Environmental Geotechnics: Two Case Histories

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ABSTRACT

About 20 million gallons of liquid hazardous and toxic industrial wastes were disposed at the Hardage Site, about 35 miles south-southwest of Oklahoma City, from 1972 to 1980. Following Superfund designation of this site by the EPA, identification of a few hundred companies as Potentially Responsible Parties, and a court ordered excavate, incinerate, and re-entomb remedy for its remediation, many of the companies joined to form the Hardage Site Remedy Corporation (HSRC) for implementation of design, construction, operation, and maintenance of the facility. With the aid of a panel comprised of experts in the disciplines relevant to contaminated site remediation, an alternative remedy was developed and shown to be both more protective of the environment and more cost effective than the EPA remedy. The HSC was successful in its lawsuit for adoption of its plan. The key geotechnical components of the remediation included (1) demonstrating that the clay-shale formation underlying the site was intact and not susceptible to adverse interactions with the liquid wastes, thereby justifying the use of this formation as a bottom barrier; (2) determining the hydraulic properties of the soil formations above the bottom barrier and analysis of the NAPL and soluble contaminant transport, (3) the construction of a 2700 ft long, 67 ft deep (on average), and 3 ft wide gravel-filled trench, keyed into the bottom barrier, that serves to intercept wastes that migrate and diffuse from the buried waste liquid sources, and which would otherwise flow offsite, and (4) the design and construction of a low permeability composite cap over the disposal area.

In the second project described in this paper, plans for the redevelopment of a large rail yard area in Sacramento, CA were significantly impacted by the presence of a variety of soil contaminants in potentially liquefiable sandy soils. A portion of the approved remedy for this site included consolidation of contaminated soil in a fully lined and capped containment structure (know as the “rail berm”, or simply the “berm”) that would ultimately be used to provide secondary flood protection and would be used to elevate up to seven sets of rail tracks above grade. This project was significant in that it essentially represented construction of a waste containment facility in the middle of a major metropolitan area. As a result, both environmental and seismic safety issues had significant impacts on design and considerable subsurface investigation, laboratory testing, and engineering analyses were completed to address these issues. Perhaps the most noteworthy geotechnical issue for this project was subsurface soils along the alignment of the planned containment berm that were subject to liquefaction. One of the key aspects towards securing approval of this project was frequent communication with a number of regulatory organizations and project team members (both informally and through regular project meetings) to discuss the rationale for project concepts, planned investigatory procedures, the results of field and laboratory studies, and the results and implications of the design analyses. Through this process, early regulatory agency “buy-off” on project concept minimized the delays that frequently plague environmental projects. Notwithstanding the significant costs associated with a planning, analysis, and design process that addressed redevelopment of a 240 acre former industrial area and despite securing preliminary regulatory approval for construction of the containment structure, the project sponsor opted to terminate the project in favor of contaminated soil excavation and off-site disposal. The principal reasons for the change were not costs, geotechnical issues, or regulatory or public acceptance. Rather, land use plans for the property were modified to support sale of a portion of the property and an elevated rail corridor and its supporting berm were no longer necessary. Nonetheless, the studies and analyses that had been completed provide useful guidance for development of similar sites in the future.

These two case histories are illustrative of the types of geotechnical issues and problems that must be dealt with in remediation of contaminated sites and site development. Regulatory, legal, social, political, and economic considerations are often of equal or greater importance in reaching acceptable solutions than are the scientific and technical aspects of the project.

INTRODUCTION

The last two decades of the Twentieth Century witnessed the explosive growth of Geoenvironmental Engineering in response to the public’s demand for clean water, clean up of contaminated sites, safe disposal and containment of wastes of all types, and the protection of the environment for future generations. Geotechnical aspects of the many problems and projects stimulated development of Environmental Geotechnics, with its focus on soils, rocks, groundwater, and earthwork construction, and their roles in waste containment, waste landfills, contaminant transport, and site cleanup.

Owing to the newness of the field and the lack of prior experience, many projects have required development of new methods, new materials, and innovative solutions under conditions that were regulation driven and under intense public scrutiny. Success in many cases depended on the proper application of the "Observational Method" in spite of the requirements of a plethora of Federal and State laws and regulations; e.g., the Resource Conservation and Recovery Act (RCRA) of 1976 with its Amendments in 1984 and 1986 and the Comprehensive Environmental Response, Compensation, and Liability Act of 1980 (CERCLA), known popularly as "Superfund." In this paper we review and extract the lessons learned from two projects that are illustrative of the types of geotechnical issues and problems that must be dealt with in remediation of contaminated sites and site development for future beneficial use.

The Hardage hazardous and toxic waste disposal site in Oklahoma was targeted as an early Superfund site requiring cleanup. It is illustrative of the interplay of regulatory, technical, legal, and economic forces on the development and implementation of containment, cleanup, and long-term maintenance of a badly contaminated site. It shows also that good science and engineering can lead to more cost effective and environmentally protective solutions than might result without challenge of initially mandated remedies.

The Sacramento Railyard Project in downtown Sacramento, CA focuses on the geotechnical issues related to proposed redevelopment of a site underlain by contaminated, potentially liquefiable sandy soils. It illustrates how geotechnical issues can initially govern the design of remedial activities, but also how later decisions regarding land use and redevelopment can overshadow the geotechnical considerations.

THE HARDAGE SUPERFUND SITE CASE HISTORY

<u>Background</u>

This site, located in Criner, OK, about 35 miles south-southwest of Oklahoma City, served as the only permitted industrial waste disposal facility in Oklahoma from 1972 to 1980. Liquid wastes in both bulk form and in drums were received from almost 400 companies in Texas and Oklahoma. The bulk liquids were open dumped into the Main Pit or North Pond (see Fig. 1), and then transferred to temporary evaporation ponds, mixed with soil and placed in source areas. Most of the drummed wastes and sludges were placed in the Barrel Mound and west side of the Main Pit and covered with soil (Costello and Wogsland, 1997). About 20 million gallons of wastes were shipped to the site during its operating period. The wastes included pesticides, solvents, alcohols, waste oils, paints, acids, caustics, and metal sludges (EPA, 1983).

The U.S. Environmental Protection Agency (EPA) did site investigations from 1980 to 1986. In 1984 the EPA informed the many companies that had disposed their wastes at the site that they were designated as Potentially Responsible Parties (PRP) for cleanup under the provisions of CERCLA. The EPA's original remedy was to excavate the waste, incinerate it, and rebury it in a secure landfill, at an estimated cost of \$300 million. In response, 100 of the PRPs formed the Hardage Steering Committee (HSC) in 1985 to act on behalf of the PRPs in challenging the EPA-mandated judgments and remedies.

From 1986 until Fall 1989 the HSC, with input from an Expert Panel representing the geological, hydrological, geotechnical, geophysical, construction, and risk analysis disciplines, developed an alternative to the EPA's remedy. A trial before a Federal District Judge in late 1989 resulted in a decision in favor of the HSC remedy. Major factors in this decision were that the risks associated with excavation, transportation, and incineration associated with the EPA's plan were estimated at 1600 times greater than with the HSC plan described below. At the time of the trial the estimated cost of the EPA plan was about \$125 million, whereas, the HSC remedy was estimated at \$72 million. A court order in August 1990 told the HSC to implement its remedy.

In response, the HSC established the Hardage Site Remediation Corporation (HSRC) to oversee the design, construct, operate, and maintain the Court Ordered Remedy. Nationwide Environmental Services, Inc. serves as Technical Manager for the HSRC, IT Corporation was responsible for the design, and Canonie Environmental Services was the construction contractor.

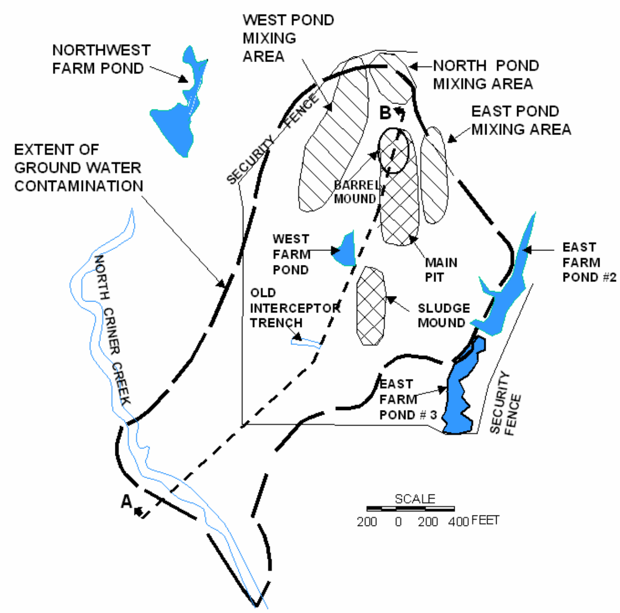

Fig. 1. Plan of the Hardage Superfund waste disposal site.

Site Conditions[1]

Geology. Fine-grained sandstones and siltstones comprise most of the surface geology. Unconsolidated alluvial sediments cover the southwestern and southern parts of the site along North Criner Creek (see Fig. 1). Six flat-lying geologic units, referred to as Stratum I through Stratum VI, Fig. 2, extend to a depth of about 300 ft:

Stratum I – sandstone and silty sandstone with some silty mudstone, exposed at elevations above 1090 ft.

Stratum II – predominantly mudstone and silty mudstone.

Stratum III – sandstone, with a one-ft thick marker bed at its top.

Stratum IV and V – about 180 ft of mudstone and silty mudstone, with some siltstone. Stratum IV is also referred to as the Bison Shale. These strata are laterally continuous across the site.

Stratum VI – sandstone and siltstone.

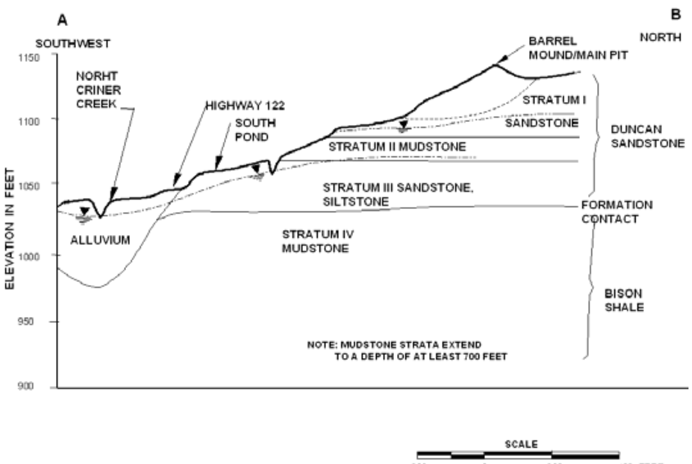

Fig. 2. Geologic profile at the Hardage Superfund site.

Material Properties. The shallow bedrock, Strata I, II, and III, contains fresh water that flows southwest towards North Criner Creek. Strata IV and V rock has very low hydraulic conductivity, of the order of 10^{-9} to 10^{-10} cm/sec in the vertical direction based on laboratory tests, and 10^{-10} to 10^{-12} cm/sec based on regional hydrologic simulation. There is no evidence of fractures in Strata IV and V through which significant groundwater flow is possible.

Extensive series of laboratory tests of the permeability, slaking and compatibility characteristics of the Stratum IV Bison shale when exposed to site groundwater and to non-aqueous phase liquids (NAPL) from the site indicated essentially no changes in the state or properties of the rock. A large decrease in permeability accompanied permeation with NAPL owing to clogging of pores and the interfacial tension between the original pore water and NAPL. Mineralogical determinations indicated illite and quartz, with minor amounts of smectites, to dominate the composition, with a small amount of iron oxide accounting for the red color of the shale. The non-expansive mineralogy, the low porosity, the stable structure in the slaking tests, and the continuity of the stratum across the site all supported the conclusion that it would provide an effective and permanent barrier to vertical migration of chemicals from above. This conclusion was essential to the site remediation plan that was developed by the HSC.

Extent of Groundwater Contamination. The estimated extent of migration of contaminants from the disposal areas is shown in Fig. 1. The distribution of chemicals was ascribed mainly to non-point surface infiltration sources originating at the pond mixing areas and waste runoff from the Barrel and Sludge Mound areas. Only a small portion of the alluvium is contaminated because of the relatively small amount of groundwater movement from the source zones.

Site Remediation Objectives

It was concluded from a Public Health and Environmental Endangerment Assessment (PHEEA) (ERM-Southwest, 1989) that human health and the environment would be protected if two pathways for exposure were eliminated:

1. Direct contact with affected materials in the source areas by humans or surface water.
2. Use of institutional controls and continuing supply of domestic water to the area residents from public water supply sources; thereby avoiding use of alluvial ground water adjacent to North Criner Creek.

The HSC remedy sought to attain these objectives by (IT Corporation, 1989):

1. Protecting North Criner Creek by meeting the surface water quality criteria of Oklahoma.
2. Preventing use of ground water on the original site and in the alluvium.
3. Allowing natural attenuation of chemical constituents in the alluvium by intercepting and treating the ground water emanating from the source areas and controlling surface water runoff.
4. Containing, capturing, and treating contaminated groundwater and NAPL coming from the source areas now and in the future.

[1] The information in this section is summarized from IT Corporation (1989)

5. Removing and treating liquids from areas where pumpable NAPL is found.

6. Limiting unauthorized access to the site.

7. Continuing public water supply to the area residents.

Remedy Components

Source Control. In 1985 EPA divided the site into source control and management of migration operable units. HSC proposed a source control comprised of several elements as described below and shown in plan on Fig. 3 (ERM-Southwest, 1988)

1. Construction of a 2-ft thick, 70 to 130-ft deep plastic concrete cutoff wall around the source zones. The wall was to penetrate about 20-ft into the Stratum IV Bison Shale, which, as noted earlier, was judged to be a thick, near impervious barrier against downward migration of contaminants.

2. Removal and destruction or treatment of liquids in the Barrel Mound using temporary vertical recovery wells and permanent horizontal drains at the base of the Mound.

3. Dynamic compaction of the Barrel Mound to reduce subsequent maintenance of a long-term cap.

4. Construction of an approximately seven ft thick cap over the previously compacted Barrel Mound. The cap was to consist of a compacted clay liner, geomembrane, and drainage layer. The purpose of the cap was to preclude generation of leachate by preventing infiltration of rain and surface water.

5. A system of recovery wells, screened within Strata I, II, and III, to reduce fluid levels in the source areas, thereby inducing inward flow from the cutoff wall.

6. Construction and operation of a system to treat the groundwater recovered from the source control area.

Pursuant to a court order, the HSC conducted a Remedial Investigation/Feasibility Study for the management and migration operable unit and submitted its report to the EPA in May 1989. It was then possible to develop a comprehensive site remedy that addressed both the source control and management of migration. Following the HSC vs. the EPA trial in Federal District Court in late 1989, as well as continuing discussions among the affected parties, a court order was issued in August 1990 in which the HSC remedy was selected, with some modifications, and the EPA remedy was rejected as unsafe and not protective of human health and the environment.

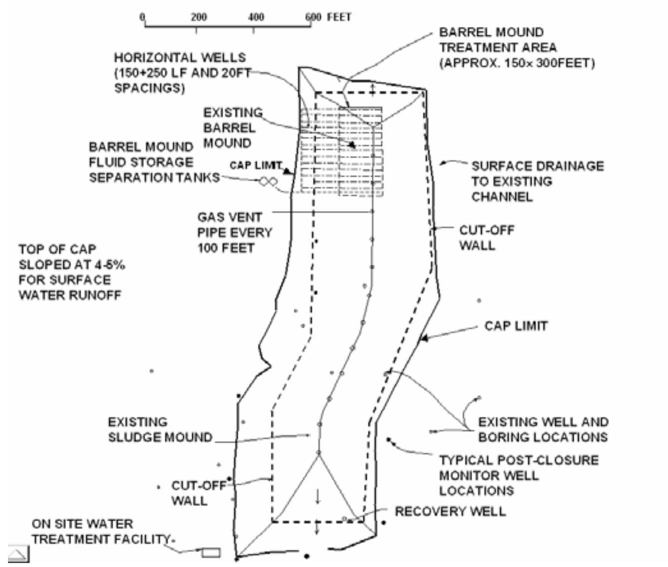

Fig. 3. Components of proposed source control remedy.

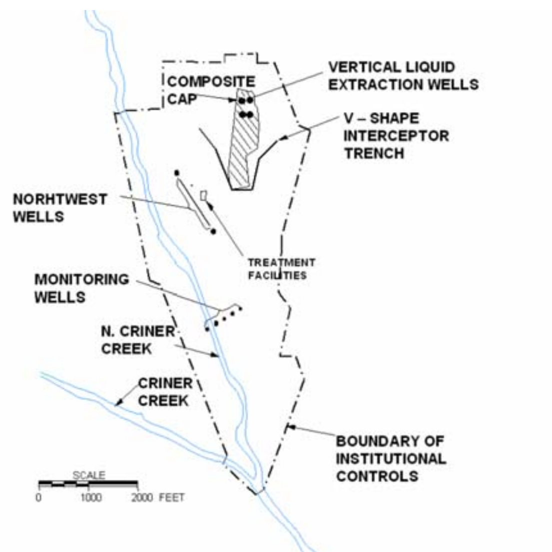

Fig. 4. Plan of approved final comprehensive site remedy.

Comprehensive Remedy. The final approved remedy that addresses both operable units retains, with some modifications, the same elements for source control as listed above, plus the following two additional components (see Fig. 4):

1. A series of Southwest interceptor wells to prevent migration of affected groundwater into North Criner Creek.

2. A groundwater and surface water monitoring system for evaluating the continued effectiveness of the remedy.

In addition, there were four significant geotechnical changes in the source control components:

1. Deep Dynamic Compaction was not used for compaction of the Barrel Mound area, probably a wise decision in view of the possibility of rupturing drums and spreading liquid waste throughout the area that might occur during the DDC process.

2. Liquid recovery wells were to be installed in the Main Pit as well as the Barrel Mound.

3. The cap over the Barrel Mound area was to have a compacted clay layer at least 2.5-ft thick with a hydraulic conductivity less than 1×10^{-7} cm/sec.

4. Owing to concerns by the EPA over the possibilities of fractures in the Stratum III rock, the buildup of fluid pressure against the cutoff wall, and leakage through the wall, the plastic concrete cutoff wall that would surround the source area (see Fig. 3) was replaced by a V-shaped gravel-filled interceptor trench, shown in plan in Fig.4. This innovative approach for preventing migration of NAPLs and confining contaminated groundwater within the source area boundary is described in more detail below.

The court issued an additional order on August 31, 1993 pertaining to several elements of the comprehensive site remedy (Costello and Wogsland, 1997). These included a reduction in the number of recovery wells in the Barrel Mound and Main Pit from 68 to 16, operational issues, the water treatment system, and on-site injection well operation. Also of interest in this court order was agreement with the HSRC that leaving a two- to three-ft layer of a viscous, tarry waste-sediment at the bottom of the Barrel Mound and Main Pit would be allowed, as attempts at removal would expose personnel to unacceptable health and safety risks.

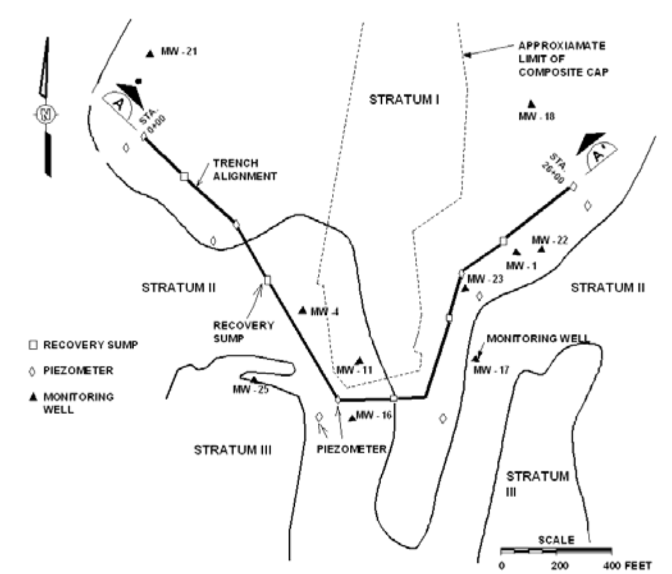

Fig. 5. Location and layout of the gravel-filled interceptor trench.

<u>V-shaped Trench</u>.

The gravel-filled V-shaped interceptor trench, located as shown in Fig. 5, is 3-ft wide, 2700-ft long, and an average of 60-ft deep, keyed a minimum of 2-ft into Stratum IV. A profile along the trench is shown in Fig. 6. The trench intersects and drains saturated zones of Strata I, II, and III. Numerical groundwater flow modeling was used to establish the trench location so that it would capture any up-gradient contaminated groundwater. The trench bottom is sloped to a series of liquid recovery sumps, with pumps for removal of the captured flow. The water level in the trench is maintained at elevation 1040 ft, or from 0 to about 15-ft above the Stratum III- Stratum IV contact. The performance of the trench segments is monitored by water level observations in the recovery sumps and piezometers located along the trench between the sumps.

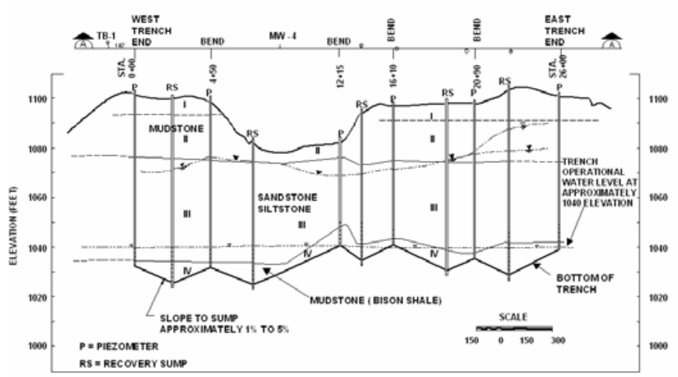

Fig. 6. Profile along the V-shaped interceptor trench showing rock strata, fluid levels, recovery sump and piezometers locations.

The free-draining trench backfill is a crushed rhyolite rock with 100 percent passing a 3-in screen, 0 to 10 percent passing a 1/2-in sieve, less than 1 percent fines, and a uniformity coefficient less than 2.5. The upper few feet of the trench are filled successively above the gravel with a 2-ft thick graded sand filter and clay cover to prevent flow of surface and runoff water into the trench,

A 100-ft long, 69-ft deep and 2-ft wide test trench was constructed along the western part of the alignment to evaluate constructability, stability, and hydrological testing (IT Corporation, 1993). Fifteen large diameter auger holes were drilled, and then a cable-operated clamshell and chisel were used to excavate the rock panels remaining between the auger holes. It was demonstrated that a trench of the specified dimensions could be constructed, the vertical trench walls would remain stable prior to backfilling without lateral support, and the gravel backfill could be placed by a tremie method.

A somewhat different excavation method was used for the main trench.[2] The contractor used a Kajima rig with an 80-ft long boom and counter-rotating auger and casing. All excavation was done using this rig; i.e., no clamshell was used for removing panels between auger holes. Instead spaced primary holes were drilled, and secondary holes were drilled to remove the material in between. About 90 percent material removal efficiency was achieved with the primary holes, and 70 percent efficiency was attained in the secondary holes. A video camera was mounted on the boom for inspection of the trench walls and bottom during construction. The specified ±6-in tolerance was met throughout the process. The bid price for excavation by this method was about $2 million less than by the clamshell method.

Sloughing of vertical sidewalls during construction only occurred along about a 40-ft long section that passed through poorly cemented sandstone in a shallow valley area. This problem was handled by sloping the sidewalls and modifying the filter arrangement to prevent migration of fines into the gravel.

The use of a cutoff trench would seem to be every bit as effective as a cutoff wall for containment of liquid wastes. It offers the additional advantages that liquids reaching the trench can be removed and treated, quantities are known, and an inward gradient is maintained. The trench requires continuously operable sumps and pumps, whereas, a barrier wall is passive. However, both systems must be monitored over the life of the project, and corrective actions, if needed, are likely to be more easily made in the trench system.

Project Performance.

The Hardage Site Remedy Corporation Superfund Site Update of February 2002 summarizes the current status and operational modifications. Among the main points are:

1. The site remedy has been operational continuously since September 1995.

2. The automated water treatment plant is operable 24 hours per day as required. By the end of 2001 over 53,000,000 gallons of water had been treated.

3. The V-Trench and Southwest Wells combined produce flow of about 18 gallons per minute.

4. The recovery wells in the Barrel Mound and Main Pit have removed almost 500,000 gallons[3] of aqueous waste and NAPL that has been shipped offsite for destruction by incineration.

5. A phytoremediation test plot was installed in March 2002 and is being monitored.

[2] Personal communication from Ben Costello, July 1, 2003.
[3] Updated quantity to July 2003, Personal Communication from Ben Costello, National environmental Services.

Costs

Costello (2000) lists the following costs for different elements of the Hardage-Criner Superfund site remediation:

1. Hardage Steering Committee cost to the end of the 1989 trial – about $25 million.

2. U.S. EPA cost to the end of the 1989 trial – about $17 million.

3. Hardage Site Remedy Corporation (HSRC) design cost – about $4 million.

4. Mounds Liquid Recovery System – about $4 million.

5. HSRC remedy construction cost (includes V-Trench, wells, pumps, monitoring systems, etc.) - $16.2 million for construction, $4.5 million for equipment and supplies. Overall this construction was complete on time, within budget, and with only 2 percent in change orders.

6. Present value of long term operation and maintenance – about $17 million.

The total HSC and HSRC cost of about $72 million compares with the EPA estimates for its solution of $300 million at the initiation of the Superfund process and $125 million at the time of the trial in 1989. How much might have been saved and what the adopted remediation plan might have been had the project been developed with the EPA and the HSC working cooperatively rather than adversarially remain questions for speculation and debate.

Conclusions and Lessons Learned

Developing and implementing a suitable plan for remediation of the Hardage-Criner Superfund Site was lengthy, litigious, and expensive. The outcome has been good in terms of achieving the objectives of health, safety, and environmental protection. In this sense, the Superfund Process has worked. By utilizing sound interdisciplinary science and technology that was made available through the HSC Expert Panel members, it was possible to challenge an EPA Record of Decision and gain court approval for an alternative site remedy.

This project was initiated relatively early in the life of CERCLA, and in the early 1980s a remove and treat approach to remediation was pervasive. Containment alternatives were only beginning to be given careful consideration as being environmentally acceptable, even where any other use of the site is unlikely in the future. As barrier technology has developed and field performance data have become available, containment has gained acceptance as suitable for many sites.

The Hardage site, with its intact, continuous, and nearly impervious Stratum IV bedrock serving as a bottom barrier, is well suited for vertical containment barriers. The gravel filled trench used to cut off and remove contaminated water and NAPL moving away from the source areas was innovative and should be a useful strategy for other sites.

THE SACRAMENTO RAILYARD CASE HISTORY

Background

The Sacramento Rail Yard was Southern Pacific Transportation Company's (SPTCo's) principal locomotive maintenance and rebuilding facility since 1863. From 1863 to 1980, the 240 acre Rail Yard was used to construct and overhaul locomotives, passenger and freight cars. SPTCo and the City of Sacramento, with support from the California Department of Toxic Substances Control (DTSC), the community, and other entities in the greater Sacramento area embarked on a substantial planning process to create a plan for the development and future reuse of the Rail Yard. The plan called for more than 2,500 units of medium- to high-density housing, over nine million square feet of office space, approximately 30 acres of parks and open spaces, a retail shopping area, and a state-of-the-art transportation center to be located at the north side of the development.

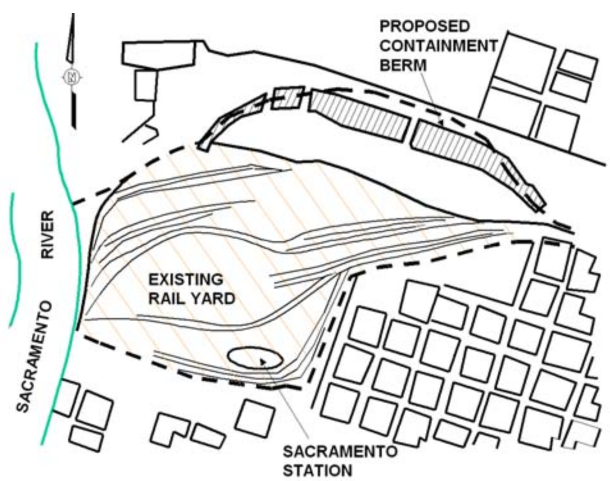

Fig. 7. Sacramento Rail Yard site plan showing existing track locations and proposed containment berm. The relocated rail lines would pass along the top of the berm.

A key aspect of the transportation center and one of the provisions of the Rail Yards Specific Plan was that the main line rail tracks which traversed the southern site boundary would be moved to the northern property boundary and placed on top of an earthen embankment. This track realignment would permit redevelopment of the central and southern portions of the site to proceed unimpeded by rail traffic. This embankment, referred to as the rail berm (or simply as the berm), would elevate the rail traffic to permit at-grade passage of vehicular traffic through the north portion of the property. The Rail Berm was also intended to act as a secondary flood protection levee for the downtown Sacramento Area. A plan of the site is shown in Fig. 7.

In addition to transportation support and flood protection, a portion of the berm would be used to provide secure on-site containment for metals-affected soil removed from the Rail Yard. The feasibility of the on-site metals-affected soil containment strategy was approved in concept by DTSC based on considerable site testing and analysis and was included in the approved Feasibility Study (FS) and Remediation Action Plans (RAP) for different areas of the Rail Yard.

Berm Contaminated Soil Acceptance Criteria

Material to be placed in the containment berm would consist of metals-affected soil excavated from the Rail Yard during remediation activities. Pursuant to the DTSC-approved RAP, soil acceptance criteria and limitations for the berm included:

- RCRA-regulated metals-affected soil must be managed as RCRA waste and could not be placed within the containment structure;
- With the exception of lead, copper, and zinc, metals-affected soils at concentrations exceeding the regulatory total threshold limit concentration (TTLC) must be managed as a California hazardous waste and could not be placed within the berm;
- With the exception of lead, metals-affected soils at concentrations exceeding the regulatory soluble threshold limit concentration (STLC) must be managed as California hazardous waste and could not be placed in the berm; and
- Lead-affected soils exceeding the STLC and/or TTLC and copper- and zinc-affected soils exceeding only the TTLC could be placed within the berm as long as the concentrations of organic constituents were protective of groundwater.

To demonstrate that excavated soil was suitable for placement within the landfill, a detailed soil sampling and analysis plan (SAP) was prepared for implementation during site remediation. The SAP defined the soil stockpile sampling frequency and the required analytical methods to verify that the soil met the criteria set forth in the approved RAP for the berm.

Berm Design Overview

The rail berm was to be approximately 5,570 feet long and was designed to accommodate from two to seven sets of rails, depending upon the location. The height of the top of the berm varied from approximately 2 to 30 feet above existing grade, the top width ranged from 40 to 230 feet; the base of the berm varied from 50 to 270 feet in width (as measured from the toe of the berm slope). The berm design incorporated a number of containment and environmental control systems, including (Figures 8 and 9):

- Composite Liner and Leak Detection Systems. The berm design included a primary composite liner that consisted of a synthetic geomembrane liner over a geosynthetic clay liner (GCL). A secondary geomembrane liner provided double containment for any areas of the berm located within five feet of the highest anticipated groundwater. In addition to primary and secondary containment, the berm included a leak detection system along its entire length.

- Leachate Collection and Removal System. Design analyses showed little to no leachate would be generated following construction of the berm. Nonetheless, in part to address regulatory concerns, a leachate collection and removal system was designed on top of the primary composite liner.

- Cover System. The cover system design for the berm included (from bottom to top): a foundation layer, a prepared subgrade soil layer with hydraulic conductivity of 1 x 10^{-5} cm/sec or less, a 60-mil HDPE geomembrane, and a drainage layer. Asphalt concrete and bituminous treated base and ballast were included in the top deck of the cover to support rail traffic.

- Retaining Walls. Retaining walls up to 30 feet high were located along the berm alignment where property and other real estate restrictions limited construction of the sloping sides for the berm. In addition to the retaining walls, rail bridge abutments were designed at street crossings to permit at-grade vehicular traffic to pass through the berm.

- Surface Water Drainage and Flood Protection. Surface water drainage control structures were designed for the 1,000 year, 24-hour storm and to prevent ponding, erosion, and run-on. Pursuant to City requirements, the containment berm was also designed to provide secondary flood protection for the downtown Sacramento area.

Subsurface Conditions

The Rail Yard is located on up to about 3,000 feet of relatively flat alluvial-deposited sediments within the Central Valley of California. Detritus from the Sierra Nevada range comprises the youngest sediments in the area of the Rail Yard and are thought to be of late Pleistocene age. Prior to development, the Rail Yard area was dominated by flood plain depositional processes that formed river and stream channels, terraces, swamps, levees, and over bank deposits.

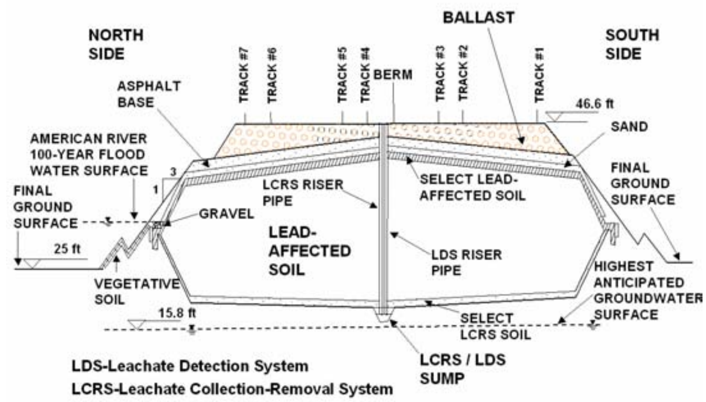

Fig. 8. Cross section of proposed containment berm and flood protection levee showing environmental control systems, material types, liners, and relocated rail lines.

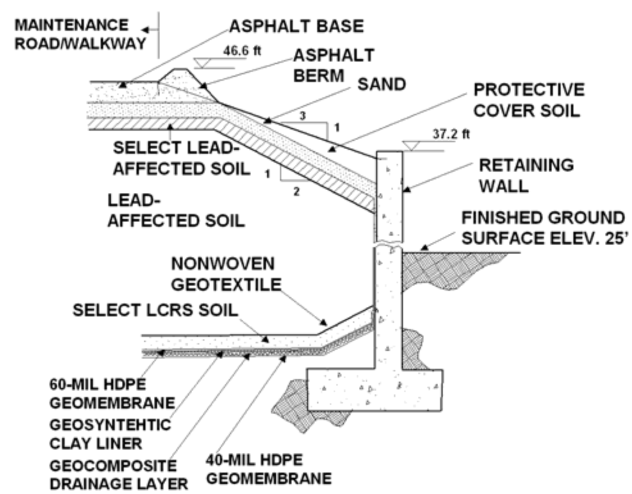

Fig. 9. Cross-section of waste containment berm showing retaining wall where embankment width is limited by site restrictions. Details of the liner and cover systems are indicated.

Subsurface conditions along the alignment of the berm are shown in Figure 10. Most of the berm alignment is located on fill soil reclaimed from the Sacramento and American Rivers. The generalized stratigraphy along the berm alignment includes a layer of fill, an underlying layer of silt and sand (known as the Upper Sand Zone), and deeper layers of gravel, sand, silt, and occasional clay. Depending on location, groundwater is encountered about 10 to 30 feet below the ground surface along the alignment.

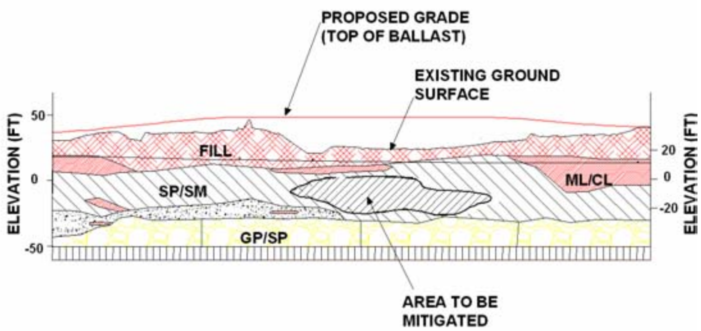

Fig. 10. Subsurface conditions at the site of the existing Sacramento Rail Yard showing zone of liquefiable soils to be mitigated.

Data from site borings indicate the fill ranges from a few feet to about 25 feet thick. In general, the fill consists of a heterogeneous mixture of borrow material and debris. The borrow material consists predominantly of sand and silt and is believed to be from dredging of sloughs along the Sacramento and American Rivers and various borrow pits in the Sierra foothills. Debris within the fill typically includes wood, metal, concrete rubble, construction and demolition wastes, bricks, drums, black staining, asbestos-containing material, and general mixed refuse.

The Upper Sand Zone underlying the fill consists mostly of silty sand, although a silt to clayey silt layer is locally present in the upper portion of the unit. Boring and CPT data indicate the lower portion of the Sand Zone is composed of primarily of silty sand, although apparently discontinuous layers and lenses of clean sand, silt, and sandy silt may be present throughout the unit. A zone of dense gravel underlies the Sand Zone at an elevation of approximately -30 ft MSL.

Seismic Setting and Seismic Response Analysis

The Sacramento Rail Yard is located within the Sierran block tectonic province, and this province is surrounded by zones of tectonic deformation on major and well-defined faults, including the San Andreas and Coastal Range systems to the west, the Big Pine, Garlock, and White Wolf faults to the south, and the Sierra Frontal faults to the east. The maximum potential site seismic ground motion likely to affect the Sacramento Rail Yard would be associated with the maximum credible earthquake (MCE) capable of being generated along one of these faults or fault zones. Evaluation of the seismogenic capability of these faults indicated a M_w 6 represented an appropriate and conservative MCE for design.

Peak horizontal ground accelerations (PHGA) and maximum cyclic shear stress profiles for the Rail Yard were determined based on a site-specific site response analysis that accounted for the design earthquake MCE magnitude, the peak horizontal acceleration in bedrock below the site, and the influence of site geologic characteristics (alluvium) on the design earthquake motions. The general procedure used to develop an acceleration profile for the site included:

- Evaluation of the peak acceleration resulting from the design MCE in bedrock. The peak horizontal acceleration was evaluated using the Abrahamson and Silva (1996, 1997) relationship for soft rock.

- Earthquake time histories representative of the design MCE were selected on the basis of earthquake magnitude and predominant style of faulting. A mean and mean plus one standard deviation target acceleration response spectra for the design event was then developed for the site using the Abrahamson and Silva (1996, 1997) model.

- Subsurface site characteristics were evaluated based on site boring, CPT, and laboratory test data to develop a representative subsurface geologic profile of the Rail Yard.

- The scaled suite of time histories and the site-specific geologic and geotechnical characteristics were used as input to the one-dimensional computer model SHAKE as bedrock outcrop motions. The "outcrop motion" option in SHAKE was used to deconvolve these motions to "bedrock at depth" motions. The deconvolved motion was input to the bottom of the site soil profile and propagated to the surface using an equivalent-linear 1-dimensional wave propagation analysis to develop profiles of cyclic shear stresses and PHGA. The SHAKE results indicated a shear stress profile that increased with depth and PHGAs at the ground surface that varied between 0.12g and 0.18g (Figure 11). Subsequent liquefaction and lateral spread evaluations were based on the maximum (highest and most conservative) shear stress or acceleration.

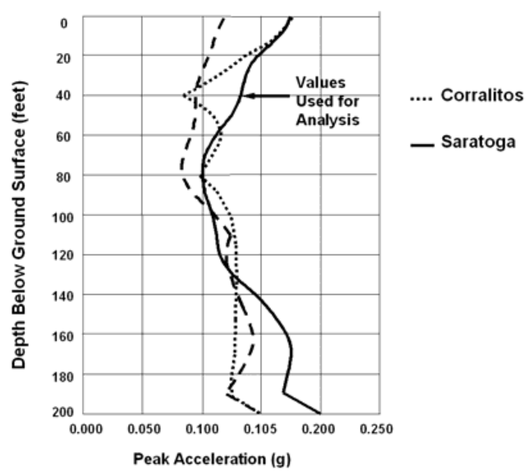

Fig. 11. Peak horizontal acceleration profiles for two input motions and the profile used for ground response and liquefaction analysis.

Liquefaction Evaluations

Liquefaction Triggering. Liquefaction triggering evaluations were performed for the containment berm area of the Rail Yard using the results of the seismic response analysis and the SPT and CPT data collected along the berm alignment. In general, SPT and CPT evaluation procedures followed protocols described in NCEER (1997). The results of these evaluations indicated a zone of liquefiable soil approximately 1,400 to 1,500 feet long and 10 to 30 feet thick near the center of the containment berm alignment.

Effects of Liquefaction. Analyses performed to assess potential effects of liquefaction included: seismically-induced settlement; ground surface liquefaction effects (sand boils); and lateral spreading. Potential seismically-induced settlements at the ground surface were estimated surface using the procedure described in Tokimatsu and Seed (1987) that relates volumetric strain to the level of ground shaking and the pre-earthquake SPT blowcount. Vertical settlements were estimated to be on the order of 1 to 3 inches, which was judged to be within design tolerances.

The potential for surface manifestations of sand boils and associated ground loss were evaluated using the methods of Ishihara (1984) to assess the amount of non-liquefiable overburden soil required to prevent sand boils. Based on this procedure, approximately 15 feet of non-liquefiable overburden soil was required to prevent sand boils at the Rail Yard.

Lateral spreading evaluations were based on the assumption that the soil underlying the berm liquefies, and as a result, post-liquefaction stability would depend on the residual shear strength of the foundation soil. The post-liquefaction (or residual) soil shear strength was estimated using the relationship in Seed and Harder (1990) between residual undrained shear strength (S_r) and the pre-liquefaction fines-corrected SPT blowcount (or $(N_1)_{60\_CS}$) for the area or zones of interest. Based on the field data and on the Seed and Harder (1990) relationship, two zones of liquefaction with differing residual shear strengths were identified, including:

- A zone with an average $(N_1)_{60\_CS}$ of approximately 8 and a resulting S_r of about 160 pounds per square foot (psf); and

- A relatively wider zone with an average $(N_1)_{60\_CS}$ of approximately 10 and a resulting S_r of about 230 psf.

The lateral spreading evaluation included assessment of the potential for unrestrained deformation (e.g. a flow slide) following liquefaction, and upper-bound estimates of seismically-induced deformation assuming unrestrained deformation does not occur.

The potential for unrestrained deformation was evaluated by calculation of the post-earthquake static factor of safety using residual shear strength in the zone of liquefaction (FS_{res}). If FS_{res} was greater than 1.0, then unrestrained lateral spreading was judged unlikely. If FS_{res} was less than 1.0, then there was a potential for large, unrestrained lateral spreading following liquefaction. For cases where FS_{res} was greater than 1.0, the upper limit on seismically-induced lateral deformation was calculated using two alternative approaches, including: (i) the Makdisi-Seed (1978) chart solution that relates deformation to the ratio of yield acceleration (k_y) to the maximum average acceleration for a potential sliding mass (k_{max}) for earthquakes of different magnitude; and (ii) a Newmark-type analysis to provide an estimate of permanent, seismically-induced deformation. For the analysis, two locomotives were assumed to be present on the top of the berm at the time MCE-level seismic activity occurred and liquefaction was assumed to be triggered.

The results showed that liquefaction of the subsurface soils below the containment berm potentially could result in static safety factors of 1.0 or less and large lateral deformation. Additionally, for those cases where the post-liquefaction static safety factor was greater than 1.0, calculated lateral deformations were marginally higher than the generally accepted value of 12 inches. As a result, a ground improvement program was recommended to mitigate liquefaction at the site.

Evaluation of Liquefaction Mitigation Methods**.** Ground improvement technologies that were identified and evaluated included dynamic compaction; blasting; vibro-compaction; vibro-replacement; compaction grouting; deep and shallow soil mixing; and geosynthetic reinforcement. The evaluation criteria were based primarily on feasibility with respect to meeting the target ground improvement requirements, considering limitations imposed by subsurface conditions, land use, and precedent. Based on these considerations, the most applicable treatment alternatives for the Rail Yard included:

- Blast densification of liquefiable soils under the containment berm to an elevation of about -10 ft MSL; and/or

- Either vibro-compaction or vibro-replacement methods along the northern and southern perimeter of the berm. To meet deformation requirements, the treated zones were estimated to be about 20 feet wide and extend to a depth about five feet above the dense sand and gravel layer.

Actual spacing, treatment depths, and amount of improvement attained would be determined through a test program prior to production ground improvement.

Stability Evaluations

Post-ground improvement static and seismic stability analyses were performed for the berm assuming slip surfaces that passed through the berm and the underlying subgrade soils;

slip surfaces that passed through the berm and along the base liner system; and slip surfaces that passed along the final cover system. Potential seismically induced deformations were estimated using a Newmark (1965) type analysis as modified by Makdisi and Seed (1977).

Material and interface properties were obtained using site-specific data or experience with similar materials on similar projects. For cases where material property data were not available, conservative values were assumed. Surcharge loads were used to model the impact of trains on berm stability, with the most severe loading condition assumed to be one locomotive on each of the seven tracks across the alignment (Figure 12).

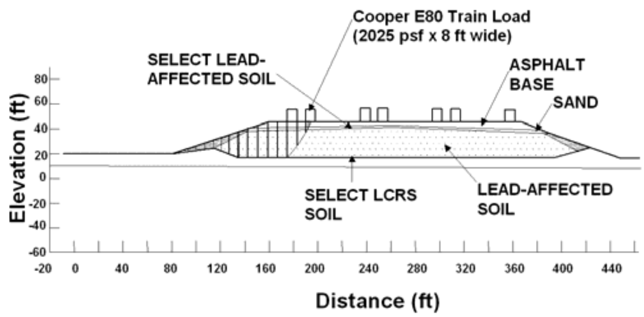

Fig. 12. Typical berm cross section used for post ground improvement evaluation of static and seismic stability.

Initial liner stability analyses assumed the base liner system and cover system met at the same elevation as the outboard toe of the berm. They indicated static safety factors that ranged from about 1.3 to about 1.5. Because some analyses resulted in a static safety factor less than 1.5, additional analyses were performed assuming an alternative configuration whereby the intersection of the top of the base liner system and the lower portion of the cover system was elevated about 5 feet above the outboard toe elevation of the berm. Static safety factors for this configuration were greater than 1.5 for all cases analyzed, and seismic analyses indicated deformations within acceptable limits. Analyses indicated acceptable static and seismic stability for slip surfaces that passed through the subgrade or through the final cover system.

Regulatory Acceptance of Analyses and Design

In addition to difficult siting, environmental, and geologic conditions, other challenging aspects of this project were developing appropriate, cost-effective remediation design concepts and then securing the necessary approvals from a number of city, county, and state agencies with regulatory or permit authority at the Rail Yard. Towards this end, input from the various agencies was actively solicited, agency personnel were viewed as essential project partners, and key project and regulatory participants were encouraged to communicate informally as often as necessary. In addition, project meetings were held on a monthly basis over a several year period to address investigation findings, proposed remediation concepts, the inevitable changes to these concepts, regulatory concerns, and to generally keep the project on schedule.

Through these meetings, it became apparent that many of the regulatory agencies and a number of other project participants did not fully understand the seismic characterization and liquefaction analyses that were performed to support design of the rail berm. As a result, there was some skepticism regarding the feasibility of safely constructing the berm in a metropolitan area. To address these concerns, the project sponsor committed to preparing and presenting a full day workshop to educate project participants and other regulatory agency personnel on seismic characterization methods, liquefaction evaluation procedures, and their specific application to the Rail Yard project. The objective of this well-attended workshop was to "demystify" the evaluation procedures, with a net effect being preliminary regulatory approval of the seismic evaluation and proposed design without the need for time-consuming and costly third-party review.

As an interesting sidelight, notwithstanding the significant costs associated with a planning, analysis, and design process that took a number of years, despite substantially completing final permit and construction documents for the berm, and despite securing preliminary regulatory approval for construction of the containment structure, the project sponsor opted to terminate the project in favor of contaminated soil excavation and off-site disposal. The principal reasons for the change were not costs, geotechnical issues, or regulatory or public acceptance. Rather, land use plans for the Rail Yard were modified to support sale of a portion of the property; consequently, and relocation of the tracks with an elevated rail corridor and its supporting berm were deemed to be no longer necessary.

Conclusions and Lessons Learned

The Sacramento Rail Yard project was to involve construction of a waste containment facility in the middle of a major metropolitan area. As a result, considerable effort was expended addressing the geotechnical issues governing design of the structure. Seismic safety issues had significant impacts on design, and considerable subsurface investigation, laboratory testing, and engineering analyses were completed to address them. Perhaps the most noteworthy geotechnical problem for this project was the presence of contaminated liquefiable subsurface soils along the alignment of the planned containment berm.

The rail berm was an integral, though by no means only, part of a large and complex environmental remediation and redevelopment project that required input and decisions from many ownership, management, planning, environmental, engineering, regulatory, and city personnel. Concerted efforts to foster and maintain communications between all project stakeholders were viewed as important towards managing project changes, costs, and limiting the potential for litigation and/or third party intervention. Similarly, extra effort to explain the fundamental approaches used to address what are perceived to be complex geotechnical issues paid dividends in securing approval of the containment portion of the project.

Ironically, despite approval and substantial completion of design, late changes in land use plans for the Rail Yard negated the need to move and elevate the rail corridor. As a result, the containment berm project was terminated in favor of contaminated soil excavation and offsite disposal. Nevertheless, the studies and analyses that have been completed provide useful guidance for development of similar sites in the future.

CONCLUSION

Conclusions and lessons specific to the geoenvironmental aspects of each of the two cases described in this paper were given earlier. To summarize, we learn from these experiences that:

- Developing and implementing a suitable plan for remediation of environmental projects may be lengthy, litigious, and expensive. Including and communicating with all project stakeholders early and throughout the project are important in reducing costs, maintaining schedules, and reducing the possibility of litigation.

- The extra effort required to explain what are thought to be complex geotechnical analysis procedures and analyses is frequently warranted and may help limit project costs and delays.

- Securing approval for innovative alternatives or alternatives without appreciable precedent may be difficult in a regulatory environment.

- Ultimate land use designations and/or changes in these designations may be more important than expended costs or previous approvals.

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