ENGINEERING AND DESIGN REPORT

B-18 CLASS I LANDFILL PHASE III EXPANSION AND FINAL CLOSURE

KETTLEMAN HILLS FACILITY
KETTLEMAN CITY, CALIFORNIA

Prepared for:
Chemical Waste Management, Inc.
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November 2008
Revision 1: February 2010
Revision 2: August 2011
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1.0 INTRODUCTION

1.1 Purpose of Report and Background Information

This report provides engineering data and analyses to support the Construction Drawings (Drawings), the Technical Specifications (Specifications), and the Construction Quality Assurance (CQA) Plans for Landfill Unit B-18 (B-18) at the Kettleman Hills Facility (KHF) in Kettleman City, Kings County, California. B-18 is located in the southeast portion of the KHF, as shown on Figure 1.1 and on the Site Location Map portion of Sheet T-1[1] in Appendix A.1.2

B-18 is an existing active Class I/II landfill that has been accepting waste continually since 1992. The existing B-18 landfill was constructed in two phases (Phases I and II), both of which were completed in the early 1990s. The design of Phases I and II of B-18 was completed by Environmental Solutions, Inc. (ESI) and was presented in the original Engineering and Design Report for B-18 (ESI, 1990a)[2].

Chemical Waste Management, Inc. (CWM), the owner and operator of B-18, wishes to construct Phase III to expand B-18 to provide additional waste capacity. Hence, this report has been prepared to supersede and serve as an updated revision to the original ESI (1990a) Engineering and Design Report for B-18. The updates contained in this report pertain primarily to the proposed Phase III expansion of B-18 and to the revised final closure design for B-18. The contents of the original ESI (1990a) Engineering and Design Report have been preserved herein - as appropriate - such that this report should be used as a stand-alone reference for the entirety of the Landfill B-18 engineering and design.

Reduced-size copies of the Drawings for Phases I through III and final closure are included in Appendix A. The Specifications and CQA Plan for Phases I and II of B-18 were prepared as a separate document by ESI (1990b). The Specifications and CQA Plans for Phase III and final closure are included in Appendices O and P, respectively.

1.2 Landfill B-18 Design

As described in Section 1.1, ESI (1990a) designed the existing Phases I and II of B-18 while Golder Associates Inc. (Golder) designed the proposed Phase III expansion and the revised final closure configuration. Golder's design of Phase III and the final closure of B-18 is based largely on the design of the existing B-18 Phases I and II completed by ESI (1990a).

The design of B-18 described in this report follows the master plan for the KHF, including the approved Kings County Conditional Use Permit (CUP) requirements. A Subsequent Environmental Impact Report (SEIR) has been prepared in accordance with the California Environmental Quality Act (CEQA). The SEIR is currently under review and certification of the SEIR will be required prior to construction of B-18 Phase III. The B-18 design generally follows procedures used for prior KHF

[1] The term "Sheet" refers to the specific page of the Drawings in Appendix A
[2] References are provided in Section 6

waste management units (WMU) for land disposal and complies with the following regulatory documents:

- United States Environmental Protection Agency (USEPA) Draft Minimum Technology Guidance on Double Liner Systems (USEPA, 1985)[3].
- USEPA Resource Conservation and Recovery Act (RCRA) Hazardous Waste Facility Permit, Part B (USEPA, 1990)[4].
- USEPA PCB regulations for Chemical Waste Landfills, Code of Federal Regulations, Title 40, Section 761.75.
- California Department of Toxic Substances Control (DTSC) Hazardous Waste Facility (Part B) Permit (DTSC, 2003).
- DTSC Environmental Health Standards for the Management of Hazardous Waste, Title 22, Division 4.5 of the California Code of Regulations (CCR).
- California Regional Water Quality Control Board (RWQCB), Central Valley Region, Waste Discharge Requirements No. 98-058 (RWQCB, 1998).
- California State Water Resources Control Board and RWQCBs, Discharges of Hazardous Waste to Land, Title 23, Division 3, Chapter 15 of the CCR.
- Kings County Conditional Use Permit (CUP) No. 1412, Administrative Approval Nos. 90-23 and 90-24 for the B-18 Landfill Phases I and II (Kings County, 1990).
- Kings County CUP No. 05-10 (application under review).

The primary differences between the B-18 design and the design of prior (i.e., pre-1990) KHF landfill units are the use of textured high density polyethylene (HDPE) geomembrane and the avoidance of operating waste slopes directly on the base liner system. These changes improve stability conditions throughout the operating period. These design concepts were initiated in Phase I and continue through Phase III and Closure.

Key aspects of the B-18 design are:

- The facility is developed in three phases (see Sheet 2 in Appendix A.1 and Sheet C-3 in Appendix A.2). Phase I is located on the west side of the existing landfill and was constructed in 1990 thru 1992 (ECS, 1992f). Phase II is located on the east side of the existing landfill and was constructed in 1992 thru 1993 (GCS, 1993h). Phase III will include a vertical expansion primarily over the western half (approximately) of the existing landfill as well as a lateral expansion up the existing rock cut slope along the west side of the landfill. Phase III is anticipated to be constructed in 2010/2011 and operational in 2010/2011.

[3] This reference is applicable only to Phases I and II.
[4] This reference is applicable only to Phases I and II.

- Phases I and II each have two independent sump areas for leachate collection, detection, and removal. No other sumps will be installed for Phase III since only sideslope liner systems will be constructed for this phase. The existing portions of each phase draining to the separate sumps are designated as Areas IA and IB for Phase I and Areas IIA and IIB for Phase II. The Phase III sideslope liner will drain to all four of the existing Areas (IA, IB, IIA, and IIB).

- The source of clay for the existing Phase I and Phase II liners was from an overburden claystone stratum (herein referred to as Stratum 18-8) that was excavated from the Phase II footprint. The primary source of clay for the Phase III liner is anticipated to be from the Landfill Unit B-17 excavation. The clay borrowing and preparation procedures that were used for Phases I and II as well as the procedures to be used for Phase III are described in Section 4.6.

1.3 Report Organization

This report is organized into the following sections that provide detailed descriptions and background information for the design of B-18:

- Section 2.0 – Site Description;
- Section 3.0 – Geotechnical Investigations;
- Section 4.0 – Landfill B-18 Description;
- Section 5.0 – Engineering Analyses; and
- Section 6.0 – References.

Supporting information on the B-18 engineering and design is provided in the following appendices to this report:

- Appendix A – Construction Drawings;
- Appendix B – Boring Logs;
- Appendix C – Trench and Test Pit Logs;
- Appendix D – Laboratory Data;
- Appendix E – Clay Liner Test Pad Data;
- Appendix F – Liner System Material Data;
- Appendix G – Settlement Analyses;
- Appendix H – Stability Analyses;
- Appendix I – Soil Erosion Analyses;

- Appendix J – Surface Water Drainage Analyses;
- Appendix K – LCRS Analyses;
- Appendix L – Riser Pipe Analyses;
- Appendix M – Cover Infiltration Analyses;
- Appendix N – Frost and Biotic Protection Evaluation;
- Appendix O – Technical Specifications; and
- Appendix P – CQA Plan.

2.0 SITE DESCRIPTION

2.1 General

This section describes the general location of B-18 as well as its pre-development and existing conditions. Sections 2.2 and 2.3 describe the current and pre-development B-18 site conditions, respectively. Sections 2.4 to 2.6 provide brief descriptions of the B-18 site's geologic, seismic, and hydrogeologic conditions, respectively.

2.2 Current Site Layout and Conditions

The KHF is located approximately midway between San Francisco and Los Angeles (see the Regional Location Map on Sheet T-1 in Appendix A.1) along the western edge of the San Joaquin Valley in central California. The KHF property consists of approximately 1,600 acres that occupies 2.5 Sections (1/2 of Section 33 and all of Section 34, R18E, T22S, and all of Section 3, R18E, T23S, Mount Diablo Base and Meridian). Landfill B-18 is located in the southeast portion of the KHF (see Figure 1.1) and currently has a footprint area of approximately 53 acres. The proposed final footprint area of B-18 will be approximately 68 acres.

Figure 1.1 shows the KHF in relation to the west side of the San Joaquin Valley floor. The KHF is located in the Kettleman Hills, approximately four miles from the valley's edge. The existing ground surface elevations (USGS Datum) at the KHF range from approximately 750 to 1,010 feet above mean sea level, making the KHF approximately 600 feet higher than the adjacent portion of the valley floor. The most recent (March 28, 2008) topographic survey of the KHF indicates that, as of March 28, 2008, the top deck waste elevations of B-18 range between 885 and 905 feet above mean sea level (see Sheet C-2 in Appendix A.2).

Access to the KHF is from State Route 41 and Interstate 5, located along the west side of the San Joaquin Valley as shown on Figure 1.1. The entrance to the KHF is approximately three miles west of Interstate 5 and 60 miles northeast of San Luis Obispo, California. Within the KHF, access to the B-18 area is through the existing Guard Station at the Main Gate, northwestward past Landfill Unit B-15, westward along the road that is south of Surface Impoundment P-9, and southward past Surface Impoundments P-10 and P-11 and past the Final Stabilization Unit (FSU), which is located immediately north of B-18 (see Sheet C-1 in Appendix A.2). Waste trucks currently enter at the northwest corner of the B-18 area; as waste elevations increase, the trucks will use the western access road and closure cover access road as shown on Sheet C-4 in Appendix A.2.

The layout of the existing Phases I and II of B-18 was based on the August 1990 CUP Facilities Boundary during the original design of B-18 (see Sheet 2 in Appendix A.1). Physical constraints for the Phases I and II areas included the following:

- The existing FSU facility to the north of Phase I.
- The existing KHF truck access road surface water control basin located along the northeast portion of Phase II.
- The requirement for a B-18 surface water containment basin (referred to as the Northeast Containment Basin herein) near the northeast corner of Phase II. This surface water basin was constructed as part of Phase II in 1992 thru 1993 and is shown on Sheet 2 in Appendix A.1.

The layout of Phase III of B-18 was developed based on the proposed modified CUP Facility Boundary the three above-mentioned physical constraints, and the following additional physical constraints:

- The existing Phases I and II geometry.
- The requirement for a second surface water containment basin (referred to as the South Containment Basin herein) to the south of B-18, as shown on Sheet C-3 in Appendix A.2.

The clean soil stockpile from the Phases I and II excavation is located outside of the immediate B-18 area as shown on Sheet C-1 in Appendix A.2. The "B-17 Borrow Area," within the boundary of Landfill B-17, to the northwest of B-18 is used for clay borrow and processing activities, but is primarily utilized as the source of daily and final cover soil.

No major KHF utilities are located within the B-18 area. Power for the existing light poles that surround B-18 and for the B-18 leachate control pumps is currently provided from an electrical transformer located along the north side of B-18 (see Sheet C-2 in Appendix A.2). For the Phase III construction, this electrical transformer will be removed and relocated to the north. The existing lighting system that surrounds B-18 is no longer required and will be removed during the Phase III construction.

2.3 Pre-Development Site Conditions

Figure 2.1 shows the B-18 site topography prior to the construction of B-18. This area was defined by a central, east/northeast-draining dry wash (i.e., swale) flanked on either side by several roughly northwest-trending ridge spurs. A former elongated, northeast-facing ridge slope formed the southwest boundary between Phases I and II and was used to develop these phases. In the Phase I area, two former tributary swales drained (northwest and southeast, respectively) along the toe of this slope into the former central swale. Another former swale drained northward through the south portion of the Phase II area, joining the former central swale near the northeast corner of the B-18 area. Typical relief between the former swales and adjacent ridge tops varied up to about 100 feet; however, the long ridge that currently borders B-18 on its southwest side rises over 250 feet above the lower portion of the former central swale. Former slopes in the B-18 footprint were gentle to moderate, ranging from nearly flat up to inclinations of about 3H:1V (horizontal:vertical). Locally steep (2H:1V) former slopes occurred on the east/northeast side of the former ridge spur in the south-central portion of B-18.

2.4 Geologic Conditions

Geologic conditions at the KHF are well-documented in the many studies completed for previous site activities. In general, subsurface conditions are relatively straightforward and consistent in comparison with other sites in California.

Figure 2.2 shows the general geologic conditions in the vicinity of the KHF, based on the work of Woodring, et al. (1940). The KHF is located along the southwest limb of North Dome, which is a broad northwest-trending anticline that forms the north portion of the Kettleman Hills. The bedrock in the vicinity of B-18 mainly consists of the stratigraphically lowest units of the Upper and Lower San Joaquin Formation, which are comprised of discrete beds of sandstone, siltstone, and claystone. The

prevailing strike of these beds in the KHF area is about N45°W, with dips ranging from 25° to 35° southwest.

Figure 2.3 summarizes the geologic conditions at the KHF based on data from a variety of prior investigations. The most important characteristic of the site geology with respect to the B-18 site investigation program (Section 3) was the continuity and uniformity of the bedrock strata. Of special importance was the thick claystone stratum which passes through the western portion of the B-18 Phase II area. This material served as the clay source for the Phases I and II liner. Additionally, for the Phase III area, the dip of the bedrock strata is to the southwest, representing the most favorable bedding orientation for stability of the excavation.

Neither Figure 2.2 nor 2.3 indicates the existence of faults within the KHF, which would disrupt the general bedrock strike and dip trends and/or the continuity of the individual sandstone, siltstone, and claystone strata. Two studies by Roger Foott and Associates (1990a and 1990b) concluded that there is no surface or recent (i.e., Holocene) faulting in the B-18 area. This conclusion was corroborated by geologic mapping of the completed landfill subgrades during the construction of Phases I and II (Golder, 1992; GCS, 1993b).

2.5 Design Ground Motions

2.5.1 General

CCR Titles 22 and 23 require Class I landfills to be designed and maintained to withstand the Maximum Credible Earthquake (MCE) event. Hence, the design ground motions used in the analyses of B-18 were based on the MCE event(s), as described in the following two sections.

2.5.2 Ground Motions Used in the Original Design

In the original design of B-18, ESI (1990a) used the MCE event and associated ground motion parameters that had been developed by Golder (1988). The Golder (1988) MCE event for the KHF corresponded to a moment magnitude (M_w) 7.0 earthquake occurring at a depth of 10 km below the site on the Ramp Thrust Kettleman Hills North Dome segment of the blind Ramp Thrust Faults. The deterministic peak horizontal ground acceleration (PHGA) associated with this MCE event was calculated to be 0.43g (Golder, 1988), where g is the acceleration due to gravity.

2.5.3 Ground Motions Used in the Current Design

Hushmand Associates, Inc. (HAI), under subcontract to Golder, updated the design ground motions for the KHF as part of the current B-18 design. Appendix H.5 contains HAI's slope stability report that explains the methods used to develop the updated design ground motions. HAI performed deterministic seismic hazard analyses to evaluate the MCE ground motions for the controlling near-field and far-field events using a variety of state-of-the-practice attenuation relationships. Based on their analyses, HAI has developed the following deterministic MCE ground motion parameters for the KHF that were used in the current design of B-18:

- *Near-Field Event:* The controlling near-field MCE event is considered to be a M_w 7 earthquake occurring 10 km from the site on the Ramp Thrust Kettleman Hills North Dome segment. The PHGA associated with this event was calculated to be 0.62g.

- *Far-Field Event:* The controlling far-field MCE event is considered to be a M_w 8 earthquake occurring 35 km from the site on the San Andreas Fault. The PHGA associated with this event was calculated to be 0.16g.

2.6 Hydrogeology

Groundwater conditions are extensively monitored at several existing monitoring wells located throughout the KHF site. Sheet C-2, C-3 and C-4 in Appendix A.2 shows the locations of monitoring wells in the vicinity of B-18. Recent data from these wells indicate that the depth to groundwater is about 250 feet below the bottom of the existing B-18 base liner system. No shallow perched groundwater or perennial springs are known to occur in the B-18 area.

Because groundwater conditions do not affect the design or construction of B-18, an extensive evaluation of hydrogeology is not provided in this report. A report by EMCON (1986) contains a detailed description of the hydrogeological conditions at the KHF. The current Groundwater Monitoring Plan for the KHF was prepared by Geosyntec (2001).

3.0 GEOTECHNICAL INVESTIGATIONS

3.1 General

This section describes the geotechnical field and laboratory investigations previously undertaken by ESI and others to characterize the B-18 subsurface conditions and to evaluate soil and rock properties necessary for the geotechnical design of B-18. No additional field or laboratory investigations were performed by Golder in the preparation of this report. Field and laboratory testing on the proposed clay source for Phase III of B-18 was performed by Geosyntec (2008), as discussed in Section 3.2.2.

Section 3.2 describes the main field investigation activities associated with the design of B-18 and the evaluation of on-site claystone for use in the B-18 liner construction. Section 3.3 summarizes the subsurface conditions for B-18 based on the results of the geotechnical investigations. Section 3.4 describes the procedures used to select the soil and rock samples for laboratory testing in order to evaluate the required geotechnical design parameters. Results of the laboratory tests, including prior KHF data not directly associated with the design of B-18, are discussed in Section 3.5.

3.2 Field Investigations

3.2.1 Geotechnical Exploration Program

The locations of geotechnical field exploration activities are shown in plan view on Figure 3.1 and in sectional views on Figure 3.2. The field explorations were undertaken in several phases designed to verify the anticipated site geologic characteristics and to obtain representative soil and rock samples.

The initial phase of B-18 field exploration activities was conducted from February 20 through 27, 1990, and consisted of the following:

- Excavating long dozer trenches DT-A to DT-F to observe the thickness of colluvium, identify stratum contacts, and measure the strike and dip of rock discontinuities/bedding. These dozer trenches were generally between about 3 and 10 feet deep.

- Excavating test pits TP-l through TP-21 to penetrate colluvium at the base of the dozer trenches and to observe soil and rock conditions throughout the B-18 area. These test pits were generally about 6 to 18 feet deep.

Several disturbed bulk samples of representative colluvium and rock materials were collected for general laboratory analyses during this initial program.

The second phase of B-18 field exploration activities was conducted from March 12 through 23, 1990, and consisted of the following:

- Drilling nine geotechnical borings (L18-A through L18-I) to further verify the depth of contacts between individual rock strata and to collect relatively undisturbed samples of the rock materials to be encountered during excavation and/or to be used as embankment borrow material. These borings were advanced to depths ranging between approximately 18.5 and 89 feet below ground surface.

- Excavating test pits TP-22 through TP-28 to confirm colluvium thicknesses and characteristics of bedrock strata at locations between the borings. These test pits were excavated to depths of approximately 5.5 to 13.5 feet.

Table 3.1 summarizes the purpose for and key information about each boring. The borings were drilled using a Pitcher Barrel rig so that core samples could be recovered at the various intervals shown in Table 3.1. A total of 65 Pitcher Barrel samples of bedrock and 6 drive samples of colluvium were collected from the initial borings (L18-A through L18-I).

Data from the two initial field exploration phases were used to develop the preliminary versions of the geologic cross sections shown in Figure 3.2. Existing monitoring well logs were also reviewed to confirm interpretations of rock strike and dip. These data consistently verified the relatively uniform site conditions and indicated that site characterization for the purposes of the B-18 design was complete. However, it was concluded that additional clay samples were required to complete the characterization of Stratum 18-8 for use as the onsite clay source for the Phases I and II liner.

Therefore, the third and final phase of B-18 field exploration activities was conducted from May 8 through 11, 1990, and consisted of the following:

- Drilling boring L18-J to penetrate the entire Stratum 18-8 (clay borrow source) thickness to confirm uniformity of the claystone throughout this stratum. Sampling was accomplished by collecting approximately 2.5 feet of relatively undisturbed sample for each 5 feet of penetration. This boring was drilled to a depth of approximately 172.5 feet below ground surface.

- Drilling boring L18-K through the entire thickness of Stratum 18-9 and into the underlying Stratum 18-8. Stratum 18-9 was excavated concurrently with the Stratum 18-8 claystone during the construction of Phases I and II. This boring was also sampled by collecting approximately 2.5 feet of relatively undisturbed sample for each 5-foot penetration interval. This boring was drilled to a depth of approximately 97.5 feet below ground surface.

- Excavating test pits TP-29 through TP-43 to obtain larger bag samples of materials from Strata 18-8, 18-9, 18-10, 18-11, 18-12, and 18-13. These larger samples were used to evaluate properties of compacted borrow materials that could be mixed with the clay. These test pits were excavated to depths of approximately 3.5 to 16 feet.

Appendix B contains the logs of the above-described borings. Logs of the dozer trenches and test pits are included in Appendix C.

The information contained in Table 3.2 demonstrates that the colluvium and rock strata of interest to the design of B-18 have been adequately investigated by the above-described field exploration activities. Table 3.2 also includes the existing and previous monitoring wells which pass/passed through each geologic stratum underlying B-18.

Supplemental geologic field investigations were undertaken by Roger Foott and Associates (1990a and 1990b) to evaluate the potential for recent faulting in the vicinity of B-18. The findings of these investigations are discussed in Section 3.3.2.

3.2.2 Clay Liner Test Pads

In 1991, a clay liner test pad was constructed to evaluate the B-18 Phases I and II clay borrow source (i.e., Stratum 18-8). A sealed double-ring infiltrometer (SDRI) test was conducted on this test pad. Results of the SDRI test confirmed that the Stratum 18-8 clay source met the permeability requirements under actual field conditions. The construction of the 1991 clay liner test pad and the SDRI testing are discussed in detail in the test fill and infiltrometer report by ESI (1992), which is provided in Appendix E.1. CQA testing of the clay liner material during construction of Phases I and II of B-18 (ECS, 1992a, 1992b, and 1992d; GCS, 1993a, 1993d, and 1993f) verified the results of the SDRI test and indicated that the as-built clay liners for Phases I and II have permeabilities that do not exceed the specified maximum of 1×10^{-7} cm/s.

The clay source for the Phase III liner is anticipated to be the on-site Pecten Claystone stratum that lies along the eastern boundary of Landfill Unit B-17 (see Sheet C-1 in Appendix A.2). Geosyntec (2008) performed laboratory testing on samples of this stratum of Pecten Claystone. Results of Geosyntec's laboratory tests indicate that the Pecten Claystone stratum is a suitable clay borrow source for the Phase III liner. The Geosyntec (2008) report on the Pecten Claystone testing is summarized in Table 3.13 and presented in Appendix E.2. A clay liner test pad consisting of Pecten Claystone was constructed in July 2008. A SDRI test was conducted by Geosyntec (2008a) on this test pad to further evaluate the Pecten Claystone and validate its use as the clay borrow source for the Phase III liner. The SDRI test report was completed in December 2008 and is presented in Appendix E.3. The report concludes the Pecten clay is suitable for Phase III clay liner.

3.3 Site Subsurface Conditions

3.3.1 Surficial Soils

The majority of the B-18 site was blanketed with colluvial soils prior to its development. These deposits consisted of low to moderately plastic, silty and/or sandy clays and very fine-grained clayey sands. The colluvium was generally stiff to very stiff and dry to slightly damp when encountered during the field explorations. Occasional laminated lenses of fine-grained sand, probably representative of intermittent alluvial deposits within the colluvium, were encountered in some swale areas. The colluvium varied in thickness from less than 1 foot along the uppermost ridge slopes to over 18 feet within the swales, as shown in Figure 3.2. Considerable variation in the thickness of colluvium beneath uniform slopes was indicative of differential weathering of the underlying San Joaquin Formation bedrock and surficial soil compaction.

During the Phases I and II construction, the colluvium was excavated from the vast majority of foundation areas within the B-18 footprint due to its shallow depths. The only areas where colluvium was left in place were at the crest of the 2H:1V slope along the western boundary of Phase I. The colluvium that remained in these areas was less than 5 feet thick (GCS, 1993h). Areas where colluvium remained above landfill cut slopes outside of the waste footprint (e.g., the steep northeast facing slope above the southwestern edge of B-18) were graded to control soil erosion and/or sloughing.

Prior to the development of B-18, minor portions of the B-18 footprint were covered with fill. The northern portion of the Phase I area included the toe of a fill slope associated with the FSU construction. Several small fills in the Phase II area were apparently associated with drilling pads and a former access road that traversed the area. These fill materials were removed during the Phases I and II excavations.

3.3.2 Bedrock Lithology, Structure, and Stratigraphy

The San Joaquin Formation underlying B-18 is similar to the other portions of this formation found throughout the KHF area. The San Joaquin Formation consists of three major lithologic units: sandstone, siltstone, and claystone. Principal variants include silty sandstone and sandy siltstone. These units of the San Joaquin Formation were relatively easy to excavate using conventional earthmoving equipment (e.g., scrapers and dozers) during the Phases I and II construction. The physical characteristics of each of the discrete lithologic units, as well as qualities common to the overall San Joaquin Formation, are:

- *Sandstone:* Beds of both clean and silty sandstone occur within the B-18 area. The sandstones are variably white, gray, tan, and orange-brown. They are typically slightly weathered, soft, friable, very fine- to fine-grained, thick-bedded, and uncemented to weakly cemented. Occasional thin beds are moderately- to well-cemented. Numerous veins of gypsum and thin, orange interbeds of hard, cemented, iron-rich material occur along bedding planes and joints within the sandstones, as well as within the other lithologic units. The sandstone excavations generated fine, loose, clean, and silty sand. Several fossiliferous, well-cemented sandstone beds (including "Trachycardium" and "Mya") occur at various stratigraphic positions within the B-18 area, as shown on Figures 2.3, 3.1, and 3.2. These fossil beds are generally excavated as hard, gravel- to boulder-sized blocks.

- *Siltstone:* The siltstone units within the B-18 area are of variable character and include siltstone, sandy siltstone, and occasionally clayey siltstone. Each siltstone type is typically slightly weathered, soft, and laminated to thin-bedded. The siltstones vary from non- to low-plastic materials. Atterberg limits of selected samples, visually classified as siltstone, indicate that some "siltstones" consist of silty clays that plot just above the A-line on the plasticity chart. The siltstones are usually light brown or gray. Excavation of siltstone generates thin, angular fragments or slabs ranging from about 1/2 inch to 1 foot in largest dimension.

- *Claystone:* The claystone is usually either light gray, gray-brown, or dark olive-gray. It is typically slightly weathered, soft, laminated to thin-bedded, highly plastic, and frequently exhibits pronounced slickensides (striations) along glossy or waxy-appearing fracture and bedding surfaces. Dozer excavation of the claystone opposite the direction of dip yielded generally uniform, angular gravel-sized fragments, while excavation in the direction of dip yielded gravel- and larger-sized blocks and slabs in the range of 12 to 18 inches in maximum size. The largest slabs of claystone generated during the initial excavation tended to break down after repeated passes with the dozer.

The prevailing structural characteristic of the bedrock is its consistent bedding, which trends generally N30°W to N50°W and dips about 24 to 45°SW throughout the B-18 area (Golder, 1992; GCS, 1993b). Most measured joints dip steeper than the bedding and trend both across and generally parallel to the bedding (bedding and joint attitudes are shown on Figure 3.2). As shown on the cross sections in Figure 3.2, the cut slopes required for the Phases I and II construction were excavated shallower than the bedding plane angles to prevent adverse daylight conditions (e.g., along generally southwest-facing cut slopes).

Anomalous and contorted bedding (measured at N55°E, 73°SE) was encountered in dozer trench DT-C (Figure 3.1) within the thick Stratum 18-8 claystone. The length of the dozer trench characterized by this feature was logged as trench T-3 (see Appendix C). A detailed examination of the geologic units along this portion of DT-C suggest that the anomalous bedding was a result of old, intraformational deformation (e.g., localized folding, faulting, or slumping). The colluvium that overlaid this feature appeared undisturbed and displayed no evidence of offset or displacement that would be indicative of recent slope instability or faulting. Excavations for the construction of Phases I and II confirmed that the contorted bedding observed in DT-C was a localized, anomalous feature.

Contorted beds of cemented sandstone, appearing to be folded or compressed in a down-dip direction, were encountered immediately beneath colluvial soils in dozer trench DT-B (Figure 3.1). The contorted bedding is apparently related to settlement of the near-surface, cemented bedrock following erosion or animal burrowing of underlying, softer, uncemented sandstone (refer to the log of trench T-1 in Appendix C). The disturbed, near-surface bedrock was removed during the excavations for Phases I and II.

Stratigraphically, the San Joaquin Formation beds underlying B-18 have been grouped into a sequence of 13 stratigraphic units. Each unit is defined according to either a discrete lithology or a distinctive interbedding of various lithologic units. The units are designated 18-1 through 18-13, from northeast (oldest) units to southwest (youngest) units. The approximate contacts of these units are depicted in plan on Figure 3.1 and in profile on Figure 3.2. The stratigraphy shown in Figures 3.1 and 3.2 was based on the results of the dozer trenching, exploration drilling, air photo analysis, mapping of cut exposures prior to the development of B-18, and excavation of test pits. Comparisons were also made with logs of existing and previous monitoring wells in the B-18 area. In general, the geologic conditions in the B-18 area were found to be straightforward and consistent. Geologic mapping of the B-18 subgrades performed during the construction of Phases I and II revealed 15 units within the Phase I area (Golder, 1992) and 26 units within the Phase II area (GCS, 1993b). The results of this mapping confirmed the general geologic conditions shown on Figures 3.1 and 3.2 with the only significant discrepancies being local adjustments of some of the contact locations.

Geologic field investigations undertaken by Roger Foott and Associates (1990a and 1990b) indicated that recent (i.e., Holocene) faulting has not occurred in the vicinity of B-18. Additionally, geologic mapping of the completed B-18 subgrades during the construction of Phases I and II did not reveal any evidence of recent faulting (Golder, 1992; GCS, 1993b).

3.4 Laboratory Investigations

3.4.1 Sample and Testing Selection Process

This section describes the approach for selecting appropriate geotechnical laboratory tests to evaluate the necessary geotechnical design parameters and to establish geochemical background data for the on-site materials.

Initially, the large amount of existing geotechnical data available from pre-1990 KHF landfill designs was evaluated to assess the usefulness of this data for the B-18 design. Also, index property tests were initially conducted on many of the B-18 samples for comparison with properties of materials previously tested and to characterize the range of material types which may be important for design. Representative samples for strength, consolidation, compaction, shrink/swell, permeability, and geochemical testing were selected based on the results of index property tests, past data, and the importance of specific strata to the design analyses.

3.4.2 Prior (Pre-1990) Geotechnical Data

The most pertinent pre-1990 KHF geotechnical data used in the design of B-18 is summarized in Table 3.3 and is based on the results of investigations reported for the following activities:

- The design of Phases II and III of Landfill Unit B-19 (Donohue and Associates, 1988a);
- The slope failure investigation for Phase IA of Landfill Unit B-19 (Seed et al., 1988); and
- Generic investigations of closure alternatives for various landfills at the KHF (Golder, 1988b, 1989c, and 1989a).

Table 3.3 also summarizes reported properties from published literature and vendor data on geosynthetic liner interface testing.

3.4.3 Laboratory Testing for Landfill Design

Tables 3.4 and 3.5 summarize the B-18 geotechnical laboratory testing program and show the samples collected from the borings and test pits/trenches, respectively. An "X" is provided in each table to indicate the types of tests conducted on each sample. The analysis methods used for the various tests are summarized in Table 3.6.

A large number of index tests were initially conducted to evaluate the consistency of characteristics for the various rock and soil types. The index tests performed on a particular sample were selected based on the type of material. For example, plasticity index testing was conducted only on fine-grained claystone or siltstone samples. Index property comparisons were then used to select representative samples to be tested for the various engineering properties (e.g., compaction, strength, settlement, permeability).

Tests that were conducted under conditions to simulate the B-18 site-specific conditions are indicated by the footnotes in Tables 3.4 and 3.5. These included:

- Conducting unconsolidated undrained (UU) triaxial tests at confining pressures ranging between 4 and 16 kips per square foot (ksf) to represent the anticipated range of overburden pressures due to the weight of the overlying waste.
- Conducting most of the clay permeability tests at a dry unit weight equal to 90 percent of the Modified Proctor (ASTM D1557) maximum dry density and at water contents ranging from 2 to 6 percent above the optimum moisture content to represent clay liner material that has been compacted in accordance with the Specifications.

All of the compaction tests except two were conducted using the Modified Proctor procedures (ASTM D1557), which are specified for the B-18 construction. Standard Proctor procedures (ASTM D698) were conducted on one sandstone sample from Boring L18-K and one claystone sample from test pit TP-37 for comparison purposes only.

Table 3.4 also shows that background geochemistry was analyzed for three rock samples (one each from borings L18-A, L18-C, and L18-F) representing a range of the claystone, siltstone, and sandstone. The geochemical analyses conducted are listed in Table 3.6. These are the same background analyses conducted for prior KHF landfill investigations.

3.4.4 Special Testing of the Phases I and II Clay Borrow Material

Plasticity and unit weight/water content tests were conducted on 15 samples of Stratum 18-8 claystone collected from Boring L18-J to assess the uniformity of the claystone. Hydrometer tests were then conducted on five of the 15 samples that were considered representative of the range of conditions in the stratum to compare grain size characteristics. Shrinkage tests were conducted on four of the five samples to quantify the clay shrink/swell characteristics. Plasticity index and hydrometer tests were also conducted on shallow claystone samples from dozer trenches DT-A and DT-C and from test pits TP-36, TP-38, and TP-40 to provide a comparison of conditions derived from the shallow weathered rock.

Modified Proctor (ASTM D1557) compaction tests were conducted on three “pure” claystone composites of borehole samples to determine their water content-dry density relationship. Permeability tests were also performed on each of these three Modified Proctor composite samples using material compacted at approximately 90 percent relative compaction and water contents ranging from 0 to 2 percent above optimum. Unconsolidated-undrained (UU) triaxial and consolidation tests were conducted on one Modified Proctor test sample from dozer trench DT-A that was considered to be representative of the clay material.

The above-described tests on “pure” claystone samples provided conservative characteristics of strength and consolidation parameters for the clay materials. In order to assess the potential for mixing the claystone with other rock materials, an additional series of tests was conducted on mixtures of claystone from Stratum 18-8 and sandstone/siltstone from the adjacent Stratum 18-9. Mix ratios of 70:30 and 50:50 (claystone:sandstone/siltstone) percent were used.

Long-term leachate compatibility testing for the B-18 clay was not conducted in light of the results of an extensive testing program by EMCON (1989) using on-site claystone materials. That program included soil/waste compatibility tests performed consistent with the California Administrative Code, Title 23, Chapter 3, Subchapter 15, Section 2541(b) and (c); the United States Environmental Protection Agency (USEPA) regulations in 40 CFR 270.17(b)(1), 270.21(b)(1), 264.22l(a)(1), and 264.301(a)(1)(i); and the Resource Conservation and Recovery Act (RCRA) Method 9100.

The EMCON (1989) compatibility tests showed no significant increase in clay permeability after displacing two volumes of pore water with a representative leachate obtained from another hazardous waste site operated by CWM EMCON (1989) therefore concluded that the leachate did not have a significant effect on the permeability of the clay. EMCON (1989) also considered this conclusion to be consistent with findings reported by others in published literature, which indicate that dilute organic liquids do not adversely affect the permeability of clay soils.

3.5 Laboratory Testing Results

3.5.1 General

This section summarizes the B-18-specific laboratory testing results in the following order:

1. Index properties.
2. Compaction tests.
3. Strength tests.
4. Permeability tests.
5. Consolidation tests.
6. Shrink/swell potential tests.
7. Geochemical analyses.

The complete laboratory test results and supporting information are presented in Appendices D and E.

The laboratory test results described in this section were performed prior to the development of B-18 and provided the necessary information that guided the B-18 design. CQA reports prepared for the Phases I and II construction (see Section 4.1) contain additional laboratory and field test results that were performed as part of the Phases I and II CQA program. These CQA test results generally confirmed that the actual properties of the various as-built materials met or exceeded the material properties that were assumed during design. Hence, no attempt has been made to fully incorporate the CQA test data into the discussions in this report. Rather, CQA test results are only mentioned herein when deemed appropriate to reinforce an earlier assumption or finding. More recent testing, Geosyntec 2008a and b, has been conducted on proposed clay liner materials. Test results indicate the clay liner is similar to that used for Phase I and II and it will be suitable for use as a clay liner in Phase III. Test results that are included in Appendix E.2 and E.3 are summarized herein.

3.5.2 Index Property Tests

Plasticity index tests (i.e., Atterberg limits tests) were performed on claystone samples from Strata 18-2 through 18-5, 18-7, 18-8 (the clay borrow source for Phases I and II), 18-9, 18-10, and 18-12. Table 3.7 summarizes the plasticity index data and indicates that the majority of the claystone is classified as high-plasticity clay (CH) having a liquid limit ranging from about 55 to 90 and a plasticity index ranging from about 30 to 60. One sample from Stratum 18-8 and several samples from other fine-grained strata were classified as low-plasticity clay (CL). These materials have a liquid limit ranging from about 30 to 49 and a plasticity index ranging from about 6 to 29. In addition, one of the plasticity index tests performed on a minor claystone/siltstone sample of Stratum 18-3 showed the characteristics of a low plasticity silt (ML).

Figure 3.3 shows a plasticity chart with plotted data points for the majority of the Stratum 18-8 samples that were tested. It can be seen from this figure that the Stratum 18-8 claystone material consistently lies in the CH (i.e., high-plasticity clay) range. Additional plasticity charts containing plotted data for the other strata that were tested are included in Appendix D.1.

Table 3.8 summarizes the tests performed to evaluate the percentage of material passing the U.S. No. 200 sieve (i.e., fine-grained silt and clay) for the various samples tested. The colluvial soil samples generally have a fairly high percentage (about 33 to 81 percent) of fine-grained materials. This variation apparently relates to the origin of the colluvial materials with the highest percentage of fines being derived from siltstone or claystone. The data in Table 3.8 for sandstone shows a relatively low

percentage of fines that ranges from about 11 to 37 percent. Clayey and silty sandstone samples showed the largest variation of percent fines (from about 21 to 73 percent) which reflects the varying amount of fine-grained laminations in these samples.

Figure 3.4 presents the grain size envelopes obtained from sieve analyses on sandstone samples from Stratum 18-9 and hydrometer tests on claystones from Stratum 18-8. Individual test results for samples from these strata and other rock units are included in Appendix D.2. These results further show the relative uniformity of the various rock types at B-18. The sandstones have relatively uniform grain sizes that fall mostly in the 4 to 0.1 millimeter diameter range. The percentage fines in the sandstones is approximately 10 to 40 percent. The claystone is well-graded with at least 80 percent fines and about 5 to 30 percent of the particles being smaller than 0.001 millimeters. The clay-size fraction (particles with a diameter less than 0.002 millimeters) varies between about 12 and 40 percent.

Natural moisture contents and dry densities for samples tested are presented on the boring logs in Appendix B. Typically, the sandstone materials have a natural moisture content varying between about 8 and 20 percent and a natural dry density varying between about 95 and 121 pounds per cubic foot (pcf). The claystone material's natural moisture content typically varies between about 15 and 30 percent and its dry density range is approximately 90 to 105 pcf.

Figure 3.5 contains a plot showing the relationship of the natural water content to the Atterberg limits for claystone samples from Stratum 18-8. This relationship is useful for qualitatively evaluating the compressibility and strength behavior of the claystone. The data in Figure 3.5 show that the natural water content of the Stratum 18-8 claystone is typically less than or roughly equal to its plastic limit. This condition is indicative of a material with relatively low compressibility and high strength. The importance of this condition is that it allowed the settlements of the B-18 foundation to be calculated based on the theory of elasticity. Other interesting features that can be seen from Figure 3.5 are the following:

- The natural water content of the Stratum 18-8 claystone is relatively close to its optimum moisture content as determined from Modified Proctor compaction tests (see Section 3.5.3); and

- The Stratum 18-8 claystone plasticity characteristics are relatively uniform throughout its entire depth, although a lower-plasticity zone was encountered in the 70- to 90-foot depth range.

- Recent testing on the proposed clay liner material, summarized in Table 3.13, indicates the plasticity index data of the claystone is generally classified as high-plasticity clay (CH) having a liquid limit ranging from about 58 to 105 and a plasticity index ranging from about 29 to 72. The fines content of the claystone ranged from 76 percent to nearly 100 percent. This is consistent with clay liner materials used for Phases I and II.

3.5.3 Compaction Tests

Table 3.9 summarizes the results of compaction tests conducted on a variety of composited samples and on individual bag samples from test pit TP-42 and dozer trenches DT-A and DT-C. With the exception of composite Samples No. 4 and No. 11, all of the tests were performed using the Modified Proctor test method (ASTM D1557), which is the method that was specified for the Phases I and II

construction (ESI, 1990b) and is specified for the Phase III and final closure construction (see Appendix O). The Standard Proctor test method (ASTM D698) was utilized for Samples No. 4 and No. 11 in order to assess the differences in densities resulting from the use of a lower compactive energy (the Modified Proctor method utilizes an energy of 56,000 foot-pounds per cubic foot as compared to an energy of only 12,400 foot-pounds per cubic foot for the Standard Proctor method). Individual plots for the Modified and Standard Proctor tests are provided in Appendices D.3 and D.4, respectively.

The Modified Proctor compaction data indicate that the optimum water content for the claystone is on the order of 21 to 25 percent and the corresponding maximum dry density is approximately 96 to 104 pcf. As the percentage of sandstone increases, the optimum moisture content is expected to decrease and the maximum dry density to increase. The recent testing by Geosyntec, presented in Appendix E.2, indicates lower optimum moisture contents and higher maximum dry density than previous testing. The optimum moisture content and maximum dry density ranged from 12.5 to 20.1 percent and 105.5 to 122.0 pcf, respectively. These variations are not significant and do not necessarily indicate a change in the clay quality.

The compaction tests using the Standard Proctor method indicate that the claystone's optimum water content for this lower compactive energy increases to about 30 percent while the maximum dry density decreases to below 90 pcf. A similar amount of change was also observed for the mixture of sandstone and claystone tested.

3.5.4 Strength Tests on Relatively Undisturbed Samples

Strength properties of the in-situ rock materials that form the sidewalls of the majority of B-18 were evaluated by the following two types of tests:

1. Unconsolidated–undrained (UU) triaxial compression tests performed on relatively undisturbed samples of silty sandstone and claystone as summarized in Appendix D.5. These tests allowed failure to occur on the weakest plane in the sample and provided representative data for evaluating the stability of slopes in which failure along bedding planes may occur.

2. Direct shear tests performed on relatively undisturbed samples of sandstone and claystone, as summarized in Figure 3.6 and Appendix D.8. The direct shear samples were oriented such that failure occurred across bedding planes. These test results were then used for the stability evaluation of slopes which are not parallel or nearly parallel to the bedding.

Figure 3.6 also includes direct shear test data from the previous B-19 Phases II and III investigation (Golder, 1988b) for comparison.

The in-situ rock strengths along bedding planes obtained from the UU triaxial tests were consistently higher than the minimum strengths obtained for similar conditions for the design of B-19 Phases II and III (Golder, 1988b). Therefore, in order to be conservative for B-18 cut slopes in the west-facing direction, it was concluded that the appropriate rock strength along bedding planes should be represented by a friction angle (ϕ) = 36 degrees and a cohesion intercept (c) = 0, as recommended by Golder (1988b).

As shown on Figure 3.6, the shear strength parameters for evaluating slope stability for crossbed conditions was evaluated to be ϕ = 40 degrees and c = 800 pounds per square foot (psf). As can be seen in Figure 3.6, these strength parameters provide an approximately lower-bound limit of the direct shear test data conducted for the B-18 and B-19 (Golder, 1988b) investigations.

3.5.5 Strength Tests on Remolded Samples

The following two types of tests were performed to evaluate the strength of the clay liner for use in assessing the stability of B-18 at different times throughout its life:

1. UU triaxial compression tests were conducted on remolded sandstone and claystone samples to provide strength parameters to assess landfill stability for short-term conditions (i.e., prior to significant clay liner consolidation occurring due to the weight of the overlying waste).

2. Consolidated–undrained (CU) triaxial compression tests were conducted on remolded claystone samples for use in evaluating the long-term stability of B-18 (i.e., after the clay liner consolidation is essentially complete).

The results of the UU and CU triaxial tests are included in Appendices D.6 and D.7, respectively.

The UU triaxial test results indicate that the short-term strength of the clay liner can be represented by ϕ = 8 degrees and c = 3,600 psf. After consolidation is essentially complete, the clay liner is significantly stronger and can be represented by ϕ = 15 degrees and c = 1,500 psf.

CU triaxial compression tests conducted on silty sandstone materials from Stratum 18-9 indicate that the shear strength of these materials when compacted to 95 percent relative compaction can be represented by ϕ = 30 degrees and c = 3,000 psf. These strength parameters are considered appropriate for evaluating the stability of structural fill and embankments constructed from low plasticity borrow materials.

3.5.6 Permeability Tests

The five permeability tests summarized in Table 3.10 were conducted on clay samples derived from the Stratum 18-8 claystone. These tests show that the anticipated permeability under laboratory conditions varies between about 2×10^{-8} and 2×10^{-9} cm/s. For comparison, the field SDRI test performed by ESI (1992) indicated that the permeability of a clay liner constructed of Stratum 18-8 claystone is on the order of approximately 5×10^{-8} cm/s.

The laboratory permeability tests were conducted under a variety of conditions to evaluate the degree to which the particle size and weathering of the claystone may affect its permeability. The first two tests in Table 3.10 were conducted using a maximum particle size of 3/8-inches in the Proctor mold. Although small with respect to field compaction equipment, the 3/8-inch particle size is relatively large for small-scale laboratory permeability tests. The second two tests in Table 3.10 were conducted using a 1/4-inch maximum particle size, which corresponds to the ASTM procedures. The final test in Table 3.10 was designed to simulate the field conditions anticipated for B-18. This test was conducted by allowing the material to weather over a two-week period and without controlling the particle size. This procedure best represents the conditions which are realized in the field as the clay borrow material is mixed, worked, stockpiled, and recovered with wetting operations at various times

during these activities. Experience from the Phases I and II construction indicates that adequate permeabilities are realized if a maximum particle size of 1 to 2 inches is maintained.

Recent permeability tests, Geosyntec 2008, indicate the proposed clay liner material has a permeability of less than 1 x 10^{-7} cm/sec. The tests show that the anticipated permeability under laboratory conditions varies between about 9 x 10^{-8} and 4 x 10^{-9} cm/s, and 4.2 x 10^{-8} cm/s based on the field SDRI (see Appendix E.2 and E.3).

3.5.7 Consolidation Tests

Consolidation tests were conducted on two samples of the Stratum 18-8 clay that were compacted to conditions similar to those specified for construction. These tests provided information for:

- Estimating the amount of settlement that will occur in the clay liners as a result of the waste loading; and
- Estimating the rate at which pore pressures will dissipate from the clay liner in order to evaluate if there is a potential for excess pore pressure build-up.

The two consolidation tests showed similar compressive stress versus void ratio relationships. The results of the consolidation tests are included in Appendix D.9.

3.5.8 Shrink/Swell Potential Tests

Tables 3.11 and 3.12 summarize the shrink/swell test results for relatively undisturbed and remolded Stratum 18-8 claystone samples, respectively. The test results for the relatively undisturbed samples in Table 3.11 indicate that the in-situ claystone has low to moderate swell potential under low confining pressures. At high confining pressures, such as those on the base liner system, the swelling potential of the claystone is considered negligible based on the test results in Table 3.11.

The data in Table 3.12 shows that remolded Stratum 18-8 clay has a moderate to high swelling potential under low confining pressures. This indicates that it is important to keep the clay liner materials wet after placement and prior to deployment of the overlying geosynthetics in order to prevent significant desiccation cracking. Appropriate steps were taken to prevent excessive drying of the clay liner during the Phases I and II construction. Similar preventative procedures are specified for the Phase III clay liner construction (see Appendix O.1).

3.5.9 Geochemical Tests

Background geochemical analyses were conducted on representative claystone, siltstone, and sandstone samples prior to the development of B-18. These test results are presented in Appendix D.10.

Additional geochemical analyses were performed on seven bedrock samples collected from the B-18 excavations during the construction of Phases I and II (Golder, 1992; GCS, 1993b). The results of these tests were consistent with the typical natural background composition (in terms of analytes and concentrations) of the San Joaquin Formation bedrock.

3.6 Method 9090 (Liner/Leachate Compatibility Testing

As a condition of the Hazardous Waste Facility Permit (DTSC, 2003) "the Permittee shall test all components of landfill liners for waste/leachate compatibility using EPA Method 9090 or other more appropriate methods approved by DTSC. The liner components include seamed portions of 60-mil [HDPE], [HDPE] geomembrane material, [HDPE] geonet, geotextiles fabric, graded gravel used as drainage material, and [HDPE] piping used in the leachate collection systems."

For Landfill B-18 Phases I and II, leachate samples from an on-site hazardous waste landfill were used to test compatibility with the liner components. The following reports were submitted to the agencies, confirming the acceptability of the materials:

- Chemical Compatibility Testing of National Seal 60 mil Geomembrane with Kettleman Hills Waste Leachate, Soltex Resin, NSC#CO2A, Final Report (TRI/Environmental, Inc., October 14, 1991)
- Leachate Compatibility of Geosynthetic Materials – Kettleman Hills Facility, Final Report (J&L Testing Company, November 4, 1991)
- Geotechnical Laboratory Test Results Aggregate/Leachate Compatibility Testing, Kettleman Hills Facility, (J&L Testing Company, November 7, 1991)
- NSC 60 mil Textured HDPE Chemical Compatibility Testing EPA Method 9090 – Kettleman Hills Facility, (J&L Testing Company, September 8, 1992)

The materials that were tested in 1991 and 1992 (during the B-18 construction) by J&L Testing Company included:

Gundle XL-14 Geonet
NSC PN-3000 Geonet
Trevira 1125 Geotextile
Gundle 60mil HDPE Geomembrane
Gundle 60mil Textured HDPE Geomembrane (New Resin)
NSC 60 mil Textured HDPE Geomembrane (1992 testing)
PVC Pipe
HDPE Pipe
LCRS Gravel

Testing was conducted in accordance with the Test Protocol and Methodology for Compatibility Testing (CWMI, May 31, 1988, revised August 31, 1989). This Test Protocol was approved with the issuance of EPA Permit Modification #2 and DTSC Permit Modification #1. Results of the testing indicate that the liner components, when exposed to leachate, would function satisfactorily and had no adverse cumulative effect on the physical and/or engineering properties.

Phase III will utilize similar materials for the construction of the liner components. The previous test results as well as industry-wide testing of liner materials with leachate (see Appendix F, Attachment 3), indicate that the proposed materials will function without adverse effect due to the exposure to leachate. Based on these data, no additional compatibility testing is proposed for materials to be used in the construction of Phase III. As allowed by the Hazardous Waste Facility Permit, the "existing

test data from similar studies, and manufacturer supplied specifications [may be] used as an alternative [to testing].”

4.0 LANDFILL B-18 DESCRIPTION

4.1 General

This section describes the B-18 design configuration, the key elements of B-18, and the supporting reasoning for the B-18 design.

B-18 development includes the following three phases:

- Phase I, which was constructed from October 1990 to February 1992 and has a footprint area of approximately 21 acres as shown on Sheet 2 in Appendix A.1.
- Phase II, which was constructed from August 1992 to November 1993 and has a footprint area of approximately 32 acres as shown on Sheet 2 in Appendix A.1.
- Phase III, which is anticipated to be constructed in 2012 and will have a footprint area of approximately 13.8 acres as shown on Sheet C-3 in Appendix A.2.

The landfill components and construction procedures for the three phases and final closure of B-18 are described in the following documents:

1. The Drawings for Phases I and II provided in Appendix A.1. It is noted that the original final closure design of B-18 shown on the Drawings in Appendix A.1 is superseded by the final closure design shown on the Drawings in Appendix A.2 and discussed herein.
2. The Drawings for Phase III and final closure provided in Appendix A.2.
3. The Specifications and CQA Plan for Phases I and II (ESI, 1990b).
4. The Specifications for Phase III and final closure contained in Appendices O.1 and O.2, respectively.
5. The CQA Plans for Phase III and final closure presented in Appendices P.1 and P.2, respectively.
6. The CQA Reports prepared for Phase I, which consist of the following:
    a. Volume 1 – Subgrade Geologic Mapping Report (Golder, 1992).
    b. Volume 2 – Clay Liner Source Report (ECS, 1992a).
    c. Volume 3 – Secondary Clay Liner Construction Report (ECS, 1992b).
    d. Volume 4 – Secondary HDPE Liner and Leachate Collection System Construction Report (ECS, 1992c).
    e. Volume 5 – Primary Clay Liner Construction Report (ECS, 1992d).

f. Volume 6 – Primary HDPE Liner and Leachate Collection System Construction Report (ECS, 1992e).

g. Volume 7 – Summary Construction Observation Report (ECS, 1992f).

h. Volume 8 – Operational Features Report (ECS, 1992g).

i. Volume 9 – Design Changes and Design Clarifications Report (ECS, 1992h).

7. The CQA Reports prepared for Phase II, which consist of the following:

a. Volume 1 – Clay Liner Source Report (GCS, 1993a).

b. Volume 2 – Subgrade Geologic Mapping Report (GCS, 1993b).

c. Volume 3 – Excavation and Structural Fill Placement Construction Report (GCS, 1993c).

d. Volume 4 – Secondary Clay Liner Construction Report (GCS, 1993d).

e. Volume 5 – Secondary and Vadose HDPE Liner and Leachate Collection System Construction Report (GCS, 1993e).

f. Volume 6 – Primary Clay Liner Construction Report (GCS, 1993f).

g. Volume 7 – Primary HDPE Liner and Leachate Collection System Construction Report (GCS, 1993g).

h. Volume 8 – Summary Construction Observation Report (GCS, 1993h).

i. Volume 9 – Operational Features Report (GCS, 1993i).

An overview of the existing Phases I and II of B-18 is provided on the following sheets in Appendix A.1:

- Sheet 2 shows the Phases I and II areas and the former stockpile areas that were used for temporary storage of excavated materials during the construction of Phases I and II.
- Sheet 3 shows the Phase I subgrade elevations and the initial Phases I and II clay borrow area configuration in the Stratum 18-8 claystone described in Section 3 (see Figures 3.1 and 3.2).
- Sheet 7 generally shows how the Phases I and II clay borrow area was expanded after completion of Phase I but prior to the construction of Phase II. However, the grades shown on Sheet 7 were adjusted such that overexcavation below the Phase II subgrade was avoided.
- Sheet 8 shows the interim closure of Phase I and the Phase II subgrade elevations.

An overview of the proposed Phase III and final closure of B-18 is provided on the following sheets in Appendix A.2:

- Sheet C-1 and C-2 show the existing conditions of the B-18 area (as of March 28, 2008) and the location of the Phase III clay borrow area (borrow is within Landfill B-17).
- Sheet C-3 shows the subgrade elevations for all of B-18.
- Sheet C-4 illustrates the configuration of the B-18 closure cover final development grades (including benches, drainage, and access roads) for B-18.
- Sheets C-5 and C-6 show critical cross sections that further detail the development of B-18
- Sheet C-7 provides critical details for the liner cross section, liner termination, liner tie-in, and other items required for development of B-18.
- Sheet C-8 provides critical details for the extension of the existing leachate riser system as well as development of a replacement leachate riser and tank station.
- Sheet C-9 provides details to convey drainage into the new southern retention basin.
- Sheet C-10 provides additional drainage bench details and the final cover profile.

Detailed descriptions of the B-18 design are provided in the following sections:

- Section 4.2 – Phase I;
- Section 4.3 – Phase II;
- Section 4.4 – Phase III;
- Section 4.5 – Final Closure;
- Section 4.6 – Clay Borrow Operations;
- Section 4.7 – Liner Systems;
- Section 4.8 – Leachate Collection and Recovery Systems;
- Section 4.9 – Surface Water Control; and
- Section 4.10 – Utilities.

4.2 Phase I

Phase I comprises the western 40 percent (approximately) of the existing B-18 area (see Sheets 2 to 6 in Appendix A.1). Phase I of B-18 was configured so that:

- Disposed wastes are located within the 1990 CUP Facilities Boundary (Kings County, 1990).
- The number of boundary curves, which complicate excavation and liner construction, were minimized.
- Approximately 1,000,000 cubic yards of waste (including daily cover) were disposed of in Phase I.
- The waste is adequately stable under the operating and interim fill conditions.

The north, south, and west sides of Phase I form the originally-planned ultimate B-18 limits and were constructed as the final waste containment boundary. However, the proposed Phase III expansion will extend the ultimate limits of B-18 such that the Phase I area will be bordered by Phase III along the full length of its north, south, and west sides. The entire east side of Phase I is bordered by Phase II. Prior to the construction of Phase II, the east boundary of Phase I consisted of a berm (the Phase I/II Berm) that rises approximately 40 to 45 feet above the landfill base (see Sheets 3 and 8 in Appendix A.1). The Phase I/II Berm allowed waste to be filled in horizontal lifts in Phase I without having a laterally-unsupported waste slope on the Phase I base liner system. This minimized the risk of slope instability during Phase I disposal operations. The Phase I/II Berm is a permanent feature of the B-18 floor.

Almost all of Phase I is within excavated rock of the San Joaquin Formation. The only significant areas that required structural fill during the construction of Phase I were along the B-18 Perimeter Road near the northwest and southwest corners of B-18, as shown on Sheet 3A and in Section A-3A/15 on Sheet 15 in Appendix A.1.

The former access route into the Phase I area during its initial filling is illustrated on Sheet 5 in Appendix A.1. The main waste truck access included the following segments:

- Entering B-18 near the northwest corner of Phase I.
- Proceeding southward along the northern two–thirds (approximately) of the B-18 Perimeter Road on the west side of B-18.
- Proceeding down the 35-foot-wide access ramp on the west, south, and east Phase I waste area slopes. Special liner and road construction details for this access ramp are discussed in Section 4.7.3.3.

The Phase I waste area access ramp discussed above was aligned to intersect the top of the Phase I/II Berm near the southeast corner of Phase I. This allowed operations personnel to move landfill equipment and daily soil cover into the Phase I waste area along a temporary road on top of the Phase I/II Berm without impacting the main waste truck access. The appropriate manner for handling site traffic was refined on an on-going basis as operational experience was gained.

The existing B-18 Perimeter Road (see Section A-3,8/15 on Sheet 15 and Section A on Sheet 17 in Appendix A.1) is typically set back approximately 20 feet from the waste disposal limit to allow for the future construction of the final closure cover (Section 4.5). This separation is wider at two locations along the western portion of the B-18 Perimeter Road to accommodate the leachate collection and recovery system (LCRS) riser pads.

The base of Phase I is subdivided into two separate leachate collection zones that are referred to as Areas IA and IB, as shown on Sheets 2 through 5 in Appendix A.1. These areas are sloped toward two separate leachate sumps. This arrangement reduces the flow length for leachate to be collected as compared to an arrangement with only one sump at either end of the Phase I base. The sump areas are described in Section 4.8.

Waste was placed in nearly level, 10-foot-thick lifts across the entire Phase I area. This filling method avoided the condition of having interim waste slopes that were supported directly on the liner system. The interim waste surface was sloped slightly toward the north to allow for collection of surface water at a single location away from the waste fill access ramp.

Sheet 6 in Appendix A.1 shows the Phase I Intermediate Closure configuration that provided approximately 1,000,000 cubic yards of initial airspace in the Phase I area. The intermediate closure was primarily an operational condition to allow time for the then newly constructed Phase II to be filled to an elevation above the Phase I/II berm. Design considerations for this intermediate fill plan consisted of the following:

- A maximum waste elevation of approximately 810 feet to provide the 1,000,000 cubic yards of airspace.
- The east-facing intermediate closure slope was configured to provide adequate stability against a potential wedge failure occurring along the liner system. This stability consideration is discussed in Section 5.3.4.
- The south-facing intermediate closure slope was configured such that the access ramp to the Phase I waste area was maintained to provide access into the Phase II disposal area.
- The north-facing intermediate closure slope was provided to avoid having any laterally-unsupported portion of the waste fill directly on the liner system.

The Phase I Intermediate Closure top deck included a run-off collection sump to temporarily collect direct rainfall run-off from the top deck area. Control of this run-off is described in Section 4.9.3.

Once the Phase I Intermediate Closure elevations were reached, the flatter portions of the intermediate closure slopes (i.e., the top deck slopes) were temporarily covered by a nominal soil foundation layer and an overlying temporary 40-mil HDPE geomembrane for infiltration control. The temporary run-off collection sump in the top deck area was also lined with a 40-mil HDPE geomembrane to minimize infiltration during the infrequent periods when surface water was temporarily retained in this sump. The sideslope portions of the Phase I Intermediate Closure area were covered with soil as shown in Section A-6/23 on Sheet 23 in Appendix A.1 to provide stability and infiltration control. This soil was recovered and used for daily cover when waste disposal in the Phase I area resumed. Additionally, the 40-mil HDPE geomembrane was removed prior to covering the interim Phase I top deck with additional waste.

4.3 Phase II

Phase II comprises the eastern 60 percent (approximately) of the existing B-18 area (see Sheets 2, 8, 9, and 10 in Appendix A.1). This area encompasses the initial Phases I and II clay borrow area (excavated during the Phase I construction), the Phases I and II clay mixing area shown on the Drawings in Appendix A.1, and the Phase II clay borrow area expansion shown on Sheet 7 in Appendix A.1 (excavated prior to the construction of Phase II). Phase II of B-18 was configured so that:

- Disposed wastes are located within the 1990 CUP Facilities Boundary (Kings County, 1990).
- The number of boundary curves, which complicate construction, were minimized.
- The waste is adequately stable under the operating and interim fill conditions.

The eastern portion of Phase II was located to allow for the construction of the Northeast Containment Basin (see Sheet C-2 and C-3 in Appendix A.2).

Most of the Phase II area was also formed by excavation into rock of the San Joaquin Formation. A fill embankment was constructed along the eastern portion of Phase II where the former main natural drainage channel formed a low spot in the B-18 perimeter (see Figure 2.1). This fill embankment contains waste on its western (Phase II) side and forms the western sideslope of the Northeast Containment Basin on its eastern side. A cross-section of this embankment is shown in Section A-8/23 on Sheet 23 in Appendix A.1. Fill was also placed along the southern portion of the B-18 Perimeter Road (see Section D-8/15 on Sheet 15 in Appendix A.1) and to form the remaining upper sideslopes of the Northeast Containment Basin.

Waste truck access into Phase II occurred along the previously-described (see Section 4.2) access route into Phase I. This was accomplished by extending the Phase I access ramp across the Phase I/II Berm and then constructing an access ramp down the southern Phase II sideslope, as shown on Sheet 9 in Appendix A.1. This access ramp was 44 feet wide (see Section A on Sheet 16A in Appendix A.1) to provide adequate room for waste truck and operations equipment traffic.

As with Phase I, the existing B-18 Perimeter Road (see Section A-3,8/15 on Sheet 15 and Section A on Sheet 17 in Appendix A.1) is typically set back approximately 20 feet from the Phase II waste disposal limit to allow for the future construction of the final closure cover (Section 4.5). This separation is wider at two locations along the northeastern and southeastern portions of the B-18 Perimeter Road to accommodate the Phase II LCRS riser pads.

Phase II is also provided with two separated sumps (see Sheets 8, 9, and 10 in Appendix A.1) serving areas designated as Areas IIA and IIB. These sumps are located to optimize drainage distances within the Phase II LCRS and to provide access for the leachate pump and storage facilities along the Phase II perimeter.

Waste placement in the Phase II area also occurred in nearly level lifts to avoid laterally-unsupported slopes against the liner system. A slight slope on the waste surface was maintained toward one or two low areas during filling to allow for collection of direct rainfall run-off in the Phase II area. Procedures that were used for handling this run-off are described in Section 4.9.5.

When the waste elevation in Phase II reached the Phase I/II berm height, the entire existing B-18 Landfill began operating as a single contiguous disposal area.

4.4 Phase III

The proposed Phase III vertical and lateral expansion of B-18 will increase the footprint area of the landfill by approximately 14 acres. Most of the lateral expansion area will be along the existing western, northwestern, and southern edges of B-18. Sheet C-3 in Appendix A.2 shows the limit of the existing B-18 liner system and the limit of the Phase III expansion area. Phase III of B-18 is configured so that:

- Disposed wastes are located within the modified CUP Facilities Boundary.
- The number of boundary curves, which complicate construction, will be minimized.
- The maximum waste elevation is increased from 965 feet to 1,018 feet, which provides B-18 with a total airspace of approximately 15,700,000 cubic yards. Of this total airspace capacity (volume between base grades and final grades which includes lining and final cover systems), the expansion of B-18 accounts for approximately 5,000,000 cubic yards of airspace.
- The waste is adequately stable under the operating, interim, and final fill conditions.

Construction of the Phase III liner system will be completed in one continuous construction sequence in accordance with the certified EIR. However, to facilitate early use of a portion of the expansion area, KHF will submit a CQA certification report for the 3.5-acre Phase IIIA area in the northwestern portion of the Phase III expansion area. Once approval from the regulatory agencies is obtained, the site will begin placement of waste within the approved Phase IIIA limits. Construction of the Phase IIIB liner system will continue and would be expected to be completed within 6 months of the initiation of waste placement in Phase IIIA. A separate CQA certification report will be prepared and submitted for Phase IIIB.

The configuration of the Phase IIIA waste fill is shown on Sheet C-4A in Appendix A.2. As can be seen on Sheet C-4A, the waste placement in Phase IIIA will involve filling to final design grades along the north, east, and west portions of the landfill. The south limit of waste in Phase IIIA will terminate in a 2H:1V interim waste fill slope. A lined temporary stormwater containment berm will be provided a minimum of 10 feet from the toe of the Phase IIIA interim waste slope as shown in Detail 1 on Sheet C-4A. This temporary berm will prevent stormwater run-off from the 24-hour PMP storm event from leaving the Phase IIIA area and will also prevent stormwater run-on from entering the Phase IIIA area from the south, as discussed in Sections 5.5.5 and 5.5.4, respectively. The Phase IIIA area will not involve the construction of any leachate controls; the temporary stormwater containment berm will also serve to contain leachate and direct this leachate to the adjacent Phase IA leachate collection system.

The southern limits of Phase III are located to allow for the construction of a second surface water run-off containment basin for B-18, herein referred to as the South Containment Basin, which will be built during the construction of Phase IIIB. The layout of Phase III and the South Containment Basin also allows two of the existing groundwater monitoring wells along the south side of B-18 (K-51 and K-32R) to be protected during the construction of Phase III. Monitoring well K-68 will be extended due to soil fill placement in the vicinity of this well.

The Phase III expansion will involve the construction of an additional sideslope liner system only (i.e., no additional base liner will be installed). Most of the Phase III sideslope liner will be constructed over either the existing B-18 Perimeter Road or the existing rock cut slopes located above the existing B-18 Perimeter Road. The existing rock cut slope will be regraded to the proposed design subgrade for Phase III. A fill embankment will be required along much of the southern and southeastern boundary of the Phase III limits to build this area up to the design subgrade elevations. This fill embankment will contain the waste on its northern (Phase III) side and will form the northern sideslope of the South Containment Basin on its southern side. A cross-section of this embankment is shown in Section D on Sheet C-5 in Appendix A.2.

Waste truck access into the Phase III disposal area will be initially through the existing entry point into B-18 at its northwest corner. Access to B-18 will eventually be relocated to the west side perimeter access road as the waste fill is extended above the surrounding topography.

The new B-18 Perimeter Road (see Sheet C-3 and C-4 in Appendix A.2) will typically be set back a minimum of 20 feet from the Phase III waste disposal limit to allow for the future construction of the final closure cover (Section 4.5). This separation will be wider at the locations of the three LCRS riser pads (the riser pads for Areas IA, IB, and IIB) that will be relocated up to the new B-18 Perimeter Road during the Phase III construction.

Phase III will not include any additional floor areas. Hence, no new LCRS sumps will be constructed as part of Phase III. Depending upon where leachate originates within Phase III, it will flow to one of the four existing sumps (IA, IB, IIA, and IIB).

Similar to Phases I and II, waste will be placed in the Phase III area in nearly level lifts to avoid laterally-unsupported slopes against the liner system. A slight waste surface slope will be maintained toward one or two low areas during operations to allow for collection of direct rainfall run-off in the Phase III area. Procedures that will be used for handling this run-off are described in Section 4.9.7.

4.5 Final Closure

Sheet C-4 in Appendix A.2 shows the proposed B-18 final closure configuration, which is based on the following parameters:

- Overall closure slope inclinations of 4H:1V.
- Approximately 25-foot wide benches at maximum vertical intervals of 50 feet.
- Approximately 3.5H:1V slope inclinations between the individual benches.

Access to each final cover bench and the top deck will be provided by either the new B-18 Perimeter Road or the Cover Access Road that will run up the west sideslope of B-18 at the approximate location shown on Sheet C-4 in Appendix A.2. This Cover Access Road will be developed during operations to haul waste onto the above-grade disposal areas.

The final cover benches will be sloped to direct surface water flow to the Cover Access Road and/or the new B-18 Perimeter Road. The longitudinal slope of the benches will generally be about 2 percent to control flow velocities and to allow adjustment for differential settlement of the waste. The Cover Access Road and the new B-18 Perimeter Road will both be sloped at about 8 percent in most locations.

Each final cover bench will be configured as a trapezoidal drainage ditch to provide the necessary capacity to adequately convey surface water flows resulting from the 6-hour Probable Maximum Precipitation (PMP) storm event. The 6-hour PMP is used to design conveyance structures (e.g. channels) since the rainfall intensity is greater than the 24-hour PMP, and is therefore conservative. The Cover Access Road will be configured with a lined V-ditch to convey surface water flows from the 100-year 24-hour storm event. In the event of the 6-hour PMP storm event, the flow will be contained within the road width.

In accordance with the current Hazardous Waste Facility Permit for the KHF (DTSC, 2003), the final cover system for B-18 will consist of the following components (from bottom to top):

- Intermediate soil cover (minimum of 1 foot) over the last lift of waste.
- A foundation layer consisting of a minimum of 1 foot of compacted soil having a maximum permeability of $1x10^{-5}$ cm/s.
- 40-mil textured HDPE geomembrane.
- A 12 oz/sy nonwoven geotextile.
- A minimum 2.5-foot-thick vegetative cover soil layer. The top surface of the vegetative cover soil layer will be vegetated with plants having shallow root depths. Seed types for the final cover vegetation are provided in Section 02924 of the final closure Specifications in Appendix O.2.

The portions of the geomembrane located under the Cover Access Road and benches will be sloped at a minimum of 2 percent toward the outside of the landfill so that any water in the geotextile drainage layer can flow toward the toe of the cover system around the perimeter of the landfill.

Detail 4 on Sheet C-7 in Appendix A.2 shows the typical perimeter detail of how the final cover system will be terminated and toed out onto the B-18 Perimeter Road. The HDPE geomembrane and geotextile of the cover system will be terminated approximately 5-feet out beyond the limit of the foundation layer.

In accordance with 22 CCR 66264.111 and 66264.310, B-18 has been designed to be closed in a manner that will:

- Minimize the need for further maintenance;
- Control, minimize or eliminate, to the extent necessary, to protect human health and the environment, post-closure escape of hazardous waste, hazardous constituents, leachate, contaminated rainfall or run-off, or waste decomposition products to groundwater, surface water or the atmosphere;
- Prevent the downward entry of water into the closed landfill throughout a period of at least 100 years;
- Promote drainage;
- Accommodate settling and subsidence so that the cover's integrity is maintained; and

- Accommodate lateral and vertical shear forces generated by the MCE.

After waste acceptance ceases in B-18, the intermediate cover/foundation layer will be graded per the final closure grading plan, as shown on Sheet C-4. A 40 mil HDPE geomembrane, geotextiles and vegetative cover will be constructed over the foundation layer. The entire cover will be vegetated for erosion control. The final cover has been designed to avoid ponding, control run-off, minimize erosion and withstand the MCE event. Therefore the cover will function with minimum maintenance. The base liner and closure cover will provide barriers to protect human health and the environment.

Post-closure inspections will be performed and post-closure maintenance will occur in accordance with the Hazardous Waste Facility Permit (DTSC, 2003).

4.6 Clay Borrow Operations

4.6.1 Phases I and II

The initial Phases I and II clay borrow area excavation was completed during the Phase I excavation as shown on Sheet 3 in Appendix A.1. Sheet 7 in Appendix A.1 shows how the Phases I and II clay borrow area was extended toward the southeast before the Phase II construction began. The cross-sections on Figure 3.2 illustrate how the area was excavated to borrow clay from the thick Stratum 18-8 claystone in the Phase II footprint.

The west-facing sideslope of the clay borrow area was excavated along the rock's dip as the claystone was recovered down to the underlying stratum. This sideslope had an inclination of approximately 25 to 30 degrees on average. The other sideslopes of the clay borrow area cut across bedding planes and were inclined at 2H:1V.

A bench was provided around the initial borrow area (see Sheet 3 in Appendix A.1) at an elevation of approximately 720 feet. This bench was used to anchor the 40-mil HDPE geomembrane (see Section B on Sheet 13 in Appendix A.1) in the bottom of the borrow area. This lined area served as an interim containment basin for run-off from the Phase I access roads.

Sheet 3 in Appendix A.1 also shows how the eastern portion of the Phase II area was initially graded to create a relatively flat clay mixing area. KHF construction crews prepared the claystone in this area to achieve the required engineering properties for clay liner material. The clay preparation procedures used for the Phases I and II construction included the following activities:

- Ripping and excavation of the claystone in a manner that reduced the friable material to relatively small particle sizes;
- Mechanical breakdown of the excavated material to further reduce particle sizes; and
- Moisture conditioning of the clay liner material on mixing tables.

The clay liner test pad described in Appendix E.1 was constructed of Stratum 18-8 clay from the Phases I and II borrow source. This clay liner test pad program demonstrated that the Stratum 18-8 clay was suitable for use as clay liner material under field conditions.

4.6.2 Phase III

The Phase III clay borrow area will be located adjacent and north of Landfill Unit B-17 (i.e., northwest of B-18), as shown on Sheet C-1 in Appendix A.2. This clay borrow source consists of a thick bed of the Pecten Claystone (see Appendix E.2). The contractor will be responsible for excavating and processing all of the required clay liner material for Phase III. Excavation and processing of the Pecten Claystone may be performed as part of the construction of various phases of Landfill B-17. KHF personnel will instruct the contractor on the appropriate excavation configurations to be used when mining the Pecten Claystone from the borrow area.

Sheet C-1 in Appendix A.2 also shows the designated clay mixing area that will be used by the contractor to process and prepare the clay liner material. This mixing area will be located adjacent to the clay borrow area (i.e., northwest of B-18). The final clay preparation procedures, to be determined by the contractor, may include combinations of the following activities:

- Ripping and excavation of the claystone in a manner that reduces the friable material to relatively small particle sizes;
- Crushing of the excavated material to further reduce particle sizes;
- Blending different portions of the claystone by the use of a pugmill or discing the material in lifts; and
- Pre-wetting stockpiled clay material with fresh water and/or a weak dispersant solution to accelerate weathering prior to re-excavation of the stockpiled clay.

A clay liner test pad was constructed from the Pecten Claystone material at the end of July 2008. The SDRI test report was completed in December 2008 (Geosyntec, 2008a). This clay liner test pad program verified the adequacy of the clay material from the Phase III borrow source when placed and compacted under actual field conditions. Based on laboratory testing and the SDRI test by Geosyntec (see Appendices E.2 and E.3), the Pecten Claystone material meets the requirements for use as clay liner. Additional pre-construction testing will be performed to confirm materials used for the construction meet the specified properties.

For Phase III, the compacted clay liner will be constructed using the same specifications as were used for the Phases I and II clay liner (see Section 4.7.2.1). Similar construction equipment will be used to compact the Phase III clay liner as was used to construct the Phases I and II clay liner test pad.

4.7 Liner Systems

4.7.1 General

Liner system details and sections for Phases I and II are shown on Sheets 16 through 22A in Appendix A.1. The Phase III and final closure liner system details and sections are shown on Sheets C-7 through C-10 in Appendix A.2. The B-18 liner configurations are generally the same as those successfully used for prior KHF disposal WMUs. The primary modifications to the B-18 liner system design compared to KHF landfills designed prior to 1990 are:

- The use of textured HDPE geomembranes throughout B-18 and the use of geocomposites and geotextiles on the sideslope areas to improve the stability of B-18.

- The use of protective liner material on the B-18 sideslopes to provide temporary ultraviolet protection to the underlying geotextile component of the geocomposite. This protective liner is removed as the operations layer soil is periodically extended up the slope in advance of the waste mass.

Each of the basic liner systems used in B-18 are described in Section 4.7.2. Special liner construction details (e.g., anchor trenches) are described in Section 4.7.3. Appendix F.1 contains data sheets that list representative properties of the geosynthetic materials used in the construction of Phases I and II. Similarly, Appendix F.2 contains data sheets that list typical properties of the geosynthetic materials that will be used in the construction of Phase III and the final cover. The CQA reports for Phases I and II contain detailed information on the properties of the existing B-18 liner systems.

4.7.2 Liner System Configurations

4.7.2.1 Base Liner

The existing base (i.e., floor) areas of B-18 were each graded to drain toward a sump where leachate is monitored and collected in the three separate zones (primary, secondary, and vadose) described in Section 4.8. The entire base of each of the four areas (IA, IB, IIA, and IIB) was graded at 2 percent toward a central flow line. The central flow line in each area was sloped at 2.4 percent toward that area's respective collection sump. No new base area will be constructed for Phase III.

Detail 2 on Sheet 16 in Appendix A.1 shows the general B-18 base liner system configuration, which consists of the following components (from top to bottom):

- A 2-foot-thick (minimum) base soil operations layer. This operations layer was constructed from on-site granular material with a maximum particle size of 6 inches. The purpose of this layer was to provide a working surface for waste trucks and landfill equipment while protecting the underlying liner system components.

- A 1-foot-thick (minimum) primary LCRS consisting of the following components (from top to bottom):

  An 8 oz/sy nonwoven geotextile (Trevira 1125) to function as a filter below the operations layer soil.

  A 12-inch-thick (minimum) drainage gravel layer. A single-sided geocomposite filter/drainage layer consisting of an 8 oz/sy nonwoven geotextile (Trevira 1125) thermally-bonded to one side of a Polynet 3000 geonet. The geocomposite was placed with the geotextile facing up.

  An 8 oz/sy nonwoven geotextile (Trevira 1125) to provide increased interface shear strength.

- A composite primary liner consisting of the following components (from top to bottom):

  A 60-mil textured HDPE geomembrane.

A 1.5-foot-thick (minimum) layer of compacted clay having a maximum permeability of 1 x 10^{-7} cm/s.

- A secondary LCRS consisting of the following components (from top to bottom):

  A 16-oz/sy nonwoven geotextile (Trevira 1155) to function as a filter below the primary clay liner.

  An approximately 12-inch-thick layer of drainage gravel.

  A single-sided geocomposite filter/drainage layer consisting of a 16-oz/sy nonwoven geotextile (Trevira 1155) thermally-bonded to one side of a Polynet 3000 geonet. The geocomposite was placed with the geotextile facing up.

  An 80-foot-wide layer of Polynet 3000 geonet centered along the entire secondary LCRS flow line above the vadose trench.

  A 16-oz/sy nonwoven geotextile (Trevira 1155) to provide increased interface shear strength.

- A composite secondary liner consisting of the following components (from top to bottom):

  A 60-mil textured HDPE geomembrane.

  A 3.5-foot-thick (minimum) layer of compacted clay having a maximum permeability of 1x10-7 cm/s.

- A prepared subgrade that was graded smooth and proof-rolled to assure that soft or loose zones did not exist.

Both of the clay liners (primary and secondary) were placed in 8-inch-thick (maximum) loose lifts before compaction. The Phases I and II Specifications (ESI, 1990b) required the clay to be compacted to a dry density of at least 90 percent of its Modified Proctor maximum dry density (ASTM D1557) at a water content wet of optimum (ASTM D1557). During the Phase I construction, the clay liner placement specifications were modified to allow the compacted clay's dry density and moisture content to lie within the window defined by the following four points on a moisture-dry density plot:

- Two (2) percent above the optimum moisture content for a dry density equal to 90 percent of the Modified Proctor maximum dry density.

- Five (5) percent above the optimum moisture content for a dry density equal to 90 percent of the Modified Proctor maximum dry density.

- One (1) percent above the optimum moisture content for a dry density equal to 98 percent of the Modified Proctor maximum dry density.

- Three (3) percent above the optimum moisture content for a dry density equal to 97 percent of the Modified Proctor maximum dry density.

This window, which allowed a lower water content for higher compactive efforts, was established to:

- Assure that both the required strength and permeability characteristics of the clay were achieved; and
- Provide the flexibility needed for controlling the clay's moisture content in an arid environment.

It should be noted that an allowance was made for up to 20 percent of the clay moisture-density test results to be slightly outside the above-described compaction window by ±0.5 percent for moisture content and -0.5 percent for relative compaction as long as the average of all acceptable tests for the day fell within the compaction window. The above-described compaction window (along with the allowance for outliers) was used for both the Phases I and II clay liner construction and was formally documented in two design change letters prepared by ESI and contained in the CQA reports for Phases I and II (ECS, 1992h; GCS, 1993h). Copies of both of these ESI letters are included in Appendix E.4.

4.7.2.2 Vadose Zone Trench

Section C on Sheet 16 in Appendix A.1 shows the 12-foot-wide vadose trench that is located directly below the secondary clay liner and along the flow line of the LCRS. Key elements of the vadose trench are:

- An 80-mil smooth HDPE geomembrane, which extends approximately 2.5 feet beyond both sides of the trench; and
- A 1-foot-thick layer of drainage gravel wrapped in a 16-oz/sy nonwoven geotextile (Trevira 1155).

4.7.2.3 Phases I and II Sideslope Liner

Detail 1 on Sheet 16 in Appendix A.1 shows the typical existing Phases I and II sideslope liner system configuration. This system includes each basic component of the base liner system except the drainage gravel layers and the primary clay liner, none of which are required due to the relatively steep inclination of the sideslope liner system and the resulting rapid drainage of any liquids in the LCRS. The Phases I and II sideslope liner system consists of the following components (from top to bottom):

- A 2-foot-thick (minimum) soil operations layer to protect the liner system from the disposal operations. A 1-inch maximum particle size criterion was established for the slope operations layer because this material was placed directly against the geosynthetic layers. The slope operations layer was placed in increments at least 3 feet but not more than 10 feet above the rising waste level.
- A temporary protective liner to protect the underlying geotextile component of the geocomposite from ultraviolet light prior to placement of the operations layer. This protective liner consisted of white 40-mil smooth HDPE geomembrane and was removed as the operations layer was placed.

- A primary LCRS consisting of a single-sided geocomposite underlain by an 8 oz/sy nonwoven geotextile (Trevira 1125). The single-sided geocomposite consisted of an 8 oz/sy nonwoven geotextile (Trevira 1125) thermally-bonded to a Polynet 3000 geonet and was placed with the geotextile facing up. In construction of Phase II the components were combined in a double-sided geocomposite.

- A primary liner consisting of a 60-mil textured HDPE geomembrane.

- A secondary LCRS consisting of a single-sided geocomposite underlain by an 8 oz/sy nonwoven geotextile (Trevira 1125). The single-sided geocomposite consisted of an 8 oz/sy nonwoven geotextile (Trevira 1125) thermally-bonded to a Polynet 3000 geonet and was placed with the geotextile facing up. In construction of Phase II the components were combined in a double-sided geocomposite.

- A composite secondary liner that is the same as that used in the base liner system and consists of the following components (from top to bottom):

  A 60-mil textured HDPE geomembrane.

  A 3.5-foot-thick (minimum) layer of compacted clay having a maximum permeability of 1 x 10^{-7} cm/s.

- The sideslope subgrade that was prepared differently than the subgrade for the base liner in order to increase stability of the clay liner. The sloped subgrade surface was scarified to a depth of approximately 4 inches as the clay liner was placed to create a rough interface between these two soil layers.

4.7.2.4 Phase III Sideslope Liner

Detail 1 on Sheet C-7 in Appendix A.2 shows the typical Phase III sideslope liner system configuration. This system is the same as the existing sideslope liner system for Phases I and II except the secondary clay liner will have a minimum thickness of 3 feet instead of 3.5 feet and double-sided geocomposites will be used instead of single-sided geocomposites with an underlying geotextile. The Phase III sideslope liner system will consist of the following components (from top to bottom):

- A 2-foot-thick (minimum) soil operations layer with a 1-inch maximum particle size criterion. The slope operations layer will be placed in increments at least 3 feet but not more than 10 feet above the rising waste level.

- Prior to the placement of the 2-foot-thick operations layer on the slope, a temporary 40 mil thick white HDPE protective liner will be installed. The protective liner will be removed as the operations layer is placed.

- A primary LCRS consisting of a double-sided geocomposite.

- A primary liner consisting of a 60-mil textured HDPE geomembrane.

- A secondary LCRS consisting of a double-sided geocomposite.

- A composite secondary liner consisting of the following components (from top to bottom):

  A 60-mil textured HDPE geomembrane.

  A 3-foot-thick (minimum) layer of compacted clay having a maximum permeability of 1 x 10^{-7} cm/s.

- A prepared subgrade that will be scarified to a depth of approximately 4 inches as the clay liner is placed to create a rough interface between these two soil layers.

4.7.2.5 Base to Sideslope Liner Transition

Details 4 and 5 on Sheet 16 in Appendix A.1 show how geotextiles were wrapped around the ends of the drainage gravel and clay layers where the existing base and sideslope liner systems meet. Since there will be no connection between the new Phase III side slope liner and any of the base liner areas of Phases I and II, this does not apply to construction of the Phase III area.

4.7.2.6 Final Cover Liner

The final closure cover liner system for B-18 will be similar to the final cover liner system approved in the Part B Permit and which has been used for closure of several other WMUs at KHF. The final closure cover liner system is described in Section 4.5.

4.7.3 Special Liner Details

4.7.3.1 Sideslope Liner Anchor Trenches

Detail 3 on Sheet 16 in Appendix A.1 shows the typical existing sideslope liner system anchor trench around the perimeter of the existing landfill. The primary requirements of the anchor trench are to prevent the geosynthetic components of the liner system from being pulled down the slope and to minimize the potential for surface water to enter the LCRSs. A vertical separation of 0.5 feet was maintained between the individual geosynthetic components in the anchor trench to provide soil friction against each of these geosynthetics. A 3-foot-tall soil berm was installed above the anchor trench to increase the frictional resistance on the geosynthetic components and to control surface water drainage.

The construction of Phase III will result in some of the existing anchor trenches being removed, as shown in Detail 5 on Sheet C-7 in Appendix A.2. The new anchor trenches for the Phase III sideslope liner system will typically have the configuration shown in Detail 2 on Sheet C-7 in Appendix A.2. The new anchor trenches will be configured similar to the existing anchor trenches except that the 0.5-foot vertical separation between geosynthetic components in the anchor trench is not required and the depth and width of the Phase III anchor trench are slightly less than that of the existing anchor trench.

The final closure cover system will be installed above the perimeter anchor trench as shown in Detail 4 on Sheet C-7 in Appendix A.2. This detail was described in Section 4.5.

4.7.3.2 Temporary Phase I/II Transition Anchors

The B-18 liner system was temporarily terminated along the eastern edge of Phase I prior to the construction of Phase II. During the construction of Phase II, the Phases I and II liner systems were spliced together at the following locations:

- Along the top of the Phase I/II Berm.
- Along the north and south sideslopes above the Phase I/II Berm.

Section B-5,15,23/17 on Sheet 17 in Appendix A.1 shows the temporary liner system configuration at the top of the Phase I/II Berm at the end of the Phase I construction. The temporary anchor trench at the Phase I/II Berm was relatively far from the slope (12 feet) to provide room for the splicing of the Phases I and II liner systems. Also, the portion of the liner system on top of the Phase I/II Berm was sloped 2 percent toward Phase I to assure that leachate ponding did not occur. A small soil berm was constructed on top of the temporary anchor trench to increase frictional resistance of the geosynthetics, control surface water drainage, and provide a foundation for the temporary lights as shown in Section B on Sheet 23 in Appendix A.1.

The procedure that was used to splice the Phases I and II liner systems at the Phase I/II Berm included the following steps:

- The temporary small soil berm, drainage ditches, and light poles were removed.
- The liner system was cut a minimum of 3 feet back (i.e., toward Phase I) from the temporary anchor trench and then this anchor trench was removed.
- The east side of the Phase I/II Berm was graded to match the base of the existing clay liner while maintaining the 2 percent slope toward Phase I.
- The existing 4-foot-thick secondary clay liner on top of the Phase I/II Berm was extended to connect with the clay liner on the west sideslope of Phase II.
- Each individual geosynthetic component was spliced at the cut location, resulting in a continuous liner system over the top of the Phase I/II Berm as shown in Section B-9,10/17 on Sheet 17 in Appendix A.1.

Section B-4,5/16 on Sheet 16 in Appendix A.1 shows the temporary anchoring procedure that was used for the Phase I/II transition on the south and north sideslopes above the Phase I/II Berm. The temporary anchors at these locations were different than those at the tops of the sideslopes since there are no significant liner stresses acting perpendicular to the anchoring. The temporary edge of the Phase I sideslope liner was anchored by:

- Securing approximately 6 feet of the secondary 60-mil HDPE geomembrane beneath 2 feet of compacted clay. Clay was used to provide increased erosion resistance on the slope.
- Cutting and welding the primary 60-mil HDPE geomembrane and temporary protective liner to the secondary 60-mil HDPE geomembrane above the temporary

anchor trench. The geocomposites and geotextiles were also cut to end just inside these welds.

Section B-4,5/16 on Sheet 16 in Appendix A.1 also shows how clay was used to contain a temporary 18-inch diameter corrugated metal pipe (CMP) on the southside slope only. This pipe was used to convey surface water run-off from the B-18 Perimeter Road to the temporary drainage ditch along the top of the Phase I/II Berm.

The splicing of the Phases I and II liner systems on the north and south sideslopes was similar to that described above for the liner system splice at the top of Phase I/II Berm. Section B-9,10/16 on Sheet 16 in Appendix A.1 shows how the Phases I and II sideslope liner systems were spliced together.

4.7.3.3 Phases I and II Access Ramp Liner

Section D-4,5/17 on Sheet 17 in Appendix A.1 shows a typical cross-section through the 35-foot-wide access ramp along the Phase I sideslopes. Key aspects of the design of this access ramp are:

- Both the primary and secondary LCRSs were sloped 2 percent toward the landfill to promote drainage without the need for water to flow the entire length of the ramp.
- An extra layer of 16 oz/sy nonwoven geotextile (Trevira 1155) was placed over the primary LCRS geocomposite to provide added cushioning under traffic loading.
- Three feet of operations layer soil (1-inch maximum particle size) was placed above the extra 16 oz/sy geotextile to further protect the liner system. The operations layer was also extended at least 10 feet up the sideslope areas adjacent to the access ramp to avoid the potential for traffic to accidentally drive onto the liner system.
- The roadway was finished with 1 foot of Class 2 aggregate base to provide all-weather access. Note that this aggregate base layer was used instead of the 4-inch-thick asphalt pavement shown in Section D-4,5/17 on Sheet 17 in Appendix A.1.

A special detail to weld the secondary 60-mil HDPE geomembrane was provided to facilitate continuous access along the ramp during construction and to minimize the potential for the liner to lift off of the ramp before the other materials and operations layer were placed. This was accomplished by anchoring the secondary HDPE geomembrane from the bottom sideslope in a trench about 15 feet from the toe of the upper sideslope. This 15-foot zone was then used for access until the upper sideslope portion of the secondary HDPE geomembrane was installed.

Section A on Sheet 16A in Appendix A.1 shows the arrangement of the Phase II access ramp. This ramp into the Phase II area was 44 feet wide to provide adequate space for the waste trucks and landfill equipment. The special liner details and protection details for the Phase II access ramp were similar to those described above for the Phase I access ramp.

4.8 Leachate Collection and Recovery System (LCRS)

4.8.1 General

The main features of the B-18 LCRS are the collection sumps where leachate is detected and removed. Phases I and II have two sumps each (see Sheets 5 and 10 in Appendix A.1), which

subdivide these phases into Areas IA and IB and Areas IIA and IIB, respectively. Phase III will utilize the existing sumps for Phases I and II and, therefore, no additional sumps will be installed.

Phase IIIA will be constructed such that leachate from Phase IIIA will be able to flow directly into the Phase IA LCRS. No interim control measures, except a temporary lined containment berm at the edge of Phase IIIA/IIIB (see Sheet C-4A in Appendix A.2) will be required. Stormwater contained on the north side of this temporary berm (i.e., between the berm and the Phase IIIA waste mass) will be treated as leachate and will be handled in the same manner as leachate that is collected in the existing B-18 leachate storage tanks (located on the concrete riser pads). The temporary berm has been sized such that stormwater run-off from the 24-hour PMP event will be fully contained on the north side of the berm with greater than 1 foot of freeboard.

The layout of the sumps in Areas IA and IB are shown on Sheet 18 in Appendix A.1. The layout of the sumps for Areas IIA and IIB are shown on Sheet 18A in Appendix A.1. Representative cross-sections through the sumps are provided on Sheets 19, 19A, 20, 20A, 22, and 22A in Appendix A.1. Sheet 21 in Appendix A.1 provides typical details for the vertical riser pipe that was installed at each of the four primary sump locations. Sheets 22 and 22A in Appendix A.1 also show details for the existing four riser pads (one riser pad is above each sump) that enable the removal and handling of the leachate. Of the four existing riser pads, three of them (the pads for Areas IA, IB, and IIB) will be removed during the construction of Phase III to allow for the expansion of the B-18 waste footprint in these areas. Three new riser pads will be constructed during Phase IIIB as replacements for the three pads to be removed as shown on Sheet C-3 in Appendix A.2. Typical details for the new riser pads to be constructed during Phase III are shown on Sheet C-8 in Appendix A.2. These new riser pads will be very similar to the existing ones.

Each B-18 sump includes the following three leachate (or leak detection) collection and recovery zones (from top to bottom):

- The primary LCRS, which collects liquids that have infiltrated through the overlying waste and operations layer. The design of the primary LCRS is based on providing adequate pump capacity so that the liquids level in each sump will not exceed the height of that sump, which is taken to be the elevation of the landfill base at the sump perimeter.

- The secondary LCRS, which is typically not expected to be affected by liquids infiltration but drains consolidation water from the overlying clay liner and any seepage that may pass through leaks in the primary liner system.

- The vadose zone collection system, which is located in a 12-foot-wide trench below the secondary liner system. The purpose of this zone is to detect any seepage through leaks in the secondary liner system. Some clay consolidation water may also be collected in the vadose zone as increasing amounts of waste are placed within B-18.

Each of the four sump areas is located so that leachate pumps are lowered into the respective gravel collection zones through sideslope riser pipes. The primary LCRS design also includes a vertical riser pipe in each sump area that is extended upward in segments as the surrounding waste is placed. Details of the individual systems are described in the following three subsections.

4.8.2 Primary LCRS

The maximum operating liquid level for each of the four primary LCRSs is 1 foot above the top of each sump, where the top of a sump corresponds to the lowest point where the 5H:1V sump sideslope meets the toe of the landfill sideslope. The liquid level in each sump is currently maintained as low as possible using a pump, which is lowered to the sump through an existing 8-inch diameter steel riser pipe that lies on a 60-mil HDPE rub sheet (see Section A on Sheet 22 in Appendix A.1). The bottom portion of this riser pipe consists of Type 304 stainless steel to resist corrosion. Above the level of normal liquids exposure, the existing riser pipe is carbon steel that is double-wrapped to protect against corrosion. During the Phase III construction, the primary sideslope riser pipes in Areas IA, IB, and IIB will be extended up the new Phase III sideslopes by connecting a 10-inch diameter HDPE pipe to the existing 8" diameter steel pipe as shown in Detail 3 on Sheet C-8 in Appendix A.2. The existing pumps and controls will be replaced with a system capable of reaching the extended length of the riser and providing a pumping capacity that will maintain the liquid level below the allowable limit.

At each sump, the primary leachate pump is lowered through the sideslope riser pipe into a 4-foot by 8-foot by 1.5-foot-deep gravel-filled pumping zone. This arrangement maximizes the pumping effectiveness of the system. Pumping is controlled manually as required. The pumping need is determined by a water level control bubbler system or equivalent system that provides liquid level sensing at the top of the riser pipe when sufficient liquids for pumping exist in the sump.

As shown in Section A and Detail 1 on Sheet 18 in Appendix A.1, the bottom of each sideslope riser pipe is connected to a perforated collection tee that is also made of stainless steel. This tee lies on and against 2-inch-thick HDPE flatstock to protect the primary liner system from impact and pipe movements as the pumps are operated and periodically removed for maintenance and repair.

Historically (January 2001 to December 2007), the primary LCRS has removed an average of 360 gallons per day from the 4 sumps. During this period, the peak flow in a primary LCRS sump was 98,000 gallons (Phase IB) during January 2006, or 3,300 gallons per day. The volume of liquids removed during Phase III waste placement and after closure is expected to remain the same or diminish after closure.

In addition to the sideslope riser pipes, each of the four sumps has a redundant vertical riser pipe that can also be used to pump leachate from the primary LCRS. Each of the four vertical riser pipes is extended upward in about 10-foot increments as the level of the surrounding waste rises. The design of the vertical riser pipes includes several features, shown on Sheet 21 in Appendix A.1, to control the drag loads transmitted to the pipe as the surrounding waste settles. These features include the following:

- The bottom 7 feet (approximately) consists of perforated, 18-inch diameter stainless steel pipe founded in the small pumping sump adjacent to the perforated tee for the sideslope riser pipe. This bottom pipe telescopes into the main vertical riser pipe through a concrete footing such that drag loads are not transmitted to the pumping area.

- The main 24-inch diameter carbon steel vertical riser pipe is surrounded by a thin-walled corrugated HDPE pipe. This corrugated pipe, which is very flexible in the longitudinal direction, deforms as the adjacent wastes settle, thereby reducing the potential for large drag loads to act on the steel vertical riser pipe.

- A 6-foot by 6-foot reinforced concrete footing provides support for the 24-inch diameter carbon steel pipe and any drag loads that may act on the steel pipe.

The corrugated HDPE pipe has a diameter of 30 inches to provide an annular space for lateral deformation to occur around the 24-inch steel pipe. Spacers were provided inside the corrugated pipe to keep the riser pipe alignment nearly vertical at the time of its initial installation.

As shown in Detail 6 on Sheet C-7 in Appendix A.2, slip connections will be installed in the Area IA and IB vertical risers due to the increased height of waste that will be placed over these areas during the Phase III expansion. These slip connections will be placed at an elevation of between 900 and 910 feet and will reduce the potential drag loads transmitted to the existing 24-inch diameter vertical riser pipes. Analysis of the existing concrete pad at the base of the liner to withstand the additional vertical force related to the expansion was performed and is contained in Appendix L. As shown, the existing pads can satisfactorily carry the additional load related to the extended risers and the down-drag forces related to waste settlement.

Initially, a backup 350 gpm, 20 horsepower submersible pump was installed through each vertical riser pipe and into the primary pumping sump. However, these backup pumps were never used and are no longer provided since they presented difficulties when extending the vertical riser pipes. In the future, backup pumps can be provided through the vertical risers on an as-needed basis.

During the Phase III construction, there will be a period of approximately 6 months between the demolition of the existing LCRS riser pads and the installation of the new riser pads. The primary LCRS will continue to be monitored on a daily basis for liquid level and liquids will be removed in accordance with site protocol during this period by placing pumps down the existing vertical riser pipes (which will not be disturbed during the Phase III construction) and/or by placing pumps down the sideslope risers that will be cut off at the existing riser pads. As the sideslope risers are extended, but before the new riser pads are constructed, a wye fitting will be installed in each sideslope pipe close to the locations of the demolished riser pads (i.e., near the bottom of Phase III). Pumps and level monitoring equipment can be inserted through the wye and then lowered down the sideslope riser. Once the new riser pads are completed, the wyes will be removed and the sideslope risers repaired at those locations.

4.8.3 Secondary LCRS

The existing secondary LCRS is provided with two 8-inch diameter sideslope riser pipes as shown in Section A-18/19 and Detail 2 on Sheet 19 in Appendix A.1 and in Section C-18/20 on Sheet 20 in Appendix A.1. The bottom portion of one of these pipes is Type 304 stainless steel connected to a perforated stainless steel tee. The portion of this existing pipe above the level of normal liquids exposure is double-wrapped carbon steel. The other riser pipe is SDR 8.3 HDPE pipe for its entire length. During the Phase III construction, the secondary sideslope riser pipes in Areas IA, IB, and IIB will be extended up the new Phase III sideslopes. The existing HDPE riser pipe will be extended by splicing to a similar diameter pipe while the existing steel riser pipe will be extended by splicing to a 10-inch diameter HDPE pipe as shown in Detail 3 on Sheet C-8 in Appendix A.2.

A 4-foot by 4-foot by 1.5-foot-deep pumping zone was provided for the secondary LCRS. The perforated tee lies on and against a 2-inch-thick HDPE flatstock to protect the underlying HDPE geomembrane. The HDPE riser pipe terminates against the tee.

Both secondary sideslope riser pipes are sized to contain a pump and a bubbler liquid level control gauge. However, the current operating configuration has the pump in the steel riser pipe and the level control gauge in the HDPE riser pipe.

Historically (January 2001 to December 2007), there has been very little liquid in the secondary LCRS, averaging less than 10 gallons per day from B-18 sumps. Peak flow in the secondary LCRS approached 150 gallons per day in January 2006. This peak resulted from damage to the primary liner system that allowed rainfall to enter the secondary LCRS. The damage was subsequently repaired and leachate volumes have reduced to zero since March 2007.

The existing pumps and controls will be replaced with a system capable of reaching the extended length of the riser and providing a pumping capacity that will maintain the liquid level below the allowable limit. The maximum operating liquid level is 1 foot above the top of each secondary LCRS sump.

During the Phase III construction, there will be a period of approximately 6 months between the demolition of the existing LCRS riser pads and the installation of the new riser pads. The secondary LCRS will continue to be monitored on a daily basis for liquid level and liquids will be removed in accordance with site protocol during this period by placing pumps down the sideslope risers that will be cut off at the existing riser pads. As the sideslope risers are extended, but before the new riser pads are constructed, a wye fitting will be installed in each sideslope pipe close to the locations of the demolished riser pads (i.e., near the bottom of Phase III). Pumps can be inserted through the wye and then lowered down the sideslope riser. Once the new riser pads are completed, the wyes will be removed and the sideslope risers repaired at those locations.

4.8.4 Vadose Zone Collection System

Section B-18A/19A on Sheet 19A in Appendix A.1 shows the configuration of the existing sideslope riser pipe used for pumping liquids from the vadose zone collection sump. A single 8-inch diameter pipe is provided since only a small volume of liquids (e.g., clay liner consolidation water) was expected to be removed from this sump. Historic (January 2001 to December 2010) LCRS pumping data indicate no liquids have been removed from the vadose LCRS. The bottom portion of the vadose riser pipe is stainless steel. During the Phase III construction, the vadose sideslope riser pipes in Areas IA, IB, and IIB will be extended up the new Phase III sideslopes by splicing a 10-inch diameter HDPE pipe to the existing steel pipe as shown in Detail 3 on Sheet C-8 in Appendix A.2.

A 4-foot by 4-foot by 1.5-foot-deep pumping zone is provided below the main 12-foot-wide vadose trench in the sump areas. The 80-mil smooth HDPE geomembrane at the bottom of the main vadose trench extends below the pumping sump to provide continuous containment. Two-inch-thick HDPE flatstock was provided as impact protection above the 80-mil geomembrane.

A pump is currently installed through the vadose sideslope riser pipe and into the vadose sump. A bubbler level control device is also provided through the 8-inch pipe. The maximum operating liquid level is 1 foot above the top of the vadose trench at each sump. During the period of January 2001 to December 2007 no liquids have been detected in the vadose sumps.

The existing pumps and controls will be replaced with a system capable of reaching the extended length of the riser and providing a pumping capacity that will maintain the liquid level below the allowable limit.

During the Phase III construction, there will be a period of approximately 6 months between the demolition of the existing LCRS riser pads and the installation of the new riser pads. Since no liquids have been detected in the vadose zone in the last 10 years, monitoring of the three vadose sumps whose riser pads are to be removed will be suspended for the duration of the Phase III construction. Once the new riser pads are constructed, monitoring of the vadose zones will resume.

4.8.5 Leachate Storage

The following discussion of the leachate tank system provides information peripheral to the permitted unit's design, but supports the operational needs for understanding. Sheets 22 and 22A in Appendix A.1 show the existing top-of-slope riser pads above each sump area where liquids from the three (primary, secondary, and vadose) collection zones are handled. Flows from each pumping zone are monitored individually by a totalizer on each of the four (one primary, two secondary, and one vadose) pump discharge pipes. These pipes are then individually discharged into the top portion of a 6,000-gallon, double-walled, HDPE tank.

The double-walled tank is provided with a centrifugal pump for transferring liquids into vacuum trucks. The removed liquids are treated in on-site treatment facilities located away from B-18.

A curbed concrete slab (i.e., a concrete riser pad) is provided at each pump collection and storage tank area to contain any spilled liquids. A collection sump is provided in each concrete slab for removal of spilled liquids or rainwater.

As discussed in Section 4.8.1, three of the existing concrete riser pads (the pads for Areas IA, IB, and IIB) will be removed during the construction of Phase III. Three new riser pads will be constructed as replacements for the three pads to be removed. These new riser pads will be very similar to the existing ones and it is anticipated that the existing HDPE tanks and appurtenances will be salvaged and re-used for the new system or replaced with similar components.

4.9 Surface Water Control

The B-18 design includes surface water control features for the following conditions described in the indicated sections:

- Section 4.9.1 - Offsite Diversion of Run-on
- Section 4.9.2 - Active Area Run-off Control for Phase I Operations
- Section 4.9.3 - Phase I Direct Rainfall Control
- Section 4.9.4 - Active Area Run-off Control for Phase II Operations
- Section 4.9.5 - Phase II Direct Rainfall Control
- Section 4.9.6 - Run-off Control During the Above Grade Filling Period
- Section 4.9.7 - Phase III Direct Rainfall Control
- Section 4.9.8 - Run-off Control at Closure

Run-on is surface water that originates outside of the B-18 limits and it can therefore be discharged into natural stream channels. Active area run-off is surface water that flows from roads used for waste truck and/or landfill equipment access or from portions of the landfill with intermediate cover. This run-off does not come into direct contract with the waste disposal area.

The primary surface water design criteria for B-18 are the following:

- Precipitation falling directly into the disposal area must be contained within the waste prism. Resulting ponding on the waste surface must be managed as stipulated in the existing KHF permits.

- Active area run-off must be contained in surface water retention (i.e., containment) basins.

The permanent surface water controls that are used for conveying peak flows are designed based on rainfall intensity and duration relationships for the PMP storm event. The 24-hour PMP rainfall event for the site is 10.3 inches as reported by the National Oceanic and Atmospheric Administration (NOAA, 1998). The original design of B-18 (ESI, 1990a), used a 24-hour PMP storm event for the site of 7.4 inches, based on data from the California Department of Water Resources (CDWR, 1976). The CDWR used a statistical method, based on historic rainfall data, to determine the PMP rainfall depth. The CDWR no longer publishes data pertaining to PMP events as they apparently now defer to the PMP data provided by NOAA. The NOAA value is "theoretically, the greatest depth of precipitation for a given duration that is physically possible over a given storm area at a particular geographical location at a certain time of the year." Hence, the current design PMP storm event of 10.3 inches has increased substantially from the 7.4 inches used in the original design.

4.9.1 Off-site Diversion of Run-On

Sheets 5 and 10 in Appendix A.1 show the existing configuration for diverting run-on away from the B-18 area. This is accomplished by providing run-on collection ditches outside of the entire existing B-18 Perimeter Road that are capable of diverting run-on from the PMP storm event. These run-on collection ditches begin near the northwest corner of Phase I. One of these ditches diverts run-on flows along the outside of the B-18 Perimeter Road along the north side of the existing landfill. The other ditch diverts run-on flows along the outside of the B-18 Perimeter Road along the west and south sides of the existing landfill. These V-shaped run-on collection ditches are 5-feet-wide, earthen, and vary in depth from one to three feet. Sheets 5 and 10 in Appendix A.1 show the locations where changes in the depths of these ditches occur.

A buried culvert is used to convey flows from the southern-most run-on collection ditch under the B-18 Stockpile access road. This culvert is a 24-inch diameter corrugated metal pipe (CMP) that is capable of conveying the 25-year storm event. Larger storms can be safely conveyed over a swale at this road intersection. This swale is graded to maintain the entire surface water flow from the PMP storm event in the run-on collection ditch system.

Upon the completion of the Phase I construction (Sheet 5 in Appendix A.1), the run-on collection ditches were discharged into the natural drainage channel at the northeast corner of the B-18 area, near the limit of the Phases I and II clay mixing area. Upon the completion of Phase II (Sheet 10 in Appendix A.1), the run-on collection ditches were directed to an existing culvert under the KHF main access road. That culvert is capable of conveying flows from most of the storms that may be expected

to occur during the B-18 operations. Surface water from large, infrequent storms is allowed to flow across the KHF main access road with no potential for drainage to the existing waste disposal areas.

The final elements of the existing run-on collection ditch system are smaller V-ditches (brow ditches) along the tops of the existing B-18 Perimeter Road cuts. These brow ditches divert run-on from the natural hill slopes into drop inlets that discharge into the collection ditches that run along the outside

of the B-18 Perimeter Road. Due to the relatively steep slopes along the tops of these road cuts, the brow ditches are lined with asphalt to reduce erosion. The locations of the brow ditches and drop inlets are shown on Sheets 5 and 10 in Appendix A.1.

During the Phase III construction, most of the existing collection ditches along the outside edge of the existing B-18 Perimeter Road as well as the smaller brow ditches at the tops of the existing B-18 Perimeter Road cuts on the north, south, and west sides of B-18 will be removed and relocated to accommodate the expanded landfill footprint. The new collection ditches and brow ditches generally have a similar or increased capacity design as those of the existing ditches and they will be located along the outside edge of the new B-18 Perimeter Road (collection ditches) and at the tops of the cuts along the new B-18 Perimeter Road (brow ditches), as shown on Sheet C-3 in Appendix A.2. The permanent stormwater controls for B-18 are designed to convey the design PMP storm event. To be conservative, the stormwater controls were designed to convey the flow of the 6-hour PMP. The 6-hour PMP has a higher rainfall intensity and therefore greater peak flow than the 24-hour PMP event. During a PMP storm event, the drainage channels will reach capacity and some flows extend into the roadway. This design approach maintains reasonable size channels that convey a majority of storm events, however, flows from the PMP are controlled within the roadway assuming trafficable conditions would not need to be maintained during such an event.

During operation of Phase IIIA (while Phase IIIB is being constructed), there will be no potential for stormwater run-on into Phase IIIA from the adjacent portion of Phase IIIB due to the presence of the lined temporary Phase IIIA stormwater containment berm that will be constructed along the Phases IIIA-IIIB interface (see Detail 1 on Sheet C-4A in Appendix A.2). The top of this berm will be approximately 8 feet higher than the nearby high point on the Phase IIIB “bench liner” area, meaning that stormwater run-on from Phase IIIB will flow to the existing Northeast Containment Basin before it could overtop the temporary Phase IIIA berm. It is estimated that a maximum of approximately 2 feet of stormwater run-on could accumulate on the south side of the temporary Phase IIIA berm; this stormwater would be clean and would therefore be pumped by site personnel to just south of the nearby high point, where it could then flow to the Northeast Containment Basin. Once Phase IIIB is completed, the need for conveying stormwater run-on will be eliminated.

4.9.2 Active Area Run-Off Control for Phase I Operations

Sheets 5 and 6 in Appendix A.1 shows how surface water run-off from active areas for the PMP storm event was controlled during the Phase I disposal operations. The main aspects of this control were:

- Active area roads were sloped inward to a ditch system that was separate from the run-on ditch system described in Section 4.9.1.
- The active area run-off was directed to a lined retention basin located in the bottom of the initial clay borrow pit.

The west and south portions of the B-18 Perimeter Road adjacent to Phase I slope inward to an asphalt-lined V-ditch as shown in Section A-3,8/15 on Sheet 15 in Appendix A.1. This ditch flows in

the south and east direction to a former culvert inlet on the south portion of the B-18 Perimeter Road approximately 12.5 feet beyond the limit of the sideslope liner system for Phase I. The former culvert was an 18-inch diameter CMP that dropped down the south sideslope (see Section B-4,5/16 on Sheet 16 in Appendix A.1) and into an 18-inch diameter, concrete-encased CMP culvert beneath the entrance of the Phase I/II Berm road to the Phase I access ramp. This culvert then discharged into a V-ditch (see Section B-5,15,23/17 on Sheet 17 in Appendix A.1) that flowed along the west side of the Phase I/II Berm to a drop inlet. This drop inlet discharged into a 30-inch diameter CMP that passed beneath the Phase I/II Berm crest and down its eastern embankment slope and into the lined basin in the clay borrow pit. The V-ditch and culvert also collected run-off from the road on top of the Phase I/II Berm, which sloped at two percent toward the west for its entire length.

An additional concrete-encased, 12-inch diameter CMP culvert was provided where the Phase I access ramp met the west portion of the existing B-18 Perimeter Road. This culvert drained the small flow from the northern part of this portion of the B-18 Perimeter Road and was concrete-encased to support waste truck traffic.

The north portion of the B-18 Perimeter Road adjacent to Phase I and the access road around the top of the initial clay borrow pit were also considered to have active area run-off because of the daily cover and landfill equipment that used these roads. Hence, these roads were also sloped toward an inside ditch that conveyed surface water flows to an 18-inch diameter CMP culvert along the southeast corner of the initial clay pit slope.

The clay pit retention basin was lined with 40-mil HDPE geomembrane from a bench at an elevation of 720 feet to its base at an elevation of 680 feet. Although not normally required for retention basins of this type, the liner was provided to avoid saturation of the clay borrow area because this area would later form part of Phase II floor. The retention basin and its liner system were completely removed when Phase II was constructed. The capacity of this temporary containment basin was sufficient to contain the 24-hour PMP run-off from the areas discussed above. Section A on Sheet 13 in Appendix A.1 shows a cross-section through this retention basin.

4.9.3 Phase I Direct Rainfall Control

The control of direct rainfall into the Phase I disposal area consisted of sloping the waste surface toward a low spot where the collected surface water was removed by vacuum trucks (or other appropriate means) and transported to the appropriate storage, treatment, and/or disposal facilities.

The amount of water that was handled by vacuum trucks was reduced by installing slope gutters on the temporary liner portions of the slopes. Water collected in these gutters drained into a clean tank and was then pumped into the run-on collection ditch system discussed in Section 4.9.1.

Initially, the low spot on the waste surface was located in the northern portion of Phase I to minimize interference with traffic at the Phase I access ramp. As the waste level in Phase I rose, the low spot was moved to best suit operational conditions. Once the waste height rose above the top of the Phase I/II Berm, it was necessary to keep the low spot away from the east-facing Phase I waste slope to avoid localized instability of this slope.

4.9.4 Active Area Run-Off Control for Phase II Operations

Sheet 10 in Appendix A.1 shows the configuration that was used for containing active area run-off during the Phase II operations. The collection system for this period consisted of an extension of the

Phase I ditches located along the inside of the B-18 Perimeter Road as described in Section 4.9.2. These ditches sloped toward the east-northeast portion of Phase II and conveyed run-off into the lined Northeast Containment Basin that was constructed in conjunction with Phase II. This basin is sized to contain run-off from both the existing B-18 Perimeter Road and from waste slopes with interim cover that are located above the level of the existing B-18 Perimeter Road. Surface water from the inside ditches flows into either of two culverts (one culvert is a 30-inch diameter CMP while the other culvert is a 24-inch diameter CMP) at the eastern portion of the B-18 Perimeter Road above the Northeast Containment Basin. These culverts discharge into asphalt-lined swales in the corners of the Northeast Containment Basin. For very large storms, the roadway profile above the culverts is depressed to direct flows across the road and into the asphalt-lined containment basin swales.

4.9.5 Phase II Direct Rainfall Control

Direct rainfall into the Phase II waste disposal area was handled using procedures similar to those described in Section 4.9.3 for the Phase I operations.

4.9.6 Run-Off Control During the Above Grade Filling Period

Currently, the B-18 waste mass is in the above grade filling period. As the waste mass rose above the level of the existing B-18 Perimeter Road, rainfall run-off began to be handled in the following two ways:

- The top deck area, where waste disposal is ongoing, is graded to direct surface water toward a low spot (with temporary earthen berms, as needed) where the collected surface water is removed as discussed for the Phase I and II areas in Sections 4.9.3 and 4.9.5, respectively.

- Surface water from sloped areas with interim cover is handled as run-off water. This run-off is collected from the sideslopes of the existing landfill and conveyed into the inside B-18 Perimeter Road ditches, which ultimately direct the run-off into the Northeast Containment Basin.

The asphalt-paved ditches on the inside of the existing B-18 Perimeter Road contain run-off from storms with recurrence intervals less than 100 years (based on original design storm event). During larger storms and up to the PMP event, the flow may extend outside of these paved ditches but is maintained within the larger channel formed by the inward-sloping perimeter road. Flows up to the PMP storm condition are currently directed into the Northeast Containment Basin. During operations within Phase IIIA (while Phase IIIB is still being constructed), stormwater run-off from the north, east, and west facing Phase IIIA waste slopes will be diverted to the Northeast Containment Basin. A temporary lined containment berm will be constructed at the interface of Phases IIIA and IIIB (see Sheet C-4A in Appendix A.2) to provide containment of stormwater run-off from the lower portions of the south-facing temporary Phase IIIA waste slope. This temporary berm has been sized to contain the entire run-off volume generated by the 24-hour PMP event (while maintaining a freeboard of greater than 1 foot); this run-off will be contained on the north side of the temporary berm. Therefore, no pumping will be required to prevent the overtopping of the temporary berm. Site personnel will treat the stormwater run-off contained on the north side of the temporary berm as leachate. Accordingly, the impounded stormwater run-off will be pumped into tanker trucks and transported to the appropriate on-site facility for treatment as leachate.

The Phase III expansion of B-18 will involve the construction of a new B-18 Perimeter Road around most of the landfill. Similar to the existing configuration, the new B-18 Perimeter Road will be sloped inward toward the landfill during the above grade filling period. Also, asphalt or shotcrete-lined V-ditches will run along the entire inside edge of the new B-18 Perimeter Road to convey run-off to containment basins in a similar manner as is currently done.

Due to the change in the PMP storm event (see the discussion at the beginning of Section 4.9) and the increased landfill area after the construction of Phase III, a second stormwater basin, the South Containment Basin, will be required to provide adequate run-off storage capacity for the PMP storm event after Phase III is completed. The new South Containment Basin will be constructed as part of the Phase IIIB expansion as shown on Sheet C-3 in Appendix A.2. The surface water flow patterns around the expanded landfill will be such that all run-off will be directed to and contained in the South and Northeast Containment Basins.

4.9.7 Phase III Direct Rainfall Control

Direct rainfall into the Phase III waste disposal area will be handled using procedures similar to those described in Section 4.9.3 for the Phase I operations.

4.9.8 Run-Off Control at Closure

The surface water drainage capacities of the final benches, Cover Access Road, and the new B-18 Perimeter Road are designed to be capable of conveying the 6-hour PMP storm water run-off without damage to the landfill. This storm water flow will drain to either the existing Northeast Containment Basin or the new South Containment Basin, as shown on Sheet C-4 in Appendix A.2. The stormwater containment basins are designed to contain the runoff from the 24-hour PMP storm event. After site closure, the basins may be modified to release the stormwater run-off in a controlled manner.

4.10 Utilities

The existing utilities for B-18 consist of the following:

- A perimeter lighting system that currently serves the existing B-18 Perimeter Road and that formerly served the waste disposal area before the waste mass grew to its current level above the lights.
- Electrical power to operate the various leachate control pumps and the perimeter lighting system.

As part of the B-18 Phases I and II construction, permanent lighting fixtures were placed around the entire existing B-18 Perimeter Road as illustrated in Section A-5,10/23 on Sheet 23 in Appendix A.1. However, this existing lighting system is no longer needed and it will be removed as part of the Phase III construction.

Power for the existing lighting system and leachate pumps is provided by an electrical transformer located along the northern boundary of B-18, as shown on Sheet C-2 in Appendix A.2. During the Phase III construction, this transformer will be removed and relocated to allow for the construction of the Phase III expansion.

5.0 ENGINEERING ANALYSES

5.1 General

This section describes the engineering analyses that were performed to support the design of B-18. The B-18 analyses are discussed in Sections 5.2 through 5.9 with calculations provided in Appendices G through N for Settlement, Stability, Cover Soil Erosion, Surface water drainage, leachate collection and removal, riser pipes, cover infiltration, and frost and biotic protection, respectively.

5.2 Settlement Analyses

5.2.1 Conditions Evaluated

The following four settlement conditions were analyzed for B-18:

1. The elastic settlement of the landfill's foundation to assess the minimum slope of the LCRSs. Ideally, the post-settlement slopes of the LCRSs should be maintained at 2 percent or greater.
2. Evaluation of the degree of consolidation of the primary and secondary clay liners at interim and final closure to assign appropriate strength properties for these materials for use in the stability analyses.
3. Estimation of the magnitude of settlements of the primary and secondary clay liners to assure that adequate thicknesses (i.e., 3 feet or more) will be maintained after compression and consolidation of the liners are completed.
4. Estimation of the post-closure waste settlements to assure positive drainage of the cover, benches, and the Cover Access Road.

The settlement calculations are provided in Appendix G and described in the following sections.

5.2.2 Foundation Settlement

The rock strata beneath B-18 have and will continue to settle in an approximately elastic manner as each layer of waste is placed. Therefore, essentially all of the foundation settlement is anticipated to occur prior to closure and, hence, foundation settlement will not be a factor with regard to the closure cover configuration or drainage control. The foundation settlement will, however, result in slope changes at the base of the landfill that will impact the LCRS. These settlements will not cause abrupt changes on the base (e.g., large differential settlements over short distances), but they could potentially reduce the slopes of the LCRS to below the desired minimum of 2 percent. A minimum LCRS slope of 2 percent is desired to assure positive leachate drainage to the sumps.

The foundation settlement estimates presented in Appendix G.1 were calculated using Boussinesq's stress distribution theory in conjunction with the elastic properties of the foundation materials. Elastic properties of the claystone and siltstone were conservatively considered to be the same and were estimated based on the measured plasticity index and strength characteristics of the undisturbed claystone samples using the procedure described by Duncan and Buchignani (1976). The elastic modulus and Poisson's ratio of these fine-grained rocks were estimated to be 6,000 ksf and 0.38, respectively. The sandstone was assumed to be incompressible since information obtained from the boring logs and laboratory test data indicate that deformation of the coarse-grained rock strata (i.e., sandstone) would be negligible in comparison to that of the claystone and siltstone.

In Appendix G.1, landfill base settlement profiles are estimated for four representative cross-sections through the waste mass to evaluate both the total and differential settlements. The total estimated settlements range up to about 15 inches. According to the computed settlements, the resulting slope of the base of the landfill will be greater than 2 percent. Hence, it is concluded that the LCRS will remain sloped at a minimum of 2 percent.

5.2.3 Degree of Consolidation of Clay Liners

Calculations of the time rate of consolidation of the clay liners due to placement of the overlying waste are provided in Appendix G.2. These calculations were carried out in order to evaluate the appropriate clay strength parameters for use in the stability analyses. Shortly after clay liner construction, UU triaxial strength parameters are appropriate. After "complete" consolidation (i.e., an average degree of consolidation greater than about 95 percent) of the clay due to the weight of the overlying waste has occurred, CU triaxial strength parameters are appropriate. The selection of clay strength parameters to assess stability at a certain stage of waste disposal is based on the expected degree of consolidation of the clay liner at that particular time.

The clay liners will be continually consolidating during waste placement such that the undrained shear strength of the clay liners will be continually increasing. Based on the consolidation test data for compacted (remolded) clay discussed in Section 3.5.7, it is estimated that the average degree of consolidation for both the primary and secondary clay liners will be greater than 95 percent by the time the final closure cover is installed. Therefore, the undrained shear strength of the clay liners for long-term, post-closure conditions should be based on its fully-consolidated strength as evaluated from the CU triaxial tests.

Another calculation was performed to evaluate the degree of consolidation of the clay liners when the Phase I Intermediate Closure was completed about two years after initial clay liner construction. This calculation shows that approximately 30 percent of clay consolidation will have occurred at that time, resulting in a modest gain in strength. To be conservative, the UU triaxial test data were used in the stability analyses for the Phase I Intermediate Closure condition. Also, because all of the excess porewater pressures were not dissipated at that time, the friction angle (ϕ) portion of the UU triaxial test data was not relied upon in the Phase I Intermediate Closure stability evaluation.

5.2.4 Magnitude of Clay Liner Consolidation

The clay liner consolidation calculations in Appendix G.3 were performed to estimate the clay liner thickness reductions once "complete" consolidation of the clay had occurred under the weight of the overlying waste mass. Calculations are provided for the following:

1. A primary and secondary clay liner thickness of 1.5 feet and 3.5 feet, respectively, at the landfill base and a maximum waste thickness of about 300 feet (compared to a maximum waste thickness of 230 feet without Phase III).

2. A clay liner thickness of 5 feet beneath the vertical riser pipe, where additional load will be applied to the clay liner by the vertical riser pipe foundation (see Section 5.7).

The calculations show that the maximum consolidation settlement of the primary and secondary clay liners at the landfill base will be approximately 0.1 and 0.3 feet, respectively, under loading from 300 feet of waste (similar to the result for Phase I and II). Therefore, it is concluded that the primary and

secondary clay liner's required minimum thickness of 1 and 3 feet, respectively, will be maintained after consolidation settlement is complete.

The maximum primary and secondary clay liner settlements beneath the vertical riser pipe foundation were calculated to be about 0.4 and 0.7 feet, respectively, under loading from 300 feet of waste. These magnitudes of settlement are acceptable because the original clay liner thicknesses in this area were about 3 and 5 feet for the primary and secondary clay liners, respectively.

5.2.5 Post-Closure Waste Settlement

Appendix G.4 contains the post-closure waste settlement calculations that were performed in order to assess the minimum closure cover slopes for positive drainage. It was assumed that the primary consolidation of waste material will fully occur during waste placement. Therefore, the post-closure waste settlements were estimated based on the fo1lowing factors:

- Crushing of disposed drums and the related settlements due to:

  Closure of void spaces in the drums within the waste mass; and

  Consolidation of loosely-placed materials in the drums.

- Secondary consolidation (or creep) settlement of the main, soil-like waste matrix.

It is anticipated that settlement of waste within B-18 will be less than prior WMUs at the KHF because of more restrictive disposal regulations, especially since a significant portion of the waste has been and will continue to be solidified prior to disposal. The waste settlement estimates discussed below do not fully account for these changes and are therefore considered to be conservative.

The waste settlement calculations were based on KHF's estimate that 15 percent of the waste volume within B-18 consists of drums that are distributed randomly throughout the waste. The drums are conservatively assumed to contain 10 percent voids and the wastes within the drums were calculated to consolidate an additional 30 percent. Long-term creep settlement of the waste was estimated based on characteristics for normally-consolidated soft to medium stiff clay.

By analyzing representative cross-sections through the entire B-18 waste mass, it is estimated that the post-closure waste settlements will vary from approximately zero at the edge of the landfill to a maximum of about 27 feet where the waste thickness is greatest. On the basis of the calculations in Appendix G.4, it is concluded that the proposed final cover grades shown on Sheet C-4 in Appendix A.2 will be adequate to assure appropriate drainage after waste settlement is complete.

It is recommended that survey monuments be monitored at areas with interim cover on the landfill sideslopes once these areas are filled to their final elevation. As this settlement data is obtained, it may be appropriate to re-evaluate whether or not a portion of the primary settlement may occur during the post-closure period and to re-assess the final cover grading plan.

5.3 Stability Analyses

5.3.1 General

Appendix H includes static and seismic slope stability analyses for the conditions discussed in the following sections:

- Section 5.3.2 - Temporary Rock Cut Slopes;
- Section 5.3.3 - Compacted Fill Slopes;
- Section 5.3.4 - Temporary Phase I Intermediate Fill Slopes;
- Section 5.3.5 - Temporary Phase IIIA Intermediate Fill Slope; and
- Section 5.3.6 - Final Closure Conditions.

Table 5.1 summarizes the shear strength and unit weight parameters assigned to each of the materials and material interfaces modeled in the stability analyses. The right-hand column in Table 5.1 identifies the data sources for these parameters, which include the site-specific laboratory testing discussed in Section 3, prior KHF landfill investigations, and published data and information.

Table 5.2 summarizes the results of the stability analyses and lists the criteria used to evaluate acceptability. For static conditions, the minimum acceptable factor of safety is considered to be 1.5. This criterion was satisfied for all of the critical conditions analyzed. For seismic conditions, acceptability is evaluated based on a design displacement during the MCE event (see Section 2.5.1). A maximum design displacement of 6 inches was established for all cases where the failure plane could intersect the base liner system, based on the recommendations of Seed and Bonaparte (1992). This minimizes the potential for large displacements that could potentially disrupt the HDPE geomembrane/clay composite liner or the LCRS. As shown in Table 5.2, the estimated displacement for most of the cases considered is less than 1 inch for the applicable MCE event.

The following seismic displacements during the MCE event were considered maximum acceptable values for locations where permanent liner systems are not affected:

- A 6-inch seismic displacement for the Northeast Containment Basin embankment, primarily to minimize the potential of overall embankment instability occurring.
- A 12-inch displacement entirely within the waste mass for the Phase I intermediate fill slope because it is temporary and regrading improvements could easily be made without exposing the underlying liner systems.
- A 12-inch displacement would be allowable for the geotextile/HDPE geomembrane and vegetative cover soil/geotextile interfaces of the final cover system. This maximum displacement minimizes the potential for damage to the HDPE geomembrane and any resulting near-surface cracking could be repaired relatively easily.

The static factors of safety were calculated using:

1. The STABL5 computer program, developed by Purdue University (1986), for the stability analyses performed during the original design of B-18 (ESI, 1990a).
2. The computer program GSTABL7 version 2.003, developed by Gregory Geotechnical Software, for the stability analyses performed for the current design.

Displacement estimates for the seismic conditions were calculated using the following procedures:

1. Newmark (1965), as modified by Franklin and Chang (1977), for temporary rock cut slopes, compacted fill slopes, and the Phase I intermediate waste fill conditions. A conservative velocity-to-acceleration ratio of 30 was used for the Newmark and Franklin and Chang methods for these cases, based on measured velocity/acceleration ratios published by Donovan (1983). The maximum acceleration of the waste mass for these cases was assumed to be 80 percent of the PHGA, based on comparisons with site response analyses performed by Woodward-Clyde (1987) for the OII Landfill in Monterey Park, California. However, no attenuation was allowed for very shallow potential failure surfaces because the lower portions of the slide mass were close to the ground surface.
2. Makdisi and Seed (1978), which is based on the Newmark (1965) method, for the final closure configuration. In order to evaluate the average acceleration time histories of the critical slide mass, two-dimensional dynamic finite element analyses were performed using the computer program QUAD4M (Hudson et al., 1994).
3. Bray et al. (1998) for the final cover veneer stability analyses. Two-dimensional site response analyses were also performed for this case using QUAD4M to evaluate the average acceleration of the cover system.

The PHGAs corresponding to the MCE events (near-field and far-field) used in the current design are discussed in Section 2.5.3. These PHGAs and MCEs were used in the stability analyses of the final closure configuration and in the cover veneer analyses. During the original design of B-18 (ESI, 1990a), a single MCE event was considered, as discussed in Section 2.5.2. This MCE event and its associated PHGA differ from those used in the current design due to advances in geotechnical earthquake engineering since the time of the original design. The temporary rock cut slopes, compacted fill slopes, and Phase I intermediate waste fill conditions were not re-analyzed for the current design since these conditions no longer exist due to the placement of overlying waste. The rock cut slope on the west side of B-18 will be flattened and will be more stable than the current configuration. Hence, the computed seismic displacements for these conditions correspond to the MCE event from the original design (ESI, 1990a), as discussed in Section 2.5.2.

5.3.2 Temporary Rock Cut Slopes

Appendix H.2 provides stability analyses for the two rock cut slope conditions illustrated in Figure 5.1. East-, south-, and north-facing slopes were excavated at a 2H:1V inclination across bedding planes. West-facing slopes, which were in the general direction of the weaker bedding planes, were excavated at a shallower 3H:1V inclination. Table 5.2 shows that the static factors of safety for these two conditions are 2.4 and 2.2 for the 2H:1V and 3H:1V slopes, respectively. Table 5.2 also shows that essentially no displacement of either slope configuration is anticipated during the original design's MCE event (Section 2.5.2). Therefore, the designed temporary rock cut slopes were adequate for both the static and seismic criteria for Phases I and II and are considered to be adequate for the comparatively minor proposed Phase III temporary rock cut slopes.

5.3.3 Compacted Fill Slopes

Phases I and II of B-18 were formed primarily by excavation into rock and there were no large fill embankments required. A relatively small fill embankment was necessary in the former natural drainage area at the northeast edge of Phase II. This fill embankment forms a portion of the eastern Phase II sideslope and also forms one of the sideslopes for the Northeast Containment Basin. A cross-section through this embankment is shown in Section A-8/23 on Sheet 23 in Appendix A.1. Stability analyses were performed for this fill embankment.

Table 5.2 shows that the Phase II/Northeast Containment Basin fill embankment is satisfactory for both static and seismic conditions. The static factor of safety for this embankment is 2.2 and essentially no displacement was projected for this slope during the original design's MCE event (Section 2.5.2). Appendix H.3 contains the detailed stability computations for this condition.

Phase III will require some relatively small fill embankments along the south side of the existing landfill. The slopes of these fill embankments will have inclinations equal to or less than those of the existing Phase II/Northeast Containment Basin fill embankment. Based on the stability analysis results for the Phase II/Northeast Containment Basin fill embankment, the proposed fill embankments for Phase III are considered to be adequately stable for both the static and seismic conditions.

5.3.4 Temporary Phase I Intermediate Fill Slopes

The typical configuration of the temporary Phase I intermediate waste fill slopes is shown in Section A-6/23 on Sheet 23 in Appendix A.1. This slope condition existed from the time that initial filling of Phase I was completed until the Phase II area was filled to the top of the Phase I/II Berm. This temporary waste slope was constructed at the relatively steep 1.5H:1V inclination because the stability analyses showed that the factor of safety along the critical liner interface is higher when the waste slope is steeper due to the added frictional resistance along the toe portion of the potential sliding wedge.

Appendix H.4 presents the detailed results of the stability analyses conducted for the Phase I intermediate fill slopes. Figure 5.2 illustrates the various potential failure mechanisms considered for the Phase I temporary fill slopes. The most important potential sliding mechanism (Cases A1 and A2 in Figure 5.2) involved shearing through the waste, along the base liner of the landfill, and up the lined Phase I/II Berm sideslope, resulting in a wedge-shaped slide mass. This mechanism is critical because the liner interface strengths are much lower than those of the underlying foundation or the overlying waste materials. Also, a failure at this location could damage the liner and LCRSs. Figure 5.2 shows the following two potential shear surfaces for this sliding mechanism:

- Case Al: A shear surface that would occur entirely along the liner interface, including at the intersection of the landfill base and the Phase I/II Berm sideslope.

- Case A2: A shear surface that would include an approximately 50-foot-long diagonal shear zone through the waste near the base/berm intersection to simulate a circular failure surface across that intersection angle.

The results in Table 5.2 for these cases show a static factor of safety of at least 2.5 and essentially no displacement during the original design's MCE event (Section 2.5.2). These results are acceptable for this important stability condition.

The potential for a shallow circular failure (Case B in Figure 5.2) to occur through the intermediate fill slope without intersecting the liner system was an additional consideration for the Phase I temporary waste slope. This condition was less important than Cases Al or A2 because, if some movement were to occur along the potential sliding surface, the underlying liner system would not be damaged. Table 5.2 shows that the static factor of safety for Case B is approximately 1.5 and the estimated displacement for the original design's MCE event (Section 2.5.2) is about half an inch. These results are based on shallow waste strength parameters of ϕ = 27 degrees and c = 300 psf as compared to ϕ = 31 degrees and c = 0 used for deeper zones of the landfill. Use of the cohesionless waste strength parameters would result in a lower static factor of safety (approximately 1.3) for very shallow potential failure surfaces. However, it is reasonable to assume that the waste will have a small degree of cohesion. Therefore, the factor of safety and estimated MCE displacement discussed above were considered to be appropriate and the intermediate slope was acceptable for this temporary and non-critical condition.

During the preliminary stability analyses, calculations were also performed to evaluate whether a failure could potentially occur along the entire Phase I liner interface (i.e., along the west sideslope, the landfill base, and the Phase I/II Berm sideslope). The factor of safety for this condition was found to be much higher than for the Cases A1 and A2 conditions.

Finally, because of prior experience at the KHF, an evaluation was made as to whether or not it would be appropriate to analyze the Phase I intermediate fill slope for a three-dimensional potential failure surface. However, because of the designed configuration, it was concluded that such an analysis was not necessary. Phase I was purposely configured to avoid three-dimensional driving forces that were significantly different than those realized for the two-dimensional (Cases Al and A2) condition. Therefore, the factor of safety for a three-dimensional case would essentially be the same as the acceptable values for the two-dimensional cases discussed above.

5.3.5 Temporary Phase IIIA Intermediate Fill Slope

The configuration of the temporary Phase IIIA intermediate waste fill slope is shown on Sheet C-4A in Appendix A.2. As can been seen on Sheet C-4A, the Phase IIIA interim waste slope is a south-facing, 2H:1V slope that contains an approximately 30-foot wide bench to allow for access to the landfill during waste placement operations. This interim slope condition will exist for only a short time, if at all, as it is anticipated that no more than 6 months of waste will be placed in Phase IIIA before waste placement in Phase IIIB begins. After waste placement in Phase IIIB commences, the interim Phase IIIA waste slope will be covered with waste relatively quickly. The north, east, and west facing Phase IIIA waste slopes will be built to the final cover grades (i.e., 3.5H:1V inclination between benches); therefore, these waste slopes were analyzed as part of the final closure configuration (Section 5.3.6).

Appendix H.4 presents the detailed results of the stability analyses conducted for the south-facing 2H:1V Phase IIIA intermediate fill slope. Due to its temporary nature, only static stability analyses were conducted for the Phase IIIA intermediate waste slope. Also, as can be seen in Appendix H.4, the shear strength parameters of the Phase IIIA liner system were assumed to lie between its peak and residual values since the slope will be temporary and the portion of the Phase IIIA liner modeled in the stability analyses will be graded at only 10 percent, thereby making it similar to a base liner from a shear strength standpoint.

Figure 5.3 illustrates the potential failure mechanism considered for the south-facing 2H:1V Phase IIIA temporary fill slope. This critical potential sliding mechanism involves shearing down through

the waste and then along the Phase IIIA liner toward the toe of the south-facing 2H:1V temporary waste slope, resulting in a wedge-shaped slide mass. This mechanism is critical because the liner interface shear strength is much lower than that of the underlying foundation or the overlying waste materials. The result in Table 5.2 for this interim case indicates a static factor of safety of 1.5, which is considered acceptable for this temporary condition.

The potential for a shallow circular failure to occur through the south-facing 2H:1V Phase IIIA intermediate fill slope without intersecting the liner system was not performed since analyses for the Phase I temporary waste slopes indicated that a temporary waste fill slope inclined at 1.5H:1V would be adequately stable (Section 5.3.4) with respect to this potential sliding mechanism. Since the Phase IIIA intermediate waste fill slope will be inclined at 2H:1V, it will be expected to be more stable than the Phase I temporary waste slopes were. In addition, the Phase I temporary waste slopes were observed to perform well after their construction, which further demonstrates that the 2H:1V Phase IIIA intermediate fill slope should be adequately stable with respect to a circular failure entirely through waste.

5.3.6 Final Closure Conditions

The three potential failure scenarios of importance for the final closure configuration, as shown conceptually on Figure 5.3, are:

- Displacement of the final cover system on the 3.5H:1V sideslopes between benches.
- A large wedge failure through the waste and along the base and perimeter sideslope liner systems.
- A circular failure entirely through the waste.

The stability analyses for the final closure conditions were performed by HAI, under subcontract to Golder, as part of the current design. Appendix H.5 contains HAI's slope stability report while the results of HAI's stability analyses are summarized in Table 5.2 and discussed in the following paragraphs.

The weakest interface in the B-18 final cover system is the nonwoven geotextile/40-mil textured HDPE geomembrane interface. Large seismic displacements at this interface could result in a tensile failure of the HDPE geomembrane. The current state-of-practice is to generally limit seismic displacements of the cover system to less than about 12 inches. Table 5.2 shows that the static factor of safety for veneer-type sliding of the cover system along the geotextile/HDPE geomembrane interface is 1.6, which is well above the allowable 1.5. The estimated seismic displacement along this interface was calculated to be about 2.7 inches for the updated MCE event (Section 2.5.3). Hence, the final cover system of B-18 is considered to be adequately stable under the design static and seismic loading. Minor repairs to the cover system may be required if the seismic displacement of the cover system exceeds about 2 to 3 inches.

In order to evaluate a deep, wedge-shaped failure mechanism (along the base liner) for the final closure condition, HAI evaluated 6 representative cross-sections (A-A' to F-F') through B-18. The locations of these 6 cross-sections are shown in Appendix H.5. For each cross-section, multiple sliding mechanisms were considered.

HAI reviewed the interface shear strength test data for B-18 to determine the weakest interface for each phase of construction. The original design report for B-18 (ESI, 1990) assumed the most critical interface would be located at the interface of the textured geomembrane and the geonet side of the geocomposite. The interface was found to have a residual friction angle of 9 degrees and 800 psf adhesion. However, during construction of Phase II, a bonded geotextile was included below the geocomposite to increase the shear strength of the liner system (i.e., a geocomposite with geotextiles heat bonded to both sides of the geonet was used during construction). Therefore, the lower shear strength properties used for the Phase I side slope liner were based on a different critical interface in the liner system. Based on the URS 2005 report, archived samples of the existing Phase II liner system components were recovered from storage and tested for interface shear strength. Two shear tests were performed on multiple components that represented all of the possible interfaces in the existing B-18 Phase II liner systems. Tests performed on these "sandwich-like" specimens of the Phase II liner system demonstrated that failure occurred along the clay/textured geomembrane interface in one test and the clay/geotextile interface in the other test. The results of these tests yielded a peak interface friction angle of 20 degrees with an adhesion of approximately 1,900 psf, and a residual interface friction angle of 19 degrees with approximately 1,800 psf adhesion. For the stability analysis the adhesion was conservatively considered to be 0 psf.

It should be noted that failure in a liner system occurs along the interface with the lowest peak shear strength. Therefore, the weakest interface in the liner system is the interface with the lowest peak strength. Based on the direct shear testing performed by SGI Testing Services, LLC (2003) on the archived Phase II liner system materials (see Appendix A of the URS 2005 report), the lowest peak shear strength is along either the textured geomembrane/compacted clay liner interface or the geotextile/compacted clay interface (peak friction angle of 20 degrees). Even though the clay liner has a lower residual friction angle of 13 degrees with lower adhesion (SGI, 2003), the higher internal peak shear strength of the clay (26 degrees) in comparison with the two above-mentioned interfaces (20 degrees) results in detrimental shear displacements occurring along either of the two geosynthetic liner/clay interfaces and not within the clay liner itself. The "sandwich-like" specimen tests presented in the URS 2005 report demonstrated this.

To confirm the friction angles for the B-18 Phase III liner system, similar "conformance" testing was completed on stock materials from geosynthetic manufacturer's that will supply materials for the Phase III expansion. Based on the testing conducted by Precision Geosynthetic Laboratories (results included in Appendix H.5), the weakest interface is the geocomposite to textured HDPE geomembrane. The measured peak friction angle was approximately 28 degrees with a residual friction angle of 12 degrees. For the stability analysis, the 12 degree residual friction angle was used for the Phase III expansion area. The initial stability report by HAI assumed that the friction angle for Phase III would be similar to Phase II. Based on the recent testing, HAI lowered the Phase III friction angle. An addendum to the original report is included in Appendix H.5 to address the stability analysis using the lower strength values.

Using the above interface shear strengths, Sections D-D' and F-F' were found to be the most critical of the 6 cross-sections considered since their potential failure mechanisms had the lowest static factor of safety and lowest yield accelerations. HAI therefore performed two-dimensional site response analyses (using QUAD4M) and seismic displacement analyses for sections D-D' and F-F'. In the two-dimensional response analyses, four different input ground motion time histories were used to provide a range of anticipated waste mass accelerations (both from the near-field and far-field events). The results of the seismic displacement analyses performed for section D-D' and F-F' indicate that, under the updated MCE ground motions (Section 2.5.3), the maximum seismic displacements will be less

than I inch. Therefore, the overall final landfill configuration is considered to be adequately stable with respect to deep, wedge-shaped failure mechanisms.

The last closure configuration condition analyzed was a circular failure entirely through the waste. Section D-D' contained the critical circular shear surface. As can be seen on Table 5.2, the calculated static factor of safety for this case was 2.2 and the seismic displacement during the updated MCE event (Section 2.5.3) is anticipated to be less than 12 inch. Hence, the final landfill configuration is considered to be acceptable with regards to static and seismic stability of circular shear surfaces through the waste.

Finally, the need for a three-dimensional stability analysis for the final landfill configuration was not deemed necessary since no conceivable three-dimensional movements could be hypothesized that would lead to significantly lower factors of safety than those for the two-dimensional cases discussed above.

5.4 Cover Soil Erosion

Appendix I contains the analyses performed to assess the erosion rate of the B-18 vegetative cover soil layer based on the planned vegetation, slope steepness, slope length, and climatological conditions. The soil erosion analyses were performed using the computer program RUSLE2 (NRCS, 2004), which uses the Revised Universal Soil Loss Equation (RUSLE) along with the appropriate site-specific parameters listed above to calculate the potential soil erosion loss in tons per acre per year.

The results of the soil erosion analyses indicate a maximum cover soil erosion rate of about 1 ton per acre per year for the typical 3.5H:1V closure slopes between benches. This maximum rate is approximately half of the maximum allowable rate of 2 tons per acre per year suggested by the USEPA (1989). This cover soil erosion rate assumes that vegetation is established on the final cover. If no vegetation (i.e., bare ground) is assumed, the calculated cover soil erosion rate is about 9 tons per acre per year. Hence, it will be important to establish and maintain an acceptable amount of vegetation on the B-18 final cover to control soil erosion losses. As such, a specification for revegetation of the final landfill slopes is provided for in Appendix O.

5.5 Surface Water Drainage

5.5.1 General

The surface water analyses presented in Appendix J are divided into sections that address the following requirements:

- The general hydrology and design criteria.
- Phases I and II run-on control.
- Phases I and II run-off control.
- Phase III run-on control.
- Phase III run-off control.

- Closure drainage control.

The B-18 surface water drainage control systems are described in Section 4.9. As discussed in Section 4.9, run-on control refers to the collection and off-site diversion of surface water that has originated outside of the B-18 limits and, therefore, has not been affected by the active disposal area. Run-off control refers to the collection of surface water from roads used for waste truck or landfill equipment access and from portions of the waste area with intermediate cover. This run-off does not come into direct contact with the waste disposal area, but it will be retained on-site in accordance with the KHF permitting requirements.

The basic surface water drainage control design criteria for B-18 are:

- Hydraulic structures should be capable of conveying the 6-hour PMP storm event that is considered to have maximum intensities based on the rainfall-duration curve data shown Appendix J. Culverts may be designed for flows as low as those from the 25-year storm event as long as water overtopping the culvert is contained within a drainage control system that has been designed for the peak PMP discharge.

- All run-off containment basins, except the temporary one for the Phase I intermediate condition, are designed for the 24-hour PMP event, which is 10.3 inches of rainfall (NOAA, 1998). It is noted that during the original design of B-18 (ESI, 1990a), the 24-hour PMP event used in the analyses was only 7.4 inches based on the available data at that time (CDWR, 1976). The temporary Phase I intermediate basin was required only during the Phase II construction period and was designed for the 25-year, 24-hour storm.

- During the original design of B-18 (ESI, 1990a), peak storm run-offs were calculated using the rational method for small watersheds along with run-off coefficients of 0.40 for natural areas, 0.90 for roads, and 0.60 for interior cover areas. Volume requirements for the former and existing containment basins were also based on these run-off coefficients. These methods and parameters are applicable for the Phases I and II run-on and run-off calculations presented in Appendices J.2 and J.3.

- For the current design, peak storm run-offs were calculated using the computer program HEC-HMS version 3.1.0, developed by the United States Army Corps of Engineers. In the HEC-HMS analyses, an SCS curve number of 81 was used to model the final cover soil and areas surrounding B-18, except a SCS curve number of 74 was used for the natural terrain west of B-18. The volume requirement for the proposed South Containment Basin was based on the design flows calculated from the HEC-HMS analyses. These methods and parameters are applicable for the closure drainage control calculations presented in Appendix J.4.

- In the original B-18 design (ESI, 1990a), Manning's roughness coefficients used in the design of ditches and culverts were 0.013 for smooth asphalt, 0.018 for earth channels (which extend onto roadway slopes), and 0.019 for CMP. In the current design for the closure condition, the following Manning's roughness coefficients (i.e., "n" values) were used:

Channel Lining	Manning's n for Stability	Manning's n for Capacity
Grass	0.030	0.035
Turf Reinforcement Mat	0.030	0.035
Rip-rap	0.035	0.040
Asphalt or Shotcrete	0.016	0.016

5.5.2 Phases I and II Run-On Control

Figure 5.5 shows individual water shed areas that contribute run-on flow to the diversion ditches located along the outside edge of the B-18 Perimeter Road. These ditches intercept the run-on and direct it away from the B-18 active area. Table 5.3 summarizes the previously calculated maximum flows for Phase I and II during the PMP event at key locations along the diversion ditch system and shows the estimated peak flow in comparison with the ditch capacity. In each case, the asphalt-lined ditch is capable of containing the design flow.

A 24-inch diameter CMP runs beneath the B-18 stockpile access road to convey the computed 25-year peak flow. The roadway above the culvert is graded as a swale to convey higher flows for the PMP event, across the road and into a natural drainage channel at the toe of the B-18 Perimeter Road fill slope, which eventually discharges back into the asphalt V-ditch system.

5.5.3 Phases I and II Run-Off Control

Table 5.4 summarizes previously calculated run-off conditions for Phase I and II at each of the hydraulic structures required to convey the PMP run-off from roads used for waste truck or landfill equipment access. In each case, the ditch or culvert capacity exceeds the appropriate estimated peak flow.

Table 5.5 compares the capacity of the former temporary Phase I containment basin and the Northeast Containment Basin (see Sheets 5 and 10, respectively, in Appendix A.1) with the estimated run-off volumes from the respective design storms. The capacity shown for the temporary Phase I containment basin in the clay borrow pit is for a height to the top of the lined area. Freeboard is not required for this basin because the basin could not be overtopped above the clay pit walls which extend high above the lined area.

The Phase I Intermediate Closure basin was on the top of a liner that was installed to prevent infiltration into the waste. This containment basin was sized only for run-off from the interim closure slope. This basin was only required until the Phase II construction was completed.

The capacity for the Northeast Containment Basin is based on allowance for a 2-foot freeboard because embankment overtopping could occur at this location.

5.5.4 Phase III Run-on Control

Run-on control during Phase III will be performed in a similar manner as it has historically been performed during Phase I and II with the use of ditches on the outside of the perimeter road as well as brow ditches at the top of cut slopes. Figure 5.5 shows individual water shed areas that contribute run-on flow for Phase III and at Closure to the diversion ditches located along the outside edge of the B-18 Perimeter Road. These ditches intercept the run-on and direct it away from the B-18 active area. The run-on areas have decreased in size compared to Phases I and II; however, the run-on

control ditches have remained the same size. To avoid ponding around the southeast side of B-18 some additional fill is added to provide positive drainage from localized low points. Buried 18-inch diameter HDPE solid wall pipe is to be installed below the B-18 south berm to provide drainage of two localized low points. The drainage pipe is maintained outside the limits of the landfill.

The permanent stormwater run-on controls, such as the brow ditches, within the Phase IIIA watershed will be constructed as part of Phase IIIA. During the subsequent placement of waste in Phase IIIA (while Phase IIIB is being constructed), a temporary lined stormwater containment berm will be utilized to prevent stormwater run-on from contacting the Phase IIIA waste. The configuration of this temporary lined containment berm is shown on Sheet C-4A in Appendix A.2. The top of this berm will be approximately 8 feet higher than the nearby high point on the Phase IIIB "bench floor"; therefore, overtopping of this berm by stormwater run-on from the south will not occur. If necessary, portable pumps will be used by site personnel to convey the clean ponded stormwater run-on retained on the south side of the temporary berm to just south of the nearby high point on the Phase IIIB "bench floor," where it can then gravity flow to the Northeast Containment Basin. A maximum of approximately 2 feet of stormwater could pool against the south side of the temporary berm. However, it should be noted that only small amounts of stormwater are anticipated since Phase IIIB is planned to be constructed during the dry season and, once Phase IIIB is completed, run-on control between Phase IIIA and Phase IIIB will no longer be required.

5.5.5 Phase III Run-Off Control

Run-off control during Phase III operations will be very similar to the current operations. The existing B-18 waste fill is above the existing perimeter road and capable of diverting flows to the Northeast Containment Basin. Initially Phase III waste will be placed in the "valley" formed by the existing waste and the Phase III lined slope. Active areas will control run-off with soil berms to keep stormwater from reaching the basin. Areas covered with interim soil cover will drain within the "valley" to the Northeast Containment Basin. In the event of a PMP prior to the Phase III waste fill reaching the elevation of the perimeter road, where run-off can be diverted to the South Containment Basin, active pumping from the Northeast Containment Basin to the South Containment Basin would be required to maintain adequate capacity.

Appendix J.4, Table 4 summarizes run-off conditions at each of the hydraulic structures required to convey the PMP run-off. In each case, the ditch capacity exceeds the appropriate estimated peak flow.

Table 5.5 compares the capacity of the Northeast Containment Basin and South Containment Basin (see Sheets C-3 and C-4 in Appendix A.2) with the estimated run-off volumes from the 24-hour PMP design storm. Sufficient on-site storage capacity exists to contain all run-off within the basins, however, some active pumping during the 24-hour PMP storm will be required (approximately 1 acre foot) from the northeast basin to the south basin.

The permanent stormwater run-off controls, such as the perimeter channels, within the Phase IIIA watershed will be constructed as part of Phase IIIA. During subsequent placement of waste in Phase IIIA (while Phase IIIB is being constructed), a temporary lined stormwater containment berm will be utilized to contain stormwater flowing off of the south-facing Phase IIIA temporary waste fill slope. This temporary berm will be 10 feet tall (see Sheet C-4A in Appendix A.2). As shown in the Phase IIIA hydrology calculations in Appendix J.3, the basin that will be formed on the north side of this berm will have sufficient capacity to contain the entire run-off volume generated by the 24-hour PMP event while maintaining a freeboard of greater than 1 foot. Stormwater run-off that is impounded on the north side of the temporary berm will be considered leachate and will be managed in an

appropriate manner by site personnel (i.e., through the use of portable pumps to convey the stormwater into tanker trucks for transportation to an on-site treatment facility). Stormwater run-off from the north, east, and west facing waste slopes of Phase IIIA will flow to the Northeast Containment Basin. It should be noted that only small amounts of stormwater are anticipated for this temporary case as Phase IIIB is planned to be constructed during the dry season and, once Phase IIIB is completed, run-off control between Phase IIIA and Phase IIIB will no longer be required.

As described in the Phase IIIA hydrology calculations in Appendix J.3, during the construction of Phase III (i.e., before the South Containment Basin is constructed), the existing Northeast Containment Basin will not have sufficient capacity to contain the stormwater volume generated by the 24-hour PMP event. Specifically, there will be approximately 14 acre-feet of water that will have to be conveyed from the Northeast Containment Basin to the site's existing East Retention Basin (located approximately 2,000 feet north of the Northeast Containment Basin, as shown on Sheet C-1 in Appendix A.2). A 21-inch orifice outlet device will be set 3 feet below the top of the existing embankment of the Northeast Containment Basin. Water will flow through this orifice and into a pipeline (preliminarily sized at 21-inch inside diameter) that will convey this overflow by gravity to the East Retention Basin. During the Phase III construction, the orifice outlet device and gravity flow pipeline will prevent the overtopping of the existing Northeast Containment Basin during the 24-hour PMP event.

5.5.6 Closure Drainage

At closure, the trapezoidal earthen ditches along benches and the Cover Access Road are designed to convey the flow from the 6-hour PMP event as shown in Appendix J, Table 4. As previously discussed, the 6-hour PMP event has a higher rainfall intensity than the 24-hour PMP and is therefore more conservative for sizing the conveyance structures. Conveyance of surface water from the cover and the adjacent areas will occur along the final B-18 Perimeter Road, which will be graded to slope into the closure cover as shown in Detail 4 on Sheet C-7 in Appendix A.2. The peak surface water run-off generated by the 24-hour PMP event will be contained in either the Northeast Containment Basin or the South Containment Basin.

5.6 Leachate Collection and Removal

In accordance with USEPA guidance for landfill geosynthetic designs (USEPA, 1987), Appendix K includes the following calculations to support the design of the B-18 LCRS:

- The capacity of the primary LCRS at each area to transmit the mean annual precipitation during the operating period.
- The capacity of the primary LCRS at each sump to convey significantly greater volumes of leachate than historically measured without the build-up of 12 inches of hydrostatic head on the base liner system.
- The suitability of the geotextile to act as a filter between the base operations layer and the primary LCRS drainage gravel layer.

The calculations are based on the conservative assumption that, during operations, all rainfall will percolate through the waste and into the primary LCRS.

The calculations show that the transmissivity of the existing Trevira 1125/Polynet 3000 geocomposite drainage layer used on the sideslopes will have a transmissivity greater than that required to convey the mean annual precipitation considering the maximum waste overburden pressure of nearly 25,000 psf.

The transmissivity of the existing base geocomposite is approximately equal to that required to convey the mean annual precipitation drainage case. However, considerable redundant capacity is provided by the 12 inches of drainage gravel above the base geocomposite in the primary LCRS. The base portion of the secondary LCRS also includes a redundant, 12-inch-thick drainage gravel layer. Furthermore, a geonet layer was provided for a width of 80 feet above the vadose trench to provide drainage in this area.

Filter calculations for the geotextile between the base operations layer and the primary LCRS drainage gravel indicate that the existing Trevira 1125 geotextile is adequate.

Historic records (January 2001 to December 2007) indicate that the primary LCRS generates an average of 360 gallons per day for all four sumps. This volume is significantly less than the 36,000 gallon per day system capacity.

The secondary LCRS, has averaged 10 gallons per day over the period of January 2001 to December 2007. Peak flows have approached 150 gallons per day. Recent monitoring has indicated that no liquids are being collected in the secondary LCRS. The Response Action Plan (RAP) (SEC Donohue 1992a) determined that the Action Leakage Rate (ALR) for each sump ranged from 1,250 to 3,500 gallons per day. These values have a minimum calculated safety factor of 3. Liquids collected in the secondary LCRS have not approached the ALR. The current ALRs remain valid for the proposed expansion since the limiting factor was the length of geocomposite at the toe of slope and this length remains unchanged, conservative transmissivity values are used in the calculations.

The vadose zone collection system has not collected any liquids during the period January 2001 to December 2007. The Vadose Zone Response Plan (SEC Donohue 1992b) allows from 3.5 to 6.6 gallons per day to be collected in the vadose sumps based on a de minimus leakage rate of 20 gallons per acre per day. The vadose system will not be modified by the proposed expansion: therefore, the allowable leakage rates remain valid.

In the event leakage in the secondary or vadose collection systems exceeds the ALR, then appropriate actions in accordance with the RAP will be implemented.

5.7 Riser Pipe Designs

Appendix L includes the engineering calculations performed to support the following riser pipe requirements:

- Design of the main 24-inch diameter steel vertical riser pipe and its foundation, located about 6.5 feet above the primary liner system in the sump area, as illustrated in Section B on Sheet 21 in Appendix A.1.

- Provision of adequate crushing strength for the lower 18-inch diameter stainless steel vertical riser pipe that is approximately 7 feet long and is located immediately below the 24-inch diameter steel vertical riser pipe.

- Assurance that the pressure exerted by the main vertical riser pipe foundation will not create a bearing capacity failure in the underlying clay liners.

- Evaluation of the stresses and deflections in the bottom portions of the sideslope riser pipes.

As shown in Section B on Sheet 21 in Appendix A.1, the main vertical riser pipe will be placed inside of a larger, 30-inch diameter corrugated HDPE pipe to avoid the development of high drag loads on the 24-inch diameter steel pipe. The corrugated pipe will not develop significant vertical stresses because it will readily deflect in an accordion-like manner in the longitudinal direction as the surrounding waste settles. The corrugated pipe will be held away from the steel pipe with flexible spacers at about 10-foot centers to avoid long sections where the two pipes would otherwise be in continuous contact. To be conservative, the calculations assume that a contact area of 10 percent of the total contact area between the two pipes develops and that it is capable of transferring drag loads to the inner steel pipe. In addition, significant factors of safety are provided for each of the key components of the system to minimize the potential for large deformation of the riser pipe to occur.

The 6-foot by 6-foot by 1.5-foot-thick reinforced concrete foundation for the main riser pipe was designed to spread any load on the pipe over a sufficient area so that there is a high factor of safety against potential bearing capacity deformations of the underlying clay liner. This is accomplished because the minimum bearing capacity factor of safety is estimated to be between 7 and 10 when using a conservative estimate of the strength of the clay liner. A slip connection has been included in the riser pipe at approximately elevation 900 ft-MSL to reduce the load on the foundation. The loads above the slip connection will be transferred within the waste.

Compression loading on the lower stainless steel pipe is calculated considering at-rest pressures resulting from the main riser pipe foundation loading. The factor of safety against crushing is greater than 5, which is considered appropriately conservative for this system.

Finally, calculations of deflection for the lower portions of the sideslope riser pipes show that:

- The carbon and stainless steel pipe deflections will be on the order of 0.1 inches, which is relatively small with regard to the allowable deflections for these pipes.

- The maximum deflection of the 8-inch diameter HDPE SDR 8.3 pipe is estimated to be on the order of 20 to 30 percent of its diameter.

Although the deflection of the HDPE pipe is higher than would normally be desirable, it is considered to be acceptable because this pipe is included as a second (i.e., redundant) system for pumping from the secondary LCRS. From an operations viewpoint, it was determined that it would be better to have these two redundant types of pipes to minimize the potential for pump access loss because of material deterioration.

5.8 Cover Infiltration

Appendix M includes calculations similar to those performed to support the design of the currently-permitted final closure cover system for B-18 (DTSC, 2003). These calculations were performed using the updated computer program HELP version 3.07 (Schroeder et al., 1994) to estimate the amount of infiltration through and head on the final cover system. The infiltration analyses were conducted by assuming the following parameters:

- The vegetative cover soil layer is 2.5-feet-thick with a permeability of 2 x 10^{-4} cm/s, based on past experience with this type of material at the KHF.

- The HELP program's default climate data for Fresno, California and Bakersfield, California were used to model the climatological conditions at the KHF. Use of Fresno climate is a conservative assumption since Fresno receives approximately 50 percent more rainfall annually than the KHF, based on historic precipitation data.

- The HDPE geomembrane will have 0.50 holes per acre resulting from manufacturer's flaws (where each hole has a 1 mm diameter), 1.0 hole per acre resulting from installation defects (where each hole has an area of 1 cm^2), and "excellent" placement quality.

- The nonwoven geotextile component of the final cover system has an in-plane permeability of 0.25 to 0.40 cm/s.

- The foundation layer has a permeability of 1 x 10^{-5} cm/sec

- The slope of the final cover system is 25 percent (i.e., 4H:1V).

Based on the above assumptions, the results of the HELP analyses indicate that a maximum head of about 3 to 4 inches will develop on the geomembrane during extended rainy seasons (similar to Fresno) and that acceptably low infiltration rates will occur through the HDPE geomembrane. Since a head of only 3 to 4 inches on the geomembrane is not anticipated to compromise the stability of the final cover system, the permitted final cover system is considered adequate for B-18. Additionally, the permitted cover system has been used to close other facilities at KHF since 1994 without failure or apparent build up of liquids within the cover.

5.9 Frost and Biotic Protection

The effects of frost penetration on the B-18 final closure cover were evaluated. Appendix N.1 provides two maps that each show estimated depths of frost penetration for the United States. Both of these maps indicate that the maximum depth of frost penetration at the KHF is less than 6 inches. Past experience and observations at the KHF corroborate this finding. Hence, any frost penetration that occurs is anticipated to be confined to the uppermost portions of the 2.5-foot-thick vegetative cover soil layer and should not present a significant potential to deteriorate the cover system or to cause special maintenance requirements.

The effects of burrowing animals on the B-18 final cover system were also evaluated. Information presented in Appendix N.2 indicates that burrowing animals will be confined to the vegetative cover soil layer due to the presence of the underlying HDPE geomembrane. Past experience at the KHF indicates that HDPE geomembranes are effective barriers to burrowing animals. Hence, the performance of the B-18 final cover is not expected to be impacted by burrowing animals.

6.0 REFERENCES

California Department of Water Resources (CDWR), 1976, Rainfall Analysis for Drainage Design Volume I, Short-Duration Precipitation Frequency Data, October.

Department of Toxic Substances Control (DTSC), 2003, Hazardous Waste Facility Permit, Kettleman Hills Facility, June 16.

Donohue and Associates, Inc., 1988a, Engineering Report, Chemical Waste Management, Inc., Kettleman Hills Facility, Landfill B-19, Phases II and III, Kings County, California, October.

Donovan, N., 1983, A Practitioner's View of Site Effects on Strong Ground Motion, Proceedings of Conference XXII on Site-Specific Effects of Soil and Rock on Ground Motion and the Implications for Earthquake-Resistant Design, July 25-27, USGS.

Duncan, J.M., and Buchignani, A.L., 1976, An Engineering Manual for Settlement Studies, Department of Civil Engineering, University of California – Berkeley, June.

EMCON Associates, 1989, Progress Report, Soil/Waste Compatibility Testing, Kettleman Hills Facility, Kettleman City, California, December 26.

EMCON Associates, 1986, Hydrogeologic Characterization, Kettleman Hills Facility, Kings County, California, December.

Environmental Construction Services, Inc. (ECS), 1992a, Clay Source, Landfill B-18, Phases IA and IB, Kettleman Hills Facility, Kettleman City, California, January 9.

Environmental Construction Services, Inc. (ECS), 1992b, Secondary Clay Liner, Landfill B-18, Phases IA and IB, Kettleman Hills Facility, Kettleman City, California, March 30.

Environmental Construction Services, Inc. (ECS), 1992c, Secondary HDPE Liner and Leachate Collection System, Landfill B-18, Phases IA and IB, Kettleman Hills Facility, Kettleman City, California, March 30.

Environmental Construction Services, Inc. (ECS), 1992d, Primary Clay Liner, Landfill B-18, Phases IA and IB, Kettleman Hills Facility, Kettleman City, California, March 30.

Environmental Construction Services, Inc. (ECS), 1992e, Primary HDPE Liner and Leachate Collection System, Landfill B-18, Phases IA and IB, Kettleman Hills Facility, Kettleman City, California, March 30.

Environmental Construction Services, Inc. (ECS), 1992f, Summary Construction Observation, Landfill B-18, Phases IA and IB, Kettleman Hills Facility, Kettleman City, California, March 30.

Environmental Construction Services, Inc. (ECS), 1992g, Operational Features, Landfill B-18, Phases IA and IB, Kettleman Hills Facility, Kettleman City, California, June.

Environmental Construction Services, Inc. (ECS), 1992h, Design Changes and Design Clarifications, Landfill B-18, Phases IA and IB, Kettleman Hills Facility, Kettleman City, California, February 18.

Environmental Solutions, Inc. (ESI), 1992, Test Fill and Infiltrometer Test Results, Landfill Unit B-18, Phases I and II and Final Closure, January 23.

Environmental Solutions, Inc. (ESI), 1990a, Engineering and Design Report, Landfill Unit B-18, Phases I and II and Final Closure, Kettleman Hills Facility, Kings County, California, Volumes I and II, August.

Environmental Solutions, Inc. (ESI), 1990b, Construction Specifications and Quality Assurance Plan, Landfill Unit B-18, Phases I and II and Final Closure, Kettleman Hills Facility, Kings County, California, July 31.

Franklin, A.G., and Chang, F.K., 1977, Permanent Displacements of Earth Embankments by Newmark Sliding Block Analysis, Earthquake Resistance of Earth and Rock-Fill Dams, Report 5, U.S. Army Engineer Waterways Experiment Station, November.

Geosyntec Consultants, 2008a, Clay Stockpile and Test Pad Report, Kettleman Hills Facility, Kettleman City, California, December.

Geosyntec Consultants, 2008b, Clay Source Testing: Summary of Field and Laboratory Test Results, Kettleman Hills Facility, Kettleman City, California, February 11.

Geosyntec Consultants, 2001, Site-Specific Groundwater Monitoring Plan, Kettleman Hills Facility, Kings County, California, May.

Golder Associates Inc., 1992, Subgrade Geologic Mapping and Chemical Analysis for Landfill B-18, Phase I, Kettleman Hills Facility, Kettleman City, California, March 30.

Golder Associates Inc., 1989a, Supporting Calculations for the Landfill B-15 Closure Plan, Kettleman Hills Facility, December.

Golder Associates Inc., 1989c, Waste Settlement Report for Final Landfill Cover Design, Kettleman Hills Facility, July.

Golder Associates Inc., 1988, Updated Evaluation of Site Design Ground Motions for the CWM Kettleman Hills Facility, September.

Golder Construction Services, Inc. (GCS), 1993a, Clay Liner Source Report (Volume 1), Landfill B-18, Phases IIA and IIB, Kettleman Hills Facility, Kettleman City, California, May.

Golder Construction Services, Inc. (GCS), 1993b, Subgrade Geologic Mapping and Chemical Analysis Report (Volume 2), Landfill B-18, Phases IIA and IIB, Kettleman Hills Facility, Kettleman City, California, May.

Golder Construction Services, Inc. (GCS), 1993c, Excavation and Structural Fill Placement Construction Report (Volume 3), Landfill B-18, Phases IIA and IIB, Kettleman Hills Facility, Kettleman City, California, August.

Golder Construction Services, Inc. (GCS), 1993d, Secondary Clay Liner Construction Report (Volume 4), Landfill B-18, Phases IIA and IIB, Kettleman Hills Facility, Kettleman City, California, September.

Golder Construction Services, Inc. (GCS), 1993e, Secondary and Vadose HDPE Liner and Leachate Collection System Report (Volume 5), Landfill B-18, Phases IIA and IIB, Kettleman Hills Facility, Kettleman City, California, October.

Golder Construction Services, Inc. (GCS), 1993f, Primary Clay Liner Construction Report (Volume 6), Landfill B-18, Phases IIA and IIB, Kettleman Hills Facility, Kettleman City, California, October.

Golder Construction Services, Inc. (GCS), 1993g, Primary HDPE Liner and Leachate Collection System Construction Report (Volume 7), Landfill B-18, Phases IIA and IIB, Kettleman Hills Facility, Kettleman City, California, November.

Golder Construction Services, Inc. (GCS), 1993h, Summary Construction Report (Volume 8), Landfill B-18, Phases IIA and IIB, Kettleman Hills Facility, Kettleman City, California, November 8.

Golder Construction Services, Inc. (GCS), 1993i, Operational Features Report (Volume 9), Landfill B-18, Phases IIA and IIB, Kettleman Hills Facility, Kettleman City, California, December 6.

Hudson, M.B., Beikae, M., and Idriss, I.M., 1994, QUAD4M, a Computer Program to Evaluate the Seismic Response of Soil Structures Using Finite Element Procedures and Incorporating a Compliant Base, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, 1994-05.

J&L Testing Company, 1991a, Leachate Compatibility of Geosynthetic Materials – Kettleman Hills Facility, Final Report, November 4.

J&L Testing Company, 1991b, Geotechnical Laboratory Test Results Aggregate/Leachate Compatibility Testing, Kettleman Hills Facility, November 7.

J&L Testing Company, 1992, NSC 60 mil Textured HDPE Chemical Compatibility Testing EPA Method 9090 – Kettleman Hills Facility, September 8.

Kings County, 1990, Chemical Waste Management, Kettleman Hills Facility, B-18 Administrative Approval Permits 90-23 and 90-24, Kings County Planning Agency, August.

Makdisi, F.I., and Seed, H.B., 1978, Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformation, Journal of the Geotechnical Engineering Division, ASCE, Vol. 104, No. GT7, July, pp. 849-868.

National Oceanic and Atmospheric Administration (NOAA), 1998, Hydrometeorological Report No. 58, Probable Maximum Precipitation for California – Calculation Procedures.

Natural Resources Conservation Service (NRCS), 2004, RUSLE2 – Instructions & User Guide, United States Department of Agriculture, May.

Newmark, N.M., 1965, Effects of Earthquakes on Dams and Embankments, Fifth Rankine Lecture.

Purdue University (Ronald A. Siegel), 1986, STABL User Manual, West Lafayette, Indiana, June 4.

Regional Water Quality Control Board (RWQCB), Central Valley Region, 1998, Order No. 98-058, Waste Discharge Requirements for Chemical Waste Management, Inc., Kettleman Hills Facility, Kings County, February 27.

Roger Foott and Associates, Inc., 1990a, Final Report, Unit B-18 Landfill, Holocene Fault Investigation, Kettleman Hills Facility, August 14.

Roger Foott and Associates, Inc., 1990b, Phase I Holocene Surface Faulting Study, Nunez Fault and Kettleman Hills Facility, April 2.

Schroeder, P.R., Aziz, N.M., Lloyd, C.M., and Zappi, P.A., 1994, The Hydrologic Evaluation of Landfill Performance (HELP) Model: User's Guide for Version 3, EPA/600/R-94/168a, September.

SEC Donohue Environment & Infrastructure, 1992a, Response Action Plan, Landfill B-18, Kettleman Hills Facility, June.

SEC Donohue Environment & Infrastructure, 1992a, Vadose Response Plan, Landfill B-18, Kettleman Hills Facility, June.

Seed, R.B., and Bonaparte, R., 1992, Seismic Analysis and Design of Lined Waste Fills: Current Practice, ASCE Geotechnical Special Publication No. 31, Stability and Performance of Slopes and Embankments, June 29-July 1.

Seed, R.B., Mitchell, J.K., and Seed, H.B., 1988, Slope Stability Failure Investigation: Landfill Unit B-19, Phase I-A, Kettleman Hills, California, Report No. UCB/GT/88-01, Department of Civil Engineering, University of California, Berkeley, July.

TRI/Environmental, Inc., 1991, Chemical Compatibility Testing of National Seal 60 mil Geomembrane with Kettleman Hills Waste Leachate, Soltex Resin, NSC#CO2A, Final Report, October 14.

United States Environmental Protection Agency (USEPA), 1989, Technical Guidance Document: Final Covers on Hazardous Waste Landfills and Surface Impoundments, EPA/530-SW-89-047, July.

United States Environmental Protection Agency (USEPA), 1987, Minimum Technology Guidance on Double Liner Systems for Landfills and Surface Impoundments - Design, Construction and Operation, EPA/530/SW-85/014, April.

Woodring, W.P., Stewart, R., and Richards, R.W., 1940, Geology of the Kettleman Hills Oil Field, California; Stratigraphy, Paleontology, and Structure, United States Geological Survey Professional Paper No. 195, 170 p.

Woodward-Clyde Consultants, 1987, Seismic Stability Evaluation for the Toe Buttress Construction, Operating Industries, Inc. Site, Monterey Park, California, Document No. 120-R12-RT-EPNN-1, Prepared for U.S. EPA, May 27.

The following references were used in the Phase I and Phase II engineering report but have not specifically been included in the Phase III and Closure. The references are included herein for completeness:

Chemical Waste Management, Inc. (CWMI), 1989, Operation Plan for Landfill Unit B-19, Kettleman Hills Facility. June.

Donohue and Associates, Inc., 1988b, Construction Documents, Landfill Unit B-19, Phases II and III, Kettleman Hills Facility. September.

Geoservices, Inc., 1990, Consulting Engineers. Direct Shear Interface Strength Testing Selected Geotextile Interfaces. April 5.

Geosyntec, Inc., 1988, Final Report on the Direct Shear Testing for the Chemical Waste Management, Inc. New Milford, Connecticut Landfill. Prepared for Geoservices, Inc. Consulting Engineers. October 11.

Golder Associates Inc., 1990a, Combined Closure Plan for Landfills B-1/4/S/6/7, B-8/9/9 Expansion /9 Extension /10/11, Surface Impoundments P-5/12/12A/13/17 and Area S-3, Kettleman Hills Facility. Revision 2, Draft. June.

Golder Associates Inc., 1990b, Direct Shear Testing Program for Landfill Cover Design, Kettleman Hills Facility. June 29.

Golder Associates Inc., 1990c, Closure Plan for Landfill B16, Kettleman Hills Facility, Kettleman City, California. April 23.

Golder Associates Inc., 1989b, Feasibility Study for Final Landfill Cover Systems, Kettleman Hills Facility (Draft), August.

Golder Associates Inc., 1989d, Filling Plan, Landfill B-19, Phases II and III, Kettleman Hills Facility, May.

Golder Associates Inc., 1988a, Filling Plan, Phase IB and III, Kettleman Hills Facility, August.

Golder Associates Inc., 1988a, Dynamic Stability of Cut Slopes in Bedrock, Landfill B-19, Kettleman Hills Facility, June.

Golder Associates Inc., 1988c, Filling Plan, Phase IB and III, Kettleman Hills Facility, August.

Gundle Lining Systems, Inc., 1989, Direct Shear Friction Testing, Gundle Friction Sheet, Mirafi 140N Geotextile, March 26.

Gundle Lining Systems, Inc., 1988a, Direct Shear Test Results, Textured HDPE and Soil, October.

Gundle Lining Systems, Inc., 1988b, Direct Shear Test Results, Gundle HDT and Fabric, Covered XL-4, Dry and Saturated Conditions, September.

Gundle Lining Systems, Inc., 1988c, Direct Shear Test Results, Textural HDPE and Sand, Dry and Saturated Conditions, June 15.

Gundle Lining Systems, Inc., 1988d, Lab Report #2683.

Gundle Lining Systems, Inc., 1987a, Direct Shear Testing of Friction Liner Material, March 11.

Gundle Lining Systems, Inc., 1987b, Direct Shear Friction Testing, Gundnet XL-4 versus 80 mil Gundline, November.

Gundle Lining Systems, Inc., (not dated), Polyethylene VS. PVC Liners.

Gundle Lining Systems, Inc., (not dated), Direct Shear Testing of Friction Liner Materials (Several Separate Reports).

Koerner, R., 1986, Designing with Geosynthetics.

Lambe, T.W., and Whitman, R.V., 1969, Soil Mechanics, New York: John Wiley & Sons, Inc.

Murphy, W.L. and Gilbert, P.A., 1985, Settlement and Cover Subsidence of Hazardous Waste Landfills, Prepared for U.S. EPA. Document No. 600/2-85-035, April.

SLT North America, Inc., (not dated), Physical Properties of Textured HDPE Liners, Friction Flex Application Data.

U.S. Department of Agriculture, 1987, Agriculture Handbook 537, Predicting Rainfall Erosion Losses, December.

U.S. Department of the Navy, Naval Facilities Engineering Command, 1986, Soil Mechanics Design Manual 7.01, September.

U.S. EPA, 1990, RCRA Hazardous Waste Facility Permit (Part B) for Kettleman Hills Facility, March.

U.S. EPA, 1988, Lining of Waste Containment and Other Impoundment Facilities, EPA-600/2-88/052, September.

U.S. EPA, 1979, Design and Construction of Covers for Solid Waste Landfills, EPA-600/2-79-165, August.

U.S. EPA, 1983, The Hydrologic Evaluation of Landfill Performance (HELP) Model, Version 2, EPA/530-SW-84-OlO, August.

Water Resources Engineers, Inc., 1977, County of Solano, Hydrology and Drainage Design Procedure, October.

Winterkorn, H.F., and Fang, H.-Y. (eds.), 1975, Foundation Engineering Handbook, Chapter 23, Buried Structures, Van Nostrand Reinhold Company.

Yen, B.C., and Scanlan, B., 1975, Sanitary Landfill Settlement Rates, ASCE Journal of the Geotechnical Engineering Division, May.

TABLES

TABLE 3.1

KEY BORING INFORMATION

BORING	NORTHING EASTING	GROUND ELEVATION (Feet) TOTAL DEPTH	SAMPLE NUMBER (Mode)[1]	SAMPLE DEPTH (Feet)	STRATIGRAPHIC UNIT DESCRIPTION[2]	STRATIGRAPHIC UNIT NUMBER	RECOVERY (%)	PURPOSE
L18-A	8,943	856	S-1 (PB)	5 - 6.1	ss	18-10	91	• Obtain samples of Unit 18-10. This unit occurs at the toe of the western cut slope and bottom of excavation, Phase I/II (See Sections 8 and 14). Unit 18-10 also occurs at toe and middle section of eastern cut slope at Section 18.
	840	38.7	S-2 (PB)	10 - 12.5	cs/ss	18-10	84	
			S-3 (PB)	17-19.5	cs/ss	18-10	96	
			S-4 (PB)	25-27.5	silty ss	18-10	100	
			S-5 (PB)	33-35.5	silty ss w/cs	18-10	88	
			S-6 (PB)	37-38.7	silty ss w/cs	18-10	88	
L18-B	8,755	861	S-1 (PB)	6-8.5	cs	18-12	92	• Determine depth to top of silty ss (Unit 18-11).
	860	39.5	S-2 (PB)	11.5-14	silty ss	18-11	96	• Obtain samples of Unit 18-12. This unit occurs near the toe of western cut slope, Phase I/II (See Sections 14+00 and 18+00).
			S-3 (PB)	20-22.5	ss	18-11	64	
			S-4 (PB)	28-30.5	ss w/cs/slt	18-11	88	
			S-5 (PB)	37-39.5	ss	18-11	80	• Obtain samples of Unit 18-11. This unit occurs at toe of western cut slope - See Sections 14+00 and 18+00.
L18-C	8,623	754	B-1 (DR)	6-7.5	colluvium (cl)	Qc	87	• Determine colluvium thickness.
	2,120	89	B-2 (DR)	12-13.5	colluvium (cl)	Qc	53	• Obtain samples of ss, slt (Unit 18-7). This unit occurs at bottom of excavation, Phase III/IV (See Sections 14+00 and 18+00) and at toe of western in-dipping cut slope, Phases III & IV (See Section 14+00).
			S-1 (PB)	15-17.3	slt	18-7	91	
			S-2 (PB)	20-22.5	slt	18-7	92	
			S-3 (PB)	25-27.5	slt	18-7	88	
			S-4 (PB)	33-35	cs/slt	18-7	95	
			S-5 (PB)	41-43.1	silty ss	18-7	95	
			S-6 (PB)	48-50.1	slt clayey slt	18-7	90	
			S-7 (PB)	56-58	slt	18-7	90	
			S-8 (PB)	64-66.3	clayey slt	18-7	78	
			S-9 (PB)	72-74.3	ss, silty ss	18-7	100	
			S-10 (PB)	80-82	slt/cs	18-7	95	
			S-11 (PB)	87-89	slt	18-7	85	
L18-D	8,785	740	B-1 (DR)	5-6.5	colluvium	--	87	• Determine colluvium thickness.
	2,480	86	B-2 (DR)	10-11.5	colluvium	--	100	• Clarify contact of Units 18-4 and 18-3.
			B-3 (Bag)	14	colluvium	--	--	• Clarify contact of Units 18-4 and 18-5.
			S-1 (PB)	15-17.3	silty ss	18-5	78	• Obtain samples of these units.
			S-2 (PB)	22-24.3	slt	18-5	96	• Units 18-4 and 18-5 occur at eastern out-dipping cut slope, Phase III and IV (See Sections 14+00 and 18+00).
			S-3 (PB)	29-31.5	cs/slt	18-4	80	
			S-4 (PB)	36-38.5	cs/slt	18-4	92	
			S-5 (PB)	44-46.5	cs	18-4	60	
			S-6 (PB)	52-54.3	cs	18-4	65	
			S-7 (PB)	60-62.5	cs/slt	18-4	48	
			S-8 (PB)	68-70.5	cs	18-4	56	
			S-9 (PB)	76-78.5	silty ss	18-3	32	
			S-10 (PB)	81-83.5	silty ss	18-3	36	
			S-11 (PB)	83.5-86	silty ss	18-3	68	
L18-E	8,895	727	B-1 (DR)	5-6.5	colluvium (cl)	--	87	• Determine colluvium thickness.
	2,670	62.5	S-1 (PB)	16-12.5	colluvium (cl)	--	68	• Clarify contact of Units 18-2 and 18-3.
			S-2 (PB)	15-17.5	ss	18-3	84	• Obtain samples of colluvium Units 18-3 and 18-2. These units may serve as foundation materials or run-off retention berms or larger berm for Phase III & IV configuration.
			S-3 (PB)	20-22.5	ss	18-3	56	
			S-4 (PB)	28-30.2	ss	18-3	86	
			S-5 (PB)	35-37.5	sandy slt	18-3	88	
			S-6 (PB)	43-45	sandy slt	18-3	80	
			S-7 (PB)	51-53	cs	18-2	95	
			S-8 (PB)	60-62.5	sandy cs	18-2	96	

(1) PB = Pitcher Barrel; DR = Drive Sampler
(2) cs = Claystone; ss = Sandstone; slt = Siltstone

TABLE 3.1

KEY BORING INFORMATION
(Continued)

BORING	NORTHING EASTING	GROUND ELEVATION (Feet) TOTAL DEPTH	SAMPLE NUMBER (Mode)(1)	SAMPLE DEPTH (Feet)	STRATIGRAPHIC UNIT DESCRIPTION(2)	STRATIGRAPHIC UNIT NUMBER	RECOVERY (%)	PURPOSE
L18-F	8,930	821	S-1 (PB)	6-8.5	cs	18-8	100	• Obtain samples of Unit 18-8 (clay). This unit occurs in eastern out-dipping cut slope, Phase I/II (See Section 8+00).
	1,538	58.5	S-2 (PB)	16-18.5	cs	18-8	88	
			S-3 (PB)	26-28.5	cs	18-8	92	
			S-4 (PB)	36-38.5	cs	18-8	76	
			S-5 (PB)	46-48.1	cs	18-8	100	
			S-6 (PB)	56-58.5	cs	18-8	84	
L18-G	7,500	853	S-1 (PB)	6-8.5	ss	18-13	56	• Obtain samples of Units 18-13 and 18-12. Unit 18-13 (sandstone) comprises the western cut slope, Phase I/II. Unit 18-12 (claystone) occurs near the toe of the cut slope, Phase I/II, at Section 18+00.
	1,914	77.5	S-2 (PB)	14-16.5	ss	18-13	56	
			S-3 (PB)	22-23.7	ss	18-13	88	
			S-4 (PB)	30-32	ss	18-13	60	
			S-5 (PB)	40-41.3	ss	18-13	92	
			S-6 (PB)	45-47.5	cs	18-12	60	
			S-7 (PB)	50-51.8	cs	18-12	72	
			S-8 (PB)	60-62.5	cs	18-12	40	
			S-9 (PB)	65-67	cs	18-12	70	
			S-10 (PB)	75-77.5	ss	18-12	32	
L18-H	8,065	865	S-1 (PB)	6-8.5	cs	18-10	72	• Obtain samples of Unit 18-10 underlying the fossil bed. This unit occurs at out-dipping cut slope, eastern side, Phase I/II (see Sections 14+00 and 18+00). In Section 8+00 Unit 18-10 occurs at bottom of excavation.
	2,055	57.5	S-2 (PB)	15-17.5	silty ss	18-10	76	
			S-3 (PB)	25-27.5	cs with sand	18-10	68	
			S-4 (PB)	35-37.5	cs	18-10	92	
			S-5 (PB)	45-47.5	silty ss	18-10	96	
			S-6 (PB)	55-57.5	cs with sand	18-10	48	
L18-I	8,022	766	B-1 (DR)	6-7.5	colluvium (cl)	--	87	• Determine colluvium thickness.
	2,415	18.5	S-1 (PB)	12-13.8	colluvium (cl)	--	78	• Obtain sample of Unit 18-8 (claystone).
			S-2 (PB)	16-18.5	cs	18-8	92	
L18-J	228,390	820	S-1 (PB)	5 - 7.5	cs	18-8	68	• Obtain samples of Unit 18-8. This unit will be used as liner material • The primary purpose is to evaluate the continuity of the claystone and to determine the existence and thickness of thin sandstone strata as coted on the log of Monitoring Well K-8.
	1,701,985	172.5	S-2 (PB)	10 - 12.5	cs	18-8	88	
			S-3 (PB)	15 - 17.5	cs	18-8	88	
			S-4 (PB)	20 - 22	cs/ss	18-8	75	
			S-5 (PB)	25 - 27.5	cs	18-8	76	
			S-6 (PB)	30 - 32.5	cs	18-8	96	
			S-7 (PB)	35 - 37.5	cs	18-8	100	
			S-8 (PB)	40 - 42.5	cs	18-8	72	
			S-9 (PB)	45 - 47.5	cs	18-8	92	
			S-10 (PB)	50 - 52.5	cs	18-8	92	
			S-11 (PB)	55 - 57.5	cs	18-8	92	
			S-12 (PB)	60 - 62.5	cs	18-8	96	
			S-13 (PB)	65 - 67.5	cs	18-8	84	
			S-14 (PB)	70 - 72.5	cs	18-8	88	
			S-15 (PB)	75 - 77.5	cs	18-8	100	
			S-16 (PB)	80 - 82.5	cs	18-8	96	
			S-17 (PB)	85 - 87.5	cs	18-8	88	
			S-18 (PB)	90 - 92.5	cs	18-8	84	
			S-19 (PB)	95 - 97.5	cs/ss	18-8	32	
			S-20 (PB)	100 - 101.5	cs	)8-8	60	
			S-21 (PB)	105 - 107.5	cs	18-8	100	
			S-22 (PB)	110 - 112.5	cs	18-8	100	
			S-23 (PB)	115 - 117.5	cs	18-8	96	
			S-24 (PB)	120 - 122.5	cs	18-8	100	
			S-25 (PB)	125 - 127.5	cs	18-8	96	

(1) PB = Pitcher Barrel; DR = Drive Sampler
(2) cs = Claystone; ss = Sandstone; slt = Siltstone

TABLE 3.1

KEY BORING INFORMATION
(Continued)

BORING	NORTHING EASTING	GROUND ELEVATION (Feet) TOTAL DEPTH	SAMPLE NUMBER (Mode)(1)	SAMPLE DEPTH (Feet)	STRATIGRAPHIC UNIT DESCRIPTION(2)	STRATIGRAPHIC UNIT NUMBER	RECOVERY (%)	PURPOSE
L18-J (Cont.)			S-26 (PB)	130 - 132.8	cs	18-8	100	
			S-27 (PB)	135 - 137.5	cs	18-8	100	
			S-28 (PB)	140 - 142.5	cs	18-8	100	
			S-29 (PB)	145 - 147.5	cs	18-8	100	
			S-30 (PB)	150 - 152.5	cs	18-8	100	
			S-31 (PB)	155 - 157.5	cs	18-8	100	
			S-32 (PB)	160 - 162.5	cs	18-8	100	
			S-33 (PB)	165 - 167.5	slt	18-7	92	
			S-34 (PB)	170 - 172.5	ss	18-7	88	
L18-K	228,390	804	S-1 (PB)	5 - 7.5	ss	18-9	88	• Obtain samples of Unit 18-9. This strata will be excavated concurrently with the claystone in order to develop the clay borrow pit.
	1,701,722	97.5	S-2 (PB)	10 - 12.5	silty ss	18-9	92	
			S-3 (PB)	15 - 17.5	silty ss	18-9	100	
			S-4 (PB)	20 - 22.5	silty ss	18-9	60	• Samples from this boring will be utilized for various combinations of claystone to determine the potential for mixing of the two rock types as they are excavated from the borrow pit.
			S-5 (PB)	25 - 27.5	ss	18-9	96	
			S-6 (PB)	30 - 32.5	sandy slt	18-9	100	
			S-7 (PB)	35 - 37.2	silty ss	18-9	86	
			S-8 (PB)	40 - 42.5	silty ss	18-9	92	
			S-9 (PB)	45 - 47	slt	18-9	95	
			S-10 (PB)	50 - 52.5	sandy slt	18-9	88	
			S-11 (PB)	55 - 57.5	sandy slt	18-9	68	
			S-12 (PB)	60 - 62.5	sandy slt	18-9	56	
			S-13 (PB)	65 - 67.2	ss	18-9	86	
			S-14 (PB)	70 - 72.5	silty ss	18-9	100	
			S-15 (PB)	75 - 76.8	silty ss	18-9	94	
			S-16 (PB)	80 - 82.3	ss	18-9	52	
			S-17 (PB)	85 - 87.5	cs	18-8	88	
			S-18 (PB)	90 - 92.5	cs	18-8	100	
			S-19 (PB)	95 - 97.5	cs	18-8	100	

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(1) PB = Pitcher Barrel; DR = Drive Sampler
(2) cs = Claystone; ss = Sandstone; slt = Siltstone

TABLE 3.2

MATRIX OF INVESTIGATION ACTIVITY VS. ROCK STRUCTURE

INVESTIGATION ACTIVITY				
STRATIGRAPHIC UNIT	TEST TRENCHES	TEST PITS	BORINGS	EXISTING AND PREVIOUS MONITORING WELLS
Colluvium	All Test Trenches	All Test Pits	L18-C, L18-D, L18-E, L18-I	–
18-1	–	–	–	K-19, K-29
18-2	–	–	L18-E	K-19, K-29
18-3	–	–	L18-D, L18-E	K2, K-19
18-4	DT-E	–	L18-D	K-1, K-2, K-26
18-5	DT-E	–	L18-D	K-1, K-2, K-26, K-38
18-6	DT-E	–	–	K-1, K-2, K-26, K-38
18-7	DT-B, DT-E, T-1, T-2	TP-5, TP-6, TP-20, TP-21, TP-28	L18-C, L18-F	K-1, K-2, K-8, K-26, K-32, K-38
18-8	DT-A, DT-B, DT-C, T-3	TP-1, TP-6, TP-7, TP-8, TP-9, TP-27, TP-36, TP-37, TP-38, TP-39, TP-40, TP-41	L18-F, L18-1 , L18-J	K-8, K-32, K-33, K-38
18-9	DT-A, DT-D	TP-1, TP-2, TP-3, TP-4, TP-26, TP-42, TP-43	L18-K	K-8, K-33
18-10	DT-A, DT-D	TP-4, TP-12, TP-13, TP-18, TP-29, TP-35, TP-35A	L18-A, L18-H	K-8, K-33
18-11	DT-A, DT-D	TP-11, TP-19, TP-25, TP-30	L18-B	K-8, K-18, K-33
18-12	DT-A, DT-D	TP-10, TP-16, TP-17, TP-31, TP-34	L18-B, L18-G	K-21, K-33
18-13	DT-A, DT-D, DT-F	TP-14, TP-15, TP-22, TP-23, TP-24, TP-32, TP-33	L18-G	K-18, K-21, K-33
18-14	–	–	–	K-18, K-21

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TABLE 3.3

GEOTECHNICAL PARAMETERS FROM PRIOR INVESTIGATIONS

PARAMETER		PHASES II & III B-19 LANDFILL DESIGN			SEED, RAYMOND B. ET AL. INVESTIGATION OF PHASE I B-19 LANDFILL FAILURE	GENERIC EVALUATIONS OF KHF LANDFILL CLOSURES	EMPIRICAL OR MEASURED DATA FROM PUBLISHED LITERATURE	
I. STRENGTH PROPERTIES								
SANDSTONE Cross-bed Strength (In-dipping)	C (psf) Ø (Degrees)	800 (1) 40						
Anisotropic Strength (Out-dipping)	C (psf) Ø (Degrees)	-- --						
SILTSTONE Cross-bed Strength (In-dipping)	C (psf) Ø (Degrees)	800 (1) 40						
Anisotropic Strength (Out-dipping)	C (psf) Ø (Degrees)	0 (1) 36						
CLAYSTONE Cross-bed Strength (In-dipping)	C (psf) Ø (Degrees)	800 (1) 40						
Anisotropic Strength (Out-dipping)	C (psf) Ø (Degrees)	0 (1) 36						
COMPACTED FILL	C (psf) Ø (Degrees)	600 (1) 26	400 38	500 (2) 31				
CLAY LINER	C (psf) Ø (Degrees)				2,200 (5)	1300 (peak) (6), 600 (residual)		
WASTE	Unit Weight (pcf) C (psf) Ø (Degrees)	85 (3) 700 0	85 (3) 0 27	110 (3) 0 28		31 (6)		
LINER SYSTEM INTERFACES Waste/Protective Soil	Ø (Degrees)	--			>>8°	31 (6)		
Protective Soil/Geotextile	Ø (Degrees)	--			>>8°			
Geotextile/Granular Layer	Ø (Degrees)	--			>>8°	>30		
Geotextile/Geonet	Ø (Degrees)	--			>12 (Residual)			
HDPE/Geonet - Smooth HDPE	Ø (Degrees)	--			8.5 (Residual)			
- Textured HDPE	Ø (Degrees)	--			--		23 (8)	
HDPE/Compacted Clay - Smooth HDPE	Ø (Degrees) τ (psf)	-- --			0 900	11 (6) 0	17 (9) --	
- Textured HDPE	Ø (Degrees) τ (psf)	-- --			-- --		26 (9) 0	27 (4) 0
Compacted Clay/Geotextile	Ø (Degrees)	--			24 (Residual)			

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(1) See Footnotes next page.

TABLE 3.3

GEOTECHNICAL PARAMETERS FROM PRIOR INVESTIGATIONS (Continued)

PARAMETER	PHASES II & III B-19 LANDFILL DESIGN	SEED, RAYMOND B. ET AL. INVESTIGATION OF PHASE I B-19 LANDFILL FAILURE	GENERIC EVALUATIONS OF KHF LANDFILL CLOSURES	EMPIRICAL OR MEASURED DATA FROM PUBLISHED LITERATURE
HDPE/Geotextile				
- Smooth HDPE Ø (Degrees)	--	8° (Residual)	9 (6)	9 (9)
- Textured HDPE Ø (Degrees)	--	--		32 (9)
HDPE/Soil				
- Smooth HDPE Ø (Degrees)	--	--		18,18,26,23,15,21,15
- Textured HDPE Ø (Degrees)		--		29.8,34,33,35,29,27, 26,25,35
HDPE/Geocomposite				
- Smooth HDPE Ø (Degrees)	--			11°, 16.6
- Textured HDPE Ø (Degrees)				42°, 38.5 (10)
II. CONSOLIDATION PROPERTIES				
SANDSTONE				
E (KSF)	--			
Recompression Ratio (Percent) (1)	0			
SILTSTONE				
e_o	.74			
E (KSF)	--			$(250\text{-}500)(\sigma_1 - \sigma_3)_f$ (11)
Recompression Ratio (Percent) (1)	1.4			
CLAYSTONE				
e_o	.93			
E (KSF)	--			$(250\text{-}500)(\sigma_1 - \sigma_3)_f$ (11)
Recompression Ratio (Percent) (1)	1.4			
COMPACTED FILL				
Typical Value of Compression (percent of original height)				4 (7)
CLAY LINER				
e_o		.82, .83, .87		
Second Compression Index			.005 to .03	
Cv (Ft2/Yr)		2		
WASTE				
Typical Value of Compression (percent or original height)			10,14.5 (6)	
Second Compression Index				.1 to .4 (7), .09 (12)

(1) Donohue 1988, Appendix F.
(2) EMCON (Pond P-9 levees). Reference 3.
(3) Donohue 1988, Appendix F, Figure V-9.
(4) Koerner, 1986.
(5) Seed, et al., 1988)
(6) Golder, 1988c.
(7) Navy, 1982.
(8) SLT, Friction Flex.
(9) Gundle, 1988a.
(10) Gundle, 1988c.
(11) Winterkorn et al., 1975.
(12) Yen, et al., 1975.
(13) Gundle, 1987.

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TABLE 3.4

LABORATORY TESTS OF BORING SAMPLES

BORING NO.	SAMPLE TYPE	SAMPLE NO.	SAMPLE DEPTH (Feet)	STRATIGRAPHIC UNIT	MATERIAL TYPE	INDEX PROPERTIES: VISUAL CLASSIFICATION	INDEX PROPERTIES: PLASTICITY INDEX	INDEX PROPERTIES: PERCENT PASSING NO. 200	INDEX PROPERTIES: SIEVE ANALYSIS/ HYDROMETER (5)	INDEX PROPERTIES: UNIT WEIGHT/ WATER CONTENT	INDEX PROPERTIES: SPECIFIC GRAVITY	COMPACTION PROPERTIES: STANDARD PROCTOR	COMPACTION PROPERTIES: MODIFIED PROCTOR	STRENGTH: UNCONSOLIDATED UNDRAINED (UU) TRIAXIAL (2)	STRENGTH: CONSOLIDATED UNDRAINED (CU) TRIAXIAL	STRENGTH: CONSOLIDATED DRAINED (CD) TRIAXIAL (16)	STRENGTH: UNSATURATED DIRECT SHEAR (1)	STRENGTH: EXTRUDE FOR MIX	MISC.: PERMEABILITY	MISC.: CONSOLIDATION	MISC.: SHRINK/SWELL POTENTIAL	GEOCHEMISTRY	REMARKS
L18-A	PB	S-1	5-6.1	18-10	SS	X			X														
	PB	S-2	10-12.5	18-10	CS																		
	PB	S-3	17-19.5	18-10	CS/SS	X		X		X													
	PB	S-4	25-27.5	18-10	SS	X		X															
	PB	S-5	33-33.5	18-10	SS																	X	
	PB	S-6	37-38.7	18-10	SS	X		X		X													
L18-B	PB	S-1	6-8.5	18-12	CS	X	X		X	X													
	PB	S-2	11.5-14	18-11	SS																		
	PB	S-3	20-22.5	18-11	SS	X				X				X	1 pt.		X	3 pts.					
	PB	S-4	28-30.5	18-11	CS/SLT																		
	PB	S-5	37-39.5	18-11	SS	X			X	X				X	2 pts		X						
L18-C	DR	B-1	6-7.5	Qc	cl	X	X			X													
	DR	B-2	12-13.5	Qc	cl																		
	PB	S-1	15-17.3	18-7	SLT	X	X		X	X													
	PB	S-2	20-22.5	18-7	SLT																	X	
	PB	S-3	25-27.5	18-7	SLT	X	X			X													
	PB	S-4	33-35	18-7	CS/SLT																		
	PB	S-5	41-43.1	18-7	SS	X				X													
	PB	S-6	48-50.1	18-7	SLT									X	1 pt.								
	PB	S-7	56-58	18-7	SLT	X	X		X					X			X	3 pts.					
	PB	S-8	64-66.3	18-7	SLT									X	2 pts.		X						
	PB	S-9	72-74.3	18-7	SS	X				X													
	PB	S-10	80-82	18-7	SLT/CS																		
	PB	S-11	87-89	18-7	SLT	X	X			X													
L18-D	DR	B-1	5-6.5	Qc	cl	X	X		X														
	DR	B-2	10-11.5	Qc	cl																		
	PB	S-1	15-12.3	18-5	SS																		
	PB	S-2	22-24.3	18-5	SLT	X	X		X					X	2 pts.		X						
	PB	S-3	29-31.5	18-4	CS/SLT									X	1 pt.								
	PB	S-4	36-38.5	18-4	CS/SLT	X				X													
	PB	S-5	44-46.5	18-4	CS																		
	PB	S-6	52-54.3	18-4	CS	X	X			X													
	PB	S-7	60-62.5	18-4	CS/SLT																		
	PB	S-8	68-70.5	18-4	CS																		
	PB	S-9	76-78.5	18-3	SS																		
	PB	S-10	81-83.5	18-3	SS	X		X		X													
	PB	S-11	83.5-86	18-3	SS																		
L18-E	DR	B-1	5-6.5	Qc	cl																		
	PB	S-1	10-12.5	Qc	cl	X				X													
	PB	S-2	15-17.5	18-3	SS	X		X															
	PB	S-3	20-22.5	18-3	SS																		
	PB	S-4	28-30.2	18-3	SS																		
	PB	S-5	35-37.5	18-3	SLT/CS																		
	PB	S-6	43-45	18-3	SLT	X	X			X													
	PB	S-7	51-53	18-2	CS																		
	PB	S-8	60-62.5	18-2	CS/SLT	X	X			X													
L18-F	PB	S-1	6-8.5	18-8	CS	X																X	
	PB	S-2	16-18.5	18-8	CS																		
	PB	S-3	26-28.5	18-8	CS	X	X		X	X													
	PB	S-4	36-38.5	18-8	CS																		
	PB	S-5	46-48.1	18-8	CS									X	1 pt.								
	PB	S-6	56-58.5	18-8	CS	X	X		X					X	2 pts		X						
L18-G	PB	S-1	6-8.5	18-13	SS	X		X		X													
	PB	S-2	14-16.5	18-13	SS																		
	PB	S-3	22-23.7	18-13	SS																		
	PB	S-4	30-32	18-13	SS									X	2 pts.								
	PB	S-5	40-41.3	18-13	SS	X		X						X	1 pt.								
	PB	S-6	45-47.5	8-12	CS									X	2 pts.								
	PB	S-7	50-51.8	8-12	CS	X	X		X					X	1 pt.								
	PB	S-8	60-62.5	8-12	CS																		
	PB	S-9	65-67	8-12	CS	X	X		X	X													
	PB	S-10	75-77.5	8-12	SS/CS																		
L18-H	PB	S-1	6-8.5	18-10	CS	X	X			X													
	PB	S-2	15-17.5	18-10	SS/CS	X		X		X													
	PB	S-3	25-27.5	18-10	CS																		
	PB	S-4	35-37.5	18-10	CS	X	X			X													
	PB	S-5	45-47.5	18-10	SS																		
	PB	S-6	55-57.5	18-10	CS	X	X			X													
L18-I	DR	B-1	6-7.5	Qc	cl																		
	PB	S-1	12-13.8	Qc	cl																		
	PB	S-2	16-18.5	18-8	CS	X	X		X	X													
TEST TOTALS FOR INITIAL BORINGS						35	20	8	13	30	–		–	6 (Series)			4		–		–	3	
ANTICIPATED TESTS FOR NEW BORINGS B18-J AND B18-K (Sheet 2)						21	16	5	5	19	2		9	–			–		9		11	–	

PB = Pitcher Barrel
DR = Drive Sample
Qc = Colluvium
CS = Claystone
CLT = Siltstone

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TABLE 2.4

LABORATORY TESTS OF BORING SAMPLES (Continued)

BORING NO.	SAMPLE TYPE	SAMPLE NO.	SAMPLE DEPTH (Feet)	STRATIGRAPHIC UNIT	MATERIAL TYPE	INDEX PROPERTIES: VISUAL CLASSIFICATION	PLASTICITY INDEX	PER CENT PASSING NO. 200	SIEVE ANALYSIS/ HYDROMETER (9)	UNIT WEIGHT/ WATER CONTENT	SPECIFIC GRAVITY	COMPACTION PROPERTIES: STANDARD PROCTOR	MODIFIED PROCTOR	STRENGTH: UNCONSOLIDATED (2) UNDRAINED (UU) TRIAXIAL	CONSOLIDATED UNDRAINED (CU) TRIAXIAL	CONSOLIDATED DRAINED (CD) TRIAXIAL	UNDRAINED DIRECT SHEAR (1)	RETRUDE FOR MIX	MISC: PERMEABILITY	CONSOLIDATION	SIEVE/SILT POTENTIAL	DISCONTINUITY	REMARKS
L18-J	BB	S-1	5-7.5	18-8	CS	X	X		X	X								X			X		
	PB	S-2	10-12.5	18-8	CS													X					
	PB	S-3	15-17.5	18-8	CS	X	X			X								X					
	PB	S-4	20-22	18-8	CS/SS																		
	PB	S-5	25-27.5	18-8	CS	X	X			X								X					
	PB	S-6	30-32.5	18-8	CS								(7)					X	(7)				
	PB	S-7	35-37.5	18-8	CS	X	X		X	X	X							X			X		
	PB	S-8	40-42.5	18-8	CS																		
	PB	S-9	45-47.5	18-8	CS	X	X			X								X					
	PB	S-10	50-52.5	18-8	CS													X					
	PB	S-11	55-57.5	18-8	CS	X	X			X								X					
	PB	S-12	60-62.5	18-8	CS													X					
	PB	S-13	65-67.5	18-8	CS	X	X			X								X					
	PB	S-14	70-72.5	18-8	CS								(8)					X	(8)				
	PB	S-15	75-77.5	18-8	CS	X	X		X	X								X					
	PB	S-16	80-82.5	18-8	CS																		
	PB	S-17	85-87.5	18-8	CS	X	X			X													
	PB	S-18	90-92.5	18-8	CS																		
	PB	S-19	95-97.5	18-8	CS/SS																		
	PB	S-20	100-101.5	18-8	CS	X				X													
	PB	S-21	105-107.5	18-8	CS																		
	PB	S-22	110-112.5	18-8	CS	X	X			X								X					
	PB	S-23	115-117.5	18-8	CS																		
	PB	S-24	120-122.5	18-8	CS	X	X		X	X								X			X		
	PB	S-25	125-127.5	18-8	CS								(7)						(7)				
	PB	S-26	130-132.5	18-8	CS	X	X			X								X					
	PB	S-27	135-137.5	18-8	CS																		
	PB	S-28	140-142.5	18-8	CS	X	X			X								X					
	PB	S-29	145-147.5	18-8	CS																		
	PB	S-30	150-152.5	18-8	CS	X	X			X													
	PB	S-31	155-157.5	18-8	CS																		
	PB	S-32	160-162.5	18-8	CS	X	X		X	X											X		
	PB	S-33	165-167.5	18-7	SL,T																		
	PB	S-34	170-172.5	18-7	SS																		
L18-K	PB	S-1	5-7.5	18-9	SS	X		X		X								X					
	PB	S-2	10-12.5	18-9	SS													X					
	PB	S-3	15-17.5	18-9	SS													X					
	PB	S-4	20-22.5	18-9	SS	X		X					(8)					X	(8)		(8)		
	PB	S-5	25-27.5	18-9	SS						X	X						X					
	PB	S-6	30-32.5	18-9	SL,T													X					
	PB	S-7	35-37.5	18-9	SS													X					
	PB	S-8	40-42.5	18-9	SS	X		X										X					
	PB	S-9	45-47	18-9	SL,T													X					
	PB	S-10	50-52.5	18-9	SL,T								(7)					X	(7)		(7)		
	PB	S-11	55-57.5	18-9	SL,T													X					
	PB	S-12	60-62.5	18-9	SL,T													X					
	PB	S-13	65-67.5	18-9	SS	X		X										X					
	PB	S-14	70-72.5	18-9	SS								(8)					X	(8)		(8)		
	PB	S-15	75-76.8	18-9	SS													X					
	PB	S-16	80-82.5	18-9	SS	X	X																
	PB	S-17	85-87.5	18-8	CS																		
	PB	S-18	90-92.5	18-8	CS																		
	PB	S-19	95-97.5	18-8	CS																		
TEST TOTALS FOR L18-J AND L18-K						21	16	5	5	19	2		9	-			-		5		10	-	
TEST TOTAL ALL BORINGS						56	36	13	18	49	2		9	4			4		5		10	3	

(1) as = 4, 8, 16 K.S.F.
(2) as = 4, 8, 12 K.S.F.
(3) Composite from samples S-1, S-2, S-3, S-5, S-6, and S-7 (pure claystone). Permeability at 90% relative compaction and optimum moisture content.
(4) Composite from samples S-9, S-10, S-11, S-12, S-13, S-14, and S-15 (pure claystone). Permeability at 90% relative compaction and 2% above optimum moisture content.
(5) Composite from samples S-22, S-24, S-26, S-28, and S-30 (pure claystone). Permeability at 90% relative compaction and optimum moisture content.
(6) Composite from samples S-1 through S-3 (sandstone). Run two series by mixing with claystone samples from Boring L18-J, at 30% and 70% sandstone (by weight). Permeability at 90% relative compaction and 2% above optimum moisture content.
(7) Composite from samples S-6 through S-10 (sandstone/siltstone). Run two series by mixing with claystone samples from Boring L18-J at 30% and 70% sandstone/siltstone (by weight). Permeability at 90% relative compaction and 2% above optimum moisture content.
(8) Composite from samples S-11 through S-15 (siltstone/sandstone). Run two series by mixing with claystone samples from Boring L18-J at 30% and 70% siltstone (by weight). Permeability at 90% relative compaction and 2% above optimum moisture content.
(9) Sieve analysis if percentage passing No. 200 sieve is less than 40%. Hydrometer analysis if greater than 40%.

TABLE 3.5

LABORATORY TESTS OF
TEST PIT SAMPLES

NUMBER	TEST PIT NO.	SAMPLE TYPE	SAMPLE NO.	SAMPLE DEPTH (Feet)	STRATIGRAPHIC UNIT	INDEX PROPERTIES						COMPACTION PROPERTIES		STRENGTH		MISCELLANEOUS		
						MATERIAL TYPE	VISUAL CLASSIFICATION	PLASTICITY INDEX	PERCENT PASSING NO. 200	SIEVE ANALYSIS/ (7) HYDROMETER	SPECIFIC GRAVITY	MODIFIED PROCTOR	STANDARD PROCTOR	UNCONSOLIDATED UNDRAINED (UU) TRIAXIAL	CONSOLIDATED UNDRAINED (CU) TRIAXIAL	PERMEABILITY	CONSOLIDATION (8)	SHRINK/SWELL POTENTIAL
1	DT-A	Sack	1	5	Colluvium	cl												
2	DT-A	Sack	2	5	18-8	CS	X	X		X		X		(1)(2) X	X	(2) X	(3)(5) X	(2) X
3	DT-C	Sack	2		Colluvium	cl												
4	DT-C	Sack	1	8	18-8	CS	X	X		X		X			X	(2) X	(3)(5) X	(2) X
5	TP-1	Sack	B-1	7	18-9	SS	X			X		X						
6	TP-2	Sack	B-1	10	Colluvium	cl	X		(6) X									
7	TP-3	Sack	B-1	15	Colluvium	cl												
8	TP-7	Sack	B-1	10	Colluvium	cl												
9	TP-8	Sack	B-1	7	Colluvium	cl	X		(6) X									
10	TP-11	Sack	B-1	7	Colluvium	S/C	X		X									
11	TP-13	Sack	B-1	4	Colluvium	cl	X		X									
12	TP-15	Sack	B-1	7	Colluvium	S/C												
13	TP-17	Sack	B-1	8	Colluvium	cl												
14	TP-18	Sack	B-1	2	Colluvium	cl	X		(6) X									
15	TP-18	Sack	B-2	12	Colluvium	C/S												
16	TP-23	Sack	B-1	0-4	Colluvium	S/M	X		(6) X									
17	TP-25	Sack	B-1	0-7	Colluvium	SC/S/M	X		(6) X			(6) X				(6) X		(6) X
18	TP-26	Sack	B-1	0-10.5	Colluvium	S/C	X		(6) X									
19	TP-27	Sack	B-1	0-12	Colluvium	S/C	X		(6) X									
20	TP-28	Sack	B-1	0-12.5	Colluvium	S/C												
21	TP-28	Sack	B-2	12.5-13.5	18-7	CS/SLT												
TEST TOTALS FOR INITIAL TEST PITS							12	2	9	3	3	4		2 (Series)		3	2	3
ANTICIPATED TESTS FOR TEST PITS TP-29 THROUGH TP-43							4	4	--	3	--	--		--		--	--	--

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TABLE 3.5

LABORATORY TESTS OF TEST PIT SAMPLES (Continued)

NUMBER	TEST PIT NO.	SAMPLE TYPE	SAMPLE NO.	SAMPLE DEPTH (Feet)	STRATIGRAPHIC UNIT	INDEX PROPERTIES						COMPACTION PROPERTIES		STRENGTH		MISCELLANEOUS		
						MATERIAL TYPE	VISUAL CLASSIFICATION	PLASTICITY INDEX	PERCENT PASSING NO. 200	SIEVE ANALYSIS/ (7) HYDROMETER	SPECIFIC GRAVITY	MODIFIED PROCTOR	STANDARD PROCTOR	UNCONSOLIDATED UNDRAINED (UU) TRIAXIAL	CONSOLIDATED UNDRAINED (CU) TRIAXIAL	PERMEABILITY	CONSOLIDATION (8)	SHRINK/SWELL POTENTIAL
22	TP-29	Sack	B-1	4-65	18-10	CS												
23	TP-30	Sack	B-1	7	18-11	SS/CS												
24	TP-31	Sack	B-1	3.5	18-12	CS	X	X										
25	TP-32	Sack	B-1	5	18-13	SS												
26	TP-33	Sack	B-1	8	18-13	SS												
27	TP-34	Sack	B-1	4	18-12	CS												
28	TP-35A	Sack	B-1	7	18-10	SS												
29	TP-36	Sack	B-1	4	18-8	CS	X	X		X								
30	TP-37	Sack	B-1	6	18-8	CS							X					
31	TP-38	Sack	B-1	9	18-8	CS	X	X		X								
32	TP-39	Sack	B-1	10	18-8	CS												
33	TP-40	Sack	B-1	3	18-8	CS	X	X		X								
34	TP-41	Sack	B-1	3.5	18-8	CS												
35	TP-42	Sack	B-1	6	18-9	SS	X			X	X	X		X				
36	TP-43	Sack	B-1	6	18-9	SS/SLT												
TEST TOTALS TP29 THROUGH TP-43							4	4	--	3	--	--		--	--		--	--
TEST TOTALS ALL TEST PITS							16	6	9	6		3		4	2 (Series)	3	2	3

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NOTES:

(1) σ_3 = 4, 8, 12 k.s.f.
(2) Sample at 90% relative compaction and optimum moisture content.
(3) If permeability criteria is satisfied, sample at 90% relative compaction and optimum moisture content.
(4) Sample at 95% relative compaction at optimum moisture content.
(5) Loading sequence: 0.5, 1, 2, 4, 8, 16, 32 k.s.f.
(6) Based on the percentage passing no. 200 test results prepare a composite sample using the three samples with smaller percentage of fines. Run compaction, permeability and shrink/swell potential at 90% relative compaction and optimum moisture content. Mixing with claystone samples will be considered upon review of preliminary results.
(7) Sieve analysis if percentage passing no. 200 sieve is less than 40%. Hydrometer analysis if greater than 40%.
(8) Time readings requested.

TABLE 3.6

GEOTECHNICAL/GEOCHEMICAL ANALYSIS METHODS

TYPE OF TEST	STANDARD
Geotechnical	
Moisture Content	ASTM D2216-80
Liquid and Plastic Limits	ASTM D4318-84
Shrinkage Limit	ASTM D427-83
Grain Size Analysis	ASTM D422-63
Specific Gravity	ASTM D854-83
Moisture Density Relations:	
• (Modified Proctor)	ASTM D1557-78, Method A
• (Standard Proctor)	ASTM D698-78, Method A
Direct Shear	ASTM D3080-72
Unconsolidated Undrained Triaxial	Corps of Engineers EM 1110-2-1906
Hydraulic Conductivity	Corps of Engineers EM 1110-2-1906
Background Geochemistry	
Priority Pollutants:	
• Volatile Organics	EPA 8240
• Semi-Volatile Organics	EPA 8270
• Pesticides	EPA 8080
California Regulated Metals	CCR Title 22 Section 66699
Total Organic Carbon	EPA 9060
Specific Conductance	EPA 9050
pH	EPA 9045
Nitrate	EPA 9200
Sulfate	EPA 9035
Cyanide	EPA 9010
Chloride	EPA 9250

TABLE 3.7

SUMMARY OF PLASTICITY INDEX DATA BY STRATIGRAPHIC UNIT

STRATIGRAPHIC UNIT	BORING NO.	SAMPLE NO.	DEPTH (ft.)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USC SYMBOL
COLLUVIUM	L18-C	B-1	6.0 - 7.5	35	19	CL
COLLUVIUM	L18-D	B-2	10.0 - 11.5	29	13	CL
18-2	L18-E	S-8	60.0 - 62.5	53	32	CH
18-3	L18-E	S-6	43.0 - 45.0	32	6	ML
18-4	L18-D	S-6	52.0 - 54.3	67	42	CH
18-5	L18-D	S-2	22.0 - 24.3	49	29	CL/CH
18-7	L18-C	S-1	15.0 - 17.3	38	17	CL
18-7	L18-C	S-3	25.0 - 27.5	46	23	CL
18-7	L18-C	S-7	56.0 - 58.0	60	36	CH
18-7	L18-C	S-11	87.0 - 89.0	41	17	CL
18-9	L18-K	S-10	80.0 - 82.3	30	11	CL
18-10	L18-H	S-1	6.0 - 8.5	81	50	CH
18-10	L18-H	S-4	35.0 - 37.5	78	51	CH
18-10	L18-H	S-6	55.0 - 57.5	71	49	CH
18-12	L18-B	S-1	6.0 - 8.5	64	36	CH
18-12	L18-G	S-7	50.0 - 51.8	78	49	CH
18-12	L18-G	S-9	65.0 - 67.0	60	36	CH
18-12	TP-31	B-1	3.5	70	49	CH
18-8	L18-F	S-1	6.0 - 8.5	78	55	CH
18-8	L18-F	S-3	26.0 - 28.5	69	57	CH
18-8	L18-F	S-6	56.0 - 58.5	59	39	CH
18-8	L18-I	S-2	16.0 - 18.5	58	36	CH
18-8	L18-J	S-1	5.0 - 7.5	55	30	CH
18-8	L18-J	S-3	15.0 - 17.5	67	41	CH
18-8	L18-J	S-5	25.0 - 27.5	79	50	CH
18-8	L18-J	S-7	35.0 - 37.5	64	40	CH
18-8	L18-J	S-9	45.0 - 47.5	74	49	CH
18-8	L18-J	S-11	55.0 - 57.5	69	46	CH
18-8	L18-J	S-13	65.0 - 67.5	74	50	CH
18-8	L18-J	S-15	75.0 - 77.5	33	17	CL
18-8	L18-J	S-17	85.0 - 87.5	56	38	CH
18-8	L18-J	S-22	110.0 - 112.5	71	47	CH
18-8	L18-J	S-24	120.0 - 122.5	70	42	CH
18-8	L18-J	S-26	130.0 - 132.8	88	60	CH
18-8	L18-J	S-28	140.0 - 142.5	85	53	CH
18-8	L18-J	S-30	150.0 - 152.5	88	62	CH
18-8	L18-J	S-32	160.0 - 162.5	55	30	CH

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TABLE 3.8

PERCENT PASSING NO. 200 SIEVE

STRATIGRAPHIC UNIT	MATERIAL TYPE	BORING/ TEST PIT NO.	SAMPLE NO.	DEPTH (ft.)	PERCENT PASSING NO. 200 SIEVE
Colluvium	Soil	TP-2	B-1	10	57
Colluvium	Soil	TP-8	B-1	7	81
Colluvium	Soil	TP-11	B-1	7	39
Colluvium	Soil	TP-13	B-1	4	48
Colluvium	Soil	TP-18	B-1	2	58
Colluvium	Soil	TP-23	B-1	0-4	33
Colluvium	Soil	TP-25	B-1	0-7	57
Colluvium	Soil	TP-26	B-1	0-10.5	52
Colluvium	Soil	TP-27	B-1	0-12	69
18-3	ss/slt	L18-D	S-10	81-83.5	24
18-3	ss	L18-E	S-2	15-17.5	24
18-9	ss	L18-K	S-1	5-7.5	11
18-9	ss/slt	L18-K	S-4	20-22.5	26
18-9	ss/slt	L18-K	S-8	40-42.5	21
18-9	ss	L18-K	S-13	65-67.5	19
18-10	slt/cs	L18-A	S-3	17-19.5	55
18-10	ss/slt	L18-A	S-4	25-27.5	73
18-10	cs/ss	L18-A	S-6	37-38.7	47
18-10	ss	L-18H	S-2	15-17.5	37
18-13	ss	L18-G	S-1	6-8.5	11
18-13	ss	L18-G	S-5	40-41.3	11

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TABLE 3.9

COMPACTION TEST RESULTS
(MODIFIED PROCTOR, ASTM D1557-78)

BORING NO./ TEST PIT NO.	COMPOSITE SAMPLE NO.	MATERIAL TYPE	SAMPLE PREPARATION	OPTIMUM MOISTURE CONTENT (%)	MAXIMUM DRY DENSITY (PCF)
1. Modified Proctor Test (ASTM D1557-78)					
L18-J	No. 1	Claystone	S-1, S-2, S-3, S-5, and S-6	22.8	98.7
L18-J	No. 2	Claystone	S-9, S-10, S-11, S-12, S-14, and S-15	21.7	100.7
L18-J	No. 3	Claystone	S-22, S-24, S-26, S-28, and S-30	24.9	96.2
L18-J/L18-K	No. 4	30% Sandstone/ 70% Claystone	S-1 through S-5, Boring L18-14 plus claystone from Boring L18-J	20.8	104.9
L18-J/L18-K	No. 5	50% Sandstone/ 50% Claystone	S-1 through S-5, Boring L18-K plus claystone from Boring L18-J	19.6	106.6
L18-J/L18-K	No. 6	30% Sandstone/ 70% Claystone	S-6 through S-10, Boring L18-K plus claystone from Boring L18-J	21.9	102.9
L18-J/L18-K	No. 7	50% Sandstone/ 50% Claystone	S-6 through S-10, Boring L18-K plus claystone from Boring L18-J	20.4	103.7
L18-J/L18-K	No. 8	30% Sandstone/ 70% Claystone	S-11 through S-15, Boring L18-K plus claystone from Boring L18-J	19.8	104.0
L18-J/L18-K	No. 9	50% Sandstone/ 50% Claystone	S-11 through S-15, Boring L18-K plus claystone from Boring L18-J	19.4	104.8
TP-11, TP-25, TP-26	No. 10	Colluvium (Silty Clay)	Mixture from all test pits	12.3	123.3
TP-42, B-1	—	Sandstone	—	15.0	114.8
DT-A, B-2	—	Claystone	—	21.5	104.2
DT-C, B-1	—	Claystone	—	23.5	99.0
2. Standard Proctor, ASTM D698					
TP-36, TP-37, TP-38	No. 11	Claystone	Mixture from all test pits	29.7*	87.7*
L18-J/L18-K	No. 4	30% Sandstone/ 70% Claystone	S-1 through S-5, Boring L18-K plus claystone from Boring L18-J	27.0*	94.2*

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TABLE 3.10

SUMMARY OF PERMEABILITY TESTS

TEST PIT/BORING NO.	COMPOSITE SAMPLE NO.	MATERIAL TYPE	STRATIGRAPHIC UNIT	OPTIMUM MOISTURE CONTENT (%)	MAXIMUM DRY DENSITY (PCF)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS	PERMEABILITY TEST										
									MOLD SIZE (INCHES)	MAXIMUM PARTICLE SIZE (INCHES)	INITIAL WATER CONTENT (%)	INITIAL DRY DENSITY (PCF)	INITIAL RELATIVE COMPACTION (%)	FINAL WATER CONTENT (%)	FINAL DRY DENSITY	FINAL RELATIVE COMPACTION	CONFINING PRESSURE (psi)	GRADIENT	K (cm/sec)
L18-J	2	Claystone	18-8	21.7	100.7	63	40	CH	2.5	3/8	22.4	91.6	91	33.6	90.1	89	40	41.2	2.0×10^{-8}
DT-A, B-2	--	Claystone	18-8	21.5	104.2	82	54	CH	2.5	3/8	24.0	93.3	90	29.5	96.4	93	40	82.4	1.9×10^{-9}
TP-36, TP-37, TP-38	11	Claystone	18-8	23.5	99.0	76	45	CH	4	1/4	28.7	89.4	90	33.2	90.4	91	40	82.4	4.1×10^{-9}
TP-36, TP-37, TP-38	11	Claystone	18-8	23.5	99.0	76	45	CH	4	1/4	28.6	89.5	90	33.7	89.7	91	40	82.4	5.0×10^{-9}
L18-J, S-21	--	Claystone	18-8	24.9	99.0	76	45	CH	4	(1)	31.2	89.3	90	34.4	88.5	89	40	82.4	2.1×10^{-8}

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(1) Weathered by repeated wetting and drying for two weeks. Maximum particle size resulted from the weathering process only.

TABLE 3.11

SUMMARY OF SWELL TEST RESULTS (UNDISTURBED SAMPLES)

BORING NO.	SAMPLE NO.	SAMPLE DEPTH (ft.)	STRATI-GRAPHIC UNIT	MATERIAL TYPE	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	NATURAL		APPLIED PRESSURE (PSF)	SWELL[1] (%)
							WATER CONTENT (%)	DRY DENSITY (PCF)		
18-J	S-1	5.0 - 7.5	18-8	Claystone	55	30	23.4	87.7	600	4.0
18-J	S-1	5.0 - 7.5	18-8	Claystone	55	30	20.6	89.6	12,000	-1.0
18-J	S-7	35.0 - 37.5	18-8	Claystone	64	24	25.0	96.0	600	1.8
18-J	S-7	35.0 - 37.5	18-8	Claystone	64	24	26.5	93.9	12,000	0.7
18-J	S-24	120.0 - 122.5	18-8	Claystone	70	42	29.4	93.2	600	1.6
18-J	S-24	120.0 - 122.5	18-8	Claystone	70	42	29.7	94.0	12,000	0.7
18-J	S-32	160.0 - 162.5	18-8	Claystone	55	30	21.2	96.7	600	0.4
18-J	S-32	160.0 - 162.5	18-8	Claystone	55	30	20.6	97.6	12,000	-0.4

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[1] As a percent of the sample height after application of the pressure.

TABLE 3.12

SUMMARY OF SWELL TEST RESULTS (REMOLDED SAMPLES)

BORING/ TEST PIT NO.	COMPOSITE SAMPLE NO.	MATERIAL TYPE	STRATI-GRAPHIC UNIT	SAMPLE PREPARATION	OPTIMUM MOISTURE CONTENT (%)	MAXIMUM DRY DENSITY (PCF)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS	INITIAL WATER CONTENT (%)	INITIAL DRY DENSITY (PCF)	INITIAL RELATIVE COMPACTION (%)	APPLIED PRESSURE (PSF)	SWELL[1] %
DT-A, B-2	--	Claystone	18-8	--	21.5	104.2	82	54	CH	24.3	93.1	89	0	22.0
	--	Claystone	18-8	--	21.5	104.2	82	54	CH	24.1	93.1	89	1,200	7.1
	--	Claystone	18-8	--	21.5	104.2	82	54	CH	27.3	93.1	89	6,000	1.0
DT-C, B-1	--	Claystone	18-8	--	23.5	99.0	76	45	CH	26.0	88.7	90	0	12.9
	--	Claystone	18-8	--	23.5	99.0	76	45	CH	25.6	88.8	90	1,200	3.7
		Claystone	18-8	--	23.5	99.0	76	45	CH	28.1	89.4	90	5,000	1.0
L18-J/L18-K	4	30% Sandstone/ 70% Claystone	18-9/18-8	S-1 through S-5. Boring L18-K plus claystone from Boring L18-J.	20.8	104.9	--	--	CL	23.3	94.1	90	0	9.7
	4	30% Sandstone/ 70% Claystone	18-9/18-8	S-1 through S-5. Boring L18-K plus claystone from Boring L18-J.	20.8	104.9	--	--	CL	23.0	94.2	90	1,200	2.7
L18-J/L18-K	6	30% Sandstone/ 70% Claystone	18-9/18-8	S-6 through S-10 Boring L18-K plus claystone from Boring L18-J.	21.9	102.9	--	--	CL	24.3	92.3	90	0	11.8
	6	30% Sandstone/ 70% Claystone	18-9/18-8	S-6 through S-10 Boring L18-K plus claystone from Boring L18-J.	21.9	102.9	--	--	CL	24.2	92.3	90	1,200	3.2
L18-J/L18-K	8	30% Sandstone/ 70% Claystone	18-9/18-8	S-11 through S-15 Boring L18-K plus claystone from Boring L18-J.	19.8	104.0	--	--		22.1	93.4	90	0	13.4
	8	30% Sandstone/ 70% Claystone	18-9/18-8	S-11 through S-15 Boring L18-K plus claystone from Boring L18-J.	19.8	104.0	--	--		21.5	93.8	90	1,200	3.9
TP-11, TP-25, TP-26	10	Colluvium (silty clay)	--	Mixture from all test pits.	12.3	123.3	--	--	CL	14.9	110.3	89	0	4.5
	10	Colluvium (silty clay)	--	Mixture from all test pits.	12.3	123.3	--	--		15.0	110.3	89	1,200	0.2

[1] As a percent of the sample height after application of pressure.

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TABLE 3.13

GEOTECHNICAL LABORATORY TEST RESULTS ON PECTEN CLAYSTONE
Source: Geosyntec (2008)

Boring ID	Sample ID	Depth (ft)		USCS	As-Received Conditions		Sieve	Atterberg Limits			Laboratory Compaction		Hydraulic Conductivity	
		From	To		Moisture Content (D2216) (%)	Dry Density (pcf)	Fines (%)	LL (%)	PL (%)	PI (%)	Maximum Dry Density (pcf)	Optimum Moisture (%)	(cm/s)	Test Conditions (MC, RC)
CS-1	1-2	10.5	12	CH - Fat Clay	16.0		92.2	100	35	65			7.0E-08	+2, 90%*
CS-1	1-3	15	16.5											
CS-1	1-4	20	21.5	CH - Fat Clay	18.5	100.6	96.0	88	32	56				
CA-1A	Bulk 1A-1	0	5	CH - Fat Clay with Sand	12.4		76.0	55	21	34	122	12.5	7.4E-09, 9.2E-09	+3, 92%, +3, 89%
CA-1A	1A-2	10.5	12		19.3	100.3								
CA-1A	1A-3	16	16.5	MH - Elastic Silt	20.3		98.1	97	44	53				
CA-1A	1A-5	25	26.5	CH - Fat Clay	20.9	98.6	93.3	105	41	64			6.2E-08	+2, 90%*
CA-1A	1A-6	30	31.5	MH - Elastic Silt	23.1								9.6E-08	+2, 90%*
CA-1A	1A-7	35	36.5				89.6	89	39	50				
CA-1A	1A-8	40	41.5	CH - Fat Clay	21.2		89.5	78	31	47				
CA-1A	tA-9	45	46.5	CH - Fat Clay	24.4	98.2	87.7	82	35	47				
CA-1A	1A-11	55	56.5	CH - Fat Clay with Sand	18.6	95.8	84.2	58	29	29				
CA-1A	1A-13	65	66	SC - Clayey Sand	3.4		17.4							
CS-2	Bulk 2-1	0	5	CH - Fat Clay	14.1		96.3	104	36	68	113.1	15.6	3.7E-09	+3, 92%
CS-2	Bulk 2-2	6.5	10.5											
CS-2	2-2	10.5	12	CH - Fat Clay	16.5	106.4	95.2	100	34	66				
CS-2	2-4	20	21.5	CH - Fat Clay	21.0		95.3	92	39	53			8.6E-08	+2, 90%*
CS-2	2-7	35	36.5	CH - Fat Clay	21.0	100.6	89.8	93	33	60				
CS-2	2-9	45	46.5	CH - Fat Clay	24.7	99.7	98.6	104	37	67				
CS-2	2-11	55	56.5	CH - Fat Clay	24.0		96.7	82	33	49			8.2E-08	+2, 90%*
CS-2	2-13	65	66	CH - Fat Clay	25.t		98.4	112	40	72			1.7E-08	+2, 90%*
CS-2	2-14	70	71.5			97.7								
CS-2	2-16	80	81.5	CH - Fat Clay	25.4		91.1	129	41	88			5.3E-08	+2, 90%*
CS-2	2-17	85	86.5											
TP-1	TP1-1	9	10	CH - Fat Clay	20.8		93.0	100	37	63	109.9	17.8	1.1E-08	+3, 92%
TP-2	TP2-1	0	8	CH - Fat Clay	14.6		97.8	92	31	61	111.7	16.2	5.0E-09	+3, 92%
TP-3	TP3-1	11	12	CH - Fat Clay	21.9		91.1	102	35	67	108.2	17.4	6.5E-09	+3, 92%
TP-4	TP4-1	9	10	CH - Fat Clay	17.0		88.5	93	34	59	105.5	20.1	t.4E-08	+3, 92%
													4.5E-08	

Note: *Samples collected with a California type sampler tested for hydraulic conductivity were compacted at 90% relative compaction of the maximum dry density of similar materials 1rom the test pad construction of Landfill B-18 Phases 1A/1B (Environmental Construction Services, 199 t)

TABLE 5.1

MATERIAL AND INTERFACE PROPERTIES USED FOR STABILITY CALCULATIONS

Material or Interface	Material/Interface Properties			Source/Remarks
	Total Unit Weight (pcf)	Cohesion Intercept, c (psf)	Friction Angle, ϕ (degrees)	
Used by ESI (1990a) for the analysis of temporary rock cut slopes, compacted fill slopes, and temporary Phase I intermediate fill slopes:				
Hazardous Waste and Operations Layer (Shallow Sliding)	115	300	27	conservative parameters due to lack of site-specific testing
Hazardous Waste and Operations Layer (Deep Sliding)	115	0	31	Golder (1989a), conservative
Bedrock (Cross Bedding Strength)	130	800	40	Donohue & Associates (1988) and direct shear test results in Appendix D.8
Bedrock (Along Bedding Strength)	130	0	36	Donohue & Associates (1988) and UU triaxial test results in Appendix D.5
Structural Fill	125	2,000	30	UU triaxial test results on compacted sandstone in Appendix D.5
Clay Liner and Clay Liner/Textured HDPE Geomembrane Interface (Long Term)	125	1,150	20	CU triaxial test results on compacted claystone in Appendix D.7
Clay Liner and Clay Liner/Textured HDPE Geomembrane Interface (Short Term or Low Confining Stress)	125	3,600	0	UU triaxial test results on compacted claystone in Appendix D.6
Textured HDPE Geomembrane/Geonet Interface	-	0	15	Gundle (1987a) and Geosyntec (1988)
Geonet/Geotextile (Heat Bonded) Interface	-	0	>30	Fluid Systems, Inc. (Appendix F)
Geotextile/Drainage Gravel Interface	-	0	>21	expected to be stronger than the geotextile/clay liner interface
Geotextile/Clay Liner Interface	-	0	21	Golder (1990b)
Textured HDPE Geomembrane/Geocomposite Interface	-	0	24	Golder (1990b)
Geocomposite/Drainage Gravel Interface	-	0	>21	expected to be stronger than the geotextile/clay liner interface
Geotextile/Operations Layer Interface (Shallow Sliding)	-	300	27	the strength of the operations layer controls
Geotextile/Operations Layer Interface (Deep Sliding)	-	0	31	the strength of the operations layer controls
Used by HAI (Appendix H.4) for the analysis of the temporary Phase IIIA intermediate fill slope:				
Hazardous Waste	115	0	31	ESI (1990a), Rust E&I (1998), URS (2005)
Bedrock	150	800	40	ESI (1990a), Rust E&I (1998), URS (2005)
Phase IIIA Liner Interface	-	0	22	HAI (Appendix H.4)

TABLE 5.1 (continued)

MATERIAL AND INTERFACE PROPERTIES USED FOR STABILITY CALCULATIONS

Material or Interface	Material/Interface Properties			Source/Remarks
	Total Unit Weight (pcf)	Cohesion Intercept, c (psf)	Friction Angle, ϕ (degrees)	
Used by HAI (Appendix H.5) for the analysis of the final closure configuration:				
Hazardous Waste	115	0	31	ESI (1990a), Rust E&I (1998), URS (2005)
Clay Liner	115	1,150	20	ESI (1990a), Rust E&I (1998), URS (2005)
Bedrock	150	800	40	ESI (1990a), Rust E&I (1998), URS (2005)
Phase I Bottom Liner Interface	-	0	17	ESI (1990a), Rust E&I (1998), URS (2005)
Phase I Sideslope Liner Interface	-	800	9	ESI (1990a), Rust E&I (1998), URS (2005)
Phase II Bottom Liner Interface	-	0	19	ESI (1990a), Rust E&I (1998), URS (2005)
Phase II Sideslope Liner Interface	-	0	19	ESI (1990a), Rust E&I (1998), URS (2005)
Phase III Liner Interface	-	0	12	HAI (Appendix H.5)
Vegetative Cover Soil/Geotextile Interface	110	100	21	HAI (Appendix H.5)
Geotextile/40-mil Textured HDPE Geomembrane Interface	110	0	25	HAI (Appendix H.5), conservative based on site-specific direct shear laboratory testing
40-mil Textured HDPE Geomembrane/Foundation Layer Interface	110	0	28	HAI (Appendix H.5)
Foundation Layer/Hazardous Waste Interface	110	0	31	HAI (Appendix H.5)

TABLE 5.2

SUMMARY OF SLOPE STABILITY ANALYSES

Case[1]	Static Factor of Safety[2]	Seismic Stability	
		Allowable Design Displacement for the MCE (inches)	Estimated Displacement for the MCE (inches)
Temporary Rock Cut Slopes (see Figure 5.1 and Appendix H.2):			
2H:1V Slopes Across Bedding Planes	2.4	1	0
3H:1V Slopes Subparallel to Bedding Planes	2.2	1	<0.1
Compacted Fill Slopes (see Appendix H.3):			
Northeast Containment Basin Embankment	2.2	3	0
Temporary Phase I Intermediate Fill Slopes (see Figure 5.2 and Appendix H.4):			
Wedge Sliding Along Landfill Base and Phase I/II Berm Slope	2.5	6	<0.1
Circular Sliding Entirely Through Waste	1.5	12	0.5
Temporary Phase IIIA Intermediate Fill Slope (see Figure 5.3 and Appendix H.4):			
Wedge Sliding Along Phase IIIA Liner System	1.5	N/A	N/A
Final Closure Configuration (see Figure 5.3 and Appendix H.5):			
3.5H:1V Cover Slopes Between Benches (i.e., Veneer Stability)	1.6	12	2.7
Wedge Sliding Along Base and Sideslope of Landfill	2.3	6	<1
Circular Sliding Entirely Through Waste	2.2	12	<12

Notes:

1. Several scenarios were analyzed for most of the cases but only the critical values (i.e., lowest factors of safety and highest seismic displacements) for each case are shown. See Appendix H for complete results.
2. The minimum acceptable static factor of safety is considered to be 1.5.
3. N/A = not analyzed.

TABLE 5.3

PERIMETER RUN-ON DIVERSION DITCH FLOW AND CAPACITY SUMMARY

DITCH LOCATION	ASPHALT V-DITCH DIMENSIONS		MINIMUM DITCH SLOPE (%)	PMP FLOW ESTIMATE (cfs)	DITCH CAPACITY (cfs)
	WIDTH (feet)	DEPTH (minimum)			
Phase I Area (see Sheet 5 in Appendix A.1)					
Brow Ditches	2	1	5.6	8.5	13.5
North Portion of West Perimeter Road	5	1	0.6	10.4	13.3
South Portion of West Perimeter Road	5	3	0.6	61.5	64.6
South Perimeter Road to South Borrow Pit Road	5	2	8.3	121.5	139.6
West Portion of North Perimeter Road	5	1.25	1.0	24.0	24.3
East Portion of North Perimeter Road	5	1.0	8.0	26.4	48.5
Phase II Area (see Sheet 10 in Appendix A.1)					
North Portion of South Perimeter Road	5	2.5	3.6	121.5	124.8

89-977 (8/14/90)

TABLE 5.4

ACTIVE ROAD RUN-OFF CONTROL SUMMARY

STRUCTURE	STRUCTURE DESCRIPTION	MINIMUM SLOPE (%)	ESTIMATED FLOW (cfs)		FLOW CAPACITY (cfs)
			25-YEAR STORM	PMP	
I. **Phase I** (see Sheet 5 in Appendix A.1)					
Inside Perimeter Road V-ditch	5-feet wide x 1.25-feet deep	0.6	N/A	14.4	18.8
Concrete-enclosed CMP pipe along West Perimeter Road at access ramp.	12-inch diameter	0.6	2	N/A	4.7
CMP culvert down south landfill slope to the Phase I/II Berm.	18-inch diameter	50	N/A	14.4	15.1
Concrete-encased CMP pipe at Phase I/II Berm Access Road.	18-inch diameter	0.5	3.9	N/A	4.8
Top of Phase I/II Berm V-Ditch.	5-feet wide x 1.25-foot deep	0.5	N/A	14.4	17.2
CMP culvert beneath Phase I/II Berm crest toward the clay pit retention basin.	30-inch diameter	0.5	N/A	15.6	33.8
Corrugated pipe at top of Phase I/II Berm to convey run-off into the clay pit containment basin.	30-inch diameter	50	N/A	15.6	33.8
Corrugated pipe to convey run-off from North Bench Road into the clay pit.	18-inch diameter	50	N/A	7.8	8.5
II. **Phase II** (see Sheet 10 in Appendix A.1)					
CMP culverts to convey run-off from Bench Road to northeast containment basin at the following locations:					
- End of South Bench Road	30-inch diameter	0.5	N/A	21.0	33.9
- End of North Bench Road	24-inch diameter	0.5	N/A	11.2	21.3

89-977 (8/14/90)

N/A = Not Applicable

TABLE 5.5

CONTAINMENT BASIN CAPACITY SUMMARY

Basin	Estimated 24-hour PMP Run-off Volume (acre-feet)	Basin Capacity (acre-feet)
Temporary Basins:		
Phase I Containment Basin (see Sheet 6 in Appendix A.1)	8	33
Phase I Intermediate Closure Basin (see Sheet 6 in Appendix A.1)	5	5
Permanent Basins:		
Northeast Containment Basin (see Sheet C-3 in Appendix A.2)	34*	33
South Containment Basin (see Sheet C-3 in Appendix A.2)	32	48

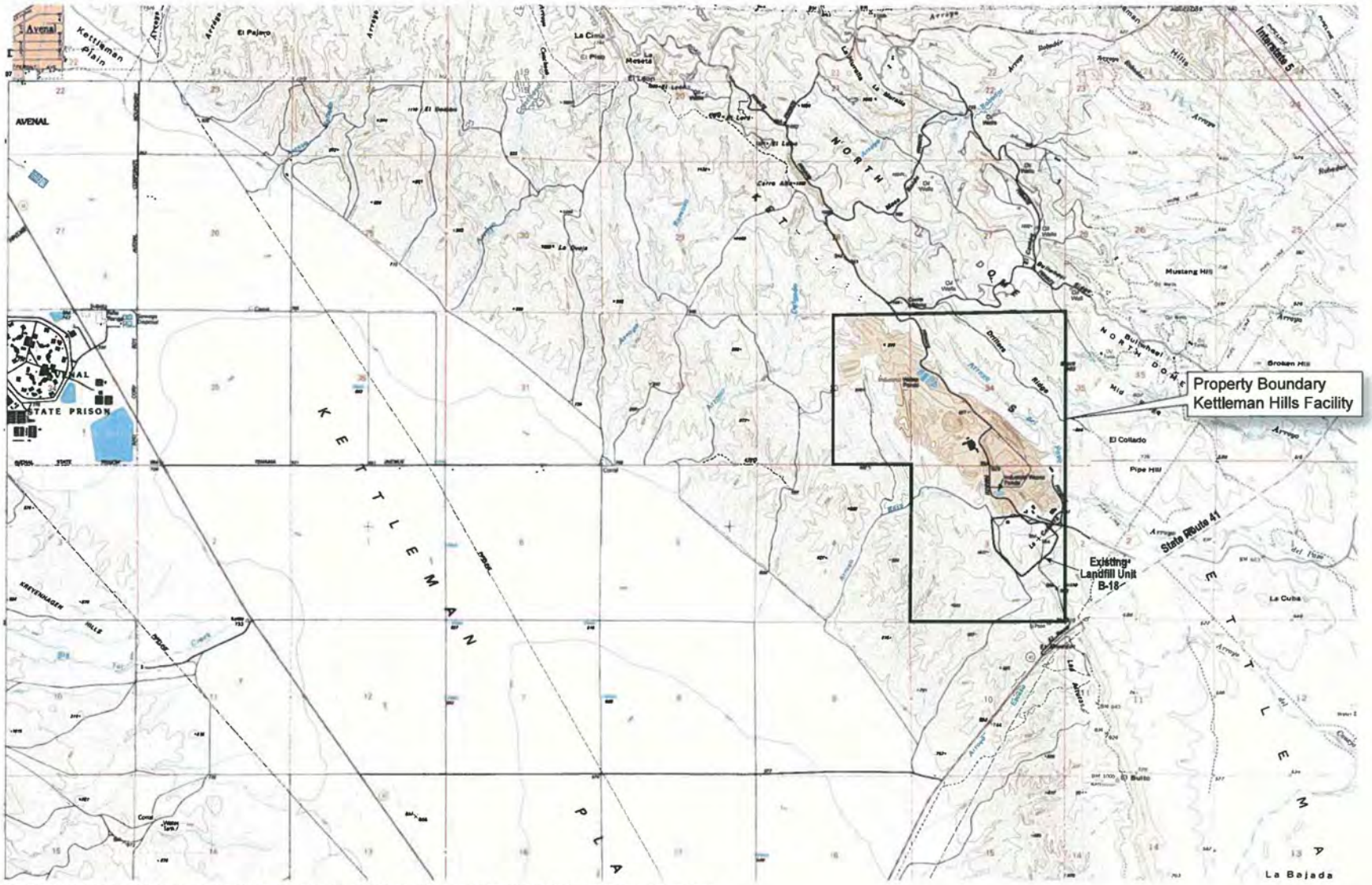
*will require pumping to South Containment Basin to maintain full containment of run-off

TABLE 5.6

FINAL CLOSURE RUN-OFF CONTROL SUMMARY

Structure	Dimensions		Minimum Slope (%)	Estimated Peak PMP Flow (cfs)	Maximum Flow Depth (feet)
	Bottom Width (feet)	Depth (feet)			
Final Drainage Benches (see Detail 2 on Sheet C-10 in Appendix A.2): trapezoidal, earthen	12	2	2	51	0.8
Cover Access Road (see Detail 5 on Sheet C-10 in Appendix A.2): Asphalt-Lined V-Ditch + Earthen Road	43.5	2 to 3.5	8	104	1.6
New B-18 Perimeter Road (see Detail 4 on Sheet C-7 in Appendix A.2): Asphalt-Lined Trapezoidal Ditch + Earthen Road + Earthen V-Ditch/2-Foot-Tall Earthen Berm)	37	2 to 3.5	1.4	270	2.9

FIGURES



Source: USGS La Cima (1978), Las Viejos (1981), Kettleman City (1981) and Kettleman Plain (1994)1:24,00 Scale Topographic Maps.

0 0.5 Scale in Miles

0 1 Scale in Kilometers

FIGURE **1.1**

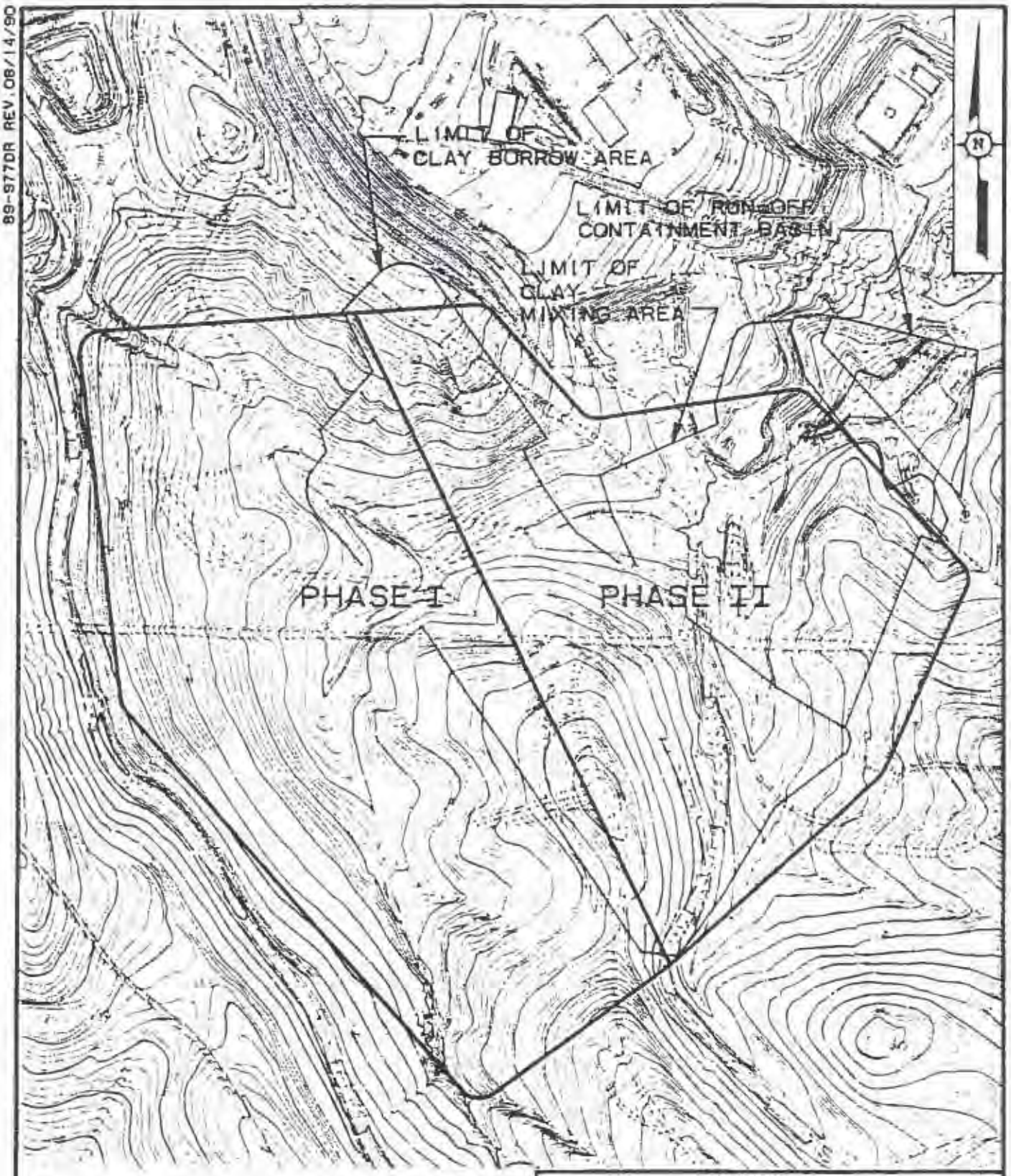
SITE VICINITY MAP

WM/LANDFILL UNIT B-18 KETTLEMAN HILLS FACILITY/CA

0839188700fig1.1.ai | Mod: 08/12/08 | JDD

Golder Associates

89-9770R REV. 06/14/90



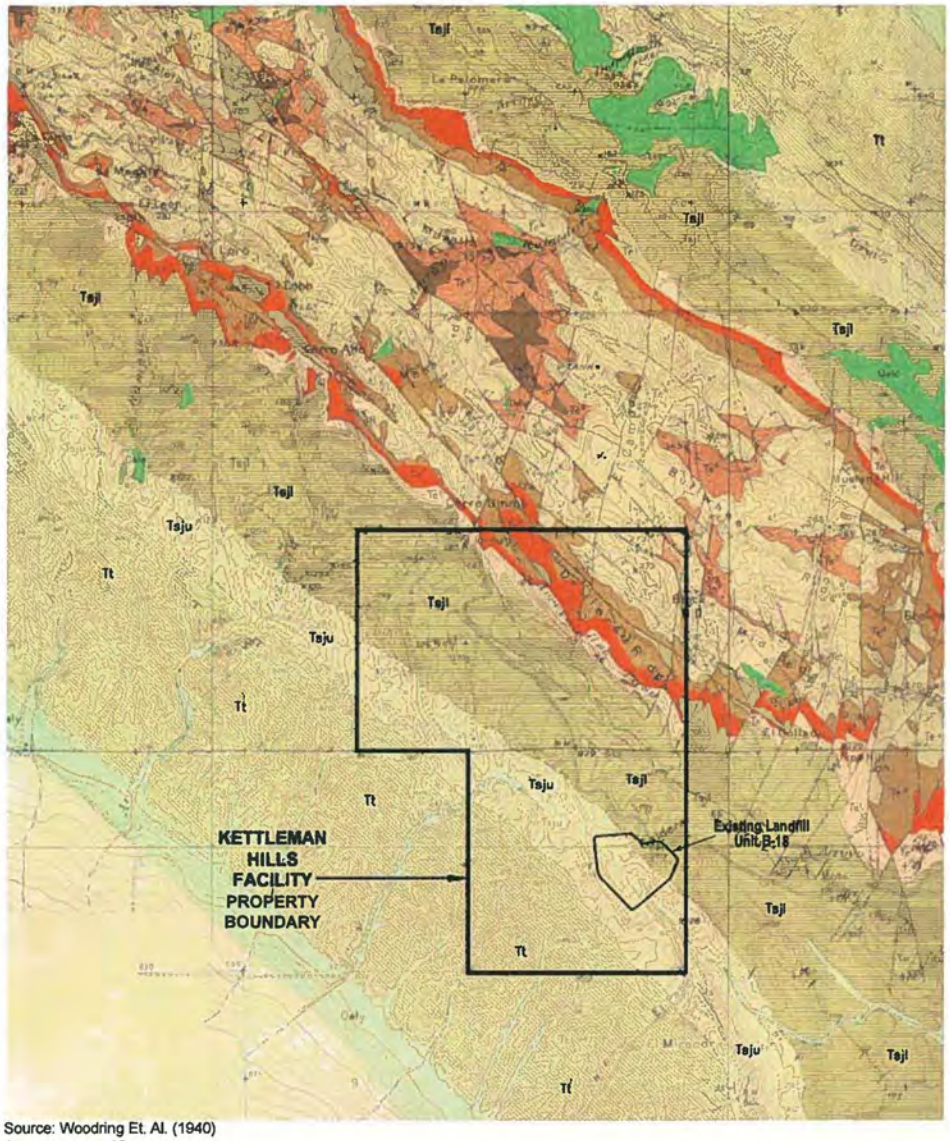
0 300 600 FEET
SCALE

FIGURE 2.1

LANDFILL B-18
PRE-DEVELOPMENT
TOPOGRAPHY

LANDFILL UNIT B-18
KETTLEMAN HILLS FACILITY

ENVIRONMENTAL SOLUTIONS, INC.



Source: Woodring Et. Al. (1940)

0 0.5
Scale in Miles

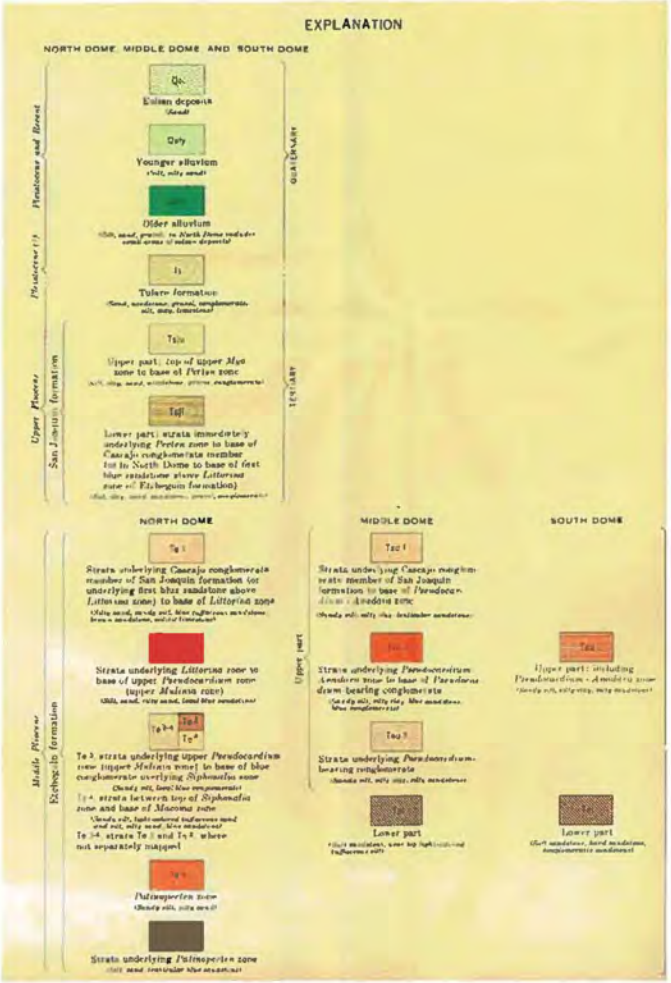


FIGURE **2.2**
AREA GEOLOGIC MAP
WM/LANDFILL UNIT B-18 KETTLEMAN HILLS FACILITY/CA

0839188700 0fig2.2.ai | Mod: 08/12/08 | JDD

Golder Associates

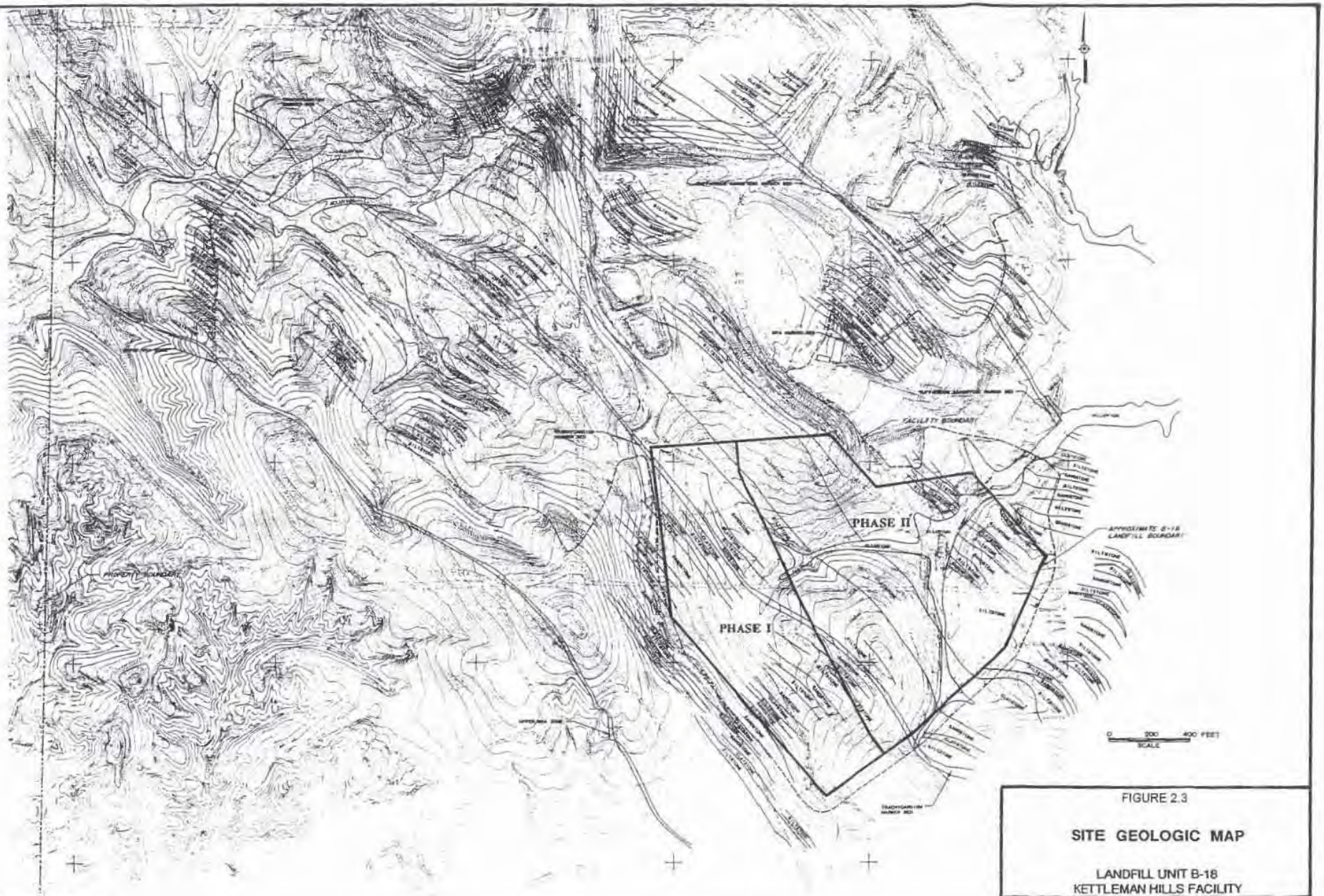


FIGURE 2.3

SITE GEOLOGIC MAP

LANDFILL UNIT B-18
KETTLEMAN HILLS FACILITY

ENVIRONMENTAL SOLUTIONS, INC.

89-977 DR REV. 8/8/90

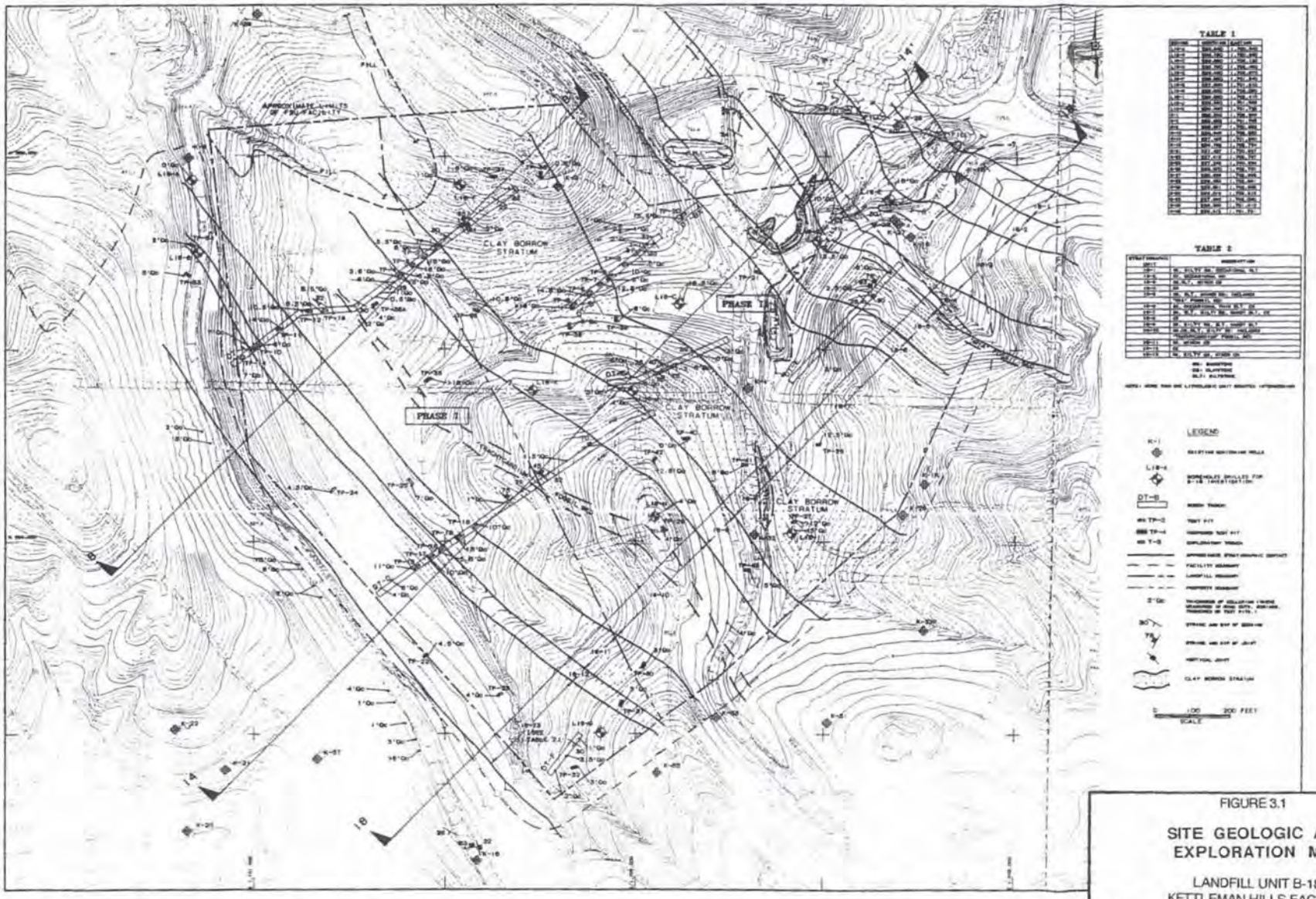


FIGURE 3.1

SITE GEOLOGIC AND EXPLORATION MAP

LANDFILL UNIT B-18
KETTLEMAN HILLS FACILITY

ENVIRONMENTAL SOLUTIONS, INC.

9-977 DR REV. 6/99

89-077 OR REV. 8/8/90

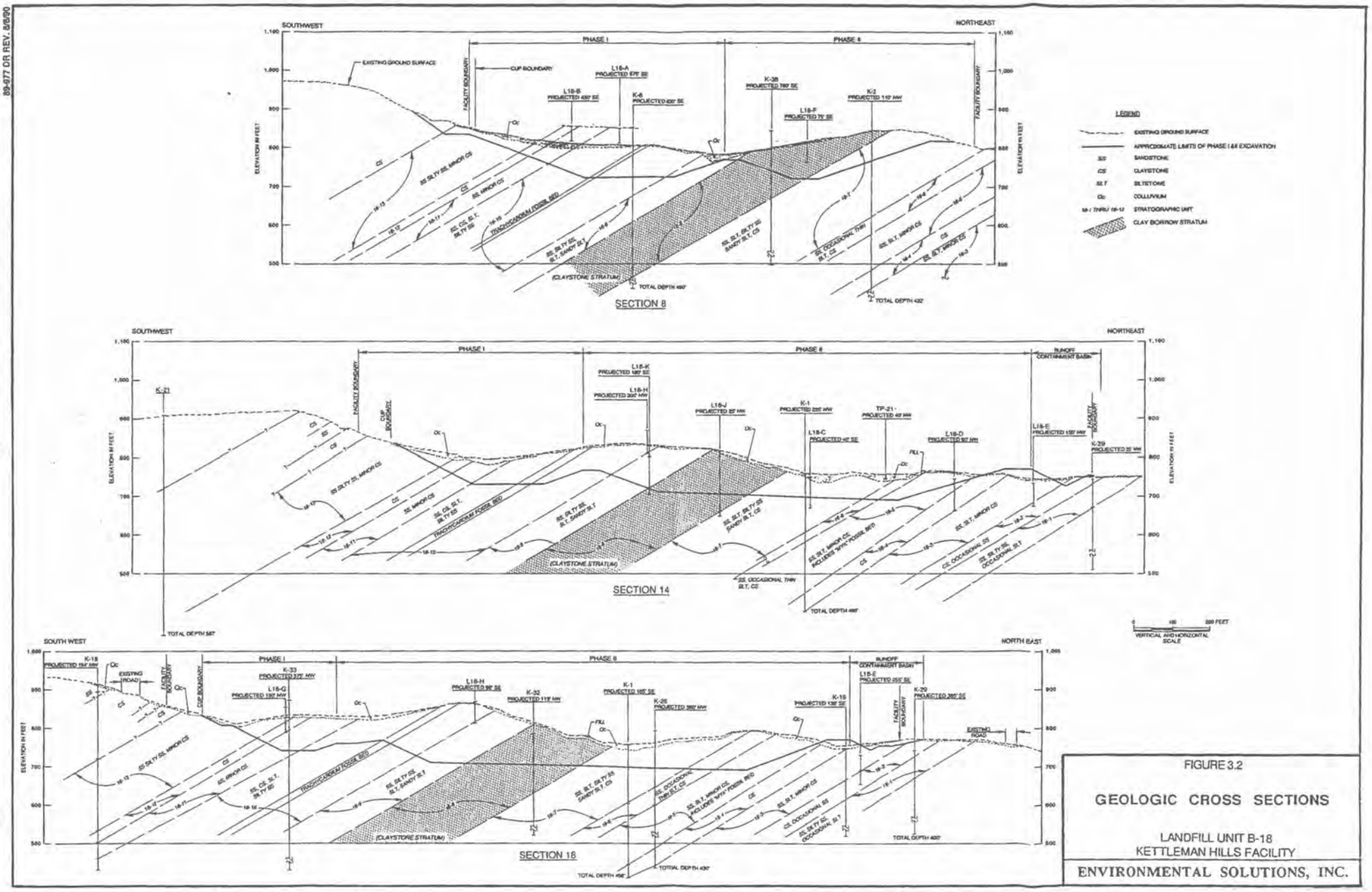


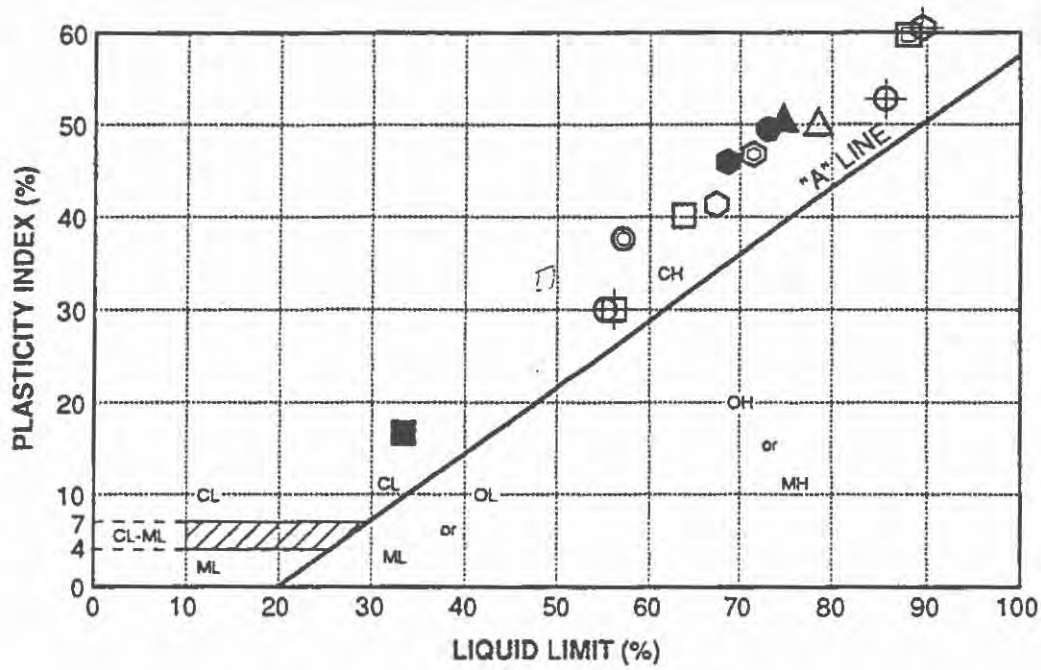
FIGURE 3.2

GEOLOGIC CROSS SECTIONS

LANDFILL UNIT B-18
KETTLEMAN HILLS FACILITY

ENVIRONMENTAL SOLUTIONS, INC.

89-97DR/Fig 3.3/PICh/U18-8/... V. 8/7/90



SYMBOL	STRATIGRAPHIC UNIT	BORING NO.	SAMPLE NO.	DEPTH (FT.)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USC SYMBOL
○	18-8	L18-J	S-1	5.0 - 7.5	55	30	CH
⬡	18-8	L18-J	S-3	15.0 - 17.5	67	41	CH
△	18-8	L18-J	S-5	25.0 - 27.5	79	50	CH
□	18-8	L18-J	S-7	35.0 - 37.5	64	40	CH
●	18-8	L18-J	S-9	45.0 - 47.5	74	49	CH
⬢	18-8	L18-J	S-11	55.0 - 57.5	69	46	CH
▲	18-8	L18-J	S-13	65.0 - 67.5	74	50	CH
■	18-8	L18-J	S-15	75.0 - 77.5	33	17	CL
◎	18-8	L18-J	S-17	85.0 - 87.5	56	38	CH
⬡	18-8	L18-J	S-22	110.0-112.5	71	47	CH
◬	18-8	L18-J	S-24	120.0-122.5	70	42	CH
▣	18-8	L18-J	S-26	130.0-132.8	88	60	CH
⊕	18-8	L18-J	S-28	140.0-142.5	85	53	CH
⊕	18-8	L18-J	S-30	150.0-152.5	88	62	CH
⊞	18-8	L18-J	S-32	160.0-162.5	55	30	CH

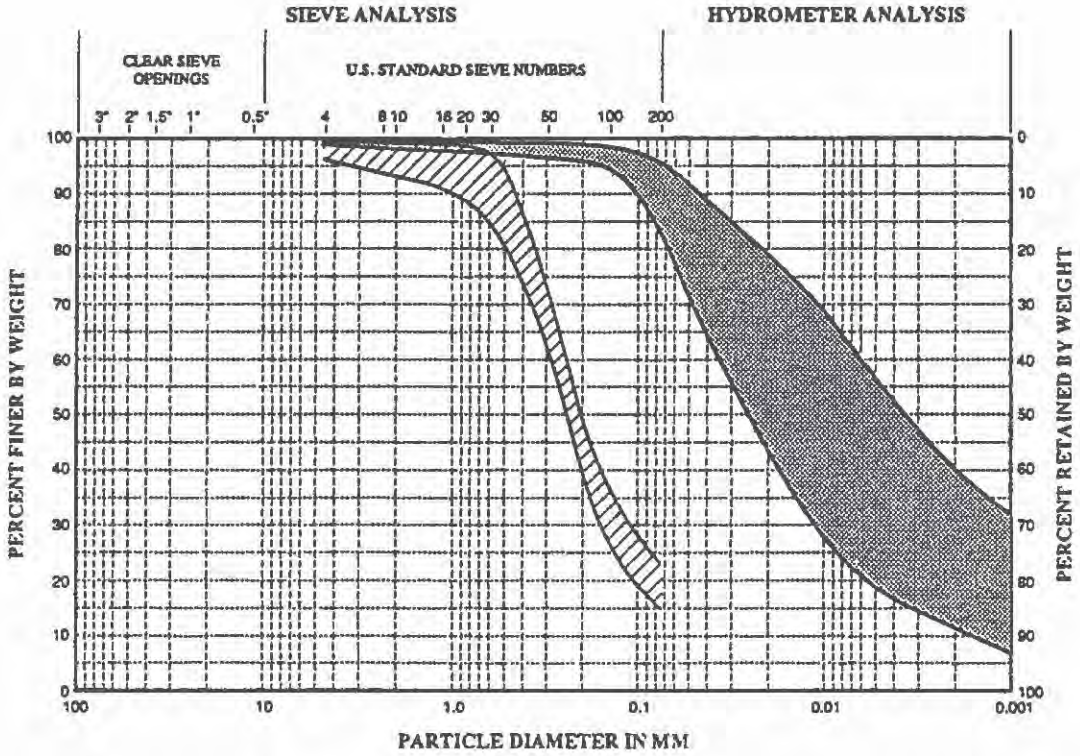
FIGURE 3.3

PLASTICITY CHART
STRATIGRAPHIC UNIT 18-8

LANDFILL UNIT B-18
KETTLEMAN HILLS FACILITY

ENVIRONMENTAL SOLUTIONS, INC.

89-977DR/Fig 3.4/GrSiCuEn REV. 8/14/90



COBBLES	GRAVEL		SAND			SILT AND CLAY FRACTION
	coarse	fine	coarse	medium	fine	

LEGEND

CLAYSTONE

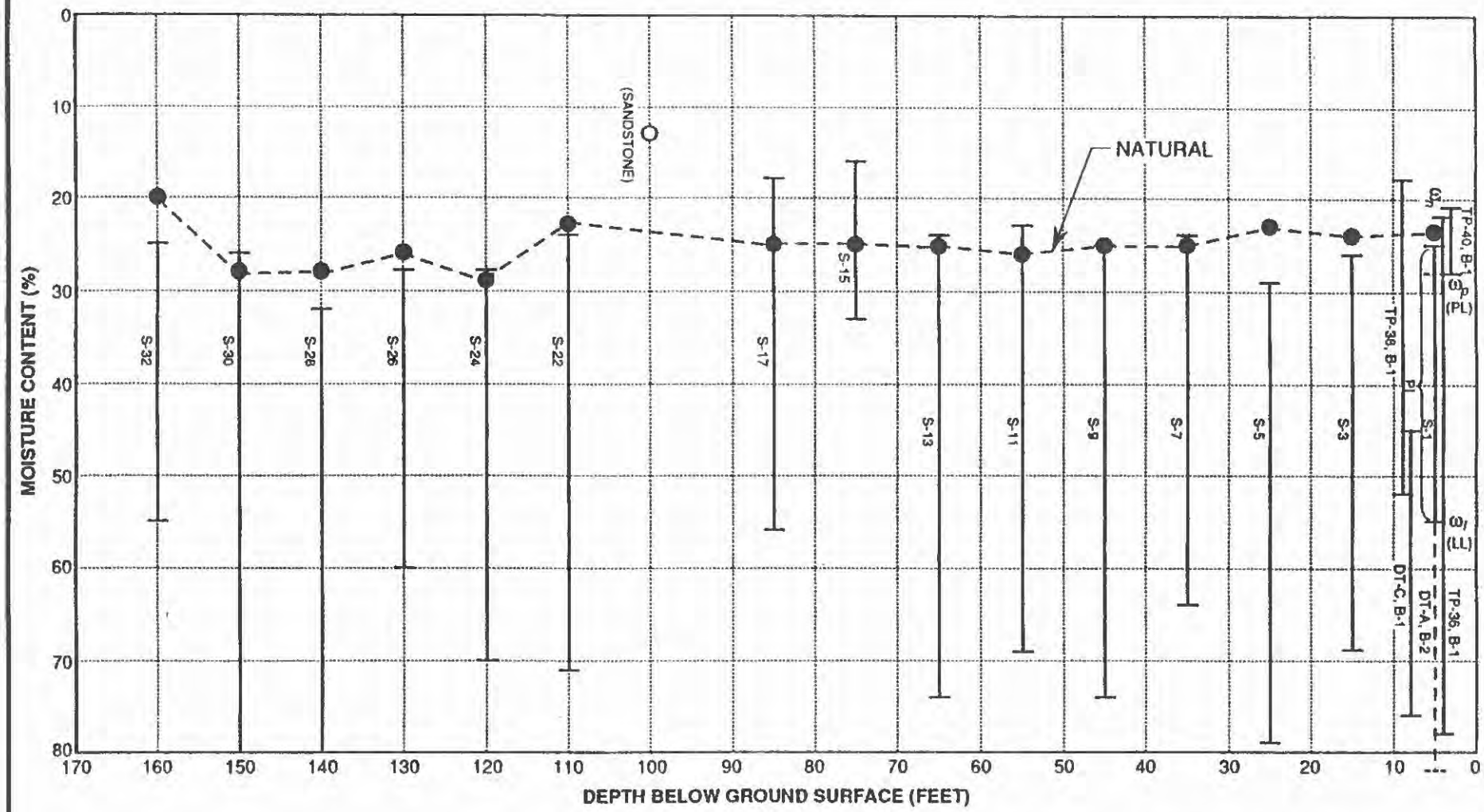
SANDSTONE

FIGURE 3.4

GRAIN SIZE CURVE ENVELOPES

LANDFILL UNIT B-18
KETTLEMAN HILLS FACILITY

ENVIRONMENTAL SOLUTIONS, INC.



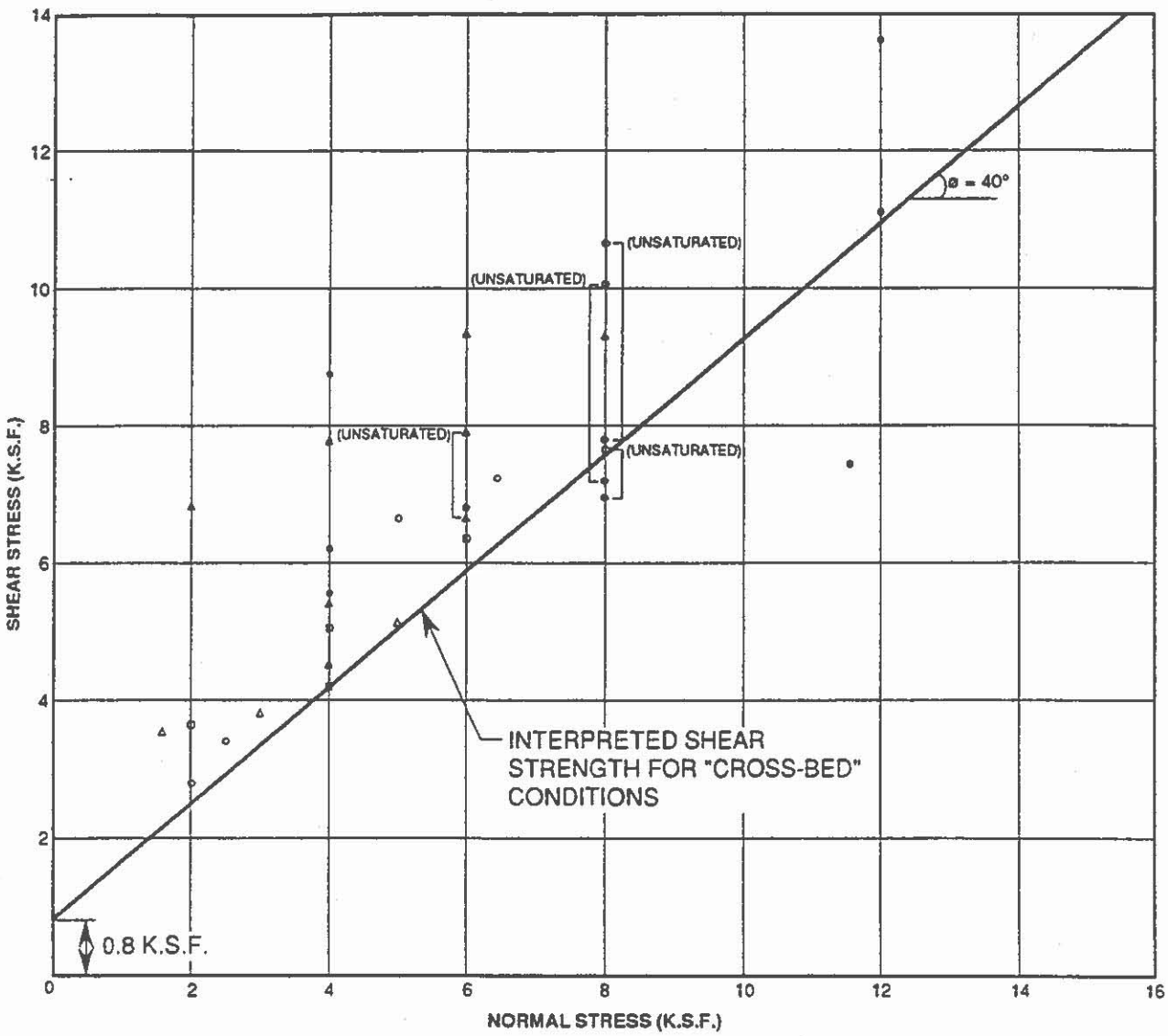
LEGEND

SHALLOW TEST PIT SAMPLE

FIGURE 3.5
MATERIAL WATER CONTENT VS. PLASTICITY INDEX
STRATUM 18-8
LANDFILL UNIT B-18
KETTLEMAN HILLS FACILITY

ENVIRONMENTAL SOLUTIONS, INC.

89-977DR/Fig 3.6/SuD/L...Re REV. 8/7/90



CURRENT INVESTIGATION (B-18, PHASES I & II)

- ▲ L18-B, S-3 @20-22.5' (SANDSTONE)
- • L18-C, S-7 @56-58' (CLAYSTONE)
  L18-D, S-2 @22-24.5' (CLAYSTONE)
  L18-F, S-6 @56-58.5' (CLAYSTONE)

PREVIOUS INVESTIGATION (B-19, PHASES II & III)

- ○ CLAYSTONE - DIRECT SHEAR
- □ SILTSTONE - DIRECT SHEAR
- △ SANDSTONE - DIRECT SHEAR

FIGURE 3.6

SUMMARY
DIRECT SHEAR TEST RESULTS

LANDFILL UNIT B-18
KETTLEMAN HILLS FACILITY

ENVIRONMENTAL SOLUTIONS, INC.

89-977DR-01 REV. 8/14/90

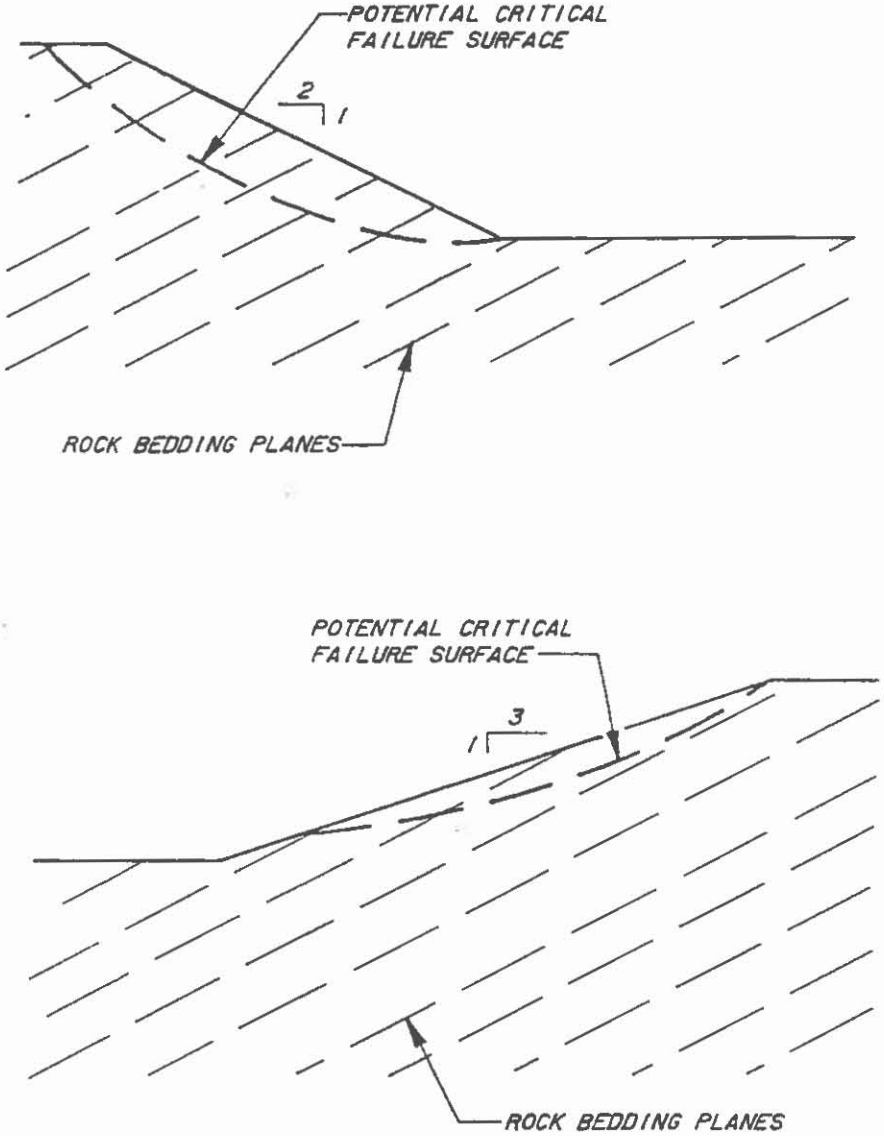
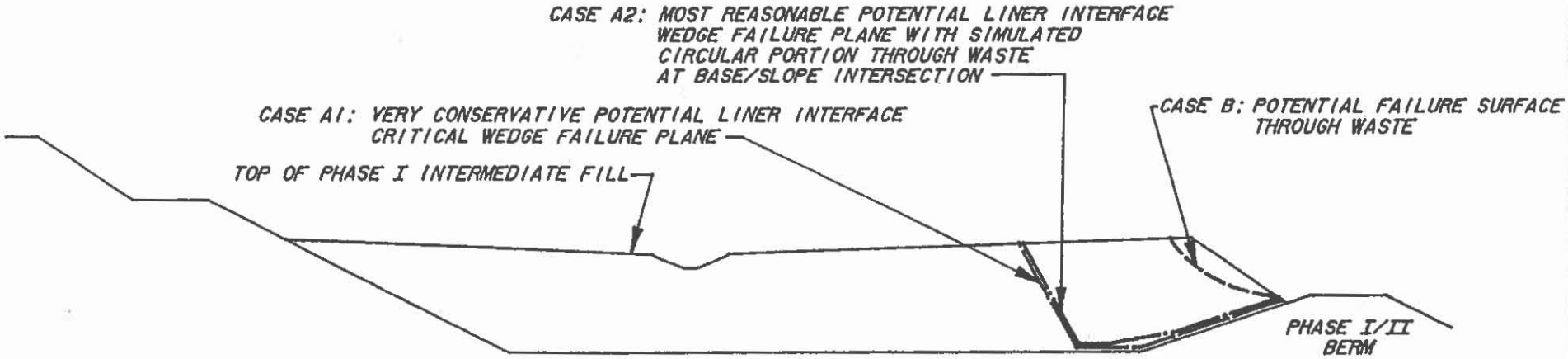


FIGURE 5.1

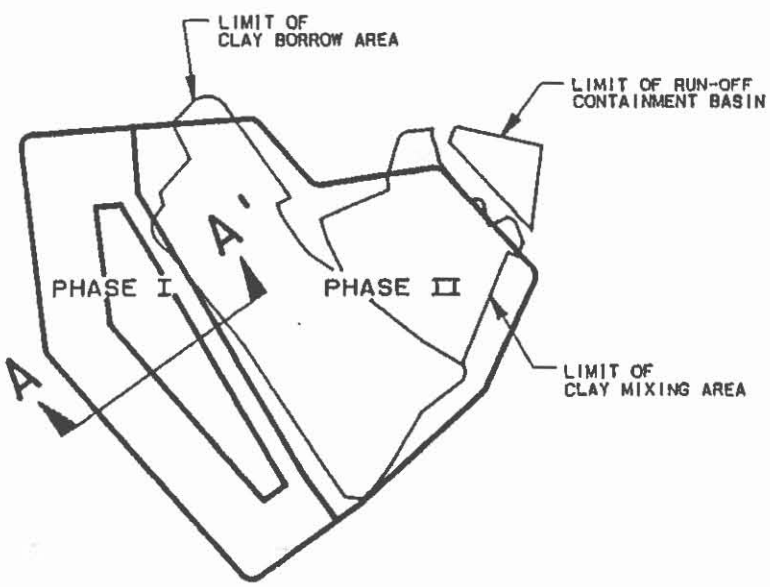
STABILITY CONSIDERATIONS FOR ROCK CUT SLOPES

LANDFILL UNIT B-18
KETTLEMAN HILLS FACILITY

ENVIRONMENTAL SOLUTIONS, INC.



SECTION A-A'



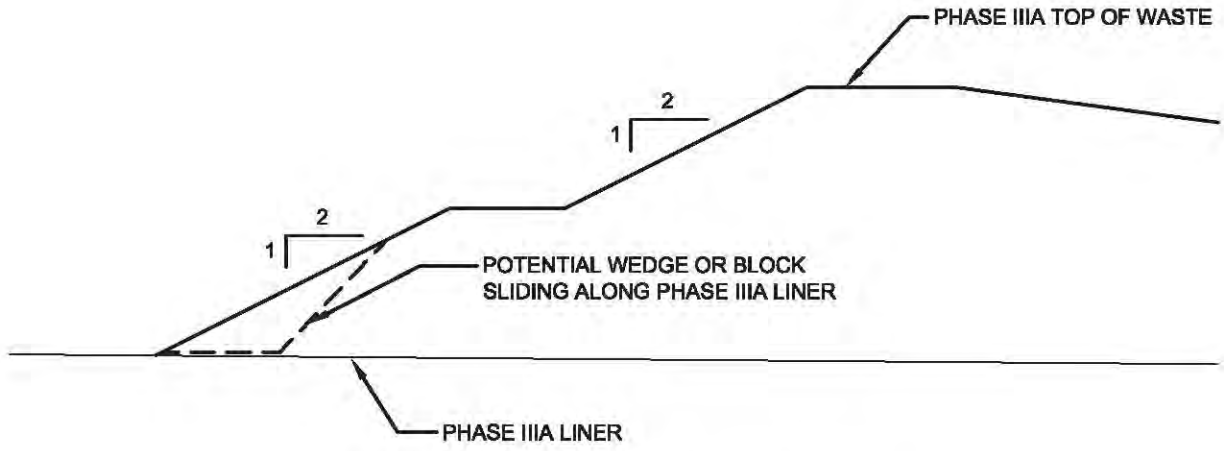
KEY PLAN

FIGURE 5.2

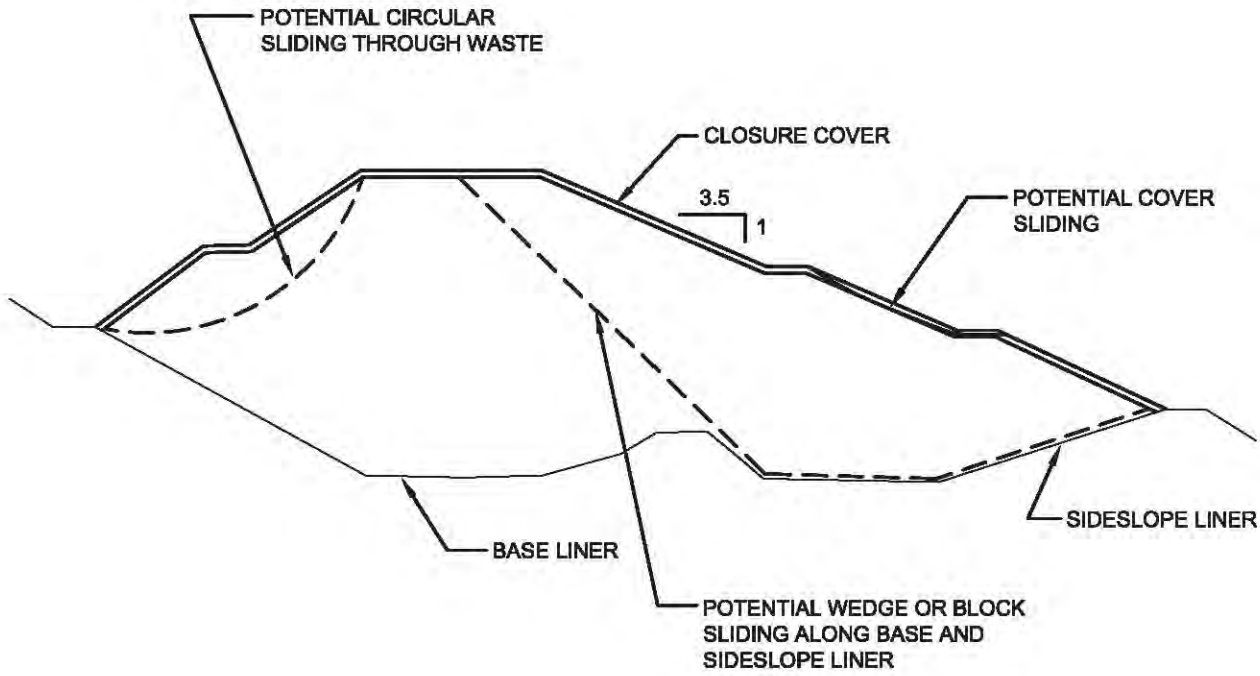
STABILITY CONSIDERATIONS FOR PHASE I INTERMEDIATE FILL SLOPE

LANDFILL UNIT B-18
KETTLEMAN HILLS FACILITY

ENVIRONMENTAL SOLUTIONS, INC.



A. PHASE IIIA INTERMEDIATE FILL SLOPE



B. FINAL CLOSURE

FIGURE 5.3

POTENTIAL SLIDING MECHANISMS FOR PHASE IIIA INTERMEDIATE FILL SLOPE AND FINAL CLOSURE

Golder Associates
Irvine, California

PROJECT No083-91887 FILE No. 08391887 5.3 CADD KJK DATE 05/02/11

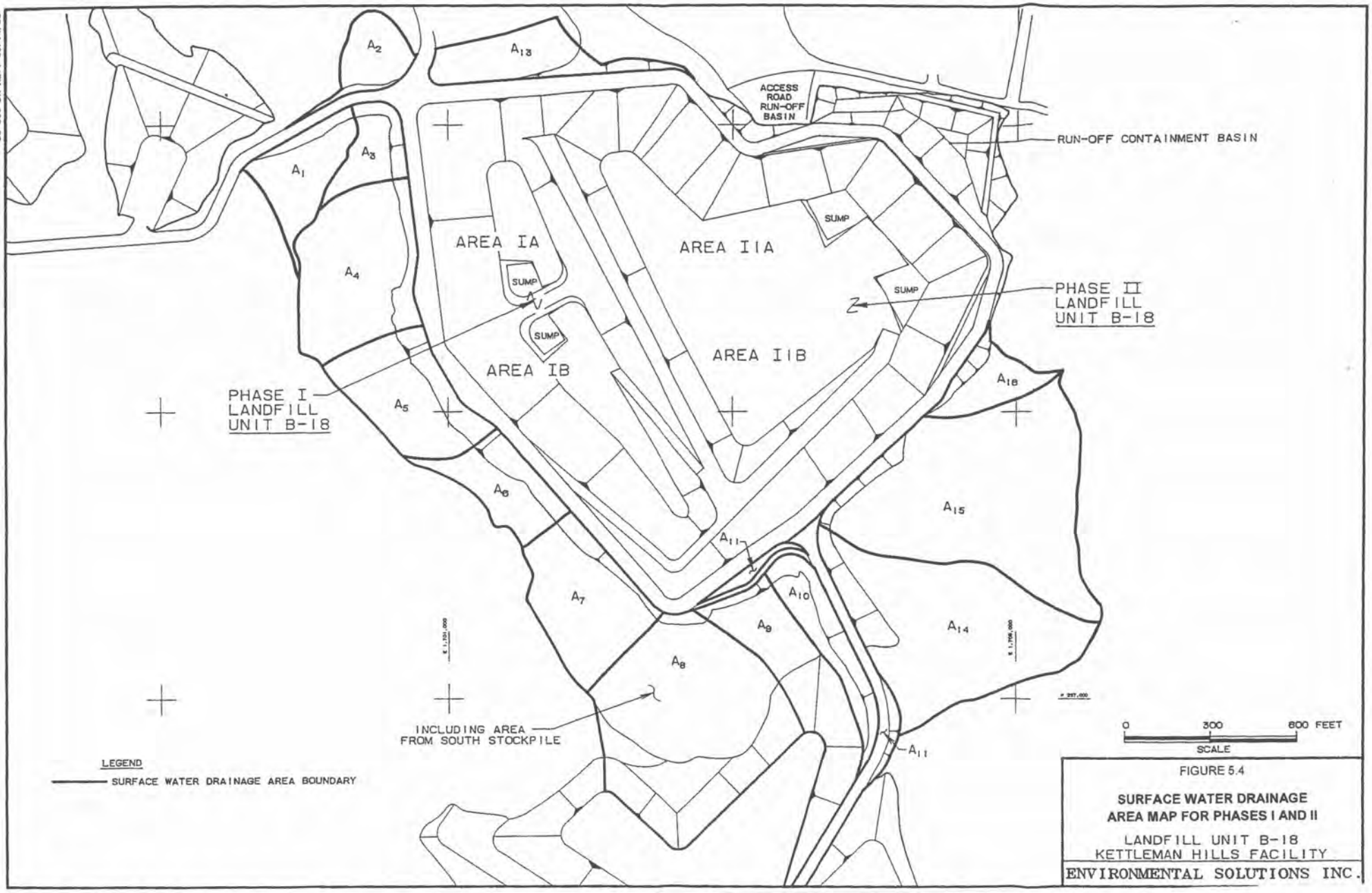


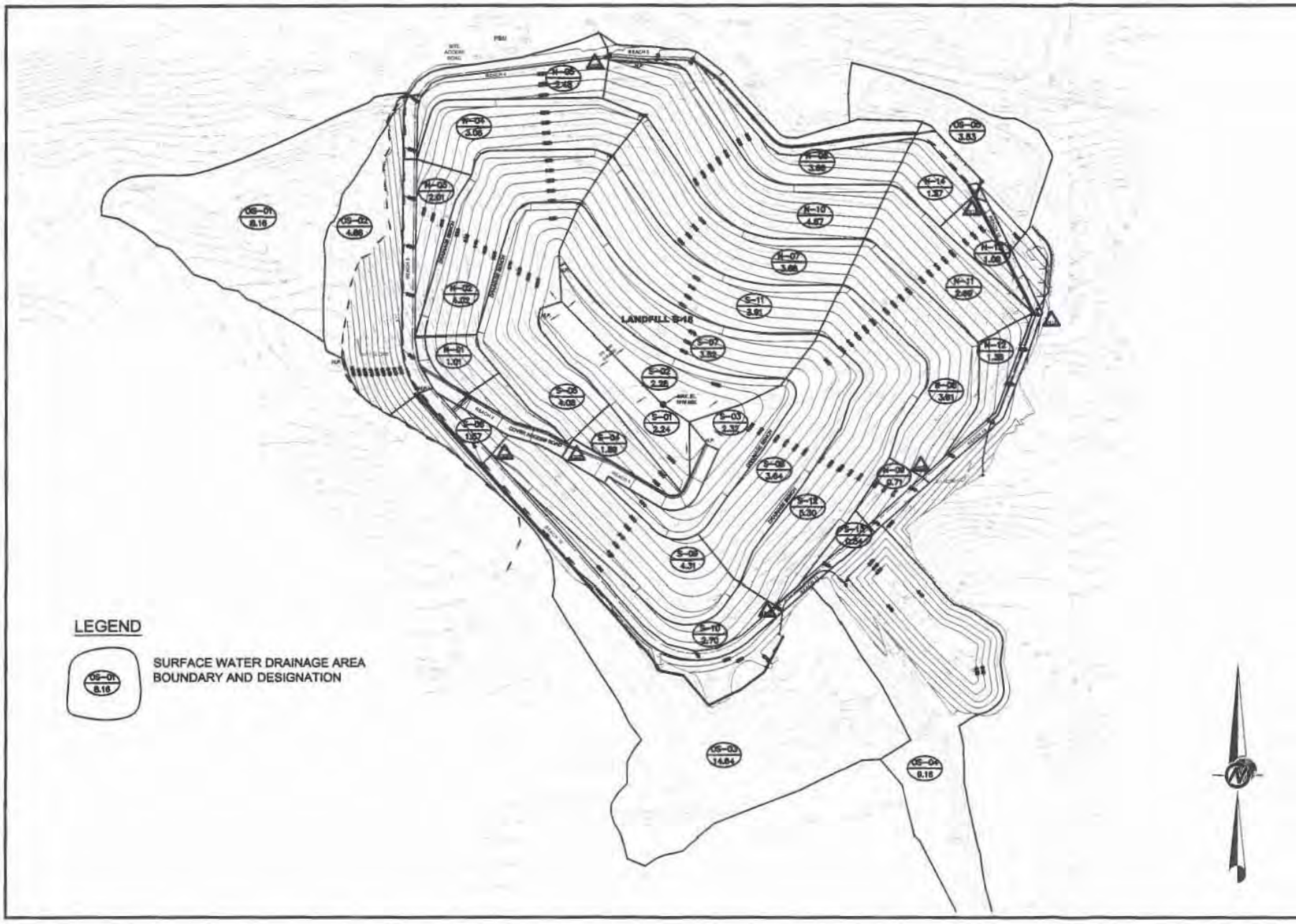
FIGURE 5.4

SURFACE WATER DRAINAGE AREA MAP FOR PHASES I AND II

LANDFILL UNIT B-18
KETTLEMAN HILLS FACILITY

ENVIRONMENTAL SOLUTIONS INC.

88-899 DR REV. 08/14/90



Golder Associates
PITTSBURG, CALIFORNIA

PROJECT
KETTLEMAN HILLS FACILITY
LANDFILL UNIT B-18
KETTLEMAN CITY, CALIFORNIA

TITLE
SURFACE WATER DRAINAGE AREA MAP FOR PHASE III AND FINAL CLOSURE

PROJECT No. 083-91887		
FILE No. 08391887A001		
REV. A	SCALE	1"=300'
DESIGN	PM	10/31/08
CADD	PM	10/31/08
CHECK	RH	11/4/08
REVIEW	SGS	11/4/08

FIGURE 5.5

APPENDIX A
CONSTRUCTION DRAWINGS

APPENDIX A.1	**PHASES I AND II DRAWINGS**
APPENDIX A.2	**PHASE III AND FINAL CLOSURE DRAWINGS**

APPENDIX A.1
PHASES I AND II DRAWINGS

CONSTRUCTION DRAWINGS
LANDFILL UNIT B-18
PHASES I AND II
AND
FINAL CLOSURE

KETTLEMAN HILLS FACILITY

KINGS COUNTY, CALIFORNIA

PREPARED FOR:

CHEMICAL WASTE MANAGEMENT, INC. - (CWMI)

PREPARED BY:

ENVIRONMENTAL SOLUTIONS, INC.

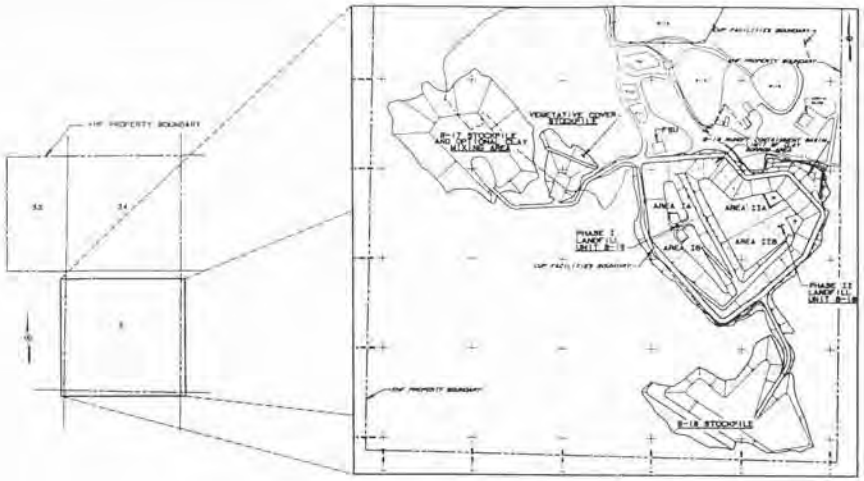
JANUARY, 1991

REVISED ISSUE FOR
PHASE II CONSTRUCTION
(JANUARY 29, 1993)

109c

REGIONAL LOCATION MAP

0 30 60 MILES
SCALE



SITE LOCATION MAP

0 600 1200 FEET
SCALE

LEGEND

GENERAL LINES

- CWM PROPERTY BOUNDARY
- MAIN TOPIC OF DRAWING
- SUBTOPIC OF DRAWING
- CENTER LINE
- HIDDEN LINE
- FLOW LINE
- EXISTING GRADE
- WORK DONE BY OTHERS
- 10 FOOT CONTOUR LINE
- 2 FOOT CONTOUR LINE
- STATION

MISCELLANEOUS

- CONTROL POINT
- FENCE
- TOP / TOE — SLOPE
- POINT OF INTERSECTION
- CURVE IDENTIFICATION
- CONTROL POINT

MONITORING WELL TYPES

- EXISTING MONITORING WELLS TO BE MAINTAINED AND PROTECTED AS REQUIRED
- EXISTING MONITORING WELLS TO BE DECOMMISSIONED DURING PHASE I
- EXISTING MONITORING WELLS TO BE DECOMMISSIONED DURING PHASE II
- PROPOSED MONITORING WELLS

GEOSYNTHETIC LINER TYPES

- HDPE
- GEOTEXTILE
- GEONET
- GEOCOMPOSITES

FILL PATTERNS

- GENERAL FILL
- STRUCTURAL FILL
- COMPACTED CLAY
- GRAVEL
- CONCRETE
- SAND
- WASTE
- NATURAL GRADE

KEY

SECTION (ALPHA) OR DETAIL (NUMERIC) IDENTIFICATION

SHEET NO. WHERE SECTION OR DETAIL IS SHOWN

SHEET NO. WHERE SECTION OR DETAIL IS CALLED OUT

ABBREVIATIONS

A.C.	ASPHALT CONCRETE
ADMIN.	ADMINISTRATION
APPROX.	APPROXIMATE
C	CUT
℄	CENTER LINE
CMP	CORRUGATED METAL PIPE
CWM	CHEMICAL WASTE MANAGEMENT, INC
DIA	DIAMETER
E	EASTING
EL	ELEVATION
F	FILL
FSU	FINAL STABILIZATION UNIT
HDPE	HIGH DENSITY POLYETHYLENE
HPFL	HIGH POINT IN FLOW LINE
KHF	KETTLEMAN HILLS FACILITY
L	LENGTH
LPFL	LOW POINT IN FLOW LINE
LT.	LEFT
MIN.	MINIMUM
N	NORTHING
N.T.S.	NOT TO SCALE
P.C.	POINT OF CURVATURE
P.I.	POINT OF INTERSECTION
P.T.	POINT OF TANGENCY
R	RADIUS
RT	RIGHT
T	TANGENT
TYP.	TYPICAL
Δ	INTERNAL ANGLE
Ø	DIAMETER

INDEX OF DRAWINGS

SHEET NO.	ESI FILE NO	TITLE
1	D-89977-FD001	INDEX, LEGEND AND GENERAL NOTES
2A	D-89977-FD002	PHASE I EXCAVATION PROJECT PLAN
2	D-89977-FD002	PROJECT PLAN
3A	D-89977-FD003	PHASE I CONSTRUCTION SEQUENCE PLAN
3	D-89977-FD003	PHASE I EXCAVATION AND SITE PREPARATION PLAN
4	D-89977-FD004	PHASE I TOP OF SECONDARY LINER
5	D-89977-FD005	PHASE I TOP OF PRIMARY LINER
6	D-89977-FD006	PHASE I INTERMEDIATE WASTE FILL PLAN
7	D-89977-FD007	CLAY BORROW PIT EXPANSION PLAN
8	D-89977-FD008	PHASE II EXCAVATION AND SITE PREPARATION PLAN
9	D-89977-FD009	PHASE II TOP OF SECONDARY LINER
10	D-89977-FD010	PHASE II TOP OF PRIMARY LINER
11	D-89977-FD011	PHASE I AND II FINAL CLOSURE PLAN
12	D-89977-FD012	GRADING DETAILS
13	D-89977-FD013	GRADING DETAILS
14	D-89977-FD014	GRADING DETAILS
15	D-89977-FD015	CROSS SECTIONS
16	D-89977-FD016	LINER DETAILS AND SECTIONS
17	D-89977-FD017	LINER DETAILS AND SECTIONS
18	D-89977-FD018	TYPICAL SUMP PLAN
18A	D-89977-SUD018	TYPICAL SUMP PLAN
19	D-89977-FD019	SUMP AND RISER DETAILS
20	D-89977-FD020	SUMP AND RISER DETAILS
21	D-89977-FD021	SUMP AND RISER DETAILS
22	D-89977-FD022	SUMP AND RISER DETAILS
23	D-89977-FD023	MISCELLANEOUS CROSS SECTIONS
24	D-89977-FD024	DRAINAGE DETAILS
25	D-89977-FD025	SURVEY CONTROL

GENERAL NOTES

1. THESE DRAWINGS AND THE SPECIFICATIONS SHOULD BE USED TOGETHER. IF A DIFFERENCE OCCURS, THE CWMI PROJECT MANAGER SHALL DETERMINE WHICH TAKES PRECEDENCE.
2. THE B-18 LANDFILL CONSTRUCTION WILL OCCUR WITHIN AREAS APPROVED BY VARIOUS GOVERNMENT AGENCIES. NO WORK SHALL OCCUR OUTSIDE OF THE CONSTRUCTION AREAS SHOWN ON THE DRAWINGS WITHOUT SPECIFIC APPROVAL OF THE CWMI PROJECT MANAGER.
3. PRIOR TO COMMENCING ANY CUT OR FILL, OR IN ANY WAY DISTURBING THE EXISTING LAND CONTOURS, CONTRACTOR SHALL VERIFY THE ACCURACY OF THE CONTOUR ELEVATIONS SHOWN IN THE DRAWINGS.

3	10/8/93	ISSUED FOR PHASE II CONSTRUCTION	KJR	
2	1/5/93	ISSUED FOR CONSTRUCTION	KJR	
1	8/24/92	ISSUED FOR BID	RD	
0	7/31/92	ISSUED FOR PERMITTING	KAD	

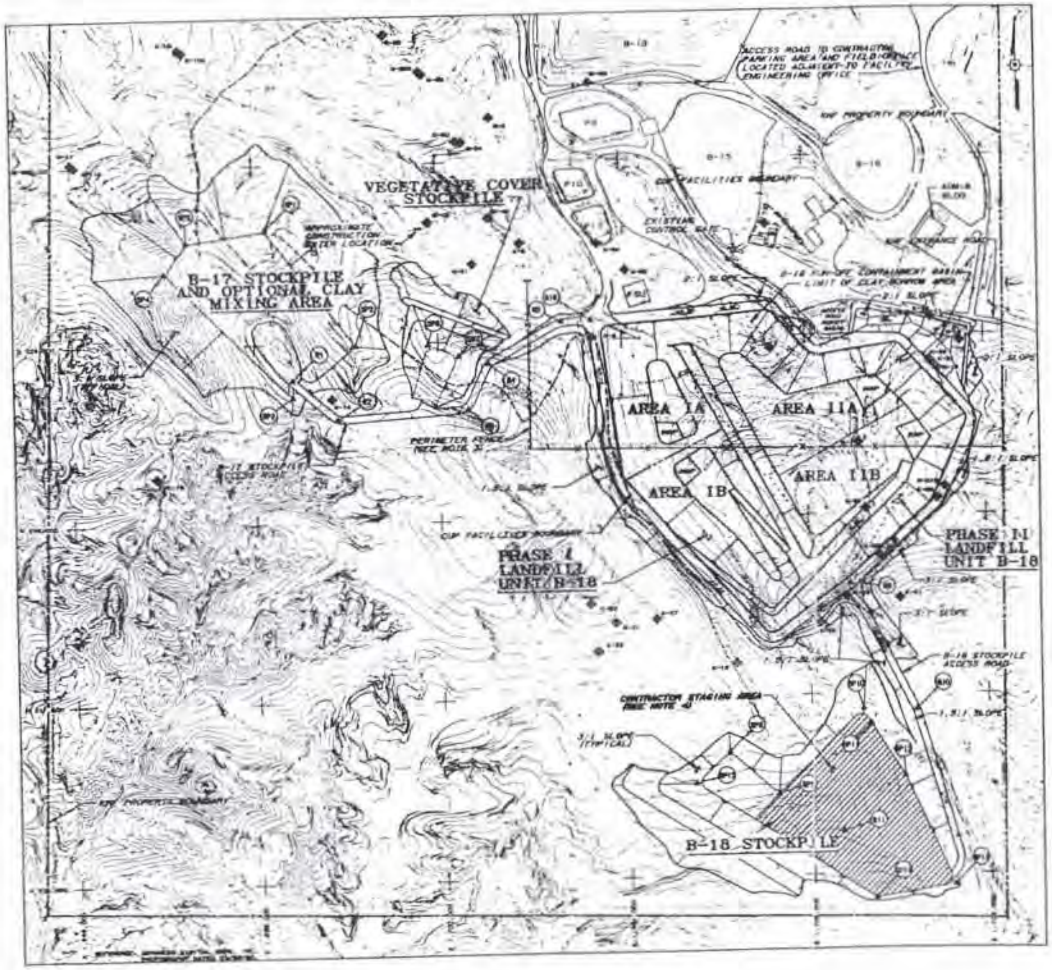
CONSTRUCTION DRAWINGS
LANDFILL UNIT B-18
CWMI-KETTLEMAN HILLS FACILITY
KINGS COUNTY, CALIFORNIA

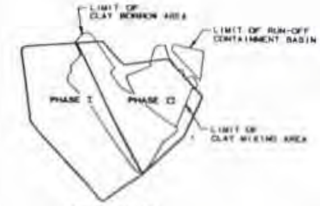
REGISTERED PROFESSIONAL ENGINEER — KENNETH J. RABENS — No. 41021 — Exp. 3-31-93 — CIVIL — STATE OF CALIFORNIA

TITLE: INDEX, LEGEND AND GENERAL NOTES

ENVIRONMENTAL SOLUTIONS, INC.

JOB NO.	DRAWING NO.	REVISION
89-977	D-89977-FD001	3
SCALE: AS NOTED	FILE NO. 89-03-304.2.1	SHEET 1





KEY PLAN

MONITORING WELL LOCATIONS

WELL	SYMBOLS	NORTHING	EASTING
A-5		229555.82	1700456.02
A-6A		229531.79	1700882.00
K-1		228398.11	1702295.71
K-2		228831.88	1701795.14
K-5		230947.04	1700301.74
K-8		229997.57	1700832.32
K-10		230837.92	1699800.78
K-10A		230823.02	1699820.92
K-11		230023.33	1699008.78
K-11A		230005.88	1699025.24
K-13		229855.78	1699941.46
K-14		229876.71	1699405.58
K-15		229262.68	1698455.72
K-16		229948.07	1702652.22
K-18		227189.59	1701580.87
K-19		228792.09	1702724.07
K-21		227406.85	1700807.31
K-22		227612.80	1700797.53
K-23		227248.07	1700926.99
K-24		230114.33	1700144.78
K-25		230507.38	1698823.07
K-25A		230450.39	1698857.36
K-26		228070.27	1702707.95
K-27		229825.41	1702880.15
K-28		229528.00	1702881.13
K-29		228080.78	1702889.34
K-30		230115.82	1700113.44
K-31		229419.24	1700178.96
K-32		228018.07	1702312.81
K-33		227398.12	1702080.82
K-36		229388.80	1701010.81
K-40		228667.86	1703264.82
K-46		230643.24	1699597.02
K-48		230452.92	1700653.00
K-49		229612.11	1701781.88
K-50		230719.43	1699730.73
K-51		227580.00	1702510.00
K-53		227545.00	1702215.00
K-57		227485.00	1701185.00
K-01R		228180.00	1702785.00
K-32R		227770.00	1702400.00

* SEE NOTE 3

STOCKPILE AND ACCESS ROAD CONSTRUCTION SURVEY CONTROL POINTS OF INTERSECTION

POINT OF INTERSECTION	NORTHING	EASTING	ELEVATION
P.I.-R1	228796.87	1699205.35	860.0
P.I.-R2	228574.33	1699446.32	840.0
P.I.-R3	228821.84	1700187.41	840.0
P.I.-R4	228873.80	1700272.07	920.0
P.I.-R5	229100.65	1700878.77	881.0
P.I.-R9	227813.32	1702234.88	830.0
P.I.-R10	228906.68	1702588.41	890.0
P.I.-R11	228238.18	1702174.36	852.4
P.I.-R18	229105.63	1700842.12	847.0
P.I.-SP1	229858.85	1699088.21	892.3
P.I.-SP2	228984.80	1699353.35	880.4
P.I.-SP3	228702.85	1699207.36	880.4
P.I.-SP4	229400.92	1699513.68	850.5
P.I.-SP5	229573.36	1699840.45	850.0
P.I.-SP6	228984.30	1699946.74	850.0
P.I.-SP7	228817.80	1700108.98	850.0
P.I.-SP8	228880.14	1701545.20	852.4
P.I.-SP9	228448.82	1701909.85	852.1
P.I.-SP10	228818.84	1702285.80	858.5
P.I.-SP11	228831.48	1702538.27	858.2
P.I.-SP12	228541.08	1702619.15	858.9
P.I.-SP13	227935.88	1702790.84	854.3
P.I.-SP14	227868.12	1703346.87	850.0
P.I.-SP15	226518.41	1701358.42	850.0

NOTES

1. SEE SHEET 1 FOR LEGEND AND GENERAL NOTES.
2. SEE LEGEND ON SHEET 1 FOR MONITORING WELL SYMBOLS.
3. EXISTING PERIMETER FENCE WILL BE EXTENDED TO ENCLOSE THE B-18 LANDFILL AREA INTO THE KHF ACTIVE FACILITY AREA.
4. LOCATION APPROXIMATE. FINAL LOCATION TO BE DETERMINED BY OWNER.

0 300 600 FEET
SCALE

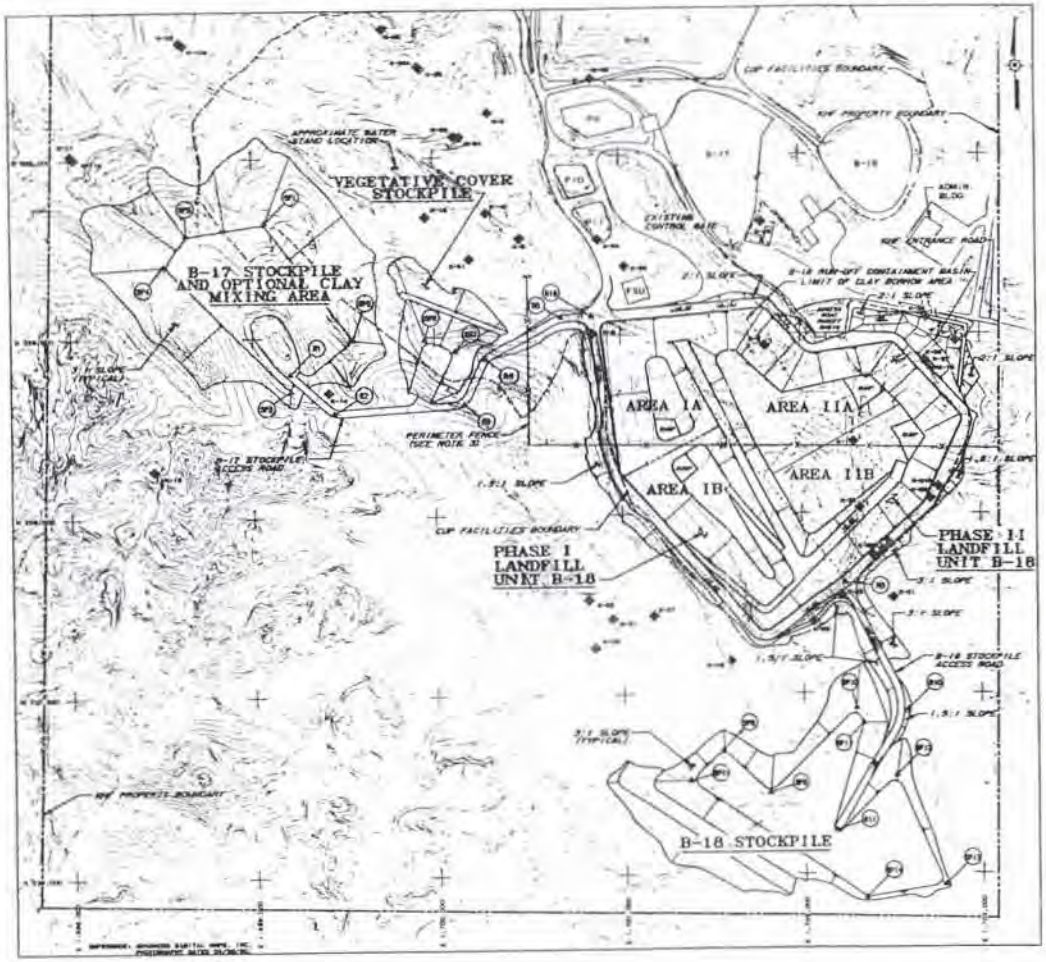
3	2/7/91	MODIFIED CONTRACTOR STAGING AREA	KJR	
2	1/9/91	ISSUED FOR CONSTRUCTION	KJR	
1	9/7/90	ISSUED FOR BID	MD	
0	2/8/90	ISSUED FOR PERMITTING	KAB	

LANDFILL UNIT B-18
CONSTRUCTION PLANS
CWMI-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

PHASE I EXCAVATION
PROJECT PLAN

ENVIRONMENTAL SOLUTIONS, INC.

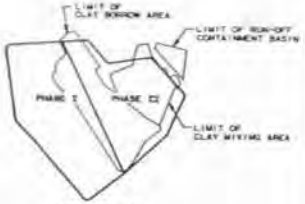
89-877	D-89977-EX002	3
1"=300'	89-03-304.2.1	2A



MONITORING WELL LOCATIONS

WELL	SYMBOL*	NORTHING	EASTING
A-5		229555.10	1700458.82
A-8A		229531.78	1700882.00
K-1		226898.11	1702295.51
K-2		226921.18	1701793.44
K-5		230247.24	1700401.74
K-6		228987.77	1700823.92
K-10		230837.32	1699400.98
K-10A		230822.32	1699420.82
K-11		230023.73	1699208.78
K-11A		230005.93	1699225.24
K-13		229655.78	1699941.96
K-14		229878.77	1699405.58
K-15		229292.98	1699453.82
K-16		228948.07	1702452.02
K-18		227189.14	1701580.87
K-19		228792.38	1702724.07
K-21		227406.85	1700927.31
K-22		227512.40	1700787.53
K-23		227248.07	1700828.98
K-24		230114.22	1700144.78
K-25		230807.23	1699923.07
K-25A		230493.38	1699917.98
K-26		229070.47	1702707.95
K-27		228825.41	1702982.15
K-28		229838.02	1702881.15
K-29		229080.78	1702989.34
K-30		230115.92	1700113.44
K-31		229418.24	1700178.86
K-32		229018.07	1702313.81
K-33		227596.12	1700080.82
K-36		229369.60	1701010.81
K-40		229687.00	1700284.82
K-46		230843.24	1699397.00
K-48		230432.92	1700853.00
K-49		229812.11	1701781.69
K-50		230719.43	1699730.73
K-51		227530.00	1702510.00
K-53		227543.00	1702215.00
K-57		227425.00	1701165.00
K-01R		226150.00	1702765.00
K-02R		227770.00	1702400.00

* SEE NOTE 2



KEY PLAN

STOCKPILE AND ACCESS ROAD CONSTRUCTION SURVEY CONTROL POINTS OF INTERSECTION

POINT OF INTERSECTION	NORTHING	EASTING	ELEVATION
P.I.-R1	226786.97	1699205.55	960.0
P.I.-R2	226574.52	1699446.32	940.0
P.I.-R3	226921.84	1700187.41	940.0
P.I.-R4	226873.80	1700273.07	920.0
P.I.-R5	229100.63	1700676.77	961.0
P.I.-R9	227819.22	1702284.86	930.0
P.I.-R10	226906.66	1702586.41	990.0
P.I.-R11	226236.16	1702174.36	952.4
P.I.-R16	229105.53	1700642.12	947.0
P.I.-SP1	229639.65	1699056.21	952.2
P.I.-SP2	229964.80	1698553.23	960.4
P.I.-SP3	229702.65	1699207.26	960.4
P.I.-SP4	229400.50	1698515.68	950.5
P.I.-SP5	229573.26	1698940.43	950.0
P.I.-SP6	229564.50	1699946.74	950.0
P.I.-SP7	229917.90	1700108.96	950.0
P.I.-SP8	229690.14	1701545.23	952.4
P.I.-SP9	229448.63	1701806.85	952.1
P.I.-SP10	228916.64	1702295.90	958.5
P.I.-SP11	228631.48	1702336.27	958.2
P.I.-SP12	228541.08	1702518.15	958.9
P.I.-SP13	228533.86	1702788.64	954.3
P.I.-SP14	225886.12	1702345.97	950.0
P.I.-SP15	226518.41	1701359.42	950.0

REGISTERED PROFESSIONAL ENGINEER
RICHARD A. FINKELSTEIN
NO. 41021
Exp. 06/31/98
CIVIL
STATE OF CALIFORNIA

NOTES

1. SEE SHEET 1 FOR LEGEND AND GENERAL NOTES.
2. SEE LEGEND ON SHEET 1 FOR MONITORING WELL SYMBOLS.
3. EXISTING PERIMETER FENCE WILL BE EXTENDED TO ENCLOSE THE B-18 LANDFILL AREA INTO THE KMF ACTIVE FACILITY AREA.

0 300 600 FEET
SCALE

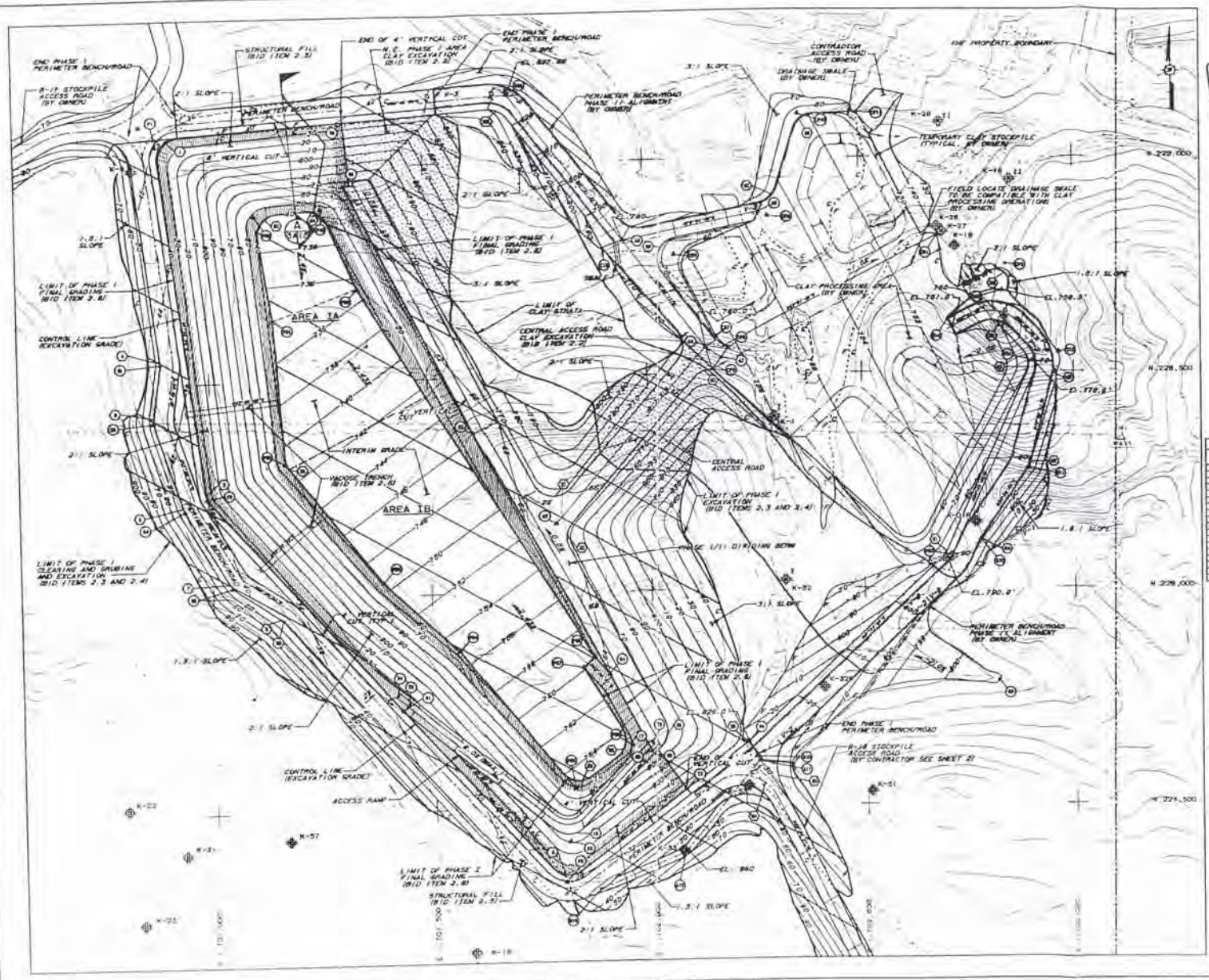
2	1/29/98	ISSUED FOR CONSTRUCTION	RJR	
0	7/31/96	ISSUED FOR PERMITTING	KAB	

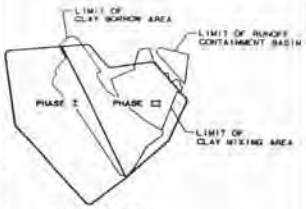
LANDFILL UNIT B-18
CONSTRUCTION PLANS
CWM-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

PHASE I EXCAVATION
PROJECT PLAN

ENVIRONMENTAL SOLUTIONS, INC.

89-877	D-89977-FD002	2
1"=800'	89-03-304.2.1	2





KEY PLAN

POINTS OF INTERSECTION			
POINT	NORTHING	EASTING	ELEV.
PIC	[illegible]	[illegible]	[illegible]
PIT	[illegible]	[illegible]	[illegible]
PIB	[illegible]	[illegible]	[illegible]
PIA	[illegible]	[illegible]	[illegible]
PIG	[illegible]	[illegible]	[illegible]
PIF	[illegible]	[illegible]	[illegible]
PIE	[illegible]	[illegible]	[illegible]
PID	[illegible]	[illegible]	[illegible]
PIC	[illegible]	[illegible]	[illegible]
PIB	[illegible]	[illegible]	[illegible]
PIA	[illegible]	[illegible]	[illegible]
PIG	[illegible]	[illegible]	[illegible]
PIF	[illegible]	[illegible]	[illegible]
PIE	[illegible]	[illegible]	[illegible]
PID	[illegible]	[illegible]	[illegible]
PIC	[illegible]	[illegible]	[illegible]
PIT	[illegible]	[illegible]	[illegible]
B18	[illegible]	[illegible]	[illegible]
B19	[illegible]	[illegible]	[illegible]
B14	[illegible]	[illegible]	[illegible]
B24	[illegible]	[illegible]	[illegible]

CURVE DATA						
CURVE NO.	LOCATION OF P.I. NORTHING	LOCATION OF P.I. EASTING	R	D	L	T
B10	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
B11	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
B8	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
B9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
B4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
B6	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
B5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
B1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
B2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
B3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
B7	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
B12	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
B13	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
B14	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
B24	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

NOTES

1. SEE SHEET 1 FOR LEGEND AND GENERAL NOTES.
2. SEE SHEET 3 FOR CROSS SECTION AND DETAIL PLAN CALLOUTS.
3. SEE SHEET 5 FOR DRAINAGE PLAN AND DETAIL PLAN CALLOUTS. (BID ITEM 2.7)

REGISTERED PROFESSIONAL ENGINEER JERRY L. TAKAMOTO No. 41021 Exp. 06/11/96 CIVIL STATE OF CALIFORNIA

0 100 200 FEET
SCALE

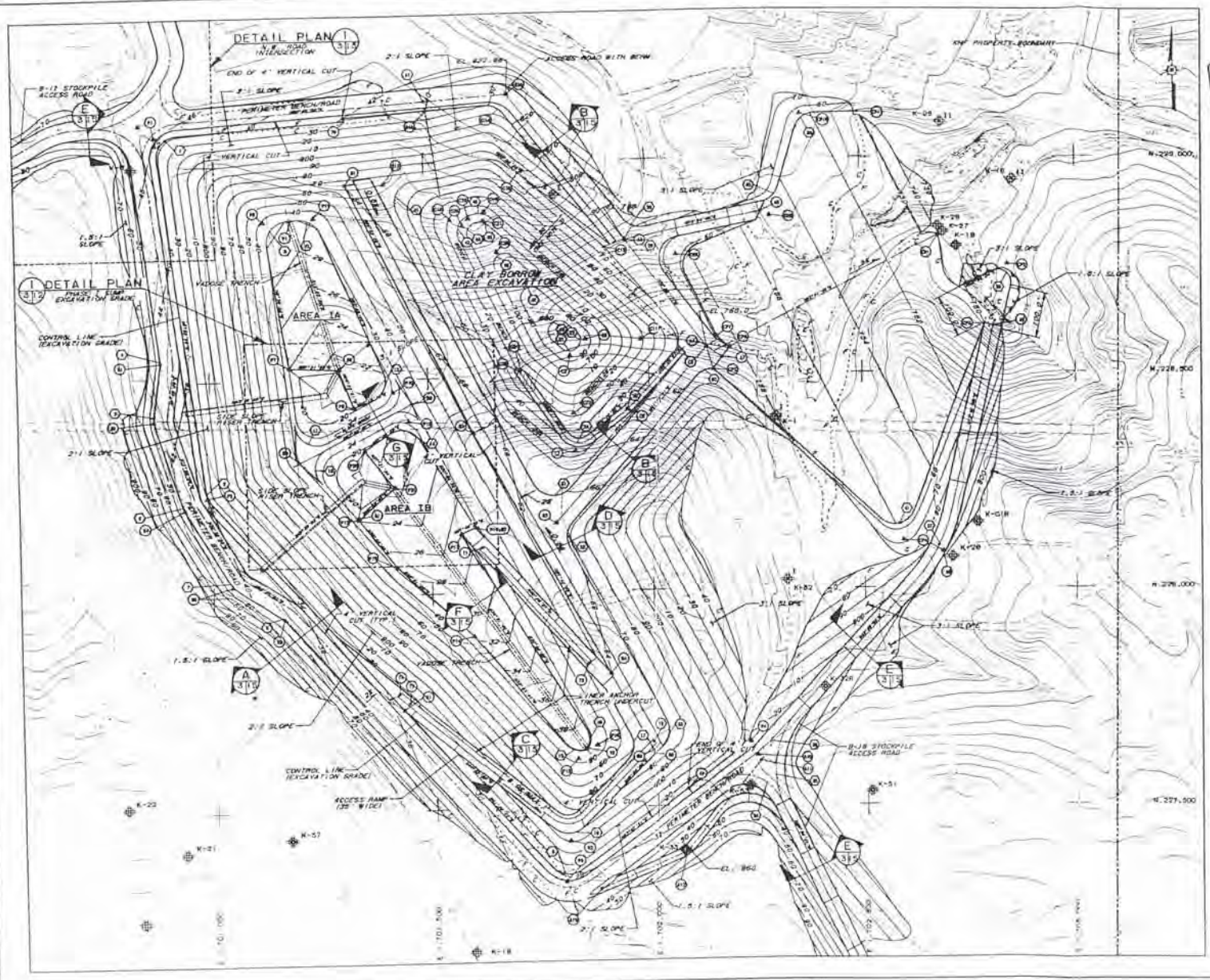
2	10/18/95	ISSUED FOR CONSTRUCTION	KJK
B	9/24/95	ISSUED FOR BID	KJK
A	8/31/95	ISSUED FOR COMMENTS	KJK

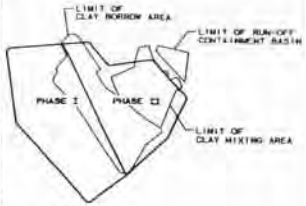
LANDFILL UNIT B-18
CONSTRUCTION PLANS
CWM-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

PHASE I
CONSTRUCTION SEQUENCE PLAN

ENVIRONMENTAL SOLUTIONS, INC.

89-977	D-89977-EX003	2
1"=100'	89-03-304.2.1	3A





KEY PLAN

REGISTERED PROFESSIONAL ENGINEER
No. 41021
Exp. 06/31/89
CIVIL
STATE OF CALIFORNIA

NOTES:

1. SEE SHEET 1 FOR LEGEND AND GENERAL NOTES.
2. TRANSITION FROM PERIMETER BENCH/ROAD (SECTION 4, 3,151 TO ACCESS ROAD WITH BERM (SECTION 8,3,151.
3. THE EAST SLOPE OF THE CLAY PIT EXCAVATION MAY BE VARIED TO CONFORM TO THE BASE OF THE CLAYSTONE STRATUM.
4. SEE SHEET 25 FOR SURVEY CONTROL.

0 100 200 FEET
SCALE

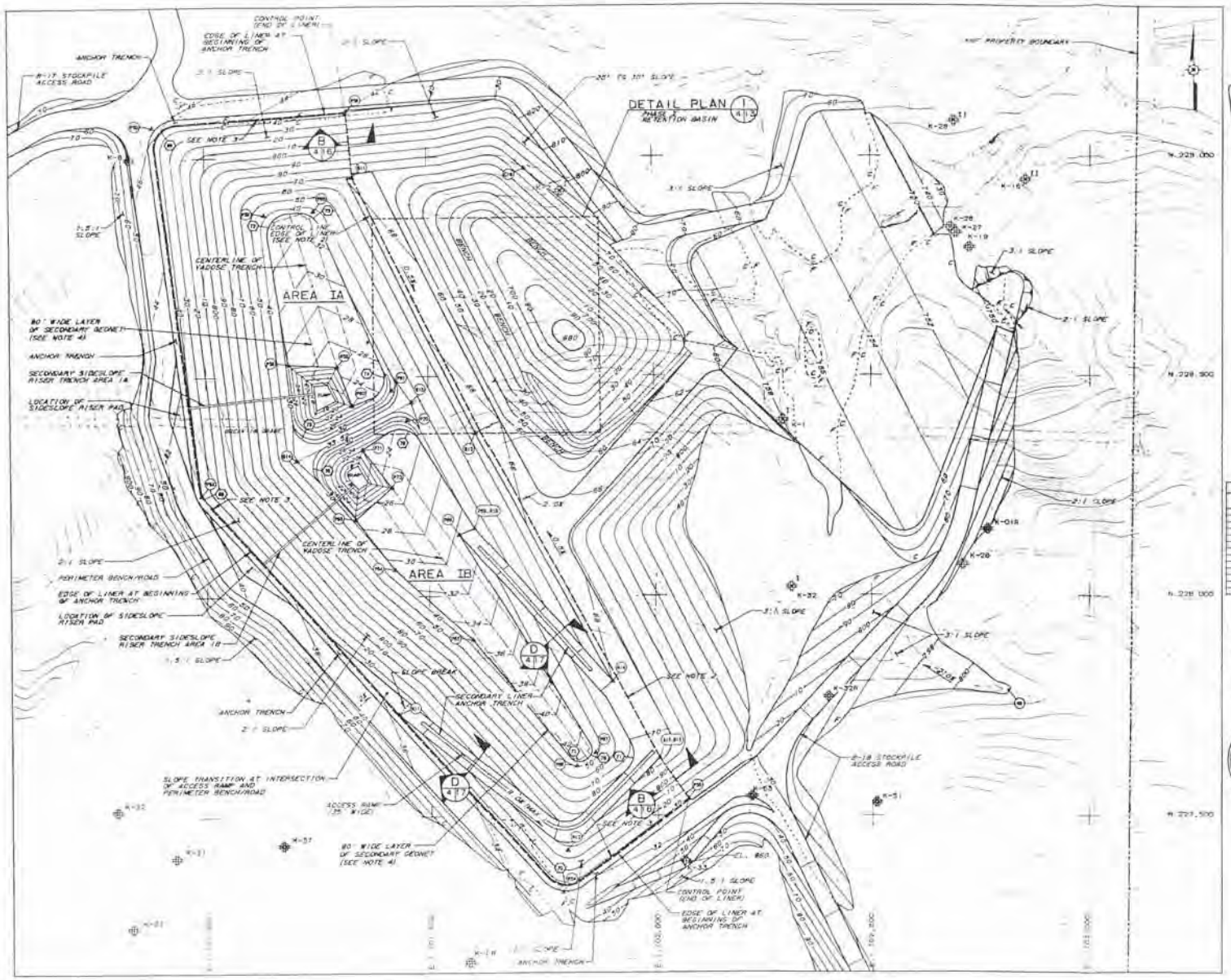
2	5/4/89	ISSUED FOR CONSTRUCTION	RKD
1	3/9/89	ISSUED FOR BIDDING	RKD
0	3/8/89	ISSUED FOR PERMITTING	RKD

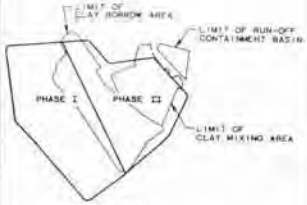
LANDFILL UNIT B-18
CONSTRUCTION PLANS
CWM-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

PHASE I EXCAVATION AND SITE PREPARATION PLAN

ENVIRONMENTAL SOLUTIONS, INC.

89-077	D-89977-F0003	2
1"=100'	89-03-304 2.1	3





KEY PLAN

POINTS OF INTERSECTION

POINT	NORTHING	EASTING	ELEV.
P22	228,071.00	1,700,889.00	848.0
P23	228,080.00	1,700,888.00	840.0
P24	227,939.00	1,701,745.00	832.5
P25	227,987.88	1,702,071.82	830.7
P26	228,101.11	1,701,823.08	841.8
P27	228,688.54	1,701,180.87	794.7
P28	228,258.21	1,701,171.76	797.8
P29	228,230.06	1,701,289.98	794.8
P30	228,183.14	1,701,288.98	793.4
P31	228,188.54	1,701,417.48	794.5
P32	228,277.80	1,701,242.44	794.5
P33	228,174.47	1,701,448.48	797.4
P34	228,080.37	1,701,420.87	798.7
P35	227,800.33	1,701,384.10	748.2
P36	227,813.24	1,701,618.90	748.5
P37	227,843.38	1,701,880.47	748.8
P38	228,188.82	1,701,262.83	780.4
P39	228,188.88	1,701,088.48	781.3
P40	228,400.98	1,701,100.80	784.7
P41	228,334.27	1,701,348.88	792.3
P42	228,248.98	1,701,418.88	794.8
P43	228,847.47	1,701,304.87	788.2
P44	228,171.14	1,701,408.38	784.3
P45	227,880.83	1,708,004.78	772.3
P46	228,808.13	1,701,181.78	768.3
P47	228,44.08	1,701,481.83	768.3
P48	227,787.81	1,701,498.80	848.3
P49	227,423.18	1,701,188.18	787.1
P50	227,880.88	1,708,004.78	772.3
P51	227,808.27	1,701,888.80	788.3
P52	228,138.88	1,701,288.88	781.8

CURVE DATA

CURVE NO.	LOCATION OF P.I. NORTHING	LOCATION OF P.I. EASTING	Δ	R	L	T
B8	228,071.00	1,700,889.00	49°32'38"	90.00	81.24	43.76
B9	228,180.00	1,700,888.00	84°33'42"	90.00	38.87	18.38
T1	227,924.00	1,701,745.00	44°46'48"	90.00	70.88	40.41
T2	227,881.87	1,701,898.87	79°44'17"	90.00	83.40	48.81
T3	228,888.34	1,701,187.87	84°14'18"	90.00	83.40	48.81
T4	228,400.78	1,701,184.84	112°28'38"	90.00	100.70	78.08
T5	228,488.34	1,701,417.48	92°27'14"	90.00	80.88	53.30
T6	228,277.80	1,701,242.44	77°17'38"	90.00	83.08	48.30
T8	228,808.13	1,701,181.78	108°10'55"	90.00	88.38	70.38
T9	227,843.38	1,701,880.47	78°28'33"	90.00	88.41	48.18
T10	228,188.82	1,701,262.83	84°38'08"	90.00	88.88	52.78
T11	228,400.98	1,701,100.80	80°08'11"	90.00	80.87	51.87

NOTES

1. SEE SHEET 1 FOR LEGEND AND GENERAL NOTES.
2. THE EAST SIDE SECONDARY LINER CONTROL LINE IS THE EDGE OF THE LINER AT THE BEGINNING OF THE ANCHOR TRENCH.
3. THE NORTH, SOUTH AND WEST SECONDARY LINER CONTROL LINE IS THE TOP OF THE SLOPE.
4. TAPER WIDTH OF GEONET AS REQUIRED TO REMAIN WITHIN THE BOTTOM AREA.

REGISTERED PROFESSIONAL ENGINEER — GARY A. TAMMINGA — No. 41022 — EXP. 06/31/96 — CIVIL — STATE OF CALIFORNIA

0 100 200 FEET
SCALE

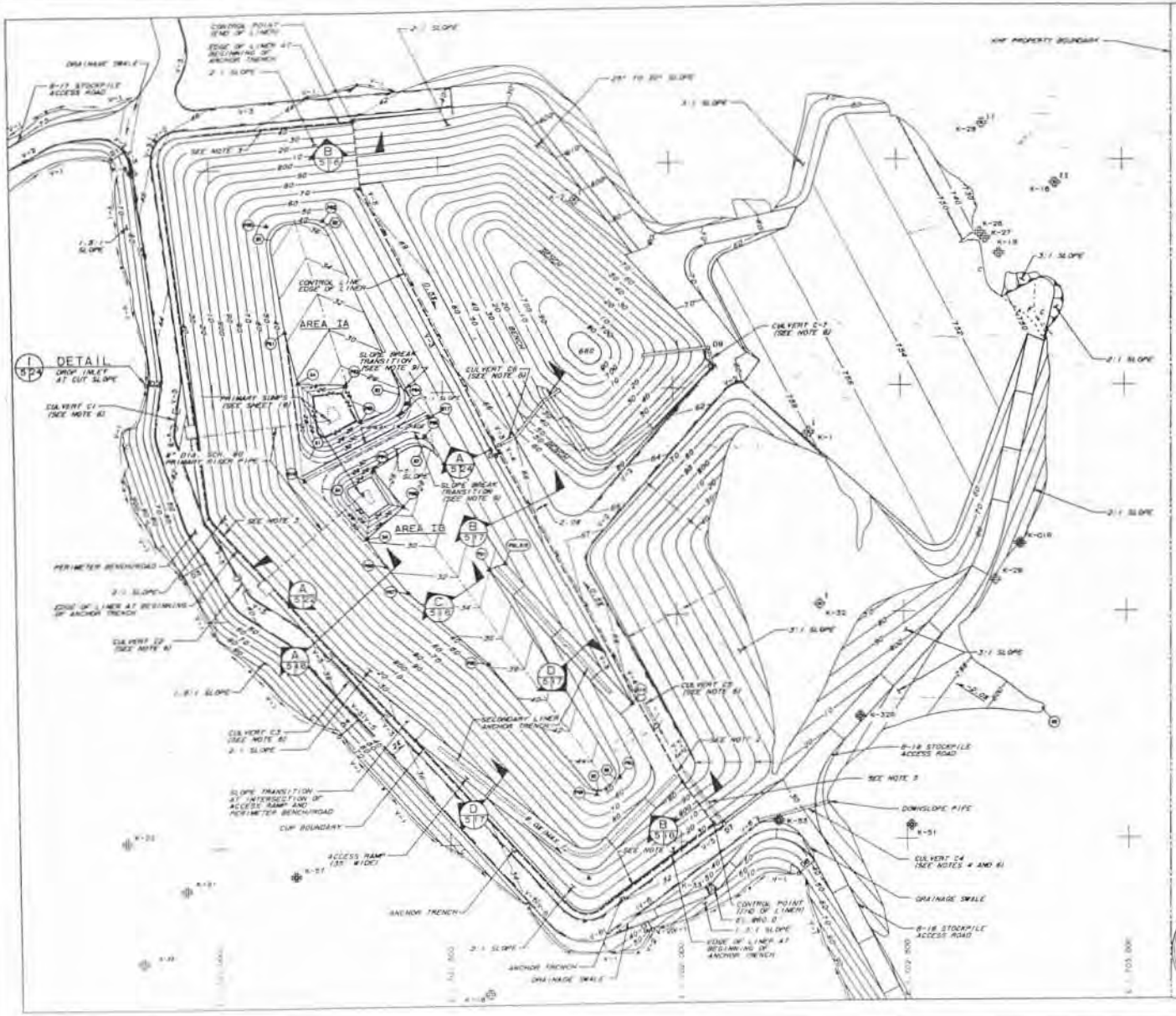
3	7/19/94	ISSUED FOR CONSTRUCTION	KJR	
1	4/94	ISSUED FOR BID	MC	
0	10/93	ISSUED FOR PERMITTING	KJR	

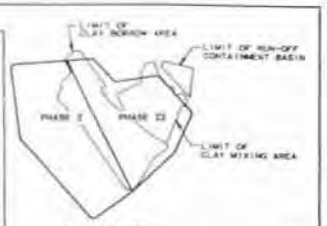
LANDFILL UNIT B-18
CONSTRUCTION PLANS
CWM-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

PHASE I
TOP OF SECONDARY LINER

ENVIRONMENTAL SOLUTIONS, INC.

89-977	D-89977-FD004	2
1"=100'	89-03-304.2.1	4





KEY PLAN

CULVERT SCHEDULE

NO.	DIA. (IN.)	LENGTH (FT.)
C1	12	80
C2	12	80
C3	12	170
C4	24	55N
C5	18	80
C6	30	170N
C7	12	50N

V-DITCH SCHEDULE

TYPE	WIDTH (FT.)	DEPTH (FT.)
V-1	5	1
V-2	5	1
V-3	5	1.5
V-4	5	2
V-5	5	2.5
V-6	5	3
V-7	5	1.25

DROP INLET SCHEDULE

NO.	DEPTH (FT.)	DIA. (IN.)	OUTLET PIPE DIA. (IN.)	OUTLET PIPE LENGTH (FT.)
D1	4	36	12	35
D2	4	36	24	45
D3	4	36	18	45
D4	4	36	12	80
D5	4	36	24N	55 EA
D6	4	36	18	80
D7	4	72	18	140
D8	4	72	30	170
D9	4	36	18	140

* NOTE: LENGTH IS APPROXIMATE. ACTUAL LENGTH WILL BE DETERMINED AT COMPLETION OF GRADING. LENGTH INCLUDES DOWNSLOPE PIPE

POINTS OF INTERSECTION

POINT	NORTHING	EASTING	ELEV.
P01	334,814.55	1,701,840.00	797.0
P02	334,852.88	1,701,188.33	798.0
P03	334,920.17	1,701,561.38	797.0
P04	334,987.88	1,701,553.04	796.0
P05	334,974.08	1,701,402.88	797.0
P06	334,894.93	1,701,346.42	798.0
P07	334,119.71	1,701,372.67	798.0
P08	334,941.91	1,701,412.42	795.0
P09	337,983.75	1,701,884.88	788.0
P10	337,918.84	1,701,918.04	790.0
P11	337,842.38	1,701,888.41	790.0
P12	336,106.31	1,701,884.88	790.0
P13	336,119.31	1,701,948.88	792.0
P14	336,408.75	1,701,883.84	792.0
P15	336,383.31	1,701,846.31	792.0
P16	336,347.64	1,701,844.31	792.0
P17	336,304.14	1,701,188.75	795.0
P18	336,440.88	1,701,470.08	795.0
P19	336,114.37	1,701,859.88	794.0

CURVE DATA

CURVE NO.	LOCATION OF P.I. NORTHING	EASTING	A	R	L	T
C1	334,814.55	1,701,840.00	90°31'00"	80.00	126.38	80.31
C2	334,852.88	1,701,188.33	110°46'00"	80.00	154.68	113.00
C3	334,974.08	1,701,402.88	90°37'00"	80.00	126.44	80.32
C4	334,894.93	1,701,346.42	110°46'00"	80.00	154.68	113.00
C5	337,918.84	1,701,918.04	120°12'44"	80.00	167.78	144.88
C6	337,842.38	1,701,888.41	90°00'00"	80.00	125.66	79.79
C7	336,106.31	1,701,884.88	90°17'27"	80.00	126.08	80.08
C8	336,119.31	1,701,948.88	90°00'00"	80.00	125.66	79.79
C9	336,408.75	1,701,883.84	90°00'00"	80.00	125.66	79.79
C10	336,383.31	1,701,846.31	90°00'00"	80.00	125.66	79.79
C11	336,347.64	1,701,844.31	90°00'00"	80.00	125.66	79.79

NOTES

1. SEE SHEET 1 FOR LEGEND AND GENERAL NOTES.
2. THE EAST SIDE PRIMARY LINER CONTROL LINE IS THE EDGE OF THE LINER AT THE BEGINNING OF THE ANCHOR TRENCH.
3. THE NORTH, SOUTH AND WEST PRIMARY LINER CONTROL LINE IS THE TOP OF THE SLOPE.
4. FOR CLAY PROCESSING AREA DRAINAGE SEE SHEET 38.
5. EXTEND 18" D.I.P. PIPE 20 FEET BEYOND TOE OF SLOPE TOE OF SLOPE.
6. DUE TO SHALLOW BURIAL DEPTH CULVERT BACKFILL SHALL BE CONCRETE. PROVIDE 6 INCHES OF COVER MINIMUM.
7. ALL DROP INLETS SHALL BE OF 12 GAGE THICKNESS.
8. ALL CULVERTS AND OUTLETS SHALL BE OF 16 GAGE THICKNESS.
9. BLEND 3:1 TO 2:1 SLOPE BREAK INTO LANDFILL BOTTOM TO CREATE SMOOTH TRANSITION.

0 100 200 FEET
SCALE

NO.	DATE	REVISIONS	DRAWN BY	CHECKED BY
3	1/08/99	REMOVED SECTION A-3:24	KJK	RCF
2	11/02/98	ISSUED FOR CONSTRUCTION	KJK	RCF
1	8/04/98	ISSUED FOR BID	MD	RCF
0	7/31/98	ISSUED FOR PERMITTING	KJK	RCF

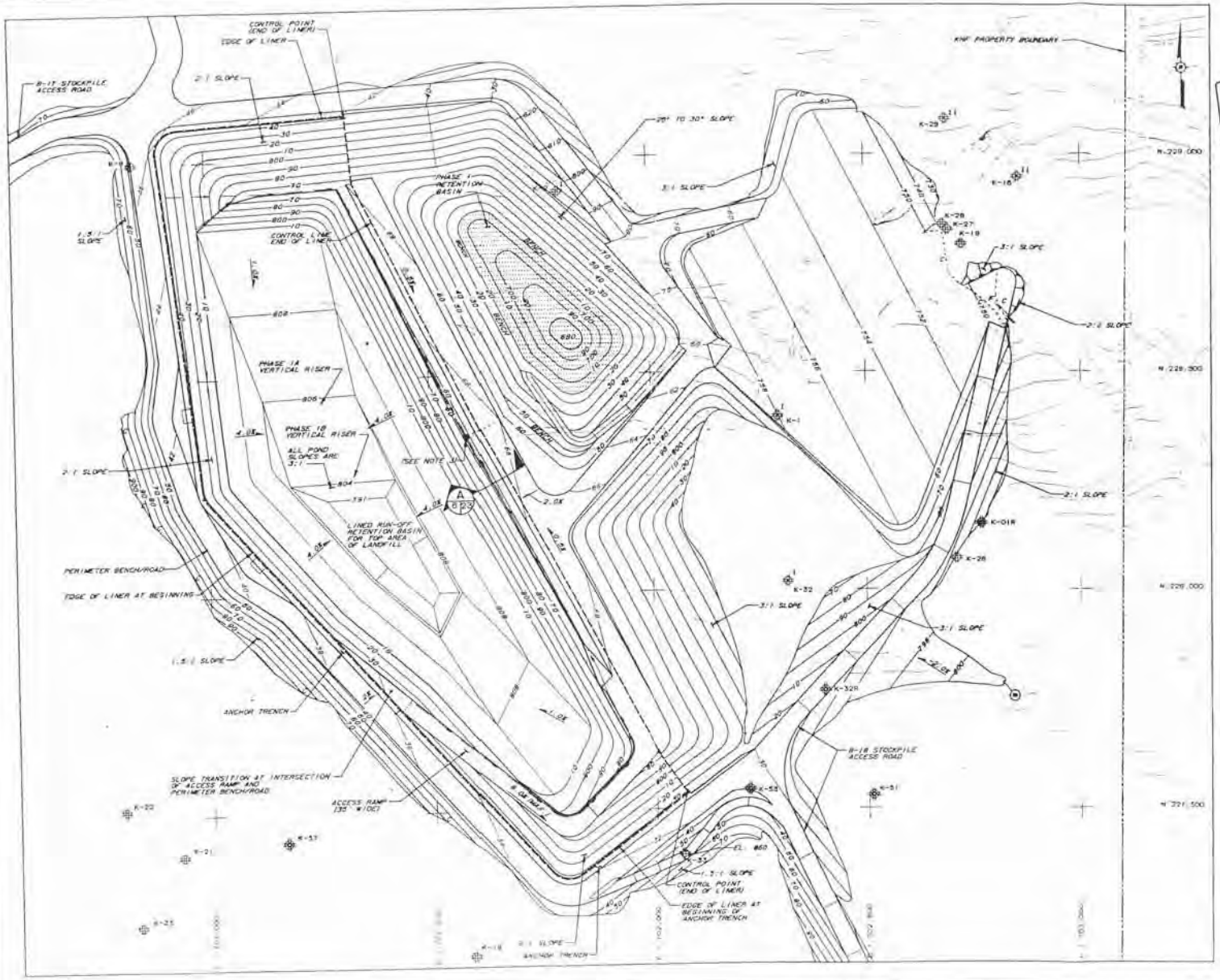
REVIEWED BY: RJE

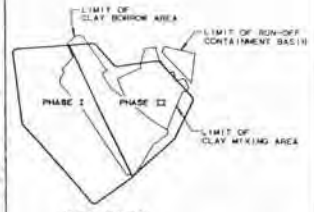
LANDFILL UNIT B-18
CONSTRUCTION PLANS
SYNAL-KETTLEMAN RILLY FACILITY
KINGS COUNTY CALIFORNIA

PHASE I
TOP OF PRIMARY LINER

ENVIRONMENTAL SOLUTIONS, INC.

89-977	D-89977-F0005	3
1"=100'	89-03-304.2.1	5





KEY PLAN

REGISTERED PROFESSIONAL ENGINEER
ROBERT B. TARKINGTON
No. 41021
Exp. MAR 31 90
CIVIL
STATE OF CALIFORNIA

NOTES:

1. SEE SHEET 1 FOR LEGEND AND GENERAL NOTES.
2. SEE SHEET 3 FOR DRAINAGE CONTROL.
3. RUN-OFF FROM LANDFILL SLOPES SHALL BE DETAINED IN A TEMPORARY POND FORMED AT THE THE LANDFILL TOE OR PUMPED TO THE PHASE 1 RETENTION BASIN AT THE OPTION OF THE OWNER.

0 100 200 FEET
SCALE

2	9/15/89	ISSUED FOR CONSTRUCTION	BRH
0	1/4/89	ISSUED FOR PERMITTING	KJK

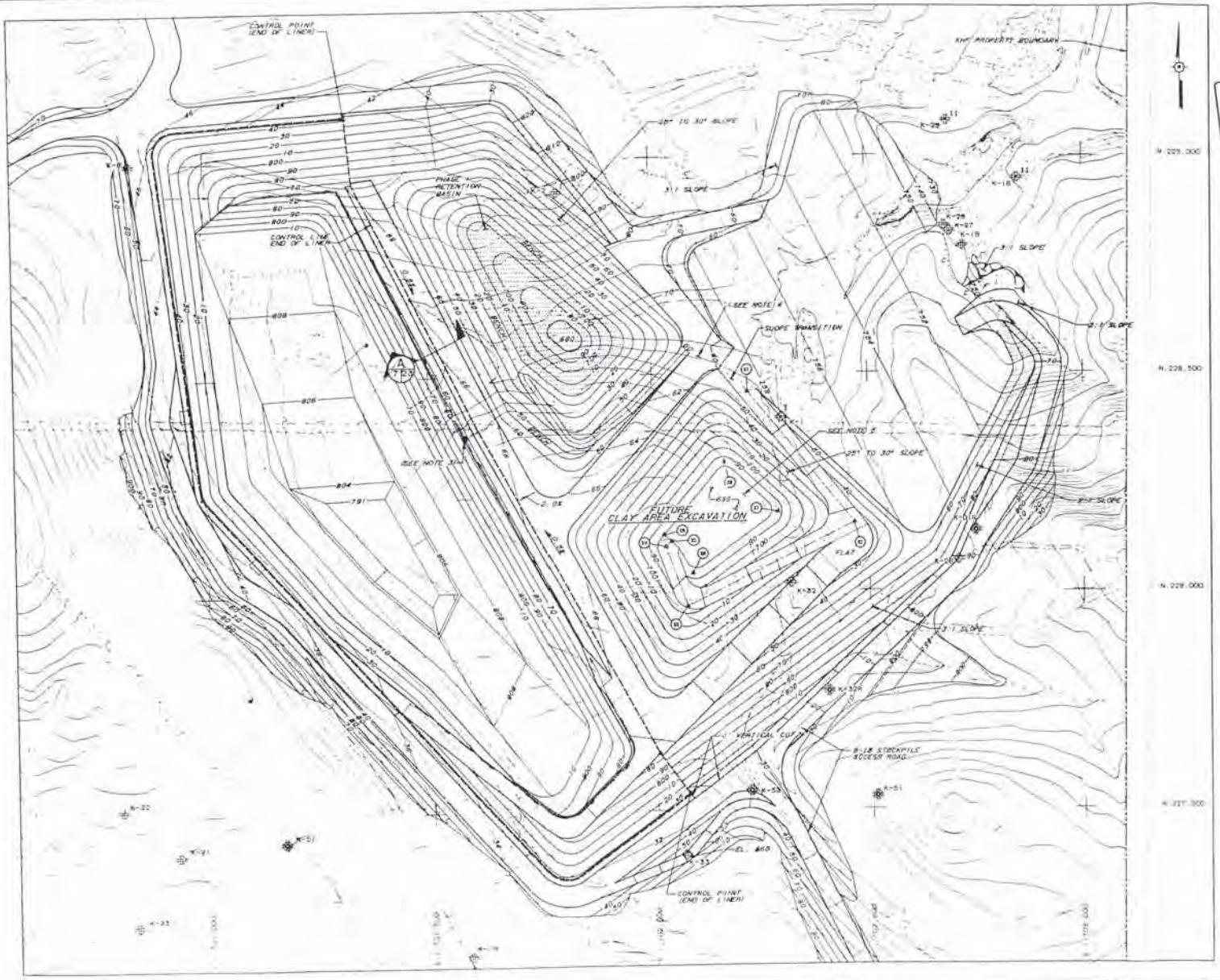
DATE

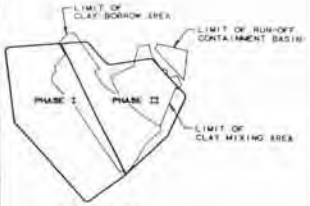
LANDFILL UNIT B-18
CONSTRUCTION PLANS
CWMI-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

PHASE I
INTERMEDIATE WASTE FILL PLAN

ENVIRONMENTAL SOLUTIONS, INC.

89-977	D-89977-F0006	2
1"=100'	89-03-304.2.1	6





KEY PLAN

POINTS OF INTERSECTION			
POINT	NORTHING	EASTING	ELEV.
A1	226,484.08	1,702,314.70	788.1
B2	226,196.82	1,701,488.35	782.5
X2	227,888.86	1,702,074.46	887.8
X4	228,104.19	1,706,057.80	882.1
B3	226,118.43	1,706,040.61	880.0
A8	226,043.44	1,705,987.68	880.0
A7	226,141.81	1,702,188.58	880.0
A6	226,287.00	1,702,193.41	880.0
A5	226,182.82	1,702,055.82	880.0

NOTES

1. SEE SHEET 1 FOR LEGEND AND GENERAL NOTES.
2. SEE SHEET 5 FOR DRAINAGE CONTROL.
3. RUN-OFF FROM LANDFILL SLOPES SHALL BE DETAINED IN A TEMPORARY POND FORMED AT THE THE LANDFILL TOE OR PUMPED TO THE PHASE I RETENTION BASIN AT THE OPTION OF THE OWNER.
4. THE BERM SEPARATING THE PHASE I RETENTION BASIN AND THE FUTURE CLAY AREA EXCAVATION MAY BE EXCAVATED AT THE OPTION OF THE OWNER.
5. THE BOTTOM OF THE FUTURE CLAY EXCAVATION AREA IS LOWER THAN THE EXCAVATION DESIGN GRADE FOR PHASE II (SEE SHEET 6). AT THE OPTION OF THE OWNER THE CLAY EXCAVATION GRADE MAY BE RAISED TO PREVENT THE USE OF BACKFILL AT THE BOTTOM.

0 100 200 FEET
SCALE

2	7/24/15	ISSUED FOR PHASE II CONSTRUCTION	K.J.K.	JKP
0	7/24/08	ISSUED FOR PERMITTING	K.J.K.	JKP
REV.	DATE	DESCRIPTION	DRAWN BY	CHKD BY

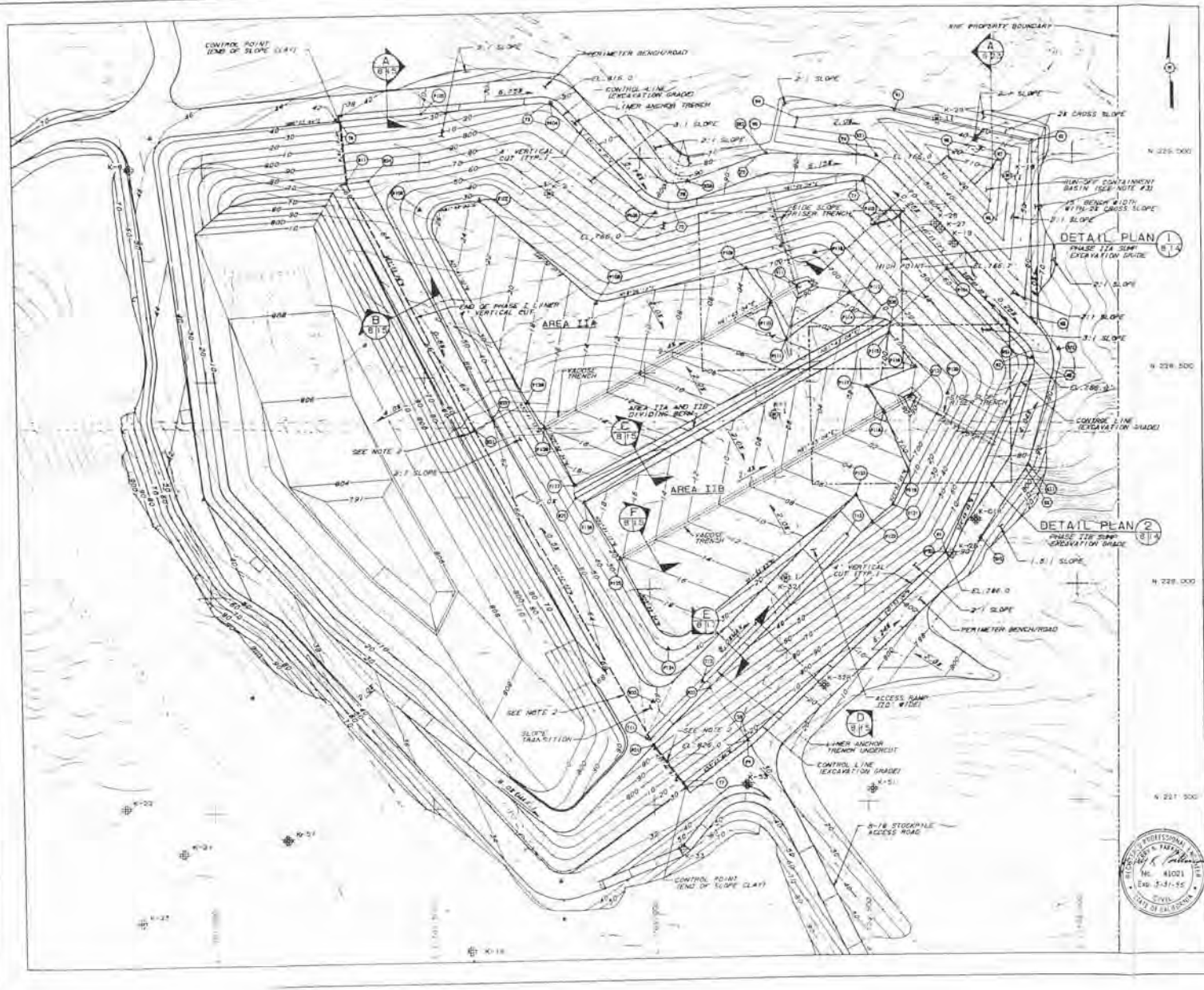
DRAWN BY K.J. Kelly CHECKED BY J. Parker APPROVED BY K.J.K.

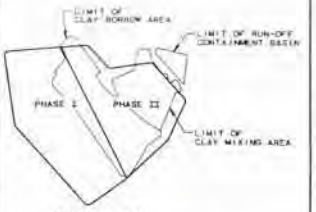
LANDFILL UNIT B-18
CONSTRUCTION PLANS
CWMI-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

CLAY BORROW PIT
EXPANSION PLAN

ENVIRONMENTAL SOLUTIONS, INC.

JOB NO. 89-977	DWG. NO. D-89977-FD007	REVISION 2
SCALE 1"=100'	DWG. FILE NAME 89-03-304.2.1	SHEET 7





KEY PLAN

POINTS OF INTERSECTION			
POINT	NORTHING	EASTING	ELEV
P101	228,731.87	1,702,866.02	788.7
P102	228,841.80	1,702,878.80	788.7
P103	228,872.41	1,702,882.40	788.7
P104	228,134.14	1,702,802.08	818.0
P105	228,119.80	1,702,800.47	820.0
P106	228,860.19	1,702,888.81	793.7
P107	228,813.76	1,702,888.21	793.7
P108	228,863.34	1,702,888.37	793.8
P109	228,788.28	1,702,888.47	798.8
P110	228,865.42	1,702,888.54	793.7
P111	228,872.08	1,702,888.84	798.8
P112	228,878.83	1,702,888.84	800.0
P113	228,773.80	1,702,888.85	800.0
P114	228,890.83	1,702,887.08	800.0
P115	228,883.83	1,702,888.7	800.0
P116	228,907.18	1,702,888.10	807.7
P117	228,881.18	1,702,887.08	807.8
P118	228,803.78	1,702,883.43	808.3
P119	228,884.00	1,702,884.82	808.7
P120	228,883.38	1,702,828.73	808.0
P121	228,188.08	1,702,870.88	808.4
P122	228,181.83	1,702,888.88	808.0
P123	228,284.48	1,702,884.13	808.8
P124	227,883.88	1,702,880.80	791.8
P125	228,088.88	1,702,888.11	718.4
P126	228,188.07	1,702,880.82	790.0
P127	228,080.08	1,702,888.83	790.0
P128	228,374.02	1,702,778.83	718.8
P129	228,484.07	1,702,728.08	717.8
P130	228,378.80	1,702,813.88	781.0
P131	228,388.07	1,702,898.00	783.0
P132	227,784.80	1,702,800.07	----
P133	228,880.08	1,702,883.00	----
P134	228,312.80	1,702,838.07	748.1
P135	228,874.01	1,702,872.78	748.0
P136	227,848.81	1,703,080.73	788.8
P137	227,788.18	1,703,130.88	780.8
P138	228,817.87	1,703,148.48	748.38
P139	228,018.28	1,703,188.48	751.7
P140	228,844.08	1,703,151.11	748.7
P141	227,848.08	1,703,883.83	783.8
P142	227,783.18	1,703,118.84	748.8
P143	228,118.08	1,703,883.88	713.3
P1	228,108.81	1,703,581.80	713.0
P2	228,08.44	1,703,808.03	723.0
P3	228,812.38	1,703,800.48	788.8
P4	228,088.83	1,703,888.70	780.3
P5	228,181.18	1,703,883.48	780.0
P6	228,888.27	1,703,117.82	713.0
P7	228,888.08	1,703,817.08	713.0
P8	228,884.08	1,703,808.48	718.0

*COORDINATES AND ELEVATIONS SHALL BE DETERMINED FROM PHASE I AS-BUILT SURVEY.

CURVE DATA						
CURVE NO.	LOCATION OF P.I. NORTHING	EASTING	Δ	R	L	T
C1	228,841.80	1,702,878.80	28°47'52"	48.00	48.48	28.88
C2	228,872.41	1,702,882.40	88°30'38"	15.00	23.18	14.68
C3	228,134.14	1,702,802.08	23°23'17"	48.00	28.17	14.88
C4	228,887.57	1,702,848.48	48°41'37"	18.00	28.28	18.80
C5	228,328.28	1,702,888.83	30°18'38"	15.00	28.31	18.27
C6	228,844.08	1,702,888.17	18°22'88"	90.00	8.28	4.14

NOTES

1. SEE SHEET 1 FOR LEGEND AND GENERAL NOTES.
2. REMOVE EXISTING DRAINAGE STRUCTURES AND TRANSPORT TO STORAGE AREA AS DIRECTED BY THE OWNER. BACKFILL EXCAVATIONS WITH CLAY AS SPECIFIED FOR CLAY LINER.
3. FIELD LOCATE SUMP AS APPROVED BY OWNER.

0 100 200 FEET
SCALE

2	1/31/89	ISSUED FOR PHASE II CONSTRUCTION	KJK / RTF
1	1/31/89	ISSUED FOR PERMITTING	KJK / RTF
NO.	DATE	REVISION	DRAWN / CHKD

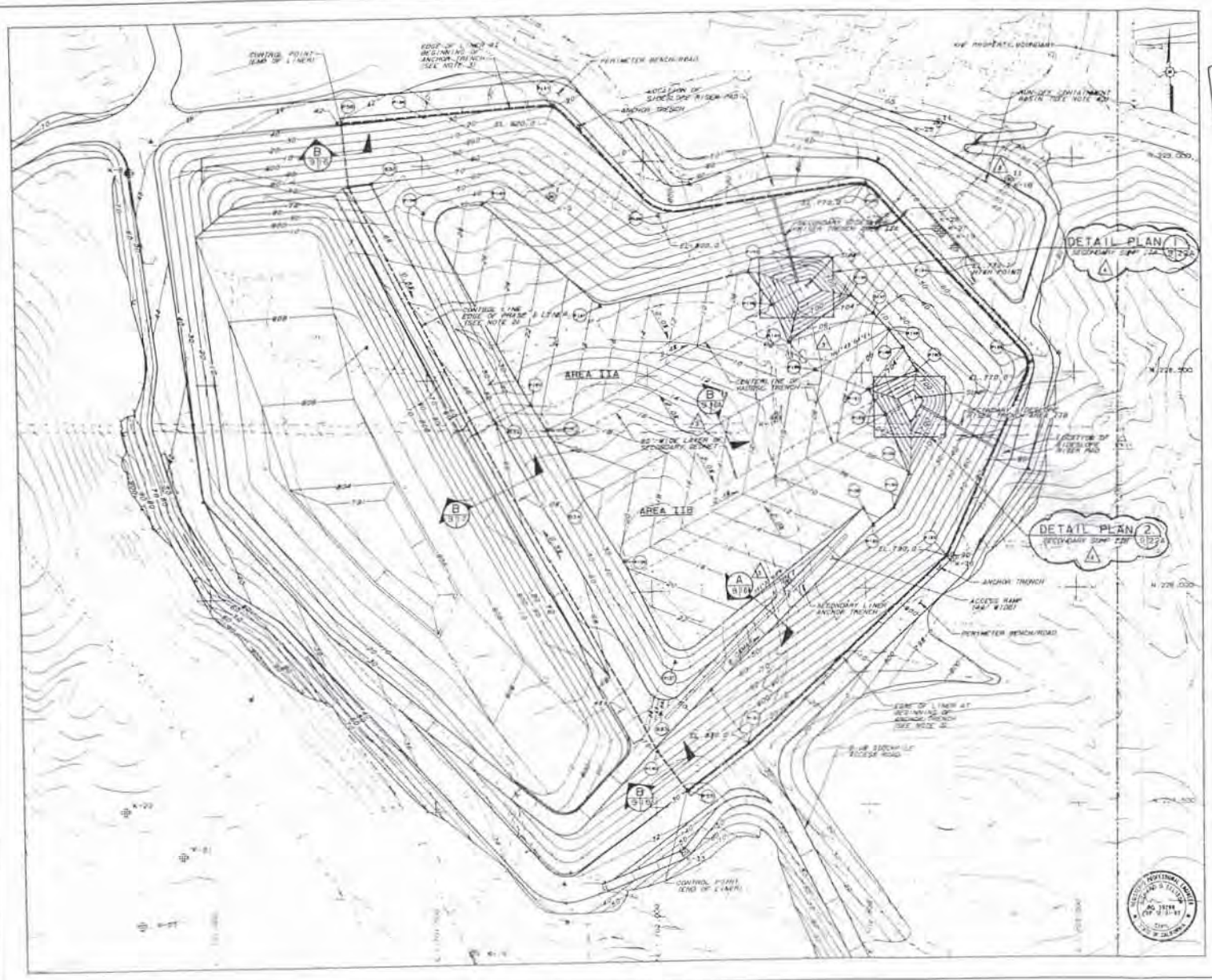
DRAWN BY: KJK CHKD BY: RTF DSGN BY: RDE

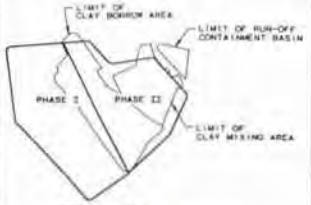
LANDFILL UNIT B-18
CONSTRUCTION PLANS
CWM-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

PHASE II EXCAVATION
AND
SITE PREPARATION PLAN

ENVIRONMENTAL SOLUTIONS, INC.

JOB NO. 89-977	DWG NO. D-59977-F0008	REVISION 2
SCALE 1"=100'	DATE 89-03-304 2 1	SHEET 8

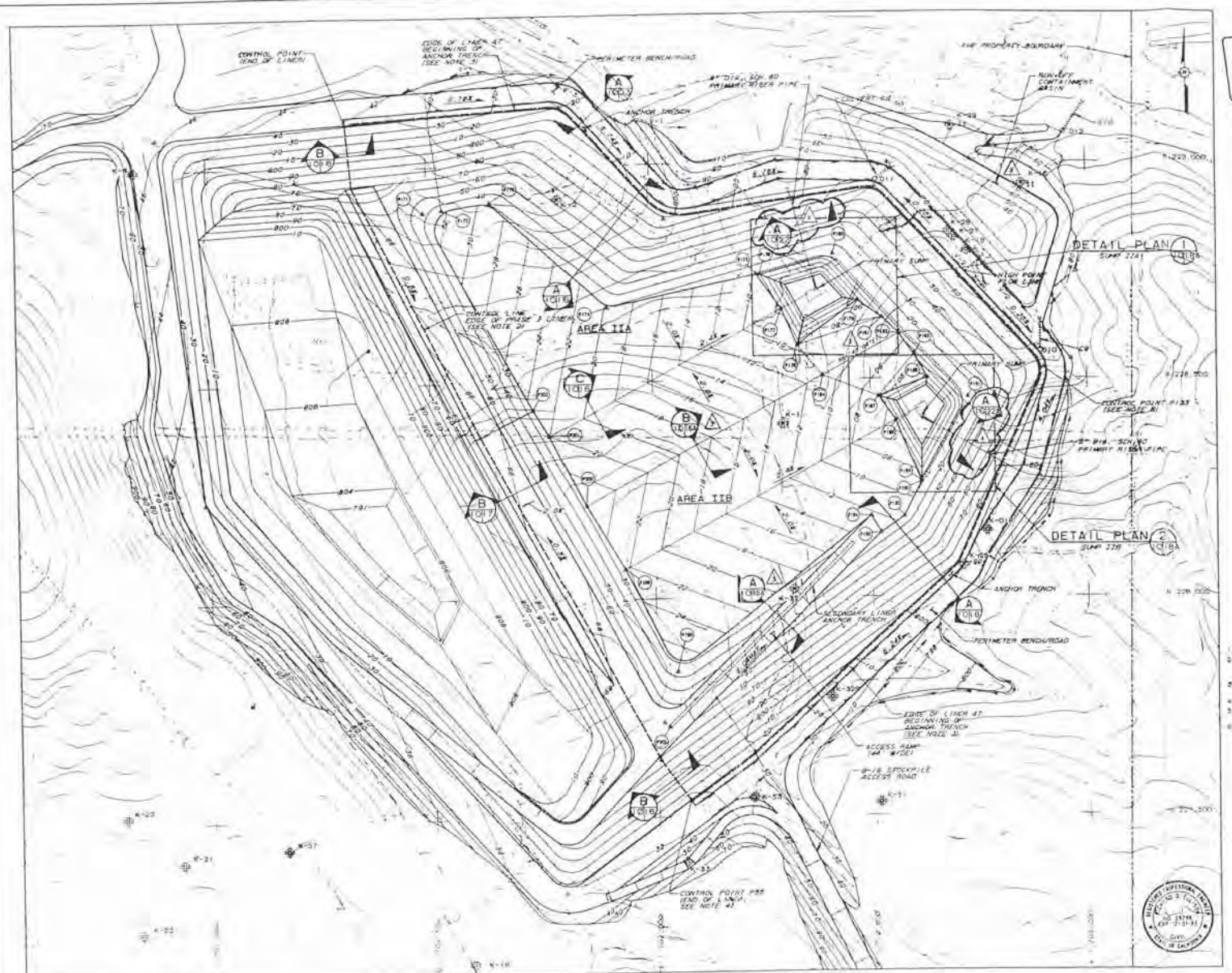


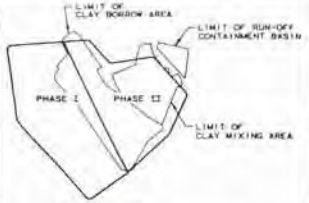


KEY PLAN

POINTS OF INTERSECTION

POINT	NORTHING	EASTING	ELEV.
P101	227,583.86	755,171.92	801.5
P102	227,780.17	755,216.37	801.5
P103	227,980.67	755,264.16	795.0
P104	227,842.01	755,314.25	795.0
P105	228,014.14	755,448.8	790.0
P106	228,154.05	755,601	790.0
P107	228,154.05	755,601	790.0
P108	228,445	755,537.98	790.0
P109	228,445	755,537.98	790.0
P110	228,445	755,512.98	790.0
P111	228,445	755,512.98	790.0
P112	228,445	755,512.98	790.0
P113	228,445	755,512.98	790.0





KEY PLAN

POINTS OF INTERSECTION

POINT	NORTHING	EASTING	ELEV.
P-111	228,884.06	1,701,481.73	708.7
P-112	228,900.39	1,701,534.86	702.4
P-113	228,912.15	1,701,540.85	708.3
P-114	228,870.78	1,701,888.33	708.8
P-115	228,742.72	1,702,040.37	708.8
P-116	228,740.52	1,702,048.73	708.3
P-117	228,863.08	1,702,086.70	707.8
P-118	228,917.87	1,702,081.46	708.8
P-119	228,880.45	1,702,472.38	708.0
P-120	228,787.43	1,702,514.73	708.0
P-121	228,640.86	1,702,538.23	708.0
P-122	228,804.48	1,702,588.13	708.8
P-123	228,924.51	1,702,588.81	708.8
P-124	228,908.32	1,702,888.13	711.0
P-125	228,588.83	1,702,583.18	708.1
P-126	228,301.34	1,702,838.48	708.1
P-127	228,458.42	1,702,918.38	708.8
P-128	228,403.42	1,702,944.37	708.1
P-129	228,265.06	1,703,815.80	708.4
P-130	228,263.32	1,703,818.06	708.7
P-131	228,258.34	1,703,721.38	----
P-132	228,483.1	1,702,274.30	711.8
P-133	228,[illegible].1	1,703,330.43	718.2
P-134	228,183.43	1,703,487.88	718.8
P-135	227,922.74	1,703,348.88	708.4
P-136	228,088.75	1,701,893.48	723.3
P-137	228,808.43	1,701,833.58	730.8
P-138	228,811.38	1,701,887.08	730.3
P-139	228,814.37	1,701,892.30	729.2
P-200	228,818.70	1,701,842.59	728.8
P-201	228,877.18	1,701,787.38	733.0
P-202	228,888.05	1,701,781.75	733.3
P-203	227,708.33	1,703,288.24	----

CULVERT SCHEDULE[3]

NO.	DIA. (IN.)	LENGTH (FT.)
C4	30	70
C5	24	40
C13	18	80

DROP INLET SCHEDULE

NO.	DEPTH (FT.)	DIA. (IN.)[4]
D10	4	30
D11	4	24
D12	4	18

V-DITCH SCHEDULE

TYPE	W (FT)	DEPTH (IN)
V-1	5	1
V-2	5	1
V-3	5	1.5
V-4	5	2
V-5	5	2.5
V-6	5	3
V-7	5	1.25

NOTES

1. SEE SHEET 1 FOR LEGEND AND GENERAL NOTES.
2. THE WEST SIDE SECONDARY LINER CONTROL LINE IS THE EDGE OF LINER AT THE BEGINNING OF THE ANCHOR TRENCH.
3. THE NORTH, SOUTH AND EAST SECONDARY LINER CONTROL LINE IS THE TOP OF THE SLOPE.
4. ALL DROP INLETS SHALL BE OF 12 GAGE THICKNESS.
5. ALL CULVERTS AND OUTLETS SHALL BE OF 16 GAGE THICKNESS.
6. THE DRAINAGE FROM CONTROL POINT P55 TO CONTROL POINT P133 IS NOT A PART OF PHASE II CONSTRUCTION.

0 100 200 FEET
SCALE

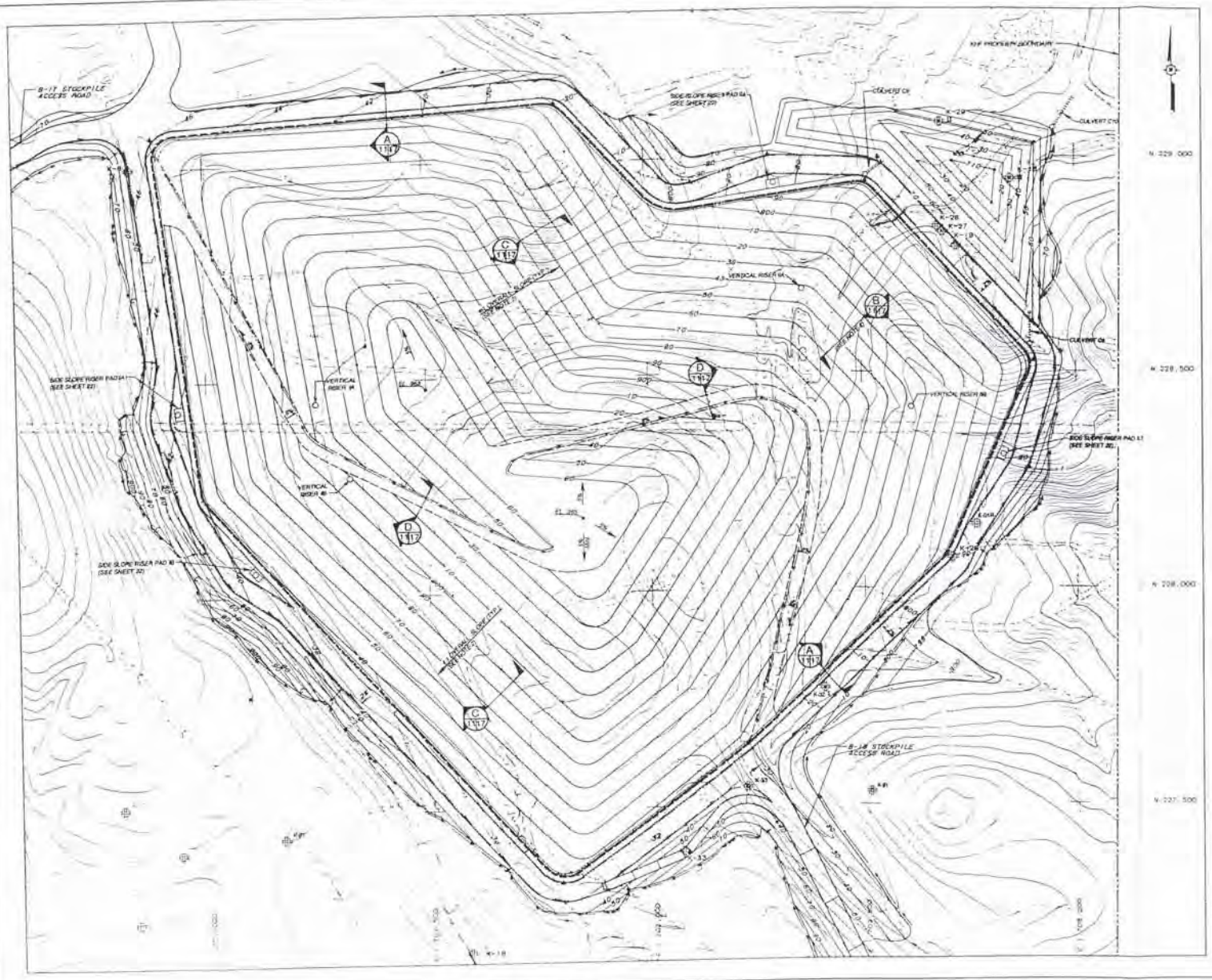
REV.	DATE	DESCRIPTION	BY	APP.
4	12/06/89	REVISED CALL OUTS FOR RAMPS AND RISERS	GGI	
3	1/29/89	ACCESS ROAD WIDTH REDUCED TO 44'. ELIMINATED PHASE I14-8 (1/12/84) DEMOLISHED TYPICAL SECTIONS 6-10/84, 8-10/84 MODIFIED GRADING IN SUMP AREA MODIFIED SUMPS CONTROL TO REFLECT WORK CHANGES	GGI	
2	10/84	ISSUED FOR PHASE II CONSTRUCTION	KJR	
0	7/30/83	ISSUED FOR PERMITTING	KJR	

LANDFILL UNIT B-18
CONSTRUCTION PLANS
CWM-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

PHASE II
TOP OF PRIMARY LINER

ENVIRONMENTAL SOLUTIONS, INC.

JOB NO.	DWG. TITLE NUMBER	REV. NO.
89-977	D-89977-FD010	4
SCALE 1"=100'	DWG. TITLE NUMBER 89-03-304 2 1	SHEET 10



NOTES

1. SEE SHEET 1 FOR LEGEND AND GENERAL NOTES.
2. BENCHES WILL BE PROVIDED AT APPROXIMATELY 50-FOOT INTERVALS AS SHOWN ON SECTION B, SHEET 17. SLOPES BETWEEN BENCHES WILL HAVE A SLOPE OF APPROXIMATELY 2.5:1. THE BENCHES WILL SLOPE AT 3% TO CONTROL DRAINAGE.

0 100 200 FEET
SCALE

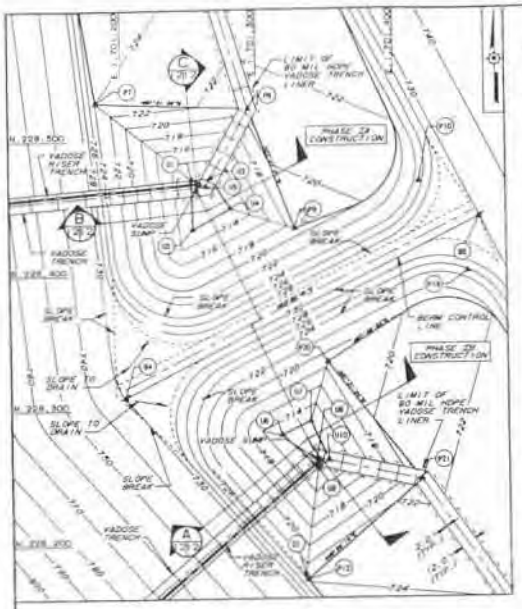
2	7/26/95	ISSUED FOR PHASE II CONSTRUCTION	KJW	
0	7/26/95	ISSUED FOR PERMITTING	MD	
REV	DATE	DESCRIPTION	DRAWN BY	CHK'D BY

LANDFILL UNIT B-18
CONSTRUCTION PLANS
CWMI-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

PHASE I AND II
FINAL CLOSURE PLAN

ENVIRONMENTAL SOLUTIONS, INC

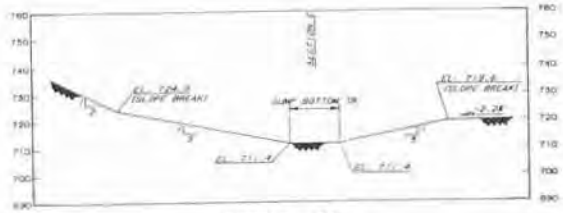
89-877	D-89977-FD011	2
1"=100'	89-03-304.2 1	11



DETAIL PLAN (1/3)

PHASE 2 SUMPS

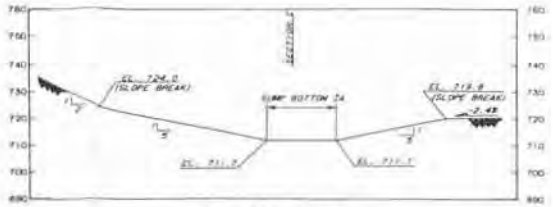
0 40 80 FEET
SCALE



SECTION (A/3)

TRANSVERSE SECTION SUMP 2B

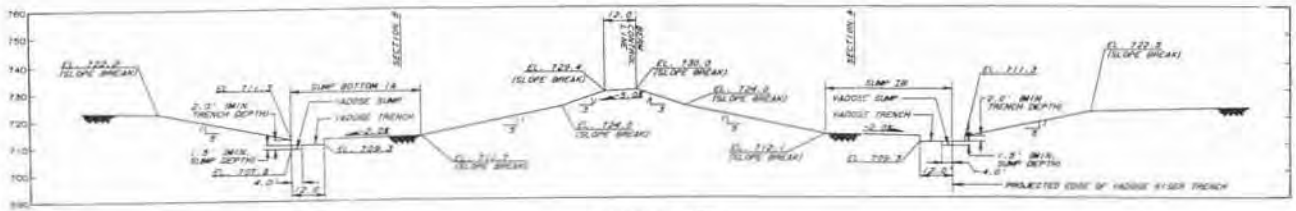
0 20 40 FEET
SCALE



SECTION (B/3)

TRANSVERSE SECTION SUMP 2A

0 20 40 FEET
SCALE



SECTION (C/3)

LONGITUDINAL SECTION SUMPS 2A AND 2B

0 20 40 FEET
SCALE

SURVEY CONTROL			
STATION	COORDINATE N	COORDINATE E	ELEVATION
P7	228,528.63	1,701,181.87	724.0
P8	228,523.45	1,701,294.07	721.0
U	228,471.09	1,701,455.04	711.4
U2	228,472.23	1,701,265.72	709.8
U3	228,433.91	1,701,252.13	713.0
U4	228,449.43	1,701,279.94	711.8
U5	228,465.95	1,701,292.13	709.5
P9	228,433.86	1,701,327.34	716.8
P4	228,509.28	1,701,202.96	730.0
P20	228,534.73	1,701,550.01	718.6
U6	228,240.94	1,701,316.10	712.1
U7	228,231.11	1,701,357.20	712.5
U8	228,263.80	1,701,341.30	709.4
U9	228,228.80	1,701,342.26	711.3
U10	228,261.96	1,701,350.04	709.9
P21	228,246.92	1,701,411.46	721.4
P12	228,173.47	1,701,333.23	724.0
P10	228,468.26	1,701,418.16	723.3
P13	228,401.18	1,701,452.35	720.0
S5	228,441.73	1,701,482.00	720.0
S1	228,177.31	1,701,231.98	729.5

REGISTERED PROFESSIONAL ENGINEER
JOSEPH A. FREDERICK
No. 41021
Exp. 06/31/98
CIVIL
STATE OF CALIFORNIA

NOTES:

1) ELEVATIONS SHOWN ARE PHASE 1 EXCAVATION

3	01/08/98	ISSUED FOR CONSTRUCTION	RVH	
2	08/14/96	ISSUED FOR BID	RVH	
1	03/13/96	ISSUED FOR PERMITTING	MD	
NO.	DATE	DESCRIPTION	BY	CK'D

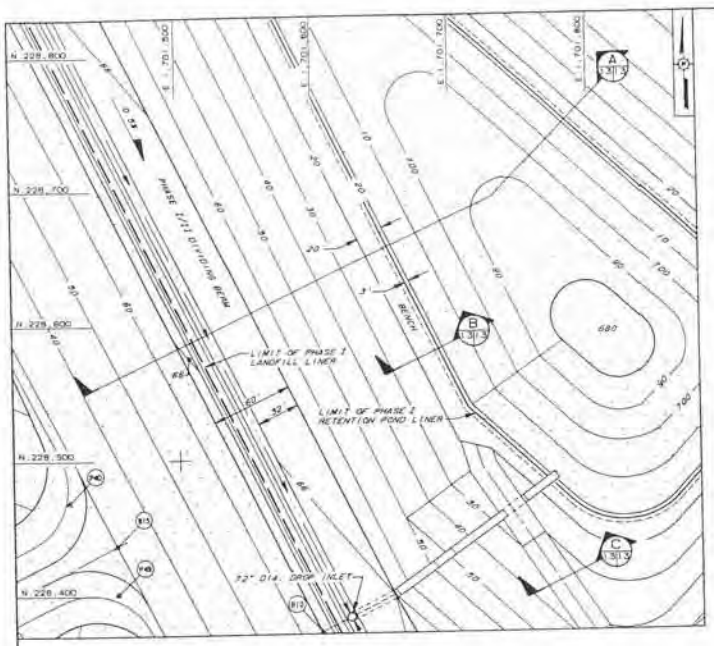
DRAWN BY RVH/MD DESIGNED BY CHECKED BY KDC

LANDFILL UNIT B-1B
CONSTRUCTION PLANS
CWMI-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

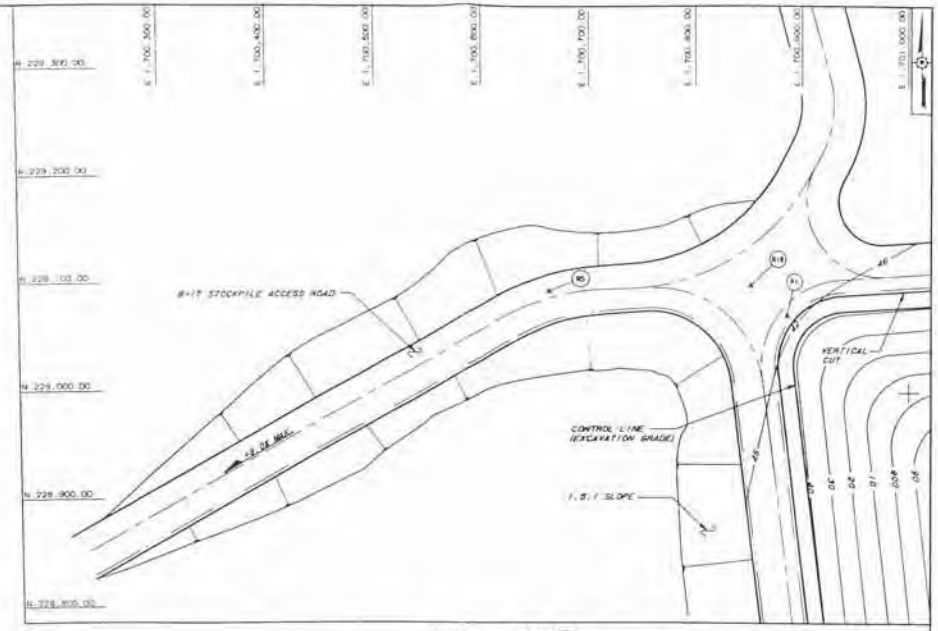
GRADING DETAILS

ENVIRONMENTAL SOLUTIONS, INC.

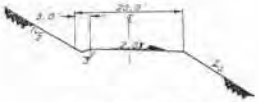
JOB NO.	DWG NO.	REVISION
89-877	D-89977-FD012	2
SCALE AS NOTED	DWG TITLE 89-03-304.2 1	SHEET 12



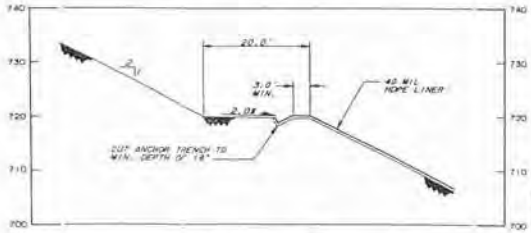
DETAIL PLAN (1) (4/3)
PHASE I RETENTION BASIN
0 40 80 FEET SCALE



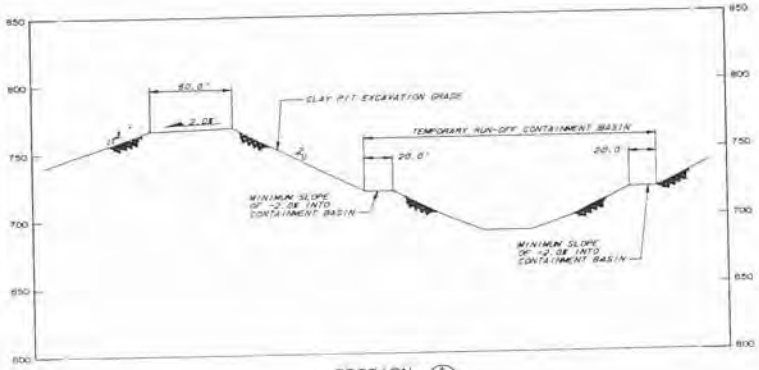
DETAIL PLAN (1) (3/3)
S.W. ROAD INTERSECTION
0 50 100 FEET SCALE



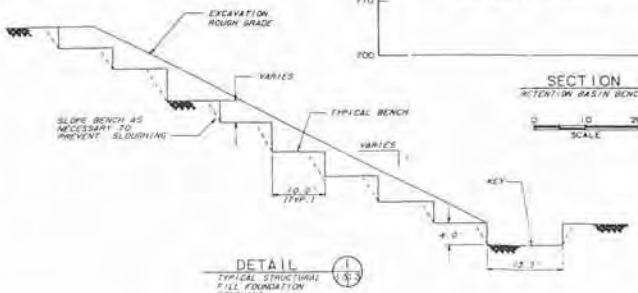
SECTION (C) (3/3)
RETENTION BASIN ACCESS ROAD
0 10 20 FEET SCALE



SECTION (B) (3/3)
RETENTION BASIN BENCH
0 10 20 FEET SCALE



SECTION (A) (3/3)
PHASE I BERM FILL RETENTION BASIN EXCAVATION GRADE
0 40 80 FEET SCALE



DETAIL (1) (3/3)
TYPICAL STRUCTURAL FILL FOUNDATION BENCHING
0 10 20 FEET SCALE

NOTE
1. SEE SHEET 1 FOR LEGEND AND GENERAL NOTES

REGISTERED PROFESSIONAL ENGINEER JEFFREY A. FARENWALD No. 41021 Exp. MAY 31 19 CIVIL STATE OF CALIFORNIA

2	1/29/91	ISSUED FOR CONSTRUCTION	RVW	KKJ
0	7/31/90	ISSUED FOR PERMITTING	MC	KKJ
REV	DATE	DESCRIPTION	DRWN	CHK

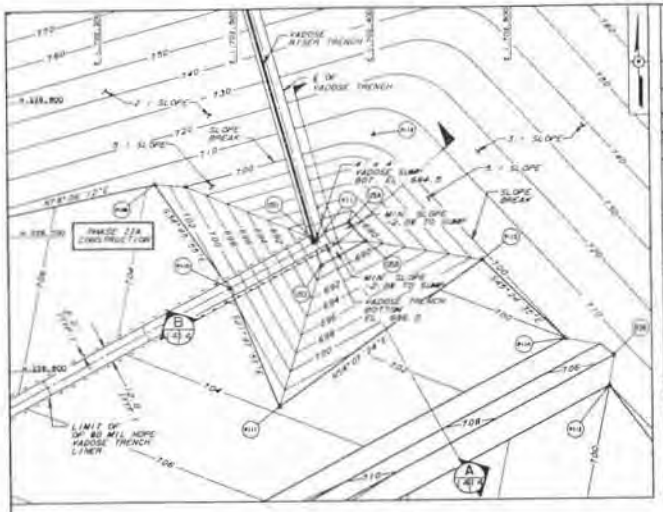
DESIGNED R.Villanueva DRAWN E.Fabiano CHECKED KDE

LANDFILL UNIT B-18
CONSTRUCTION PLANS
CWM-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

GRADING DETAILS

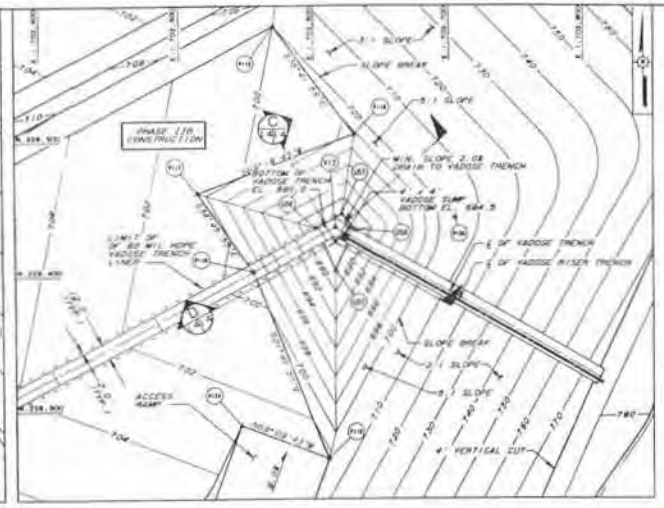
ENVIRONMENTAL SOLUTIONS, INC.

89-977	D-89977-FD013	2
AS NOTED	89-03-304.2.1	13



DETAIL PLAN 1 (89-4)
PHASE 2A SUMP EXCAVATION GRADE

0 40 80 FEET
SCALE



DETAIL PLAN 2 (89-4)
PHASE 2B SUMP EXCAVATION GRADE

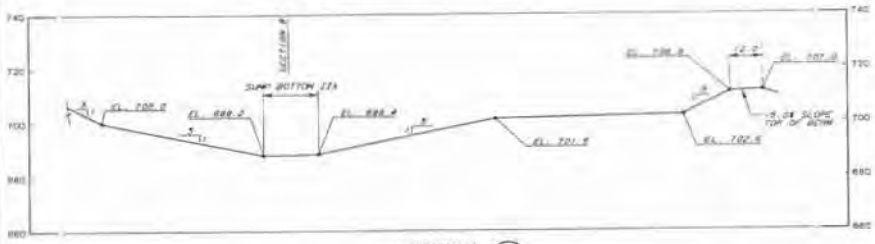
0 40 80 FEET
SCALE

SURVEY CONTROL

STATION	COORDINATE N	COORDINATE E	ELEVATION
US1	228,694.65	702,353.30	688.0
US2	228,678.57	702,360.08	688.4
US3	228,694.30	702,393.04	688.4
US4	228,708.34	702,380.42	688.4
US5	228,403.94	702,614.92	689.5
US6	228,454.34	702,604.51	688.0
US7	228,445.93	702,619.52	688.1
US8	228,434.71	702,629.71	688.0
V11	228,893.14	702,317.83	686.0
V12	228,455.78	703,614.73	686.0
P108	228,738.06	702,339.47	703.8
P110	228,860.43	702,293.34	700.0
P111	228,512.06	702,328.69	703.9
P112	228,679.61	702,477.39	699.0
P113	228,771.80	702,399.87	700.0
P114	228,820.21	702,437.03	700.0
P115	228,585.63	702,589.13	700.0
P116	228,305.15	702,629.03	697.7
P117	228,481.28	702,512.83	701.0
P118	228,402.73	702,553.42	698.5
P119	228,284.20	702,606.85	700.8
P120	228,453.26	702,703.72	700.0
P124	228,289.03	702,943.82	702.8
926	228,606.01	702,572.79	706.0

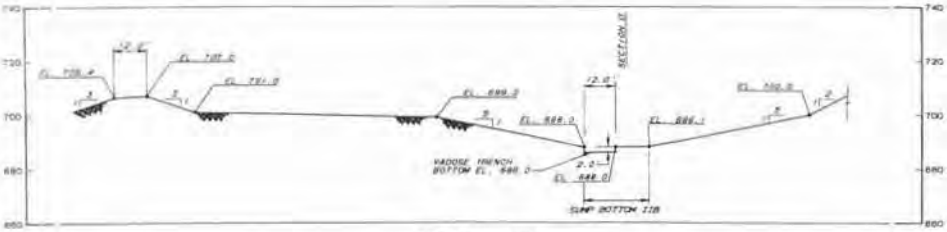
NOTES:

1. ELEVATIONS CALLED OUT IN SECTION MAY ONLY APPLY ALONG THAT SECTION LINE; CHECK AGAINST PLAN.
2. THE LIMIT OF SUMPS 11A AND 11B ARE DEFINED BY THE 3:1 SLOPE BREAK. EVERYTHING DOWNSLOPE OF THE SLOPE BREAK IS TO BE CONSIDERED SUMP 11A OR 11B AS APPROPRIATE



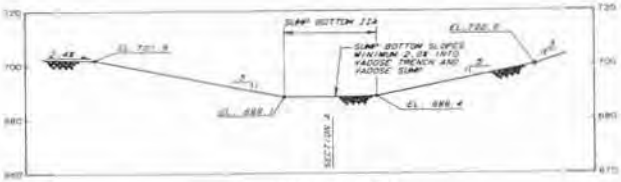
SECTION A (4/4)
SUMP 11A

0 20 40 FEET
SCALE



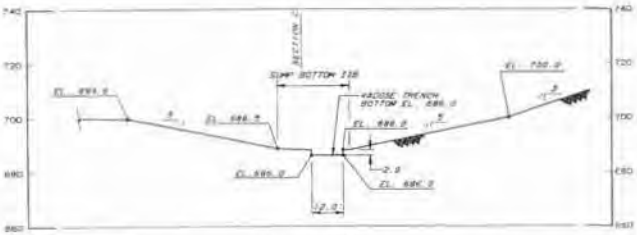
SECTION C (4/4)
SUMP 11B

0 20 40 FEET
SCALE



SECTION B (4/4)
SUMP 11A

0 20 40 FEET
SCALE



SECTION D (4/4)
SUMP 11B

0 20 40 FEET
SCALE

REGISTERED PROFESSIONAL ENGINEER
RICHARD K. FRANKLIN
No. 41021
Exp. 7-31-92
CIVIL
STATE OF CALIFORNIA

D	10/89	ISSUED FOR PHASE II CONSTRUCTION	RVH	RKF
C	10/89	ISSUED FOR PERMITTING	KJF	RKF
REV	DATE	DESCRIPTION	DRN	CHK

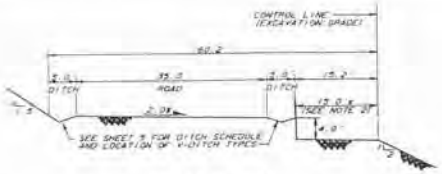
RKF

LANDFILL UNIT B-1B
CONSTRUCTION PLANS
CWMI-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

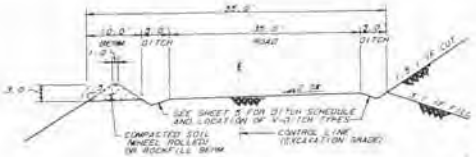
GRADING DETAILS

ENVIRONMENTAL SOLUTIONS, INC.

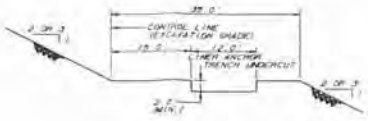
89-977	D-89977-014	2
AS SHOWN	89-03-304.2.1	14



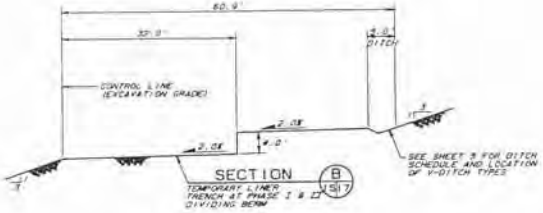
SECTION A (3|8|5)
PERIMETER BENCH/ROAD
0 10 20 FEET SCALE



SECTION E (3|5)
ACCESS ROAD WITH BERM
0 10 20 FEET SCALE

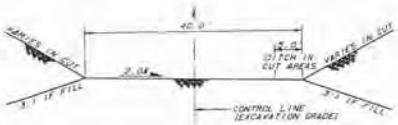


SECTION C (3|5)
PHASE 2 ACCESS RAMP
0 10 20 FEET SCALE

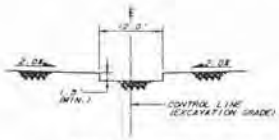


SECTION B (V5|7)
TEMPORARY LINER TRENCH AT PHASE I & II (DIVIDING BERM)

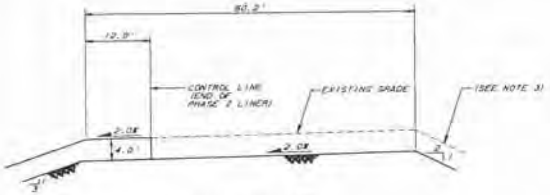
SECTION D (3|5)
PHASE I & II (DIVIDING BERM)
0 10 20 FEET SCALE



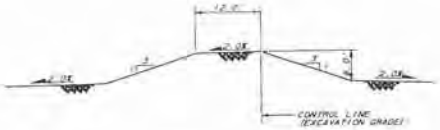
SECTION E (3|5)
ACCESS ROAD (TYP.)
0 10 20 FEET SCALE



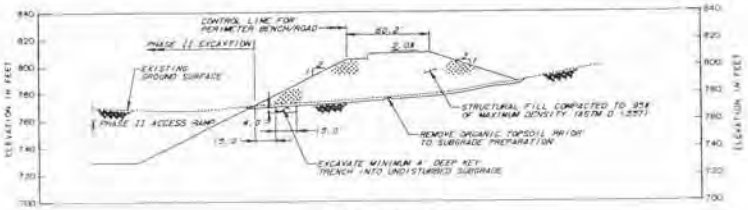
SECTION F (3|5)
VADOSE TRENCH
0 10 20 FEET SCALE



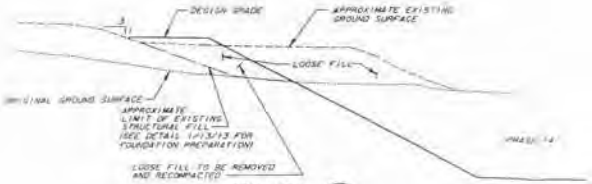
SECTION B (8|5)
LINER EXTENSION AT PHASE I & II DIVIDING BERM
0 10 20 FEET SCALE



SECTION C (8|5)
AREA IIA AND IIB DIVIDING BERM
0 10 20 FEET SCALE



SECTION D (8|5)
PERIMETER BENCH/ROAD AT FILL (TYPICAL)
0 40 80 FEET SCALE



SECTION A (3|8|5)
EXISTING FILL @ NORTH END OF PHASE 1A (LOOKING EAST)
0 40 80 FEET SCALE

REGISTERED PROFESSIONAL ENGINEER
DAVID K. PARKINSON
No. 41021
Exp. 3-31-95
CIVIL
STATE OF CALIFORNIA

NOTES

1. SEE SHEET 1 FOR LEGEND AND GENERAL NOTES.
2. SEE DETAIL 3.16.16 FOR COMPLETED ANCHOR TRENCH DETAILS. WIDTH OF TOP LINER TRENCH EXCAVATION MAY BE VARIED TO SUIT EXCAVATION AND BACKFILLING EQUIPMENT.
3. REMOVE THE MATERIAL TO 3.5 FEET BELOW ORIGINAL DESIGN GRADE TO ALLOW FOR PLACEMENT OF PHASE II LINER

0 10 20 FEET SCALE

3	10/94	ISSUED FOR PHASE II CONSTRUCTION	KJR	RCP
2	2/94	ISSUED FOR CONSTRUCTION	KJR	RCP
1	9/93	ISSUED FOR BID	KJR	RCP
0	1/93	ISSUED FOR PERMITTING	KAB	RCP
REV	DATE	DESCRIPTION	DRAWN BY	CHK'D BY

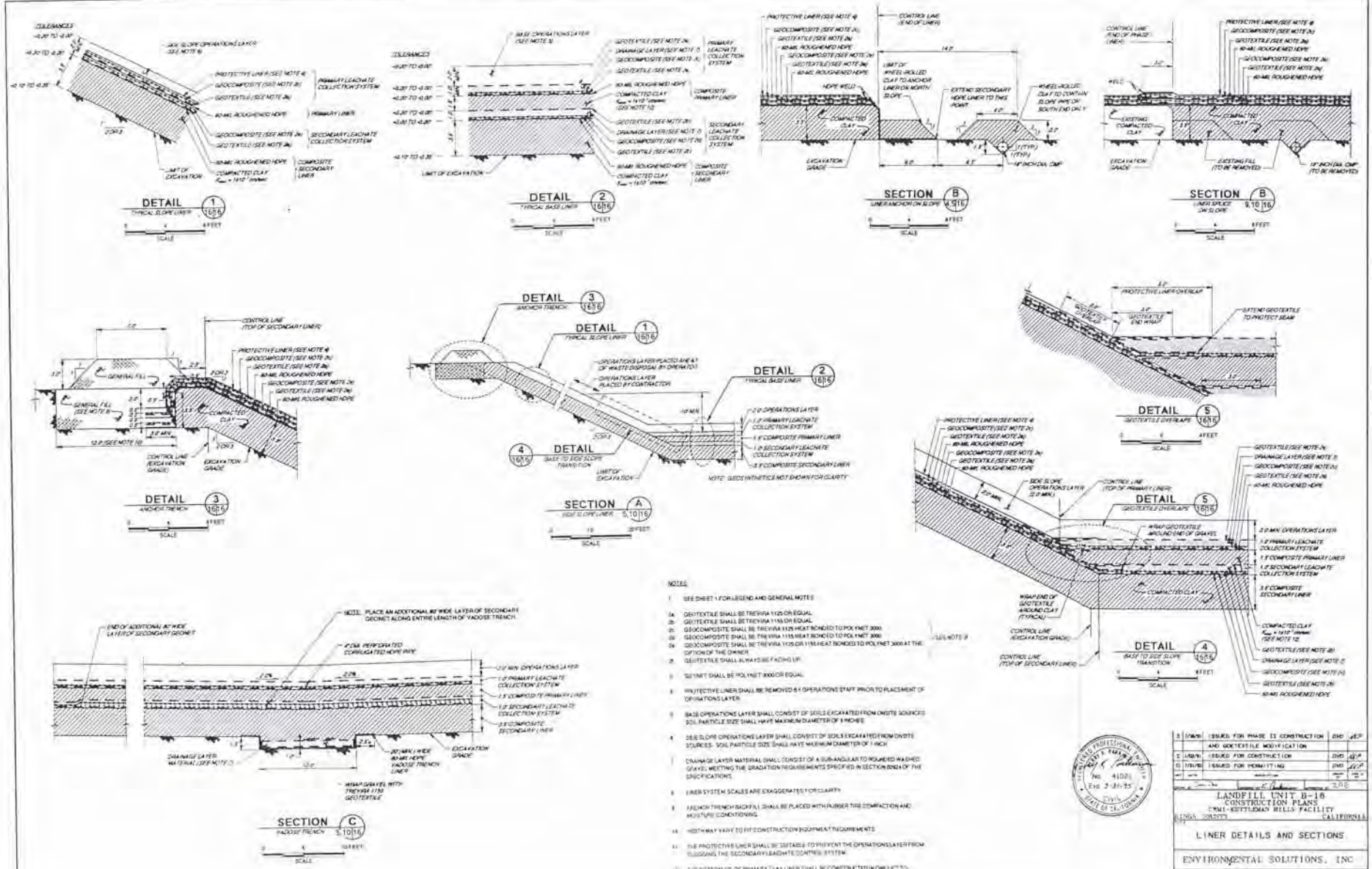
DRAWN: K. Kaiser CHK'D: R. Parkinson APP'D: V.O.C.

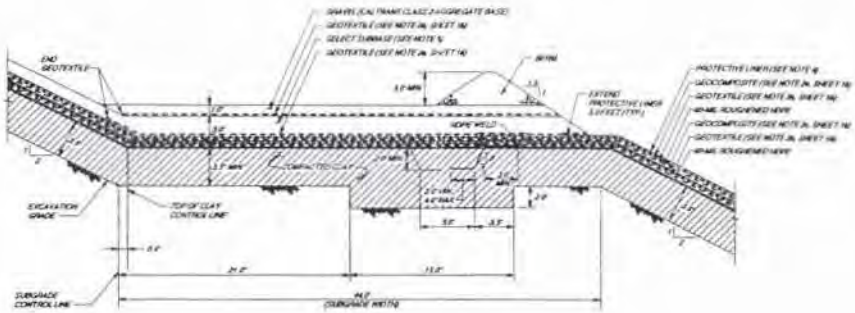
LANDFILL UNIT B-18
CONSTRUCTION PLANS
CWM-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

CROSS SECTIONS

ENVIRONMENTAL SOLUTIONS, INC.

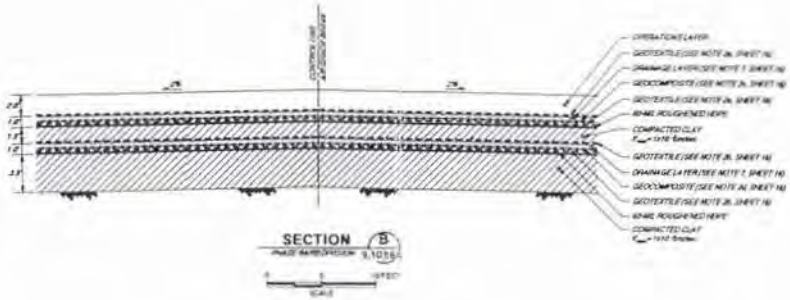
JOB NO.	DETAIL NUMBER	REVISION
89-977	D-89977-FD015	3
SCALE 1"=10'	FILE NAME 89-03-304.2.1	SHEET 15


DETAIL 1 (16/16) TYPICAL SLOPE LINER
DETAIL 2 (16/16) TYPICAL BASE LINER
SECTION B (9,10/16) LINER ANCHOR ON SLOPE
SECTION B (9,10/16) LINER SPLICE ON SLOPE
DETAIL 3 (16/16) ANCHOR TRENCH
DETAIL 1 (16/16) TYPICAL SLOPE LINER
DETAIL 2 (16/16) TYPICAL BASE LINER
DETAIL 4 (16/16) BASE TO SIDE SLOPE TRANSITION
SECTION A (5,10/16) SIDE TO SIDE LINER
DETAIL 5 (16/16) GEOTEXTILE OVERLAPS
DETAIL 5 (16/16) GEOTEXTILE OVERLAPS
DETAIL 4 (16/16) BASE TO SIDE SLOPE TRANSITION
DETAIL 3 (16/16) ANCHOR TRENCH
SECTION C (5,10/16) HADOSE TRENCH
NOTE: GEOSYNTHETICS NOT SHOWN FOR CLARITY
NOTES
1 SEE SHEET 1 FOR LEGEND AND GENERAL NOTES
2a GEOTEXTILE SHALL BE TREVIRA 1125 OR EQUAL
2b GEOTEXTILE SHALL BE TREVIRA 1115 OR EQUAL
2c GEOCOMPOSITE SHALL BE TREVIRA 1125 HEAT BONDED TO POLYNET 3000
2d GEOCOMPOSITE SHALL BE TREVIRA 1115 HEAT BONDED TO POLYNET 3000
2e GEOCOMPOSITE SHALL BE TREVIRA 1125 OR 1115 HEAT BONDED TO POLYNET 3000 AT THE OPTION OF THE OWNER
3 GEOTEXTILE SHALL ALWAYS BE FACING UP
4 GEONET SHALL BE POLYNET 3000 OR EQUAL
5 PROTECTIVE LINER SHALL BE REMOVED BY OPERATIONS STAFF PRIOR TO PLACEMENT OF OPERATIONS LAYER
6 BASE OPERATIONS LAYER SHALL CONSIST OF SOILS EXCAVATED FROM ONSITE SOURCES. SOIL PARTICLE SIZE SHALL HAVE MAXIMUM DIAMETER OF 3 INCHES
7 SIDE SLOPE OPERATIONS LAYER SHALL CONSIST OF SOILS EXCAVATED FROM ONSITE SOURCES. SOIL PARTICLE SIZE SHALL HAVE MAXIMUM DIAMETER OF 1 INCH
8 DRAINAGE LAYER MATERIAL SHALL CONSIST OF A SUB-ANGULAR TO ROUNDED WASHED GRAVEL MEETING THE GRADATION REQUIREMENTS SPECIFIED IN SECTION 02924 OF THE SPECIFICATIONS
9 LINER SYSTEM SCALES ARE EXAGGERATED FOR CLARITY
10 TRENCH TRENCH BACKFILL SHALL BE PLACED WITH RUBBER TIRE COMPACTION AND MOISTURE CONDITIONING
11 HDPE MAY VARY TO FIT CONSTRUCTION EQUIPMENT REQUIREMENTS
12 THE PROTECTIVE LINER SHALL BE SUITABLE TO PREVENT THE OPERATIONS LAYER FROM CLOGGING THE SECONDARY LEACHATE COLLECTION SYSTEM
13 THE BOTTOM 12" OF PRIMARY CLAY LINER SHALL BE CONSTRUCTED IN ONE LIFT TO PROTECT UNDERLYING GEOSYNTHETIC LINER SYSTEM. THE UPPER 12" WILL HAVE A HYDRAULIC CONDUCTIVITY OF K = 10⁻⁷ CM/SEC
LICENSED PROFESSIONAL ENGINEER
No. 41021
Exp. 3-31-95
Civil
STATE OF CALIFORNIA
LANDFILL UNIT B-18
CONSTRUCTION PLANS
CRM1-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA
LINER DETAILS AND SECTIONS
ENVIRONMENTAL SOLUTIONS, INC
89-977 0-89977-FD016 3
AS NOTED 89-03-304.2 1 18



SECTION A
PHASE II ACCESS RAMP 8.1076

SCALE



SECTION B
PHASE II MANIFOLD SECTION 8.1056

SCALE

NOTES:

1. SEE SHEET 1 FOR LEGEND AND GENERAL NOTES.
2. SEE SHEET 14 FOR GEOSYNTHETIC MATERIAL NOTES.
3. REMOVE PROTECTIVE LINER AS WASTE IS ADVANCED UP THE SLOPE.
4. LINER ELEMENT THICKNESSES AND SPACING EXAGGERATED FOR CLARITY.
5. SELECT SUBBASE SHALL BE ONSITE MATERIAL WITH A MAXIMUM PARTICLE SIZE LESS THAN 1", CLASSIFIED AS SW OR SM BY THE UNIFIED SOIL CLASSIFICATION SYSTEM, AND SHALL CONFORM TO THE FOLLOWING GRADATION:

SIEVE SIZE	PERCENT PASSING SIEVE
1"	100
NO. 4	70-100
NO. 10	40-90
NO. 40	20-60
NO. 200	5-20

THE MATERIAL SHALL BE SUBANGULAR AND SHALL ALSO CONFORM TO THE FOLLOWING REQUIREMENTS:

TEST	TEST METHOD NO.	REQUIREMENT
R-VALUE[1]	CALIF 301	60 (MIN.)
SAND EQUIVALENT	CALIF 217	20 (MIN.)

UPPER TWO FEET OF SELECT SUBBASE SHALL BE COMPACTED TO 90% OF MAXIMUM DRY DENSITY PER ASTM D1557.

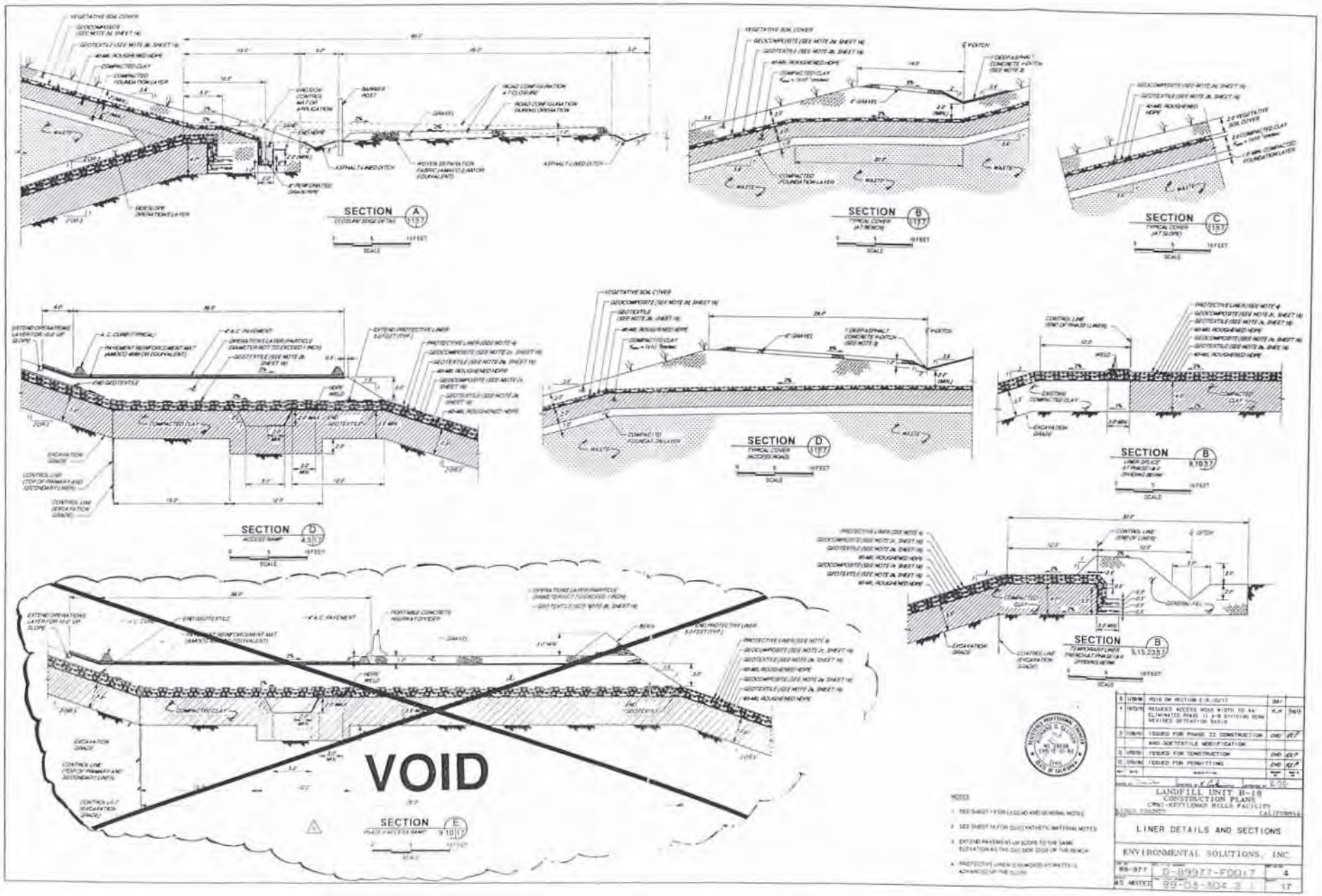
[1] R-VALUE REQUIREMENT MAY BE WAIVED IF MATERIAL HAS A SE ≥ 30.

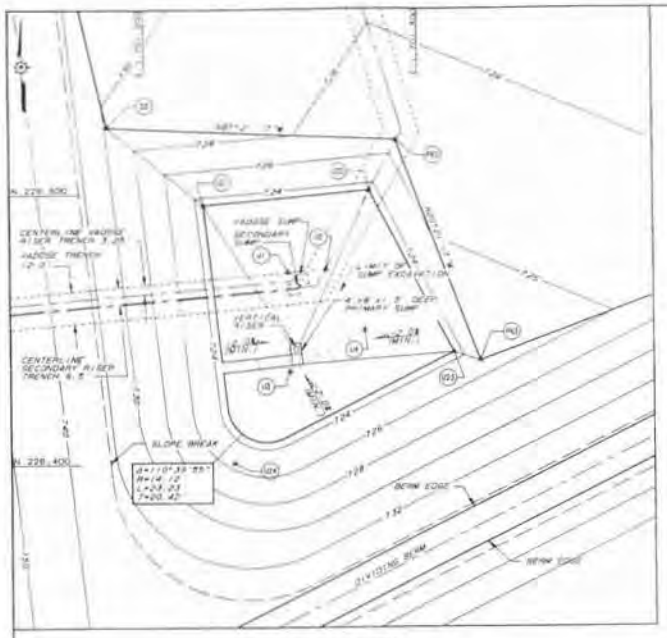
LANDFILL UNIT D-18
CONSTRUCTION PLANS
CWMI-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

PHASE II
LINER DETAILS AND SECTIONS

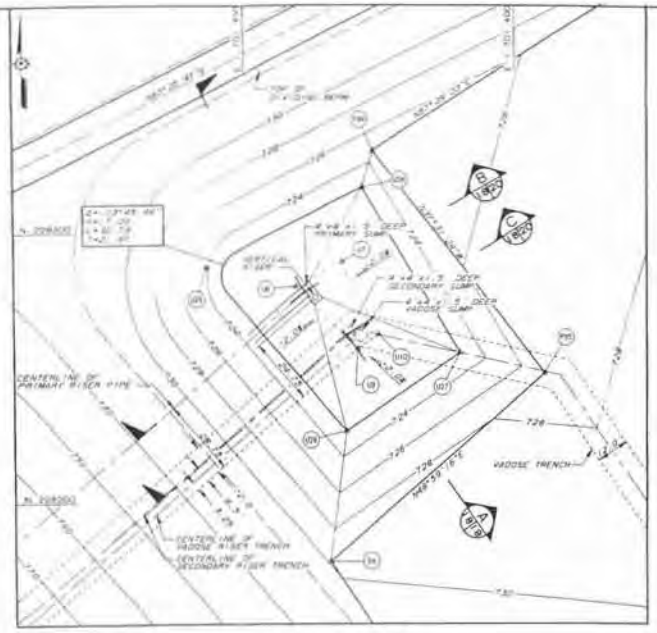
ENVIRONMENTAL SOLUTIONS, INC.

89-977	D-89977-016A	1
AS SHOWN	89-03-304-2	16A


SECTION A
SECTION B
SECTION C
SECTION D
SECTION D
SECTION B
SECTION B
SECTION E
VOID
NOTES
LANDFILL UNIT H-18
CONSTRUCTION PLANS
CWM-KETTLEMAN HILLS FACILITY
LINER DETAILS AND SECTIONS
ENVIRONMENTAL SOLUTIONS, INC.
D-89977-F0017
99-04-304



DETAIL PLAN 1



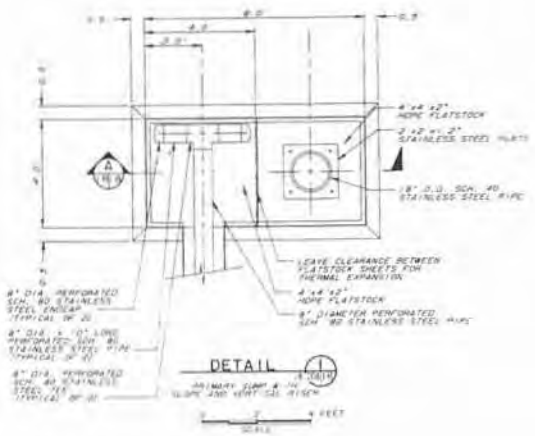
DETAIL PLAN 2

SCALE 0 20 40 FEET

SURVEY CONTROL			
STATION	COORDINATE N	COORDINATE E	ELEVATION
1A *	228,471.09	701,350.04	711.2
1B *	228,441.33	701,265.72	709.6
1C *	228,443.81	701,152.12	712.0
1D *	228,449.43	701,070.94	711.8
1E *	228,460.84	701,016.10	712.1
1F *	228,461.11	701,037.20	712.0
1G *	228,476.00	701,142.26	711.3
2A *	228,261.96	701,350.04	728.8
2B	228,326.86	701,398.33	729.8
2C	228,320.13	701,325.58	727.0
2D	228,431.96	701,320.00	724.5
2E	228,436.12	701,321.55	723.4
2F	228,301.42	701,281.80	723.4
2G	228,441.14	701,313.42	723.4
2H	228,331.20	701,347.60	723.4
2I	228,180.20	701,348.31	724.8
2J	228,243.54	701,411.69	727.4
2K	228,172.86	701,531.56	728.8
2L	228,286.81	701,385.64	723.4
2M	228,316.81	701,344.31	723.4
2N	228,203.24	701,380.12	723.4
2O	228,228.14	701,337.94	723.4

* CONTROL POINTS FROM EXCAVATION GRADE

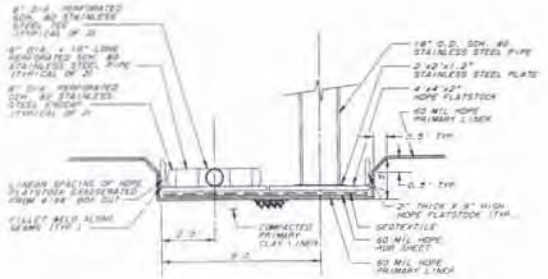
SURVEY CONTROL (SECONDARY LINER (SEE NOTE 2))			
STATION	COORDINATE N	COORDINATE E	ELEVATION
1J1	228,480.16	701,240.95	716.9
1J2	228,481.87	701,277.53	716.9
1J3	228,437.50	701,346.78	716.9
1J4	228,448.96	701,288.81	716.9
1J5	228,289.43	701,307.90	716.9
1J6	228,298.35	701,339.85	716.9
1J7	228,259.23	701,362.27	716.9
1J8	228,244.41	701,340.88	716.9



DETAIL 1

PRIMARY SUMP & 18" SLOPE AND VENT OF RISER

SCALE 0 2 4 FEET



SECTION A

PRIMARY SUMP WITH SLOPE AND VERTICAL RISER

SCALE 0 2 4 FEET

NOTES:

1. ELEVATIONS SHOWN ARE TOP OF PRIMARY LINER
2. SURVEY CONTROL GIVEN IN THE LOWER SURVEY CONTROL TABLE ARE CONTROL ELEVATIONS FOR THE TOP OF SECONDARY LINER. THE POINTS GIVEN ARE FOR THE BOTTOM OF SUMPS 1A AND 1B RESPECTIVELY.

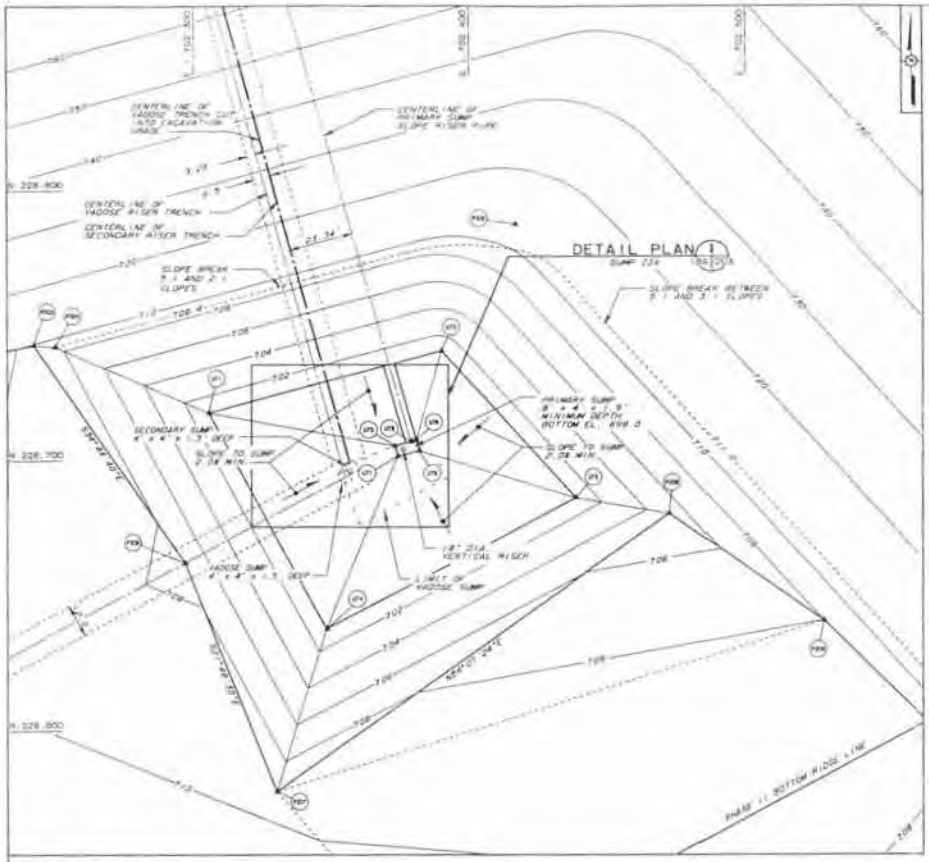
REV	DATE	DESCRIPTION	BY	CHK'D
3	2/28/98	MODIFIED CALLOUT FOR DETAIL 1/1B/1B CLARIFIED DIMENSIONS OF HDPE RAMP & SUMPS	GCL	
2	1/5/98	ISSUED FOR CONSTRUCTION	RGH	
0	12/2/97	ISSUED FOR PERMITTING	RGH	

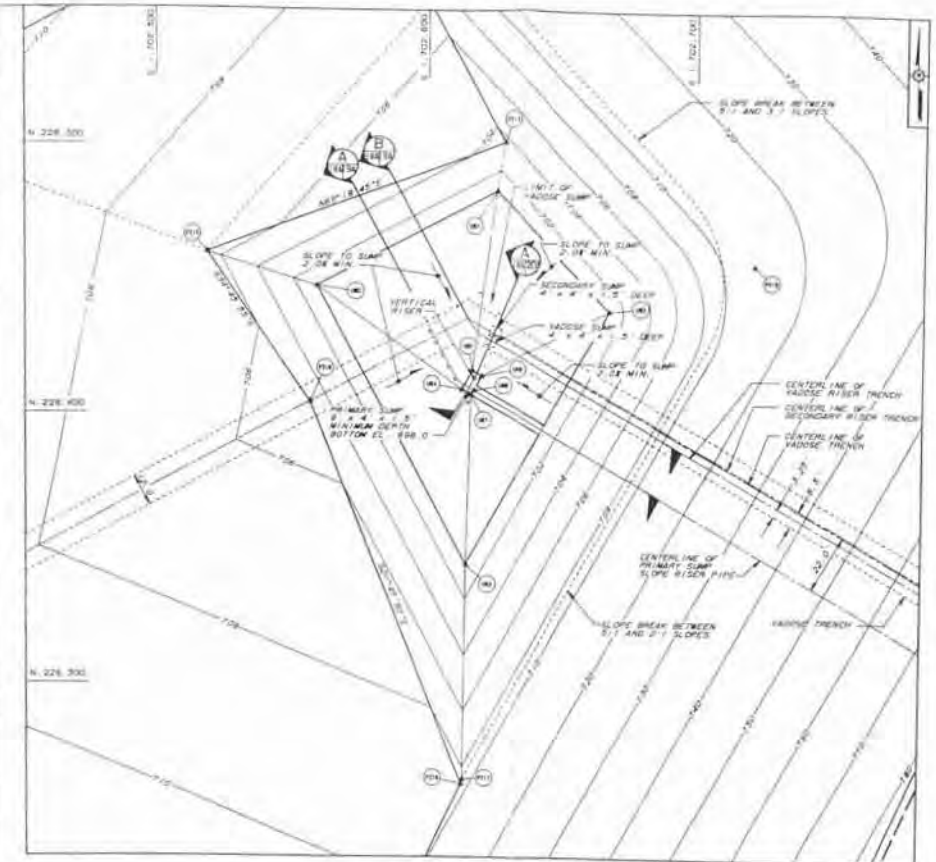
LANDFILL UNIT B-1B
CONSTRUCTION PLANS
CWMI-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

TYPICAL SUMP PLANS

ENVIRONMENTAL SOLUTIONS, INC.

89-977	0-89977-FD018	3
NONE	89-030304.2.1	18





SURVEY CONTROL

STATION	COORDINATE N	COORDINATE E	ELEVATION
P203	228,742.12	702,380.77	708.8
P204	228,742.04	702,348.15	708.4
P205	228,787.63	702,416.72	---
P206	228,602.09	702,296.70	707.6
P207	228,577.81	702,331.48	709.6
P208	228,680.45	702,472.58	708.0
P209	228,460.28	702,328.27	708.0
P214	228,301.34	702,628.46	705.7
P215	228,459.64	702,518.39	708.8
P216	228,404.49	702,356.57	705.3
P217	228,265.05	702,513.50	708.4
P218	228,263.34	702,613.06	708.1
P219	228,456.48	702,721.58	---
U71	228,717.66	702,305.45	700.8
U72	228,740.62	702,369.91	700.8
U73	228,686.35	702,439.70	700.8
U74	228,637.78	702,348.43	700.8
U75	228,705.42	702,313.31	699.5
U76	228,707.11	702,380.73	699.5
U77	228,701.55	702,374.05	699.5
U78	228,703.87	702,381.78	699.5
U79	228,706.99	702,379.80	699.0
U80	228,447.43	702,538.94	700.8
U81	228,483.28	702,561.56	700.8
U82	228,438.64	702,667.57	700.8
U83	228,344.57	702,614.31	700.8
U84	228,408.47	702,612.53	699.5
U85	228,418.39	702,610.56	699.5
U86	228,414.42	702,630.04	699.5
U87	228,407.38	702,631.11	699.5
U88	228,409.28	702,617.28	699.0

CONTROL POINTS STATIONS ARE TOP OF CLAY, NOT ON NOT INCLUDE THE 40-MILLS LINER.

DETAIL PLAN 1
SUMP 22A
0 20 40 FEET
SCALE

SURVEY CONTROL*

STATION	COORDINATE N	COORDINATE E	ELEVATION
U81	228,705.89	702,327.84	694.0
U82	228,719.71	702,393.78	694.0
U83	228,891.50	702,408.18	694.0
U84	228,862.89	702,355.97	694.0
U85	228,439.89	702,584.05	694.0
U86	228,460.22	702,601.84	694.0
U87	228,435.04	702,644.70	694.0
U88	228,382.91	702,614.70	694.0

*-STATIONS U81-U88 ARE SECONDARY LINER CONTROL POINTS AND RELATE TO CONTROL POINTS U71-U74 AND U81-U84.

DETAIL PLAN 2
SUMP 22B
0 20 40 FEET
SCALE

NOTES:
1. SECTIONS SHOWN ON SHEET 18A-B 20A ARE TYPICAL OF BOTH SUMPS. PLAN VIEW DIMENSIONS TAKE PRECEDENCE OVER DIMENSIONS FOUND ON TYPICAL SECTION VIEWS.

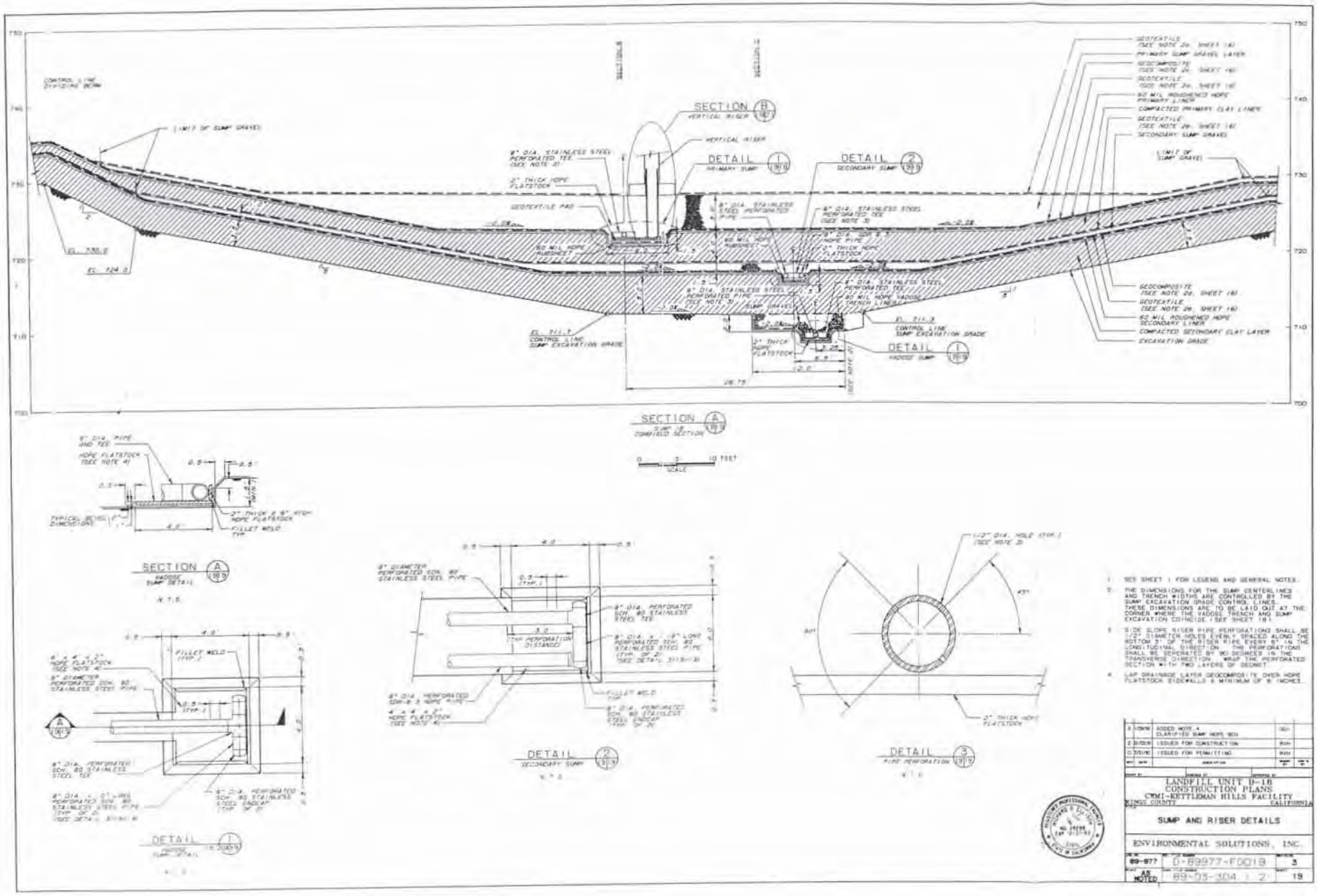
3	12/8/93	MODIFIED GRADING PER DWG. MODIFIED SURVEY CONTROL. ADDED CALLOUTS FOR DETAIL PLAN 1/18A/20A AND SECTIONS A-18A/18B, B/18A/18B, A-18A/20A.
2	7/28/93	ISSUED FOR PHASE II CONSTRUCTION
1		

LANDFILL UNIT B-18
CONSTRUCTION PLANS
CWMI-KETTLEMAN HILLS FACILITY
KINGS COUNTY, CALIFORNIA

PHASE II
PRIMARY SUMP GRADING DETAILS

ENVIRONMENTAL SOLUTIONS, INC.

89-977	D-89977-SU018	3
AS NOTED	89-03-304.2.1	18A



1. SEE SHEET 1 FOR LEGEND AND GENERAL NOTES.
2. THE DIMENSIONS FOR THE SUMP CENTERLINES AND TRENCH WIDTHS ARE CONTROLLED BY THE SUMP EXCAVATION GRADE CONTROL LINES. THESE DIMENSIONS ARE TO BE LAID OUT AT THE CORNER WHERE THE VADOSE TRENCH AND SUMP EXCAVATION COINCIDE (SEE SHEET 1B).
3. SIDE SLOPE RISER PIPE PERFORATIONS SHALL BE 1/2" DIAMETER HOLES (VERIFY) SPACED ALONG THE BOTTOM 3" OF THE RISER PIPE EVERY 6" IN THE LONGITUDINAL DIRECTION. THE PERFORATIONS SHALL BE SEPERATED BY 90 DEGREES IN THE TRANSVERSE DIRECTION. WRAP THE PERFORATED SECTION WITH TWO LAYERS OF GEONET.
4. LAP DRAINSIDE LAYER GEOCOMPOSITE OVER HDPE FLATSTOCK SIDEWALLS A MINIMUM OF 6 INCHES.

3	12/09/09	ADDED NOTE 4, CLARIFIED SUMP HDPE BOX	SDJ	
2	10/30/09	ISSUED FOR CONSTRUCTION	RVH	
0	7/31/09	ISSUED FOR PERMITTING	RVH	
REV.	DATE	DESCRIPTION	DRAWN BY	CHECKED BY

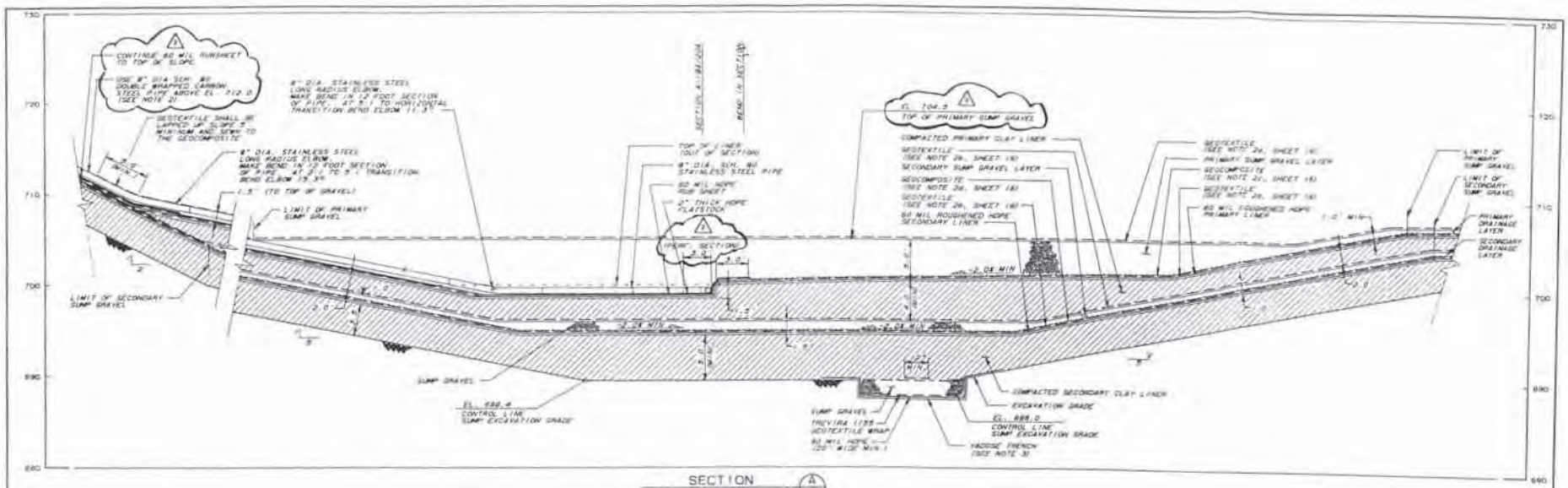
DRAWN BY: | CHECKED BY:

LANDFILL UNIT D-1B
CONSTRUCTION PLANS
CWMI-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

SUMP AND RISER DETAILS

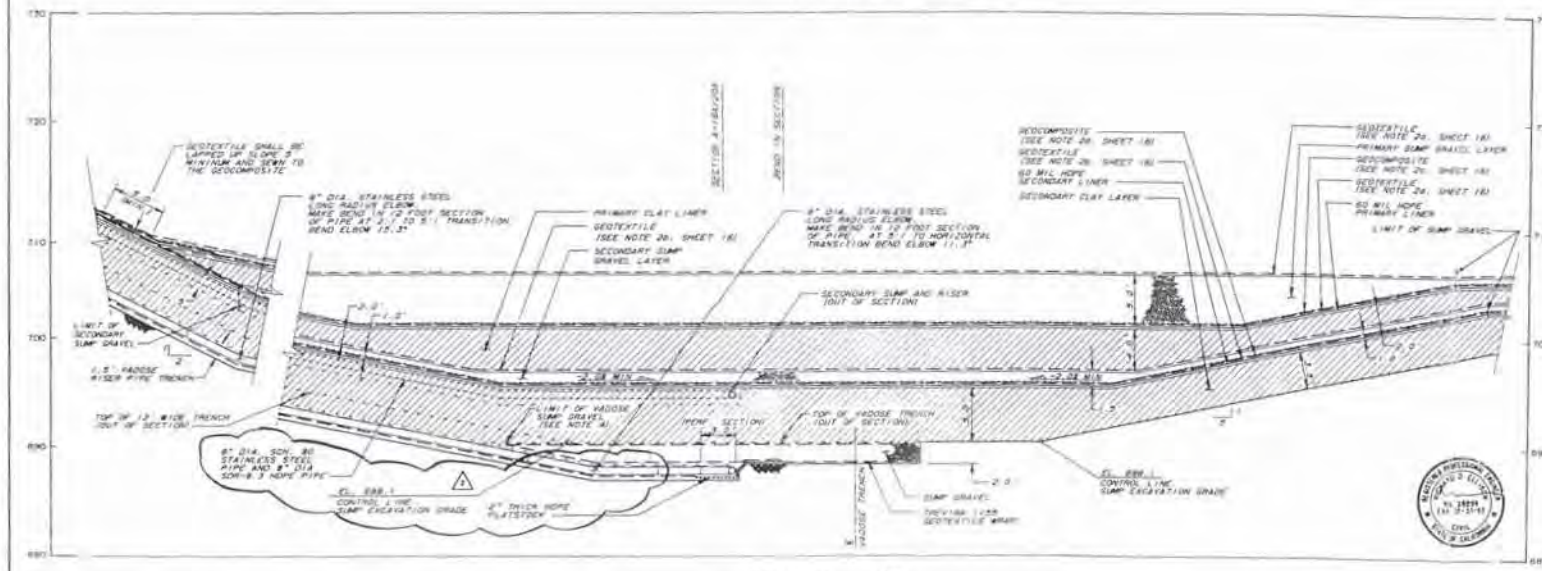
ENVIRONMENTAL SOLUTIONS, INC.

JOB NO.	DWG. NO.	REV.
89-877	D-89877-F0018	3
SCALE: AS NOTED	DATE: 89-03-304.1.2	SHEET: 19



SECTION A

TYPICAL PHASE II PRIMARY SUMP AND SIDESLOPE RISER



SECTION B

TYPICAL PHASE II HADDIE SUMP AND SIDESLOPE RISER

NOTES:

1. SEE SHEET 1 FOR LEGEND AND GENERAL NOTES.
2. WRAP CARBON STEEL PIPE WITH TWO LAYERS OF CARTOOLE OR APPROVED EQUAL.
3. DUE TO BEND IN SECTION THE HADDIE TRENCH & PRIMARY SUMP RISER TRENCH ARE SHOWN OUT OF PLANE WITH RESPECT TO EACH OTHER.
4. HADDIE SUMP GRAVEL TERMINATES BY FEATHERING INTO 5:1 SLOPE INSIDE THE HADDIE SUMP RISER PIPE (SIM).

0 5 10 FEET
SCALE

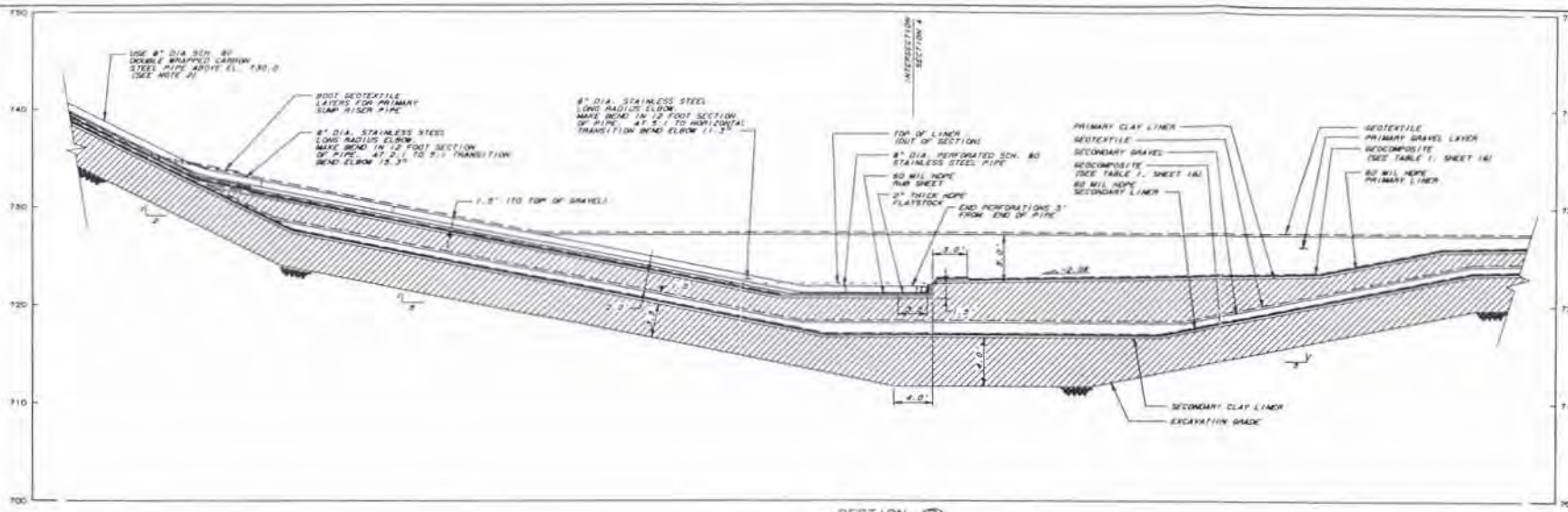
2	10/12/94	REVISED NOTES FOR PHASE II RISERS AND SUMPS	
1	12/94	ADDED SECTIONS A-A' 08/94, B-B' 08/94	
REV.	DATE	DESCRIPTION	

LANDFILL UNIT B-18
CONSTRUCTION PLANS
CWMI-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

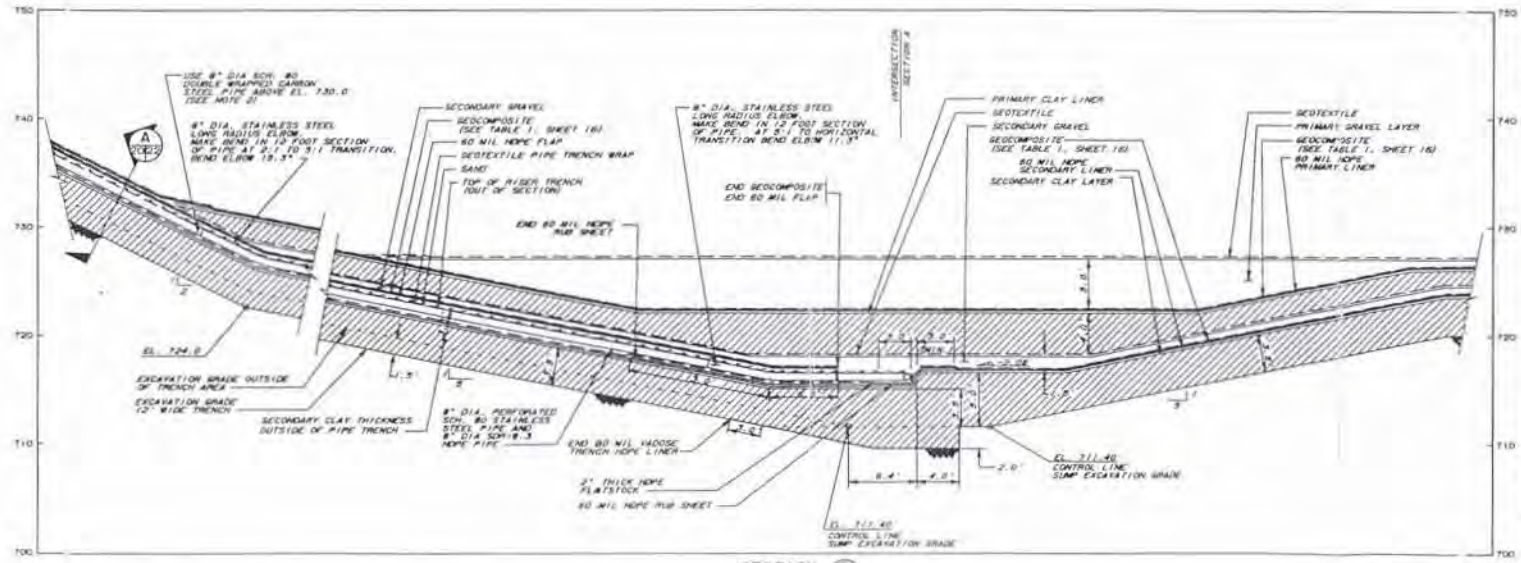
PHASE II
SUMP AND RISER DETAILS

ENVIRONMENTAL SOLUTIONS, INC.

80-977	D-89977-SU019	2
1"=5'	89-03-304.2.1	IBA



SECTION B
PRIMARY SUMP AND RISER



SECTION C
SECONDARY SUMP AND RISER AT PIPE TRENCH

NOTES:

1. SEE SHEET 1 FOR LEGEND AND GENERAL NOTES.
2. WRAP CARBON STEEL PIPE WITH TWO LAYERS OF TAPECOAT OR APPROVED EQUAL.

REGISTERED PROFESSIONAL ENGINEER JEFF E. PARTRIDGE No. 41021 Exp. 06/31/99 CIVIL STATE OF CALIFORNIA

0 5 10 FEET
SCALE

2	3/99	ISSUED FOR CONSTRUCTION	RYH
0	7/96	ISSUED FOR PERMITTING	RYH
REV	DATE	DESCRIPTION	BY

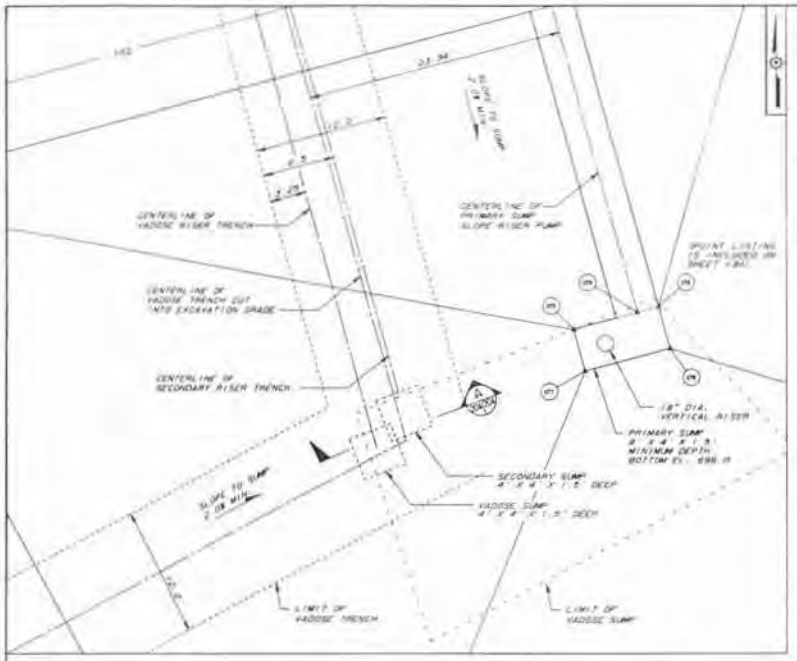
DRAWN BY R.Y. Hosoda CHECKED BY J. Partridge APPROVED BY EOG

LANDFILL UNIT B-1B
CONSTRUCTION PLANS
CWMI-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

SUMP AND RISER DETAILS

ENVIRONMENTAL SOLUTIONS, INC.

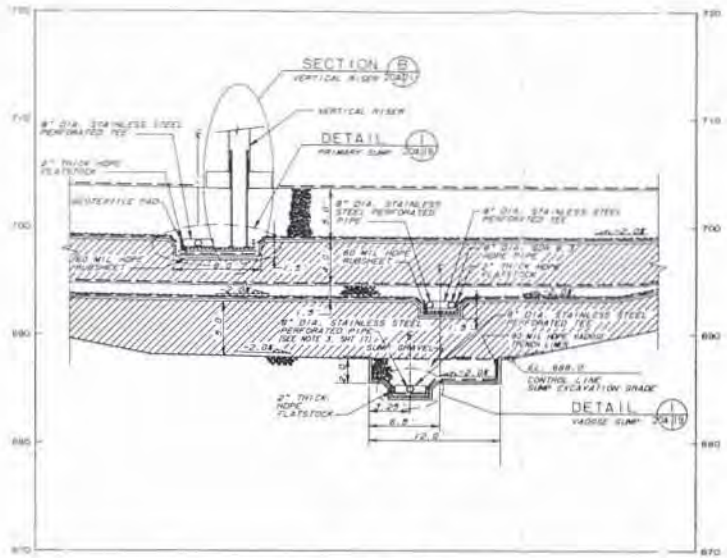
89-977	D-89977-FD020	2
1"=5'	89-03-304.2.1	20



DETAIL PLAN (1) 8A/20A

PHASE II A SUMP TOP OF PRIMARY LINER

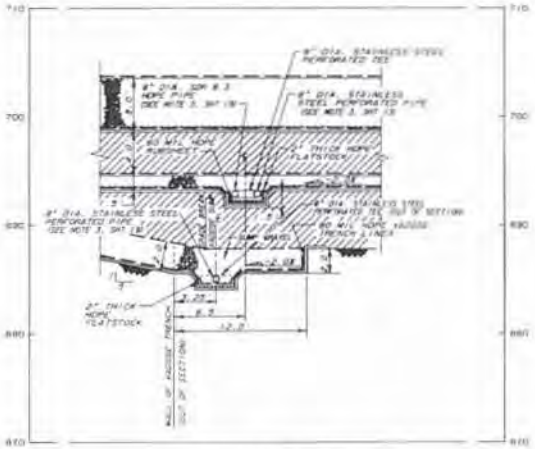
SCALE 0 5 10 FEET



SECTION (A) 8A/20A

SUMP 11A

SCALE 0 5 10 FEET



SECTION (A) 8A/20A

SUMP 11A

SCALE 0 5 10 FEET

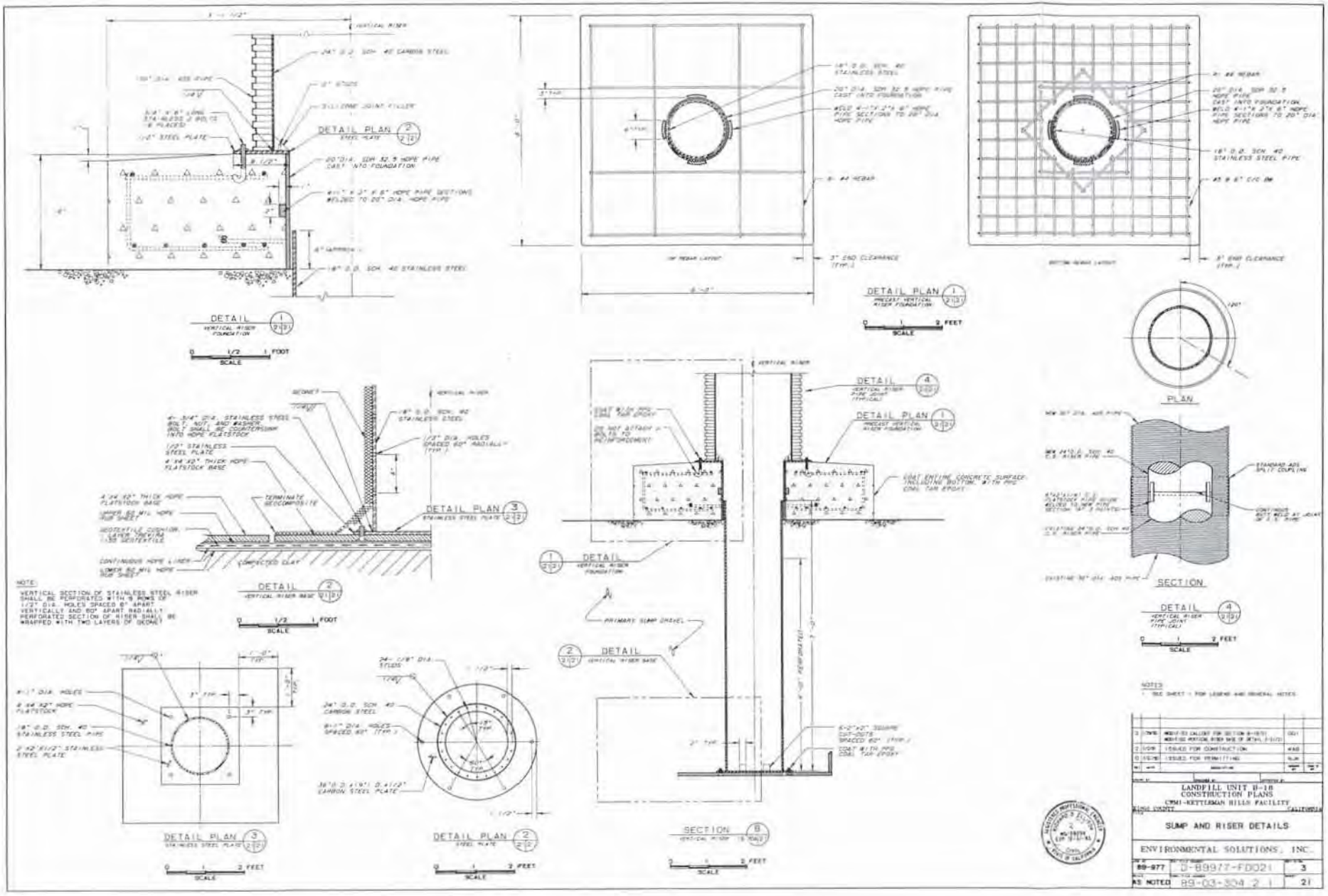
1	1/28/92	ADDED DETAIL PLAN 1-18A/20A AND SECTIONS A-1/8A/20A, A-20A/20A	
NO.	DATE	REVISION	BY

LANDFILL UNIT B-18
CONSTRUCTION PLANS
CWMI-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

PHASE II
SUMP AND RISER DETAILS

ENVIRONMENTAL SOLUTIONS, INC.

89-077	D-89977-SU020	1
AS SHOWN	89-03-304 2 1	20A


DETAIL PLAN
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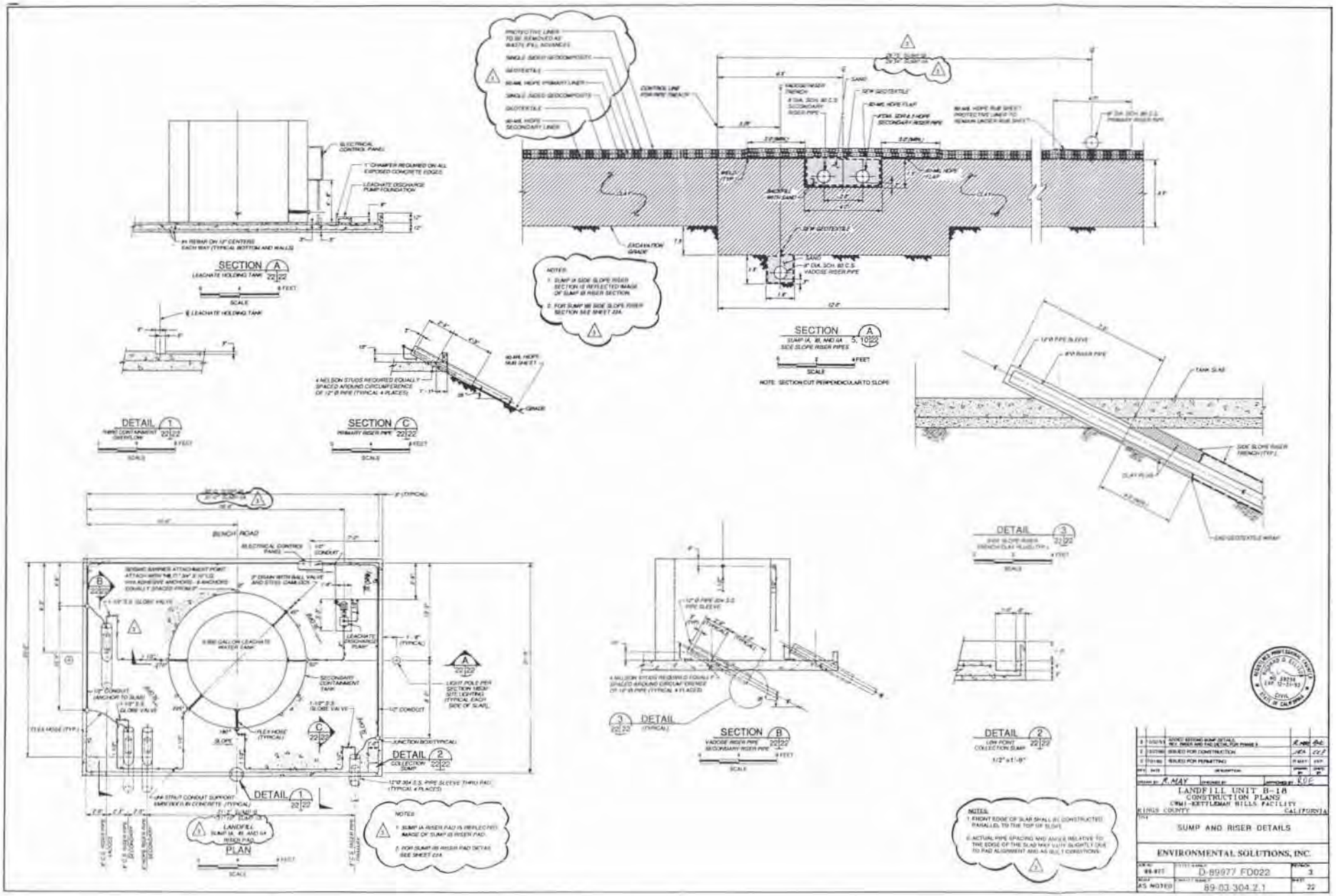
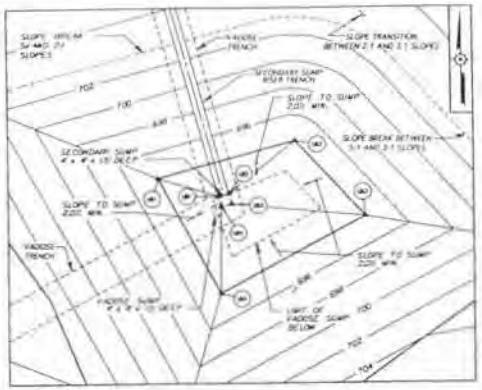
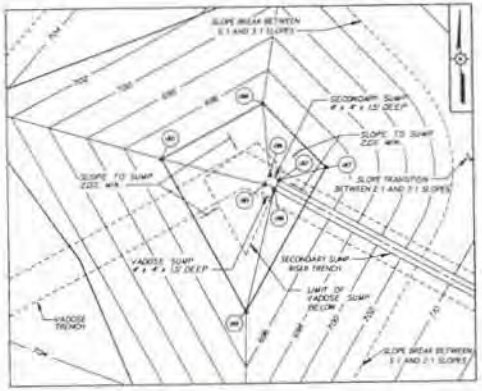
NOTES
LANDFILL UNIT B-18
CONSTRUCTION PLANS
CPM1-KETTLEMAN HILLS FACILITY
SUMP AND RISER DETAILS
ENVIRONMENTAL SOLUTIONS, INC.
D-89977-FD021
AS NOTED
89-03-304 2 1
21


SECTION A
LEACHATE HOLDING TANK 22/22
SECTION A
SUMP 1A, 1B, AND 1A SIDE SLOPE RISER PIPES 5, 10/22
NOTE: SECTION CUT PERPENDICULAR TO SLOPE
DETAIL 1
SECTION C
PRIMARY RISER PIPE 22/22
DETAIL 3
DETAIL 3
SECTION B
VADOSE RISER PIPE SECONDARY RISER PIPE 22/22
DETAIL 2
LOW POINT COLLECTION SUMP
1/2"=1'-0"
DETAIL 2
COLLECTION SUMP 22/22
DETAIL 1
LANDFILL SUMP 1A, 1B, AND 1A RISER PAD
PLAN
NOTES:
1. SUMP 1A SIDE SLOPE RISER SECTION IS REFLECTED IMAGE OF SUMP 1B RISER SECTION.
2. FOR SUMP 1B SIDE SLOPE RISER SECTION SEE SHEET 22A.
NOTES:
1. SUMP 1A RISER PAD IS REFLECTED IMAGE OF SUMP 1B RISER PAD.
2. FOR SUMP 1B RISER PAD DETAIL SEE SHEET 22A.
NOTES:
1. FRONT EDGE OF SLAB SHALL BE CONSTRUCTED PARALLEL TO THE TOP OF SLOPE.
2. ACTUAL PIPE SPACING AND ANGLE RELATIVE TO THE EDGE OF THE SLAB MAY VARY SLIGHTLY DUE TO PAD ALIGNMENT AND AS BUILT CONDITIONS.
LANDFILL UNIT B-1A
CONSTRUCTION PLANS
CRM-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA
SUMP AND RISER DETAILS
ENVIRONMENTAL SOLUTIONS, INC.
D-89977 FD022
89-03-304 Z.1
22



DETAIL PLAN (1/9|22A)
SECONDARY SUMP 1A

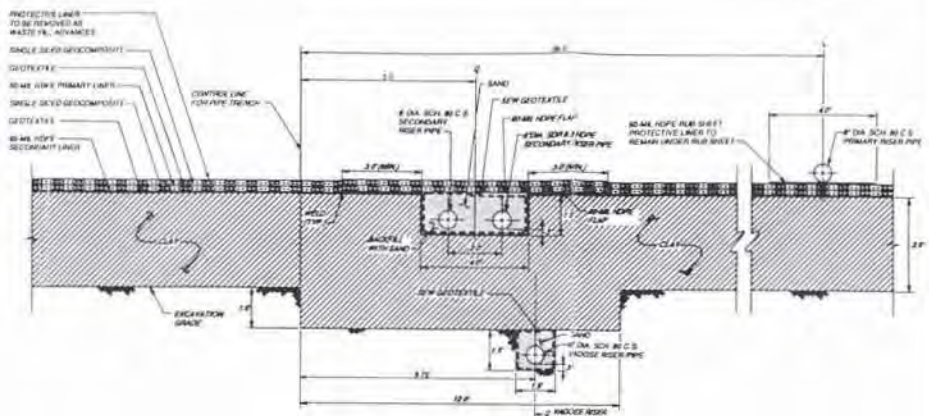
0 20 40 FEET
SCALE



DETAIL PLAN (2/9|22A)
SECONDARY SUMP 1B

0 20 40 FEET
SCALE

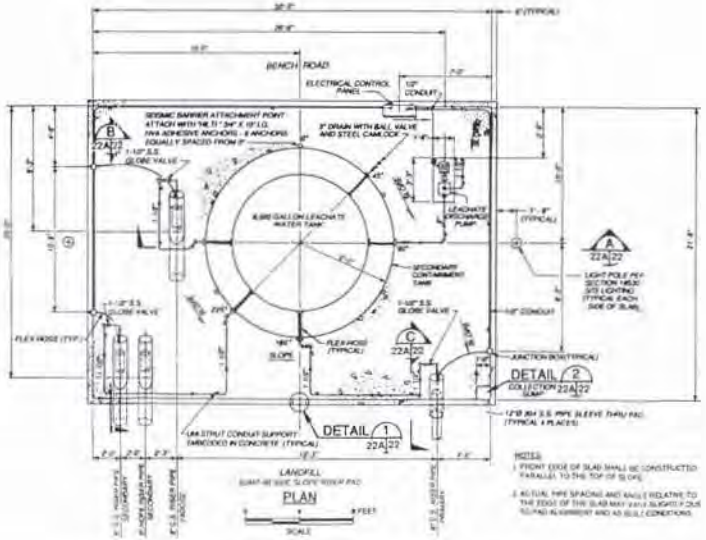
SURVEY CONTROL			
STATION	COORDINATE N	COORDINATE E	ELEVATION
U61	228,355.89	1,702,332.84	694.0
U62	228,176.31	1,702,383.76	694.0
U63	228,031.50	1,702,409.16	694.0
U64	228,652.89	1,702,359.91	694.0
U65	228,439.85	1,702,584.05	694.0
U66	228,460.27	1,702,621.84	694.0
U67	228,435.04	1,702,644.70	694.0
U68	228,382.91	1,702,614.70	694.0
U91	228,699.06	1,702,355.21	693.0
U92	228,701.10	1,702,358.07	693.0
U93	228,596.24	1,702,360.12	693.0
U94	228,595.20	1,702,356.26	693.0
U95	228,423.86	1,702,632.13	693.0
U96	228,423.14	1,702,634.33	693.0
U97	228,431.31	1,702,621.82	693.0
U98	228,423.88	1,702,625.65	693.0



SECTION (A/10|22A)
SUMP 1B SIDE SLOPE RISER PIPES

1 FEET
SCALE

NOTE: SECTION CUT PERPENDICULAR TO SLOPE



DETAIL 1 (22A/22)

DETAIL 2 (22A/22)

LANDFILL
SUMP 1B SIDE SLOPE RISER PIPE
PLAN

1 FEET
SCALE

NOTES:
1. FRONT EDGE OF SLAB SHALL BE CONSTRUCTED PARALLEL TO THE TOP OF SLOPE.
2. ACTUAL PIPE SPACING AND ANGLE RELATIVE TO THE EDGE OF THE SLAB MAY VARY SLIGHTLY DUE TO FIELD ALIGNMENT AND AS BUILT CONDITIONS.

REV	DATE	DESCRIPTION	DRN	CHK
1	10/03/94	ISSUED FOR CONSTRUCTION		
0	7/14/93	ISSUED FOR CLIENT REVIEW		

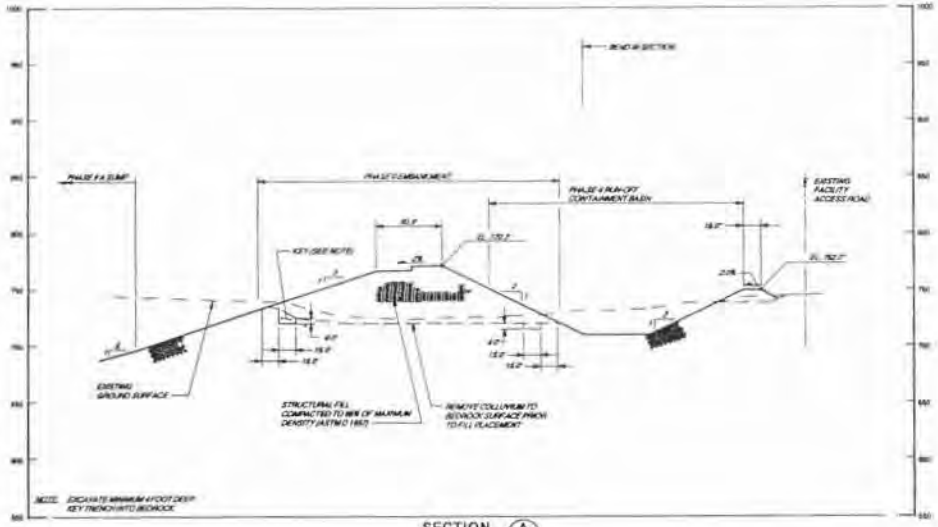
DRAWN R. MAY

CONSTRUCTION DRAWINGS
LANDFILL UNIT B-1B
CWM - KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

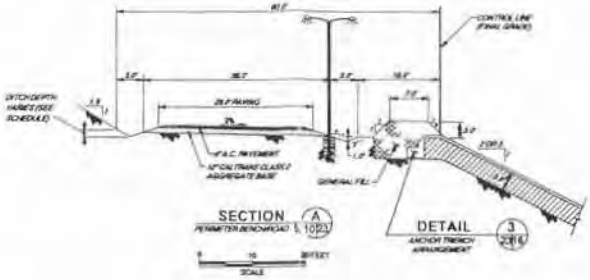
PHASE II SUMPS AND CELL 1B
SIDE SLOPE RISER DETAILS

ENVIRONMENTAL SOLUTIONS, INC.

89-877	D-89977-FD022A	1
AS NOTED	89-03-304.2.1	22A

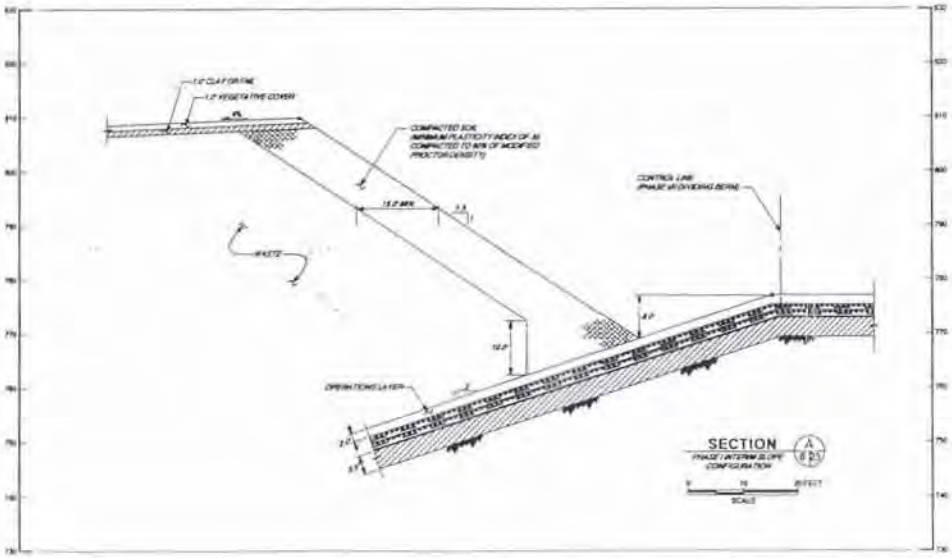


SECTION A (8/23)
PHASE II EMBANKMENT AND RETENTION BASIN
SCALE

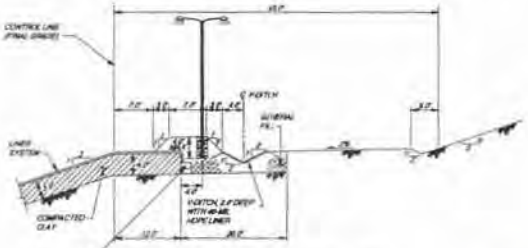


SECTION A (10/23)
PERIMETER BENCH ROAD
SCALE

DETAIL 3 (2/16)
ANCHOR TRENCH ARRANGEMENT



SECTION A (8/23)
PHASE I INTERIM SLOPE CONFIGURATION
SCALE



SECTION B (23/1)
TEMPORARY LINER TRENCH AT PHASE I & II DIVIDING BERM

SECTION B (5/23)
PHASE I & II DIVIDING BERM
SCALE

NOTES:

1. SEE SHEET 1 FOR LEGEND AND GENERAL NOTES.

2	12/8/94	ISSUED FOR PHASE II CONSTRUCTION	RAP	KJF
1	11/1/94	ISSUED FOR CONSTRUCTION	RAP	KJF
0	09/19/94	ISSUED FOR PERMITTING	CHD	KJF
REV	DATE	DESCRIPTION	DRN	CHK

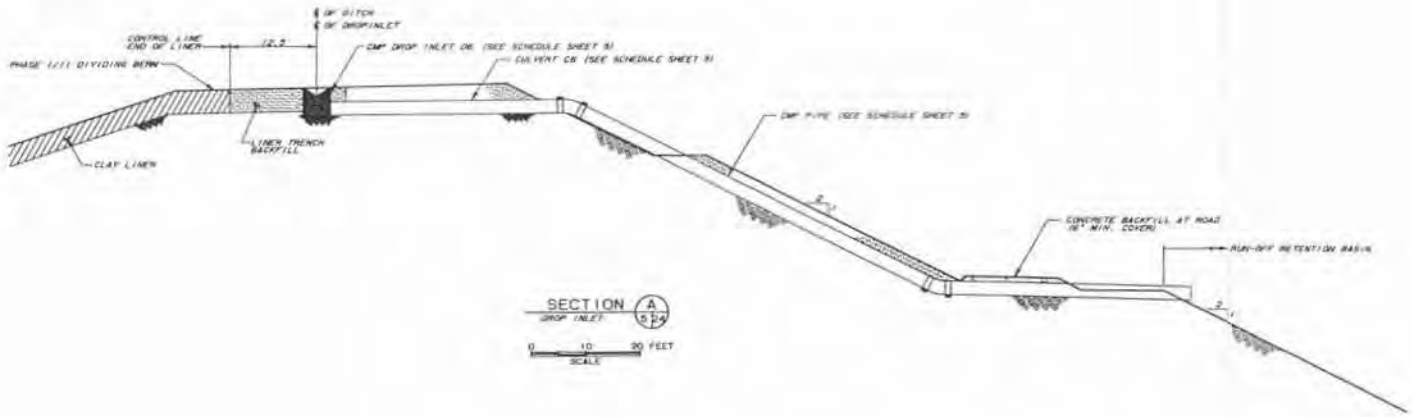
DESIGNED BY K.J. KNOLL DRAWN BY R. PARKER CHECKED BY KJK

CONSTRUCTION DRAWINGS
LANDFILL UNIT D-18
CWMI-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

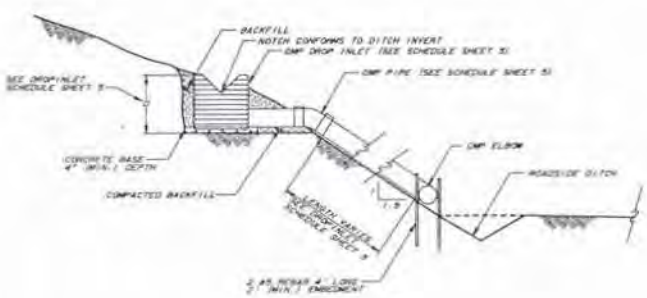
MISCELLANEOUS CROSS SECTIONS

ENVIRONMENTAL SOLUTIONS, INC.

JOB # 89-977	DETAIL TITLE NUMBER D-89977-FD023	REVISION 3
SCALE AS NOTED	DRAWING NUMBER 89-03-304.2.1	SHEET 23



SECTION A 5.24
DROP INLET
0 10 20 FEET
SCALE



DETAIL 1 5.24
DROP INLET AT CUT SLOPE (TYP.)
0 4 8 FEET
SCALE

REGISTERED PROFESSIONAL ENGINEER
GARY E. FAHRENBACH
No. 41021
EXP. MAR 31 90
CIVIL
STATE OF CALIFORNIA

2	1/24/90	ISSUED FOR CONSTRUCTION	KAB	EEP
1	8/24/89	ISSUED FOR BID	KAB	EEP
NO.	DATE	DESCRIPTION	DRAWN BY	CHK'D BY

DRAWN BY K. Blanco CHECKED BY E. Fahrenbach APPROVED BY R.D.E.

LANDFILL UNIT B-1B
CONSTRUCTION PLANS
CWMI-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

DRAINAGE DETAILS

ENVIRONMENTAL SOLUTIONS, INC.

JOB NO.	DWG FILE NAME	SHEET NO.
89-077	D-89977-F0024	2
SCALE AS NOTED	DWG FILE NAME 89-03-304.2.1	SHEET 24

SURVEY CONTROL FOR PHASE I EXCAVATION AND BACKFILL PLAN (SHEET 3)

POINTS OF INTERSECTION

POINT	DESCRIPTION	NORTHING	EASTING	ELEV
P1	N.W. CORNER TOP PHASE I EXCAVATION	[illegible]	[illegible]	[illegible]
P2	WEST SIDE TOP PHASE I EXCAVATION	[illegible]	[illegible]	[illegible]
P3	S.W. CORNER TOP PHASE I EXCAVATION	[illegible]	[illegible]	[illegible]
P4	S.E. CORNER TOP PHASE I EXCAVATION	[illegible]	[illegible]	[illegible]
P5	N.E. CORNER TOP PHASE I EXCAVATION	[illegible]	[illegible]	[illegible]
P6	N.W. TOE AREA IA	[illegible]	[illegible]	[illegible]
P7	N.W. TOP AREA IA SUMP	[illegible]	[illegible]	[illegible]
P8	N.E. TOP AREA IA SUMP	[illegible]	[illegible]	[illegible]
P9	S.E. TOP AREA IA SUMP	[illegible]	[illegible]	[illegible]
P10	S.E. TOE AREA	[illegible]	[illegible]	[illegible]
P11	N.E. TOE AREA IA	[illegible]	[illegible]	[illegible]
P12	S.W. TOP AREA IB SUMP	[illegible]	[illegible]	[illegible]
P13	WEST TOE AREA IB	[illegible]	[illegible]	[illegible]
P14	WEST TOE AREA IB	[illegible]	[illegible]	[illegible]
P15	S.W. TOE AREA IB	[illegible]	[illegible]	[illegible]
P16	S.E. TOE AREA IB	[illegible]	[illegible]	[illegible]
P17	EAST TOE AREA IB AT RAMP	[illegible]	[illegible]	[illegible]
P18	EAST TOE AREA IB AT RAMP	[illegible]	[illegible]	[illegible]
P19	N.E. TOE AREA IB	[illegible]	[illegible]	[illegible]
P20	N.E. TOP AREA IB SUMP	[illegible]	[illegible]	[illegible]
P21	N.E. TOP AREA IB SUMP	[illegible]	[illegible]	[illegible]
[illegible]	OUTSIDE EDGE BENCH/ROAD	[illegible]	[illegible]	[illegible]
[illegible]	OUTSIDE EDGE BENCH/ROAD	[illegible]	[illegible]	[illegible]
[illegible]	OUTSIDE EDGE BENCH/ROAD	[illegible]	[illegible]	[illegible]
[illegible]	WEST END ACCESS RAMP	[illegible]	[illegible]	[illegible]
[illegible]	S.W. CORNER ACCESS RAMP	[illegible]	[illegible]	[illegible]
[illegible]	S.E. CORNER ACCESS RAMP	[illegible]	[illegible]	[illegible]
[illegible]	EAST SIDE ACCESS RAMP	[illegible]	[illegible]	[illegible]
[illegible]	EAST SIDE ACCESS RAMP	[illegible]	[illegible]	[illegible]

POINT	DESCRIPTION	NORTHING	EASTING	ELEV
A2	℄ NORTH ACCESS ROAD	[illegible]	[illegible]	[illegible]
A3	℄ NORTH ACCESS ROAD AND CLAY PROCESS ACCESS ROAD	[illegible]	[illegible]	[illegible]
A4	℄ NORTH ACCESS ROAD AND CLAY PROCESS ACCESS ROAD	[illegible]	[illegible]	[illegible]
A6	℄ END NORTH ACCESS ROAD	[illegible]	[illegible]	[illegible]
A8	℄ SOUTH ACCESS ROAD	[illegible]	[illegible]	[illegible]
A10	℄ SOUTH ACCESS ROAD	[illegible]	[illegible]	[illegible]
A13	℄ WELL ACCESS ROAD	[illegible]	[illegible]	[illegible]
B2	L.P.F.L.(1) PHASE IA BENCH	[illegible]	[illegible]	[illegible]
C1	N.W. TOE CLAY PIT	[illegible]	[illegible]	[illegible]
C2	S.W. TOE CLAY PIT	[illegible]	[illegible]	[illegible]
C3	S.E. TOE CLAY PIT	[illegible]	[illegible]	[illegible]
C4	N.E. TOE CLAY PIT	[illegible]	[illegible]	[illegible]
C6	℄ CLAY PIT ROAD	[illegible]	[illegible]	[illegible]
C7	℄ CLAY PIT ROAD	[illegible]	[illegible]	[illegible]
C9	N.W. CORNER BENCH	[illegible]	[illegible]	[illegible]
C10	N.E. CORNER BENCH	[illegible]	[illegible]	[illegible]
C11	S.E. CORNER BENCH	[illegible]	[illegible]	[illegible]
C12	S.W. CORNER BENCH	[illegible]	[illegible]	[illegible]
C14	INSIDE EDGE N.E. CORNER 780 BENCH	[illegible]	[illegible]	[illegible]
CP4	S.E. CORNER CLAY PROCESS AREA	[illegible]	[illegible]	[illegible]
CP5	WEST SIDE CLAY PROCESS AREA	[illegible]	[illegible]	[illegible]
CP6	WEST SIDE CLAY PROCESS AREA	[illegible]	[illegible]	[illegible]
CP7	WEST SIDE CLAY PROCESS AREA	[illegible]	[illegible]	[illegible]
CP8	WEST SIDE CLAY PROCESS AREA	[illegible]	[illegible]	[illegible]
CP9	WEST SIDE CLAY PROCESS AREA	[illegible]	[illegible]	[illegible]
CP10	NORTH SIDE CLAY PROCESS AREA	[illegible]	[illegible]	[illegible]

CONTROL POINTS

POINT	DESCRIPTION	NORTHING	EASTING	ELEV
A1	℄ NORTH ACCESS ROAD	[illegible]	[illegible]	[illegible]
A6	℄ END CLAY PROCESS AREA ROAD	[illegible]	[illegible]	[illegible]
A7	℄ END CLAY PROCESS AREA ROAD	[illegible]	[illegible]	[illegible]
A9	℄ END SOUTH ACCESS ROAD	[illegible]	[illegible]	[illegible]
A11	℄ END WELL ACCESS ROAD	[illegible]	[illegible]	**
A13	℄ END WELL ACCESS ROAD	[illegible]	[illegible]	**
*B1	NORTH END PHASE IA BENCH	[illegible]	[illegible]	[illegible]
B3	SOUTH END PHASE IA BENCH	[illegible]	[illegible]	[illegible]
*B4	WEST END SUMP DIVIDING BERM	[illegible]	[illegible]	[illegible]
B5	EAST END SUMP DIVIDING BERM	[illegible]	[illegible]	[illegible]
C5	℄ CLAY PIT ROAD	[illegible]	[illegible]	[illegible]
C8	℄ CLAY PIT ROAD AT BENCH	[illegible]	[illegible]	[illegible]
C13	INSIDE EDGE END 780 BENCH	[illegible]	[illegible]	[illegible]
*C15	INSIDE EDGE END 780 BENCH	[illegible]	[illegible]	[illegible]
CP1	CONTROL POINT CLAY PROCESS GRADING	[illegible]	[illegible]	[illegible]
CP2	CLAY PROCESS ACCESS ROAD	[illegible]	[illegible]	[illegible]
CP3	EAST SIDE CLAY PROCESS AREA	[illegible]	[illegible]	[illegible]
CP11	NORTH SIDE CLAY PROCESS AREA	[illegible]	[illegible]	[illegible]
S1	OUTSIDE EDGE BENCH/ROAD	[illegible]	[illegible]	[illegible]
S2	OUTSIDE EDGE BENCH/ROAD	[illegible]	[illegible]	[illegible]
T1	END RAMP LINER TRENCH	[illegible]	[illegible]	[illegible]
T2	END RAMP LINER TRENCH	[illegible]	[illegible]	[illegible]
T3	END RAMP LINER TRENCH	[illegible]	[illegible]	[illegible]
T4	END RAMP LINER TRENCH	[illegible]	[illegible]	[illegible]
*T5	SLOPE BREAK IN LINER TRENCH	[illegible]	[illegible]	[illegible]
*T6	END TOP LINER TRENCH	[illegible]	[illegible]	[illegible]
*T7	END TOP LINER TRENCH	[illegible]	[illegible]	[illegible]
V1	START AREA IA VADOSE TRENCH	[illegible]	[illegible]	[illegible]
V2	START AREA IA VADOSE TRENCH	[illegible]	[illegible]	[illegible]

CURVE DATA

CURVE NO.	DESCRIPTION	LOCATION OF P.I. NORTHING	LOCATION OF P.I. EASTING	D	R	L	T
1	N.W. CORNER TOP PHASE I EXCAVATION	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
2	WEST SIDE TOP PHASE I EXCAVATION	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
3	S.W. CORNER TOP PHASE I EXCAVATION	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
4	OUTSIDE EDGE BENCH/ROAD	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
5	OUTSIDE EDGE BENCH/ROAD	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
6	OUTSIDE EDGE BENCH/ROAD	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
7	OUTSIDE EDGE BENCH/ROAD	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
8	OUTSIDE EDGE BENCH/ROAD	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
9	N.W. TOE AREA IA	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	N.E. TOE AREA IA	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	S.W. TOP DIVIDING BERM AREA IA	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	S.E. TOE AREA IA	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	N.W. TOP DIVIDING BERM AREA IB	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	N.E. TOE AREA IB	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	S.W. TOE AREA IB	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	S.E. TOE AREA IB	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	INSIDE EDGE ACCESS RAMP	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	OUTSIDE EDGE ACCESS RAMP	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	N.W. TOE CLAY PIT	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	N.E. TOE CLAY PIT	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	S.W. TOE CLAY PIT	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	S.E. TOE CLAY PIT	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	℄ CLAY PIT ROAD	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	℄ CLAY PIT ROAD	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	℄ N.E. CORNER 780 BENCH	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	℄ NORTH ACCESS ROAD	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	℄ NORTH ACCESS ROAD	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	EDGE ACCESS ROAD	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	EDGE ACCESS ROAD	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	EDGE ACCESS ROAD	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	EDGE ACCESS ROAD	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	EDGE ACCESS ROAD	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	S.E. CORNER PHASE IA BENCH	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	℄ ACCESS ROAD	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	℄ ACCESS ROAD	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	℄ ACCESS ROAD	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	℄ ACCESS ROAD	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	EAST SIDE CLAY PROCESS AREA	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	NORTH SIDE CLAY PROCESS AREA	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	WEST SIDE CLAY PROCESS AREA	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	SOUTH SIDE CLAY PROCESS AREA	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

(1) L.P.F.L. - LOW POINT OF FLOW LINE

* LOCATION GIVEN IN TABLE SUPERSEDES THE LOCATION ON THE DRAWING

** MATCH GRADE AT PERIMETER BENCH/ROAD

2	[illegible]	ISSUED FOR CONSTRUCTION	[illegible]	[illegible]
1	[illegible]	ISSUED FOR BID	[illegible]	[illegible]
0	[illegible]	ISSUED FOR PERMITTING	[illegible]	[illegible]

LANDFILL UNIT B-18
CONSTRUCTION PLANS
CWMI-KETTLEMAN HILLS FACILITY
KINGS COUNTY CALIFORNIA

SURVEY CONTROL

ENVIRONMENTAL SOLUTIONS, INC.

89-977 | D-89977-FD025 | 2
AS NOTED | 89-03-304.1.2 | 25

APPENDIX A.2
PHASE III AND FINAL CLOSURE DRAWINGS

CONSTRUCTION DRAWINGS FOR THE B-18 CLASS I LANDFILL PHASE III EXPANSION AND FINAL CLOSURE

KETTLEMAN HILLS FACILITY
KINGS COUNTY, CALIFORNIA
NOVEMBER 2008

PREPARED FOR:
CHEMICAL WASTE MANAGEMENT

SHEET INDEX

DRAWING NO.	TITLE	SHEET NO.
083-91887EXP-T1	TITLE SHEET	T-1
083-91887EXP-C1	SITE PLAN	C-1
083-91887EXP-C2	EXISTING CONDITIONS (AS OF MARCH 28, 2008)	C-2
083-91887EXP-C3	BASE LINER PLAN	C-3
083-91887EXP-C4A	PHASE IIIA FILL PLAN	C-4A
083-91887EXP-C4	FINAL CLOSURE PLAN	C-4
083-91887EXP-C5	CROSS SECTIONS A TO D	C-5
083-91887EXP-C6	CROSS SECTIONS E TO I	C-6
083-91887EXP-C7	PHASE III BASE LINER CONSTRUCITON DETAILS	C-7
083-91887EXP-C8	PHASE III LCRS SYSTEM DETAILS	C-8
083-91887EXP-C9	DRAINAGE DETAILS	C-9
083-91887EXP-C10	CLOSURE DETAILS	C-10

GENERAL NOTES

1. THESE DRAWINGS AND SPECIFICATIONS SHALL BE USED TOGETHER. IF A DIFFERENCE OCCURS, THE CWM PROJECT MANAGER SHALL DETERMINE WHICH TAKES PRECEDENCE.
2. THE LANDFILL B-18 CONSTRUCTION WILL OCCUR WITHIN AREAS APPROVED BY VARIOUS GOVERNMENT AGENCIES. NO WORK SHALL OCCUR OUTSIDE OF THE CONSTRUCTION AREAS SHOWN ON THE DRAWINGS WITHOUT SPECIFIC APPROVAL OF THE CWM PROJECT MANAGER.
3. PRIOR TO COMMENCING ANY CUT OR FILL, OR IN ANY WAY DISTURBING THE EXISTING LAND CONTOURS, CONTRACTOR SHALL VERIFY THE ACCURACY OF THE CONTOUR ELEVATIONS SHOWN IN THE DRAWINGS.

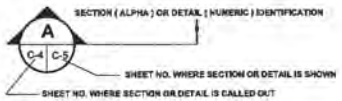
LEGEND

- STRUCTURAL FILL
- GENERAL FILL
- COMPACTED CLAY LINER
- RIP-RAP
- CONCRETE
- OPERATIONS LAYER
- SUBGRADE
- HDPE GEOMEMBRANE
- GEOTEXTILE
- GEOCOMPOSITE
- PROPERTY LINE
- 10' CONTOUR
- 2' CONTOUR
- FLOWLINE
- LIMIT OF GRADING

ABBREVIATIONS

A.C.	ASPHALT CONCRETE
C	CENTER LINE
CMP	CORRUGATED METAL PIPE
CWM	CHEMICAL WASTE MANAGEMENT
EL	ELEVATION
HDPE	HIGH DENSITY POLYETHYLENE
H.P.	HIGH POINT
MIN.	MINIMUM
TYP.	TYPICAL
Ø	DIAMETER

KEY



PLANS PREPARED UNDER THE SUPERVISION OF:

REGISTERED PROFESSIONAL ENGINEER — STATE OF CALIFORNIA

8-25-11

RE-ISSUED FOR PERMITTING

REV	DATE	DESCRIPTION	DESIGN	DRAWN	REVIEW
A	9/11/2008	ISSUED FOR SUBMITTING MEETING	K.R.	K.R.	EGD
B	11/24/2008	ISSUED FOR PERMITTING	K.R.	K.R.	EGD
C	10/20/11	RE-ISSUED FOR PERMITTING	K.R.	K.R.	RH

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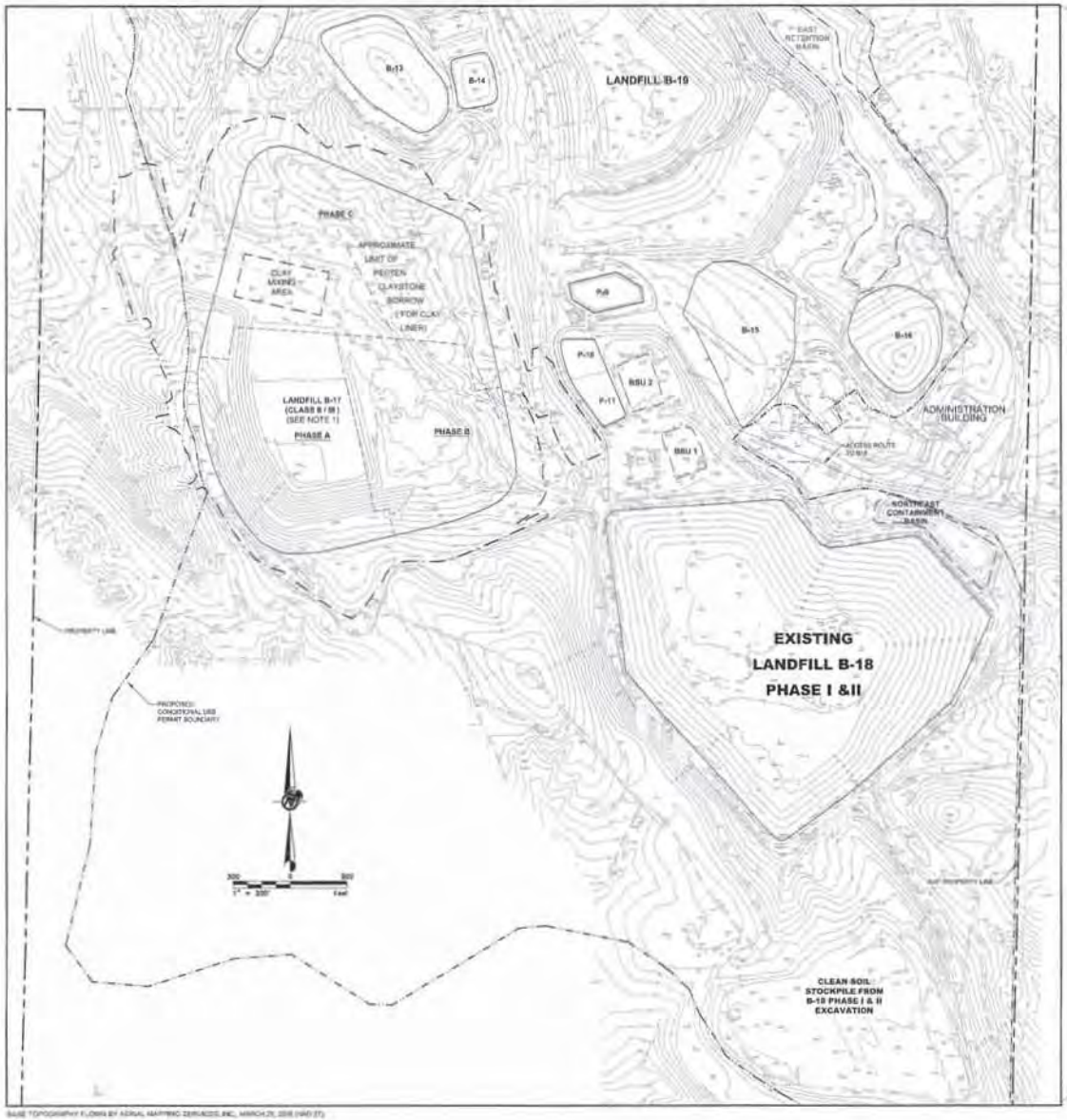
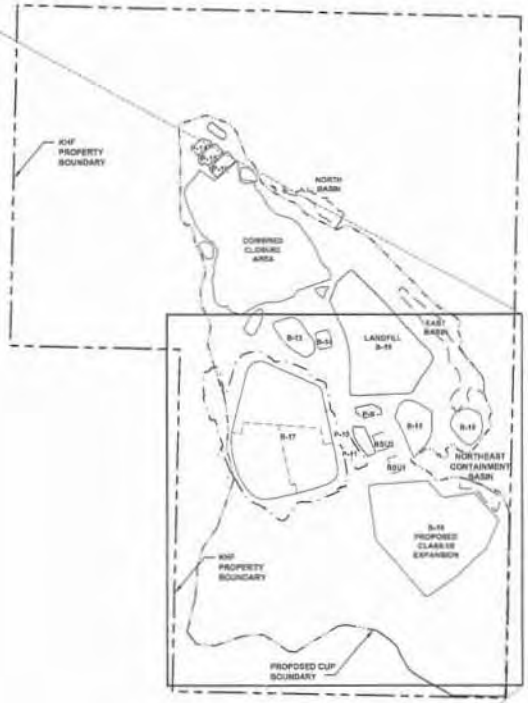
B-18 CLASS I LANDFILL PHASE III EXPANSION AND FINAL CLOSURE

TITLE SHEET

SHEET NUMBER
T-1
1 OF 11 SHEETS



BASE TOPOGRAPHY FLOWN BY AERIAL MAPPING SERVICES, INC., MARCH 25, 2008 (NAD 27)



NOTE
1. SOIL EXCAVATED FROM WITHIN LANDFILL B-17 MAY BE UTILIZED FOR STRUCTURAL FILL, CLAY LINER, OPERATIONS LAYER, DAILY COVER AND/OR FINAL COVER.

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STATE OF CALIFORNIA
CIVIL

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REV	DATE	DESCRIPTION	DESIGN	DRAWN	REVIEW
A	9/15/2008	ISSUED FOR PERMITTING MEETING	K.R.	K.R.	SCD
B	11/14/2008	ISSUED FOR PERMITTING	K.R.	K.R.	SCD

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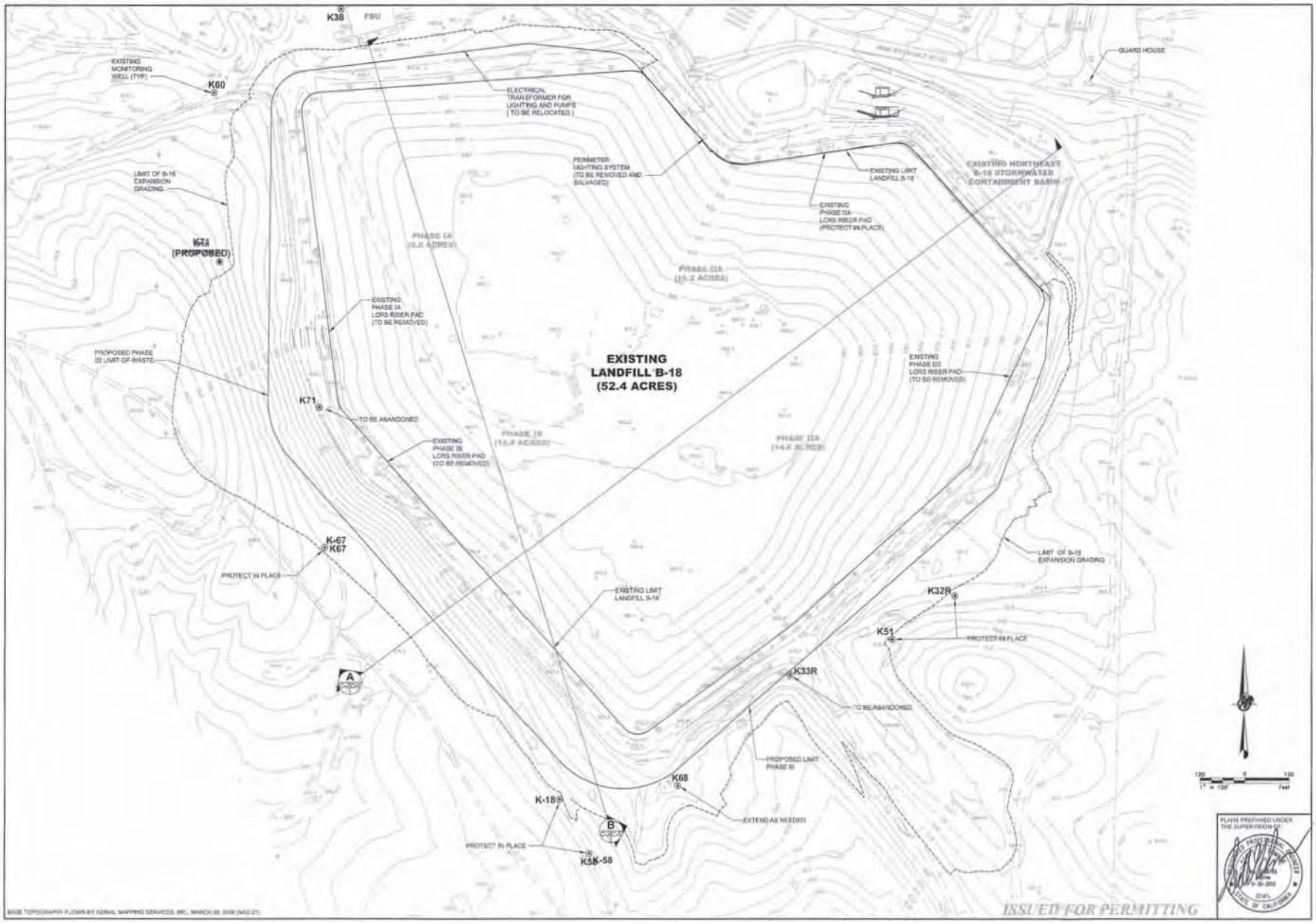
B-18 CLASS I LANDFILL
PHASE III EXPANSION AND
FINAL CLOSURE

SITE PLAN

SHEET NUMBER:

C-1

2 OF 11 SHEETS



REV	DATE	DESCRIPTION	DRAWN	REVIEW
A	8/4/2008	ISSUED FOR MEETING	KJK	GGS
B	9/15/2008	ISSUED FOR FY2007/08 SUBLINER	KJK	GGS
C	11/24/2008	ISSUED FOR PERMITTING	KJK	GGS

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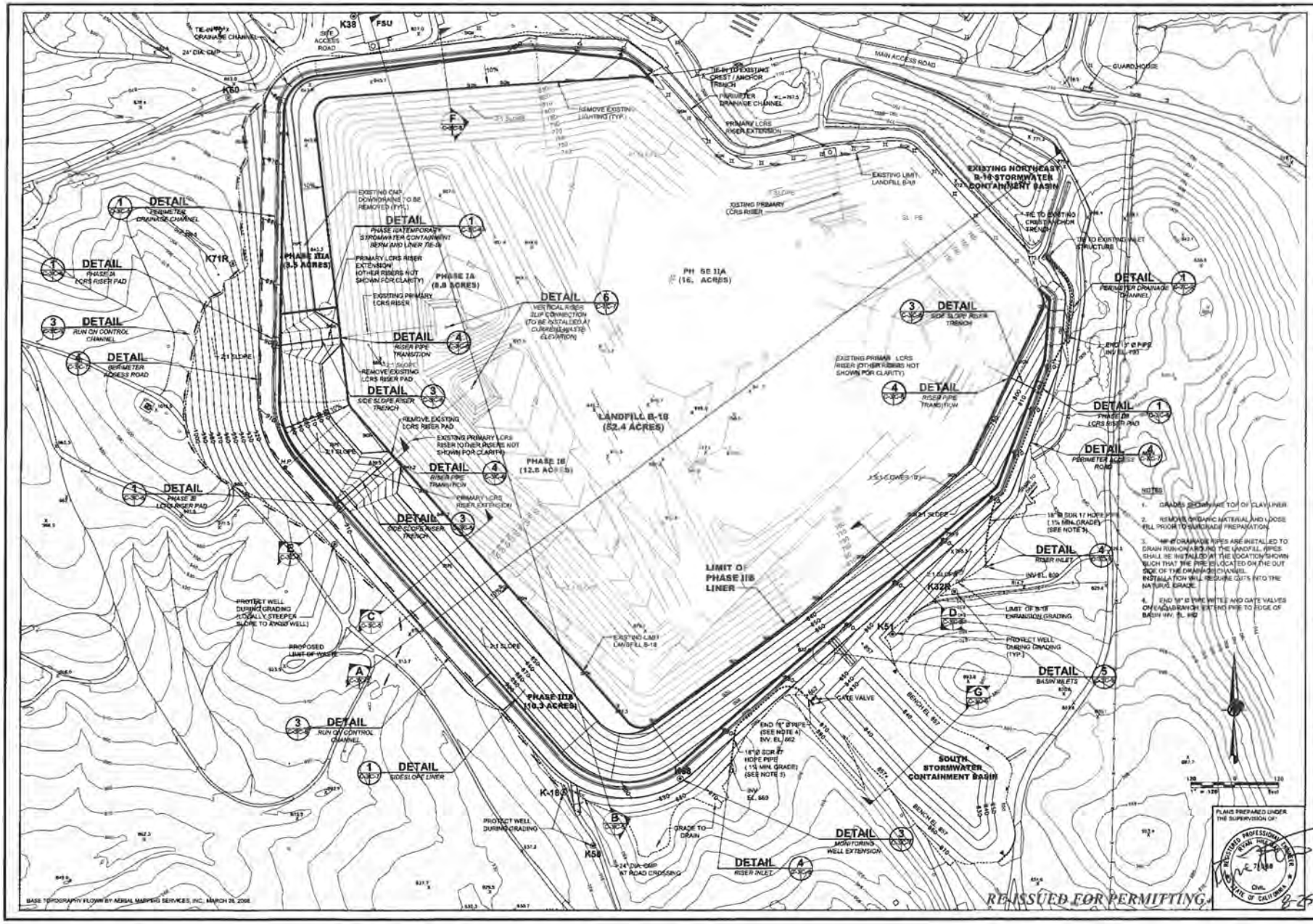
B-18 CLASS I LANDFILL PHASE III EXPANSION AND FINAL COVER
EXISTING CONDITIONS (AS OF MARCH 28, 2008)

SHEET NUMBER
C-2
3 OF 11 SHEETS



NOTES:

1. GRADES SHOWN ARE TOP OF CLAY LINER.
2. REMOVE ORGANIC MATERIAL AND LOOSE FILL PRIOR TO SUBGRADE PREPARATION.
3. 18" Ø DRAINAGE PIPES ARE INSTALLED TO DRAIN RUN-ON AROUND THE LANDFILL. PIPES SHALL BE INSTALLED AT THE LOCATION SHOWN SUCH THAT THE PIPE IS LOCATED ON THE OUT SIDE OF THE DRAINAGE CHANNEL. INSTALLATION WILL REQUIRE CUTS INTO THE NATURAL GRADE.
4. END 18" Ø PIPE WITH 2" AND GATE VALVES ON EACH BRANCH EXTEND PIPE TO EDGE OF BASIN INV. EL. 862

REV	DATE	DESCRIPTION	DESIGN	DRAWN	REVIEW
A	9/16/2008	ISSUED FOR PERMITTING WITH COMMENTS	K.J.B.	K.J.B.	J.D.D.
B	11/04/2008	ISSUED FOR PERMITTING	K.J.B.	K.J.B.	D.G.E.
C	8/26/2011	RE-ISSUED FOR PERMITTING	K.J.B.	K.J.B.	JRH

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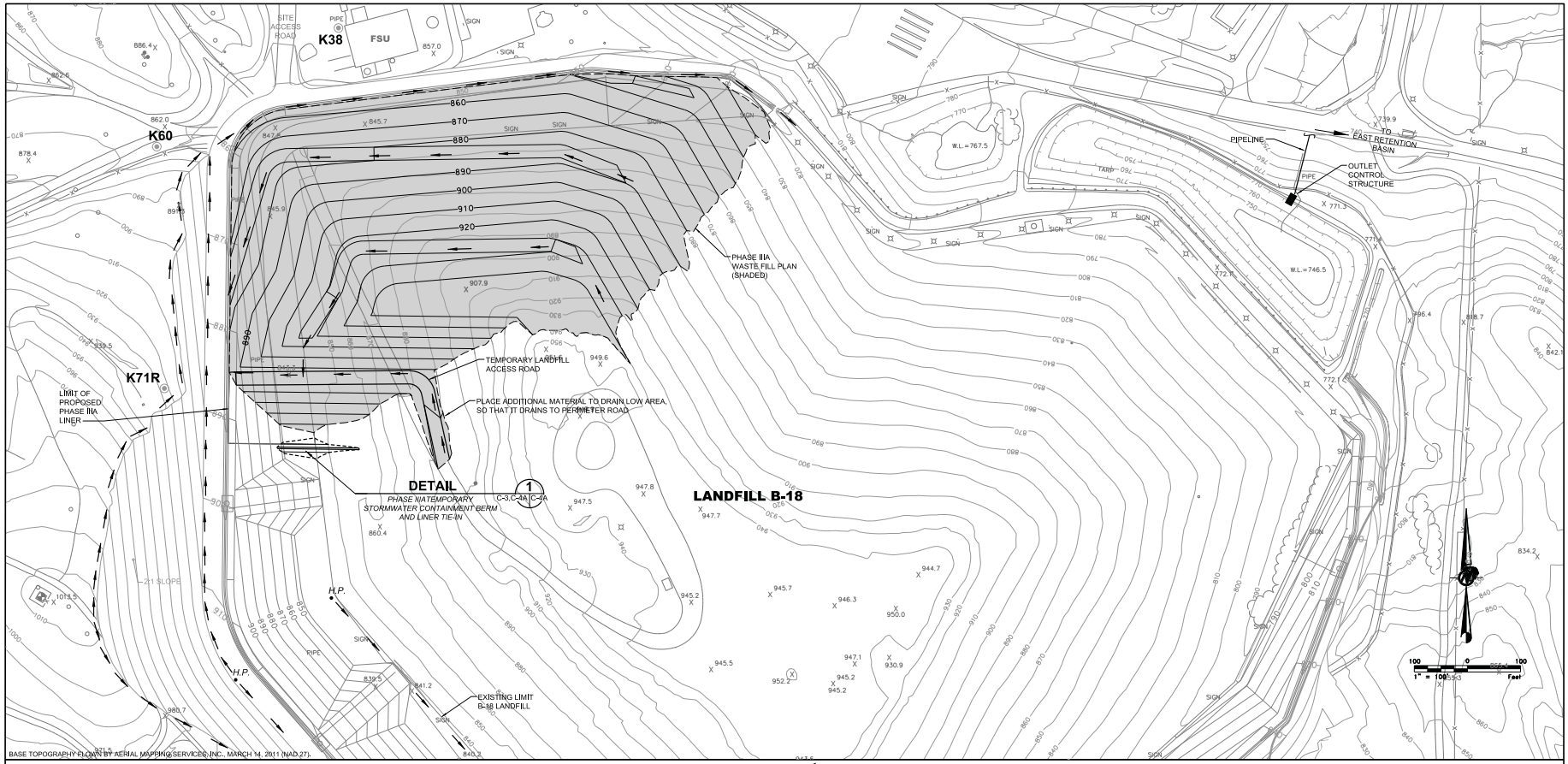
B-18 CLASS I LANDFILL
PHASE III EXPANSION AND FINAL CLOSURE

BASE LINER PLAN

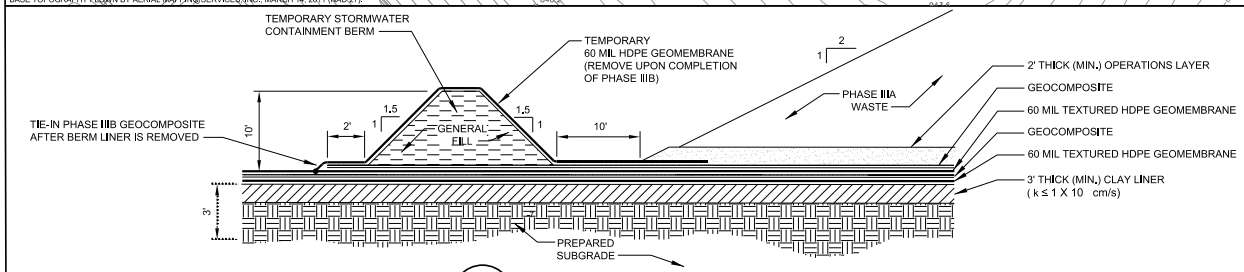
SHEET NUMBER

C-3

4 OF 11 SHEETS



BASE TOPOGRAPHY FLOWN BY AERIAL MAPPING SERVICES, INC., MARCH 14, 2011 (NAD 27).



DETAIL 1 C-3, C-4A C-4A

PHASE IIIATEMPORARY STORMWATER CONTAINMENT BERM AND LINER TIE-IN

NOT TO SCALE

NOTES:

1. GRADES SHOWN FOR PHASE IIIA (EXCEPT TEMPORARY WASTE SLOPE)ARE TO TOP OF FINAL COVER.
2. PHASE IIIB GRADING SHOWN AS TOP OF PROPOSED LINER.
3. SOIL BERMS ON DRAINAGE BENCH, COVER ACCESS ROAD AND SOUTH PERIMETER ROAD NOT SHOWN FOR CLARITY.

PLANS PREPARED UNDER THE SUPERVISION OF:

REGISTERED PROFESSIONAL ENGINEER — KEVIN KIRK — C 71648 — CIVIL — STATE OF CALIFORNIA

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REV	DATE	DESCRIPTION	DESIGN	DRAWN	REVIEW
C	8/10/2011	RE-ISSUED FOR PERMITTING	KJK	KJK	RH

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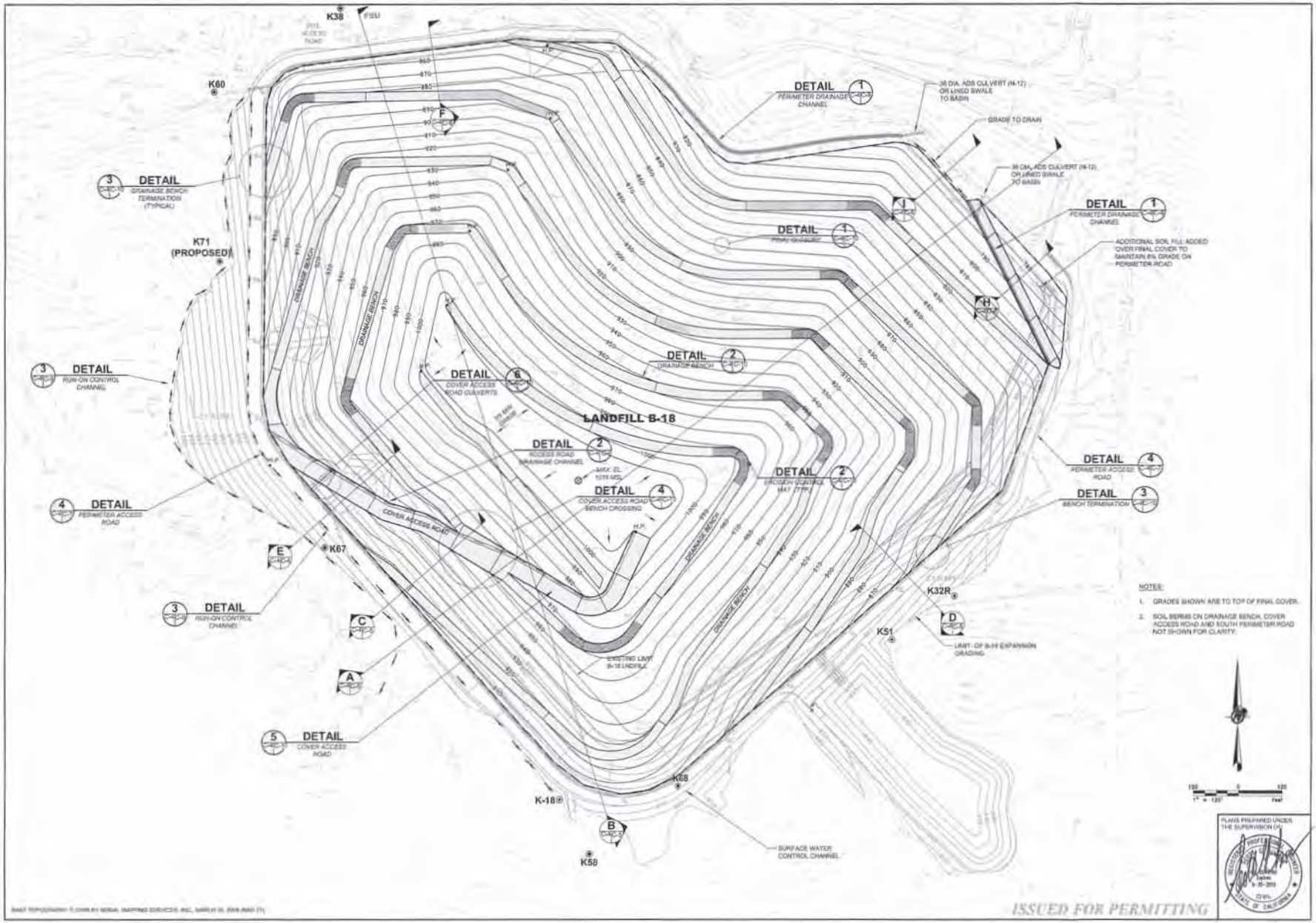
B-18 CLASS I LANDFILL
PHASE III EXPANSION AND FINAL CLOSURE

PHASE IIIA FILL PLAN

SHEET NUMBER

C-4A

5 OF 12 SHEETS



REV	DATE	DESCRIPTION	DESIGN	DRAWN	REVIEW
A	9/16/2008	ISSUED FOR PERMITTING MEETING	A.R.	K.R.	SGS
B	11/14/2008	ISSUED FOR PERMITTING	S.R.	K.R.	SGS

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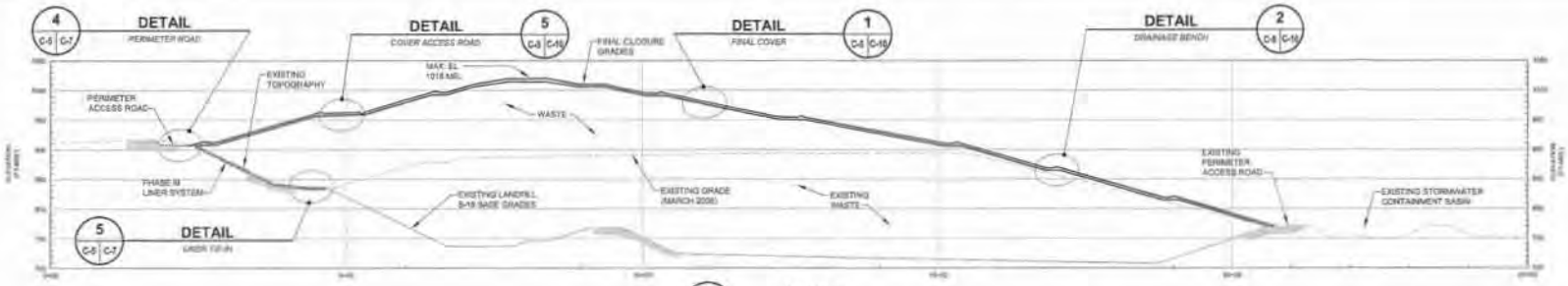
B-18 CLASS I LANDFILL
PHASE III EPANSION AND FINAL CLOSURE

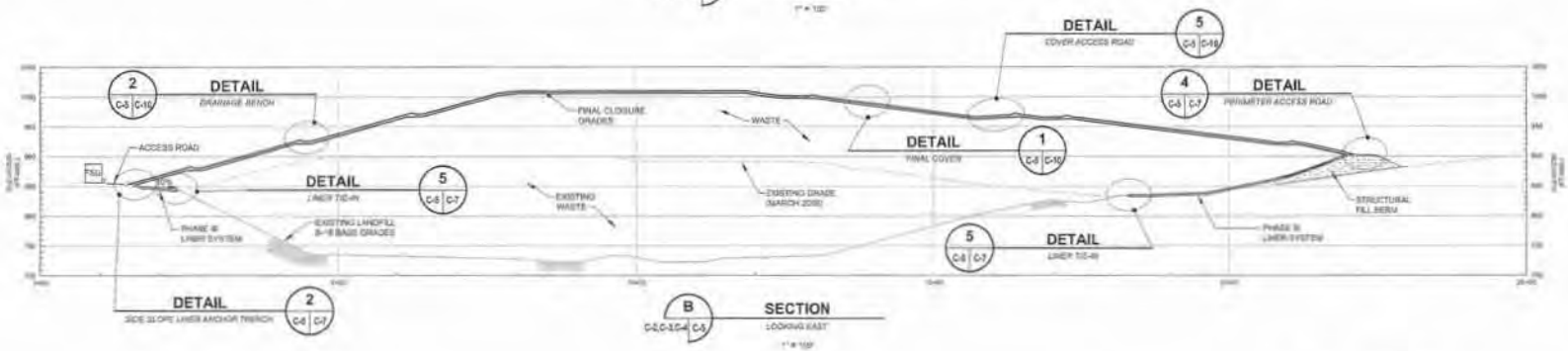
FINAL CLOSURE PLAN

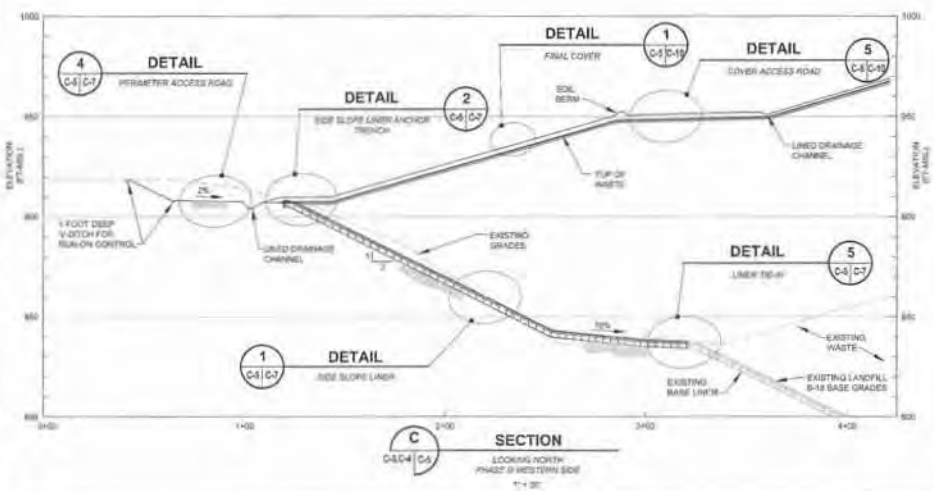
SHEET NUMBER

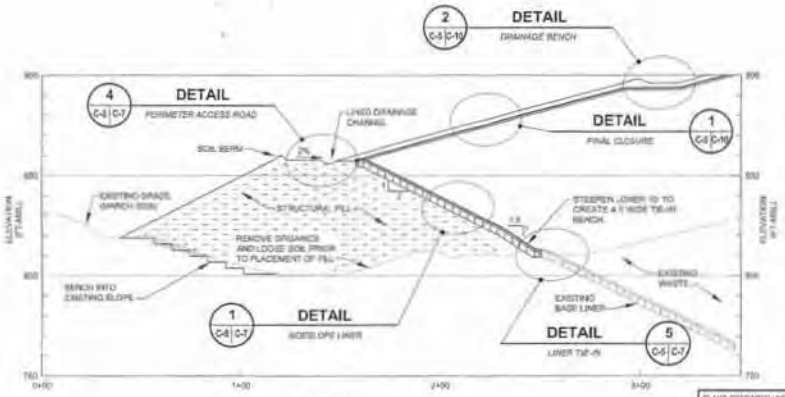
C-4

3 OF 11 SHEETS









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PLANS PREPARED UNDER THE SUPERVISION OF:

REV	DATE	DESCRIPTION	DESIGN	DRAWN	REVIEW
A	9/10/2008	ISSUED FOR PERMITTING MEETING	K.R.	K.R.	KAS
B	11/14/2008	ISSUED FOR PERMITTING	K.R.	L.R.	SGC

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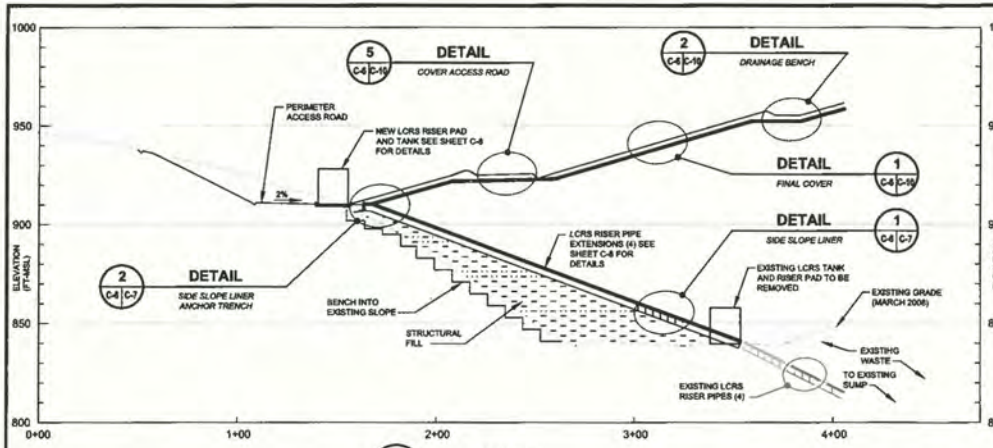
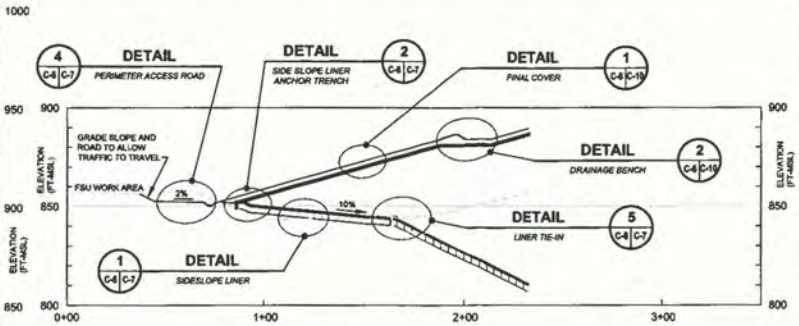
B-18 CLASS I LANDFILL
PHASE III EXPANSION AND FINAL CLOSURE
CROSS SECTIONS A TO D

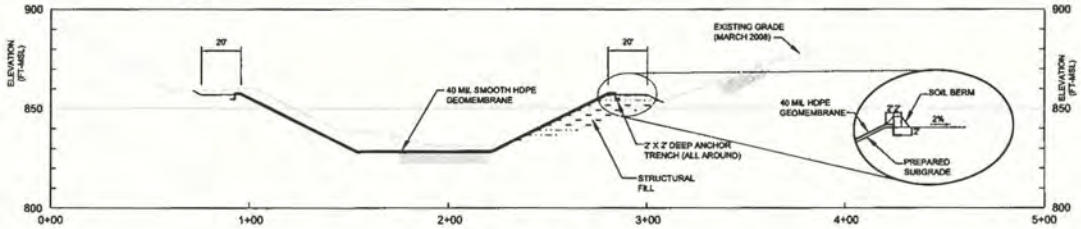
SHEET NUMBER
C-5
8 OF 11 SHEETS



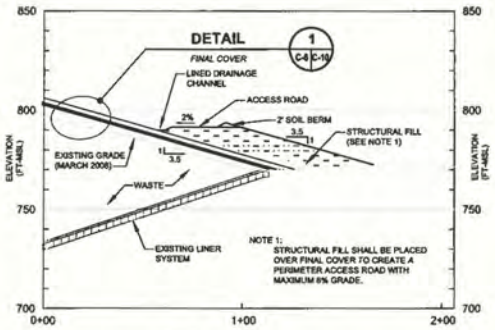
E C-3 C-4 C-6 SECTION
LOOKING NORTH
LCRS RISER PIPES
1" = 30'



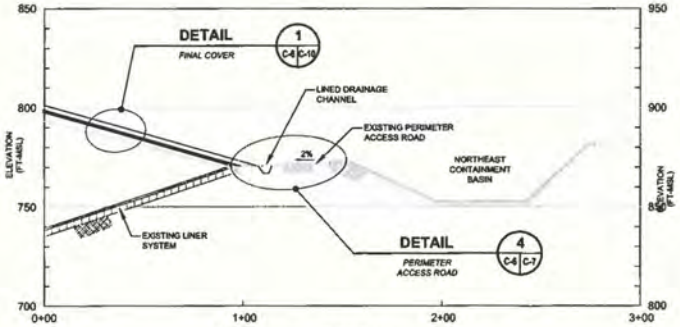
F C-3, C-4 C-6 SECTION
LOOKING EAST
PHASE III NORTH SIDE
1" = 30'



G C-3 C-4 SECTION
LOOKING NORTH
SOUTH STORMWATER CONTAINMENT BASIN
1" = 30'



H C-4 C-6 SECTION
LOOKING NORTH
NORTHEAST ACCESS ROAD
1" = 30'



I C-4 C-6 SECTION
LOOKING NORTH
EXISTING NORTHEAST PERIMETER ROAD AND STORMWATER CONTAINMENT BASIN
1" = 30'

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REV	DATE	DESCRIPTION	DESIGN	DRAWN	REVIEW
A	9/16/2008	ISSUED FOR PERMITTING MEETING	KJR	KJR	BKS
B	11/24/2008	ISSUED FOR PERMITTING	KJR	KJR	BKS
C	FEB 2010	REVISED PER PRECAUTIONS COMMENTS	KJR	KJR	BKS

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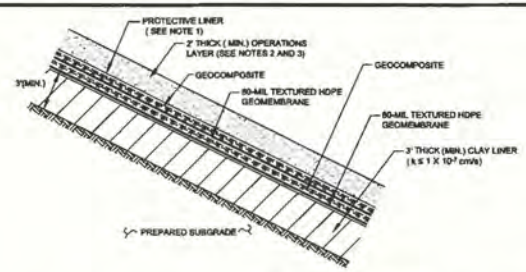
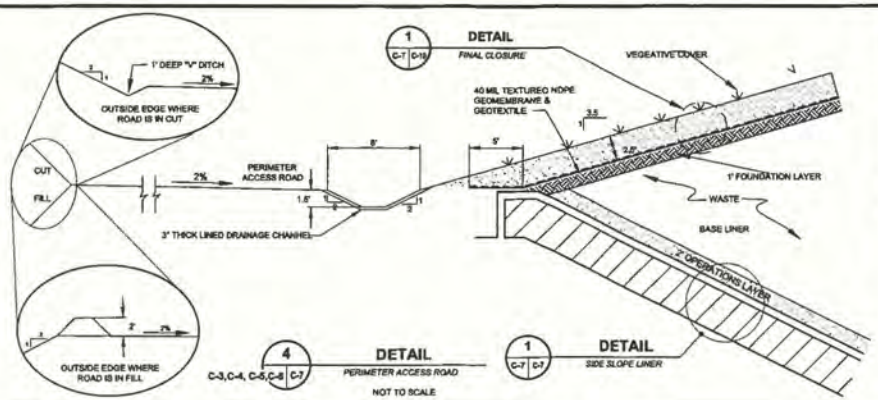
B-18 CLASS I LANDFILL
PHASE III EXPANSION AND FINAL CLOSURE
CROSS SECTIONS E TO I

SHEET NUMBER
C-6
7 OF 11 SHEETS



1 C-3,C-5, C-6,C-7 | C-7 **DETAIL**
SIDESLOPE LINER
NOT TO SCALE

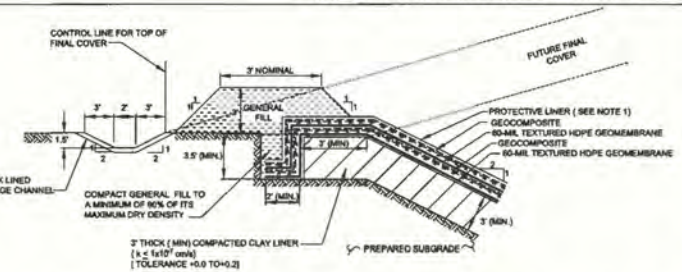
NOTE
1. NO HORIZONTAL SEAMS SHALL BE ALLOWED ON GRADES STEEPER THAN 12% OR WITHIN 3 FEET OF THE CREST OR TOE OF SLOPES. A HORIZONTAL SEAM IS DEFINED AS MORE THAN HALF OF THE PANEL WIDTH.
2. PROTECTIVE LINER SHALL BE COVERED BY OPERATIONS LAYER FOR A DISTANCE OF 3 FEET. PROTECTIVE LINER SHALL BE REMOVED BY LANDFILL PERSONNEL PRIOR TO PLACEMENT OF ADDITIONAL OPERATIONS LAYER.
3. OPERATIONS LAYER SHALL CONSIST OF SOILS EXCAVATED FROM ONSITE SOURCES. SOIL PARTICLE SIZE SHALL HAVE A MAXIMUM DIAMETER OF 1 INCH.
4. CONTRACTOR TO PLACE OPERATIONS LAYER 10' VERTICALLY UP THE SLOPE. CWM WILL PLACE REMAINING OPERATIONS SOIL AS WASTE MATERIALS ARE PLACED.



1 C-7 | C-15 **DETAIL**
FINAL CLOSURE

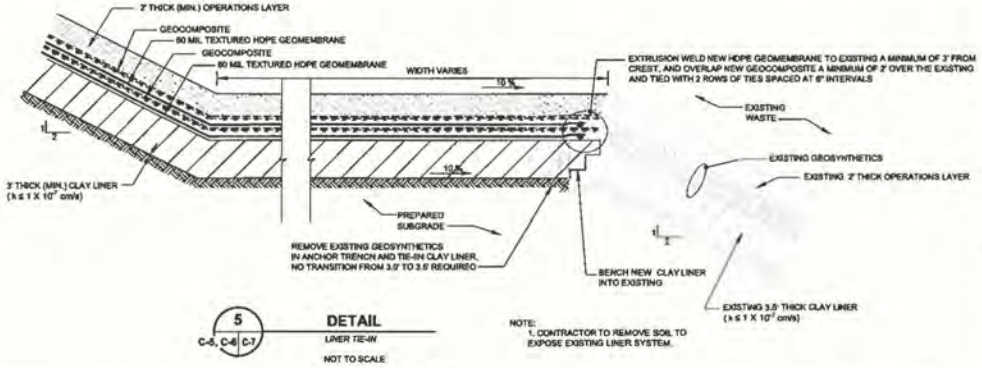
4 C-3,C-4, C-5,C-6 | C-7 **DETAIL**
PERIMETER ACCESS ROAD
NOT TO SCALE

1 C-7 | C-7 **DETAIL**
SIDE SLOPE LINER



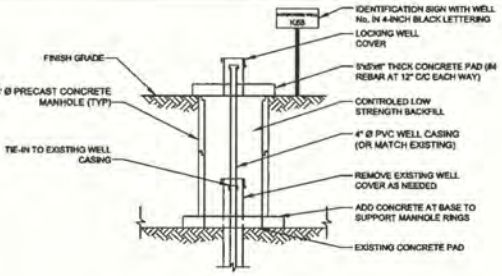
2 C-5, C-6 | C-7 **DETAIL**
SIDESLOPE LINER ANCHOR TRENCH
NOT TO SCALE

NOTE
1. PROTECTIVE LINER SHALL BE REMOVED BY LANDFILL PERSONNEL PRIOR TO PLACEMENT OF OPERATIONS LAYER.



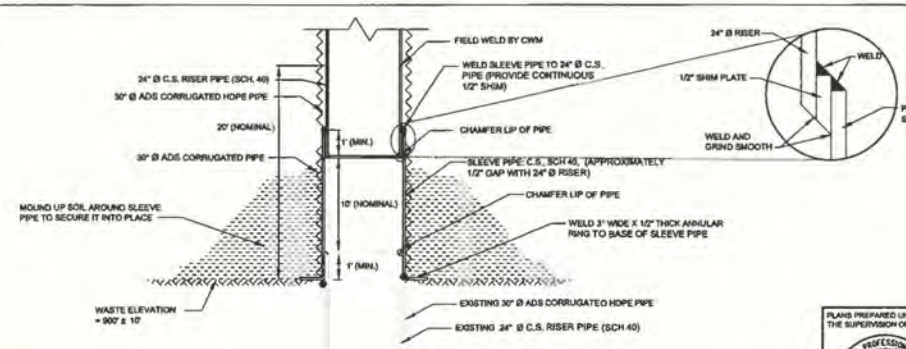
5 C-5, C-6 | C-7 **DETAIL**
LINER TIE-IN
NOT TO SCALE

NOTE:
1. CONTRACTOR TO REMOVE SOIL TO EXPOSE EXISTING LINER SYSTEM.



3 C-3 | C-7 **DETAIL**
MONITORING WELL EXTENSION
NOT TO SCALE

NOTE
1. CONTRACTOR TO EXTEND WELL AND ADD MANHOLE SECTION IN MAXIMUM 8 FOOT INCREMENTS. CONTROLLED LOW STRENGTH BACK FILL TO BE ADDED ONCE EACH 8' WELL EXTENSION IS COMPLETE.



6 C-3 | C-7 **DETAIL**
VERTICAL RISER SLIP CONNECTION
NOT TO SCALE

NOTE
1. THIS DETAIL PERTAINS ONLY TO VERTICAL RISERS IA AND IB
2. INITIAL SECTION TO BE INSTALLED BY CONTRACTOR SHALL BE 20' IN LENGTH.
3. ADDITIONAL SECTIONS OF 24" Ø RISER TO BE INSTALLED BY CWM AS WASTE PLACEMENT PROGRESSES.

PLANS PREPARED UNDER THE SUPERVISION OF:
REGISTERED PROFESSIONAL ENGINEER CIVIL STATE OF CALIFORNIA

ISSUED FOR PERMITTING

REV	DATE	DESCRIPTION	DESIGN	DRAWN	REVIEW
A	9/16/2008	ISSUED FOR PERMITTING MEETING	KJK	KJK	BSG
B	11/04/2008	ISSUED FOR PERMITTING	KJK	KJK	RGS
C	FEB 2012	REVISED PER PRODUCTIVE COMMENTS	KJK	KJK	RGS

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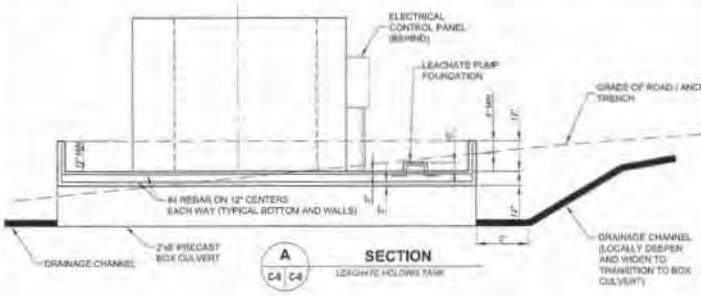
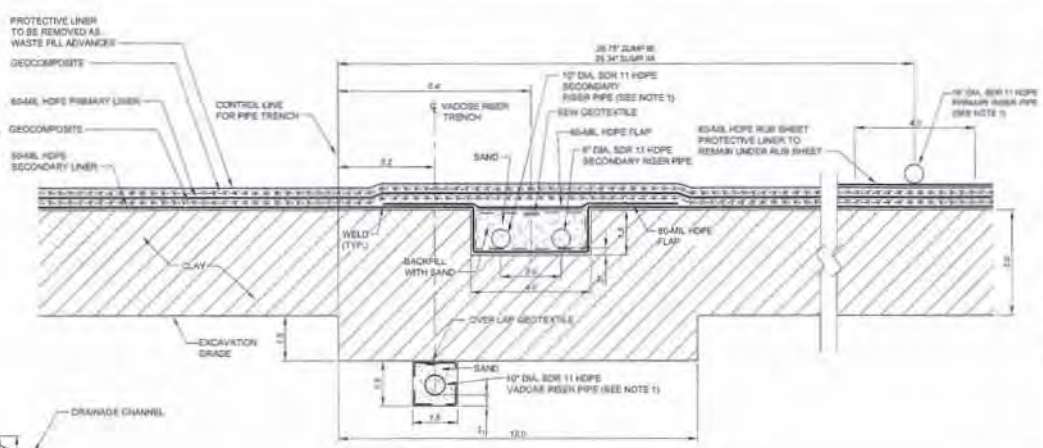
B-18 CLASS I LANDFILL
PHASE III EXPANSION AND FINAL CLOSURE
PHASE III BASE LINER CONSTRUCTION DETAILS

SHEET NUMBER
C-7
8 OF 11 SHEETS

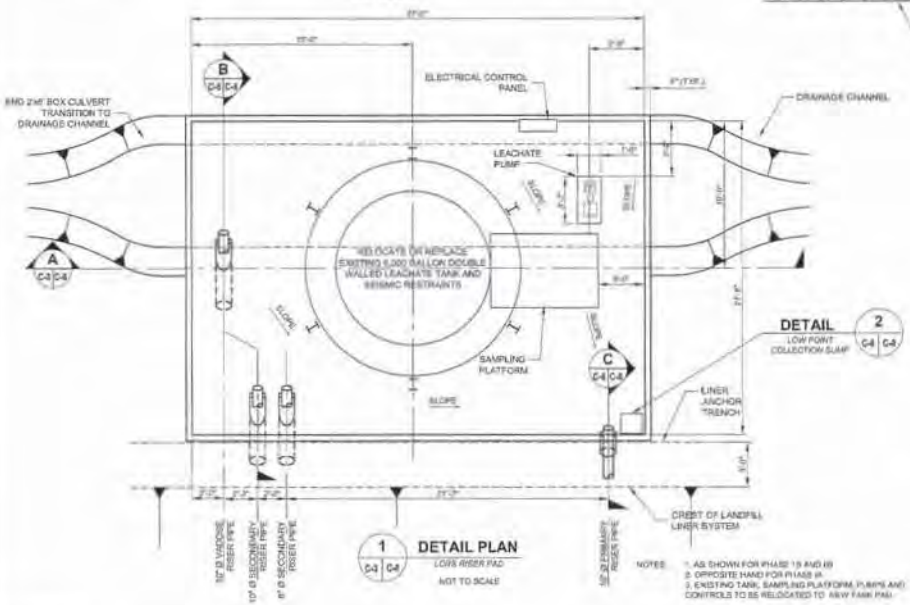


A C-8 | C-8 **SECTION**
LEACHATE HOLDING TANK
NOT TO SCALE



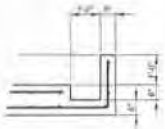
3 C-8 | C-8 **DETAIL**
SIDE SLOPE RISER TRENCH (TYP.)
NOT TO SCALE

NOTE:
1. EXISTING RISER PIPES ARE 8" Ø CARBON STEEL. SEE DETAIL 4 FOR TRANSITION BETWEEN CARBON STEEL AND HDPE PIPES
2. MATCH LOCATIONS OF EXISTING PIPES, DIMENSIONS MAY VARY

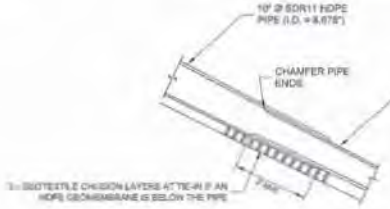


1 C-8 | C-8 **DETAIL PLAN**
LOW RISER PAD
NOT TO SCALE

NOTES:
1. AS SHOWN FOR PHASE 1B AND 1B
2. OPPOSITE HAND FOR PHASE 1A
3. EXISTING TANK, SAMPLING PLATFORM, PUMPS AND CONTROLS TO BE RELOCATED TO NEW TANK PAD

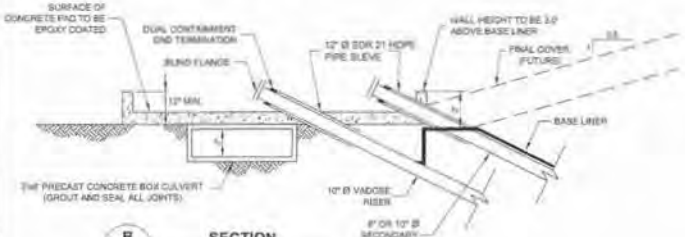


2 C-8 | C-8 **DETAIL**
LOW POINT COLLECTION SUMP
NOT TO SCALE

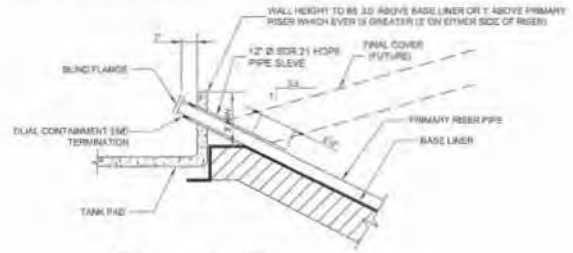


4 C-8 | C-8 **DETAIL**
RISER PIPE TRANSITION
NOT TO SCALE

NOTE: 10" Ø SDR 13.5 (I.D. = 9.062) MAY BE USED FOR 3' TRANSITION IF TOLERANCE IS TO TIGHT FOR SDR 11 PIPE



B C-8 | C-8 **SECTION**
VADOSE RISER PIPE
SECONDARY RISER PIPE
NOT TO SCALE



C C-8 | C-8 **SECTION**
PRIMARY RISER PIPE

ISSUED FOR PERMITTING

PLANS PREPARED UNDER THE SUPERVISION OF

REV	DATE	DESCRIPTION	DRAWN	DESIGN	REVIEW
4	5/16/2006	ISSUED FOR PERMITTING (REDTING)	KJP	JLB	SCB
3	11/14/2006	ISSUED FOR PERMITTING	KJK	JLB	PJG

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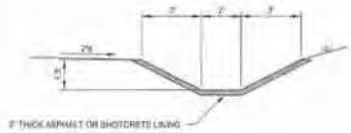
B-18 CLASS I LANDFILL PHASE III EXPANSION AND FINAL CLOSURE

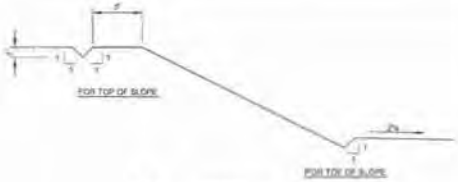
PHASE III LCRS SYSTEM DETAILS

SHEET NUMBER
C-8
8 OF 11 SHEETS

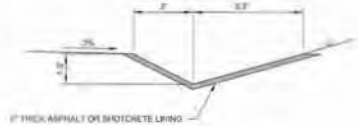


1 DETAIL
PERIMETER DRAINAGE CHANNEL

NOTE: SHOTCRETE SHALL HAVE WELDED WIRE FABRIC REINFORCEMENT

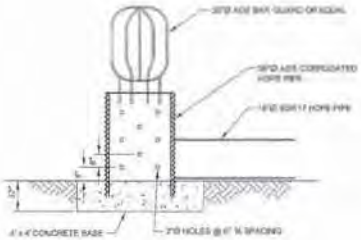


3 DETAIL
RUN-ON CONTROL CHANNEL

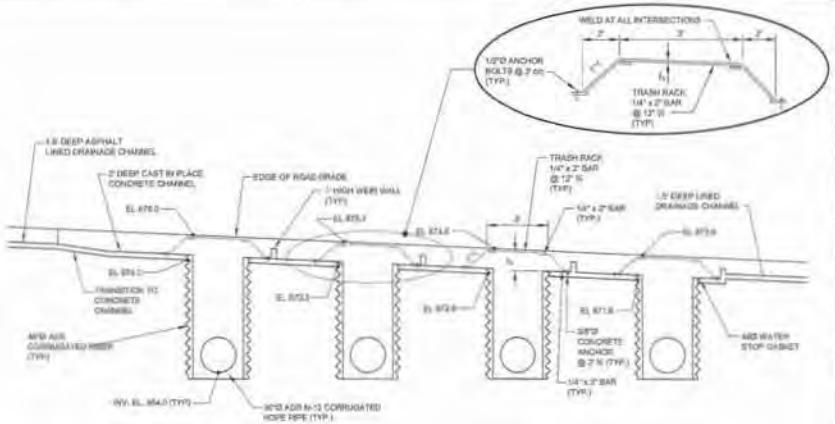


2 DETAIL
ACCESS ROAD DRAINAGE CHANNEL

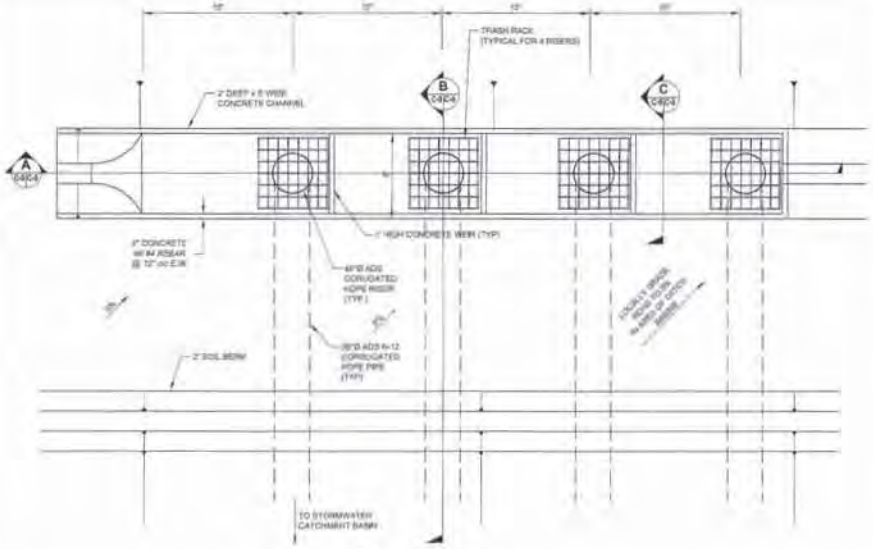
NOTE: SHOTCRETE SHALL HAVE WELDED WIRE FABRIC REINFORCEMENT



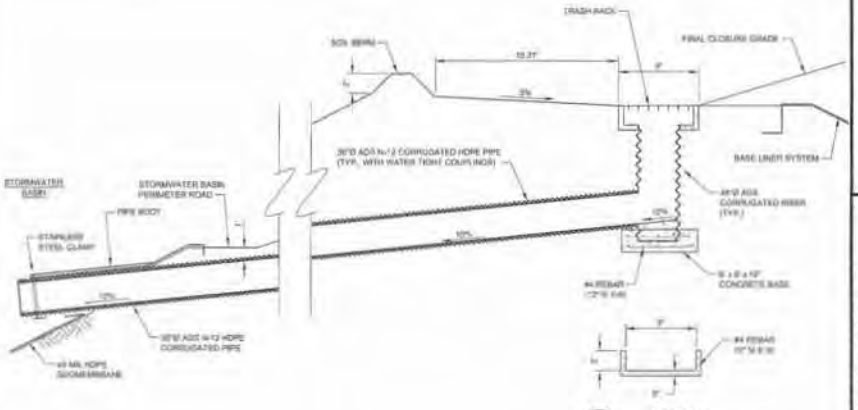
4 DETAIL
RISER INLET



A SECTION
BASIN INLETS



5 DETAIL
BASIN INLETS



B SECTION
BASIN INLETS

C SECTION
BASIN INLETS

PLANS PREPARED UNDER THE SUPERVISION OF

ISSUED FOR PERMITTING

REV.	DATE	DESCRIPTION	DESIGNED	DRAWN	REVIEWED
0	9/16/2008	ISSUED FOR PERMITTING MEETING	K.R.	K.R.	TKB
1	11/25/2008	ISSUED FOR SUBMITTAL	K.R.	K.R.	SLD

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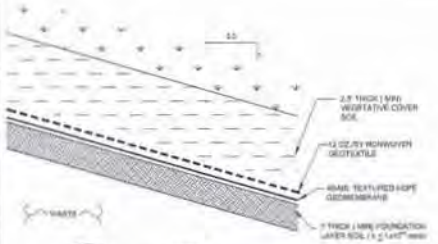
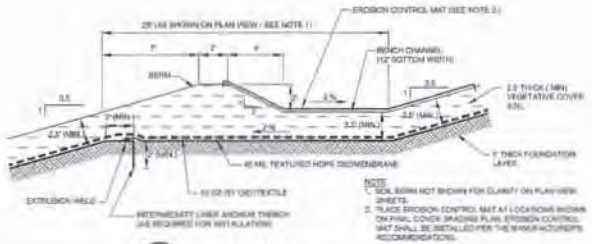
B-18 CLASS I LANDFILL
PHASE III EXPANSION AND
FINAL COVER

DRAINAGE DETAILS

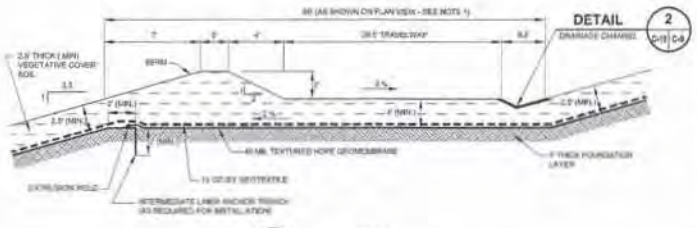
SHEET NUMBER
C-9
10 OF 11 SHEETS



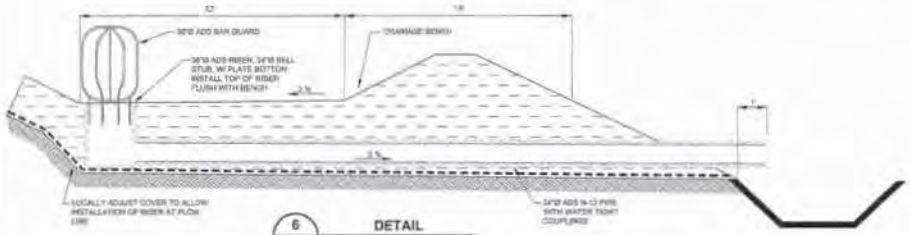
1 DETAIL
C-4, C-6, C-8, C-9
FINAL COVER
NOT TO SCALE



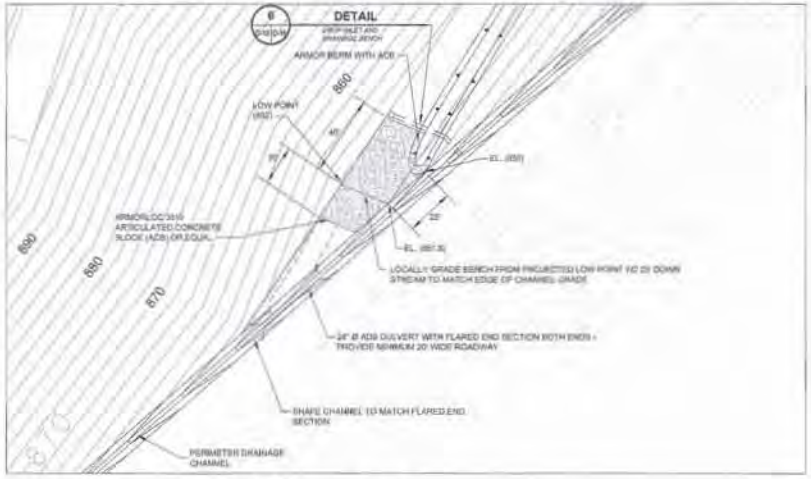
2 DETAIL
C-4, C-5, C-9
C-8 C-10
DRAINAGE BENCH
NOT TO SCALE



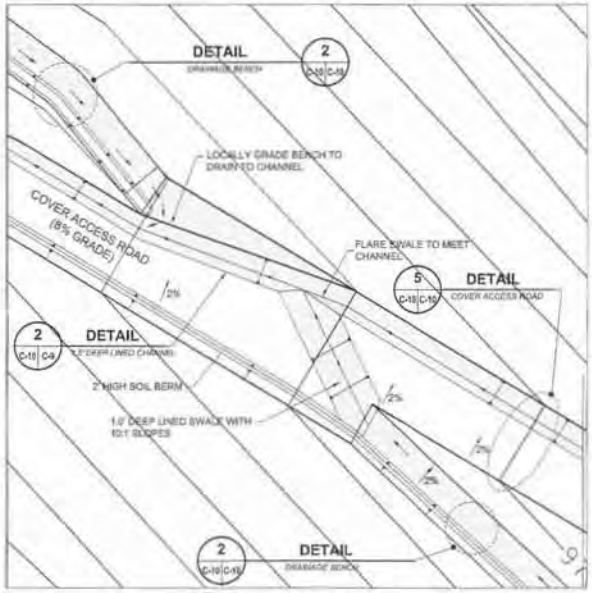
5 DETAIL
C-4, C-5, C-9
C-8 C-10
COVER ACCESS ROAD
NOT TO SCALE



6 DETAIL
C-4, C-10, C-10
COVER ACCESS ROAD
NOT TO SCALE



3 DETAIL
C-4 C-10
TYPICAL DRAINAGE BENCH TERMINATION
NOT TO SCALE



4 DETAIL
C-4 C-10
COVER ACCESS ROAD AT BENCH CROSSING
NOT TO SCALE
NOTE: DRAINAGE PATHWAYS SHADED FOR CLARITY

ISSUED FOR PERMITTING

PLANS PREPARED UNDER THE SUPERVISION OF
Expires 6-30-2010
CIVIL
STATE OF CALIFORNIA

REV	DATE	DESCRIPTION	DESIGNED	DRAWN	REVIEWED
A	8/18/2009	ISSUED FOR PERMITTING REVISED	K.R.	K.R.	D.R.
B	11/18/2009	REVISED FOR PERMITTING	K.R.	K.R.	D.R.

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B-18 CLASS I LANDFILL
PHASE III EXPANSION AND FINAL CLOSURE
CLOSURE DETAILS

SHEET NUMBER
C-10
11 OF 11 SHEETS