REMEDIATION OF CONTAMINATED MARINE SEDIMENTS BY IN-PLACE CONTAINMENT: A CASE STUDY OF RARITAN RIVER, NEW JERSEY

Newark Bay Study Area
NBSA-00076

Linden, New Jersey (Fig. 1). The Rahway River has been contaminated by historical discharge of chemicals from several industries located along its coast. During 1990, the American Cyanamid Company (Cyanamid) and the New Jersey Department of Environmental Protection (NJDEP) entered into an Administrative Consent Order (ACO) that required completion of a Remedial Investigation (RI) and Corrective Measures Study (CMS) for the Warners Plant. Cytec was formed in 1993 by the spin-off of the Cyanamid Global Chemicals. After the spin-off, Cytec assumed the responsibilities of the ACO. Capping was selected as the remedial alternative for the

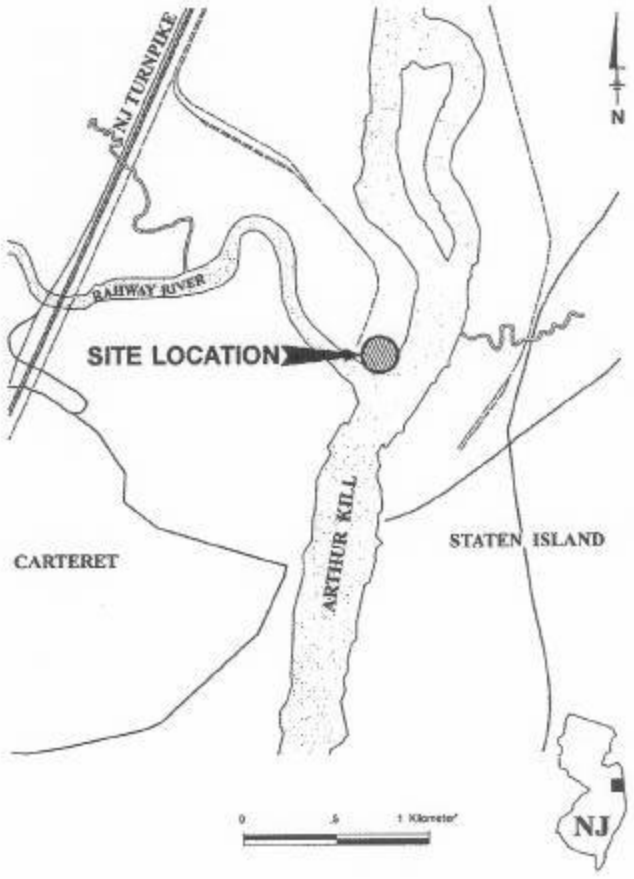


FIGURE 1 Site location map.

Rahway River in order to provide protection of human health and the environment by isolating and affected fill/sediments from the overlying water column. Once the affected sediments and fill are isolated and immobilized, they would no longer serve as a potential source media to fish, benthic organisms, or the water column.

2. SITE CHARACTERISTICS

2.1. Physical Site Description

The Warners Plant is located on Tremley Point, a land mass defined by the confluence of the Rahway River and the Arthur Kill (Fig. 2). This land was reclaimed from former tidal marshes by placement of fills, and its form has been maintained by bulkheads and rip-rap fills constructed along the banks of the Arthur Kill and the Rahway River, and by the channel dredging of the Arthur Kill. The addition of fills had created a relatively flat topographic surface, with elevations ranging from about 1 to 3 m above mean sea level. Surface water drainage from the site is to the surface water bodies on the eastern (Arthur Kill) and southern (Rahway River) limits of the site.

2.2. Hydrology and Hydraulics

The Rahway River is a freshwater river which discharges into the Arthur Kill at Tremley Point. Water conditions within the river at the Warners Plant site are primarily salty to brackish, with water depths varying from 1 to 3 m (Tab. I). Data by the U.S. Army Corps of Engineers (USACE, 1961) indicated that during a 100-year storm event, the maximum river velocity was 0.9 m/s and the maximum flow rate was 187 m^3/s at the site. Mean high water and mean low water at the site were estimated to be 1.07 m-NGVD and −0.6 m-NGVD, respectively.

2.3. Nature and Extent of Chemicals

As part of the remedial investigation (RI) and corrective measures study (CMS), several rounds of sediment sampling were conducted at

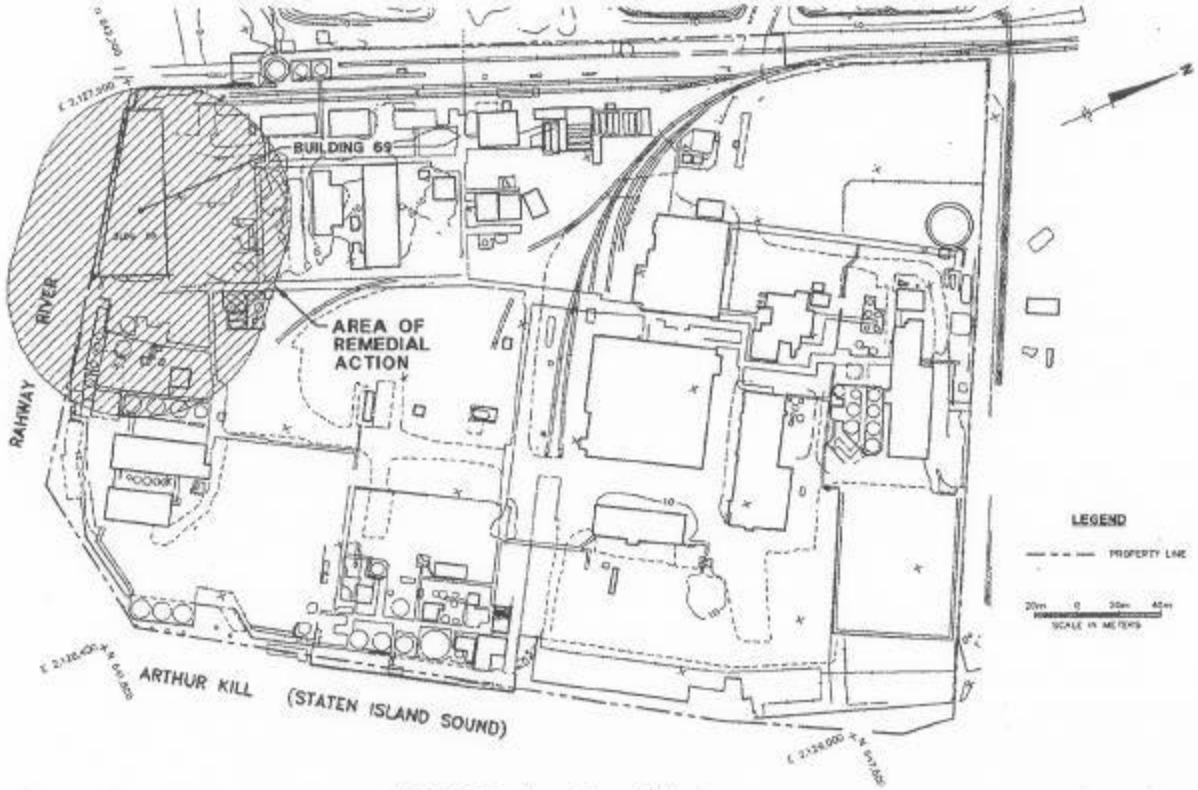


FIGURE 2 Area of remediation.

TABLE I Water depths and sediment thicknesses at the project site

Sample location	*Water depth (m)*	*Sediment thickness (m)*
FF6	2.59	0.61
FF4	2.13	0.91
FF2	1.49	1.52
DD6	2.90	0.46
BB6	2.96	0.30
BB5	n/a	0.43
AA3	n/a	0.46
A7	2.68	0.61
A4	n/a	0.46
C7	2.10	0.27
C5	n/a	0.30
E8	1.07	0.67
E6	1.83	0.91
E4	n/a	0.34
G8	2.07	0.67
H5	n/a	0.61
I9	2.20	0.76
K9	2.23	0.76
L6	1.68	0.61
M10	2.32	0.91
O10	2.50	1.07
O8	1.10	0.00
O6	2.59	0.21

Notes: 1. Refer to Figure 3 for sample location points.
2. n/a = not applicable (water depth sample was not taken at this location).

the site (BBL, 1996). These sampling events were completed to define the extent of the constituent of concern (COC) concentrations in the surface sediment. The specific COC's targeted included a ubiquitous organochlorine pesticide, DDT, and its metabolites, DDD, and DDE, as well as four trace metals (arsenic, copper, lead, and zinc). The sampling locations and chemical concentrations in the project site are illustrated in Figure 3 and Table II.

3. METHODOLOGY

3.1. Geophysical Investigation of Native Sediments

An investigation of the geophysical nature of native sediments at the project site was conducted to obtain information on the depth of native sediments, index properties, grain size distribution, organic

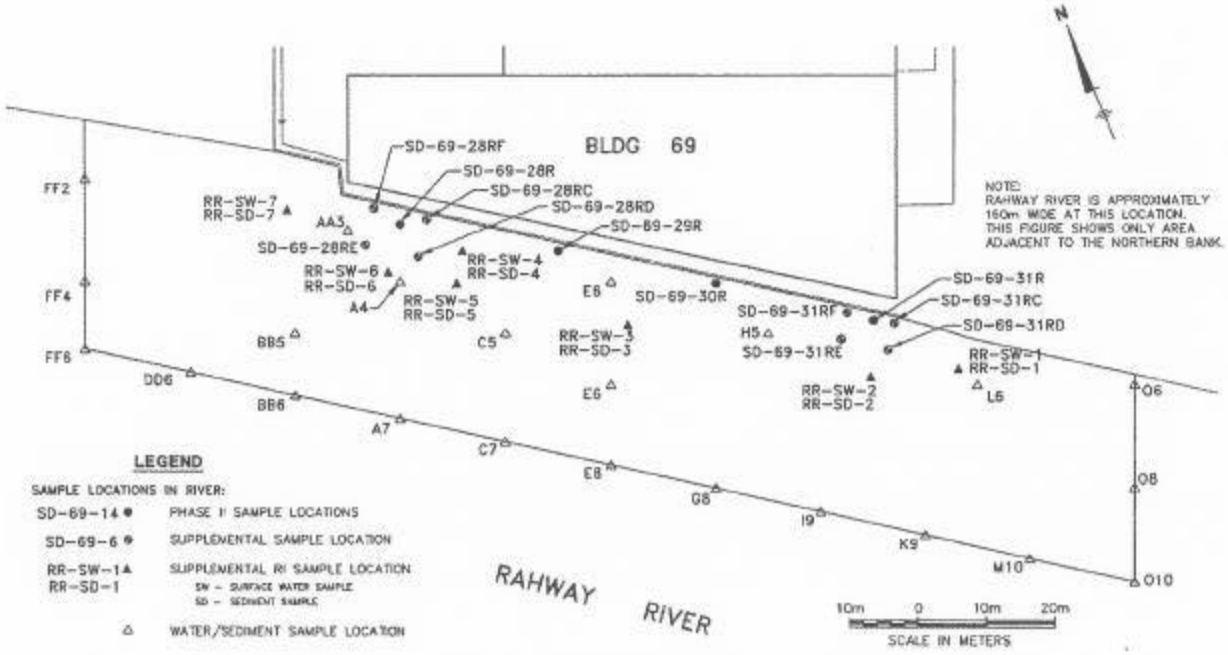


FIGURE 3 Sample location map.

TABLE II Chemical concentrations at the project site (mg/kg)

Sample location	*Arsenic*	*Copper*	*Lead*	*Zinc*	*DDD*	*DDE*	*DDT*
FF6	31	519	345	719	0.35	ND	1.10
FF4	34	850	354	859	0.36	0.07	0.23
FF2	52	850	427	1420	0.59	0.06	0.43
DD6	40	852	388	1120	0.73	ND	0.84
BB6	37	837	468	965	0.30	ND	1.30
RR-SW-7	N/A	1120	874	4500	3.80	ND	12
RR-SD-7	N/A	691	455	2470	31	ND	ND
SD-69-28RF	N/A	N/A	N/A	N/A	35	ND	130
SD-69-28RE	N/A	N/A	N/A	N/A	909	0.24	26
RR-SW-6	N/A	991	814	2180	5.10	ND	3.30
RR-SD-6	N/A	715	428	2550	28	ND	41
A7	57	699	400	1220	0.87	ND	1.70
SD-69-28RC	N/A	N/A	N/A	N/A	170	1.10	17
SD-69-28R	N/A	N/A	N/A	N/A	6.70	61	16
SD-69-28RD	N/A	N/A	N/A	N/A	5.50	0.19	5.50
RR-SW-5	N/A	654	520	1580	1600	ND	160
RR-SD-5	N/A	482	340	1150	290	ND	430
RR-SW-4	105	820	878	3950	7200	ND	5900
RR-SD-4	84	2740	506	1050	1400	ND	2300
C7	47	454	272	566	0.17	0.04	0.36
SD-69-29R	N/A	N/A	N/A	N/A	3.00	ND	ND
E8	43	895	459	908	0.18	0.07	0.56
RR-SW-3	160	11500	752	5250	13	ND	ND
RR-SD-3	246	16600	750	3620	2.70	ND	11
G8	48	1290	622	952	0.14	0.09	ND
SD-69-30R	N/A	N/A	N/A	N/A	0.70	0.16	ND
I9	49	914	505	947	0.28	0.10	1.10
SD-69-31RF	N/A	N/A	N/A	N/A	0.70	ND	1.30
SD-69-31R	N/A	N/A	N/A	N/A	32	ND	ND
SD-69-31RE	N/A	N/A	N/A	N/A	1.50	0.19	1.60
SD-69-31RC	N/A	N/A	N/A	N/A	0.36	ND	0.34
SD-69-31RD	N/A	N/A	N/A	N/A	0.49	0.05	0.13
RR-SW-2	156	1290	685	2910	8.70	ND	ND
RR-SD-2	77	947	522	2360	12	ND	3.90
K9	42	744	449	886	0.46	0.10	0.40
RR-SW-1	N/A	782	574	6360	45	ND	ND
RR-SD-1	N/A	600	558	2050	11	ND	5.50
M10	49	712	372	816	0.44	ND	1.60
O10	68	821	419	1090	0.35	0.15	0.22
O8	16	508	186	294	3.00	ND	ND
O6	29	364	354	475	0.93	ND	ND
NOAA-ER-M	85	390	110	270	no data	no data	no data

Notes: 1. Refer to Figure 3 for sample location points.
2. N/A = not applicable (the chemical was not sampled at this location).
3. ND = the contaminant concentration was below the detection limit.
4. NOAA-ER-M represents the ecological risk based limit of the contaminant suggested by NOAA.

content, consolidation and permeability, and strength and bearing capacity. Information on general subsurface conditions were collected using geophysical field surveys, sediment sampling, laboratory testing and visual inspection of the sediments.

Determining the depth of native sediments was important in defining the sediment matrix and in estimating the compressibility and settlement properties of the native sediment layer. Probing was used to estimate the depth of the water column and the thickness of the native sediments at the site. Grab samples were taken at select locations for laboratory classification and grain size testing. In addition, two cores were also taken using a 5 cm (~2-inch) diameter stainless steel sampler for laboratory consolidation samples. The sampling, probing and coring locations are shown in Figure 3. Laboratory geotechnical testing included index property tests (water content, dry density, specific gravity, and Atterberg limits), grain size tests (combined hydrometer and sieve analysis methods for the fine and coarse fractions, respectively), organic content tests and consolidation tests. Details of the test procedures can be obtained from ASTM (1993).

One-dimensional consolidation tests with complete lateral confinement was conducted on select samples to evaluate the effect of cap placement on mechanical compression of native sediments. The average consolidation settlement was modeled using Terzaghi's consolidation theory (Terzaghi and Peck, 1967) assuming that the total settlement would be comprised of primary settlement due to the expulsion of pore water from the sediments and secondary compression due to the time dependent creep of the organic component of the sediments. The primary settlement is also a function of the cap construction rate and methodology. For example, if the cap is placed in a series of vertical increments (lifts), localized differences in settling rates will be reduced considerably.

Primary consolidation settlement (ΔH_0) of the native sediments was evaluated as,

$$\Delta H_0 = [C_c/(1+e_0)][H_s \log_{10}\{(p_0+\Delta p)/(p_0)\}] \tag{1a}$$

$$p_0 = \gamma'_s(H_s/2) \tag{1b}$$

$$\Delta p = \gamma_c' H_c \tag{1c}$$

where, C_c is the compression index of the sediments, e_0 is the initial void ratio of the sediment layer, p_0 is the initial vertical effective stress, Δp is the additional vertical effective stress due to effective cap weight, γ_s' is the effective unit weight of sediments, γ_c' is the effective unit weight of the cap, H_s is the thickness of the native sediments, and H_c is the thickness of the cap.

Secondary consolidation settlement (ΔH_α) is due to the time dependent deformation (creep) of the native sediments under a relatively constant load and was evaluated as,

$$\Delta H_\alpha = H_s C_\alpha \log_{10} (t/t_0) \tag{2}$$

where, C_α is the coefficient of secondary compression, t is the time interval of interest, and t_0 is the beginning of secondary compression.

The total settlement (ΔH_T) of the native sediments due to stresses induced by the cap was then computed as the sum of primary and secondary settlements as shown below:

$$\Delta H_T = \Delta H_0 + \Delta H_\alpha \tag{3}$$

3.2. Remedial Design for Building 69

Building 69 is located immediately adjacent to the project site (see Fig. 2) and is supported on a timber pile foundation driven into an underlying sand stratum. Tidal and wave action had damaged the wooden bulkheads along the river edge of Building 69 and eroded most of the soil backfill from under the building, exposing some chemicals of concern. In order to rectify this situation, it was proposed to repair the deteriorated bulkhead with steel sheetpiles and to construct a sediment cap under the building by filling the void below the existing slab with a sand-cement-flyash mixture (commonly referred to as Controlled Density Fill or CDF).

Repair of the bulkhead by steel sheetpiles would serve two purposes: (a) it would act as a form for the CDF when it is being placed below the building; and (b) it would protect the fill below the building from

washout. The selected sheetpile section needed to resist the following forces: (a) tidal action, (b) wave action, (c) fluid CDF loadings before it hardens, and (d) lateral soil loads.

The following two scenarios were evaluated as the critical cases for design: (i) "the short-term condition", which covered the period of several hours before the CDF has hardened and begins gaining strength, and (ii) "the long-term condition", which assumed that the CDF has hardened and that it is not exerting direct pressure on the wall although the weight of the CDF would still cause a slight increase in the active pressures behind the wall due to the surcharge effect. Bending moments and required sheetpile penetration depths were then computed for a variety of material layers and loading conditions. The required sheetpile section was selected as,

$$S = M/F_y \tag{4}$$

where, S is the section modulus, M is the allowable moment for a factor of safety of 2.0, and F_y is the yield stress of steel.

3.3. In-place Sediment Containment Design

3.3.1. Aerial Extent and Cross-section of the Cap

The aerial extent of the cap was delineated to provide for an effective, implementable, and cost-effective remedy that would be protective of the human health and the environment. The extent of the cap was determined based on negotiations with the United States Army Corps of Engineers (USACE) and the New Jersey Department of Environmental Protection (NJDEP) to achieve the following objectives: (i) provide adequate isolation of the in-place contaminated sediments near Warners Plant, (ii) minimize chemical releases and other adverse effect at the site during the remedial process and (iii) achieve sediment remediation in an economical and environmentally beneficial manner (by providing isolation of the contaminants from the external environment). A cap consisting of a geotextile stabilizing layer, sand isolation layer, gravel (or geotextile) filter layer, and stone armor layer was selected for further investigation.

3.3.2. Geotextile Stabilizing Layer Design

The use of geotextile as a base layer would provide additional bearing capacity by forming a separation between native sediments and the sand cap layer, and would minimize the mixing of sediment and sand during cap construction. The main design criteria for the geotextile layer was the required strength to withstand the stresses due to the capping materials. Once this was known, the appropriate geotextile was selected from standard catalogs from the manufacturers (Mirafi, 1995).

3.3.3. Sand Isolation Layer Design

The design parameters for the sand isolation layer were determined using a theoretical evaluation of the following three aspects: (i) protection from chemical migration by diffusion, (ii) protection from chemical migration by advection/dispersion and (iii) protection from chemical migration due to sediment compression. These are discussed below.

Protection from Chemical Migration by Diffusion The breakthrough time (5 percent of maximum flux), time to achieve steady state (95 percent of maximum flux), and steady-state flux rate of chemicals through the sand layer portion of the cap were estimated using the equations presented in Wang *et al.* (1991) and Thoma *et al.* (1993).

The breakthrough time (t_b) was determined by solving,

$$0.95 = -2 \sum_{N=1}^{N=6} (-1)^N \exp\{(-D_t N^2 \pi^2 t_b)/L^2\} \tag{5a}$$

$$D_t = D_e / R \tag{5b}$$

$$R = \varepsilon + \rho_b K_p \tag{5c}$$

$$D_e = D_w \varepsilon^{1.33} \tag{5d}$$

$$K_p = K_{oc} f_{oc} \tag{5e}$$

where, t_b is the breakthrough time, L is the cap thickness, D_t is the transient transport effective diffusion coefficient, R is the retardation factor, D_e is the effective diffusivity, D_c is the chemical diffusivity, ε is the sediment porosity, K_p is the partitioning coefficient, K_{oc} is the chemical distribution coefficient, f_{oc} is the organic carbon content and ρ_b is the bulk density. These coefficients were determined for each of the chemicals from standard engineering manuals.

The steady state time, t_{ss}, (defined as the time when the flux is equal to 95 percent of maximum flux) was estimated as,

$$t_{ss} = (3.69L^2)/(D_t\pi^2) \tag{6}$$

The steady state flux (F_{ss} defined as the mass of contaminants released through the cap per year once the steady state condition is achieved), was computed by,

$$F_{ss} = D_e(P_o - P_w)/L \tag{7a}$$

$$P_o = C_s/K_p \tag{7b}$$

where, P_w is the water column chemical concentration, P_o is the average pore water chemical concentration and C_s is the depth averaged sediment chemical concentration, determined from field sampling.

The long-term flux (F_l) was estimated from,

$$F_l = P_o\{(D_eR)/(\pi t)\}^{0.5} \tag{8}$$

Protection from Chemical Migration by Advection/Dispersion The breakthrough and steady-state time associated with advective transport of chemicals through the cap was estimated using the solution to the one-dimensional advective/dispersive equation (Bedient *et al.*, 1985; Fetter, 1993; Ogata and Bank, 1961), incorporating a retardation factor to account for adsorption of chemicals.

$$C/C_o = 0.5\ \text{erfc}[(RL - Vt)/\{2(RD_Ht)^{0.5}\}] \tag{9}$$

where, C is the sand-armor interface chemical concentration, C_o is the sediment-sand interface chemical concentration, V is the groundwater velocity, t is the time, and D_H is the hydrodynamic dispersion coefficient (estimated from engineering manuals).

The value of the complementary error function was approximated using the first eight terms of the infinite series:

$$\begin{aligned}\operatorname{erfc}(x) = 1 - [\{2/(\pi)^{0.5}\}\{x - (x^3/3) \\ + (x^{2n-1}/\{(2n-1)[(n-1)!]\}) - \cdots\}].\end{aligned} \tag{10}$$

Protection from Chemical Migration due to Sediment Compression The potential for chemical migration into the sand isolation layer due to pore water release during sediment compression was evaluated by assuming that the volume change due to sediment consolidation was entirely due to the expulsion of pore water. The resulting mass migration rate of the dissolved contaminants (F_c) was estimated as follows:

$$F_c = 10 * \Delta H_T * C_t/(K_{oc} * f_{oc}) \tag{11}$$

where, ΔH_T is the total settlement of the native sediment layer, and C_t is the average concentration of the contaminant in the native sediment zone that will be available for the pore water transport (estimated from field sampling results).

3.3.4. Armor Layer Design

The main purpose of the armor layer was to protect the sand isolation layer from being eroded away due to hydraulic and environmental forces. In general, bottom velocities exceeding about 0.3 m/s will initiate sediment erosion (Vanoni, 1977). The design parameters for the armor layer were determined by considering the following: (i) river flow, (ii) wave action, (ii) propeller wash forces, and (iv) ice scour. Since the design events considered were rather extreme (meaning that they have a low probability of occurrence), a combination of the extreme river flow and maximum wave action case was not considered.

Protection from Extreme River Flow Maximum design river velocities were estimated as the higher of the river velocity and the tidal velocity during storm events. The Isbach equation (USACE, 1991b) was then used to determine the stable stone size based on expected bottom velocities as,

$$V = C[2g\{(\gamma_s/\gamma_w) - 1\}]^{0.5}(D_{50})^{0.5} \tag{12}$$

where, D_{50} is the median grain size of the armor, γ_w is the unit weight of water, γ_s is the unit weight of the armor stone, g is the gravitational constant, C is a constant (0.86 for high turbulence) and V is the bottom velocity.

Protection from Wave and Current Action The maximum expected wave action and resulting current velocities were estimated from storm records and visual observations during large ship traffic in the Arthur Kill. Resulting river bottom velocities were estimated using the Waterways Experiment Station computer program, ACES (USACE, 1985), which is based on well established wave theories outlined in the Shore Protection Manual (USACE, 1984). The Isbash equation was then used to compute the stone size which would be stable for a given velocity.

Protection from Propeller Wash Forces The effects of propeller wash were considered based on the work of Blaauw and Van de Kaa (1978) who suggested the following equation for maximum bottom velocity due to propeller wash forces from a navigating vessel

$$V_b(\max) = C_1 U_0 D_P / H_P \tag{13a}$$

$$U_0 = C_2[P_d/(D_P)^2]^{0.333} \tag{13b}$$

where, $V_b(\max)$ is the maximum bottom velocity, C_1 is a constant (0.22 for non-ducted propeller), U_0 is the jet velocity exiting the propeller, D_P is the propeller diameter, H_P is the distance from propeller shaft to channel bottom, P_d is the engine power and C_2 is a constant (9.72 for non-ducted propellers).

Protection from Ice Scour Ice scour becomes a potential concern during armor design if the possibility exists for ice being piled or ridged to the total depth (or a significant part) of the water column. The Shore Protection Manual (USACE, 1984) suggested that in general, ice piled on the shore by wind and wave action will not cause serious damage to rip–rap, but will provide additional protection against severe winter waves. Conversations with local residents and plant personnel indicated that no significant ice flows or ice damming have been observed at the site in recent years. However, a design value of 2758 KN/m^2 for crushing strength and 1034 KN/m^2 for blows from floating ice as recommended by the U.S. Army Corps of Engineers (USACE, 1984) was used for design.

3.3.5. Filter Layer Design

Hydraulic forces such as eddies (which are transmitted to the bottom through the voids in a protective layer), seepage forces (developed by percolating water), or a combination of both, may initiate movement of the bed material beneath an armor layer. A filter layer provides: (i) protection of the base material from direct attack by such hydraulic forces, and (ii) the ability to distribute the loads due to the armor layer evently to the foundation soils.

The Terzaghi-Vicksburg criteria (Terzaghi, 1943; USACE, 1949) was used as a design guideline to investigate the need for a filter layer and to design a filter layer. It consists of three criteria which a filter material must satisfy in order to provide hydraulic stability to the base material.

$$D_{15(F)} < 5d_{85(B)} \tag{14a}$$

$$20d_{15(B)} > D_{15(F)} > 5d_{15(B)} \tag{14b}$$

$$D_{50(F)} < 25d_{50(B)} \tag{14c}$$

where, $D_{15(F)}$ is the diameter through which 15 percent of the filter passes, $D_{50(F)}$ is the diameter through which 50 percent of the

filter passes, $d_{15(B)}$ is the diameter through which 15 percent of the base material passes, $d_{50(B)}$ is the diameter through which 50 percent of the base material passes and $d_{85(B)}$ is the diameter through which 85 percent of the base material passes.

The first criteria (Eq. 14a) prevents the largest base material grains from being washed into the pores of the filter material. Washout of the smaller grains will then be prevented by means of internal formation of filters in the base material (self-filtration). The second criteria (Eq. 14b) improves the drainage characteristics of the filter.

The Worman (1989) equation was then used to evaluate the stability of the protective layer (by computing the maximum velocities the cap could withstand), as given below.

$$(V^2/gS) = 6(d_{85}/D_{15}) \tag{15}$$

where, V is the mean flow velocity above the armor layer, g is the acceleration due to gravity, S is the riprap layer thickness, d_{85} is the 85 percent passing size of the filter material and D_{15} is the 15 percent passing size of the armor layer.

4. RESULTS

4.1. Geophysical Investigation of Native Sediments

The water depths and sediment thicknesses at the site are summarized in Table I. In general, the water depth at the site ranged from 1 to 3 m, with an average value of 2.3 m. The average water depth within the area to be capped was about 1.8 m. Native sediments at the site were comprised of a top layer of silty sediments (varying thickness; typically 0.3 to 0.9 m) underlain by a clay layer (varying thickness; typically 4.6 to 6.1 m). Below this was a layer of sand and gravel sediments (typical thickness of 3 to 7.6 m). The layer was underlain by a weathered bedrock layer. In general, the thickness of native sediment layer within the area to be capped varied from 0.2 to 0.9 m, with an average value of about 0.6 m. The water content of the native sediments ranged from 40–159%, with an average value of 142%. The optimum dry density of the native sediment was 1171 kg/m^3 at an optimum water content of

27%. The specific gravity of the native sediments varied from 2.48 to 2.78, with an average of 2.58. The liquid limit varied from 39–124% (92%, on average), while the plastic limit varied from 28–39% (367%, on average). The plasticity index varied from 9–85%, with an average value of 56%.

Visual observation of the samples collected indicated that the sediments at the site consisted of 'black silty sand' within the area to be capped, with some 'black clay with varying quantities of fine to medium sands' along the perimeter of the area to be capped. Based on the index properties, the sediments were classified as 'inorganic clays and silts with high plasticity' according to the United Soil Classification System (Terzaghi and Peck, 1967). The grain size distribution curves (Fig. 4) were 'well-graded' with a uniform distribution of particle sizes. The test data indicated that the native sediments had a median grain size (d_{50}) of 0.05 mm, a d_{15} of 0.011 mm (15 percent passing size), and a d_{85} of 0.23 mm (85 percent passing size) respectively. Organic content of the native sediments varied from 2.5–9.7%, with an average value of 8.7%. This indicated that the native sediments were compressible and may exhibit secondary consolidation settlement over long intervals of time (several decades).

The consolidation and settlement computation was based on the values for engineering properties of the native sediments derived from laboratory test results. The average total settlement of the native sediments due to the stresses induced by the cap was estimated to be about 49 cm ($\sim$19 inches). The permeability of native sediments was estimated to be 2×10^{-8} cm/s from the results of one-dimensional consolidation tests using standard engineering correlations (Terzaghi, 1943; MacFarlane, 1969).

4.2. Remedial Design for Building 69

Geotechnical data suggested that the site soils form a layered system (BBL, 1996). The upper 1.5 to 2.4 m layer appeared to have been eroded away by long-term wave and tidal action. These materials would be replaced by the CDF with a unit weight of 2145 kg/m^3 as discussed in Section 3.2. The layer underlying the eroded material was granular in composition (with a unit weight of 2145 kg/m^3 and a friction angle of 28 degrees) and extended to about the water table

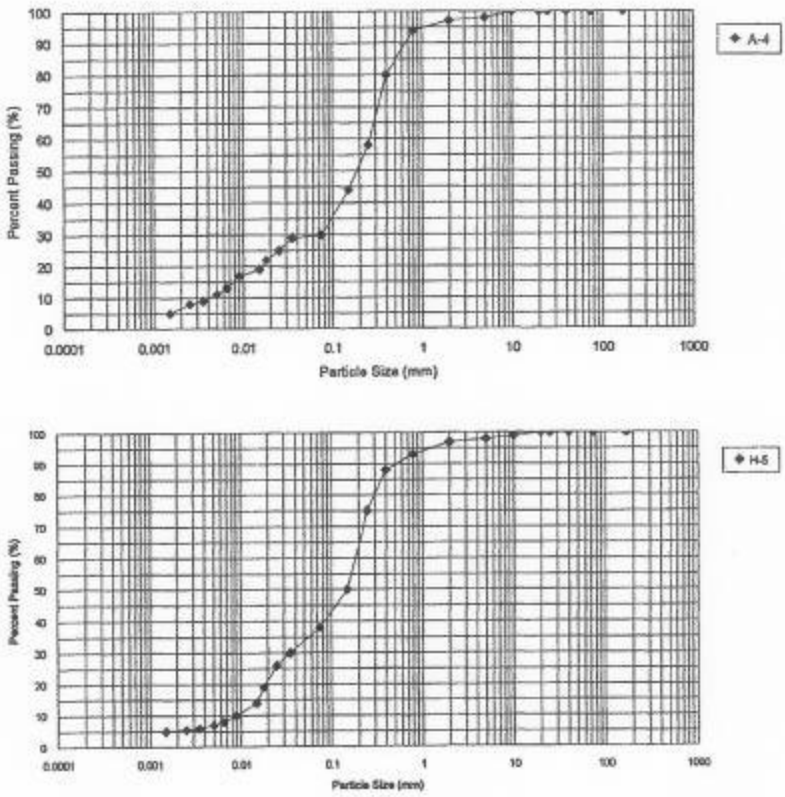


FIGURE 4 Grain size curves of typical native sediment samples.

elevation (about 2.4 m below the floor slab). The native soils below this layer consisted of a soft, cohesive, interbedded peat and silty clay deposit. For this design, it was assumed that these native peats and clays have a cohesive strength of 194 kg/m^2 which increases with depth at a rate of $0.22\gamma'h$ below 3.7 m from the floor slab. Further, since these sediments were permanently under water, they were assumed to be highly compressible with a bulk density of 636 kg/m^3 (estimated based on geotechnical test data).

The maximum moment computed for the short-term case was 337 KN-m with a required penetration depth of 13.7 m (*i.e.*, 5.2 m into the sand foundation layer). For the long-term case, an elevated water

head of 0.9 m was assumed to act in addition to the active pressure due to tidal effects (based on historical data for the site). The resulting maximum moment was 321 KN-m, with a penetration depth of 13.4 m (*i.e.*, 4.9 m into the sand foundation layer).

A cantilever sheetpiling system was recommended for the site since tie-back construction would be difficult and the cantilever system would be less susceptible to failure due to localized corrosion above the water line. Based on these results, a design sheetpile section capable of withstanding bending moments of 337 KN-m and a penetration depth of at least 5.2 m into the sand foundation layer was required for Building 69 remediation. Accordingly, a Bethlehem Steel PZ40 sheet pile section with a yield strength of 35 KN/cm^2 was selected for construction.

4.3. Sediment Containment Design

4.3.1. Aerial Extent of the Cap

Based on risk-reduction criteria and negotiations with the USACE and the NJDEP, the proposed cap was placed over a 0.57 acre area (Fig. 5) and would achieve isolation of chemicals at the Rahway River adjacent to the Warners Plant site. In addition, to assure that the affected sediments would be immobilized by the cap, the cap design extended an additional 3 m in all directions (with the exception of the side which abuts the proposed bulkhead) beyond the required extent determined from negotiations.

4.3.2. Geotextile Layer Design

Based upon the review of geotechnical test results of native sediments from the site, a polypropylene geotextile was recommended for this application in order to evenly distribute the weight of the capping materials and to minimize the resuspension and/or mixing of native sediments during the cap construction phase. Accordingly, a non-woven polypropylene geotextile (Mirafi 1100N or equivalent) with a density of 339 g/m^2, with a Mullen Burst Limit of 3617 kPa was recommended for use at this site (Mirafi, 1995). This type of geotextile

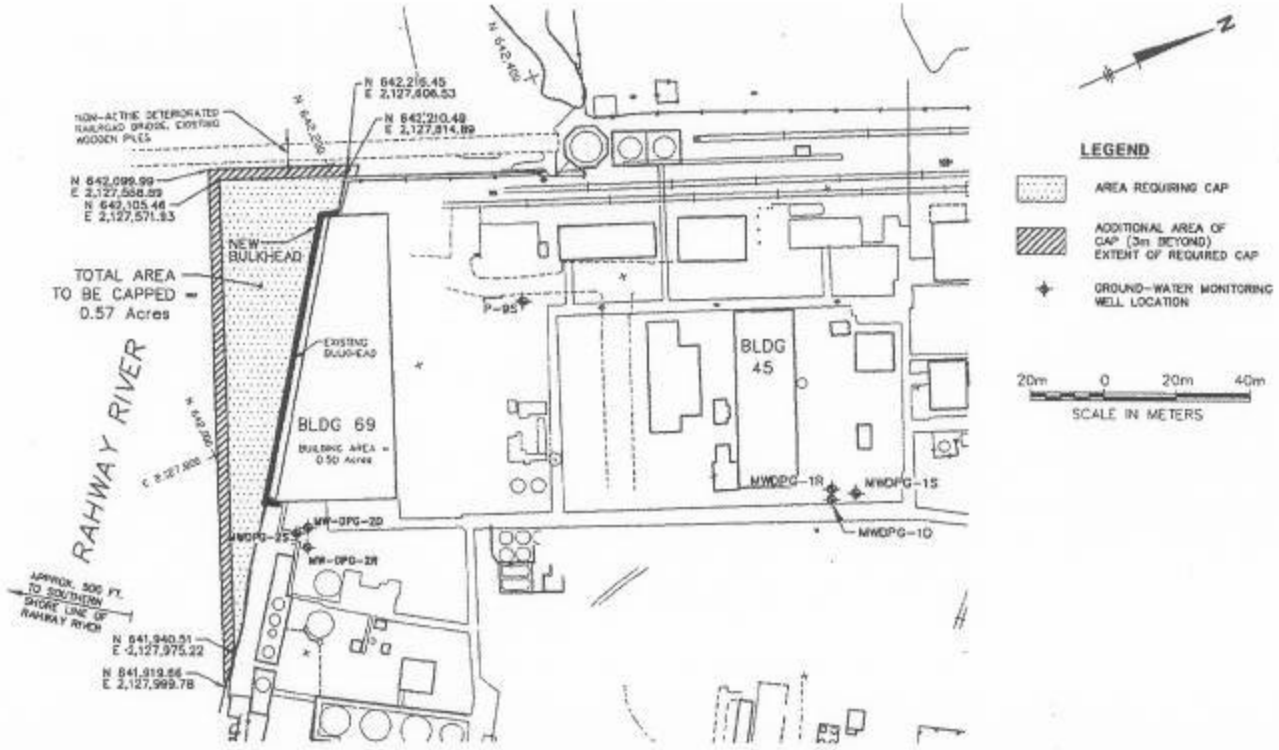


FIGURE 5 Aerial extent of the cap and remediation area.

will float and can subsequently be weighted to sink in order to facilitate construction.

4.3.3. Sand Layer Design

A design sand layer thickness of 46 cm with an organic carbon content of 0.5 percent was assumed for this analysis. The gradation of the sand layer was as follows: (i) d_{15} = 0.50 mm (15 percent passing size), (ii) d_{50} = 2.00 mm (50 percent passing size; median grain size), and (iii) d_{85} = 3.80 mm (85 percent passing size). The proposed sand layer thickness was based on the work of Gunnison *et al.* (1987) which recommends a 22 cm cap as being sufficient to "seal the contaminated sediments from the overlying water column". Bioturbation effects were assumed to be limited by the thickness of the filter layer and the armor layer. Accordingly, a 46 cm sand cap layer would mitigate the diffusive and/or advective transport of chemicals at the site. The thickness of the sand cap was evaluated separately for protection from chemical migration due to diffusion, and advection/dispersion, as shown below.

Protection from Chemical Migration by Diffusion This analysis was based on the following assumptions: (i) a design sand cap thickness of 46 cm, (ii) a value of 0 ppm for the background river water chemical concentration at the site, (iii) the depth averaged chemical concentration in the sediments would remain constant over time, (iv) local equilibrium and linear partitioning exists between the sediment (or cap) particles and the aqueous phase, (v) no resistance to chemical migration would be provided by other components of the cap, and (vi) other conservative assumptions (those which tend to predict greater rates of migration) including no fresh sediment accumulation over the cap over time, and no long term degradation of the chemicals within the native sediments.

Diffusive breakthrough analysis (Tab. III) indicated that the earliest breakthrough time occurs for DDE (456 years) while the latest breakthrough time occurs for Lead (13,427 years). Sample breakthrough curves are presented in Figures 6–8. The steady-state flux and steady-state time for the chemicals are shown in (Tab. III), which indicates that placement of the cap would achieve a reduction of about

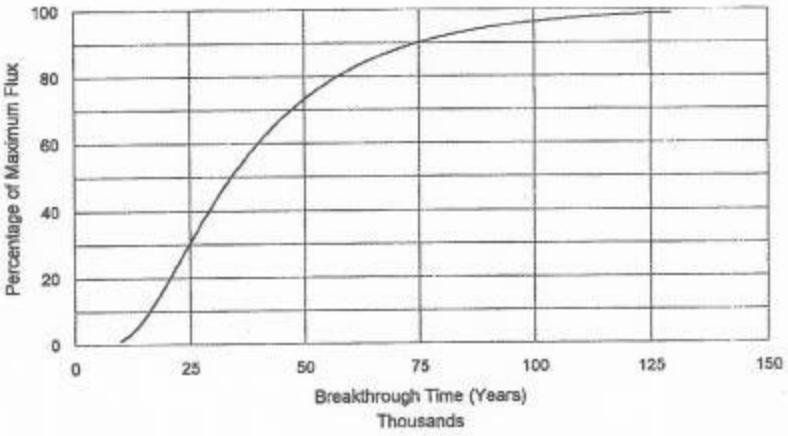


FIGURE 6 Diffusive breakthrough curve for lead.

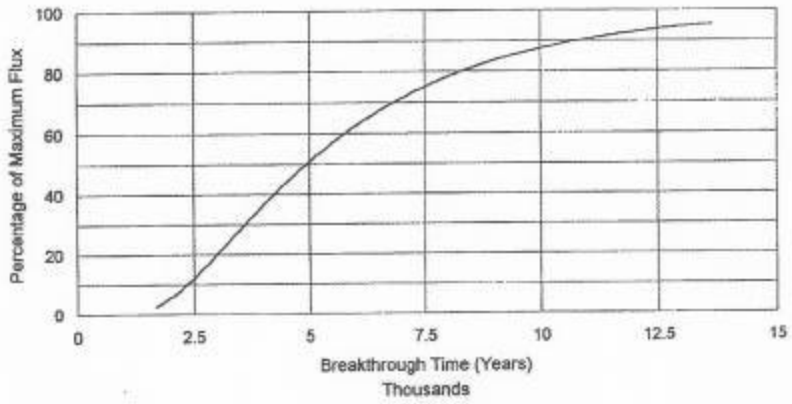


FIGURE 7 Diffusive breakthrough curve for DDT.

99 percent in chemical release to the water column when compared to the current conditions. It should also be mentioned here that these results should be considered to be conservative since they do not account for any flux reduction due to depletion of chemical mass by migration, chemical degradation over time. Also, the effects of new sediment deposition over the capping materials over time were not considered.

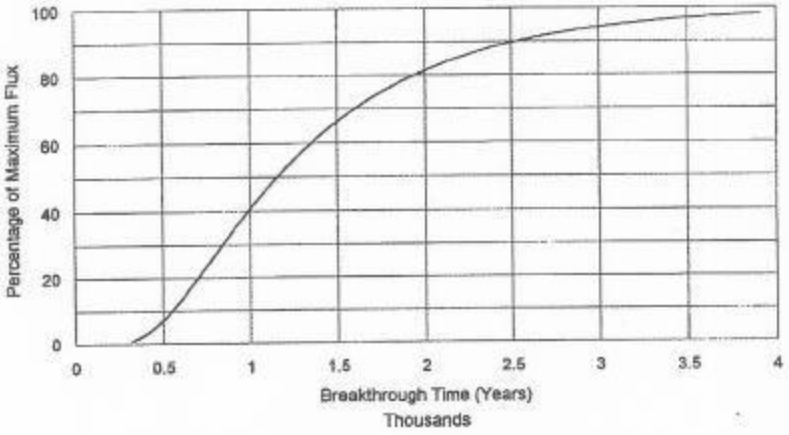


FIGURE 8 Diffusive breakthrough curve for DDE.

Protection from Chemical Migration by Advection/Dispersion A conservative value of $1 \times 10^{-10}\,m^2/s$ (Tchobanoglous and Schroeder, 1987) was used for the hydrodynamic dispersion coefficient (D_H) in Eq. (9). In order to further increase the conservative nature of the analysis, pore water velocity (0.2 cm/day) was used in Eq. (9) instead of the low groundwater velocity at the site.

Results of computer-based analysis of the advective/dispersive breakthrough curve using the above relationships and parameters are summarized in Table IV. The breakthrough curves are presented in Figures 9–11. It can be noted from Table IV that the advective/dispersive breakthrough time ranged from 91 years for DDE (corresponding flux of 0.05 gm/yr/acre) to 3398 years for Lead (corresponding flux of 0.5 gm/yr/acre). The steady-state flux and steady-state time for the chemicals are shown in Table IV, which indicates that the advective/dispersive steady-state fluxes are well below the concentrations of these chemicals currently present at the site. It should also be mentioned here that these results should be considered to be conservative since they do not account for any flux reduction due to depletion of chemical mass by migration, or chemical degradation over time. Also, the effects of new sediment deposition over the capping materials over time were not considered.

TABLE III Summary of chemical migration analysis by diffusion

	Arsenic	*Copper*	*Lead*	*Zinc*	*DDD*	*DDE*	*DDT*
Breakthrough time (yr)	1347	1653	13427	3366	853	456	1950
Breakthrough flux (gm/yr/acre)	0.05	1.15	0.02	0.40	0.18	0.00	0.06
Steady-state time (yr)	9202	11292	91723	22991	5823	3112	13319
Steady-state flux (gm/yr/acre)	0.91	21.86	0.40	7.70	3.34	0.03	1.12
Maximum_flux with cap (gm/yr/acre)	0.96	23.00	0.42	8.11	3.51	0.03	1.18
Maximum flux_without cap (gm/yr/acre)	337	8942	469	4496	980	6	496
Percent reduction in flux due to cap	99.72	99.74	99.91	99.82	99.64	99.51	99.76

TABLE IV Summary of chemical migration analysis by advection/dispersion

Parameter	*Arsenic*	*Copper*	*Lead*	*Zinc*	*DDD*	*DDE*	*DDT*
Breakthrough time (yr)	566	755	3398	1510	174	91	378
Breakthrough flux (gm/yr/acre)	0.64	13.98	0.64	4.99	4.77	0.05	1.68
Steady-state time (yr)	888	1184	5326	2367	273	143	592
Steady-state flux (gm/yr/acre)	12	265	9	95	91	1	32
Maximum flux (gm/yr/acre)	13	279	9	100	10	1	34

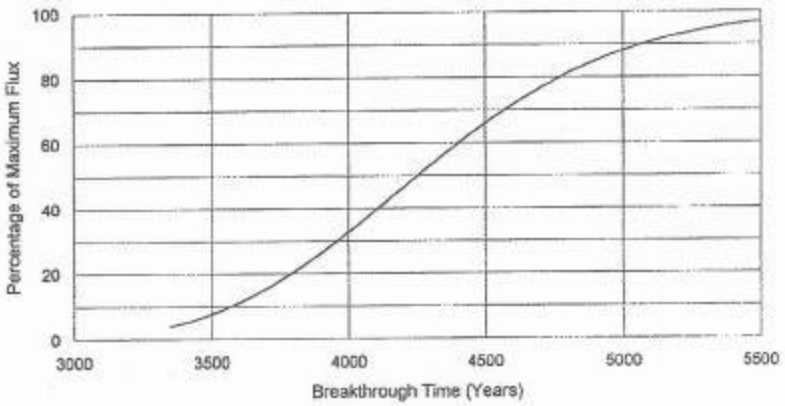


FIGURE 9 Advective/dispersive breakthrough curve for lead.

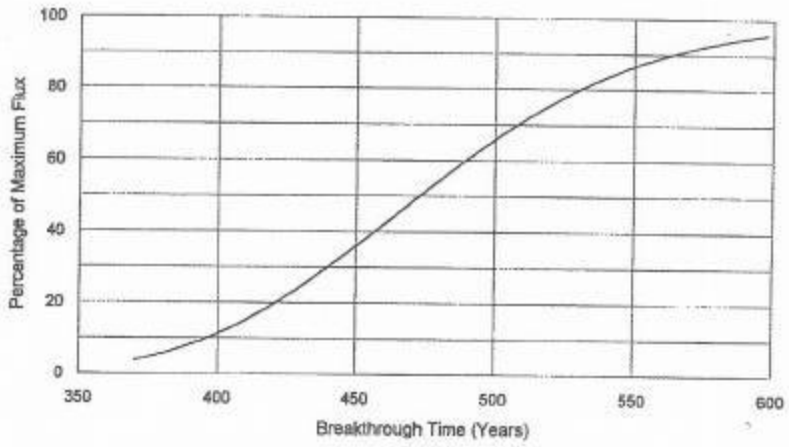


FIGURE 10 Advective/dispersive breakthrough curve for DDT.

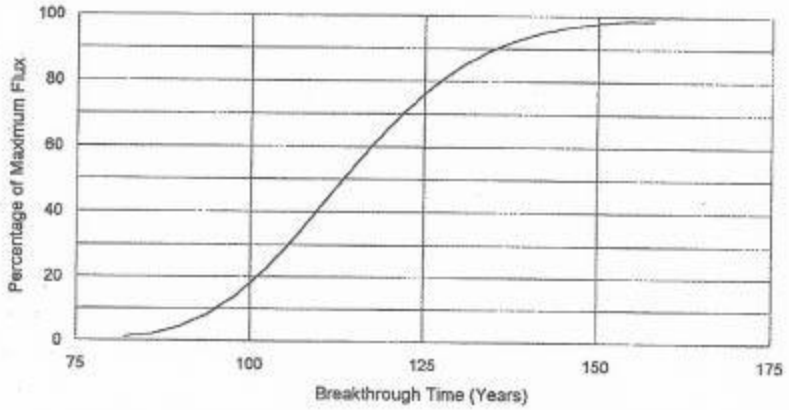


FIGURE 11 Advective/dispersive breakthrough curve for DDE.

Protection from Chemical Migration During Sediment Compression The computation was based on the following conservative assumptions: (i) sorption of chemicals onto the sand cap materials would be negligible, (ii) porosity of sand of 0.4, (iii) porosity of native sediments of 0.5, (iv) average sediment compression of 49 cm (from settlement computations), (v) surface area of the cap of 0.57 acres, (vi) thickness of sand cap of 46 cm, (vii) depth-averaged chemical concentration (as shown in (Tab. V), and (viii) native sediment TOC of 8.7%.

TABLE V Chemical release due to pore water flux during sediment compression

Parameter	*Arsenic*	*Copper*	*Lead*	*Zinc*	*DDD*	*DDE*	*DDT*
Water released from sediment (m)	0.67	0.67	0.67	0.67	0.67	0.67	0.67
Average chemical concentration (ppm)	140	4117	615	2954	324	1.45	248
Equilibrium pore water concentration (mg/L)	0.01073	0.23661	0.00785	0.08489	0.08096	0.00069	0.02851
Volume of water released from sediment (L)	772769	772769	772769	772769	772769	772769	772769
Volume of water released from cap (L)	421510	421510	421510	421510	421510	421510	421510
Mass of chemical released from sediment (gm)	8	183	6	66	62	1	22
Mass of chemical released from cap (gm)	5	100	3	36	34	0	12

The results of this computation are presented in Table V, which showed that the expected release of chemicals from native sediments due to compression range from 0.9 gm for DDE to 268 gm for Copper. The mass of chemicals released to the water column from the sand cap range from 0.5 gm for DDE to 168 gm for Copper. This indicated that even under conservative assumptions, chemical release into the water column due to native sediment compression is low, when compared to releases under the existing conditions.

Recommended Thickness and Gradation The above analyses confirmed the suitability of the proposed sand layer in providing adequate isolation to the in-place contaminants. Therefore, a 46 cm thick sand layer (with a minimum organic content of 0.5 percent) which meets the following specifications was recommended for use at this site: (i) $d_{15} = 0.50$ mm (15 percent passing size), (ii) $d_{50} = 2.00$ mm (50 percent passing size; median grain size), (iii) $d_{85} = 3.80$ mm (85 percent passing size), (iv) TOC = 0.5 percent (minimum).

4.3.4. Armor Layer Design

The armor layer design was based on conservative assumptions to account for the horizontal bottom velocities resulting from the combined action of a storm induced surge coupled with unusually high wave action in the region as described below.

Protection from River Flow The average river flow velocity at the site was estimated to be on the order of 0.3 m/s based on field measurements. Hydrographic data from the U.S. Army Corps of Engineers (USACE, 1961) indicated that the 100 year river flow velocity at the site was about 0.9 m/s. The corresponding velocity at the bottom of the river would be much less (about 50 to 70 percent) based on standard river velocity profiles developed by the U.S. Geological Survey (Rantz, 1982). However, to be conservative, a river bottom velocity of 0.9 m/s was considered in this design for the armor layer. Accordingly, a 3.6 cm diameter armor stone (median size) would be required to yield sufficient protection to the sand layer for river bottom flow velocities up to 0.9 m/s.

Protection from Wave Action The waves at the site were primarily caused by navigational ship traffic at the nearby Arthur Kill. The largest vessels navigating the Arthur Kill are commercial vessels (with approximate principal dimensions of 76 m (Length), 15 m (Width), and 12 m (Draft) respectively) with an engine power of about 20,000 HP (estimated based on USCG, 1995; Munro-Smith, 1973). Field observations and conversations with local residents and plant personnel indicated that the maximum wave heights observed at the site are on the order of 0.3 to 0.6 m, which would generate bottom velocities of the order of 0.37 to 1.2 m/s. However, conservatively assuming that large commercial vessels navigating the Arthur Kill would produce waves as high as 0.9 to 1.2 m under the worst weather conditions, the potential wave-induced bottom velocities are of the order of 1.1 to 1.7 m/s, depending on the wave theories used for analysis (USACE, 1984). Using a conservative assumption of 1.5 m high waves combined with a 0.9 m storm surge (which would inundate the shoreward portion of the cap) at the site (corresponding to a 50-year event), the resulting bottom velocities (after accounting for the effects of wave diffraction and reflection by coastal steel bulkheads) were estimated to be about 2.2 m/s. Therefore, for a conservative armor design for yielding protection against wave action, storm surge and wave diffraction/reflection at the site, the design velocity was 2.2 m/s. Accordingly, a 20 cm diameter armor stone (median size) would be required to yield sufficient protection to the sand layer for storm wave-induced bottom velocities up to 2.2 m/s.

Protection from Propeller Wash Forces As mentioned in the previous section, the largest vessels navigating the Arthur Kill are commercial vessels with an engine power of about 20,000 HP. However, the propeller-induced bottom velocities created by such traffic would be restricted to within the navigation channel (note that the area to be capped is about 245 m from the navigation channel). Further, these deep draft vessels would be unable to maneuver the shallower depths at the cap site. However, the propeller-induced bottom velocities at the cap site were estimated to be 1.7 m/s, for a conservative assumption of a 300 HP recreational craft (with a propeller diameter of 0.46 m) passing right over the cap. Based on USACE guidance for design of hydraulic channels (USACE, 1991b), the Isbash equation was then used to compute the stone size which would be stable for a given velocity. Accordingly, a 11.6 cm diameter armor stone (median size) would be required to yield sufficient protection to the sand layer for propeller wash-induced bottom velocities up to 1.7 m/s under the conservative scenario assumed for this analysis.

Protection from Ice Scour Conversations with local residents/plant personnel indicated that no significant ice flows or ice damming have been observed at the site in recent years. The U.S. Army Corps of Engineers (USACE, 1984) recommends a design value of 2758 KN/m^2 for crushing strength and 1034 KN/m^2 for blows from floating ice. Conservatively assuming that a sheet of ice would exist at the site during winter, a 46 cm thick layer of 20 cm diameter armor stone would protect the sand layer from ice scour/damage with a factor of safety of about 2.

Recommended Thickness and Gradation Based on the above analysis, a 20 cm diameter (median size) armor stone would protect the sand layer from bottom velocities of up to 2.2 m/s thereby protecting it from the conservative river flow, combined storm/wave action, propeller wash forces, and ice scour conditions assumed for the site.

The U.S. Army Corps of Engineers (USACE, 1991a and b) recommends the following guidelines for armor layer thickness: (i) 1.5 times median grain diameter (d_{50}), or (ii) 1.0 times maximum grain diameter (d_{100}), whichever is greatest. They further recommend that

these values be increased by 50 percent for underwater placement. Therefore, the recommended thickness for the armor layer was 46 cm. Accordingly, an armor stone layer which meets the following specifications was recommended for use at this site: (i) d_{15} = 15 cm (15 percent passing size), (ii) d_{50} = 20 cm (50 percent passing size; median grain size), (iii) d_{85} = 28 cm (85 percent passing size), (iv) layer thickness of 46 cm.

4.3.5. Filter Layer Design

Based on the specified properties of the sand layer (d_{15} = 0.50 mm, d_{50} = 2.00 mm, d_{85} = 3.80 mm) and armor layer (d_{15} = 15 cm, d_{50} = 20 cm, d_{85} = 28 cm), it follows that none of the filter criteria would be satisfied. Therefore, hydraulic forces at the site could potentially erode the sand layer material. In order to prevent this from occurring, a filter layer was recommended for use at this site.

Using Worman's equation, a filter layer with a d_{85} of 3.2 cm would yield sufficient protection to the underlying sand layer from hydraulic forces of upto 2.4 m/s. The specifications of the filter layer were then selected in order to satisfy the Terzaghi-Vicksburg criteria (d_{15} = 0.6 cm, d_{50} = 1.9 cm, d_{85} = 3.2 cm). Accordingly, a filter layer thickness of 11.4 cm would be required for an armor layer thickness of 46 cm. However, considering the constructability of such a thin layer underwater, a filter layer thickness of 15 cm would be more appropriate. However, this would further increase the thickness of the cap and subsequently the loading on the native sediments. Therefore, a non-woven polypropylene geotextile (Mirafi 1100N or equivalent) with a density of 339 g/m^2, with a Mullen Burst Limit of 3617 kPa was recommended for use at this site (Mirafi, 1995).

5. CONCLUSIONS

An *in-situ* sediment cap consisting of the components shown in Figure 12 would provide sufficient physical and chemical isolation to the affected sediments in Rahway River near Warners Plant: (i) *base stabilizing layer*: This consists of a non-woven geotextile fabric (Mirafi

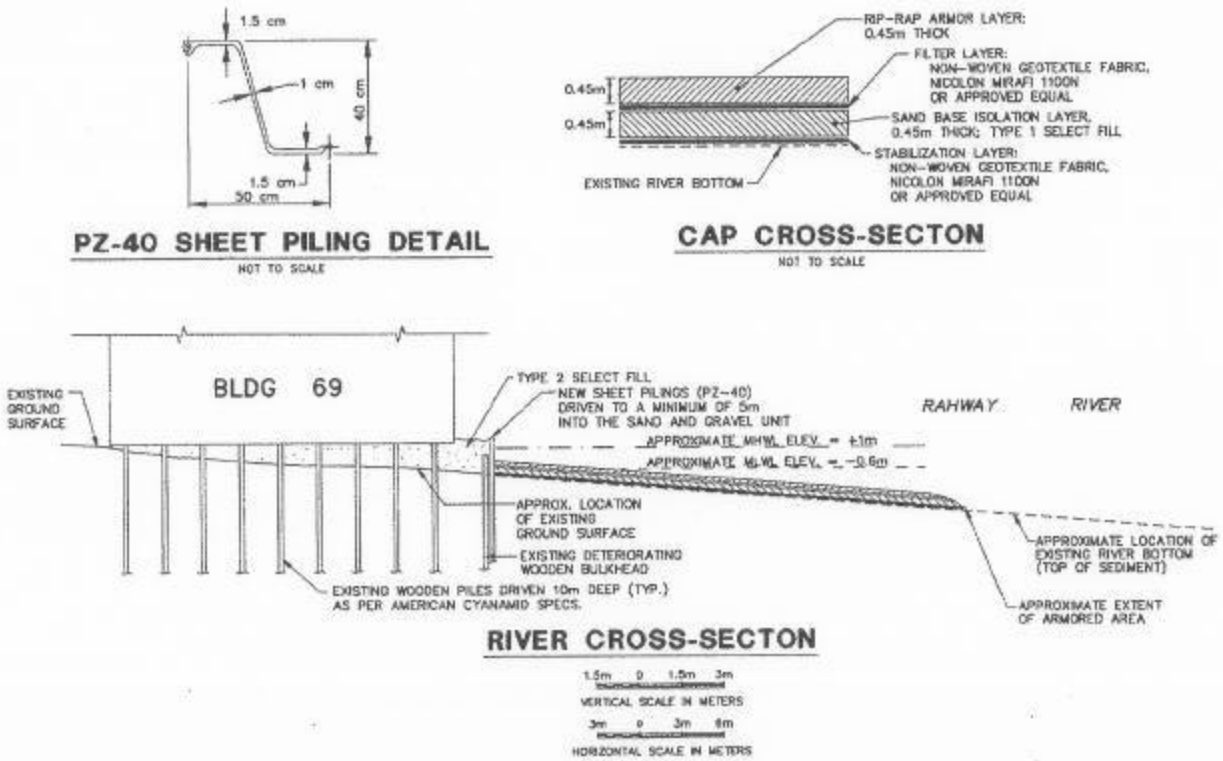


FIGURE 12 Cap cross section and details.

1100N or equivalent) to be placed over the native sediments in order to minimize mixing of the chemicals with the sand isolation layer during construction; (ii) *base isolation layer*: This consists of a 46 cm thick medium to coarse sand base ($d_{50} \sim 2.00$ mm) layer to isolate the chemicals from the environment; (iii) *filter layer*: This consists of a non-woven geotextile fabric (Mirafi 1100N or equivalent) placed over the sand layer to retard the winnowing of sand through the pore spaces of the armor layer due to hydraulic forces; and (iv) *armor layer*: This consists of a 46 cm thick stone layer ($d_{50} \sim 20$ cm) which will provide the sand layer protection against scouring under bottom velocities of up to 2.2 m/s (using conservative design assumptions).

The Rahway River cap was constructed during 1996 and is currently performing satisfactorily by providing complete isolation of the in-place contaminants from the water column, as expected. Thus, in many cases, capping can be an attractive *in-situ* remedial alternative which can be implemented with high precision and less adverse effects to the water column and biota, compared to other alternatives. It is hoped that this paper provides a cost-effective and environmentally attractive method for remediation of contaminated sediments in the coastal zone.

Acknowledgments

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NOTATION

ACES	Automated Coastal Engineering System
C	Sand-armor interface chemical concentration, or Constant (0.86 for high turbulence)
C_c	Compression index of the sediments
C_o	Sediment-sand interface chemical concentration

C_s	Depth averaged sediment chemical concentration
C_α	Coefficient of secondary compression
C_1	Constant (0.22 for non-ducted propeller)
C_2	Constant (9.72 for non-ducted propellers)
CMS	Corrective Measures Study
D_e	Effective diffusivity
D_H	Hydrodynamic dispersion coefficient
D_P	Propeller diameter
D_t	Transient transport effective diffusion coefficient
D_w	Chemical diffusivity
$d_{15(B)}$	Diameter through which 15 percent of the base material passes
$d_{50(B)}$	Diameter through which 50 percent of the base material passes
d_{85}	85 percent passing size of the filter material
$d_{85(B)}$	Diameter through which 85 percent of the base material passes
D_{15}	15 percent passing size of the armor layer
D_{50}	Median grain size of the armor
$D_{15(F)}$	Diameter through which 15 percent of the filter passes
$D_{50(F)}$	Diameter through which 50 percent of the filter passes
e_0	Initial void ratio of the sediment layer
f_{oc}	Organic carbon content
F_{ss}	Steady state flux
F_y	Yield stress of steel
g	Gravitational constant
H_P	Distance from propeller shaft to channel bottom
K_{oc}	Chemical distribution coefficient
K_p	Partitioning coefficient
L	Cap thickness
M	Allowable moment for a given factor of safety
NJDEP	New Jersey Department of Environmental Protection
P_d	Engine power (HP)
P_o	Average pore water chemical concentration
p_0	Initial vertical effective stress
P_w	Water column chemical concentration
R	Retardation factor
RI	Remedial Investigation

S	Section modulus, or Riprap layer thickness
t	time frame
t_b	Breakthrough time
t_0	Beginning of secondary compression
t_{ss}	Steady state time
U_0	Jet velocity exiting the propeller
USACE	United States Army Corps of Engineers
USEPA	United States Environmental Protection Agency
V	Bottom velocity, or Groundwater velocity, or Mean velocity above the armor layer
V_b	Maximum bottom velocity
Δh_α	Secondary consolidation settlement
ΔH_0	Primary consolidation settlement
ΔH_T	Total settlement
Δp	Additional vertical effective stress due to effective cap weight
ε	Sediment porosity
γ'_s	Effective unit weight of sediments
γ'_c	Effective unit weight of the cap
γ_s	Unit weight of the armor stone
γ_w	Unit weight of water
ρ_b	Bulk density

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