthenium-106 generic characteristics. The flux is much less dependent upon travel time. The analysis of core melt leachate is based on strontium-90 because of its longer half-life. There are two major factors beyond aquifer hydraulics that determine peak flux rate: 1) the leach release rate which is largely a function of rock chemistry 2) and the amount of retardation due to sorption. When these factors favor release and transport simultaneously, as in the case of the fractured and solutioned consolidated carbonate classification, the flux discharge is at its maximum. For this classification strontium-90 and ruthenium-106 have peak discharge fluxes of 1×10^{17} and 2×10^{16} ρCi/yr, respectively.

The generic classification fractured and solutioned carbonates has the maximum flux as expected in consideration of the initial conditions. However, the second greatest flux values for this generic classification are also

TABLE 5.8.2-1. Summary of Pre-Mitigative Core Melt Discharges to Surface Water

(1) Generic Classification	(2) Time of First Contaminant Arrival[a] (yr)	(3) Peak Activity Flux (ρCi/yr)	(4) Data Clusters	(5) Time of Clustered Contaminant Arrival[a] (yr)	(6) Clustered Activity Flux (ρCi/yr)	(7) Radionuclide
Fractured Consolidated Silicates-Crystalline	0.9	$1x10^{15}$	Yes	10	$2x10^{14}$	^{90}Sr
	4.6	$1x10^{14}$	No	--	--	^{106}Ru
Fractured and Solutioned Consolidated Carbonates	0.6	$1x10^{17}$	Y&es	5	$3x10^{16}$	^{90}Sr
	2.2	$2x10^{16}$	No	--	--	^{106}Ru
Porous Consolidated Carbonates	44.0	$4x10^{15}$	Minor	<200	$>2x10^{13}$	^{90}Sr
	>40.0	--	--	--	--	^{106}Ru
Porous Consolidated Silicate	2.0	$1x10^{15}$	Yes	50	$8x10^{13}$	^{90}Sr
	9.8	$3x10^{12}$	No	--	--	^{106}Ru
Porous Unconsolidated Silicate	4.4	$6x10^{14}$	Yes	125	$5x10^{12}$	^{90}Sr
	17.6	$1x10^{10}$	No	--	--	^{106}Ru
Fractured Consolidated Silicate-Shale	32.0	$2x10^{14}$	No	--	--	^{90}Sr
	>40.0	--	--	--	--	^{106}Ru

(a) Times are given from time of release which is assumed to be 1 year after the accident.

important because they determine what transient feature is more sensitive to a nuclear release; fracture hydraulics or the chemically controlled leach rate? Fractured flow systems have the shortest transport times and carbonate rock types have high leachate rates. Comparison of peak strontium-90 flux values for fractured consolidated crystalline silicates and porous consolidated carbonate classifications shows that the latter has about four times greater flux. This occurs despite of the much longer time to first contaminant arrival for porous consolidated carbonates. The lowest peak flux values are observed in the fractured consolidated silicate-shale classification. Here the effects of a slow silicic leach rate and a long transport time to the surface water body become evident. The peak strontiuim-90 flux for this generic classification is 2×10^{14} ρCi/yr and 2.5 is orders of magnitude less than the maximum observed flux.

Generic trends are noteworthy because they indicate the credible characteristics of a significant discharge. The generically characteristic values of arrival time and flux are given in Columns 4 and 5 of Table 5.7.1-3. Again, the time and flux relationships for generic classification are similar to those observed in the first arrival time-peak flux analysis. Ruthenium-106 decay is too rapid for it to be used as an indicator radionuclide at these long travel times of 5 to 125 years. The first arrival activity flux of ruthenium falls quickly with time making data clustering unlikely. In addition, not all generic classifications exhibit data clustering or trends other than that caused by radioactive decay. Data clusters are defined as grouped site arrival times and discharge fluxes. However, the clustering does not always include the majority of sites in a generic classification. Hence the clustered data values do not represent the most likely values. For distribution of times and flux values in a generic classification the reader should consult the individual generic discharge descriptions. Fractured geologic materials except for shale media tend to have characteristic first arrival times of about 5 to 10 years after release. Porous flow systems have much longer characteristic first contaminant arrival times of 50 to greater than 200 years.

Shale is a special case although not one of great concern. The long time of contaminant transport in shale coupled with the lack of sites in this classification prevent any observable trends to contaminant discharges. It was concluded that discharge of core melt leachate from shale media will be at long times and at low activity levels. In all generic classifications the clustered arrival times are sufficient to construct mitigative barriers.

The clustered activity fluxes of strontium-90 are presented in Table 5.8.2-1, Column 5. The early contaminant arrivals for the fractured consolidated crystalline silicates and fractured consolidated carbonates produce the highest flux values of 2×10^{14} and 3×10^{16} ρCi/yr, respectively. Contaminant clustered arrivals 50 years after the release flux values are less than 8×10^{13} ρCi/yr and generic distinctions are not clear.

5.8.3 Sump Water Discharges to Surface Water

A sump water releaswe would contain the remainaing fractions of strontium-90 and ruthenium-106 not incorporated in the core melt mass. The entire inventory of cesium-137 is also assumed to be included in a sump water release. A comparison of first arrival times and peak discharge fluxes is given in Table 5.8.3-1. The first arrival times for strontium-90 and ruthenium-106 are the same as for the core melt release. Cesium-137 is retarded equal to or greater than ruthenium-106 and thus is never the first radionuclide to be discharged into surface water. The analysis of first arrival times of strontium-90 and ruthenium-106 is presented in Section 5.8.2.

Of the two types of radionuclide releases following a core melt accident, sump water has the potential of creating the highest activity flux into the accessible environment. The flux rate is dependent upon rate of liquid release (which is a function of pressure head and melt debris permeability) and the travel time to a surface water body. Column 2 of Table 5.8.3-1 gives the maximum activity flux calculated for each generic classification. When travel times are short, cesium-137 has flux rates above the other indicator radionuclides reaching a peak value of 2.5×10^{17} ρCi/yr in fractured and solutioned consolidated carbonates. In porous ground-water systems, higher rate of sorption for cesium-137 results in discharge activities comparable to that of strontium despite the much larger initial release of cesium-137. Fractured shale media is a special case where cesium-137 is highly sorbed and discharge fluxes are about 11 orders of magnitude less than the other generic classifications. Carbonate also is a sorptive environment for cesium-137 and in the porous classification the first arrival time is long and peak flux is low. The range of significant cesium-137 is somewhat constrained in that there are only a 2.5 orders of magnitude of variation.

Strontium-90 peak fluxes are also relatively high in sump water discharges. Strontium-90 is less sorbed than cesiuim-137 and arrives at the discharge location earlier, thus preserving a high flux rate from the time effects of radioactive decay. The range of peak strontium-90 flux values is 1×10^{15} to 2×10^{17} ρCi/year yielding about the same range as cesium-137.

Ruthenium-106 (which has the greatest initial release of activity) arrives at the discharge location decayed to flux levels of 2×10^{16} to 1×10^{11} ρCi/year. Fractured shale and porous consolidated carbonates classifications discharge ruthenium-106 at long times and at insignificant levels. In all classifications rutheniuim-106 is discharged to the surface water body at fluxes at least one order of magnitude less than strontium-90 and cesium-137.

First arrival time and peak flux data clusters are given in Columns 4 and 5 of Table 5.8.3-1. The clustered times for strontium-90 are the same as in Section 5.8.2. Strontium-90 characteristics discharge occur in silicates (where sorption is less) and in fractured carbonates (where contaminant transport is more rapid). The characteristic strontium-90 sump water discharge flux levels are one to three orders of magnitude less than the peak flux

TABLE 5.8.3-1. Summary of Pre-Mitigative Sump Water Discharges to Surface Water

(1) Generic Classification	(2) Time of First Contaminant Arrival[a] (yr)	(3) Peak Activity Flux (ρCi/yr)	(4) Data Clusters	(5) Time of Clustered Contaminant Arrival[a] (yr)	(6) Clustered Activity Flux (ρCi/yr)	(7) Radionuclide
Fractured Consolidated Silicates Crystalline	0.9	6×10^{16}	Yes	10	4×10^{15}	^{90}Sr
	18.4	1×10^{17}	No	--	--	^{137}Cs
	4.6	1×10^{13}	No	--	--	^{106}Ru
Fractured and Solutioned Consolidated Carbonates	0.6	2×10^{17}	Minor	<150	$>1 \times 10^{13}$	^{90}Sr
	2.6	2.5×10^{17}	No	--	--	^{137}Cs
	2.2	2×10^{16}	No	--	--	^{106}Ru
Porous Consolidated Carbonates	>44	1×10^{15}	No	--	--	^{90}Sr
	>520	--	--	--	--	^{137}Cs
	>40	--	--	--	--	^{106}Ru
Porous Consolidated Silicate	2.0	4×10^{16}	Yes	50	1×10^{15}	^{90}Sr
	9.8	8×10^{16}	Minor	225	5×10^{13}	^{137}Sr
	9.8	3×10^{13}	No	--	--	^{106}Ru
Porous Unconsolidated Silicate	4.4	2×10^{16}	Yes	125	5×10^{13}	^{90}Sr
	26.4	2×10^{16}	No	--	--	^{137}Cs
	17.6	1×10^{11}	No	--	--	^{106}Ru
Fractured Consolidated Silicate-Shale	32	3×10^{15}	No			^{90}Sr
	1066	4×10^{4}	No			^{137}Cs
	>40	--	--			^{106}Ru

(a) Times are given from time of release which is assumed to be 6 months after accident.

rates. Ruthenium-106 decays at a rapid rate and the discharges cananot be generically characterized. Cesium-137 is retarded by sorption to the extent that this radionuclide also fails to show generic clustering except for porous consolidated silicates. For this classification the generic clustering occurs at 225 years after release and produces a moderate flux rate of 5 x 10^{13} ρCi/yr.

5.9 CONCLUSIONS FOR PRE-MITIGATIVE CONTAMINANT DISCHARGES

- The lack of clear generic trends in some classifications indicates that there are wide ranges in site specific parameters that outweigh the importance of the geologic transport media. In these cases the key generic trend is indicated by the percentage of sites with significant releases prior to 40 half-lives of decay.

- Generic characteristics that affect the severity of a core melt accident can be ranked in descending importance and are: 1) bedrock chemical type, 2) porosity type (i.e., interstitial or fracture), 3) sorption, and 4) aquifer hydraulics.

- Strontium-90 would be first of the indicator radionuclides to arrive at the discharge location. Cesium-137 which has an initially large sump water release would arrive at a slightly later time at flux levels very close to that of strontium-90. Ruthenium-106 arrives at longer times and lower activity flux than either strontium-90 and cesium-137.

- Strontium-90 is a better indicator of accident severity due to its longer half-life. Ruthenium-106 is decayed to flux levels 1 to 4.6 orders of magnitude less than strontium-90 prior to discharge.

- The time over which the radionuclides in a sump water release would be discharged into the accessible environment is on the order of weeks or months whereas core melt leachate would be discharged for hundreds of years.

- Fractured flow systems are more likely than porous flow systems to discharge contaminant at early times. The shortest arrival time of leachate in fractured silicates and carbonates is between 0.5 and 1.0 years. Shale media is an exception where first arrival times from leachate are on the order of decades.

- Porous flow systems have first arrival times of between 2.0 and 44 years after leach release. The average value is about 15 years which indicates there will be time to implement mitigative measures if needed.

- Carbonate aquifers are more sensitive to a core melt accident than a silicic aquifer. The leachate discharge flux to surface water for carbonates is expected to be 100 times greater than in fractured silicate aquifers and about 4 times greater than in porous silicate aquifers. The porous consolidated carbonate classification has a generic characteristic of long contaminant travel times preventing high flux values.

- The release of sump water to a ground-water flow system can create higher flux rates than core melt leaching. The peak discharge rates are about one order of magnitude greater than for coremelt leachate.

- Generic characteristics of sump water releases are best observed in first contaminant arrival times and peak flux values. Clustering of times and fluxes is seen clearly only for the strontium-90 discharges. Ruthenium decays at a rapid rate and cesium is most strongly sorbed making generic trends difficult to discern.

- The maximum flux rates due to sump water releases are for fractured consolidated crystalline silicates and solutioned carbonates are on the order of 1×10^{17} ρCi/yr. Minimum arrival times occur in the same classifications and are about 6 months to one year after radionuclide release which may be up to one year after the core melt accident.

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13. ABSTRACT (200 words or less)
Pacific Northwest Laboratory evaluated the feasibility of using ground-water contaminant mitigation techniques to control radionuclide migration following a severe commercial nuclear power reactor accident. The two types of severe commercial reactor accidents investigated are 1) containment basemat penetration of core melt debris, which slowly cools and leaches radionuclides to the subsurface environment; and 2) containment basemat penetration of sump water without full penetration of the core mass. Six generic hydrogeologic site classifications were developed from an evaluation of reported data pertaining to the hydrogeologic properties of all existing and proposed commercial reactor sites. One-dimensional radionuclide transport analyses were conducted on each of the individual reactor sites to determine the generic characteristics of a radionuclide discharge to an accessible environment. Ground-water contaminant mitigation techniques that may be suitable for severe power plant accidents, depending on specific site and accident conditions, were identified and evaluated. Feasible mitigative techniques and associated constraints on feasibility were determined for each of the six hydrogeologic site classifications. Three case studies were conducted at power plant sites located along the Texas Gulf Coast and the Ohio River. Mitigative strategies were evaluated for their impact on contaminant transport. Results show that the techniques evaluated significantly increased ground-water travel times and reduced contaminant migration rates.

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