01-0308

REPORT

SDMS 157791

Proposal for Supplemental Source Control Containment / Recovery Measures - Lyman Street Site

General Electric Company
Pittsfield, Massachusetts

July 1999

BBL
BLASLAND, BOUCK & LEE, INC.
engineers & scientists

WESTON Ref. No.

01-0308

Transmitted Via Federal Express

Corporate Environmental Programs
General Electric Company
100 Woodlawn Ave., Pittsfield, MA 01201

July 12, 1999

Bryan Olson
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One Congress Street
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436 Dwight Street
Springfield, MA 01103

**Re: Proposal for Supplemental Source Control Containment / Recovery Measures - Lyman Street Site, General Electric Company, Pittsfield, Massachusetts
DEP Site No. 1-0856, USEPA Area 5A**

Dear Mr. Olson, Mr. Tagliaferro, Mr. Weinberg:

Enclosed please find the document entitled *Proposal for Supplemental Source Control Containment / Recovery Measures - Lyman Street Site.* This document supplements prior General Electric Company (GE) letters regarding this topic which were submitted to the United States Environmental Protection (USEPA) and the Massachusetts Department of Environmental Protection (DEP) on February 16, 1999 and April 26, 1999, and responds to comments received from the USEPA via letters dated March 23, 1999 and May 7, 1999.

Please contact me at (413) 494-3952 if you have any comments regarding this document.

Yours truly,

John D. Ciampa / DCK for

John D. Ciampa
Remediation Project Manager

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01-0308

Transmitted Via Federal Express

Corporate Environmental Programs
General Electric Company
100 Woodlawn Ave., Pittsfield, MA 01201

July 13, 1999

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Re: Proposal for Supplemental Source Control Containment / Recovery Measures - Lyman Street Site, General Electric Company, Pittsfield, Massachusetts DEP Site No. 1-0856, USEPA Area 5A

Dear Mr. Olson, Mr. Tagliaferro, Mr. Weinberg:

Enclosed please find a revised Figure 4 for the document entitled *Proposal for Supplemental Source Control Containment / Recovery Measures - Lyman Street Site* which was submitted yesterday. The figure has been revised to indicate that light non-aqueous phase liquid (LNAPL) has been observed in monitoring wells LSSC-5 and LSSC-6 rather than in soil borings LS-5 and LS-6, as indicated on the original figure. We apologize for any confusion this inadvertent error may have caused.

Yours truly,

John D. Ciampa / DCK for

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(*with enclosures)

TECHNICAL REPORT

Proposal for Supplemental Source Control Containment / Recovery Measures - Lyman Street Site

General Electric Company
Pittsfield, Massachusetts

July 1999

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Acknowledgments

This *Proposal for Supplemental Source Control Containment / Recovery Measures* was prepared on behalf of the General Electric Company by Blasland, Bouck & Lee, Inc., with groundwater modeling being performed by HSI GeoTrans, Inc., and the Wetland Reconnaissance Report being prepared by White Engineering, Inc.

Table of Contents

1. Introduction

1.1 General

This *Proposal for Supplemental Source Control Containment / Recovery Measures* (Source Control Proposal) describes the supplemental measures to be implemented at the General Electric Company's (GE's) Lyman Street Site (the Site) in Pittsfield, Massachusetts (USEPA Area 5A/MCP Site No. 1-12792). These measures have been designed to further address non-aqueous phase liquids (NAPL) that are present within the Site. Currently, GE operates and maintains three NAPL containment/recovery systems involving active groundwater pumping and NAPL extraction from on-site recovery wells, as well as the use of oil adsorbent booms located adjacent to the riverbank. To supplement these existing measures, GE proposed to install an approximate 400-foot length of steel sheetpile to provide an impermeable containment barrier at the base of the riverbank. This report describes the various activities conducted/proposed by GE concerning the selection, evaluation, design, and installation of the proposed containment barrier and related activities.

The information provided in this document supplements the information GE previously submitted to the United States Environmental Protection Agency (USEPA) and the Massachusetts Department of Environmental Protection (MDEP) (jointly referred to as the Agencies) in letters dated February 16, 1999 and April 26, 1999. GE prepared and submitted the February 16, 1999 letter to the Agencies pursuant to a letter from the USEPA dated October 6, 1998 and as follow-up to GE's *Source Control Work Plan - Upper Reach of Housatonic River (First ½ Mile)*, dated September 1998 (Source Control Work Plan). The February 16, 1999 letter included a summary of recent field investigations and historical NAPL occurrence for the Site. With this information serving as the basis, the February 16, 1999 letter also outlined GE's conceptual plans proposed to supplement the NAPL containment/recovery measures that are currently in place in this area, and identified additional site investigations to support the detailed design of the proposed measures. Subsequently, in an April 26, 1999 letter, GE summarized the results of supplemental soil sampling and analysis activities which were proposed by GE in its February 16, 1999 letter.

The information presented in the February 16, 1999 and April 26, 1999 letters serves as the basis for the containment/recovery measures proposed in this Source Control Proposal.

1.2 Format of Report

A summary of the prior field investigations and results is provided in Section 1.3. The remainder of this Source Control Proposal is divided into two sections. Section 2 of this Proposal presents GE's plan for supplemental NAPL containment/recovery within the riverbank area, and presents information (and supporting technical information) regarding the design of the proposed sheetpile containment barrier. This section includes the final sheetpile layout, detailed design information (including the results of the hydraulic modeling), and a description of the anticipated implementation activities associated with the proposed containment barrier. This includes the anticipated construction sequence, measures to mitigate potential environmental disturbances during construction, temporary/final site restoration details, and post-construction monitoring activities. Section 3 presents a summary and anticipated project schedule.

1.3 Background/Overview of Prior Investigations

1.3.1 General

Over the last several years, GE has conducted extensive investigations and has implemented numerous activities to control, contain, and recover NAPL in this area of the GE facility. The information generated as a result of these activities was useful in defining the scope of the recent source control investigations; understanding and assessing the results of the recent activities; and selecting and designing the supplemental NAPL containment/recovery measures proposed herein. A complete summary of the historic information is beyond the scope of this Source Control Proposal. However, it can be found in other documentation previously provided by GE. Where appropriate, an expanded discussion of relevant historical information is provided with appropriate references.

This section of the Source Control Proposal also provides a brief summary of the results of field investigations conducted between December 14, 1998 and April 2, 1999. More detailed information concerning these investigations is presented in a report entitled *Source Control Investigation Report - Upper Reach of Housatonic River (First ½ Mile)* [prepared by HSI GeoTrans, Inc. (GeoTrans), dated February 9, 1999], GE's February 16, 1999 and April 26, 1999 letters mentioned above, and a report entitled *Source Control Investigation Addendum Report, Upper Reach Housatonic River (First ½-Mile)* (prepared by GeoTrans, dated June 15, 1999). The following summary focuses primarily on information that is directly related to the design of the supplemental containment/recovery measures presented herein.

1.3.2 Overview of Recent Investigations

Between December 14, 1998 and January 5, 1999, GeoTrans, on behalf of GE, advanced a total of eleven soil borings (LSSC-1 through LSSC-11) at the Site, as shown on Figure 1. Nine of the soil borings were subsequently converted into monitoring wells to gauge water table elevations and to monitor for the presence of NAPL. The results of the analyses performed as part of these activities are provided in the previously referenced February 9, 1999 report prepared by GeoTrans. During this time period, seismic refraction surveys were also conducted by Geophysical Applications, Inc. (GAI) to further assess the configuration of the till confining layer beneath the Site and adjacent areas. These results were also provided in that report. On January 29, 1999, four shallow soil borings (LSSC-12 through LSSC-15) were advanced by Blasland, Bouck, & Lee, Inc. (BBL) on behalf of GE at locations along the riverbank adjacent to the Site (Figure 1). Soil samples were collected at one-foot intervals to a depth of 6 to 8 feet below grade and submitted for laboratory analysis of total petroleum hydrocarbons (TPH). The results of these analyses were presented in GE's February 16, 1999 letter to the Agencies and are summarized on Figure 2 of this proposal.

Between March 29 and April 5, 1999, ten additional soil borings (LSSC-20 through LSSC-25, and LSSC-27 through LSSC-30) were advanced along the riverbank adjacent to the Site, and two shallow monitoring wells were installed at each end of the proposed containment barrier (LSSC-08S and LSSC-18) to assess the presence of NAPL at this location. The location of these borings are presented on Figure 1. The ten soil borings were positioned along the proposed alignment of the containment wall, as shown in GE's February 16, 1999 letter to the Agencies. A total of 108 soil samples (including duplicates) were collected from these riverbank borings. Samples were collected to a depth of at least 10 feet below grade, depending upon visual observations and sampling technique limitations. Soil samples adjacent to and below the depths previously sampled were analyzed in 1-foot increments for TPH and PCBs, and also examined for evidence of NAPLs. These data were presented in GE's April 26, 1999 letter to the Agencies; however, these data are re-presented in Table 1 of this proposal. Additionally, a cross-section of the area represented by these soil borings is presented on Figure 3.

With respect to monitoring wells LSSC-08S and LSSC-18, no indications of the presence of NAPL were noted during installation or subsequent monitoring of these wells (GeoTrans, June 15, 1999).

In general, based on the information gathered as a result of the soil boring installations and the geophysical survey performed in this area, the stratigraphy of this area is comprised of fill and fluvial deposits which overlay a dense silt and silty sand layer, interpreted to be till. The till unit is at least 40 feet thick at some locations. Finally, bedrock lies beneath this till layer at approximately 50 to 60 feet below grade.

2. Description of Supplemental NAPL Control Measures

2.1 General

Based on the results of the recent and prior investigations, supplemental containment/recovery measures are proposed to further address the known or potential presence of NAPL within the subsurface soils in this area. These proposed measures are in addition to the existing containment/recovery activities that have been implemented by GE over the last several years. As previously indicated, the NAPL monitoring, control, and recovery activities currently being conducted along the first ½ mile of the Housatonic River are preventing any significant migration of NAPL into the river. The activities proposed herein supplement the existing measures and provide further assurances of NAPL containment. The primary component of the proposed supplemental NAPL containment/recovery measure is the installation of a physical containment barrier along and parallel to a portion of the Housatonic River riverbank. Specifically, GE proposes the installation of an approximately 400-foot long steel sheetpile wall parallel to and along the edge of the river, as shown on Figure 4.

The location and depth of the proposed containment barrier was selected based on the results of field investigations to include those areas (both vertically and horizontally) where NAPL has been identified or may be potentially present. Once this area was determined, several other technical and operational factors were considered in the detailed design activities, such as possible impacts to the existing hydrogeologic conditions in the area (and the existing NAPL containment/recovery measures) and possible effects of future river flooding on the migration/containment of NAPL. In addition, the design of the proposed containment barrier also considered the future response actions to be performed within the first ½ mile of the Housatonic River. Specifically, for this section of the river, GE will be conducting response actions involving river sediments and bank soils. Accordingly, the design of the proposed containment barrier considered potential excavation and restoration requirements for the response actions for the first ½ mile.

2.2 Extent and Type of Containment Barrier

As previously indicated, the existing NAPL control/recovery measures associated with the Site are sufficient to preclude any significant NAPL migration. Nonetheless, a sheetpile wall is proposed as an additional and supplemental containment/recovery measure. The location of the proposed containment barrier has been selected based on a number of considerations, including evidence of NAPL and/or soil staining, laboratory analytical results, historic groundwater elevations, typical river elevations, and existing bank geometries. A summary of information supporting the proposed horizontal and vertical extent of the sheetpile containment barrier, and type of sheetpile, is presented below.

It should be noted that the actual alignment of the containment barrier may be adjusted somewhat during construction based on actual field conditions. These field adjustments are not anticipated to be significant and are further explained in Section 2.3.5.

2.2.1 Horizontal Extent

The horizontal extent of the proposed containment barrier is shown on Figure 4. This location has been selected based on a review of information obtained from the recent investigations summarized in Section 1.3 and pertinent data from prior investigations conducted in this area. Using this information, the location of the proposed containment barrier was established to include known areas of NAPL which could potentially migrate toward the river. The location and alignment of the proposed containment barrier has been selected considering both light non-aqueous phase liquids (LNAPL) and dense non-aqueous phase liquids (DNAPL). DNAPL has been found at the Site west of well LS-38; however, it occurs in a trough in the till layer at depths below the river bed. As further detailed and evaluated in the June 15, 1999 report prepared by GeoTrans, the till layer in this area slopes away from the river, and the trough within the till layer where the DNAPL is found slopes in a direction generally perpendicular to Lyman Street. DNAPL has not been found along the riverbank along the proposed alignment of the containment barrier.

The western extent of the proposed containment barrier will be adjacent to the Lyman Street bridge abutment. In that area, the containment barrier would include the area immediately downgradient of monitoring well LS-38. Well LS-38 appears to represent the western limit of LNAPL migration. This well has been monitored regularly since its installation in 1995. During that time period, LNAPL was detected on only three occasions in extremely small quantities (thickness of 0.01 feet). Two well points (P-6 and P-7) located immediately downgradient and downslope from well LS-38 have been monitored since the latter part of 1994 and have never indicated the presence of NAPL. Wells recently installed on the west side of the Lyman Street bridge abutment (LSSC-08I and LSSC-08S) did not detect the presence of separate phase NAPL, and there were no indications of staining or sheens near the top of the water table. The eastern end of the proposed containment barrier will extend upstream to a location near well point P-5 and well LS-24. NAPL has not been detected in these wells or in nearby wells LS-20, LS-22, LS-25, or RW-2. Perpendicular wing walls will extend up the bank approximately 40 feet at both ends of the proposed barrier wall. Based on these design parameters, the length of the proposed containment barrier along the riverbank is approximately 300 feet. With the addition of the wing walls, the overall length of the proposed containment barrier will be approximately 400 feet.

2.2.2 Vertical Extent of the NAPL Containment Barrier

Several considerations were taken into account in selecting the vertical extent of the proposed containment barrier, including the results from recent and prior investigations; historic, current, and predicted groundwater hydraulics; existing and future NAPL containment/recovery measures; and geotechnical considerations. From this information, it is anticipated that the vertical extent of the containment barrier will extend at least to the upper surface of the till unit (i.e., approximately 963 to 968 feet), which corresponds to the vertical extent to which NAPL has been detected. In addition, the sheetpile will extend approximately 10 feet into the till (at least an elevation of 956 feet) in certain areas for geotechnical reasons (i.e., structural integrity of sheetpile wall). If the sheetpile cannot be advanced to the design elevation, the embedment requirements will be re-evaluated based on actual field conditions. If necessary, tiebacks or other mechanical methods will be used to provide the required support for bank soil removal activities.

The proposed upper elevation of the containment barrier is between 977 feet to 978 feet, except near the bridge abutment where the upper elevation will be at existing grade, as shown on Figure 4. These top of sheetpile elevations are generally conservative based on the data summarized on Figure 2. Specifically, as shown on Figure 2, and explained previously in GE's February 16, 1999 letter, LNAPL sheens or staining was not observed above an elevation of 974 feet, and the TPH values decrease rapidly to low levels at elevations which approach 978 feet.

In addition to the presence of NAPL and adjacent areas with stained soils, groundwater hydraulics and geotechnical considerations were factored into the selection of the location and configuration (e.g., vertical extent) of the proposed containment barrier. A summary of this information is presented below.

2.2.2.1 Historical Groundwater Data

Historical river levels and adjacent groundwater levels were evaluated using data dating back to 1992. Specifically, for the river levels, the data set consists of weekly monitoring results available from between January 1992 and January 1999. Weekly groundwater level measurements from several riverbank wells and well points are available from September 1992 to January 1999. Weekly monitoring data for the river and three of the representative nearby well points (P-1, P-3, and P-4) are depicted on hydrographs provided in Appendix A (previously provided to the Agencies as part of GE's February 16, 1999 letter).

As shown on Figure 2 of Appendix A, river elevations ranged from 970.14 to 976.50 feet above mean sea level, with an average level of 971.20 feet. During the 7-year monitoring period that was evaluated, four monitoring events identified river elevations greater than 974 feet. [Note that modeling performed related to other first ½ mile river activities indicates that the river levels corresponding to a 2-year recurrence interval flow event at the Lyman Street Site measure in the range of about 976 to 977 feet.] Groundwater levels in the riverbank well points between September 1992 and January 1999 ranged from 970.35 feet to 976.43 feet and averaged 971.2 feet. The average river and groundwater levels in this area are significantly below the proposed elevation for the top of the containment wall of 977 to 978 feet.

In general, comparison of the river and groundwater elevation data indicates that groundwater levels are generally slightly higher than or nearly equal to river levels. However, during several of the "high-water" monitoring events (i.e., when the river level was above 974 feet), the groundwater levels were lower than that of the river, indicating a landward flow direction. A groundwater level contour map representing low-water conditions is presented in Appendix A. Less frequent high-water conditions (i.e., river level above 974 feet) are also presented in Appendix A. It should be noted that the high-flow river event that occurred on June 15, 1998 was not contoured because groundwater levels along the riverbank were not measured concurrent with that event. Although the river could conceivably overtop the proposed containment barrier wall of 977 to 978 feet on rare occasions, such occurrences, if any, would be expected to be short-term. It is also expected that during such occurrences, the groundwater gradient will be landward along the riverbank due to river water infiltration. Also, LNAPL is not expected to migrate vertically to or above 977 to 978 feet as evidenced by the TPH sampling results and visual observations of soil cores in the riverbank, summarized on Figure 3.

2.2.2.2 Geotechnical/Structural Considerations

Concurrent with the preparation of this Source Control Proposal, GE has evaluated and proposed response actions for the bank soils and sediments within the first ½ mile of the Housatonic River (i.e., between Lyman Street and Newell Street). A work plan identifying such response actions was submitted by GE to the Agencies on June 25, 1999.

With respect to the proposed containment barrier, the depth of the proposed sheetpiling (to an elevation of approximately 956 feet) facilitates the removal of approximately 2 feet of sediment from the portion of the river located immediately adjacent to the sheetpiling as proposed in the First ½ Mile Work Plan (approximate removal elevation of 967 feet). Appendix C to this Source Control Proposal provides the supporting geotechnical and structural calculations.

2.2.2.3 Predictive Groundwater Modeling

The groundwater hydraulics associated with typical hydrogeologic conditions in this area were modeled by GeoTrans using the publicly available and well-documented MODFLOW program. The model extends from 410 feet southwest of Lyman Street to 470 feet beyond the northeast edge of the parking lot. Vertically, the model extends from the water table to elevation 930 feet. This three-dimensional model uses a grid consisting of 65 rows and 95 columns represented with eight layers to simulate the flow system. The Housatonic River and the existing groundwater pumping wells are included in the simulation. A detailed discussion of the modeling with graphical output is provided in Appendix B; a summary is provided below.

The model boundary conditions consist of constant head boundaries to the northwest (East Street) and to the southeast (Housatonic River). The northeast and southwest boundaries are modeled as no flow because they represent flow lines. Groundwater discharge to three recovery wells (RW-1, RW-2, and RW-3) are modeled using constant head boundaries. The bottom boundary of the model represents no flow. The river elevation is modeled as decreasing linearly from an elevation of 971.6 feet in the east to 971.1 feet in the west.

The model was calibrated using average water levels from 37 monitoring wells and the average groundwater extraction rates from recovery wells RW-1, RW-2, and RW-3. After calibration, two simulations were performed, one with a barrier extending from the water table to an elevation of 960 feet and the second with the bottom elevation of the barrier at 950 feet. The lateral extent of the modeled wall ran from well LS-38 southward along the Lyman Street bridge abutment, east along the bank of the Housatonic River to well P-5, and north to well LS-24. The model demonstrates that the wall will not significantly affect the hydrogeologic flow regime of the Site and that the existing recovery wells will continue to effectively capture LNAPL at the Site. The wall will potentially cause a slight decrease in the water elevations across the Site and the pumping rates of the recovery wells. Pumping rates in wells RW-1R, RW-2, and RW-3 during the model simulation were reduced by 29, 24, and 20 percent, respectively. Also, there was no significant difference between the simulations with the bottom of the wall at 960 or 950 feet.

2.2.3 Selection of Containment Barrier Type

The proposed containment barrier will be constructed of a steel sheetpile wall with sealable joints. Similar steel sheetpiling has been successfully installed at two locations less than ½ mile upstream at GE's Building 68 and East Street Area 2 Site. The sheetpile wall will be constructed of Waterloo brand, heavy-wall, sealable sheetpiling (WEZ95) manufactured by Canadian Metal Rolling Mills under license to the University of Waterloo. The heavier gauge sheeting has been selected for this application, since it will be necessary to advance the sheeting approximately 10 feet into the till layer. The sheeting will be driven into place with a vibratory or impact hammer. Waterloo Barrier™ sheetpiling consists of a L-shaped steel sheetpile section, which has a cold-rolled joint that is larger than a typical hot-rolled joint and therefore facilitates sealing to form a low-permeability barrier. Extensive field testing at the CFB Borden site in Canada and at other locations has demonstrated that permeabilities of 1×10^{-8} to 1×10^{-10} cm/sec are achieved in practice. Waterloo Barrier™ has also been installed at a number of sites in the United States, including Hill Air Force Base (Utah), Lowry Air Force Base (Colorado), Dover Air Force Base (Delaware), and Alameda Naval Station (California).

Common joint sealants for the Waterloo barrier and other sealable sheetpile walls include bentonite, vermiculite or cementitious grouts; epoxies; and other organic polymers. The potential deflection of the wall has been calculated for various loading conditions that may be expected during the response actions associated with the first ½ mile of the river. The tensile stress and strain due to bending were compared to the properties of a typical grout to evaluate if significant cracking would be likely. These analyses (included in Appendix C) indicate that the wall system is very stiff and that potential deformations are insignificant and unlikely to cause cracking of the grouted joint. Nevertheless, since there will be future construction activities performed in the vicinity of the containment barrier during the sediment and bank soil response actions for the first ½ mile, the sheetpile joints will be left ungrouted until the completion of the response actions to avoid potential joint damage that may be caused by construction-related impacts.

The expected life of the sheetpile containment barrier is in excess of 60 years. Corrosion of steel sheetpiles requires either the presence of low resistivity or low pH materials in moist, aerobic environments, or the presence of sulphate-reducing bacteria in anaerobic conditions. None of these conditions appear to be present at the Site. Historic groundwater pH values from wells in the area are 6.6 to 7.8 (LS-10 through -13, LS-20, LS-22 through -25, LS-32, and LS-33). Published information indicates that sheetpile corrosion may be significantly accelerated due to acidic conditions only in environments with pH of less than 4. The specific conductance of groundwater from these wells was measured to be 0.145 to 1.9 mS/cm. These specific conductances are within the range of potable water, indicating a non-saline, and therefore low-corrosivity, environment.

Published data indicates that the buried components of the steel sheetpiling, even if installed in materials such as slag, are likely to have a corrosion loss of 0.03 mm or less per year resulting in corrosion-related lifespans of 100 years or more. Due to the lack of oxygen, normal underground anaerobic environments help protect steel. Exposed portions of the sheetpiling may have faster corrosion rates due to the presence of air and water, but would still be less than the loss rate of 0.05 mm/year that can be assumed for splash zones in marine environments. Hence, a corrosion-related expected lifespan of longer than 60 years is projected.

2.3 Proposed Implementation Activities

2.3.1 Permits and Approvals / Pre-Mobilization Activities

All permits and approvals necessary for the implementation of the activities discussed above (e.g., DIGSAFE utility clearances) will be obtained by either GE or its selected contractor(s) prior to initiation of any on-site activities. Other pre-mobilization activities to be performed/coordinated by GE prior to the start of on-site activities will include selection of a contractor; discussion with the sheetpile manufacturer regarding availability; and contact with utility companies to identify potential subsurface utilities in the riverbank area.

In addition to these activities, GE is including as Appendix E an evaluation of the potential impacts of the proposed project on areas subject to the Massachusetts Wetlands Protection Act (310 CMR 10.00). Although approval from the Pittsfield Conservation Commission is not necessary to implement this project (since the project is an on-site removal action under the Comprehensive Environmental Response, Compensation, and Liability Act), the evaluation addresses the substantive requirements of the Massachusetts Wetlands Protection Act.

2.3.2 Working Limits

The construction of the proposed containment barrier will require use of the area located north of the Housatonic River and entirely within GE-owned property. The paved areas adjacent to the top of the bank will be the primary location for contractor equipment and work areas. Current access restrictions (i.e., perimeter fencing and locked access gates) will be maintained and supplemented as necessary through temporary measures. While it is anticipated that the contractor's work will generally be restricted to the areas on the bank and immediately adjacent to the bank, other existing GE facilities may also be used for required work items such as water sources and excavated soil staging areas. The anticipated work limits are shown on the technical drawings included as Appendix D to this Source Control Proposal.

2.3.3 Site Preparation

A number of site preparation activities will be performed/coordinated by GE prior to the start of on-site construction activities, including contractor mobilization; installation of erosion and sedimentation control measures; clearing and removal of existing trees and vegetation as necessary for access; protection of existing structures and facilities; and relocation of existing utilities (as necessary). A summary of the various site preparation activities is provided below.

Once a remediation contractor has been selected and the necessary pre-mobilization submittals have been prepared and submitted to GE (including a contractor-specific health and safety plan), the contractor will mobilize to the Site. The initial mobilization may include the provision of temporary office facilities, health and safety equipment, and materials necessary to conduct the initial site preparation activities.

Prior to the initiation of vegetation clearing and soil removal actions, the contractor will install the necessary erosion control measures. Such measures -- which are expected to include the use of staked hay bales, silt fencing, and silt curtains -- will be selected in consideration of the area subject to control measures and the applicable provisions of the Massachusetts Wetlands Protection Act. For example, along the base of the river bank, a silt fence will be installed. It is also proposed that a silt curtain will be installed within the river along the base of the bank. Any control measures that are installed will be inspected on a regular basis and repaired/replaced as needed.

To facilitate access for the equipment associated with the sheetpile installation, the majority of the existing bank vegetation will need to be cleared and removed from the bank. However, with the exception of the lowest portion of the bank along the river (which is subject to removal and sheetpile installation), it will not be necessary to remove the root structures or stumps associated with the existing vegetation. All vegetation cleared from the Site will be disposed of at an appropriately-permitted off-site disposal facility or transported to the appropriate on-plant consolidation area.

With regard to existing utilities, there is an existing overhead 100 amp, 480 volt electric service and an existing telephone line located along the top of the bank. These lines will be rerouted or decommissioned prior to start of the project. Finally, the steel sheetpiling will be delivered to the Site and stored in an area located adjacent to the top of the bank near the east end of the proposed containment barrier. The sheetpiling will be stored flat on blocking per manufacturer's recommendations.

2.3.4 Removal of Soils and Sediments Adjacent to the Containment Barrier

Along the length of the proposed containment barrier, it is expected that certain bank soil and sediment removal actions will occur within the first ½ mile of the river. Additionally, certain removals are necessary to facilitate the installation of the proposed containment barrier and to minimize the potential for disturbance and mobilization of residual NAPL, if any, which may be located between the proposed containment barrier and the river. The scope of these activities is described below.

Prior to installing the proposed containment barrier, GE will remove (as needed) certain bank soils positioned between the proposed containment barrier and the edge of the river. The primary purpose of this pre-installation soil removal is to minimize the potential for sloughing of the soils located immediately adjacent to the river during the subsequent installation of the steel sheetpiling. As shown on Figure 4, the existing bank soils subject to removal will be excavated to a depth approximately equivalent to the typical elevation of 1-foot above the river level (i.e., 972.5 feet). This removal is anticipated to be required along the full length of the sheetpile wall, except for an approximate 60-foot stretch of the wall located between the existing piezometer P-7 and the Lyman Street bridge (see Figures 4 and 5).

In addition to the "pre-installation" bank soil excavations described above, additional bank soil will be excavated along the river side of the sheetpile wall based on the soil boring data collected from this area as reported in GE's April 26, 1999 letter. As discussed in that letter, the purpose of those data were to facilitate the assessment of the potential presence of LNAPL residuals along the river bank on the riverside of the proposed sheetpile wall, and to serve as the basis for additional excavation of soils in this area exhibiting the presence of such materials. Based on the evaluation of these data, GE proposes to excavate additional soil along the river side of the sheetpile to an elevation of 967.5 feet (as shown on Figure 4), generally between soil borings LSSC-20 and the upstream end of the proposed sheetpile wall (near boring LSSC-24). As shown on Figure 3, this area exhibited the highest PCB and TPH values observed along the riverbank in this area. Additionally, it corresponds to the area where bank seeps have been historically observed. Below 967.5 feet along this stretch of the riverbank, PCB and TPH values are generally low or non-detect. As for the remaining portions of riverbank adjacent to the sheetpile, "deeper" soil excavation beyond the "pre-installation" excavation does not appear to be required based on the available analytical data and field information. Specifically, as reported by GeoTrans in their June 15, 1999 report, the PCB component of the LNAPL found at the Lyman Street is comprised of Aroclor 1254. This is further supported with the results of PCB analysis of LNAPL collected recently (June 22, 1999) from recovery well RW-3 (see Figure 4 for location). Those data indicated the LNAPL from this well to contain PCBs at a concentration of 146,000 ppm, of which approximately 90 percent was comprised of Aroclor 1254. As shown in Table 1, PCB concentrations detected in the riverbank soil samples collected from borings LSSC-20 and LSSC-27 through LSSC-30 are primarily comprised of Aroclor 1260, which is not the main component of the LNAPL found at the Site. Some higher concentrations of Aroclor 1254 are present in soil at borings LSSC-27 and LSSC-28. However, it is detected above the water table, and thus it is not attributed to the presence of separate phase LNAPL at these locations. Furthermore, well point P-6 is located immediately adjacent to these borings and has never shown the presence of LNAPL.

Nevertheless as a conservative measure, GE proposes to excavate soil to a depth of approximately 2 feet below the average water table (to an elevation of 969.5 feet) along the riverbank between soil boring locations LSSC-20 and LSSC-30 (See Figure 4). This zone borders the eastern end of the oxbow where LNAPL has been observed in well LS-35 and well point P-1.

It is expected that the "deeper" soil removal activities associated with the stretch of riverbank described above will be performed along with the removal activities to be performed as part of the first ½ mile removal actions. During the interim timeframe between the initial "pre-installation" bank soil excavations and the "deeper" soil removal/final restoration activities, temporary erosion control measures will be implemented as was recently performed by GE as part of the containment barrier installations at the East Street Area 2 Site. Figure 5 presents several typical cross-sections which illustrate these removal activities.

It is expected that all soils subject to removal will be transported to and consolidated at the appropriate on-plant consolidation area per GE's *Detailed Work Plan for On-Plant Consolidation Areas*, dated June 1999. Prior to initiating the soil excavations described above, a silt curtain will be installed as shown on the technical drawings included as Appendix D to this report. Also, as the excavations are performed and prior to the initiation of sheetpile installation, a silt fence will be installed along the edge of the river. The silt fence installation is also shown on the technical drawings in Appendix D.

2.3.5 Sheetpile Alignment and Installation

The sheetpiling associated with the proposed containment barrier will be installed using a vibratory or impact hammer along the approximate alignment shown on Figure 4. Driving will be restricted to daylight hours due to safety and noise considerations. As previously mentioned, the alignment may be modified slightly in the field during construction in order to better accommodate specific obstructions or slope geometries that may be encountered. The primary goals related to installation of the barrier wall are to advance the sheetpiling a distance of 6 to 7 feet from the river's edge, allowing for the achievement of a slope of approximately one vertical to one horizontal (1:1) upon restoration.

2.3.6 Site Restoration

Figure 5 illustrates the restoration activities that will be implemented for the lower portion of the riverbank following installation of the proposed containment barrier. As described in Section 2.3.4, the soils in the area will be removed in order to minimize the potential for sloughing caused by the vibratory or impact nature of the sheetpile installation and to remove potential LNAPL residuals which might be present within soils along the riverside of the sheetpile wall. The restoration activities for this bank area have been developed in consideration of river hydraulics, existing flood storage capacity, and structural considerations. With respect to the remaining portions of the riverbank potentially affected by the proposed containment barrier installation, GE will coordinate the restoration/enhancement activities with the activities to be implemented as part of the response actions for the first ½ mile of the Housatonic River.

The primary component of the final lower bank restoration activity involves the placement of stone rip-rap (6- to 18-inch diameter) between the existing edge of the river and the proposed containment barrier. This installation -- which will be similar in design and aesthetics to the recently completed installation for the Building 68 containment barrier and the East Street Area 2 Source Control containment barrier, located approximately ¼-mile upriver -- has been selected for a number of reasons, including the ability to re-establish the relatively steep topography in this section of the river with an adequate structural capacity, the ability of the rip-rap material to withstand future river hydraulics, and the ability to provide diversity to the existing wetlands/riverbank habitat.

Modeling of river hydraulics for this section of the Housatonic River (done in association with the ½ Mile Work Plan) indicates that rip-rap with a minimum dimension of approximately 6 inches will be sufficient to withstand river flow events and velocities associated with a 25-year flood event in the range of 6 to 8 feet per second. In addition, the use of rip-rap in combination with the proposed containment barrier will provide structural stability of the riverbank in this area. From a structural perspective, the placement of the rip-rap materials will occur at an approximate 1:1 slope (i.e., the approximate angle of repose for these materials) to provide sufficient structural stability for the placed materials.

It is important to note that should the containment barrier installation activities be performed significantly prior to the first ½ mile river activities, those portions of this area which will later be subject to the first ½ mile river activities will be preliminarily restored, as necessary, to control erosion until final restoration measures can be implemented. Such erosion control will involve the temporary placement of geotextile(s), grass seed, and/or mulch over the disturbed area(s). Depending on the timing of the first ½ mile removal activities, such measures may not be needed, however, provided that the two programs are performed simultaneously.

2.3.7 Flood Storage

Restoration conditions associated with a 1:1 slope along the riverside of the proposed sheetpile wall have been evaluated to ascertain: 1) the location of the sheetpile wall necessary to result in an approximate 1:1 slope between the top of the wall (i.e., elevation of 977 or 978 feet) and the edge of the Housatonic River (i.e., considered to be at an average elevation of about 971.5 feet); and 2) the potential change in flood storage volume resulting from the proposed activities, incorporating re-alignment of portions of the sheetpile to achieve a 1:1 slope.

One component of the Wetlands Protection Act mentioned in Section 2.3.1 is to evaluate the potential changes in compensatory flood storage, if any, resulting from the proposed sheetpile installation. It is not expected that there will be a significant change in the existing flood storage capacity, but that a slight increase (i.e., gain) in current flood storage capacity may result from the proposed activities. GE has preliminarily evaluated changes in compensatory storage due to this project by developing several representative riverbank cross-sections under both existing and anticipated post-construction activities. These cross-sections (12 locations in total) were used to identify topographic changes between the existing and post-construction conditions, and subsequent calculations of lost/gained flood storage by depth increments. Table 2 summarizes the results of these calculations for each of the 12 cross-sections. Figures 6, 7, and 8 depict three typical cross sections used in this evaluation (see Figure 4 for locations of these cross sections).

Several assumptions were made as part of this evaluation: 1) both existing soil and rip-rap backfill were assumed to have similar porosities; 2) permeability differences between soil and rip-rap were ignored; 3) rip-rap will constitute the entire fill volume (i.e., triangular solids with a length of about 20 feet per transect, height of 5.5 to 6.5 feet, and base of 5.5 to 6.5 feet); and 4) potential flood storage capacity changes that may result from work in the first ½ mile of the Housatonic River stream bed have not been evaluated.

If it is assumed that the porosities and permeabilities of the existing soil and rip-rap backfill are both similar and inconsequential, assessing the change in flood storage capacity is reduced to a comparison of material present prior to, and following, removal and restoration operations. The resultant change in flood storage capacity is a gain of approximately 13 cy. Figures 6, 7, and 8 depict three cross sections typical of the minimal changes in grade along the proposed sheetpile wall.

The need for post-construction erosion control measures will be evaluated based on the condition of the bank at the end of construction and the expected timing for commencement of response actions in this area related to the first ½ mile of the Housatonic River. Since portions of the bank may be subject to response actions associated with the first ½ mile, backfilling, restoration, and erosion control activities will be limited to those that are necessary to control erosion until final measures can be implemented.

2.4 Future Monitoring Activities

The potentiometric conditions and NAPL recovery systems will continue to be evaluated based on future monitoring data acquired following installation of the containment barrier. The monitoring results will be provided in the monthly status reports. Future monitoring will be utilized to assess whether changes to the currently operating conditions are warranted. Groundwater monitoring will be done consistent with ongoing activities, with any modifications to be made with Agency approval. Monitoring wells LSSC-08S and LSSC-18 will be added to the current weekly monitoring program.

The physical condition of the riverbank will be monitored weekly following completion of the containment barrier construction and until the response actions for the first ½ mile of the Housatonic River commence in this area. Future monitoring of the bank will be incorporated into the long-term monitoring program for the first ½ mile of the Housatonic River.

Existing wells and well points along the riverbank will be protected to the extent practical during construction activities. It is anticipated that some of the well points may need to be abandoned prior to construction. Abandoned monitoring wells and well points will be replaced after final bank restoration activities are completed.

3. Summary and Anticipated Schedule

This Source Control Proposal describes the activities recently performed by GE related to the presence of NAPL in the vicinity of GE's existing recovery systems at the Lyman Street Site and the adjacent riverbank area. Included herein is an overview of field investigations conducted by GE to determine the presence and extent of NAPL within the subsurface soils along the riverbank in this area. The results of these recent investigations, combined with information available from prior investigations in this area, support the selection and design of a proposed NAPL containment barrier. Such a barrier, proposed to supplement the containment/recovery measures that are currently in place, will involve the installation of an approximately 400-foot long steel sheetpile wall located generally parallel to the river and within the lower portion of the riverbank. Both the length and depth of the proposed containment barrier have been conservatively selected to encompass areas where small quantities of separate phase and residual NAPL are potentially present. In addition, the overall design of the proposed barrier has been determined in consideration of future response actions related to the first ½ mile of Housatonic River bank soils and river sediments (i.e., between Newell Street and Lyman Street).

It is expected that the construction of the proposed containment barrier will be performed in 2000, in conjunction with the first ½ mile removal actions. GE will provide a more detailed project schedule following the Agencies review and approval of this Source Control Proposal and the First ½ mile Work Plan submitted on June 25, 1999.

Tables

BLASLAND, BOUCK & LEE, INC.
engineers & scientists

TABLE 1

GENERAL ELECTRIC COMPANY - PITTSFIELD, MASSACHUSETTS

SOURCE CONTROL MEASURES FOR LYMAN STREET SITE

SUMMARY OF RIVERBANK SOIL DATA - APRIL 1999

Sample Location	Sample Date	Sample Depth	Sample Elevation (Feet AMSL)	Field Observations/Testing			Analytical Results (ppm)			
				Sample PID Reading (Instrument Units)	Staining Observed on Soil Core	Shake Test Results	Aroclor 1254	Aroclor 1260	Total PCBs	TPH
LSSC-20	04/02/99	0-1'	974.05 - 973.05	0.0	No	No	N/A	N/A	N/A	N/A
LSSC-20	04/02/99	1-2'	973.05 - 972.05	0.0	No	No	N/A	N/A	N/A	N/A
LSSC-20	04/02/99	2-3'	972.05 - 971.05	N/A	No	N/A	ND	77	77	391
LSSC-20	04/02/99	3-4'	971.05 - 970.05	0.0	No	No	ND	0.58	0.58	ND(55.1)
LSSC-20	04/02/99	4-5'	970.05 - 969.05	N/A	No	Trace Sheen	ND	0.41	0.41	ND(60)
LSSC-20	04/02/99	5-6'	969.05 - 968.05	N/A	No	N/A	ND	0.11	0.11	ND(54.9)
LSSC-20	04/02/99	6-7'	968.05 - 967.05	N/A	No	N/A	ND	0.25	0.25	ND(58.2)
LSSC-20	04/02/99	7-8'	967.05 - 966.05	N/A	No	N/A	0.15	ND	0.15	ND(54.2)
LSSC-20	04/02/99	8-9'	966.05 - 965.05	N/A	No	N/A	1.5	ND	1.5	ND(47.1)
LSSC-20	04/02/99	9-10'	965.05 - 964.05	N/A	No	N/A	N/A	N/A	N/A	N/A
LSSC-21	04/05/99	0-1'	974.48 - 973.48	16.9	No	No	ND	15	15	74.0 [ND (40)]
LSSC-21	04/05/99	1-2'	973.48 - 972.48	16.9	No	No	140	ND	140	263 [46,000]
LSSC-21	04/05/99	2-3'	972.48 - 971.48	15.8	No	No	5,600	ND	5,600	23,500
LSSC-21	04/05/99	3-4'	971.48 - 970.48	39.0	Yes (begins at 3.2')	Yes	5,600 [ND]	ND [47]	5,600 [47]	58,100
LSSC-21	04/05/99	4-5'	970.48 - 969.48	21.8	Yes	No	30	ND	30	337
LSSC-21	04/05/99	5-6'	969.48 - 968.48	28.6	Yes	Yes	39	ND	39	723
LSSC-21	04/05/99	6-7'	968.48 - 967.48	25.3	Yes	Trace Sheen	8.3	ND	8.3	93.3 [ND (46.6)]
LSSC-21	04/05/99	7-8'	967.48 - 966.48	16.1	Yes	No	3.4	ND	3.4	50.6
LSSC-21	04/05/99	8-9'	966.48 - 965.48	17.3	No	No	2.9	ND	2.9	65.6
LSSC-21	04/05/99	9-10'	965.48 - 964.48	17.4	No	No	5.9 [1.8]	ND [ND]	5.9 [1.8]	58.1
LSSC-22	04/01/99	0-1'	974.43 - 973.43	N/A	No	N/A	ND	2.3	2.3	47 [76]
LSSC-22	04/01/99	1-2'	973.43 - 972.43	3.8	No	No	ND	2.5	2.5	45
LSSC-22	04/01/99	2-3'	972.43 - 971.43	N/A	No	N/A	ND [ND]	180 [160]	180 [160]	120
LSSC-22	04/01/99	3-4'	971.43 - 970.43	18.3	No	Yes	ND	150	150	1,900
LSSC-22	04/01/99	4-5'	970.43 - 969.43	81.2	Yes	Yes	180	ND	180	6,200
LSSC-22	04/01/99	5-6'	969.43 - 968.43	31.8	Yes	Yes	ND	40	40	3,500
LSSC-22	04/01/99	6-7'	968.43 - 967.43	9.3	Yes	No	0.31	ND	0.31	ND (40)
LSSC-22	04/01/99	7-8'	967.43 - 966.43	5.2	Yes (ends at 7.6')	No	ND	0.15	0.15	ND (40)
LSSC-22	04/01/99	8-9'	966.43 - 965.43	7.4	No	No	1.2	ND	1.2	ND (40)
LSSC-22	04/01/99	9-10'	965.43 - 964.43	6.4	No	No	0.028 J	ND	0.028 J	ND (40)

See notes on page 4.

TABLE 1 (Continued)

GENERAL ELECTRIC COMPANY - PITTSFIELD, MASSACHUSETTS

SOURCE CONTROL MEASURES FOR LYMAN STREET SITE

SUMMARY OF RIVERBANK SOIL DATA - APRIL 1999

Sample Location	Sample Date	Sample Depth	Sample Elevation (Feet AMSL)	Field Observations/Testing			Analytical Results (ppm)			
				Sample PID Reading (Instrument Units)	Staining Observed on Soil Core	Shake Test Results	Aroclor 1254	Aroclor 1260	Total PCBs	TPH
LSSC-23	04/05/99	0-1'	973.14 - 972.14	N/A	No	N/A	24	ND	24	88.6
LSSC-23	04/05/99	1-2'	972.14 - 971.14	19.3	No	No	800	ND	800	9,620
LSSC-23	04/05/99	2-3'	971.14 - 970.14	18.7	Yes (begins at 2.5')	Yes	5,200	ND	5,200	48,800
LSSC-23	04/05/99	3-4'	970.14 - 969.14	23.6	Yes	Yes	39	ND	39	1,000
LSSC-23	04/05/99	4-5'	969.14 - 968.14	17.4	Yes	Trace Sheen	28	ND	28	772
LSSC-23	04/05/99	5-6'	968.14 - 967.14	N/A	Yes	N/A	1.4	ND	1.4	ND (46.6)
LSSC-23	04/05/99	6-7'	967.14 - 966.14	17.5	Yes	No	0.24	ND	0.24	ND (45.6)
LSSC-23	04/05/99	7-8'	966.14 - 965.14	17.6	No	No	0.36	ND	0.36	ND (49.9)
LSSC-23	04/05/99	8-9'	965.14 - 964.14	15.6	No	No	0.22	ND	0.22	ND (44.5)
LSSC-23	04/05/99	9-10'	964.14 - 963.14	15.8	No	No	N/A	N/A	N/A	N/A
LSSC-24	04/05/99	0-1'	973.35 - 972.35	3.3	No	No	ND [ND]	3.2 [0.77]	3.2 [0.77]	102
LSSC-24	04/05/99	1-2'	972.35 - 971.35	3.8	No	No	ND	44	44	392 [ND (55.3)]
LSSC-24	04/05/99	2-3'	971.35 - 970.35	3.4	No	No	ND	37	37	2,920
LSSC-24	04/05/99	3-4'	970.35 - 969.35	18.5	No	No	ND	0.78	0.78	ND (55.1)
LSSC-24	04/05/99	4-5'	969.35 - 968.35	6.1	No	No	ND	0.12	0.12	ND (63.8)
LSSC-24	04/05/99	5-6'	968.35 - 967.35	8.7	No	No	ND (0.043)	ND (0.043)	ND (0.043)	ND (52.2)
LSSC-24	04/05/99	6-7'	967.35 - 966.35	3.5	No	No	ND (0.038)	ND (0.038)	ND (0.038)	ND (46.3)
LSSC-24	04/05/99	7-8'	966.35 - 965.35	N/A	No	No	0.96	ND	0.96	ND (42.6)
LSSC-24	04/05/99	8-9'	965.35 - 964.35	3.7	No	No	ND	0.6	0.6	ND (50.8)
LSSC-24	04/05/99	9-10'	964.35 - 963.35	3.8	No	No	N/A	N/A	N/A	N/A
LSSC-25	04/01/99	0-1'	974.01 - 973.01	N/A	No	N/A	ND	2.0	2.0	51
LSSC-25	04/01/99	1-2'	973.01 - 972.01	N/A	No	N/A	ND	19	19	60
LSSC-25	04/01/99	2-3'	972.01 - 971.01	1.6	No	No	ND	0.22	0.22	ND (40)
LSSC-25	04/01/99	3-4'	971.01 - 970.01	2.7	No	No	ND (0.042)	ND (0.042)	ND (0.042)	ND (40)
LSSC-25	04/01/99	4-5'	970.01 - 969.01	2.4	No	No	ND (0.045)	ND (0.045)	ND (0.045)	ND (40)
LSSC-25	04/01/99	5-6'	969.01 - 968.01	N/A	No	N/A	ND	0.067	0.067	ND (40)
LSSC-25	04/01/99	6-7'	968.01 - 967.01	2.5	No	No	0.046	ND	0.046	ND (40)
LSSC-25	04/01/99	7-8'	967.01 - 966.01	2.9	No	No	ND (0.034)	ND (0.034)	ND (0.034)	ND (40)
LSSC-25	04/01/99	8-9'	966.01 - 965.01	3.6	No	No	0.031 J	ND	0.031 J	ND (40)
LSSC-25	04/01/99	9-10'	965.01 - 964.01	3.2	No	No	ND (0.04)	ND (0.04)	ND (0.04)	ND (40)

See notes on page 4.

TABLE 1 (Continued)

GENERAL ELECTRIC COMPANY - PITTSFIELD, MASSACHUSETTS

SOURCE CONTROL MEASURES FOR LYMAN STREET SITE

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Sample Location	Sample Date	Sample Depth	Sample Elevation (Feet AMSL)	Field Observations/Testing			Analytical Results (ppm)			
				Sample PID Reading (Instrument Units)	Staining Observed on Soil Core	Shake Test Results	Aroclor 1254	Aroclor 1260	Total PCBs	TPH
LSSC-27	04/01/99	0-1'	980.97 - 979.97	7.0	No	No	700	ND	700	202
LSSC-27	04/01/99	1-2'	979.97 - 978.97	1.8	No	No	18	ND	18	65
LSSC-27	04/01/99	2-3'	978.97 - 977.97	1.8	No	No	1,200	ND	1,200	654
LSSC-27	04/01/99	3-4'	977.97 - 976.97	1.9	No	No	5,200	ND	5,200	1,960
LSSC-27	04/01/99	4-5'	976.97 - 975.97	1.8	No	No	ND	6.2	6.2	138
LSSC-27	04/01/99	5-6'	975.97 - 974.97	N/A	No	N/A	3.6	ND	3.6	94.8
LSSC-27	04/01/99	6-7'	974.97 - 973.97	1.5	No	No	11	ND	11	148
LSSC-27	04/01/99	7-8'	973.97 - 972.97	N/A	No	N/A	9.9	ND	9.9	196
LSSC-27	04/01/99	8-9'	972.97 - 971.97	1.6	No	No	6.6	ND	6.6	152
LSSC-27	04/01/99	9-10'	971.97 - 970.97	N/A	No	N/A	3.0	ND	3.0	96.1
LSSC-27	04/01/99	10-11'	970.97 - 969.97	1.8	No	No	1.2	ND	1.2	65.4
LSSC-27	04/01/99	11-12'	969.97 - 968.97	2.8	No	No	1.7	ND	1.7	144
LSSC-27	04/01/99	12-13'	968.97 - 967.97	N/A	No	N/A	ND	1.3	1.3	80.2
LSSC-27	04/01/99	13-14'	967.97 - 966.97	8.1	No	Trace Sheen	ND	0.67	0.67	51.1
LSSC-27	04/01/99	14-15'	966.97 - 965.97	22.9	No	Trace Sheen	ND	1.1	1.1	98
LSSC-27	04/01/99	15-16'	965.97 - 964.97	2.7	No	No	0.019 J	ND	0.019 J	ND (44.9)
LSSC-27	04/01/99	16-17'	964.97 - 963.97	2.0	No	No	0.022 J	ND	0.022 J	ND (47.8)
LSSC-27	04/01/99	17-18'	963.97 - 962.97	2.1	No	No	ND (0.037)	ND (0.037)	ND (0.037)	ND (45.2)
LSSC-28	04/01/99	0-1'	977.81 - 976.81	N/A	No	N/A	N/A	N/A	N/A	N/A
LSSC-28	04/01/99	1-2'	976.81 - 975.81	N/A	No	N/A	N/A	N/A	N/A	N/A
LSSC-28	04/01/99	2-3'	975.81 - 974.81	N/A	No	N/A	N/A	N/A	N/A	N/A
LSSC-28	04/01/99	3-4'	974.81 - 973.81	2.5	No	N/A	25	ND	25	323
LSSC-28	04/01/99	4-5'	973.81 - 972.81	2.5	No	No	14	ND	14	229
LSSC-28	04/01/99	5-6'	972.81 - 971.81	N/A	No	N/A	12	ND	12	128
LSSC-28	04/01/99	6-7'	971.81 - 970.81	2.5	No	No	ND	5.0	5.0	222
LSSC-28	04/01/99	6-7'	971.81 - 970.81	N/A	No	N/A	ND [ND]	35 [31]	35 [31]	636
LSSC-28	04/01/99	7-8'	970.81 - 969.81	46.8	Yes	Yes	ND	33	33	7,560
LSSC-28	04/01/99	8-9'	969.81 - 968.81	34.1	Yes	Trace Sheen	ND	20	20	4,080
LSSC-28	04/01/99	9-10'	968.81 - 967.81	23.1	Yes	Trace Sheen	ND	2.8	2.8	100
LSSC-28	04/01/99	10-11'	967.81 - 966.81	30.3	Yes (ends at 10.5')	No	ND	0.47	0.47	339
LSSC-28	04/01/99	11-12'	966.81 - 965.81	26.0	No	Trace Sheen	ND (0.05)	ND (0.05)	ND (0.05)	63.4
LSSC-28	04/01/99	12-13'	965.81 - 964.81	N/A	No	N/A	1.9	ND	1.9	265 [140]
LSSC-28	04/01/99	13-14'	964.81 - 963.81	9.5	No	No	ND	0.14	0.14	ND (44.6)
LSSC-28	04/01/99	14-15'	963.81 - 962.81	4.1	No	No	ND (0.044)	ND (0.044)	ND (0.044)	ND (44.7)
LSSC-28	04/01/99	15-16'	962.81 - 961.81	N/A	No	N/A	N/A	N/A	N/A	N/A

See notes on page 4.

TABLE 1 (Continued)

GENERAL ELECTRIC COMPANY - PITTSFIELD, MASSACHUSETTS

SOURCE CONTROL MEASURES FOR LYMAN STREET SITE

SUMMARY OF RIVERBANK SOIL DATA - APRIL 1999

Sample Location	Sample Date	Sample Depth	Sample Elevation (Feet AMSL)	Field Observations/Testing			Analytical Results (ppm)			
				Sample PID Reading (Instrument Units)	Staining Observed on Soil Core	Shake Test Results	Aroclor 1254	Aroclor 1260	Total PCBs	TPH
LSSC-29	04/05/99	0-1'	973.00 - 972.00	3.2	No	No	N/A	N/A	N/A	N/A
LSSC-29	04/05/99	1-2'	972.00 - 971.00	3.5	No	No	N/A	N/A	N/A	N/A
LSSC-29	04/05/99	2-3'	971.00 - 970.00	N/A	No	No	ND	120	120	783
LSSC-29	04/05/99	3-4'	970.00 - 969.00	N/A	No	No	ND	6.2	6.2	45.5
LSSC-29	04/05/99	4-5'	969.00 - 968.00	15.5	Yes	No	ND	7.0	7.0	1,800
LSSC-29	04/05/99	5-6'	968.00 - 967.00	7.3	Yes	No	ND	0.53	0.53	60.3
LSSC-29	04/05/99	6-7'	967.00 - 966.00	5.6	No	No	ND	0.12	0.12	ND (44.7)
LSSC-29	04/05/99	7-8'	966.00 - 965.00	4.2	No	No	ND	0.064	0.064	ND (44.8)
LSSC-29	04/05/99	8-9'	965.00 - 964.00	3.2	No	No	ND	0.041	0.041	ND (45.3)
LSSC-29	04/05/99	9-10'	964.00 - 963.00	3.5	No	No	N/A	N/A	N/A	N/A
LSSC-30	04/02/99	0-1'	973.32 - 972.32	N/A	No	N/A	ND	11	11	ND (59)
LSSC-30	04/02/99	1-2'	972.32 - 971.32	N/A	No	N/A	ND	54	54	89.6
LSSC-30	04/02/99	2-3'	971.32 - 970.32	N/A	No	N/A	ND	6.6	6.6	252
LSSC-30	04/02/99	3-4'	970.32 - 969.32	N/A	No	N/A	ND	3.8	3.8	84
LSSC-30	04/02/99	4-5'	969.32 - 968.32	N/A	No	N/A	ND [ND]	3.8 [1.6]	3.8 [1.6]	67.8
LSSC-30	04/02/99	5-6'	968.32 - 967.32	N/A	No	N/A	ND	0.18	0.18	ND (46.1)
LSSC-30	04/02/99	6-7'	967.32 - 966.32	11.7	No	No	0.083	ND	0.083	ND (45.3)
LSSC-30	04/02/99	7-8'	966.32 - 965.32	10.3	No	No	0.19	ND	0.19	ND (44.4)
LSSC-30	04/02/99	8-9'	965.32 - 964.32	9.4	No	No	ND	0.45	0.45	ND (45.0)
LSSC-30	04/02/99	9-10'	964.32 - 963.32	N/A	No	N/A	N/A	N/A	N/A	N/A

Notes:

1. Samples were collected and screened in the field with a photoionization detector (PID) by Blasland, Bouck & Lee, Inc. (BBL).
2. Water shake tests were performed by BBL on all samples to evaluate the potential presence of LNAPL residuals.
   "No" indicates that no LNAPL residuals were observed.
   "Yes" indicates that LNAPL residuals were observed, or a moderate to strong sheen formed on the water surface during the test.
   "Trace Sheen" indicates that a slight sheen formed on the water surface during the test.
3. Samples were submitted to CT & E Environmental Services, Inc., for analysis of PCBs by EPA Method 8082 and Total Petroleum Hydrocarbons (TPH) by EPA Method 418.1.
4. ppm: Dry weight parts per million.
5. Duplicate sample results are shown in brackets [ ].
6. ND: Not detected (Practical Quantitation Limit shown in parantheses).
7. N/A: Not analyzed - sample not submitted to laboratory or insufficient volume for field analyses.
8. J: Indicates an estimated value less than the Practical Quantitation Limit.
9. Feet AMSL: Feet above mean sea level.
10. The boring designation of LSSC-26 was not utilized.
11. LNAPL: Light Non-Aqueous Phase Liquid

TABLE 2

GENERAL ELECTRIC COMPANY
PITTSFIELD, MASSACHUSETTS

SOURCE CONTROL MEASURES FOR LYMAN STREET SITE

CALCULATION OF CHANGES IN FLOOD STORAGE

Transect	Length of Transect (ft)	Change in Flood Storage (ft²) Within Elevations:								Total (ft²)
		971.5 - 972	972 - 973	973 - 974	974 - 975	975 - 976	976 - 977	977 - 978	Greater than 978	
1	16	0.0	0.3	0.9	1.3	0.6	0.2	0.0	(0.4)	2.8
2	20	0.0	0.0	(0.3)	(0.8)	(0.5)	(3.3)	(5.3)	(3.0)	(13.0)
3*	20	0.0	(0.1)	(0.3)	(0.5)	(2.0)	(4.0)	(5.3)	(2.8)	(14.9)
4	20	0.0	0.3	0.6	0.8	(0.6)	(2.8)	(5.0)	(3.3)	(10.0)
5	20	0.0	0.3	0.8	1.0	1.3	1.8	1.8	2.0	8.8
6	20	0.0	0.3	1.1	2.0	3.0	3.8	4.4	13.3	27.8
7*	20	0.1	0.6	1.3	1.8	2.5	3.1	3.5	5.3	18.1
8	20	0.0	0.4	0.8	1.1	1.4	1.5	1.8	2.5	9.4
9	20	(0.1)	(0.4)	(0.8)	(1.3)	(1.8)	(2.3)	(1.5)	(0.1)	(8.1)
10	20	0.0	0.0	0.0	0.0	0.0	0.0	(0.5)	0.0	(0.5)
11*	20	0.0	(0.1)	(0.3)	(0.8)	(1.3)	(2.5)	(1.3)	0.0	(6.1)
12	20	0.0	0.3	0.8	1.0	1.4	0.0	0.5	0.0	3.9
	Total:	0.0	0.3	38.1	46.8	33.0	(35.9)	(56.0)	109.7	18.1

Transect	Length of Transect (ft)	Change in Flood Storage (cy) Within Elevations:								Change in Flood Storage (cy)
		971.5 - 972	972 - 973	973 - 974	974 - 975	975 - 976	976 - 977	977 - 978	Greater than 978	
1	16	0.0	0.1	0.5	0.7	0.4	0.1	0.0	(0.2)	1.7
2	20	0.0	0.0	(0.2)	(0.6)	(0.4)	(2.4)	(3.9)	(2.2)	(9.6)
3*	20	0.0	(0.1)	(0.2)	(0.4)	(1.5)	(3.0)	(3.9)	(2.0)	(11.0)
4	20	0.0	0.2	0.5	0.6	(0.5)	(2.0)	(3.7)	(2.4)	(7.4)
5	20	0.0	0.2	0.6	0.7	0.9	1.3	1.3	1.5	6.5
6	20	0.0	0.2	0.8	1.5	2.2	2.8	3.2	9.8	20.6
7*	20	0.1	0.4	0.9	1.3	1.9	2.3	2.6	3.9	13.4
8	20	0.0	0.3	0.6	0.8	1.0	1.1	1.3	1.9	6.9
9	20	(0.1)	(0.3)	(0.6)	(0.9)	(1.3)	(1.7)	(1.1)	(0.1)	(6.0)
10	20	0.0	0.0	0.0	0.0	0.0	0.0	(0.4)	0.0	(0.4)
11*	20	0.0	(0.1)	(0.2)	(0.6)	(0.9)	(1.9)	(0.9)	0.0	(4.5)
12	20	0.0	0.2	0.6	0.7	1.0	0.0	0.4	0.0	2.9
	Total:	0.0	1.2	3.3	4.0	2.9	(3.3)	(5.1)	10.1	13.0

Notes:

1. Area and volume estimates represent total estimated excavations associated with both the Source Control and the First 1/2 Mile River activities.

2. * - Transects 11, 7, and 3 are illustrated on Figures 6, 7, and 8, respectively.

Figures

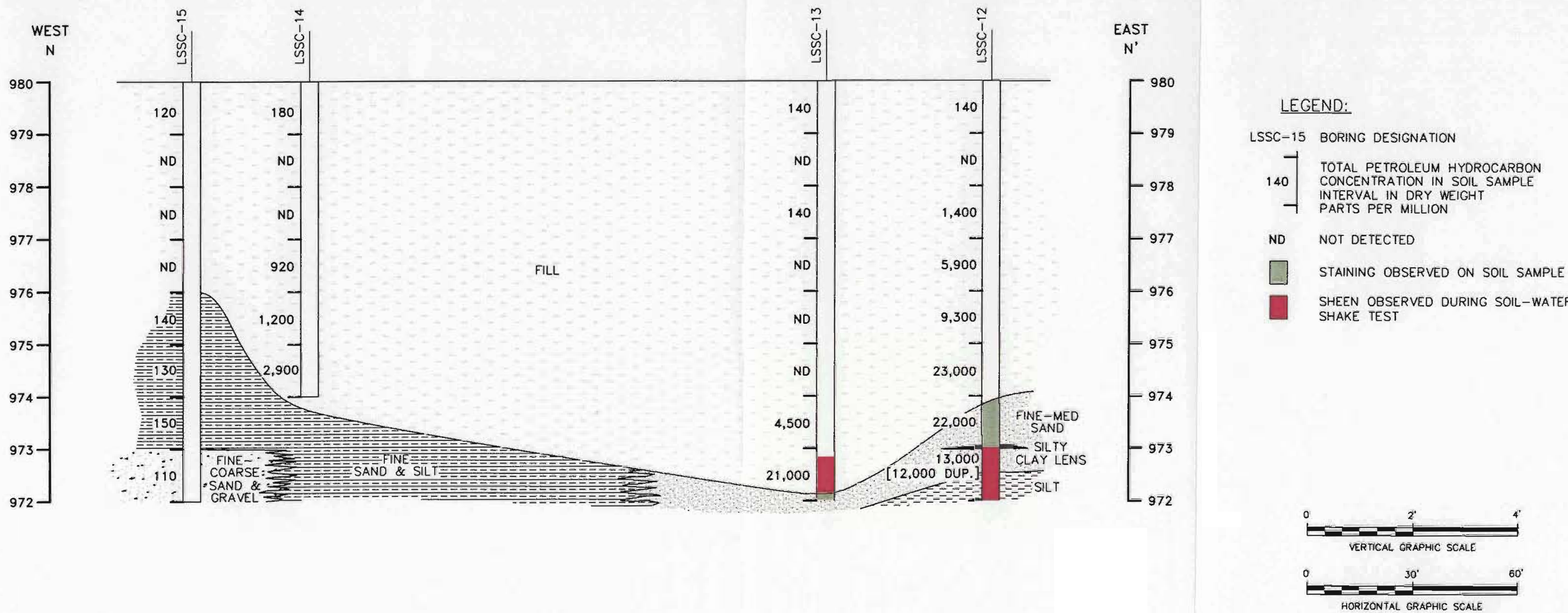
BLASLAND, BOUCK & LEE, INC.
engineers & scientists

GENERAL ELECTRIC COMPANY
PITTSFIELD, MASSACHUSETTS
LYMAN STREET SITE
SOURCE CONTROL PROPOSAL

SITE PLAN

BBL BLASLAND, BOUCK & LEE, INC. *engineers & scientists*

FIGURE 1



GENERAL ELECTRIC COMPANY
PITTSFIELD, MASSACHUSETTS

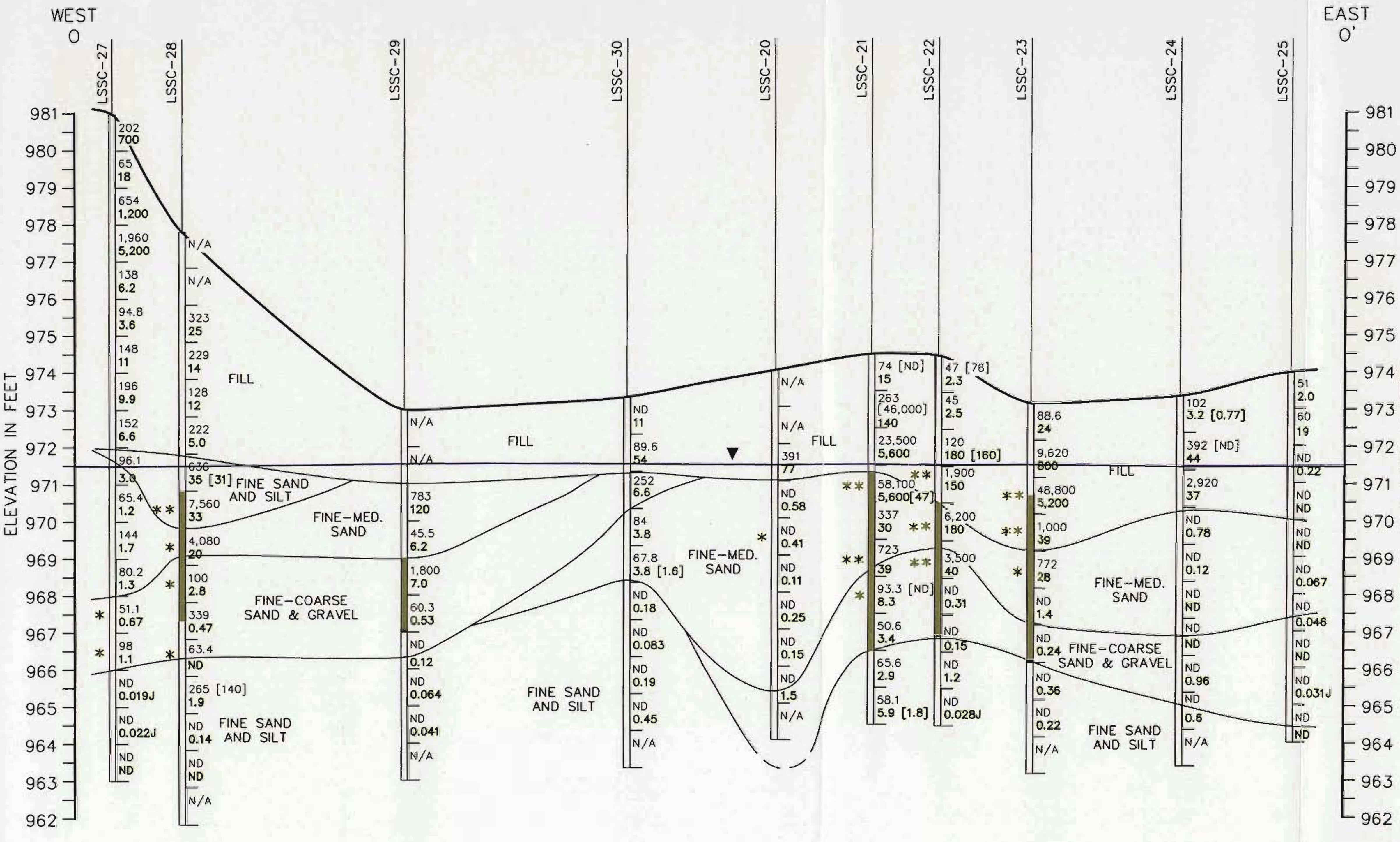
LYMAN STREET SITE
SOURCE CONTROL PROPOSAL

CROSS SECTION N-N'

BBL BLASLAND, BOUCK & LEE, INC. engineers & scientists

FIGURE 2

L: ON=*; OFF=*REF
P: STD/PCP-BL
7/12/99 SYR-54-RCA YCC
20182002/20182V02.DWG



LEGEND:

LSSC-25	BORING DESIGNATION
60 / 19	SOIL SAMPLE INTERVAL: TOTAL PETROLEUM HYDROCARBON (TPH (IN BLACK]) AND TOTAL POLYCHLORINATED BIPHENYL (PCB [IN GREEN]) CONCENTRATIONS IN DRY WEIGHT PARTS PER MILLION
ND	NOT DETECTED
[ND]	DUPLICATE SAMPLE RESULTS
N/A	NOT ANALYZED
0.031J	ESTIMATED VALUE
*	TRACE SHEEN OBSERVED DURING SOIL-WATER SHAKE TEST
**	LNAPL RESIDUAL OR MODERATE-STRONG SHEEN OBSERVED DURING SOIL-WATER SHAKE TEST
▮	STAINING/SHEEN OBSERVED ON SOIL CORE
▼	APPROXIMATE WATER TABLE ELEVATION (BASED ON HISTORICAL MONITORING DATA)

0 3' 6'
VERTICAL GRAPHIC SCALE

0 30' 60'
HORIZONTAL GRAPHIC SCALE

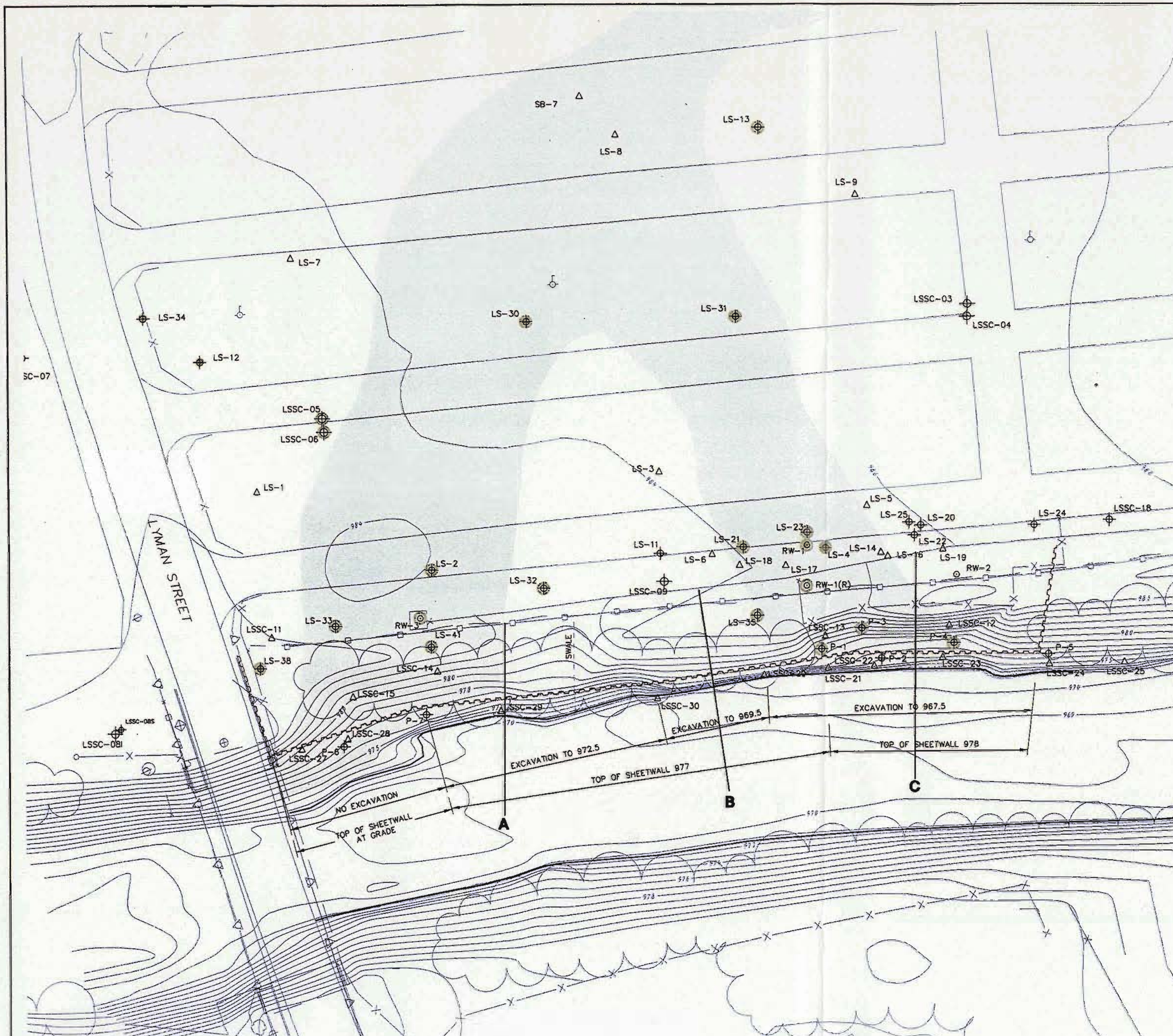
GENERAL ELECTRIC COMPANY
PITTSFIELD, MASSACHUSETTS
LYMAN STREET SITE
SOURCE CONTROL PROPOSAL

CROSS SECTION O-O'

BBL BLASLAND, BOUCK & LEE, INC. engineers & scientists

FIGURE 3

L: ON=*, OFF=REF*
P: STD-PCP/BL
6/28/99 SYR-54 RCA YCC
20182002/20182V01.DWG



X: 20182X1A, 20182X1B
L: ON=*, OFF=*REF*
P: 801-D, 801-D2B
7/13/99 SYR-54-RDM YCC
20182001/LYMAN/20182801.DWG

LEGEND:

- APPROXIMATE MEAN RIVER ELEVATION AND MEAN GROUNDWATER TABLE ELEVATION (971.5)
- APPROXIMATE LOCATION OF PROPOSED CONTAINMENT BARRIER
- A — CONCEPTUAL CROSS-SECTION LOCATION
- EXISTING INDEX ELEVATION CONTOUR
- EXISTING INTERMEDIATE ELEVATION CONTOUR
- DECIDUOUS TREE
- CONIFEROUS TREE
- MANHOLE
- CHAIN LINK FENCE
- POLE (NON-UTILITY)
- POLE (OVERHEAD UTILITY)
- APPROXIMATE DELINEATION OF FORMER OXBOWS
- ES2-1 EXISTING MONITORING WELL
- RW-3 EXISTING PUMPING WELL
- LS-1 EXISTING SOIL BORING
- LNAPL OBSERVED
- DNAPL OBSERVED

NOTES:

1. MAPPING IS BEST AVAILABLE INFORMATION AS OF 12/10/98 BASED ON MAPPING PROVIDED BY LOCKWOOD MAPPING, INC. PREPARED FROM 1990 AERIAL PHOTOGRAPHY; DATA PROVIDED BY GENERAL ELECTRIC; AND BLASLAND AND BOUCK ENGINEERS, PC. CONSTRUCTION PLANS. RIVERBANK AND RIVER BED TOPOGRAPHIC INFORMATION PROVIDED BY BBL FROM OCTOBER 12-23, 1998 FIELD SURVEY.
2. COORDINATE GRID BASED ON 1927 STATE PLANE COORDINATES.
3. ELEVATION DATUM REFERENCED TO NGVD 1929.
4. ALL SAMPLING LOCATIONS ARE APPROXIMATE.
5. LNAPL = LIGHT NON-AQUEOUS PHASE LIQUIDS
DNAPL = DENSE NON-AQUEOUS PHASE LIQUID.

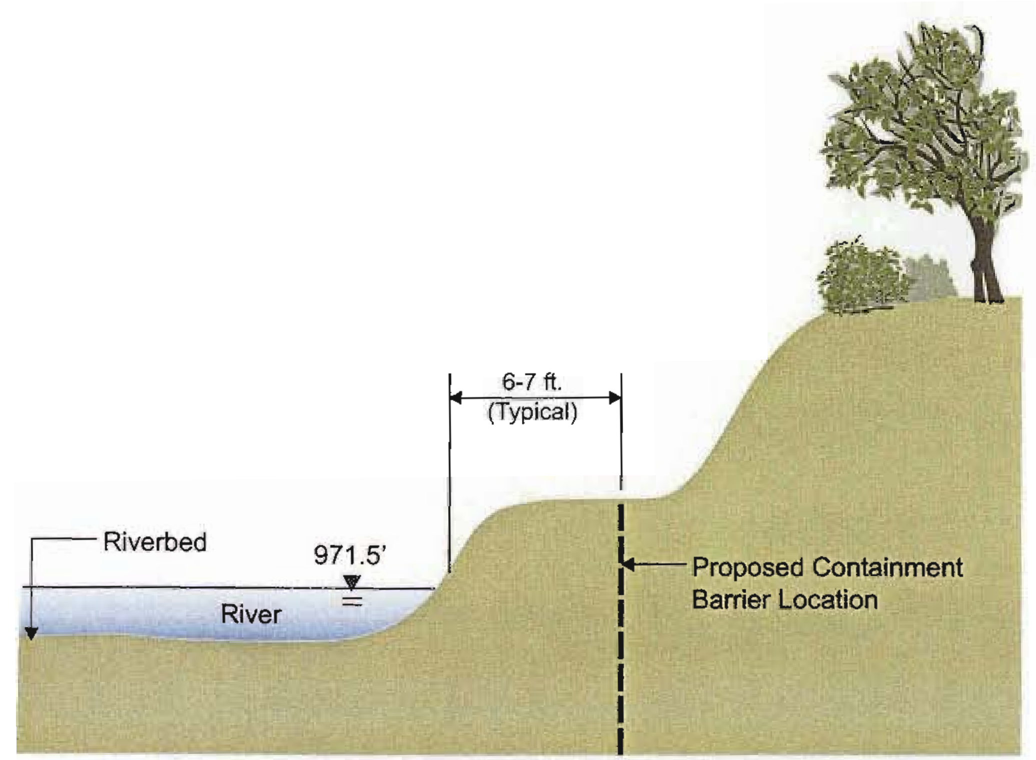
0 20' 40'
GRAPHIC SCALE

GENERAL ELECTRIC COMPANY
PITTSFIELD, MASSACHUSETTS
LYMAN STREET SITE
SOURCE CONTROL PROPOSAL

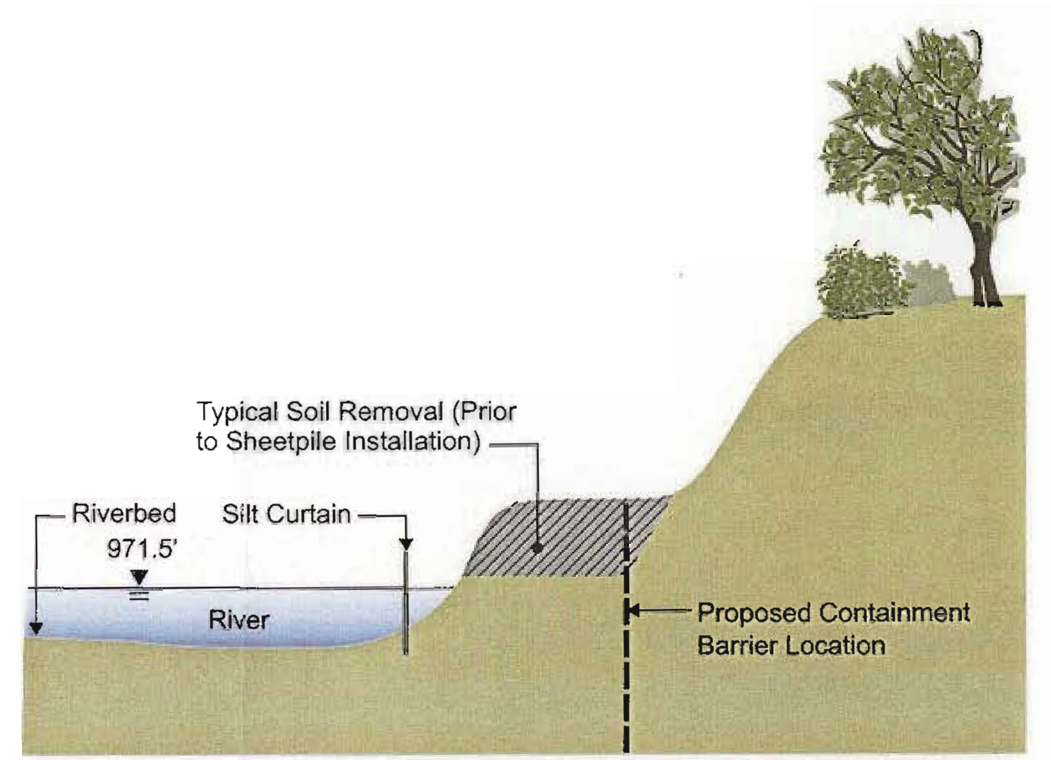
PROPOSED CONTAINMENT BARRIER LOCATION

BBL BLASLAND, BOUCK & LEE, INC.
engineers & scientists

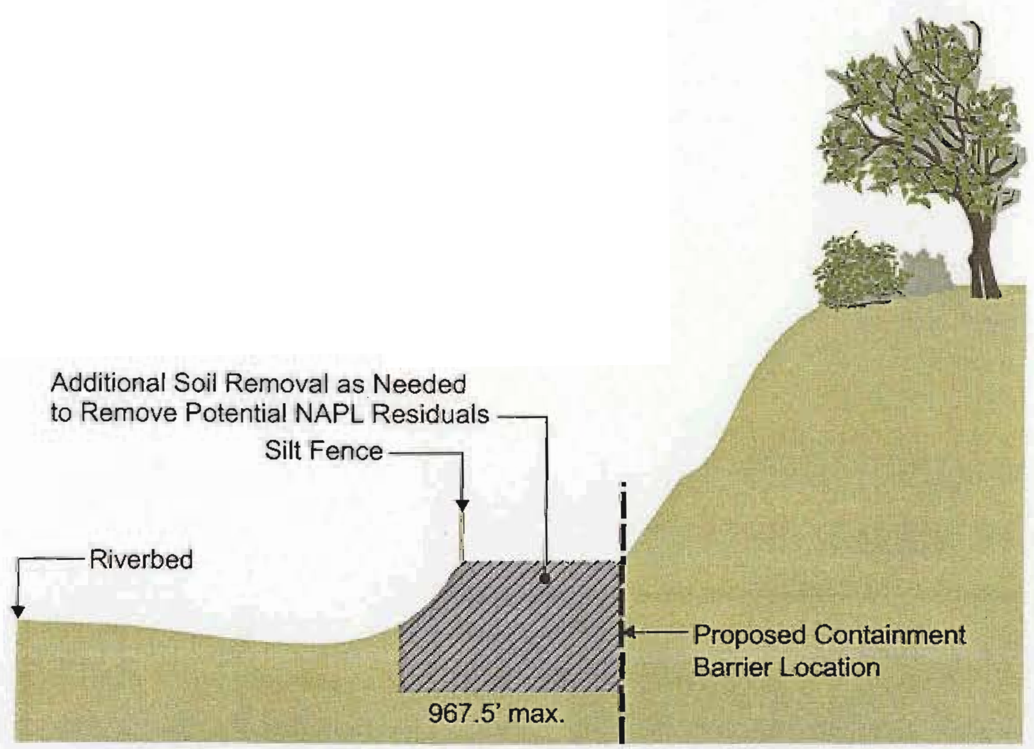
FIGURE 4
REVISED



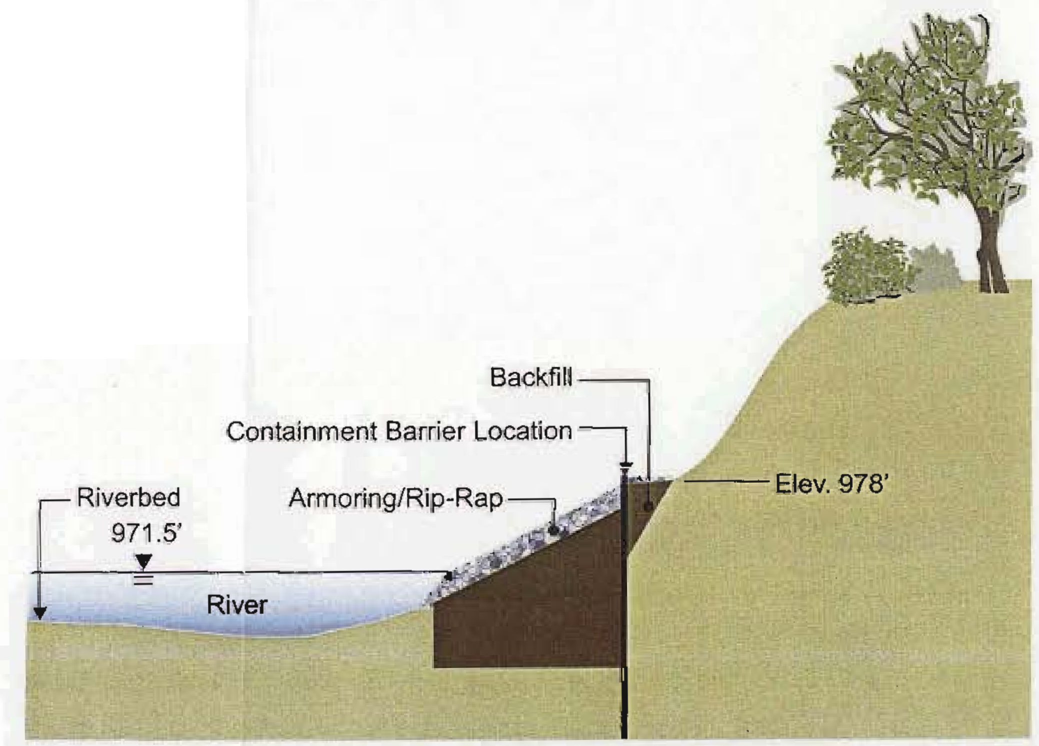
EXISTING CONDITIONS



INITIAL SOIL REMOVAL AND CONTAINMENT BARRIER INSTALLATION



DEEPER ADDITIONAL SOIL REMOVAL



RESTORATION WITH RIPRAP

NOTES:

1. Final restoration to be completed after sediment/bank removal activities associated with first ½ mile removal action are completed.
2. Extent of bank soil/sediment removal (and related restoration activities) varies as detailed on Figure 4 included with this proposal.
3. Typical river elevation is based on historical monitoring data.
4. Deeper soil removal will be done concurrent with sediment removal performed as part of first ½ mile removal action.

NOT-TO-SCALE

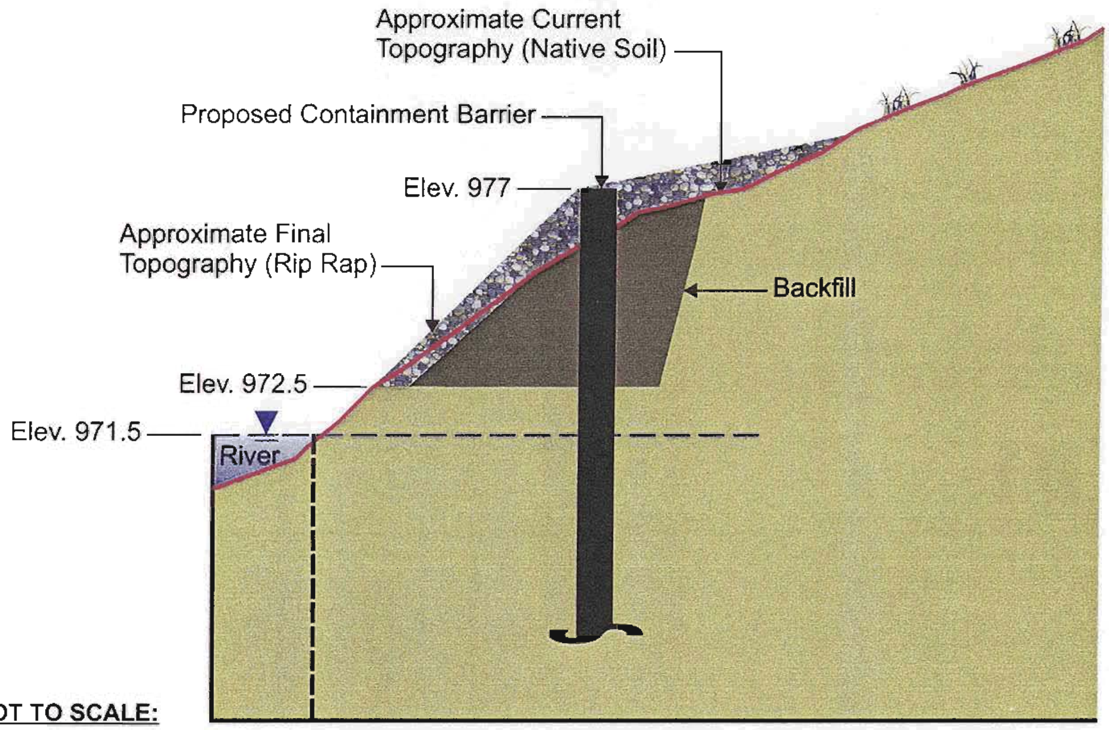
GENERAL ELECTRIC COMPANY
PITTSFIELD, MASSACHUSETTS
LYMAN STREET AREA
SUPPLEMENTAL SOURCE CONTROL MEASURES

CONCEPTUAL CROSS-SECTION OF SOIL REMOVAL, BARRIER INSTALLATION AND RIVERBANK RESTORATION

BBL BLASLAND, BOUCK & LEE, INC. engineers & scientists

FIGURE 5

7/99 SYR-D54-DJH LBR YCC
20182001/20182g01.cdr



NOT TO SCALE:

	Approximate Changes in Flood Storage at Transect A						
Elevation Interval	**971.5 - 972**	**972 - 973**	**973 - 974**	**974 - 975**	**975 - 976**	**976 - 977**	**977 - 978**
Change Flood Storage (sf)	0	-0.1	-0.25	-0.75	-1.25	-2.5	-1.25
Total Change in Flood Storage at Transect A	-6.1 (sf)						

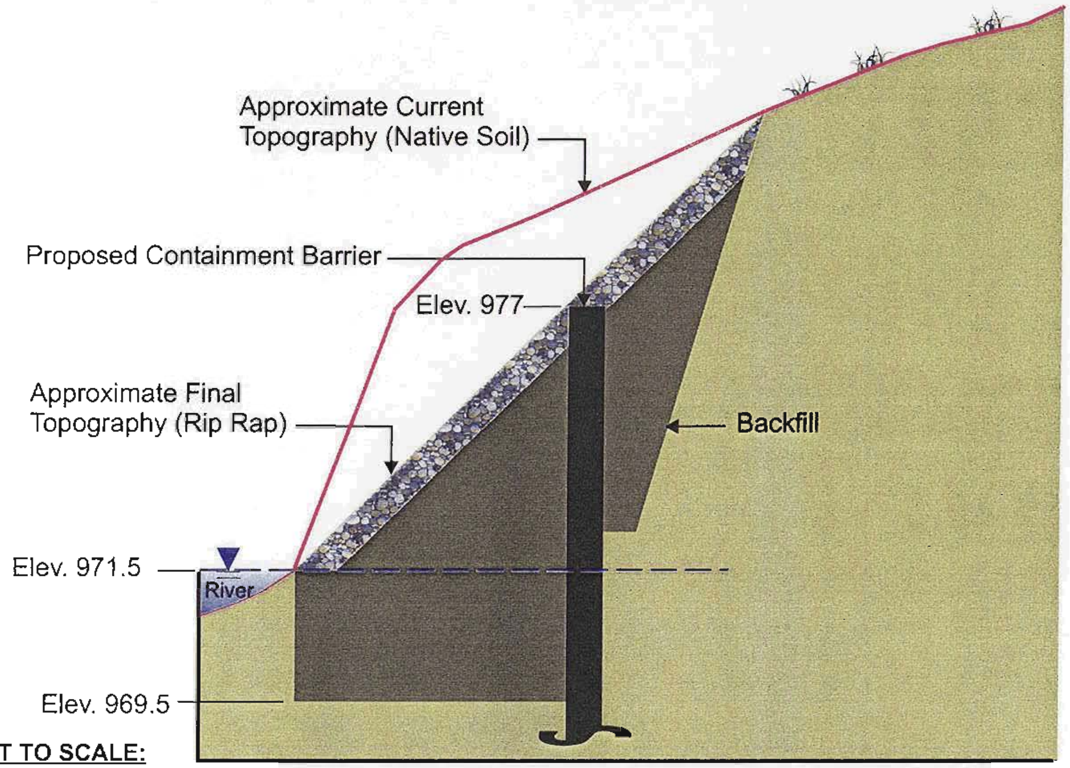
GENERAL ELECTRIC COMPANY
PITTSFIELD, MASSACHUSETTS
LYMAN STREET AREA
SUPPLEMENTAL SOURCE CONTROL MEASURES

CONCEPTUAL CROSS-SECTION
TRANSECT A

BBL BLASLAND, BOUCK & LEE, INC. engineers & scientists

FIGURE 6

06/99 SYR-D54-DJH
20182002/20182g01.CDR



NOT TO SCALE:

	Approximate Changes in Flood Storage at Transect B						
Elevation Interval	**971.5 - 972**	**972 - 973**	**973 - 974**	**974 - 975**	**975 - 976**	**976 - 977**	**977 - 978**
Change Flood Storage (sf)	0.1	0.6	1.25	1.75	2.5	3.125	3.5
Total Change In Flood Storage at Transect B	18.1 (sf)						

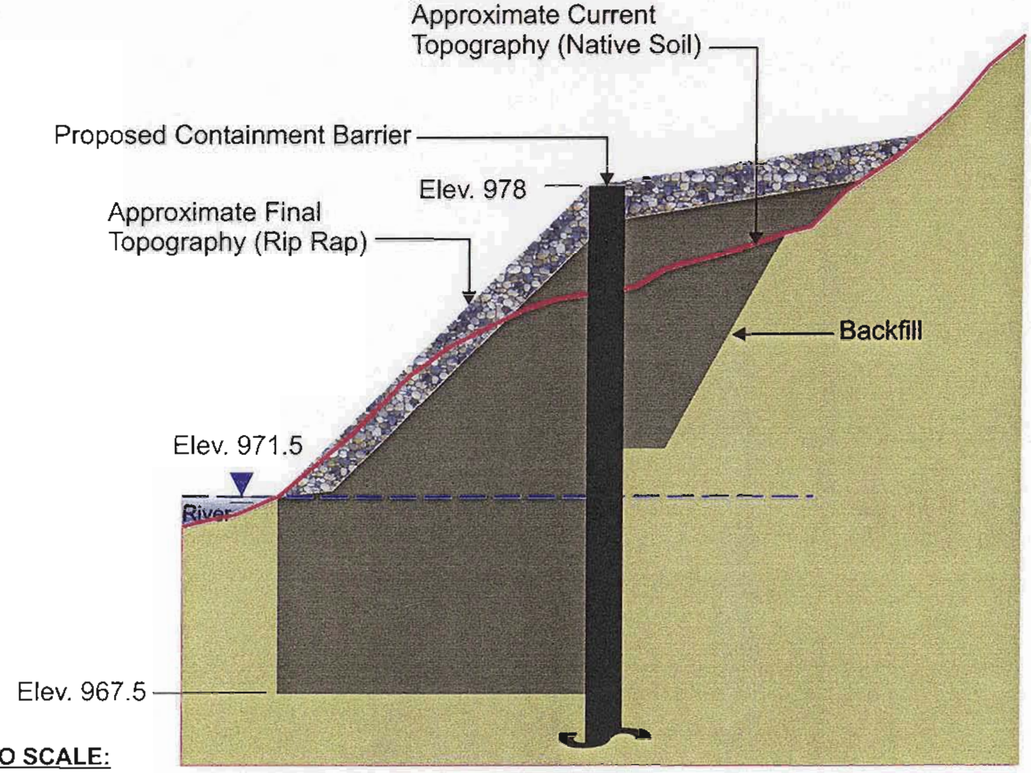
GENERAL ELECTRIC COMPANY
PITTSFIELD, MASSACHUSETTS
LYMAN STREET AREA
SUPPLEMENTAL SOURCE CONTROL MEASURES

CONCEPTUAL CROSS-SECTION
TRANSECT B

BBL BLASLAND, BOUCK & LEE, INC.
engineers & scientists

FIGURE
7

06/99 SYR-D54-DJH
20182002/20182g02.CDR



NOT TO SCALE:

	Approximate Changes in Flood Storage at Transect C						
Elevation Interval	**971.5 - 972**	**972 - 973**	**973 - 974**	**974 - 975**	**975 - 976**	**976 - 977**	**977 - 978**
Change Flood Storage (sf)	0	-0.1	-0.3	-0.5	-2	-4	-5.25
Total Change in Flood Storage at Transect C	-14.9 (sf)						

GENERAL ELECTRIC COMPANY
PITTSFIELD, MASSACHUSETTS
LYMAN STREET AREA
SUPPLEMENTAL SOURCE CONTROL MEASURES

CONCEPTUAL CROSS-SECTION
TRANSECT C

BBL BLASLAND, BOUCK & LEE, INC. engineers & scientists

FIGURE 8

06/99 SYR-D54-DJH
20182002/20182g03.CDR

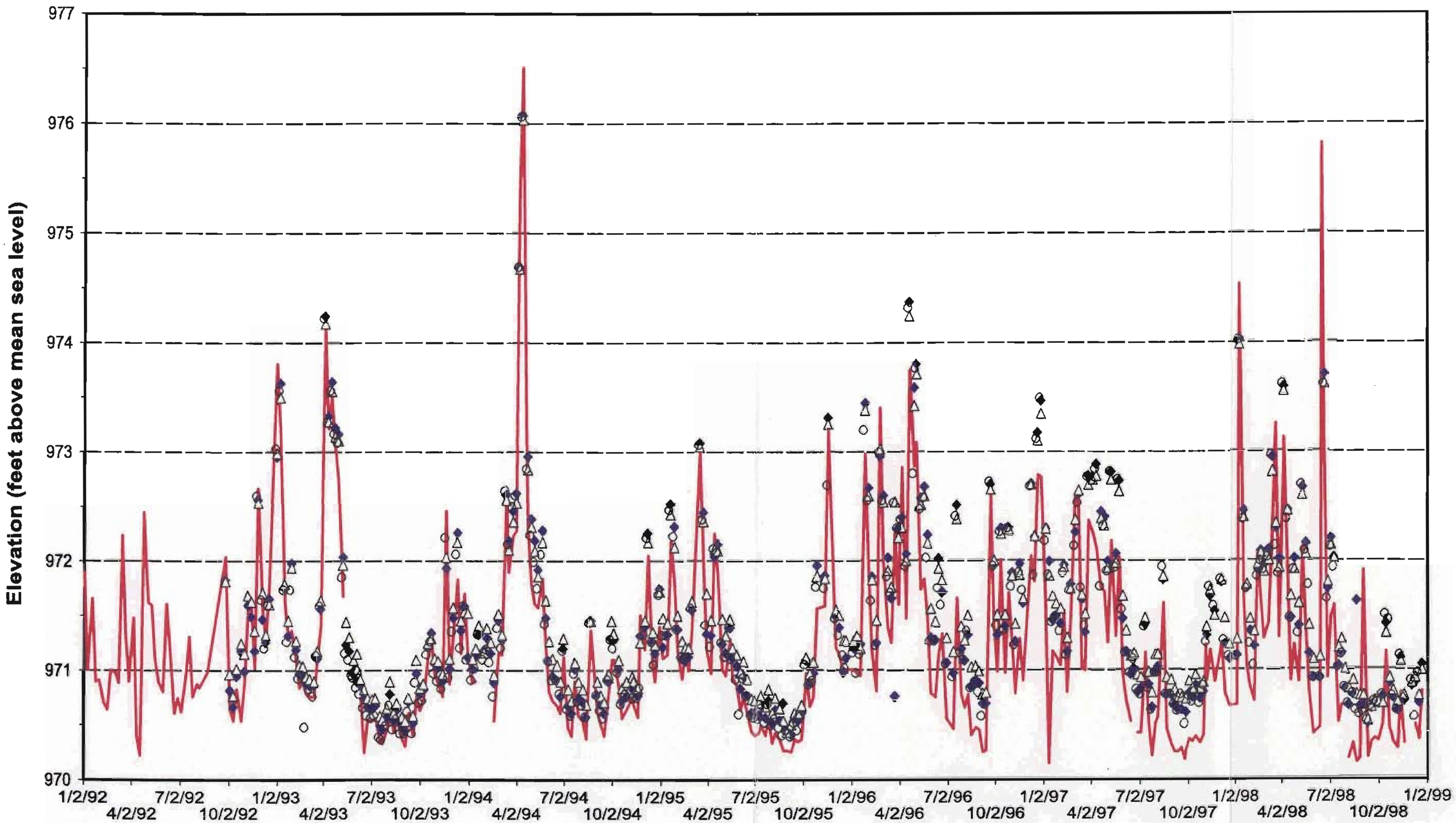
Appendices

Appendix A

BLASLAND, BOUCK & LEE, INC.
engineers & scientists

Historical Groundwater Information

(Figures 2 through 5 of GE's February 16, 1999 Letter)



NOTES:

1. Data compiled by Golder associates.
2. River elevation data was not collected on the following dates: May 13-June 3, 1993; January 20 - February 10, 1994; June 19, 1997; July 30, 1998; and December 30, 1998.

LEGEND

◆ P-1 △ P-3 ○ P-4 — RIVER ELEVATION

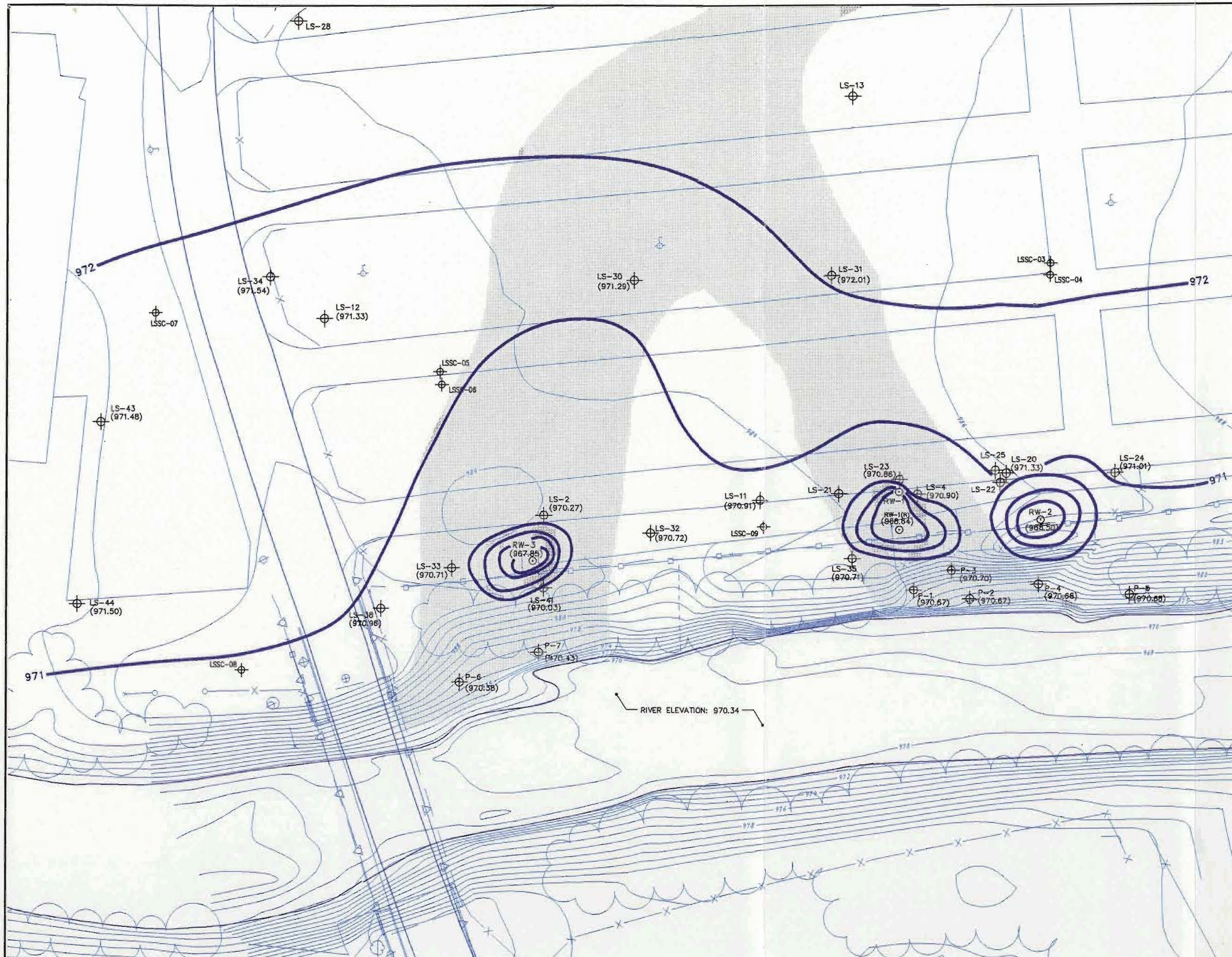
GENERAL ELECTRIC COMPANY
PITTSFIELD, MASSACHUSETTS

LYMAN STREET PARKING LOT/ USEPA AREA 5A RIVERBANK AREA HYDROGRAPHS

BBL BLASLAND, BOUCK & LEE, INC. engineers & scientists

FIGURE 2

2/99 SYR-D54-DJH
20140005/20140g08.cdr



LEGEND:

- EXISTING INDEX ELEVATION CONTOUR
- EXISTING INTERMEDIATE ELEVATION CONTOUR
- DECIDUOUS TREE
- CONIFEROUS TREE
- MANHOLE
- CHAIN LINK FENCE
- POLE (NON-UTILITY)
- POLE (OVERHEAD UTILITY)
- APPROXIMATE DELINEATION OF FORMER OXBOWS
- ES2-1 EXISTING MONITORING WELL
- RW-3 EXISTING PUMPING WELL
- GROUNDWATER ELEVATION CONTOUR (DASHED WHERE INFERRED)

NOTES:

1. MAPPING IS BEST AVAILABLE INFORMATION AS OF 12/10/98 BASED ON MAPPING PROVIDED BY LOCKWOOD MAPPING, INC. PREPARED FROM 1990 AERIAL PHOTOGRAPHY; DATA PROVIDED BY GENERAL ELECTRIC; AND BLASLAND AND BOUCK ENGINEERS, PC. CONSTRUCTION PLANS. RIVERBANK AND RIVER BED TOPOGRAPHIC INFORMATION PROVIDED BY BBL FROM OCTOBER 12-23, 1998 FIELD SURVEY.
2. COORDINATE GRID BASED ON 1927 STATE PLANE COORDINATES.
3. ELEVATION DATUM REFERENCED TO NGVD 1929.
4. ALL SAMPLING LOCATIONS ARE APPROXIMATE.
5. SOME GROUNDWATER CONTOURS AROUND RECOVERY WELLS NOT SHOWN FOR CLARITY.

0 20' 40'
GRAPHIC SCALE

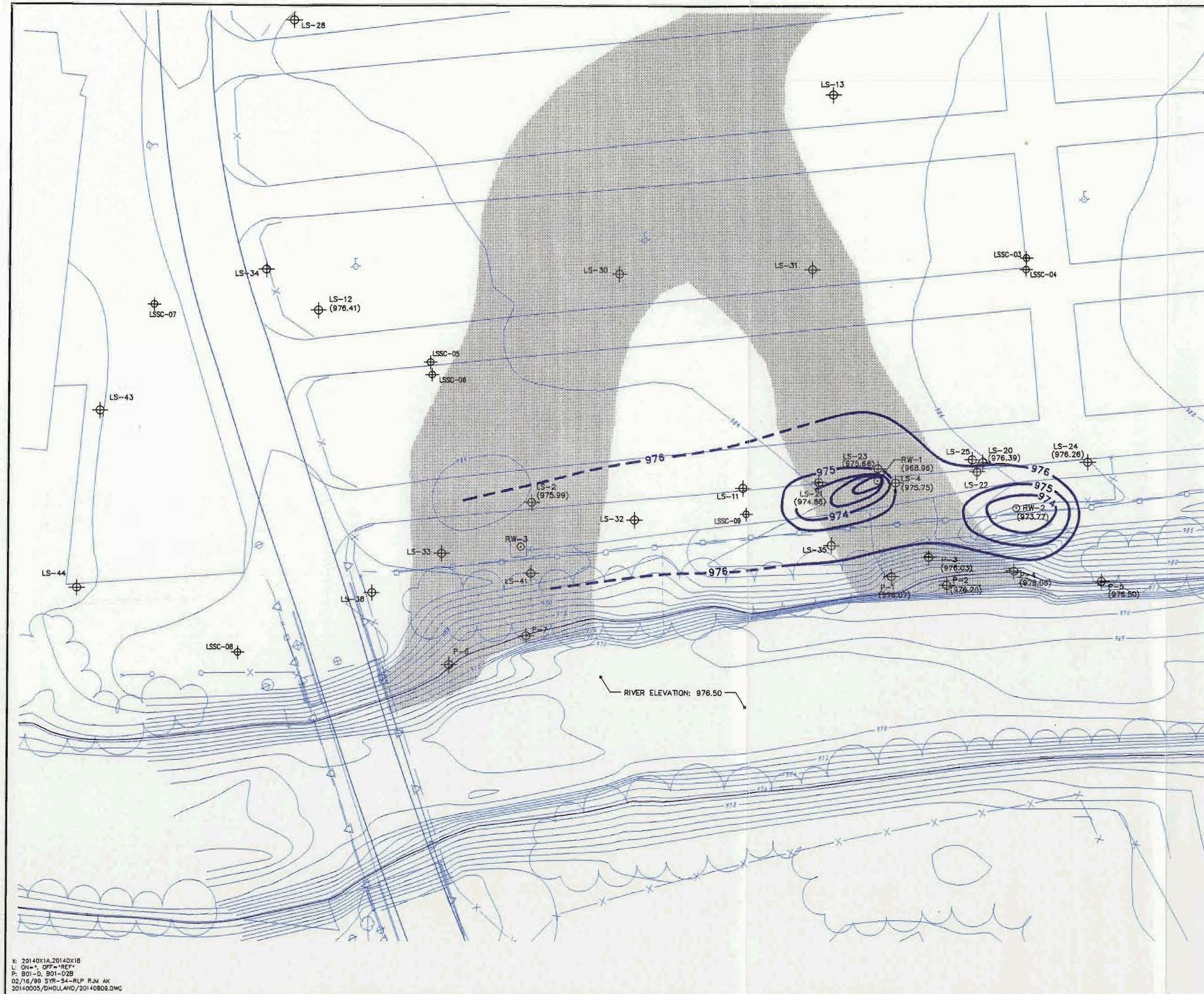
GENERAL ELECTRIC COMPANY
PITTSFIELD, MASSACHUSETTS
LYMAN STREET PARKING LOT
SOURCE CONTROL INVESTIGATION

GROUNDWATER ELEVATION CONTOUR MAP
OCTOBER 1, 1998

BBL BLASLAND, BOUCK & LEE, INC.
engineers & scientists

FIGURE
3

X: 20140X1A,20140X18
L: ON=*, OFF=*REF*
P: B01-D, B01-D2B
02/16/99 SYR-54-RLP RJM AK
20140005/DHOLLAND/20140B10.DWG



LEGEND:

- EXISTING INDEX ELEVATION CONTOUR
- EXISTING INTERMEDIATE ELEVATION CONTOUR
- DECIDUOUS TREE
- CONIFEROUS TREE
- MANHOLE
- CHAIN LINK FENCE
- POLE (NON-UTILITY)
- POLE (OVERHEAD UTILITY)
- APPROXIMATE DELINEATION OF FORMER OXBOWS
- ES2-1 EXISTING MONITORING WELL
- RW-1 EXISTING PUMPING WELL
- GROUNDWATER ELEVATION CONTOUR (DASHED WHERE INFERRED)

NOTES:

1. MAPPING IS BEST AVAILABLE INFORMATION AS OF 12/10/98 BASED ON MAPPING PROVIDED BY LOCKWOOD MAPPING, INC. PREPARED FROM 1990 AERIAL PHOTOGRAPHY; DATA PROVIDED BY GENERAL ELECTRIC; AND BLASLAND AND BOUCK ENGINEERS, PC. CONSTRUCTION PLANS. RIVERBANK AND RIVER BED TOPOGRAPHIC INFORMATION PROVIDED BY BBL FROM OCTOBER 12-23, 1998 FIELD SURVEY.
2. COORDINATE GRID BASED ON 1927 STATE PLANE COORDINATES.
3. ELEVATION DATUM REFERENCED TO NGVD 1929.
4. ALL SAMPLING LOCATIONS ARE APPROXIMATE.
5. SOME GROUNDWATER CONTOURS AROUND RECOVERY WELLS NOT SHOWN FOR CLARITY.

0 30' 60'
GRAPHIC SCALE

GENERAL ELECTRIC COMPANY
PITTSFIELD, MASSACHUSETTS
LYMAN STREET PARKING LOT
SOURCE CONTROL INVESTIGATION

GROUNDWATER ELEVATION CONTOUR MAP
APRIL 13-14, 1994

BBL BLASLAND, BOUCK & LEE, INC. engineers & scientists

FIGURE 4

LEGEND:

988	EXISTING INDEX ELEVATION CONTOUR
	EXISTING INTERMEDIATE ELEVATION CONTOUR
	DECIDUOUS TREE
	CONIFEROUS TREE
	MANHOLE
	CHAIN LINK FENCE
	POLE (NON-UTILITY)
	POLE (OVERHEAD UTILITY)
	APPROXIMATE DELINEATION OF FORMER OXBOWS
ES2-1	EXISTING MONITORING WELL
RW-3	EXISTING PUMPING WELL
	GROUNDWATER ELEVATION CONTOUR (DASHED WHERE INFERRED)

NOTES:

1. MAPPING IS BEST AVAILABLE INFORMATION AS OF 12/10/98 BASED ON MAPPING PROVIDED BY LOCKWOOD MAPPING, INC. PREPARED FROM 1990 AERIAL PHOTOGRAPHY; DATA PROVIDED BY GENERAL ELECTRIC; AND BLASLAND AND BOUCK ENGINEERS, PC. CONSTRUCTION PLANS. RIVERBANK AND RIVER BED TOPOGRAPHIC INFORMATION PROVIDED BY BBL FROM OCTOBER 12-23, 1998 FIELD SURVEY.
2. COORDINATE GRID BASED ON 1927 STATE PLANE COORDINATES.
3. ELEVATION DATUM REFERENCED TO NGVD 1929.
4. ALL SAMPLING LOCATIONS ARE APPROXIMATE.
5. SOME GROUNDWATER CONTOURS AROUND RECOVERY WELLS NOT SHOWN FOR CLARITY.

0 20' 40'
GRAPHIC SCALE

GENERAL ELECTRIC COMPANY
PITTSFIELD, MASSACHUSETTS
LYMAN STREET PARKING LOT
SOURCE CONTROL INVESTIGATION

GROUNDWATER ELEVATION CONTOUR MAP JANUARY 7-8, 1998

BBL BLASLAND, BOUCK & LEE, INC. engineers & scientists

FIGURE 5

X: 20140X1A,20140X1B
L: ON=*, OFF=*REF*
P: B01-DL B01-D2B
02/16/99 SYR-54-RLP RJM AK
20140003/DHOLLAND/20140B05.DWG

Appendix B

BLASLAND, BOUCK & LEE, INC.
engineers & scientists

Groundwater Hydraulic Modeling Results

SHEETPILE WALL GROUNDWATER FLOW MODEL ANALYSIS

LYMAN STREET SITE

Prepared for:

General Electric Company
Pittsfield, Massachusetts

Prepared by:

HSI GeoTrans, Inc.
6 Lancaster County Road
Harvard, Massachusetts 01451

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1 INTRODUCTION

A groundwater modeling analysis of the Lyman Street Site in Pittsfield, Massachusetts was done to evaluate hydrogeologic changes that could potentially result from the installation of a proposed sheetpile wall.

The proposed sheet pile wall will supplement the existing LNAPL containment systems. Monitoring data and this model indicate that the existing groundwater/NAPL pumping containment systems are preventing any significant migration of LNAPL to the Housatonic River. The proposed sheet pile wall will provide further assurance of LNAPL containment as part of the continuing remediation activities.

The model was developed to evaluate potential changes to groundwater flow directions, pumping rates and vertical hydraulic gradients which could result from the installation of the sheet pile wall. Specifically groundwater flow directions at the ends of the sheet pile wall and changes in vertical gradients near the river were evaluated.

The following sections of the report present the Site conceptual model, numerical model construction and results of the model analyses.

2 SITE CONDITIONS

2.1 SITE DESCRIPTION

The Site, designated by the Massachusetts DEP as the Lyman Street Site and by the USEPA as Area 5A, is located along the Housatonic River in Pittsfield, Massachusetts. The Site consists of a former GE parking lot, a strip of land along the riverbank to the south of the Lyman Street parking lot, and a strip of land owned by Western Massachusetts Electric along the east side of the Lyman Street parking lot. The Site covers an area of approximately five acres and is bounded by Lyman Street on the west and the Housatonic River on the south. Previous investigations have detected the presence of LNAPL and DNAPL beneath the site. GE currently maintains a NAPL containment/recovery system in this area, which includes active groundwater and NAPL extraction from three on-site recovery wells and the use of oil absorbent booms in the river at the base of the river bank. As a supplement to these measures and to further minimize the potential for any NAPL migration towards the river, GE has proposed to install a sheet pile wall adjacent to the river bank. A site map which shows the locations of monitoring and recovery wells installed during previous investigations, is presented in Figure 2-1.

2.2 STRATIGRAPHY

The Site is underlain by fill and fluvial deposits overlying a basal till layer. The fill ranges in thickness from zero to 20 feet. The underlying fluvial deposits consist of thinly bedded fine to medium sand with lenses of coarse sand and sandy gravel. The fluvial deposits range in thickness from less than one foot to more than 30 feet. These fluvial deposits overlie the basal till layer. Data from numerous borings indicate that the till is continuous beneath the Site. The top of till is highest in the central portion of the Site and slopes to the northeast and southwest. In the central portion of the Site, a fine to medium grained sand unit was observed within the till at boring LS-14, and monitoring wells LS-25 and LSSC-10. The sand unit is not continuous beneath the Site. At the eastern side of the Site (boring LSSC-02), the till was encountered between the depths of 38 to 50 feet, directly underlain by bedrock. At the western side of the Site (boring LSSC-11), the till was encountered at a depth of 22 feet and extended to the end of the boring, at a depth of 60 feet. The sand unit detected within the till at sampling locations LS-14, LS-25 and LSSC-10 was not encountered in either boring LSSC-02 or LSSC-11.

2.3 HYDROGEOLOGY

Prior to the installation of the active pumping wells, groundwater in the unconsolidated deposits beneath the Site flowed generally from the north to south. The horizontal gradient across the Site ranged from 0.007 to 0.03. Three extraction wells have created localized cones of depression that result in groundwater being diverted toward, and captured by, these wells. Vertical gradients are upward at the Site with the exception of local downward gradients within the cones of depression created by the three pumping wells

Based on prior slug test data from Site monitoring wells, the hydraulic conductivity of the fill ranges from 1.4 to 113 ft/day. The range of hydraulic conductivity values reported for the sand is 2.9 to 130 ft/day. A hydraulic conductivity value of 0.017 ft/day was measured in the till. Average precipitation in the site vicinity is 46 in/yr. Average recharge due to precipitation is estimated to be 10 in/yr regionally (Blasland, Bouck & Lee, Inc., 1999) but may vary locally. Three active recovery wells, RW-1R, RW-2, and RW-3 are located along the southern site boundary in the fill and sand deposits. Recovery well RW-1R, which is a replacement for nearby recovery well RW-1, began operation in October 1998. The pumps in the wells are operated by automated controllers which maintain the water level within a specified range. Based on flow-totalizer readings from August 1997 to July 1998, the average pumping rates in RW-1, RW-2, and RW-3 are 1.3, 2.0, and 3.3 gallons/minute, respectively. The average pumping rate in RW-1R, from October 1998 to May 1999 was 2.6 gallons/minute.

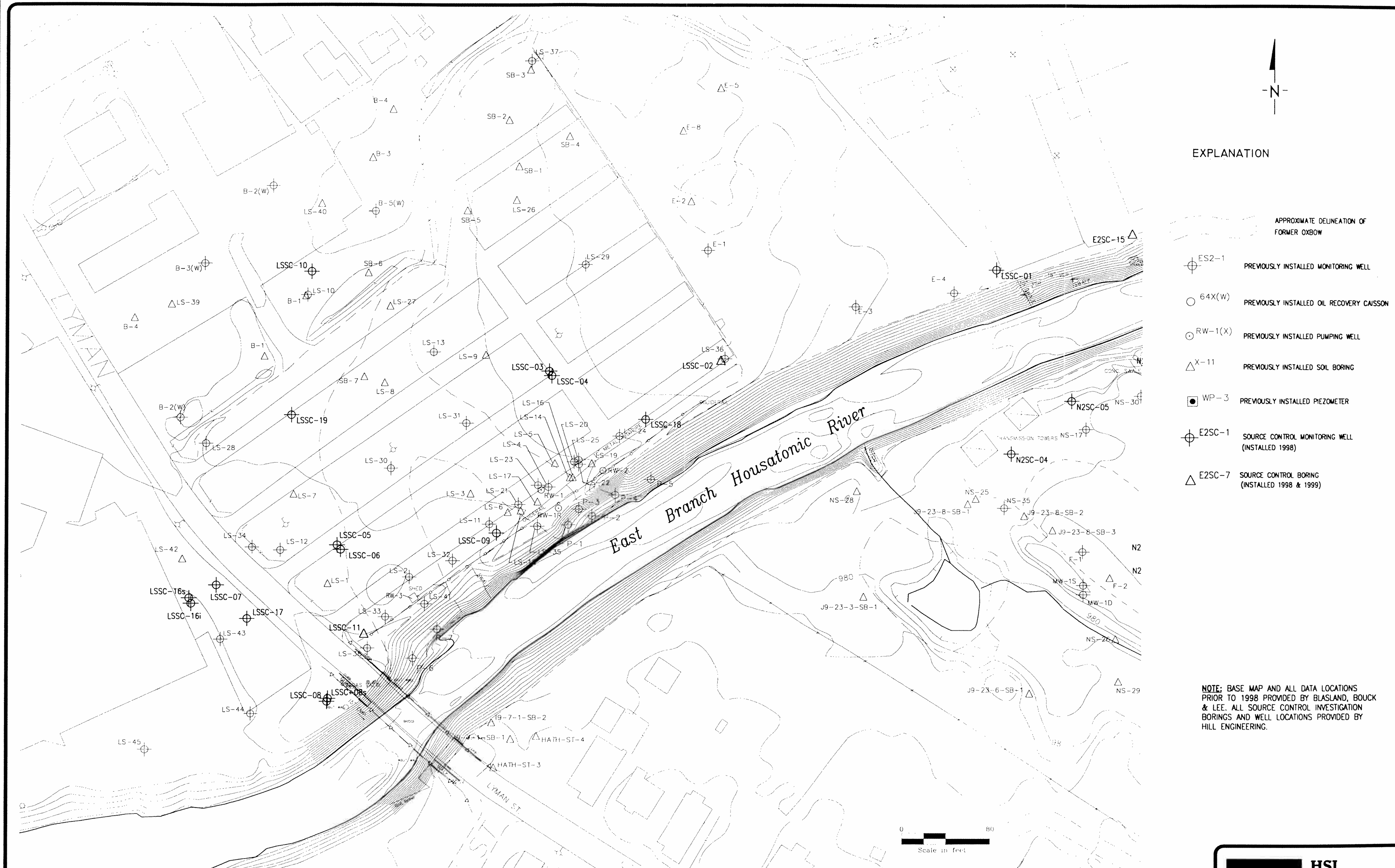

Figure 2-1 Lyman Street Site Map

3 GROUNDWATER FLOW MODEL

3.1 GENERAL MODEL CONSTRUCTION

The groundwater model covers an area from approximately East Street along the northwestern Site boundary to the Housatonic River along the southeastern Site boundary. The modeled area extends approximately 410 feet southwest of Lyman Street to approximately 470 feet beyond the northeastern edge of the parking lot area. Vertically, the modeled area extends from the water table (approximate elevation of 971 to 976 feet) to an elevation of 930 feet. The bottom layer of the model represents bedrock in the eastern portion of the Site and till in the western portion of the site. The bottom of the model is a no-flow boundary.

Groundwater flows into the modeled area through the unconsolidated deposits along the northwestern model boundary. This upgradient boundary is represented in the model as a constant head boundary. The northeastern and southwestern model boundaries, which correspond to flow lines, are represented in the model as no-flow boundaries. Precipitation infiltrates the top of the model. Groundwater exits the model near the southeastern boundary at the three recovery wells and along the Housatonic River beyond the lateral limits of the capture zone of the recovery wells. The river is included in the model as a constant head boundary.

3.2 MODEL CODE

The computer code MODFLOW which was developed by the USGS (McDonald and Harbough, 1984) was used to simulate groundwater flow. MODFLOW is a three-dimensional finite-difference groundwater flow model code that has been thoroughly tested and is widely accepted. The proposed sheetpile wall was modeled using the horizontal-flow barrier module developed to work with MODFLOW (Hsieh and Frecklton, 1993). Groundwater Vistas (ESI, 1998) a pre- and post-processor designed to work with MODFLOW was used to facilitate data input and graphical representation of the model output.

3.3 GRID AND LAYERING

The model grid consists of 65 rows and 95 columns with variable grid block spacing. Grid block lengths range from five to 50 feet. Five foot grid spacing is used in the vicinity of the pumping wells and the proposed sheet pile wall. The grid spacing gradually increases to 50 feet along the west, north and east boundaries (see Figure 3-1).

The model has eight layers to more accurately represent fill, sand, and till distribution and to allow for evaluation of variable sheetpile wall depths. Each layer has a uniform bottom elevation. The elevation of the bottom of the model is 930 feet. The bottom layer of the model (Layer 8) is 10 feet thick. Layers two through seven are five feet thick. The top layer (Layer 1) has a variable thickness (due to the variation in the water table elevation across the site) and a uniform bottom elevation of 970 feet.

3.4 FLOW PARAMETERS AND BOUNDARY CONDITIONS

The model incorporates three units of varying hydraulic conductivity. These units correspond to fill, sand, and till. The hydraulic conductivity distribution in model layers 1 through 8 is illustrated in Figures 3-2 through 3-9. Ranges of hydraulic conductivity values for the fill and sand, based on slug test data, were evaluated during the calibration process. Ranges of hydraulic conductivity values for the till were based on typical values for till. A range of recharge rates from 5 to 15 inches in/yr and spatially varied recharge for the parking lot and open areas were also evaluated.

A constant head boundary condition is specified along the northern model boundary in Layers 1 through 8. The constant head was increased by 0.5 feet increments from 976 feet in Layer 1 to 979.5 feet in Layer 8 to simulate the upward gradient observed beneath this portion of the Site. Along the southern model boundary, constant heads are specified in the river in Layer 1. Based on data from river gauges located at the GE East Street Area 2 Site and the Lyman Street Site, the modeled hydraulic head in the river is 971.6 feet in the eastern most river block and decreases linearly to 971.1 feet in the western most river block.

3.5 CALIBRATION

The model was calibrated to average water level elevations from 37 monitoring wells and average flow rates in recovery wells RW-1, RW-2, and RW-3. Water level and pumping rate data from the period August 1996 to July 1998 were used to calculate the averages.

The hydraulic conductivities used in the calibrated model are:

	K_H	K_V
Fill	40 ft/day	40 ft/day
Sand	15 ft/day	1.5 ft/day
Till	0.017 ft/day	0.0017 ft/day

Where K_H corresponds to the horizontal hydraulic conductivity and K_V corresponds to the vertical hydraulic conductivity. Recharge is specified as a uniform rate of 11.8 in/year.

A comparison of averaged measured (observed) groundwater level data to model-calculated groundwater levels is provided in Table 3-1. Model-calculated water levels compare favorably to observed water levels as indicated by the residual mean of 0.67 feet. The calculated vertical hydraulic gradient at well cluster LS-20 and LS-25 is within five percent of the measured vertical gradient. Model-calculated flow rates at the recovery wells, also compared favorably to measured flow rates.

The observed and model-calculated flow rates for recovery wells RW-1, RW-2, and RW-3 were:

WELL	OBSERVED	MODEL-CALCULATED
RW-1	1.33 gpm	2.01 gpm
RW-2	2.05 gpm	1.95 gpm
RW-3	3.35 gpm	3.94 gpm

Figure 3-10 shows the model-calculated water table contours. The model-calculated layer 1 potentiometric surface is similar to the average water table conditions at the site from August 1996 through July 1998. Figure 3-11 shows the average observed water table conditions at the site from August, 1996 through July, 1998 used for the calibration of the model.

Table 3-1. Statistical analysis of calibrated model groundwater levels

WELL	ROW	COLUMN	LAYER	OBSERVED WATER LEVEL	MODEL-CALCULATED WATER LEVEL	RESIDUAL
LS-12	20	17	2	972.79	972.65	0.14
LS-34	19	15	3	972.58	972.79	-0.21
LS-43	24	9	4	972.39	972.36	0.03
LS-45	33	4	5	972.12	971.69	0.43
LS-44	39	8	4	972.37	971.72	0.65
LS-33	38	21	1	971.52	970.53	0.99
LS-32	37	35	1	971.55	970.89	0.66
LS-2	35	28	1	970.94	970.20	0.74
LS-11	35	39	3	971.87	971.64	0.23
P-6	47	21	1	971.39	971.04	0.35
P-7	45	27	1	971.41	970.87	0.54
P-1	43	49	1	971.67	971.03	0.64
P-2	44	53	1	971.58	971.05	0.53
P-3	42	52	1	971.73	970.90	0.83
P-4	44	59	1	971.63	970.92	0.71
P-5	45	65	1	971.53	971.36	0.17
LS-23	34	40	1	971.92	970.46	1.46
LS-21	35	44	1	972.22	971.05	1.17
LS-20	34	57	1	972.43	971.14	1.29
LS-25	35	57	6	976.08	975.32	0.76
LS-24	35	65	2	972.07	971.65	0.42
LS-31	21	45	2	973.02	972.67	0.35
LS-30	20	35	1	972.17	972.58	-0.41
LS-4	36	50	2	971.96	969.99	1.97
LS-41	40	28	1	970.57	970.14	0.43
E-1	20	82	1	975.03	973.63	1.40
E-3	40	87	1	973.49	972.49	1.00
E-4	48	89	2	973.43	972.16	1.27
LS-37	9	78	1	976.75	975.52	1.23
LS-28	14	17	2	974.23	973.88	0.35
LS-10	11	38	2	974.44	974.71	-0.27
LS-13	17	47	2	975.74	973.63	2.11
LS-29	17	72	3	974.3	973.94	0.36
LS-36	35	77	2	974.05	972.49	1.56
LS-35	40	44	2	971.85	971.00	0.85
E-7	5	87	1	975.6	976.42	-0.82
LS-38	41	18	2	971.83	971.15	0.68
Residual Mean						0.67
Residual Standard Dev.						0.62
Residual Sum of Squares						30.44
Absolute Residual Mean						0.76
Minimum Residual						-0.82
Maximum Residual						2.11
Range in Observed Water Level						6.18
Residual Std. Dev./Range in Obs. Water Level						0.10

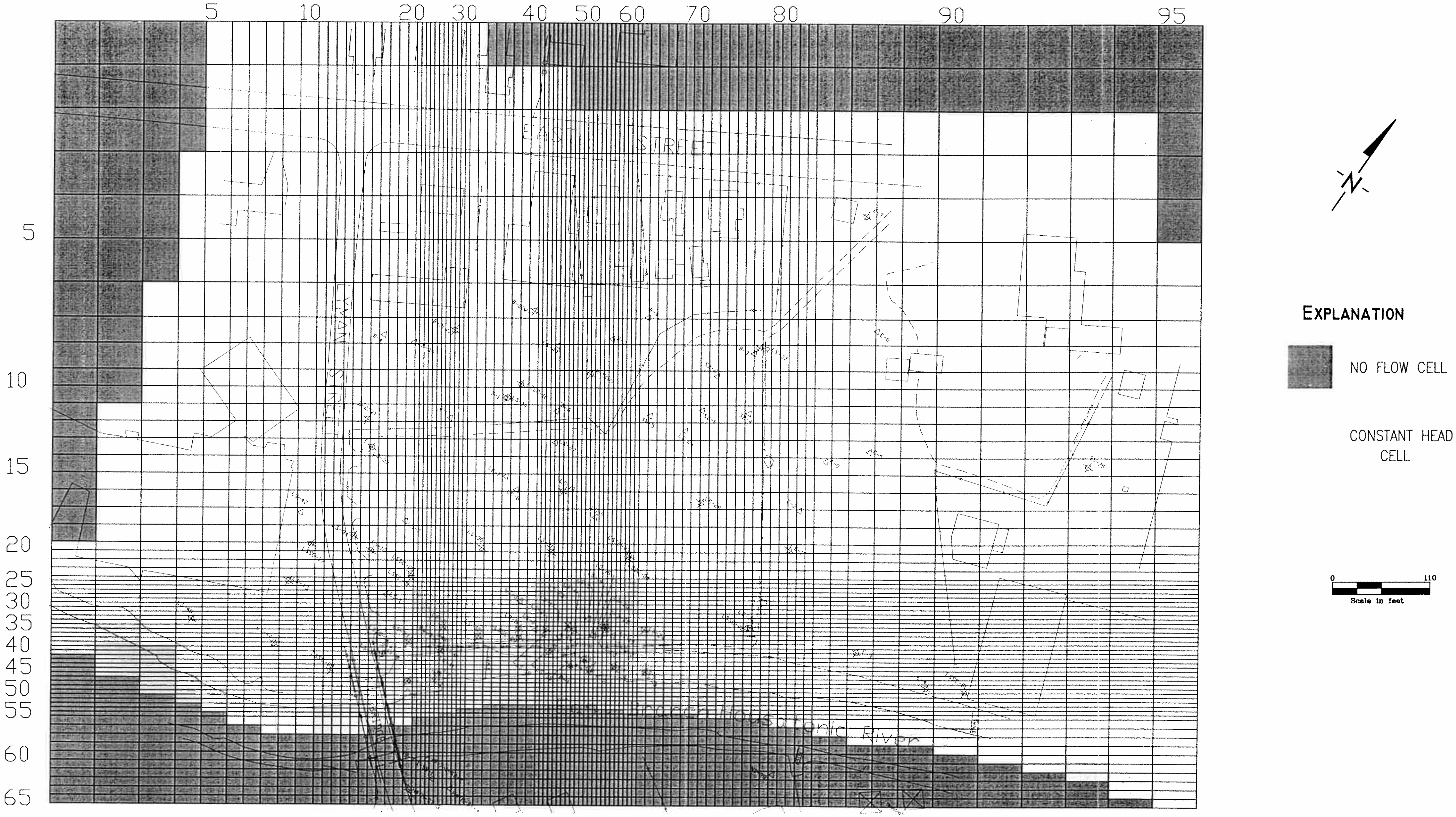

Figure 3-1 Model Grid

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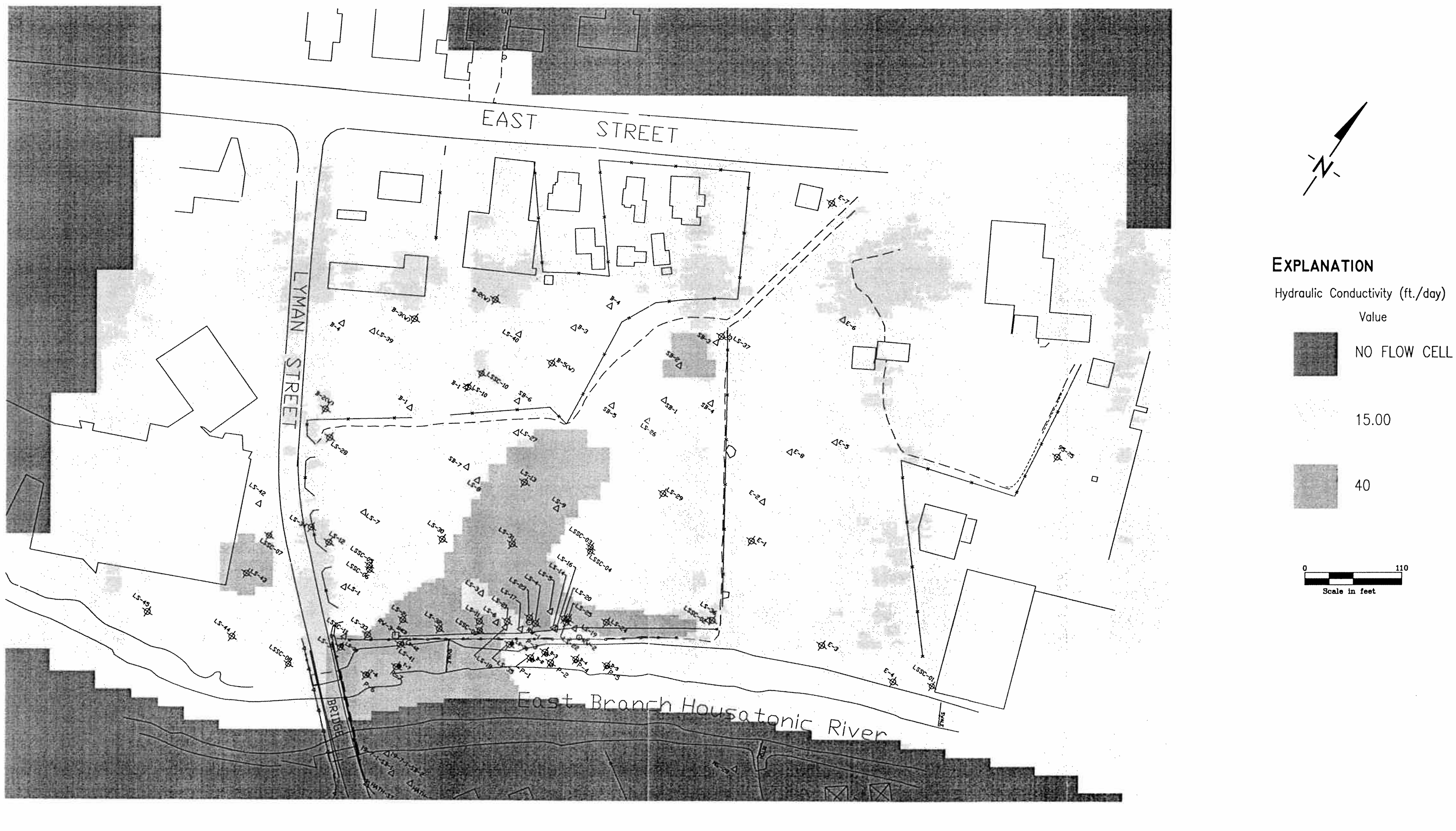

Figure 3-2 Layer 1 Hydraulic Conductivity

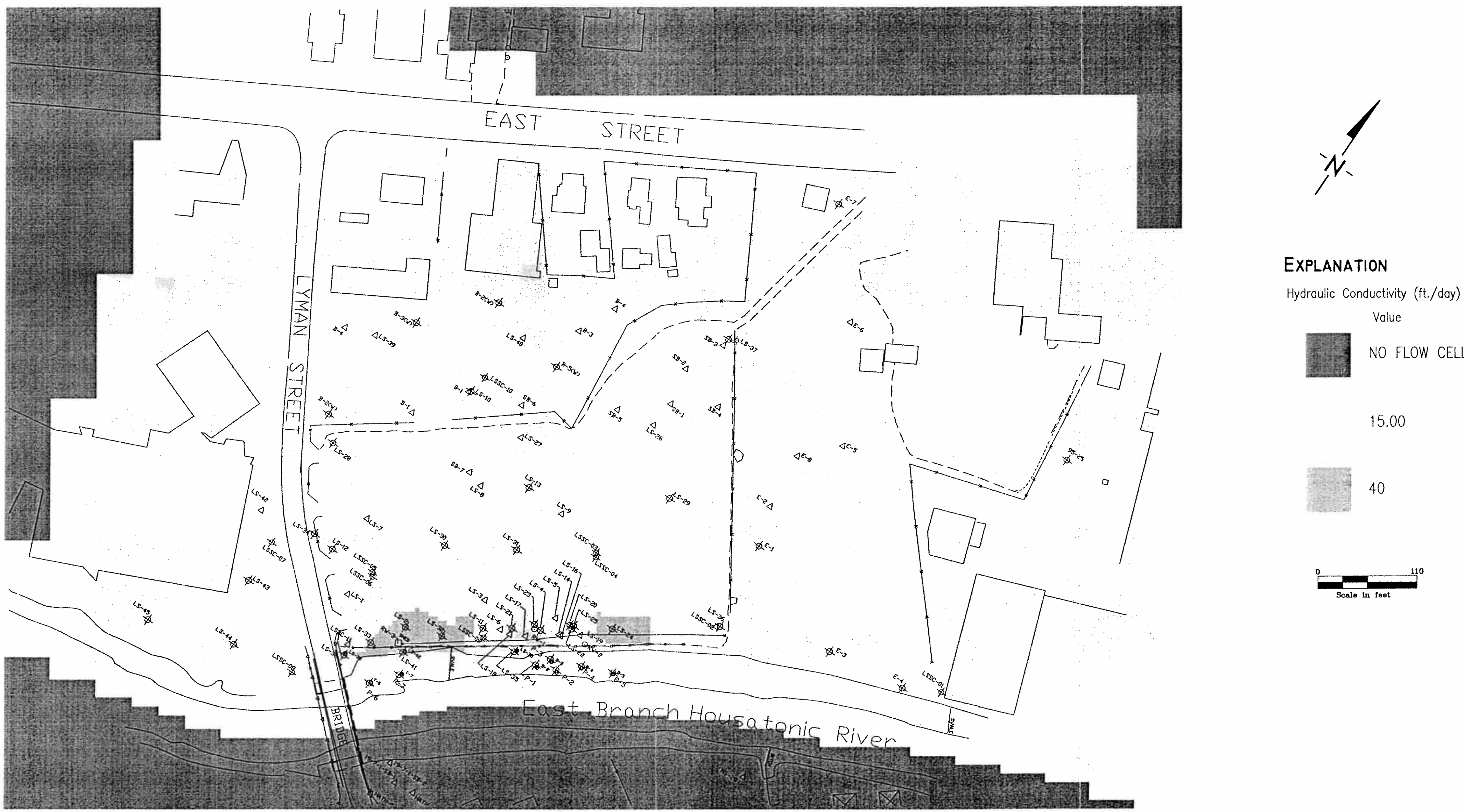

Figure 3-3 Layer 2 Hydraulic Conductivity

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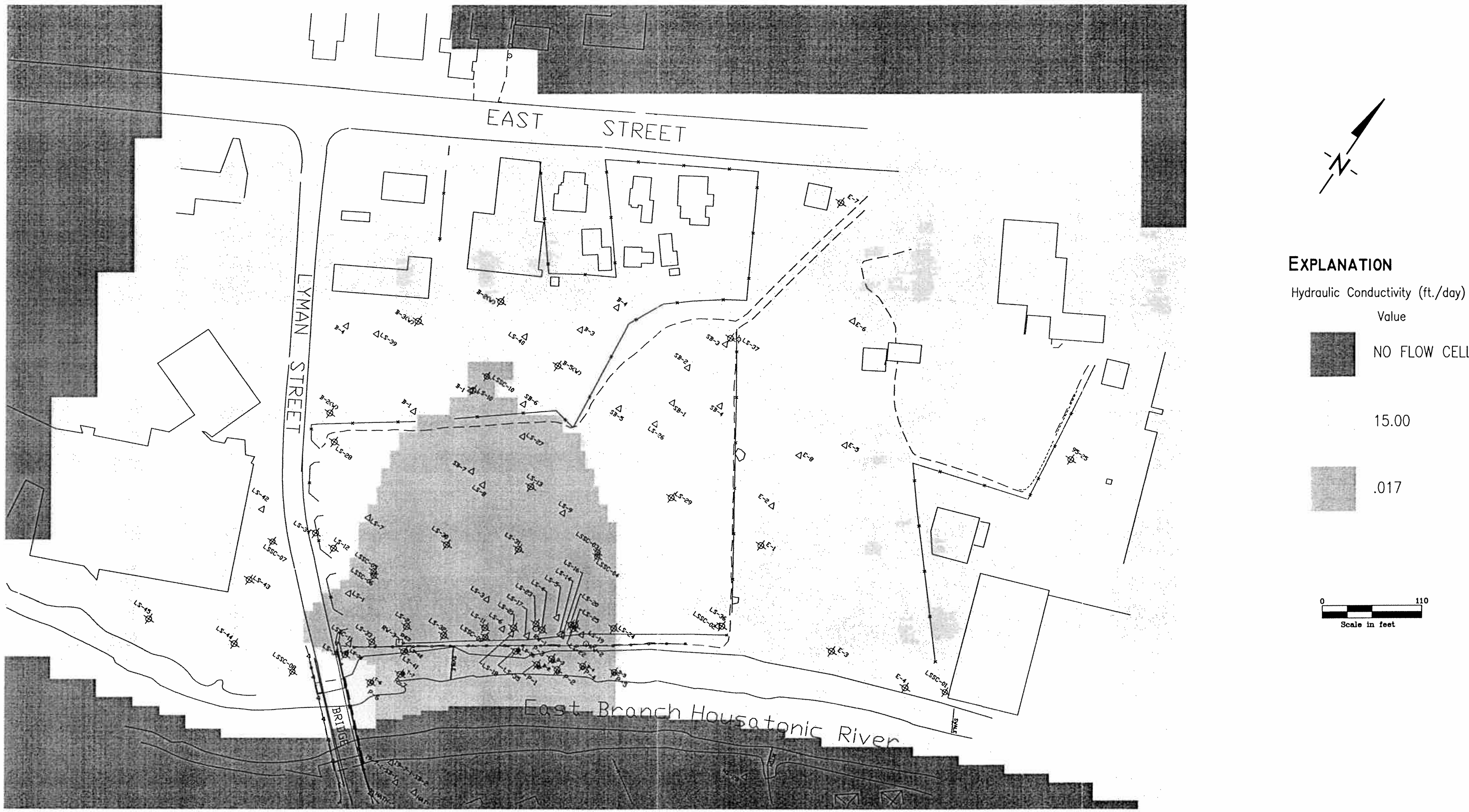

Figure 3-4 Layer 3 Hydraulic Conductivity

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Figure 3–5 Layer 4 Hyraulic Conductivity

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Figure 3-6 Layer 5 Hydaulic Conductivity

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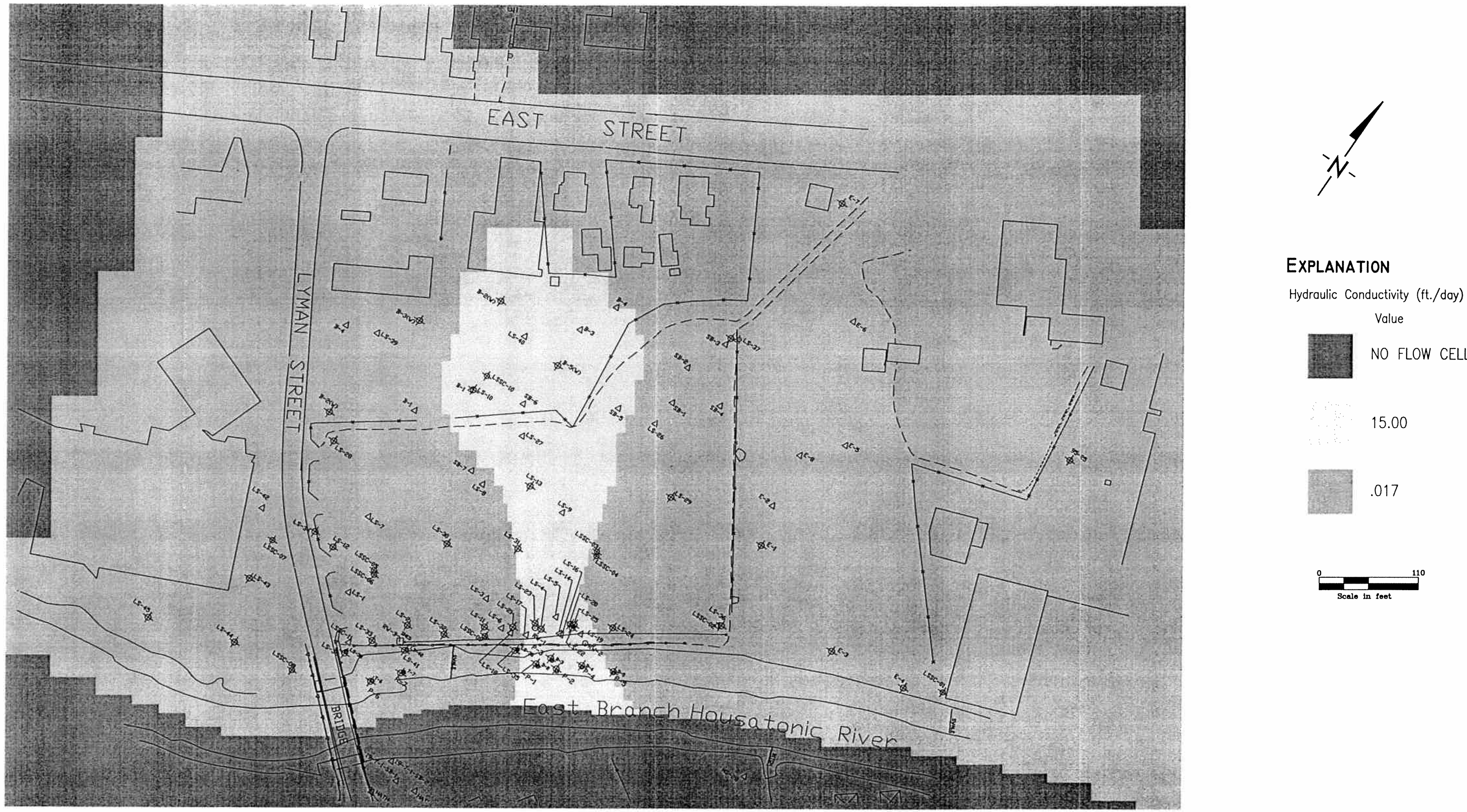

Figure 3–7 Layer 6 Hydraulic Conductivity

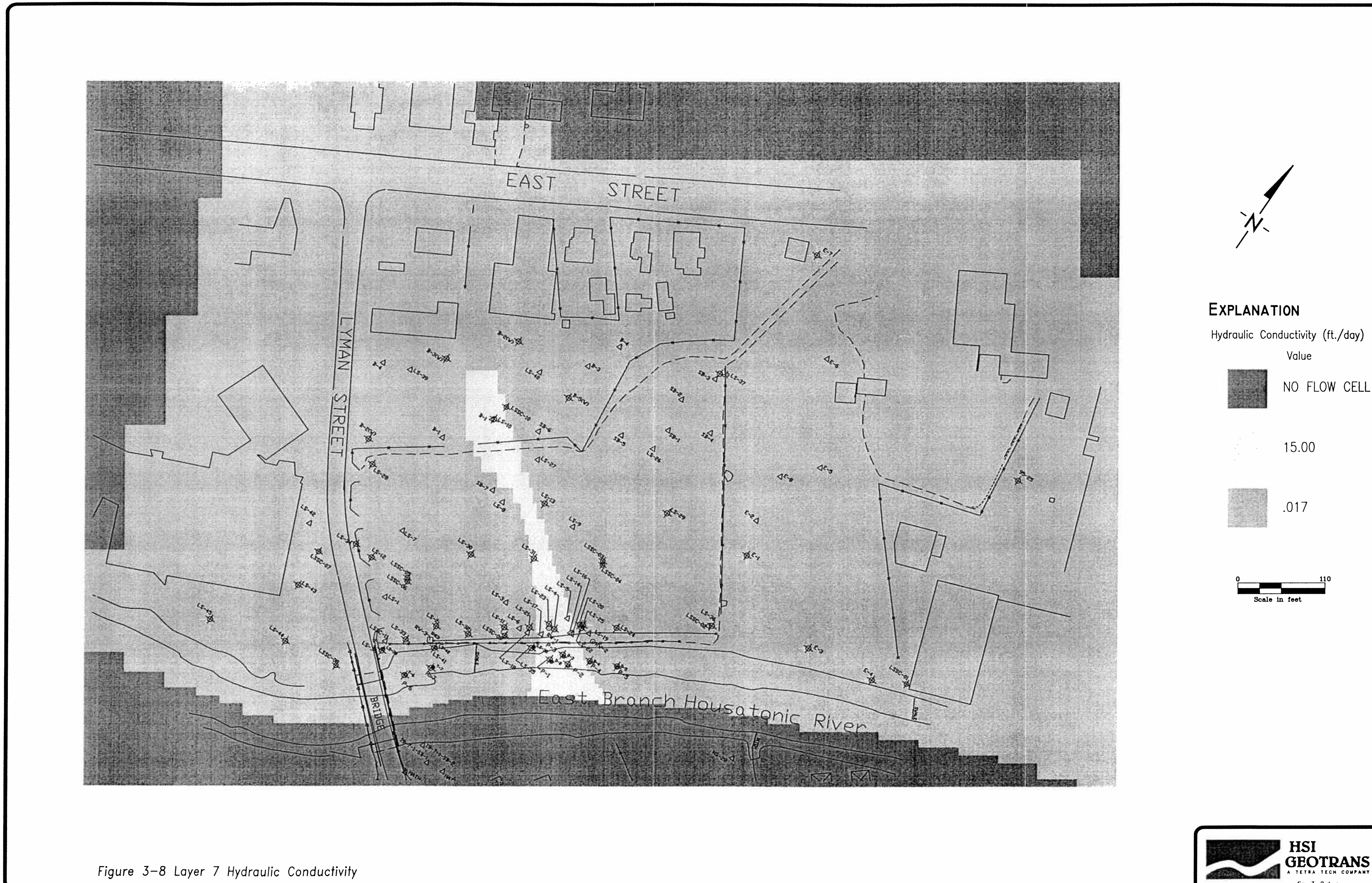

Figure 3-8 Layer 7 Hydraulic Conductivity

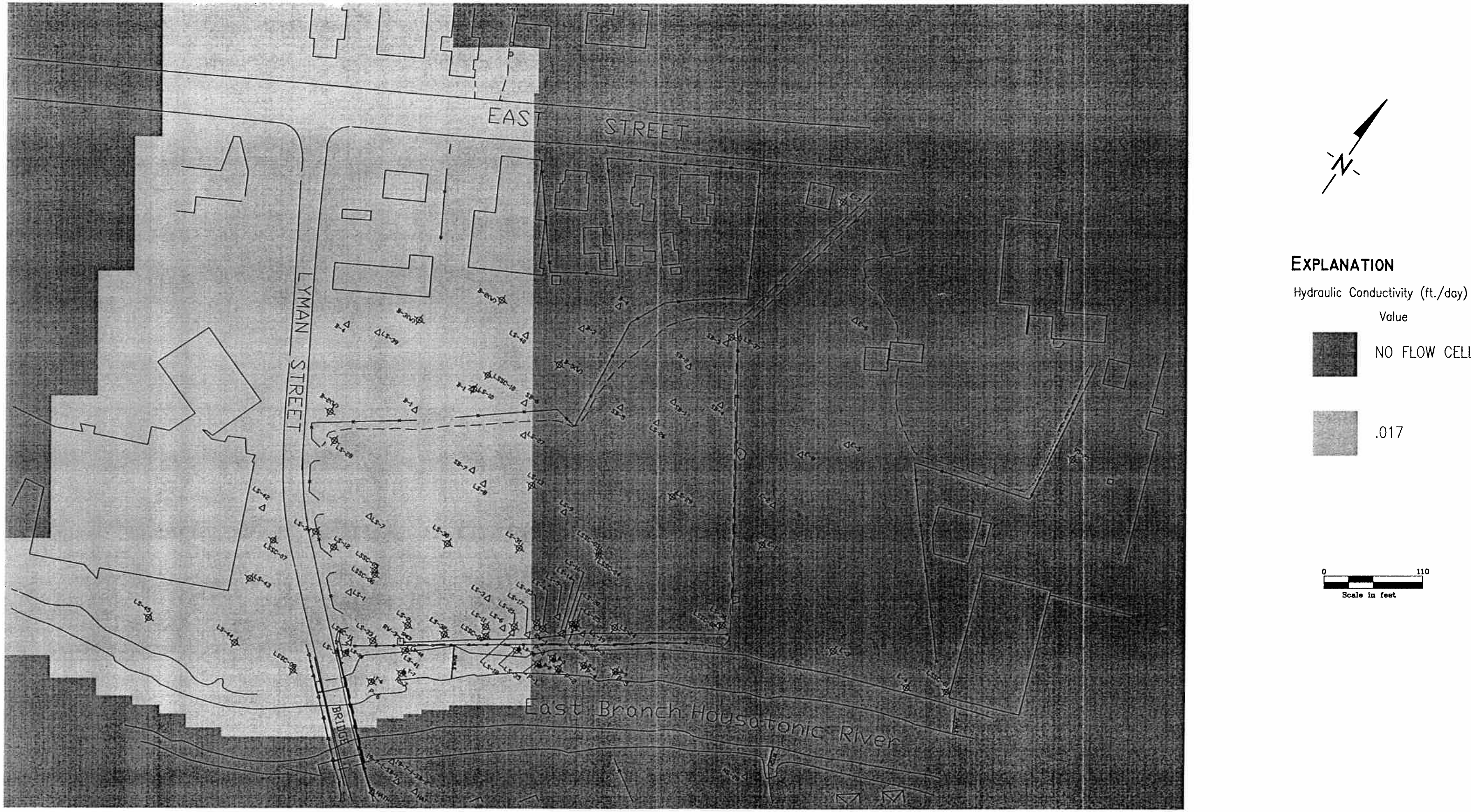

Figure 3–9 Layer 8 Hydaulic Conductivity

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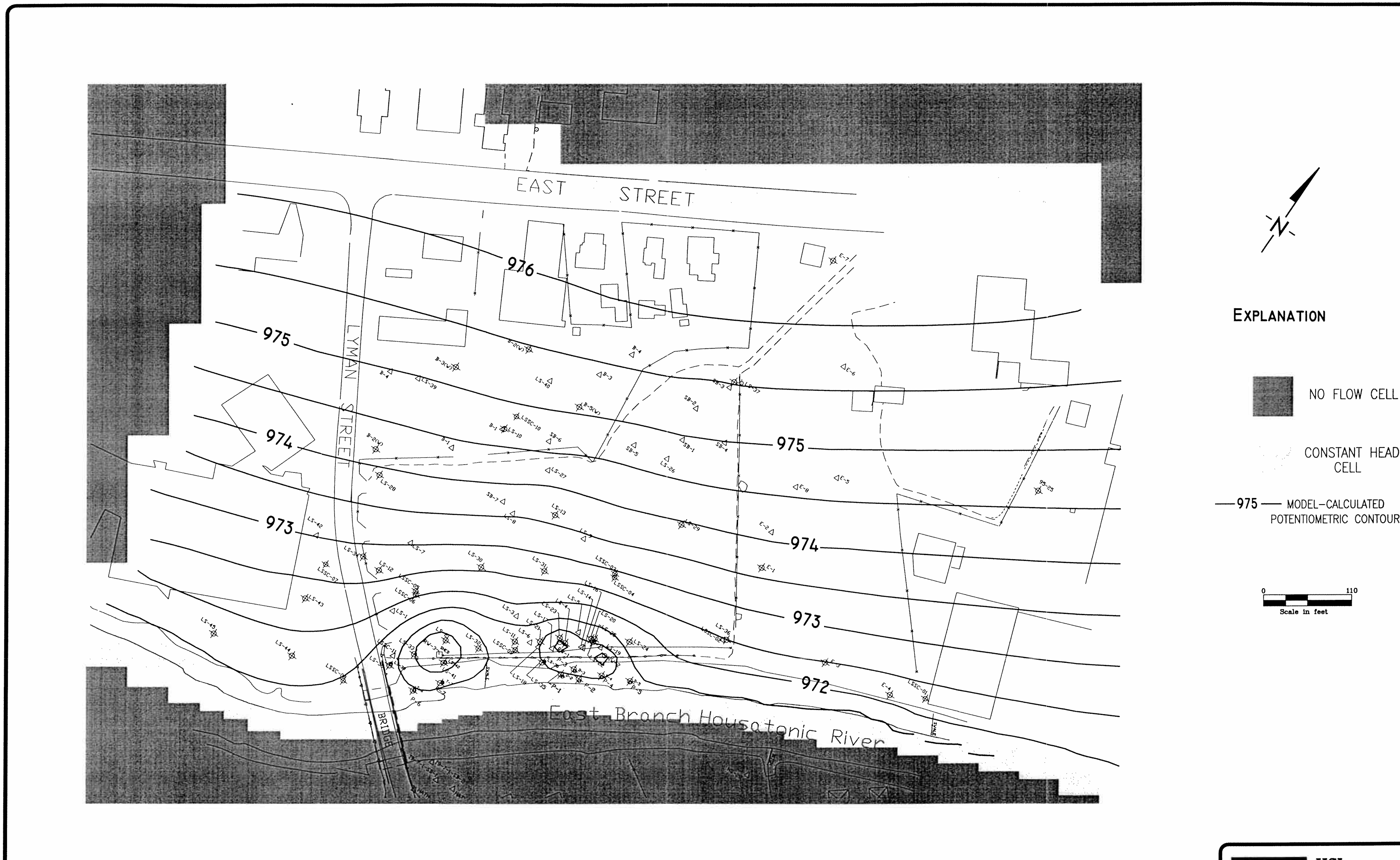

Figure 3-10 Layer 1 Model-Calculated Potentiometric Surface

Figure 3-11 Average Observed Water Levels 8/96-7/98 Potentiometric Surface

4 SHEETPILE WALL ANALYSIS

4.1 WALL SPECIFICATIONS

To simulate the potential effects of the proposed sheetpile wall on water table elevations and recovery well pumping rates, the proposed sheet pile wall was simulated by adding a vertical "wall" to the calibrated model of the Lyman Street Site. In addition, the simulation included replacement recovery well RW-1R in place of RW-1 to evaluate the hydraulic effects of the wall under the current pumping conditions. Two simulations were run to evaluate the potential effect of the wall for a possible range of embedment depths. In one simulation, the wall extended from the water table in layer 1 to a bottom elevation of 950 ft. In the second simulation, the wall extended to a bottom elevation of 960 ft The wall was assumed to have a thickness of 0.25 inches and a hydraulic conductivity of 2.84 X 10^{-5} ft/day (10^{-8} cm/sec). The simulated sheetpile wall runs from well LS-38, located near the Lyman Street bridge, in a northeasterly direction along the bank of the Housatonic River to well LS-24. Figure 4-1 shows the modeled location of the proposed sheetpile wall (flow barrier).

4.2 WATER LEVELS

The addition of the sheetpile wall to the calibrated model resulted in a decrease in water level elevations across the Site. This potential decrease was most pronounced at monitoring wells located closest to the sheetpile wall where model calculated groundwater levels decrease by up to 0.85 ft. Two wells, LS-4 and LS-23, showed increases in model calculated water levels of 0.86 and 0.65 feet. These wells are located next to recovery well RW-1, which was actively pumped in the prewall simulation but was replaced with RW-1R for the wall simulations. Table 4-1 summarizes the potential changes in water level elevation at monitoring wells due to the installation of the wall. The model-calculated potentiometric surface of layer 1 and groundwater flow directions are shown in Figure 4-1 for the simulation with the bottom of the wall at elevation 960 feet. As shown on Table 4-1, the model-calculated groundwater levels with a wall bottom elevation of 950 ft are very similar to the model-calculated water levels with the bottom elevation of the wall at 960 ft. The maximum difference in model-calculated groundwater levels between the simulations was 0.01 ft.

Figure 4-2 is a potentiometric cross-section perpendicular to the wall along column 40 in the model grid for the simulation having a bottom wall elevation of 960 ft. The cross section location is shown on Figure 4-1. Because the water level elevations in the recovery wells are controlled by level sensors, the addition of the sheetpile wall does not significantly change the recovery well capture zones or vertical gradients at the site.

Although site characterization activities indicate that no DNAPL exists near the river, an evaluation of the potential for vertical hydraulic gradients beneath the river to mobilize DNAPL toward the river was performed. With the sheetpile wall in place, the model indicates that the vertical hydraulic gradient immediately beneath the river adjacent to the wall is slightly upward ranging from zero to 0.002. An upward hydraulic gradient can theoretically counteract the downward migration of DNAPL. The minimum hydraulic gradient, i_n, sufficient to prevent DNAPL from sinking vertically downward due to gravity is related to the density contrast between the DNAPL and water. It can be obtained with the following equation (Cohen.and Mercer, 1993):

$$i_n=(\rho_n-\rho_w)/\rho_w$$

where:

i_n= The critical hydraulic gradient necessary to prevent downward DNAPL migration

ρ_n= DNAPL density

ρ_w= Water density

The theoretically derived minimum hydraulic gradient needed to prevent DNAPL from sinking vertically downward is calculated using the equation above and the measured Lyman Street DNAPL densities 1.071 to 1.165 g/ml (HSI GeoTrans, 1999). The resulting minimum upward hydraulic gradient sufficient to counteract downward migration of the

DNAPL is 0.076 to 0.165. Therefore, if the model calculated hydraulic gradient is less than 0.076, then the upward hydraulic gradient cannot cause DNAPL to flow upward towards the river.

The maximum model calculated upward hydraulic gradient beneath the river is 0.002 which is significantly less than the lowest critical hydraulic gradient based on the measured density of Lyman Street DNAPL.

4.3 PUMPING WELL EXTRACTION RATES

As shown in Figure 4-1, all groundwater flow within the lateral limits of the wall is captured by the three recovery wells (RW-1R, RW-2, and RW-3). Additionally, groundwater flow paths beyond the lateral limits of the wall (i.e., east of well LS-29 and west of well LS-34) are captured by the three pumping systems. The model results also indicate that the installation of the wall will reduce the pumping rates of the recovery wells due to the reduction of induced infiltration from the river. The simulated percent reductions in pumping rates for RW-1R, RW-2, and RW-3 were 29 percent, 24 percent, and 20 percent, respectively.

Table 4-1. Effect of sheet pile wall with bottom elevation at 960 and 950 feet on model-calculated water levels

WELL	ROW	COLUMN	LAYER	MODEL-CALCULATED GROUNDWATER ELEVATION		
				NO WALL	BOTTOM ELEVATION OF WALL AT 960 FT.	BOTTOM ELEVATION OF WALL AT 950 FT.
LS-12	20	17	2	972.65	972.59	972.59
LS-34	19	15	3	972.79	972.75	972.75
LS-43	24	9	4	972.36	972.33	972.33
LS-45	33	4	5	971.69	971.68	971.68
LS-44	39	8	4	971.72	971.71	971.71
LS-33	38	21	1	970.53	970.13	970.12
LS-32	37	35	1	970.87	970.54	970.54
LS-2	35	28	1	970.20	970.01	970.01
LS-11	35	39	3	971.64	971.45	971.45
P-6	47	21	1	971.04	970.15	970.15
P-7	45	27	1	970.87	970.01	970.01
P-1	43	49	1	971.03	970.01	970.00
P-2	44	53	1	971.05	970.43	970.43
P-3	42	52	1	970.90	970.05	970.06
P-4	44	59	1	970.92	970.01	970.01
P-5	45	65	1	971.36	971.52	971.52
LS-23	34	40	1	971.46	971.12	971.12
LS-21	35	44	1	971.05	971.09	971.09
LS-20	34	57	1	971.14	971.14	971.14
LS-25	35	57	6	975.32	975.31	975.31
LS-24	35	65	2	971.65	971.57	971.57
LS-31	21	45	2	972.67	972.68	972.68
LS-30	20	35	1	972.58	972.52	972.52
LS-4	36	50	2	969.99	970.84	970.84
LS-41	40	28	1	970.14	970.00	970.00
E-1	20	82	1	973.63	973.64	973.64
E-3	40	87	1	972.49	972.49	972.49
E-4	48	89	2	972.16	972.16	972.16
LS-37	9	78	1	975.52	975.52	975.52
LS-28	14	17	2	973.88	973.86	973.86
LS-10	11	38	2	974.71	974.70	974.70
LS-13	17	47	2	973.63	973.63	973.63
LS-29	17	72	3	973.94	973.95	973.95
LS-36	35	77	2	972.47	972.51	972.51
LS-35	40	44	2	971.00	970.50	970.50
E-7	5	87	1	976.42	976.42	976.42
LS-38	41	18	2	971.15	970.65	970.65

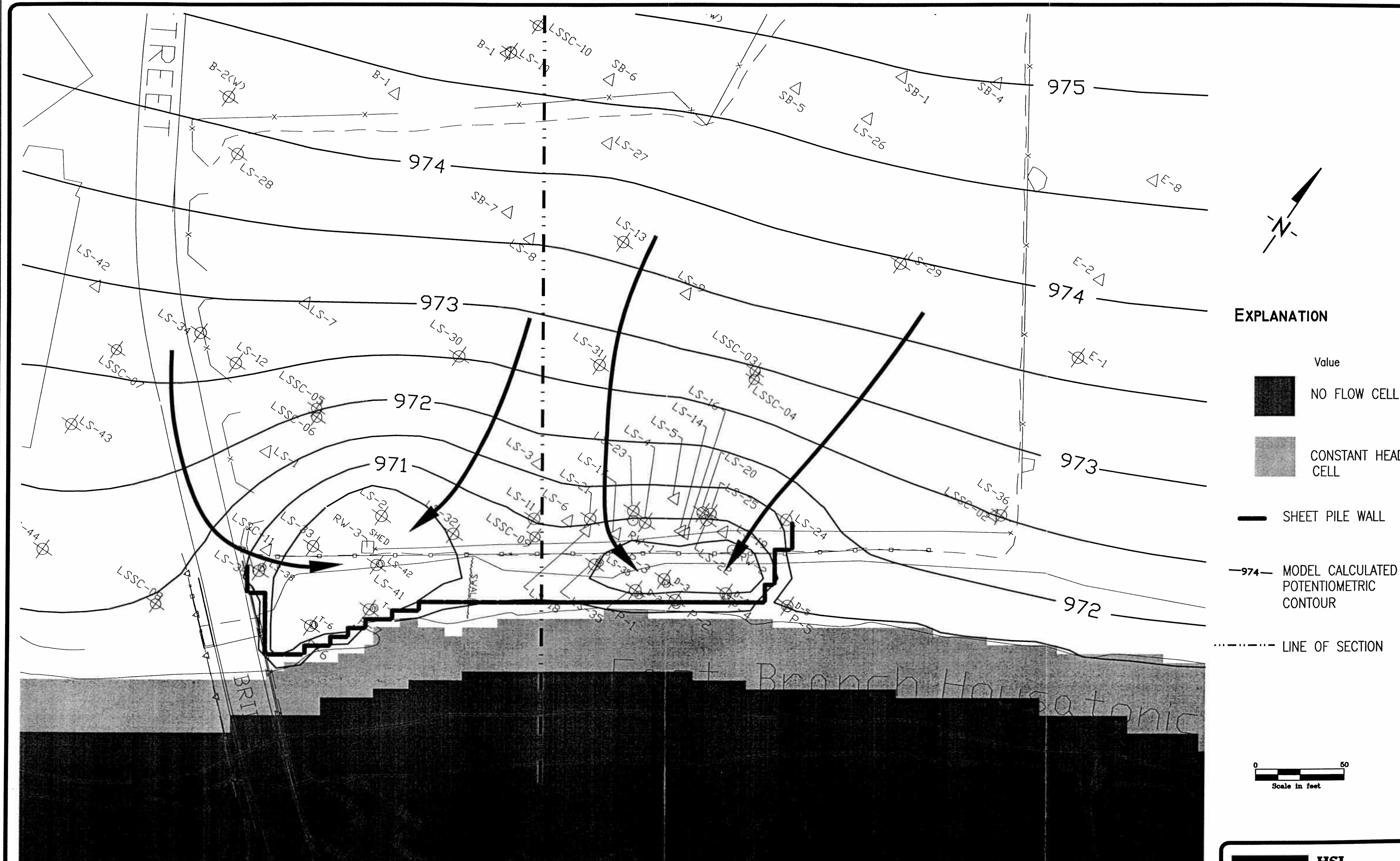

Figure 4-1 Model Calculated Potentiometric Surface with Proposed Sheet Wall

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A TETRA TECH COMPANY
fig-4-1.dwg

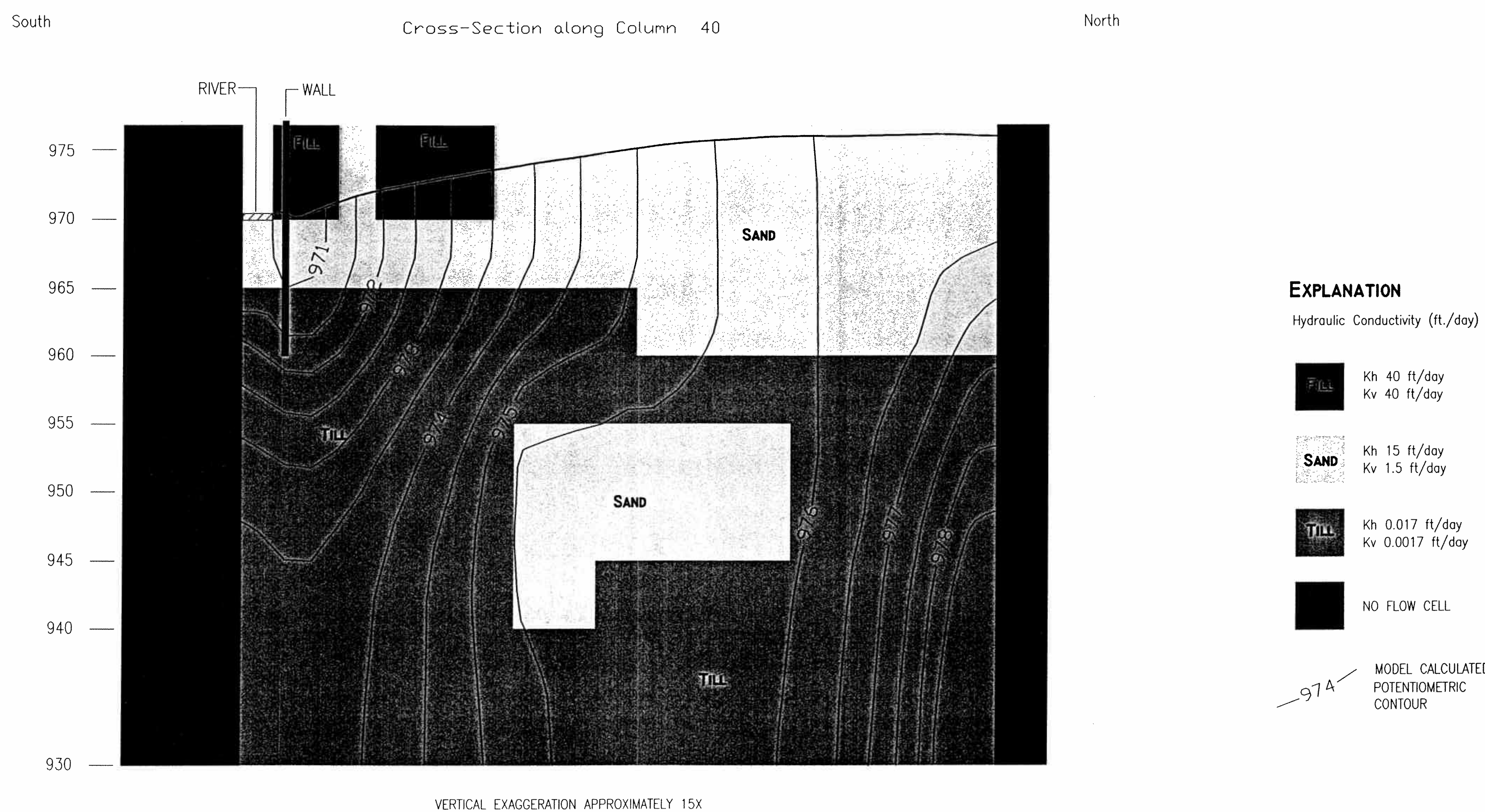

Figure 4–2 Model–Calculated Potentiometric Section Perpendicular to the Sheetpile Wall Grid

HSI GEOTRANS
A TETRA TECH COMPANY
fig-4-2.dwg

5 CONCLUSIONS

Based on the results of the model analyses performed, the proposed sheet pile wall will effectively supplement the existing LNAPL containment systems without negatively affecting the ongoing performance of these systems. The model analyses indicate that installation of the wall will cause a small reduction in the groundwater levels and recovery well pumping rates at the site. The lower pumping rates are due to the reduction in infiltration from the river caused by the installation of the sheetpile wall. The model simulations also indicate that the hydraulic gradients and groundwater flow at the ends of the wall are towards the recovery wells. As a result, there will be no groundwater or LNAPL flow around the ends of the proposed wall. Although no DNAPL has been observed near the river an evaluation of model calculated vertical hydraulic gradients was performed. The model indicates that the small upward hydraulic gradient with the wall in place will not be sufficient to cause DNAPL to migrate upward toward the river even if DNAPL was present.

6 REFERENCES

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Appendix C

BLASLAND, BOUCK & LEE, INC.
engineers & scientists

Sheetpile Geotechnical and Structural Calculations

CLIENT GE SUBJECT Sheetpile Design Calculations Prepared By LHK Date: 6/23/99
Reviewed By RDD Date 7/12/99
PROJECT Lyman Street Source Control Containment Barrier Temporary Case 1

TASK:

To calculate the required embedment depth, maximum moment, and section modulus for a sheetpile wall supporting a 2H:1V slope starting at an elevation of 974.5 feet with soil temporarily excavated to 967.5 feet in front of the wall. The elevation of the top of the wall is 978 feet.

REFERENCES:

1. NAVFAC DM-7, March 1971.
2. Das, B. M. (1990) Principles of Foundation Engineering, 2nd Edition, PWS-Kent Publishing Company.

ASSUMPTIONS:

Soil unit weight = γ = 125 pcf
Buoyant soil unit weight = γ' = 62.6 pcf
Exposed height of sheetpile = 10.5 feet
Height of water behind sheetpile wall above excavation depth = 3.0 feet

CALCULATIONS:

The following calculation method is outlined in Ref. 2 (Sheets 19 through 26).

(1) Determine net pressure diagram:

(a) Calculate K_a and K_p
Using Table 1 from Ref. 1 (Sheet 27), wall friction angle $\delta = 14°$,

For K_p, $\phi = 32°$, $\beta = 0°$, $\delta = -14°$

Using Figure 6 on Sheet 28, for $\beta/\phi = 0°/32° = 0$, and $\delta/\phi = -14°/32° = -0.44$,

$K_p = R*K_p$ (for $\delta/\phi = -1$) = 0.678(7.8) = 5.29

$K_p = 5.29$

For K_a, $\phi = 32°$, $\beta = \tan^{-1}(1/2) = 26.6°$, $\delta = 14°$,

Since Figure 6 does not provide values for $\delta \neq \phi$, use general equation on Sheet 29 instead (with $\theta = 0$).

$$K_a = \cos^2\phi / \{ \cos\delta[1 + (\sin(\phi+\delta) \sin(\phi-\beta)/ (\cos\delta \cos(-\beta)))^{0.5}]^2\}$$
$$= \cos^2(32)/ \{ \cos(14)[1 + (\sin(32+14) \sin(32-26.6)/ (\cos(14) \cos(-26.6)))^{0.5}]^2\}$$

$K_a = 0.45$

(b) Calculate pressures and forces acting on wall.

All of the following calculations are based on the information provided on Sheets 6 and 19 through 26.

(i) Calculate active pressure and water pressure on wall at EL 967.5 ft:

$p_1 = \gamma L_1 K_a$

$p_1 = 226$ psf

$p_2 = p_1 + \gamma' L_2 K_a + \gamma_w L_2$

$p_2 = 499$ psf

(ii) Determine location of zero net pressure as distance below excavation elevation (967.5 ft):

$$L_3 = \frac{p_2}{\gamma'(K_p - K_a)}$$

$L_3 = 1.65$ ft

(iii) Calculate magnitude and location of active force acting on wall, P.

$P = 0.5 p_1 L_1 + 0.5(p_1 + p_2)L_2 + 0.5 p_2 L_3$

P = 1,951 lb

$\sum M_E$ to determine location:

$Pz_1 = 1/2 p_1 L_1 (L_3 + L_2 + L_1/3) + p_1 L_2 (L_3 + L_2/2) + 1/2(p_2 - p_1) L_2 (L_3 + L_2/3) + 1/2 p_2 L_3 (2/3 L_3)$

$z_1 = 3.27$ ft

(iv) Formulate equations for pressures acting at the bottom of the sheetpile wall:

$p_3 = L_4(K_p - K_a)\gamma'$ (1)
$p_4 = \gamma L_1 K_p + \gamma' L_3 (K_p - K_a) + \gamma' L_4 (K_p - K_a) = p_5 + \gamma' L_4 (K_p - K_a)$ (2)
where $p_5 = (\gamma L_1 + \gamma' L_2) K_p + \gamma' L_3 (K_p - K_a)$

$p_5 = 4{,}136$ psf

(c) Satisfy principles of statics.

$\sum F_H = 0$

CLIENT GE SUBJECT Sheetpile Design Calculations Prepared By LHK Date: 6/23/99
Reviewed By RDD Date 7/12/99
PROJECT Lyman Street Source Control Containment Barrier Temporary Case 1

$P-0.5p_3L_4+0.5(p_3+p_4)L_5 = 0$ (3)

Solving Eq. 3 for L_5:

$$L_5=\frac{p_3L_4-2P}{p_3+p_4} \quad (4)$$

$\sum M_B = 0$
$P(L_4+z_1)-(1/2)L_4p_3(L_4/3)+(1/2)L_5(p_3+p_4)(L_5/3) = 0$ (5)

Combining Eqs. 1, 2, 4, and 5 and simplifying yields:

$L_4^4+A_1L_4^3-A_2L_4^2-A_3L_4-A_4 = 0$ (6)

where

$$A_1=\frac{p_5}{\gamma'(K_p-K_a)} \qquad A_2=\frac{8P}{\gamma'(K_p-K_a)}$$

$$A_3=\frac{6P[2z_1\gamma'(K_p-K_a)+p_5]}{(\gamma')^2(K_p-K_a)^2} \qquad A_4=\frac{P(6z_1p_5+4P)}{(\gamma')^2(K_p-K_a)^2}$$

$A_1 = 13.66$; $A_2 = 51.57$; $A_3 = 781$; $A_4 = 1894$
By trial and error:

L_4	Equation	
9	3419	
8	-353	
8.1	-40	**OK**

L_4= 8.1 ft

Using Eqs. 1, 2, and 4 :

$\underline{p_3 = 2{,}452 \text{ psf}}$
$\underline{p_4 = 6{,}588 \text{ psf}}$

$\underline{L_5 = 1.77 \text{ ft}}$

(d) Determine required embedment depth.

$D = L_3 + L_4$
$D = 1.65 + 8.1 = 9.75 \text{ ft}$

Increase D by 10 percent (F.S.=1.25 for temporary construction condition) → **$\underline{D = 10.7 \text{ ft}}$**

(2) Calculate the maximum bending moment.
(a) Determine location of maximum moment as distance from Point E (see Sheets 6 and 19 through 26 for clarification):

$$z' = \sqrt{\frac{2P}{(K_p - K_a)\gamma'}}$$

$\underline{z' = 3.59 \text{ ft}}$

(b) Calculate maximum bending moment:

$M_{max} = P(z_1 + z') - [0.5\gamma'(z')^2(K_p - K_a)](1/3)z'$

$M_{max} = 11{,}050 \text{ lb-ft/ft}$

$\underline{M_{max} = 132{,}596 \text{ lb-in/ft}}$

(3) Calculate required section modulus:

$$S = \frac{M_{max}}{f_b}$$

where f_b = 25 ksi for allowable stress on σ_y = 36 ksi steel.

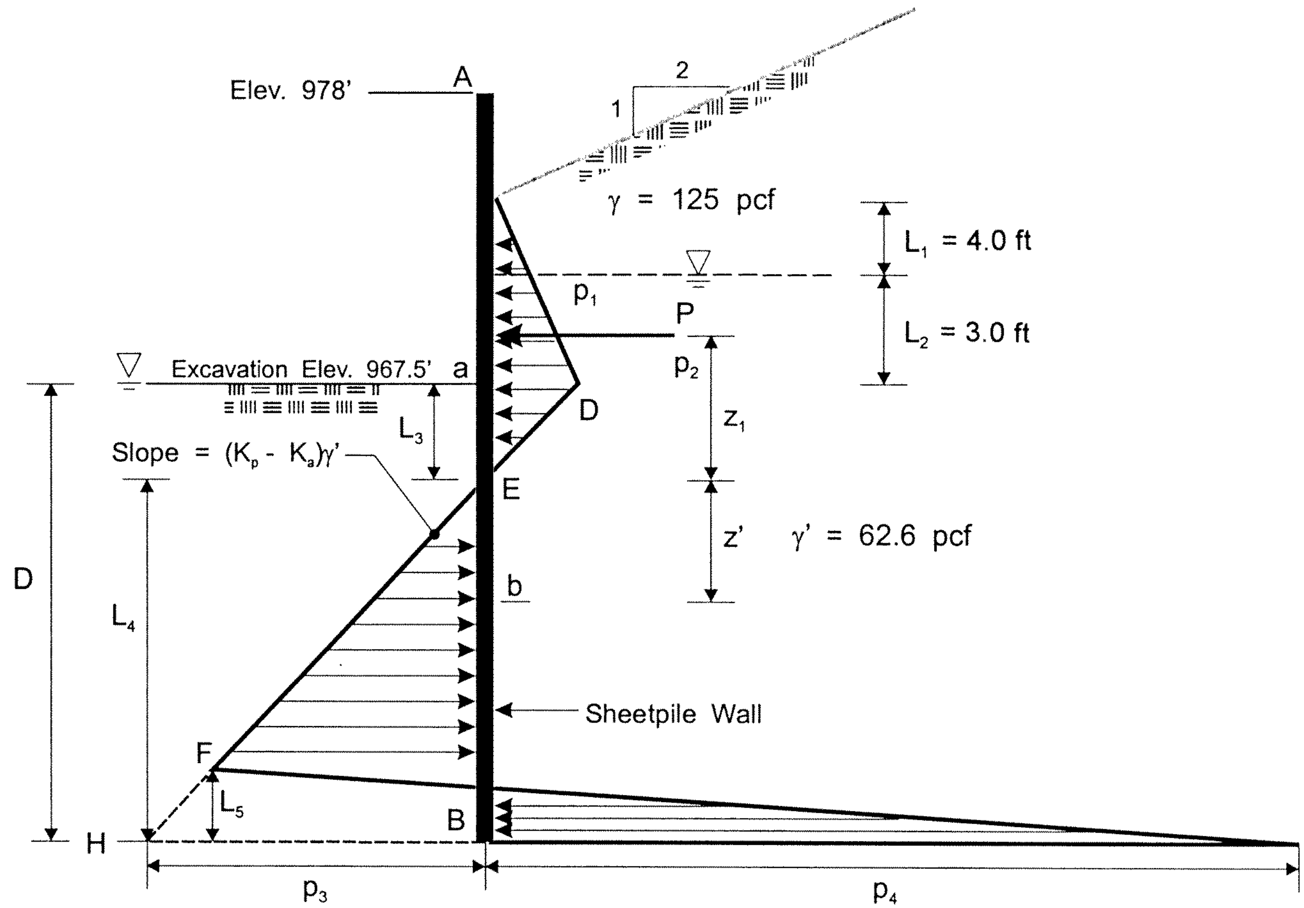
$\underline{S = 5.3 \text{ in}^3}$

The section modulus, S, is less than 24.9 in^3 for WEZ-95, therefore OK.

CLIENT GE SUBJECT Sheetpile Design Calculations Prepared By LHK Date: 6/23/99
Reviewed By RDD Date 7/12/99
PROJECT Lyman Street Source Control Containment Barrier Temporary Case 1

CONCLUSIONS

For an exposed wall height of 10.5 feet with a 2H:1V slope of soil starting at 974.5 feet, the required embedment depth is 10.7 feet for a factor of safety of 1.25 under temporary construction conditions. A 21.2-foot long sheetpile is required. The section modulus of a WEZ-95 sheetpile is acceptable.



NET PRESSURE DIAGRAM - TEMPORARY CASE

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CLIENT GE SUBJECT Sheetpile Design Calculations Prepared By LHK Date: 6/23/99

Reviewed By RDD Date 7/12/99

PROJECT Lyman Street Source Control Containment Barrier Temporary Case 2

TASK:

To calculate the required embedment depth, maximum moment, and section modulus for a sheetpile wall supporting a 2H:1V slope starting at an elevation of 974.5 feet with soil temporarily excavated to 967.5 feet in front of the wall. The elevation of the top of the wall is 977 feet. Note: this configuration is also being used to represent the deepest river excavation, which slopes down from 972.5 feet at the wall to 967 feet in the river.

REFERENCES:

1. NAVFAC DM-7, March 1971.

2. Das, B. M. (1990) Principles of Foundation Engineering, 2nd Edition, PWS-Kent Publishing Company.

ASSUMPTIONS:

Soil unit weight = γ = 125 pcf
Buoyant soil unit weight = γ' = 62.6 pcf
Exposed height of sheetpile = 9.5 feet
Height of water behind sheetpile above excavation depth = 3.0 feet

CALCULATIONS:

The following calculation method is outlined in Ref. 2 (Sheets 19 through 26).

(1) Determine net pressure diagram:

(a) Calculate K_a and K_p
Using Table 1 from Ref. 1 (Sheet 27), wall friction angle $\delta = 14°$,

For K_p, $\phi = 32°$, $\beta = 0°$, $\delta = -14°$

Using Figure 6 on Sheet 28, for $\beta/\phi = 0°/32° = 0$, and $\delta/\phi = -14°/32° = -0.44$,

$K_p = R{*}K_p$ (for $\delta/\phi = -1$) = 0.678(7.8) = 5.29

$K_p = 5.29$

For K_a, $\phi = 32°$, $\beta = \tan^{-1}(1/2) = 26.6°$, $\delta = 14°$,

Since Figure 6 does not provide values for $\delta \neq \phi$, use general equation on Sheet 29 instead (with $\theta = 0$).

$$K_a = \cos^2\phi \,/\, \{ \cos\delta[\, 1 + (\sin(\phi+\delta)\sin(\phi-\beta)/(\cos\delta\cos(-\beta)))^{0.5}]^2\}$$
$$= \cos^2(32) / \{ \cos(14)[\, 1 + (\sin(32+14)\sin(32-26.6)/(\cos(14)\cos(-26.6)))^{0.5}]^2\}$$

$K_a = 0.45$

p_3 = 2,452 psf
p_4 = 6,588 psf

L_5 = 1.77 ft

(d) Determine required embedment depth.

$D = L_3 + L_4$
$D = 1.65 + 8.1 = 9.7$ ft

Increase D by 10 percent (F.S.=1.25 for temporary construction condition) → **D = 10.7 ft**

(2) Calculate the maximum bending moment.
(a) Determine location of maximum moment as distance from Point E (see Sheets 6f and 19 through 26 for clarification):

$$z' = \sqrt{\frac{2P}{(K_p - K_a)\gamma'}}$$

z' = 3.59 ft

(b) Calculate maximum bending moment:

$M_{max} = P(z_1 + z') - [0.5\gamma'(z')^2(K_p - K_a)](1/3)z'$

M_{max} = 11,050 lb-ft/ft

M_{max} = 132,596 lb-in/ft

(3) Calculate required section modulus:

$$S = \frac{M_{max}}{f_b}$$

where f_b = 25 ksi for allowable stress on σ_y = 36 ksi steel.

S=5.3 in^3

The section modulus, S, is less than 24.9 in^3 for WEZ-95, therefore OK.

CLIENT GE SUBJECT Sheetpile Design Calculations Prepared By LHK Date: 6/23/99

Reviewed By RDD Date 7/12/99

PROJECT Lyman Street Source Control Containment Barrier Temporary Case 2

$$P-0.5p_3L_4+0.5(p_3+p_4)L_5 = 0 \qquad (3)$$

Solving Eq. 3 for L_5:

$$L_5=\frac{p_3L_4-2P}{p_3+p_4} \qquad (4)$$

$\sum M_B = 0$

$$P(L_4+z_1)-(1/2)L_4p_3(L_4/3)+(1/2)L_5(p_3+p_4)(L_5/3) = 0 \qquad (5)$$

Combining Eqs. 1, 2, 4, and 5 and simplifying yields:

$$L_4^4+A_1L_4^3-A_2L_4^2-A_3L_4-A_4 = 0 \qquad (6)$$

where

$$A_1=\frac{p_5}{\gamma'(K_p-K_a)} \qquad A_2=\frac{8P}{\gamma'(K_p-K_a)}$$

$$A_3=\frac{6P[2z_1\gamma'(K_p-K_a)+p_5]}{(\gamma')^2(K_p-K_a)^2} \qquad A_4=\frac{P(6z_1p_5+4P)}{(\gamma')^2(K_p-K_a)^2}$$

$A_1 = 13.66$; $A_2 = 51.57$; $A_3 = 781$; $A_4 = 1894$

By trial and error:

L_4	Equation	
9	3419	
8	-353	
8.1	-40	**OK**

L_4= 8.1 ft

Using Eqs. 1, 2, and 4 :

(b) Calculate pressures and forces acting on wall.

All of the following calculations are based on the information provided on Sheets 6f and 19 through 26.

(i) Calculate active pressure and water pressure on wall at EL 967.5 ft:

$p_1 = \gamma L_1 K_a$

$\underline{p_1 = 226 \text{ psf}}$

$p_2 = p_1 + \gamma' L_2 K_a + \gamma_w L_2$

$\underline{p_2 = 499 \text{ psf}}$

(ii) Determine location of zero net pressure as distance below excavation elevation (967.5 ft):

$$L_3 = \frac{p_2}{\gamma'(K_p - K_a)}$$

$\underline{L_3 = 1.65 \text{ ft}}$

(iii) Calculate magnitude and location of active force acting on wall, P.

$P = 0.5p_1L_1 + 0.5(p_1+p_2)L_2 + 0.5p_2L_3$

$\underline{P = 1{,}951 \text{ lb}}$

$\sum M_E$ to determine location:

$Pz_1 = 1/2p_1L_1(L_3+L_2+L_1/3) + p_1L_2(L_3+L_2/2) + 1/2(p_2-p_1)L_2(L_3+L_2/3) + 1/2p_2L_3(2/3L_3)$

$\underline{z_1 = 3.27 \text{ ft}}$

(iv) Formulate equations for pressures acting at the bottom of the sheetpile wall:

$p_3 = L_4(K_p-K_a)\gamma'$ (1)
$p_4 = \gamma L_1 K_p + \gamma' L_3(K_p-K_a) + \gamma' L_4(K_p-K_a) = p_5 + \gamma' L_4(K_p-K_a)$ (2)
where $p_5 = (\gamma L_1 + \gamma' L_2)K_p + \gamma' L_3(K_p-K_a)$

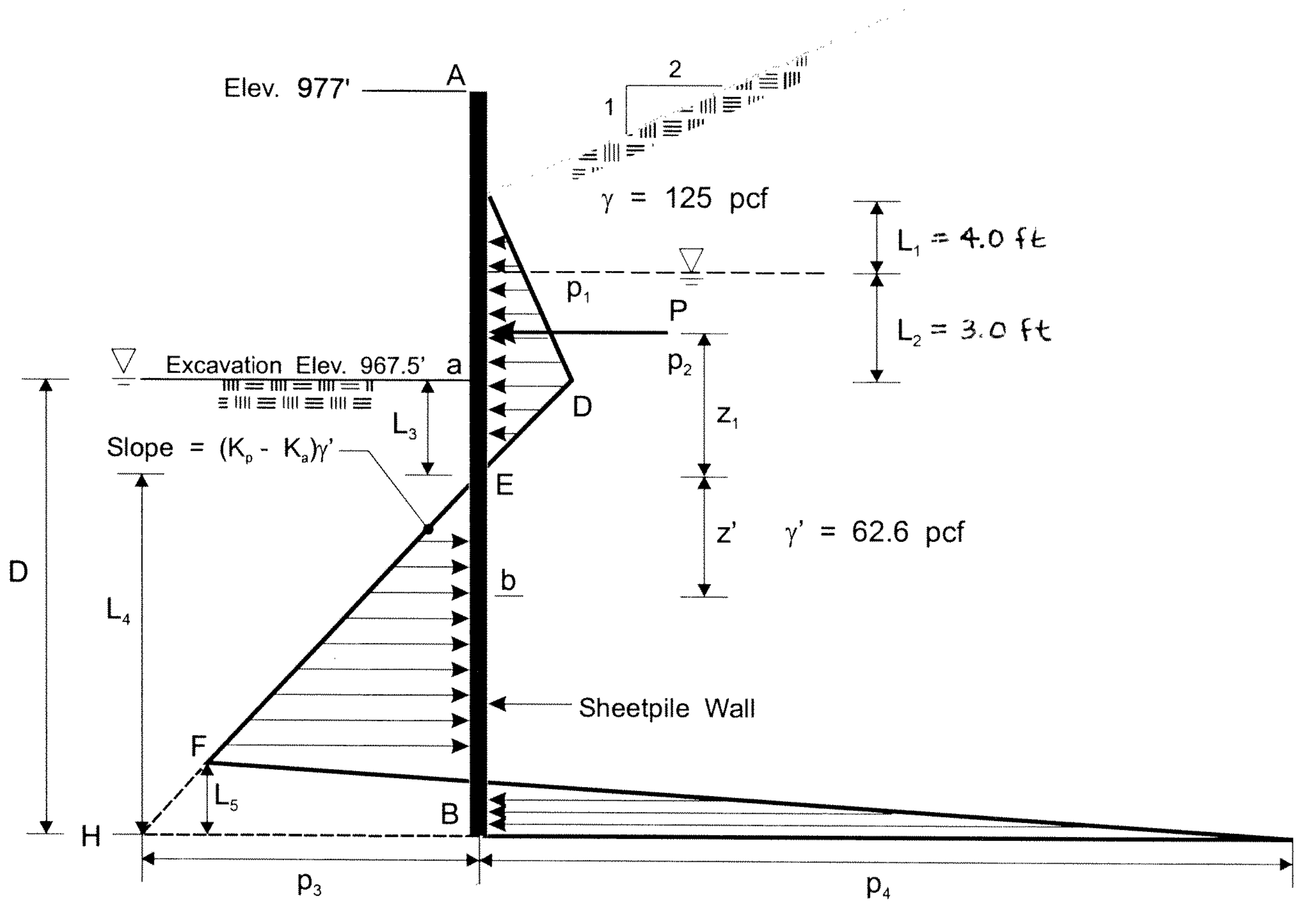
$\underline{p_5 = 4{,}136 \text{ psf}}$

(c) Satisfy principles of statics.

$\sum F_H = 0$

CONCLUSIONS

For an exposed wall height of 9.5 feet with a 2H:1V slope of soil starting at 974.5 feet, the required embedment depth is 10.7 feet for a factor of safety of 1.25 under temporary construction conditions. A 20.2-foot long sheetpile is required. The section modulus of a WEZ-95 sheetpile is acceptable.



NET PRESSURE DIAGRAM - TEMPORARY CASE

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TASK:

To calculate the required embedment depth, maximum moment, and section modulus for a sheetpile wall supporting a 2H:1V slope above the wall and a soil elevation of 970 feet in front of the wall. The elevation of the top of the wall is 978 feet.

REFERENCES:

1. NAVFAC DM-7, March 1971.

2. Das, B. M. (1990) Principles of Foundation Engineering, 2nd Edition, PWS-Kent Publishing Company.

ASSUMPTIONS:

Soil unit weight = γ = 125 pcf
Buoyant soil unit weight = γ' = 62.6 pcf
Exposed height of sheetpile = 8.0 feet

CALCULATIONS:

The following calculation method is outlined in Ref. 2 (Sheets 19 through 26).

(1) Determine net pressure diagram:

(a) Calculate K_a and K_p
Using Table 1 from Ref. 1 (Sheet 27), wall friction angle $\delta = 14°$,

For K_p, $\phi = 32°$, $\beta = 0°$, $\delta = -14°$

Using Figure 6 on Sheet 28, for $\beta/\phi = 0°/32° = 0$, and $\delta/\phi = -14°/32° = -0.44$,

$K_p = R*K_p$ (for $\delta/\phi = -1$) = 0.678(7.8) = 5.29

$K_p = 5.29$

For K_a, $\phi = 32°$, $\beta = \tan^{-1}(1/2) = 26.6°$, $\delta = 14°$,

Since Figure 6 does not provide values for $\delta \neq \phi$, use general equation on Sheet 29 instead (with $\theta = 0$).

$K_a = \cos^2\phi / \{ \cos\delta[1 + (\sin(\phi+\delta) \sin(\phi-\beta)/ (\cos\delta \cos(-\beta)))^{0.5}]^2\}$
$= \cos^2(32)/ \{ \cos(14)[1 + (\sin(32+14) \sin(32-26.6)/ (\cos(14) \cos(-26.6)))^{0.5}]^2\}$

$K_a = 0.45$

(b) Calculate pressures and forces acting on wall.

All of the following calculations are based on the information provided on Sheets 12 and 19 through 26.

(i) Calculate active pressure on wall:

$p_1 = \gamma L_1 K_a$

$\underline{p_1 = 425 \text{ psf}}$

$p_2 = p_1 + \gamma' L_2 K_a + \gamma_w L_2$

$\underline{p_2 = 439 \text{ psf}}$

(ii) Determine location of zero net pressure as distance below river bottom elevation:

$$L_3 = \frac{p_2}{\gamma'(K_p - K_a)}$$

$\underline{L_3 = 1.45 \text{ ft}}$

(iii) Calculate magnitude and location of active force acting on wall, P.

$P = 0.5 p_1 L_1 + 0.5(p_1 + p_2) L_2 + 0.5 p_2 L_3$

$\underline{P = 2{,}126 \text{ lb}}$

$\sum M_E$ to determine location:

$Pz_1 = 1/2 p_1 L_1 (L_3 + L_2 + L_1/3) + p_1 L_2 (L_3 + L_2/2) + 1/2 (p_2 - p_1) L_2 (L_3 + L_2/3) + 1/2 p_2 L_3 (2/3 L_3)$

$\underline{z_1 = 3.65 \text{ ft}}$

(iv) Formulate equations for pressures acting at the bottom of the sheetpile wall:

$p_3 = L_4(K_p - K_a)\gamma'$ (1)
$p_4 = \gamma L_1 K_p + \gamma' L_3 (K_p - K_a) + \gamma' L_4 (K_p - K_a) = p_5 + \gamma' L_4 (K_p - K_a)$ (2)
where $p_5 = (\gamma L_1 + \gamma' L_2) K_p + \gamma' L_3 (K_p - K_a)$

$\underline{p_5 = 5{,}562 \text{ psf}}$

(c) Satisfy principles of statics.

$\sum F_H = 0$

CLIENT GE SUBJECT Sheetpile Design Calculations Prepared By LHK Date: 6/23/99
Reviewed By RDD Date 7/12/99
PROJECT Lyman Street Source Control Containment Barrier Long Term Case 1

$P-0.5p_3L_4+0.5(p_3+p_4)L_5 = 0$ (3)

Solving Eq. 3 for L_5:

$$L_5=\frac{p_3L_4-2P}{p_3+p_4} \quad (4)$$

$\sum M_B = 0$

$P(L_4+z_1)-(1/2)L_4p_3(L_4/3)+(1/2)L_5(p_3+p_4)(L_5/3) = 0$ (5)

Combining Eqs. 1, 2, 4, and 5 and simplifying yields:

$L_4^4+A_1L_4^3-A_2L_4^2-A_3L_4-A_4 = 0$ (6)

where

$$A_1=\frac{p_5}{\gamma'(K_p-K_a)} \qquad A_2=\frac{8P}{\gamma'(K_p-K_a)}$$

$$A_3=\frac{6P[2z_1\gamma'(K_p-K_a)+p_5]}{(\gamma')^2(K_p-K_a)^2} \qquad A_4=\frac{P(6z_1p_5+4P)}{(\gamma')^2(K_p-K_a)^2}$$

$A_1 = 18.37$; $A_2 = 56.18$; $A_3 = 1{,}082$; $A_4 = 3{,}023$

By trial and error:

L_4	Equation	
9	2646	
8	-1769	
8.4	-205	**OK**

L_4= 8.4 ft

Using Eqs. 1, 2, and 4 :

$p_3 = 2{,}543$ psf
$p_4 = 8{,}105$ psf

$L_5 = 1.61$ ft

(d) Determine required embedment depth.

$D = L_3 + L_4$
$D = 1.45 + 8.4 = 9.9$ ft

Increase D by 20 percent (F.S.=1.50) → **$D = 11.8$ ft**

(2) Calculate the maximum bending moment.
(a) Determine location of maximum moment as distance from Point E (see Sheets 12 and 19 through 26 for clarification):

$$z' = \sqrt{\frac{2P}{(K_p - K_a)\gamma'}}$$

$z' = 3.75$ ft

(b) Calculate maximum bending moment:

$M_{max} = P(z_1 + z') - [0.5\gamma'(z')^2(K_p - K_a)](1/3)z'$

$M_{max} = 13{,}069$ lb-ft/ft

$M_{max} = 156{,}830$ lb-in/ft

(3) Calculate required section modulus:

$$S = \frac{M_{max}}{f_b}$$

where $f_b = 25$ ksi for allowable stress on $\sigma_y = 36$ ksi steel.

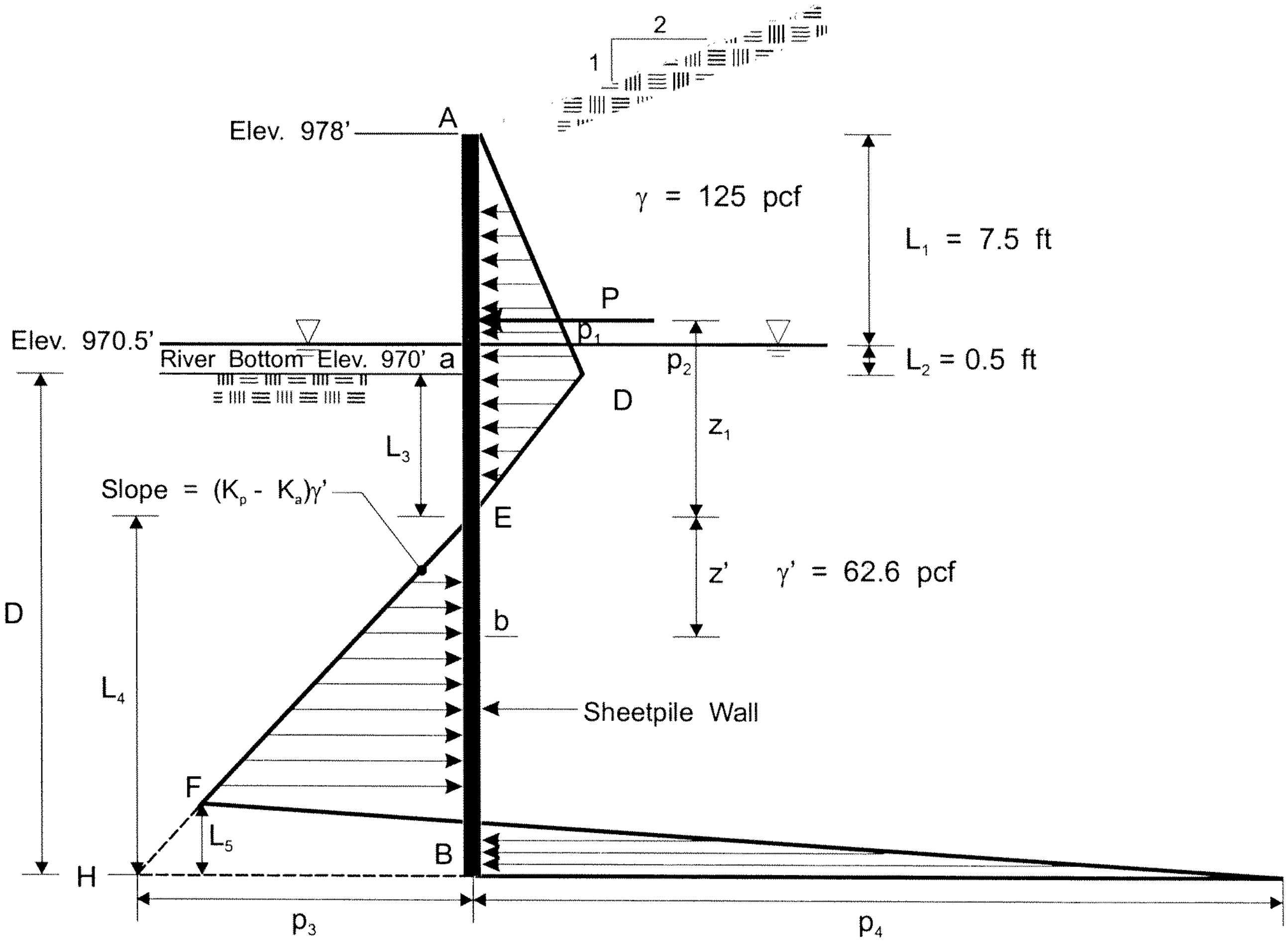
$S = 6.3$ in^3

The section modulus, S, is less than 24.9 in^3 for WEZ-95, therefore OK.

CLIENT GE SUBJECT Sheetpile Design Calculations Prepared By LHK Date: 6/23/99
Reviewed By RDD Date 7/12/99
PROJECT Lyman Street Source Control Containment Barrier Long Term Case 1

CONCLUSIONS

For an exposed wall height of 8.0 feet with a 2H:1V slope behind the wall, the required embedment depth is 11.8 feet for a factor of safety of 1.50. Rounded to the nearest foot, a 20-foot long sheetpile is required. The section modulus of a WEZ-95 sheetpile is acceptable.



NET PRESSURE DIAGRAM - LONG TERM CASE

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12

CLIENT GE SUBJECT Sheetpile Design Calculations Prepared By LHK Date: 7/09/99
Reviewed By RDD Date 7/12/99
PROJECT Lyman Street Source Control Containment Barrier Permanent Case 2

TASK:

To calculate the required embedment depth, maximum moment, and section modulus for a sheetpile wall supporting a 2H:1V slope with soil at an elevation of 969 feet in front of the wall. The elevation of the top of the wall is 977 feet.

REFERENCES:

1. NAVFAC DM-7, March 1971.

2. Das, B. M. (1990) Principles of Foundation Engineering, 2nd Edition, PWS-Kent Publishing Company.

ASSUMPTIONS:

Soil unit weight = γ = 125 pcf
Buoyant soil unit weight = γ' = 62.6 pcf
Exposed height of sheetpile = 8.0 feet

CALCULATIONS:

The following calculation method is outlined in Ref. 2 (Sheets 19 through 26).

(1) Determine net pressure diagram:

(a) Calculate K_a and K_p
Using Table 1 from Ref. 1 (Sheet 27), wall friction angle $\delta = 14°$,

For K_p, $\phi = 32°$, $\beta = 0°$, $\delta = -14°$

Using Figure 6 on Sheet 28, for $\beta/\phi = 0°/32° = 0$, and $\delta/\phi = -14°/32° = -0.44$,

$K_p = R^*K_p$ (for $\delta/\phi = -1$) = 0.678(7.8) = 5.29

K_p = 5.29

For K_a, $\phi = 32°$, $\beta = \tan^{-1}(1/2) = 26.6°$, $\delta = 14°$,

Since Figure 6 does not provide values for $\delta \neq \phi$, use general equation on Sheet 29 instead (with $\theta = 0$).

$K_a = \cos^2\phi \,/\, \{ \cos\delta[1 + (\sin(\phi+\delta) \sin(\phi-\beta)/ (\cos\delta \cos(-\beta)))^{0.5}]^2\}$
$= \cos^2(32)/ \{ \cos(14)[1 + (\sin(32+14) \sin(32-26.6)/ (\cos(14) \cos(-26.6)))^{0.5}]^2\}$

K_a = 0.45

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Reviewed By RDD Date 7/12/99
PROJECT Lyman Street Source Control Containment Barrier Permanent Case 2

(b) Calculate pressures and forces acting on wall.

All of the following calculations are based on the information provided on Sheets 18 and 19 through 26.

(i) Calculate active pressure on wall:

$p_1 = \gamma L_1 K_a$

$\underline{\mathbf{p_1 = 368\ psf}}$

$p_2 = p_1 + \gamma' L_2 K_a + \gamma_w L_2$

$\underline{\mathbf{p_2 = 410\ psf}}$

(ii) Determine location of zero net pressure as distance below excavation elevation:

$$L_3 = \frac{p_2}{\gamma'(K_p - K_a)}$$

$\underline{\mathbf{L_3 = 1.36\ ft}}$

(iii) Calculate magnitude and location of active force acting on wall, P.

$P = 0.5p_1L_1 + 0.5(p_1+p_2)L_2 + 0.5p_2L_3$

$\underline{\mathbf{P = 2{,}058\ lb}}$

$\sum M_E$ to determine location:

$Pz_1 = 1/2p_1L_1(L_3+L_2+L_1/3) + p_1L_2(L_3+L_2/2) + 1/2(p_2-p_1)L_2(L_3+L_2/3) + 1/2p_2L_3(2/3L_3)$

$\underline{\mathbf{z_1 = 3.63\ ft}}$

(iv) Formulate equations for pressures acting at the bottom of the sheetpile wall:

$p_3 = L_4(K_p-K_a)\gamma'$ (1)
$p_4 = \gamma L_1K_p + \gamma' L_3(K_p-K_a) + \gamma' L_4(K_p-K_a) = p_5 + \gamma' L_4(K_p-K_a)$ (2)
where $p_5 = (\gamma L_1 + \gamma' L_2)K_p + \gamma' L_3(K_p-K_a)$

$\underline{\mathbf{p_5 = 5{,}204\ psf}}$

(c) Satisfy principles of statics.

$\sum F_H = 0$

$P-0.5p_3L_4+0.5(p_3+p_4)L_5 = 0$ (3)

Solving Eq. 3 for L_5:

$$L_5=\frac{p_3L_4-2P}{p_3+p_4} \quad (4)$$

$\sum M_B = 0$

$P(L_4+z_1)-(1/2)L_4p_3(L_4/3)+(1/2)L_5(p_3+p_4)(L_5/3) = 0$ (5)

Combining Eqs. 1, 2, 4, and 5 and simplifying yields:

$L_4^4+A_1L_4^3-A_2L_4^2-A_3L_4-A_4 = 0$ (6)

where

$$A_1=\frac{p_5}{\gamma'(K_p-K_a)} \qquad A_2=\frac{8P}{\gamma'(K_p-K_a)}$$

$$A_3=\frac{6P[2z_1\gamma'(K_p-K_a)+p_5]}{(\gamma')^2(K_p-K_a)^2} \qquad A_4=\frac{P(6z_1p_5+4P)}{(\gamma')^2(K_p-K_a)^2}$$

$A_1 = 17.19$; $A_2 = 54.39$; $A_3 = 998$; $A_4 = 2734$

By trial and error:

L_4	Equation	
9	2974	
8	-1299	
8.3	-186	**OK**

L_4= 8.3 ft

Using Eqs. 1, 2, and 4 :

p_3 = 2,512 psf
p_4 = 7,716 psf

L_5 =1.64 ft

(d) Determine required embedment depth.

$D = L_3+L_4$
$D = 1.36 + 8.3 = 9.7$ ft

Increase D by 20 percent (F.S.=1.50 for temporary construction condition) → **D = 11.6 ft**

(2) Calculate the maximum bending moment.
(a) Determine location of maximum moment as distance from Point E (see Sheets 18 and 19 through 26 for clarification):

$$z' = \sqrt{\frac{2P}{(K_p - K_a)\gamma'}}$$

z' = 3.69 ft

(b) Calculate maximum bending moment:

$M_{max} = P(z_1+z') - [0.5\gamma'(z')^2(K_p - K_a)](1/3)z'$

M_{max} = 12,539 lb-ft/ft

M_{max} = 150,464 lb-in/ft

(3) Calculate required section modulus:

$$S = \frac{M_{max}}{f_b}$$

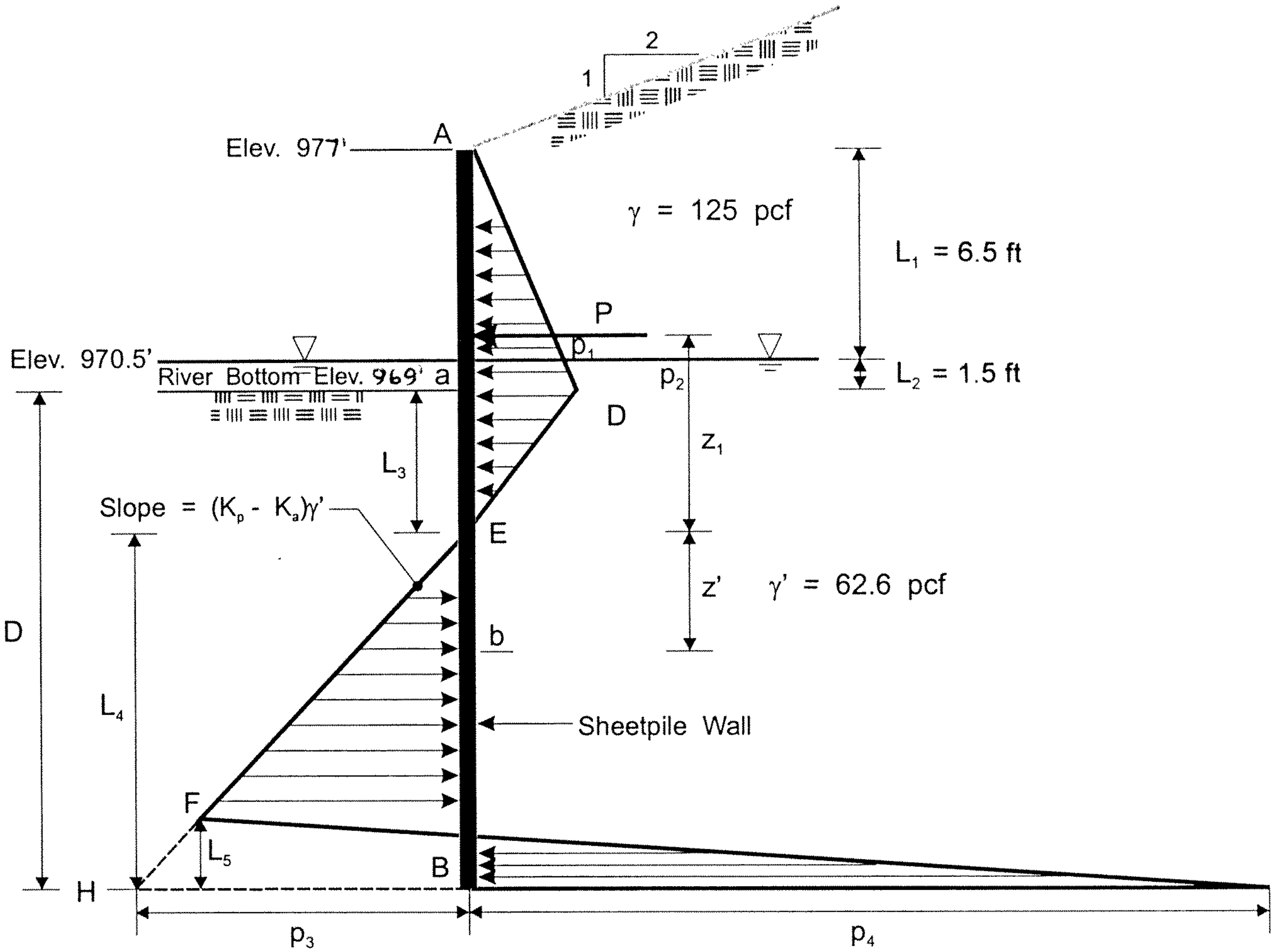
where f_b = 25 ksi for allowable stress on σ_y = 36 ksi steel.

S=6.0 in^3

The section modulus, S, is less than 24.9 in^3 for WEZ-95, therefore OK.

CONCLUSIONS

For an exposed wall height of 8 feet with a 2H:1V slope of soil behind the wall, the required embedment depth is 11.6 feet for a factor of safety of 1.50. A 19.6-foot long sheetpile is required. The section modulus of a WEZ-95 sheetpile is acceptable.



NET PRESSURE DIAGRAM - LONG TERM CASE

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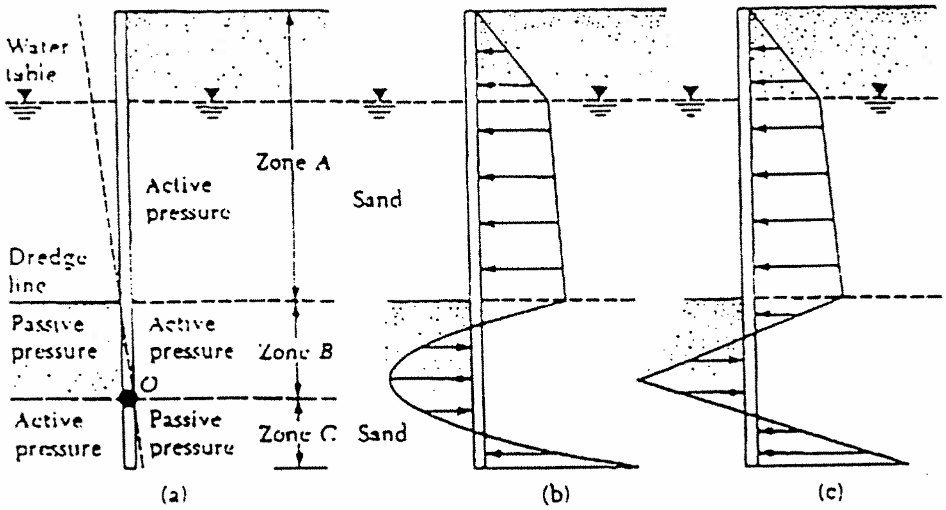

Figure 6.6 Cantilever sheet pile penetrating sand

The following sections (Sections 6.3 through 6.6) present the mathematical formulation of the analysis of cantilever sheet pile walls. Note that, in some waterfront structures, the water level may fluctuate as the result of tidal effects. Care should be taken in determining the water level that will affect the net pressure diagram.

6.3 Cantilever Sheet Piling Penetrating Sandy Soils

To develop the relationships for the proper depth of embedment of sheet piles driven into a granular soil, we refer to Figure 6.7a. The soil retained by the sheet piling above the dredge line is also sand. The water table is located at a depth of L_1 below the top of the wall. Let the angle of friction of the sand be ϕ. The intensity of the active pressure at a depth $z = L_1$ can be given as

$$p_1 = \gamma L_1 K_a \tag{6.1}$$

where K_a = Rankine active pressure coefficient = $\tan^2(45 - \phi/2)$
γ = unit weight of soil above the water table

Similarly, the active pressure at a depth of $z = L_1 + L_2$ (that is, at the level of the dredge line) is equal to

$$p_2 = (\gamma L_1 + \gamma' L_2)K_a \tag{6.2}$$

where γ' = effective unit weight of soil = $\gamma_{sat} - \gamma_w$

Note that, at the level of the dredge line, the hydrostatic pressures from both sides of the wall are of the same magnitude and cancel each other.

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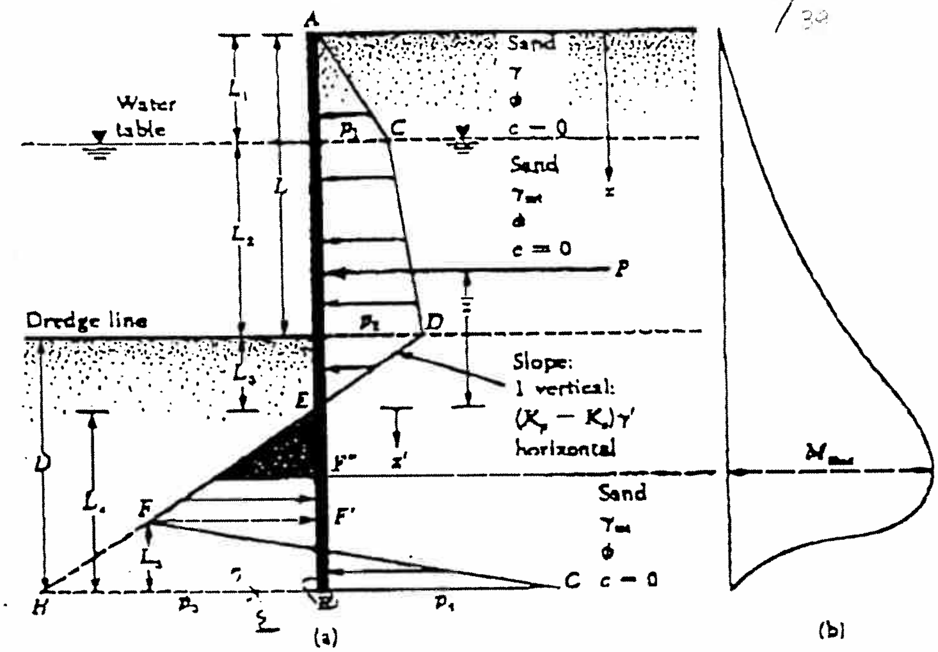

Figure 6.7 Cantilever sheet pile penetrating sand: (a) variation of net pressure diagram. (b) variation of moment

In order to determine the net lateral pressure below the dredge line up to the point of rotation O, as shown in Figure 6.6a, one has to consider the passive pressure acting from the left side (water side) toward the right side (land side) and also the active pressure acting from the right side toward the left side of the wall. For such cases, ignoring the hydrostatic pressure from both sides of the wall, the active pressure at a depth z can be given as

$$p_a = [\gamma L_1 + \gamma' L_2 + \gamma'(z - L_1 - L_2)]K_a \tag{6.3}$$

Also, the passive pressure at that depth z is equal to

$$p_p = \gamma'(z - L_1 - L_2)K_p \tag{6.4}$$

where K_p = Rankine passive pressure coefficient $= \tan^2(45 + \phi/2)$

Hence, combining Eqs. (6.3) and (6.4), the net lateral pressure can be obtained as

$$\begin{aligned} p = p_a - p_p &= (\gamma L_1 + \gamma' L_2)K_a - \gamma'(z - L_1 - L_2)(K_p - K_a) \\ &= p_2 - \gamma'(z - L)(K_p - K_a) \end{aligned} \tag{6.5}$$

where $L = L_1 + L_2$

The net pressure, p, becomes equal to zero at a depth L_3 below the dredge line; or

$$p_2 - \gamma'(z - L)(K_p - K_a) = 0$$

or

$$(z - L) = L_3 = \frac{p_2}{\gamma'(K_p - K_a)} \tag{6.6}$$

From the preceding equation, it is apparent that the slope of the net pressure distribution line DEF is 1 vertical to $(K_p - K_a)\gamma'$ horizontal. So, in the pressure diagram

$$\overline{HB} = p_3 = L_4(K_p - K_a)\gamma' \tag{6.7}$$

At the bottom of the sheet pile, passive pressure (p_p) acts from the right toward the left side, and active pressure acts from the left toward the right side of the sheet pile. So, at $z = L + D$

$$p_p = (\gamma L_1 + \gamma' L_2 + \gamma' D)K_p \tag{6.8}$$

At the same depth

$$p_a = \gamma' D K_a \tag{6.9}$$

Hence, the net lateral pressure at the bottom of the sheet pile is equal to

$$\begin{aligned} p_p - p_a = p_4 &= (\gamma L_1 + \gamma' L_2)K_p + \gamma' D(K_p - K_a) \\ &= (\gamma L_1 + \gamma' L_2)K_p + \gamma' L_3(K_p - K_a) + \gamma' L_4(K_p - K_a) \\ &= p_5 + \gamma' L_4(K_p - K_a) \end{aligned} \tag{6.10}$$

where $p_5 = (\gamma L_1 + \gamma' L_2)K_p + \gamma' L_3(K_p - K_a)$ (6.11)

$D = L_3 + L_4$ (6.12)

For the stability of the wall, the principles of statics can now be applied; or

$$\sum \text{horizontal forces per unit length of wall} = 0$$

and

$$\sum \text{moment of the forces per unit length of wall about point } B = 0$$

For summation of the horizontal forces,

area of the pressure diagram $ACDE$ − area of $EFHB$ + area of $FHBG = 0$

or

$$P - \tfrac{1}{2}p_3 L_4 + \tfrac{1}{2}L_5(p_3 + p_4) = 0 \tag{6.13}$$

where P = area of the pressure diagram $ACDE$

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Summing the moment of all the forces about point B

$$P(L_4 + \bar{z}) - \left(\frac{1}{2} L_4 p_3\right)\left(\frac{L_4}{3}\right) + \frac{1}{2} L_5(p_3 + p_4)\left(\frac{L_5}{3}\right) = 0 \tag{6.14}$$

From Eq. (6.13)

$$L_5 = \frac{p_3 L_4 - 2P}{p_3 + p_4} \tag{6.15}$$

Combining Eqs. (6.7), (6.10), (6.14), and (6.15) and simplifying them further, one obtains the following fourth-degree equation in terms of L_4.

$$\boxed{L_4^4 + A_1 L_4^3 - A_2 L_4^2 - A_3 L_4 - A_4 = 0} \tag{6.16}$$

where

$$A_1 = \frac{p_5}{\gamma'(K_p - K_a)} \tag{6.17}$$

$$A_2 = \frac{8P}{\gamma'(K_p - K_a)} \tag{6.18}$$

$$A_3 = \frac{6P[2\bar{z}\gamma'(K_p - K_a) + p_5]}{\gamma'^2(K_p - K_a)^2} \tag{6.19}$$

$$A_4 = \frac{P(6\bar{z}p_5 + 4P)}{\gamma'^2(K_p - K_a)^2} \tag{6.20}$$

Step-by-Step Procedure for Obtaining the Pressure Diagram

Based on the preceding theory, the step-by-step procedure for obtaining the pressure diagram for a cantilever sheet pile wall penetrating a granular soil is as follows:

1. Calculate K_a and K_p.
2. Calculate p_1 [Eq. (6.1)] and p_2 [Eq. (6.2)]. *Note:* L_1 and L_2 will be given.
3. Calculate L_3 [Eq. (6.6)].
4. Calculate P.
5. Calculate $\bar{z}$ (that is, the center of pressure for the area $ACDE$) by taking the moment about E.
6. Calculate p_5 [Eq. (6.11)].
7. Calculate A_1, A_2, A_3, and A_4 [Eqs. (6.17) to (6.20)].
8. Solve Eq. (6.16) by trial and error to determine L_4.
9. Calculate p_4 [Eq. (6.10)].

10. Calculate p_5 [Eq. (6.7)].
11. Obtain L_5 from Eq. (6.15).
12. Now the pressure distribution diagram as shown in Figure 6.7a can easily be drawn.
13. Obtain the theoretical depth [Eq. (6.12)] of penetration as $L_3 + L_4$. The actual depth of penetration is increased by about 20–30%.

Note: Some designers prefer to use a factor of safety on the passive earth pressure coefficient at the beginning. In that case, in Step 1

$$K_{p(\text{design})} = \frac{K_p}{FS}$$

where FS = factor of safety (usually between 1.5 to 2)

For this type of analysis, follow Steps 1 through 12 with the value of $K_a = \tan^2(45 - \phi/2)$ and $K_{p(\text{design})}$ (instead of K_p). The actual depth of penetration can now be determined by adding L_3, obtained from Step 3, and L_4, obtained from Step 8.

Calculation of Maximum Bending Moment

The nature of variation of the moment diagram for a cantilever sheet pile wall is shown in Figure 6.7b. The maximum moment will occur between the points E and F'. To obtain the maximum moment (M_{max}) per unit length of the wall, one must determine the point of zero shear. Adopting a new axis z' (with origin at point E) for zero shear

$$P = \tfrac{1}{2}(z')^2(K_p - K_a)\gamma'$$

or

$$z' = \sqrt{\frac{2P}{(K_p - K_a)\gamma'}} \tag{6.21}$$

Once the point of zero shear force is determined (point F'' in Figure 6.7a), the magnitude of the maximum moment can be obtained as

$$\boxed{M_{max} = P(\bar{z} + z') - [\tfrac{1}{2}\gamma' z'^2(K_p - K_a)](\tfrac{1}{3})z'} \tag{6.22}$$

The sizing of the necessary profile of the sheet piling is then made according to the allowable flexural stress of the sheet pile material, or

$$\boxed{S = \frac{M_{max}}{\sigma_{all}}} \tag{6.23}$$

where S = section modulus of the sheet pile required per unit length of the structure

σ_{all} = allowable flexural stress of the sheet pile

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Example 6.1

Refer to Figure 6.7. For a cantilever sheet pile wall penetrating a granular soil, given: $L_1 = 2$ m, $L_2 = 3$ m. The granular soil has the following properties:

$\phi = 32°$

$c = 0$

$\gamma = 15.9$ kN/m³

$\gamma_{sat} = 19.33$ kN/m³

Make the necessary calculations to determine the theoretical and actual depth of penetration. Also determine the miminum size of sheet pile (section modulus) necessary.

Solution

The step-by-step procedure given in Section 6.3 will be followed here.

Step 1

$$K_a = \tan^2\left(45 - \frac{\phi}{2}\right) = \tan^2\left(45 - \frac{32}{2}\right) = 0.307$$

$$K_p = \tan^2\left(45 + \frac{\phi}{2}\right) = 3.25$$

Step 2

$$p_1 = \gamma L_1 K_a = (15.9)(2)(0.307) = 9.763 \text{ kN/m}^2$$

$$p_2 = (\gamma L_1 + \gamma' L_2)K_a = [(15.9)(2) + (19.33 - 9.81)3]0.307$$
$$= 18.53 \text{ kN/m}^2$$

Step 3

$$L_3 = \frac{p_2}{\gamma'(K_p - K_a)} = \frac{18.53}{(19.33 - 9.81)(3.25 - 0.307)} = 0.66 \text{ m}$$

Step 4

$$P = \tfrac{1}{2}p_1 L_1 + p_1 L_2 + \tfrac{1}{2}(p_2 - p_1)L_2 + \tfrac{1}{2}p_2 L_3$$
$$= \tfrac{1}{2}(9.763)(2) + (9.763)(3) + \tfrac{1}{2}(18.53 - 9.763)3 + \tfrac{1}{2}(18.53)(0.66)$$
$$= 9.763 + 29.289 + 13.151 + 6.115 = 58.32 \text{ kN/m}$$

Step 5. Taking the moment about E

$$\bar{z} = \frac{1}{58.32}\left[9.763\left(0.66 + 3 + \frac{2}{3}\right) + 29.289\left(0.66 + \frac{3}{2}\right) + 13.151\left(0.66 + \frac{3}{3}\right) + 6.115\left(0.66 \times \frac{2}{3}\right)\right] = 2.23 \text{ m}$$

Step 6

$$p_5 = (\gamma L_1 + \gamma' L_2)K_p + \gamma' L_3(K_p - K_a)$$
$$= [(15.9)(2) + (19.33 - 9.81)3]3.25 + (19.33 - 9.81)(0.66)(3.25 - 0.307)$$
$$= 196.17 + 18.49 = 214.66\ \mathrm{kN/m^2}$$

Step 7

$$A_1 = \frac{p_5}{\gamma'(K_p - K_a)} = \frac{214.66}{(9.52)(2.943)} = 7.66$$

$$A_2 = \frac{8P}{\gamma'(K_p - K_a)} = \frac{(8)(58.32)}{(9.52)(2.943)} = 16.65$$

$$A_3 = \frac{6P[2\bar{z}\gamma'(K_p - K_a) + p_5]}{\gamma'^2(K_p - K_a)^2}$$
$$= \frac{(6)(58.32)[(2)(2.23)(9.52)(2.943) + 214.66]}{(9.52)^2(2.943)^2} = 151.93$$

$$A_4 = \frac{P(6\bar{z}p_5 + 4P)}{\gamma'^2(K_p - K_a)^2}$$
$$= \frac{58.32[(6)(2.23)(214.66) + (4)(58.32)]}{(9.52)^2(2.943)^2} = 230.72$$

Step 8. From Eq. (6.16)

$$L_4^4 + 7.66L_4^3 - 16.65L_4^2 - 151.39L_4 - 230.72 = 0$$

The following table shows the solution of the preceding equation by trial and error.

Assumed L_4 (m)	Left side of Eq. (6.16)
4	−356.44
5	+178.58
4.8	+36.96

So, $L_4 \approx 4.8$ m

Step 9

$$p_4 = p_5 + \gamma' L_4(K_p - K_a)$$
$$= 214.66 + (9.52)(4.8)(2.943) = 349.14\ \mathrm{kN/m^2}$$

Step 10

$$p_3 = \gamma'(K_p - K_a)L_4 = (9.52)(2.943)(4.8) = 134.48\ \mathrm{kN/m^2}$$

Step 11

$$L_5 = \frac{p_3 L_4 - 2P}{p_3 + p_4} = \frac{(134.48)(4.8) - 2(58.32)}{134.48 + 349.14} = 1.09\ \mathrm{m}$$

Step 12. The net pressure distribution diagram can now be drawn, as shown in Figure 6.7a.

Step 13. The actual depth of penetration $= 1.3(L_3 + L_4) = 1.3(0.66 + 4.8) = 7.1$ m. The theoretical depth of penetration $= 0.66 + 4.8 = 5.46$ m.

Size of Sheet Piling

Using Eq. (6.21)

$$z' = \sqrt{\frac{2P}{\gamma'(K_p - K_a)}} = \sqrt{\frac{(2)(58.32)}{9.52(2.943)}} = 2.04 \text{ m}$$

From Eq. (6.22)

$$M_{max} = P(\bar{z} + z') - \left[\frac{1}{2}\gamma' z'^2 (K_p - K_a)\right]\left(\frac{z'}{3}\right)$$

$$= (58.32)(2.23 + 2.04) - \frac{1}{2}(9.52)(2.04)^2(2.943)\left(\frac{2.04}{3}\right)$$

$$= 249.03 - 39.64 = 209.39 \text{ kN-m}$$

The required section modulus of the sheet pile

$$S = \frac{M_{max}}{\sigma_{all}}$$

With $\sigma_{all} = 172.5 \text{ MN/m}^2$

$$S = \frac{209.39 \text{ kN-m}}{172.5 \times 10^3 \text{ kN/m}^2} = 1.214 \times 10^{-3} \text{ m}^3\text{/m of wall}$$

6.4 Special Cases for Cantilever Wall (Penetrating a Sandy Soil)

Following are two special cases of the mathematical formulation shown in Section 6.3.

Case 1: Sheet Pile Wall with the Absence of Water Table

In the absence of the water table, the net pressure diagram on the cantilever sheet pile wall will be as shown in Figure 6.8, which is a modified version of Figure 6.7. For this figure

$$p_2 = \gamma L K_a \quad (6.24)$$

$$p_3 = L_4(K_p - K_a)\gamma \quad (6.25)$$

$$p_4 = p_5 + \gamma L_4(K_p - K_a) \quad (6.26)$$

$$p_5 = \gamma L K_p + \gamma L_3(K_p - K_a) \quad (6.27)$$

$$L_3 = \frac{p_2}{\gamma(K_p - K_a)} = \frac{L K_a}{(K_p - K_a)} \quad (6.28)$$

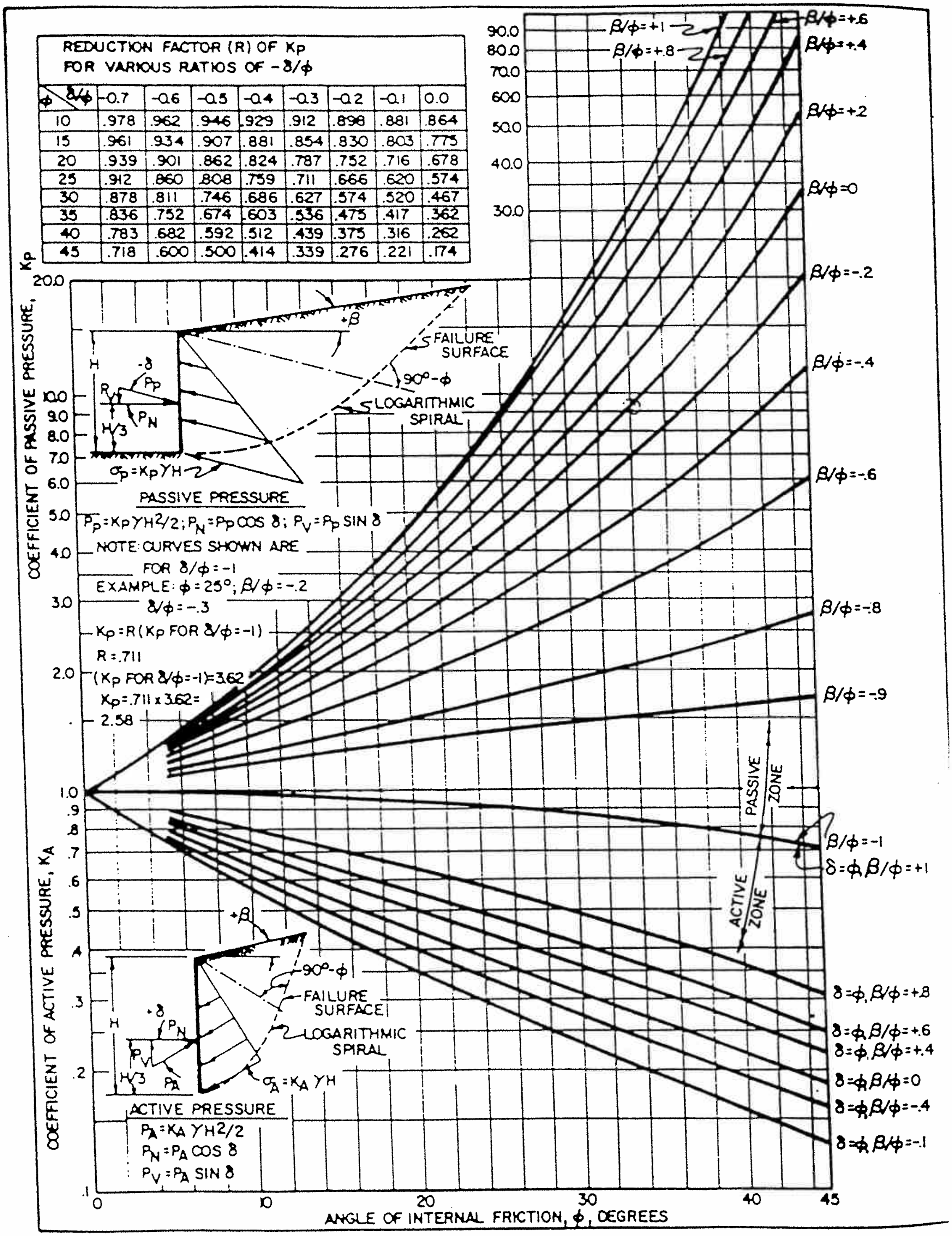
$$P = \tfrac{1}{2} p_2 L + \tfrac{1}{2} p_2 L_3 \quad (6.29)$$

$$\bar{z} = L_3 + \frac{L}{3} = \frac{L K_a}{K_p - K_a} + \frac{L}{3} = \frac{L(2K_a + K_p)}{3(K_p - K_a)} \quad (6.30)$$

TABLE 1
Ultimate Friction Factors and Adhesion for Dissimilar Materials

Interface Materials	Friction factor, $\tan \delta$	Friction angle, δ degrees
Mass concrete on the following foundation materials:		
Clean sound rock..................................	0.70	35
Clean gravel, gravel-sand mixtures, coarse sand...	0.55 to 0.60	29 to 31
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel..................	0.45 to 0.55	24 to 29
Clean fine sand, silty or clayey fine to medium sand..	0.35 to 0.45	19 to 24
Fine sandy silt, nonplastic silt.................	0.30 to 0.35	17 to 19
Very stiff and hard residual or preconsolidated clay..	0.40 to 0.50	22 to 26
Medium stiff and stiff clay and silty clay........	0.30 to 0.35	17 to 19
(Masonry on foundation materials has same friction factors.)		
Steel sheet piles against the following soils:		
Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls..........................	0.40	22
Clean sand, silty sand-gravel mixture, single size hard rock fill.................................	0.30	17
Silty sand, gravel or sand mixed with silt or clay	0.25	14
Fine sandy silt, nonplastic silt.................	0.20	11
Formed concrete or concrete sheet piling against the following soils:		
Clean gravel, gravel-sand mixture, well-graded rock fill with spalls..........................	0.40 to 0.50	22 to 26
Clean sand, silty sand-gravel mixture, single size hard rock fill.................................	0.30 to 0.40	17 to 22
Silty sand, gravel or sand mixed with silt or clay	0.30	17
Fine sandy silt, nonplastic silt.................	0.25	14
Various structural materials:		
Masonry on masonry, igneous and metamorphic rocks:		
Dressed soft rock on dressed soft rock..........	0.70	35
Dressed hard rock on dressed soft rock..........	0.65	33
Dressed hard rock on dressed hard rock..........	0.55	29
Masonry on wood (cross grain).....................	0.50	26
Steel on steel at sheet pile interlocks..........	0.30	17

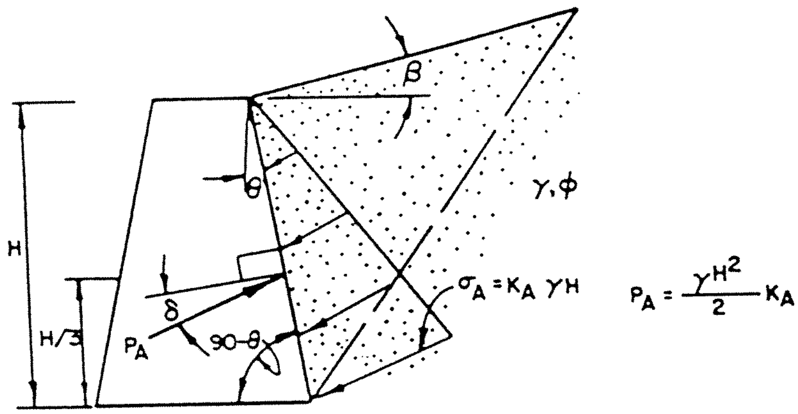
Interface Materials (Cohesion)	Adhesion C_a (psf)
Very soft cohesive soil (0 - 250 psf)	0 - 250
Soft cohesive soil (250 - 500 psf)	250 - 500
Medium stiff cohesive soil (500 - 1000 psf)	500 - 750
Stiff cohesive soil (1000 - 2000 psf)	750 - 950
Very stiff cohesive soil (2000 - 4000 psf)	950 - 1,300



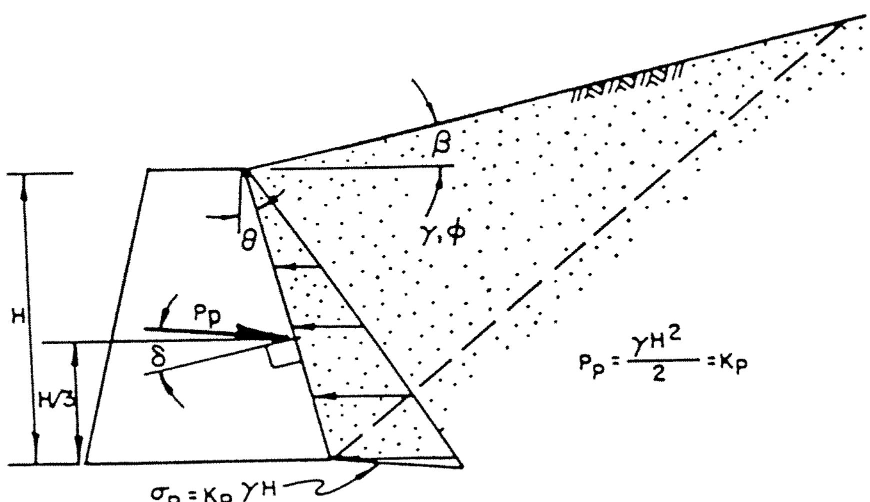
REDUCTION FACTOR (R) OF Kp FOR VARIOUS RATIOS OF $-\delta/\phi$

ϕ \ δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

FIGURE 6
Active and Passive Coefficients with Wall Friction
(Sloping Backfill)



$$K_A = \frac{\cos^2(\phi - \theta)}{\cos^2\theta \cos(\theta + \delta)\left[1 + \sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\cos(\theta+\delta)\cos(\theta-\beta)}}\right]^2}$$



$$K_p = \frac{\cos^2(\theta + \phi)}{\cos^2\theta \cos(\theta - \delta)\left[1 - \sqrt{\frac{\sin(\phi+\delta)\sin(\phi+\beta)}{\cos(\theta-\delta)\cos(\theta-\beta)}}\right]^2}$$

K_p VALUES ARE SATISFACTORY FOR $\delta \leq \phi/3$ BUT ARE UNCONSERVATIVE FOR $\delta > \phi/3$ AND THEREFORE SHOULD NOT BE USED.

FIGURE 8

Coefficients K_A and K_p for Walls with Sloping Wall and Friction, and Sloping Backfill

TASK:

To perform a bending deflection and grout cracking evaluation for a sheetpile wall supporting a slope of 2H:1V with 10.5 feet of sheetpile wall exposed (temporary case).

REFERENCES:

1. Manual of Steel Construction - Load and Resistant Factor Design (1986). First Edition. American Institute of Steel Construction.

METHODOLOGY:

The following procedure was used to evaluate the potential of grout cracking:

(1) Calculate the deflection of the sheetpile wall at the bottom of the exposed sheetpile.

(2) Calculate total equivalent load on the grout core to match the deflection of the sheetpile wall.

(3) Determine maximum moment in the grout core.

(4) Calculate tensile stress in the grout and compare it to the allowable tensile stress.

CALCULATIONS:

Assumptions:

Sheetpile:
Modulus of elasticity = E = 30,000,000 psi
Moment of inertia = I = 134 in^4 (Sheet 36 for a WEZ-95 sheetpile wall)
Exposed height of sheetpile = 10.5 ft = 126 in

1.5" Diameter Grout Core:
Modulus of elasticity = E = 4,560,000 psi (see Sheet 33 for calculation)
Allowable tensile stress = σ_t' = 740 psi (see Sheet 33 calculation)
Moment of inertia = I_x = 38.5 in^4 (see Sheet 33 for calculation)
Section modulus = S = 7.12 in^3 (see Sheet 34 for calculation)

Soil Properties:
Soil unit weight = γ = 125 pcf = 0.072 pci
Buoyant soil unit weight = γ' = 62.5 pcf = 0.036 pci (Note: 62.5 pcf is used as a simplification since it is the average value of the buoyant weight of the soil (62.6 pcf) and the unit weight of water (62.4 pcf), and it is within the required accuracy.)

From Sheet 1:
K_a = 0.45; K_p = 5.29

(1) Calculate the deflection of the sheetpile wall at the bottom of the exposed sheetpile (Point a).

Point b is the location of zero net shear, which was determined on Sheet 4. Therefore, based on Sheet 6:

$D_1 = L_3 + z' = 1.65\text{ ft} + 3.59\text{ ft}$

$D_1 = 5.24\text{ ft} = 62.88\text{ in}$

$L = L_1 + L_2 + D_1 = 12.24\text{ ft} = 146.9\text{ in}$

$L_s = L_1 + L_2 = 7.0\text{ ft} = 84\text{ in}$

Using the deflection formula for a Cantilever Beam - Load Increasing Uniformly to Fixed End and for a Cantilever Beam - Concentrated Load at Any Point in Ref. 1 (Sheet 38 and 39), the loading geometry shown on Sheet 35, and the modulus and moment of inertia for the sheetpile:

$$\Delta x_a = \frac{W_1}{60EIL^2}(L_s^5 - 5L^4 L_s + 4L^5) - \frac{W_2}{60EID_1^2}(4D_1^5) + \frac{P(L_s + D_1 - L_s)^2}{6EI}(3(L_2/3 + D_1) - (L_s + D_1) + L_s)$$

where $W_1 = 0.5K_a(\gamma L_1^2 + \gamma'(L_2 + D_1)^2)$, $W_2 = 0.5(K_p + K_a)\gamma' D_1^2$, and $P = 0.5\gamma_w L_2^2$

$\Delta x_a = 0.009\text{ in}$

(2) Calculate total equivalent load on the grout core to match the deflection of the sheetpile wall.

The deflection formula for a Cantilever Beam - Uniformly Distributed Load in Ref. 1 (Sheet 34), with the Δx_a calculated in Step 2, and the modulus and moment of inertia of the grout is used to calculate the equivalent load on the grout core. The length of this beam is assumed to be D_1 which provides a conservative overestimate of the loading condition (see Sheet 35 for loading geometry).

$$w = \frac{\Delta x_a 24EI}{3D_1^4}$$

$w = 0.84\text{ lb/in}$

(3) Determine maximum moment in the grout core.

Using the maximum moment formula for a Cantilever Beam - Uniformly Distributed Load in Ref. 1 (Sheet 38):

$$M_{max} = \frac{wD_1^2}{2}$$

M_{max} = 1,655 lb-in

(4) Calculate tensile stress in the grout and compare it to the allowable tensile stress.

$$\sigma_t' = \frac{M_{max}}{S}$$

σ_t' = 232 psi

232 psi (calculated) < 740 psi (allowable) OK

CONCLUSIONS

Based on the above calculations, it was determined that the stress in the grout is less than the the allowable tensile stress (232 psi < 740 psi) under a worst case loading condition; therefore, grout cracking is unlikely.

CLIENT GE SUBJECT Supplemental Calculations Prepared By LHK Date: 6/23/99
Reviewed By RDD Date 6/23/99
PROJECT Lyman Street Source Control Containment Barrier

TASK:

To determine the allowable tensile stress, the elastic modulus, moment of inertia, and section modulus of the grout core.

REFERENCES:

1. Merritt, F. S., M.K. Loftin, and J.T. Ricketts. (1996) Standard Handbook for Civil Engineers. Fourth Edition. McGraw- Hill Companies, Inc. New York, NY.

CALCULATIONS:

Allowable Tensile Stress

The tensile stress of the grout is usually between 7 to 10 percent of its compressive strength. Using 8.5 percent:

$$\sigma_t' = (0.085)\, f_c'$$

where f_c' = specified compressive strength at 28 days= 60 MPa (8,700 psi) from Sheet 37.

$$\underline{\sigma_t' = 740 \text{ psi}}$$

Modulus of Elasticity

Using Ref. 1 the modulus of elasticity of the grout, E, is calculated as follows:

$$E = w^{1.5}(33)\sqrt{f_c'}$$

where w = unit weight of the grout = 130 pcf.

$$\underline{E = 4{,}560{,}000 \text{ psi}}$$

Moment of Inertia

Using the parallel axis theorem (Ref. 1), the moment of inertia about the parallel axis, I_x, is calculated as follows:

$$I_x = I + Ad_1^2$$

where I = moment of inertia about centroidal axis for a circle; A = cross-sectional area; d_1 = distance between centroidal and parallel axes (see Sheet 36 for a WEZ-95 sheetpile wall).

$$I_x = \frac{\pi d^4}{64} + \frac{\pi d^2}{4}(d_1^2)$$

where d = diameter of the grout core.

$I_x = 38.5\ in^4$

Section Modulus

The section modulus, S, is calculated as follows:

$$S = \frac{I_x}{c}$$

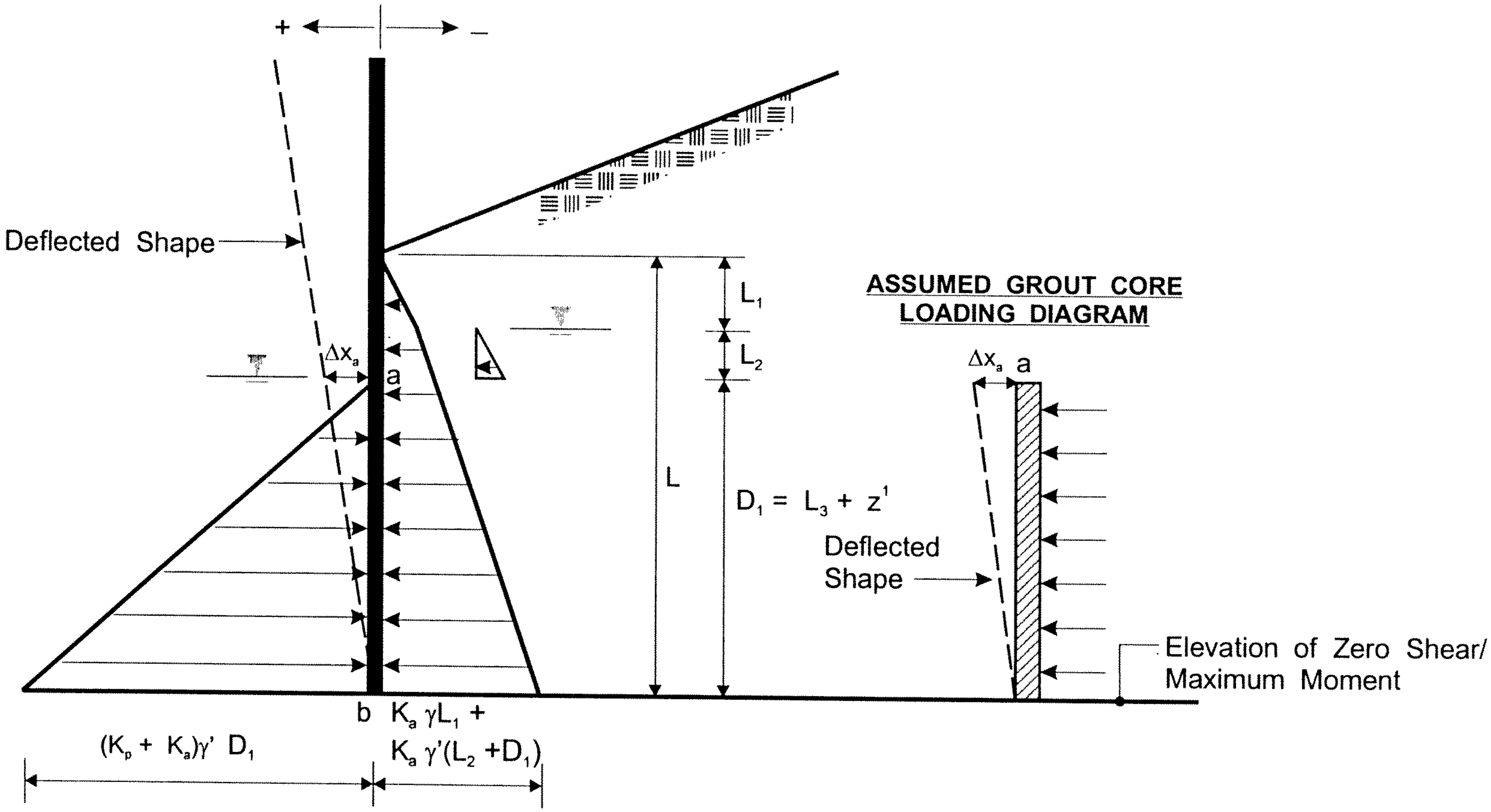
where c = distance from the outermost fiber of the grout core to the neutral axis of the sheetpile wall (see Sheet 36 for a WEZ-95 sheetpile wall).

S = 7.13 in^3

CONCLUSIONS

The allowable tensile strength of the grout core is 740 psi, the elastic modulus is 4,560,000 psi, the moment of inertia is 38.5 in^4, and the section modulus is 7.13 in^3.

SHEETPILE LOADING DIAGRAM



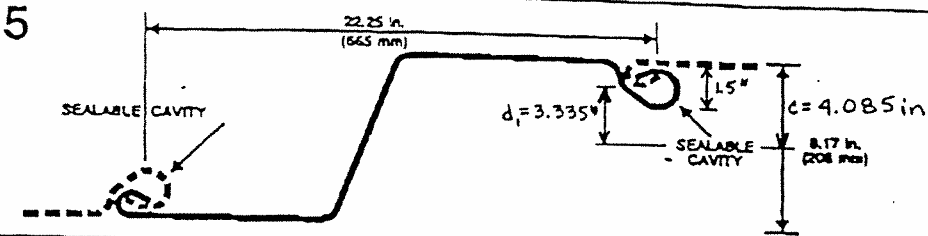
LOADING DIAGRAM

02/99 SYR-D54-DJH
20140005/20140g13.cdr

WATERLOO BARRIER™

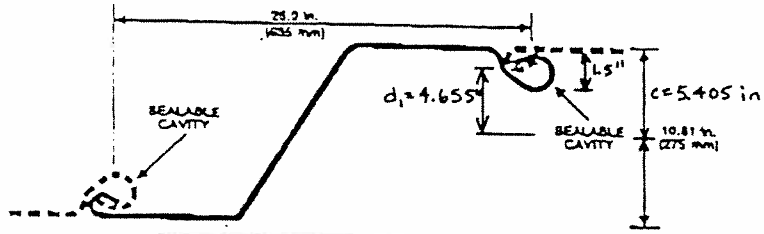
IS AVAILABLE IN TWO DESIGNS. THE MEDIUM WALL WZ75 AND THE HEAVY WALL WEZ95

WZ75



SECTION	THICKNESS in (mm)	HEIGHT in (mm)	NOMINAL WIDTH in (mm)	SECTION AREA in² (cm²)	WEIGHT lbs/lin. ft (kg/lin. m)	WEIGHT lbs/ft² (kg/m²)	MOMENT OF INERTIA (cm⁴/wall m)	RADIUS OF GYRATION	SECTION MODULUS (cm³/wall m)
WZ75	.295 (7.50)	8.17 (208)	22.25 (565)	10.5 (67.5)	35.6 (53.0)	19.2 (93.7)	64.8 (8870)	3.39 (86.1)	15.8 (850)

WEZ95



SECTION	THICKNESS in (mm)	HEIGHT in (mm)	NOMINAL WIDTH in (mm)	SECTION AREA in² (cm²)	WEIGHT	WEIGHT	MOMENT OF INERTIA (cm⁴/wall m)	RADIUS OF GYRATION	SECTION MODULUS (cm³/wall m)
[illegible]	[illegible]	10.81 (275)	25.0 (635)	14.9 (96.2)	50.5 (75.2)	24.3 (119)	134 (18300)	4.33 (110)	24.9 (1340)

SPECIFICATIONS:

RAW MATERIAL: ASTM A572 GR50
CSA G40.21 GR 350W

MANUFACTURING: ASTM A6
CSA G40.20

COATINGS:
1) GALVANIZED ASTM A123, CSA G164
2) COAL TAR EPOXY SSPC-16
3) FUSION BONDED EPOXY RESIN, MFG'S SPE

ACCESSORIES: BENDS CAN BE SUPPLIED TO ANY ANGLE. 'T' SECTIONS AND OTHER WELDED FABRICATIONS ARE AVAILABLE.

37/39

MIXING PROPERTIES:

DESCRIPTION	REQUIREMENTS
SEALANT PACKAGING:	30 kg Bags
MINIMUM WATER VOLUME (per bag):	6.25 L
MAXIMUM WATER VOLUME (per bag):	9.25 L
MIXER TYPE:	Colloidal
MINIMUM MIXING TIME:	2.5 (min.)
MAXIMUM POT LIFE:	180 (min.)

CURING AND SET TIMES:

The initial gel time of the sealant varies from 1.5 to 2.0 hours @ 20°C.
Initial set time of the sealant varies from 1 to 2 days after placement.
Ultimate strength is reached at approximately 28 days.

COMPRESSIVE TESTING	STRENGTH
1 DAY:	15 Mpa
3 DAYS:	38 Mpa
7 DAYS:	50 Mpa
28 DAYS:	60 Mpa

PERMEABILITY TESING:

Permeability testing was completed by Davroc Testing Laboratories Inc. to confirm a bulk hydraulic conductivity of 3.19×10^{-15} m/s

YIELD:

Each 30 kg (66 lb) bag of WBS 301 sealant produces 0.01 cubic metres of grout.

SAFETY PRECAUTIONS:

Waterloo Barrier® Grout - WBS 301 - contains Portland Cement, Fly Ash, Silica Fume and other admixtures. Normal safety wear, such as rubber gloves, dust masks and safety glasses that are used to handle conventional cement based products should be worn. Material Safety Data Sheets available on request.

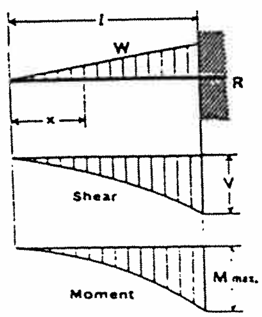
BEAM DIAGRAMS AND FORMULAS

For various static loading conditions

38/39

For meaning of symbols, see page 3-127

18. CANTILEVER BEAM—LOAD INCREASING UNIFORMLY TO FIXED END



Total Equiv. Uniform Load . . . . $= \frac{8}{3} W$

$R = V$ . . . . $= W$

V_x . . . . $= W \frac{x^2}{l^2}$

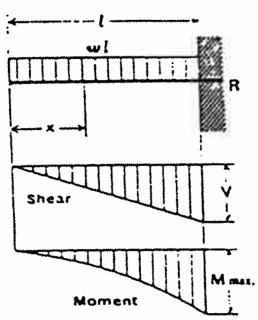
M max. (at fixed end) . . . . . $= \frac{Wl}{3}$

M_x . . . . $= \frac{Wx^3}{3l^2}$

Δmax. (at free end) . . . . . $= \frac{Wl^3}{15EI}$

Δx . . . . $= \frac{W}{60EIl^2}(x^5 - 5l^4x + 4l^5)$

19. CANTILEVER BEAM—UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load . . . . $= 4wl$

$R = V$ . . . . $= wl$

V_x . . . . $= wx$

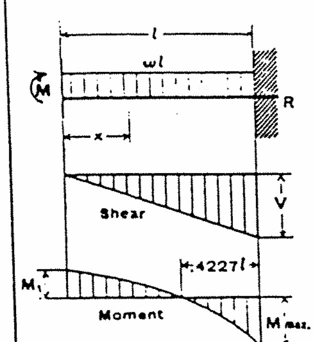
M max. (at fixed end) . . . . . $= \frac{wl^2}{2}$

M_x . . . . $= \frac{wx^2}{2}$

Δmax. (at free end) . . . . . $= \frac{wl^4}{8EI}$

Δx . . . . $= \frac{w}{24EI}(x^4 - 4l^3x + 3l^4)$

20. BEAM FIXED AT ONE END, FREE TO DEFLECT VERTICALLY BUT NOT ROTATE AT OTHER—UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load . . . . $= \frac{8}{3} wl$

$R = V$ . . . . $= wl$

V_x . . . . $= wx$

M max. (at fixed end) . . . . . $= \frac{wl^2}{3}$

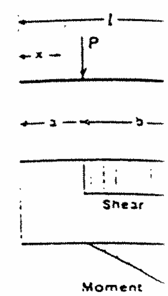
M_1 (at deflected end) . . . . . $= \frac{wl^2}{6}$

M_x . . . . $= \frac{w}{6}(l^2 - 3x^2)$

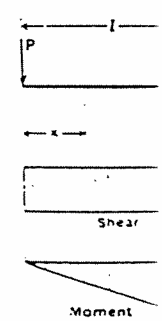
Δmax. (at deflected end) . . . . . $= \frac{wl^4}{24EI}$

Δx . . . . $= \frac{w(l^2 - x^2)^2}{24EI}$

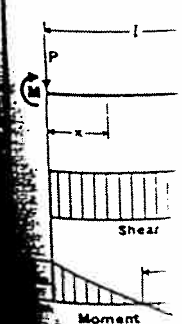
21. CANTILE



22. CANTIL



23. BEA
NOT RO



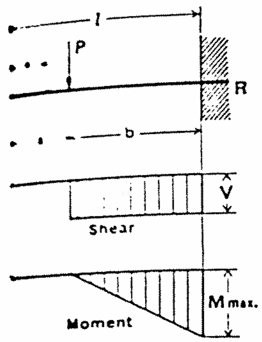
39/39

BEAM DIAGRAMS AND FORMULAS

For various static loading conditions

For meaning of symbols, see page 3-127

CANTILEVER BEAM—CONCENTRATED LOAD AT ANY POINT



Total Equiv. Uniform Load . . . . $= \frac{8Pb}{l}$

$R = V$ . . . . $= P$

M max. (at fixed end) . . . . . . $= Pb$

M_x (when $x > a$) . . . . . . $= P(x - a)$

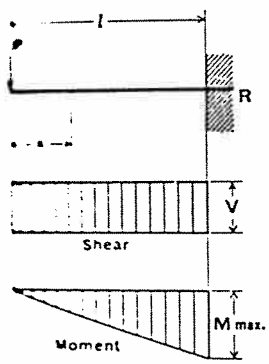
$\Delta max.$ (at free end) . . . . . . $= \frac{Pb^2}{6EI}(3l - b)$

Δa (at point of load) . . . . $= \frac{Pb^3}{3EI}$

Δx (when $x < a$) . . . . . $= \frac{Pb^2}{6EI}(3l - 3x - b)$

Δx (when $x > a$) . . . . . . $= \frac{P(l - x)^2}{6EI}(3b - l + x)$

CANTILEVER BEAM—CONCENTRATED LOAD AT FREE END



Total Equiv. Uniform Load . . . . $= 8P$

$R = V$ . . . . $= P$

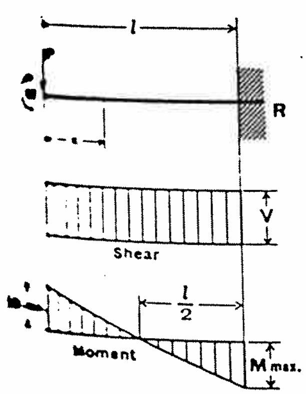
M max. (at fixed end) . . . . . . $= Pl$

M_x . . . . $= Px$

$\Delta max.$ (at free end) . . . . $= \frac{Pl^3}{3EI}$

Δx . . . . $= \frac{P}{6EI}(2l^3 - 3l^2x + x^3)$

BEAM FIXED AT ONE END, FREE TO DEFLECT VERTICALLY BUT NOT ROTATE AT OTHER—CONCENTRATED LOAD AT DEFLECTED END



Total Equiv. Uniform Load . . . . $= 4P$

$R = V$ . . . . $= P$

M max. (at both ends) . . . . $= \frac{Pl}{2}$

M_x . . . . $= P\left(\frac{l}{2} - x\right)$

$\Delta max.$ (at deflected end) . . . . $= \frac{Pl^3}{12EI}$

Δx . . . . $= \frac{P(l - x)^2}{12EI}(l + 2x)$

Appendix D

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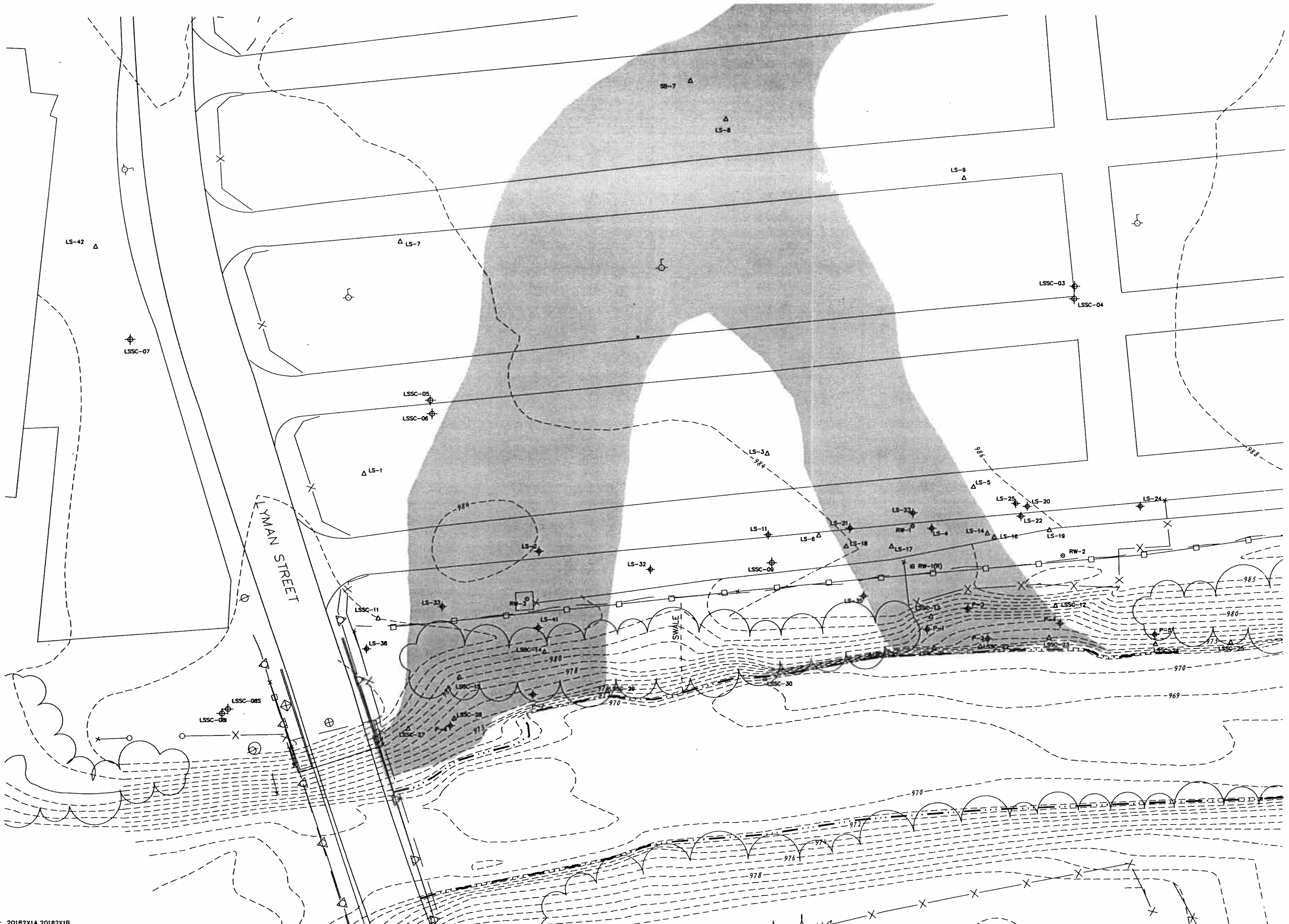
Containment Barrier Technical Drawings



LEGEND:

- APPROXIMATE MEAN RIVER ELEVATION AND MEAN GROUNDWATER TABLE ELEVATION (971.5)
- 980 EXISTING INDEX ELEVATION CONTOUR
- EXISTING INTERMEDIATE ELEVATION CONTOUR
- DECIDUOUS TREE
- CONIFEROUS TREE
- MANHOLE
- CHAIN LINK FENCE
- POLE (NON-UTILITY)
- POLE (OVERHEAD UTILITY)
- APPROXIMATE DELINEATION OF FORMER OXBOWS
- ES2-1 EXISTING MONITORING WELL
- RW-3 EXISTING PUMPING WELL
- LS-1 EXISTING SOIL BORING

NOTES:

1. MAPPING IS BEST AVAILABLE INFORMATION AS OF 12/10/98 BASED ON MAPPING PROVIDED BY LOCKWOOD MAPPING, INC. PREPARED FROM 1990 AERIAL PHOTOGRAPHY; DATA PROVIDED BY GENERAL ELECTRIC; AND BLASLAND AND BOUCK ENGINEERS, PC. CONSTRUCTION PLANS. RIVERBANK AND RIVER BED TOPOGRAPHIC INFORMATION PROVIDED BY BBL FROM OCTOBER 12–23, 1998 FIELD SURVEY.
2. COORDINATE GRID BASED ON 1927 STATE PLANE COORDINATES.
3. ELEVATION DATUM REFERENCED TO NGVD 1929.
4. ALL SAMPLING LOCATIONS ARE APPROXIMATE.

X: 20182X1A,20182X1B
L: ON=*,OFF=*REF*
P: STD-PCP/DL
7/1/99 SYR-54-YCC
20182001/CONTRACT/20182G01.DWG

Graphic Scale
20' 0 20' 40'

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Project Mgr. ________
Designed by ________
Drawn by ________
Checked by ________
Prof. Eng. ________
PE License ________

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engineers & scientists

GENERAL ELECTRIC COMPANY • PITTSFIELD, MASSACHUSETTS

LYMAN STREET SITE SUPPLEMENTAL SOURCE CONTROL CONTAINMENT

EXISTING SITE PLAN

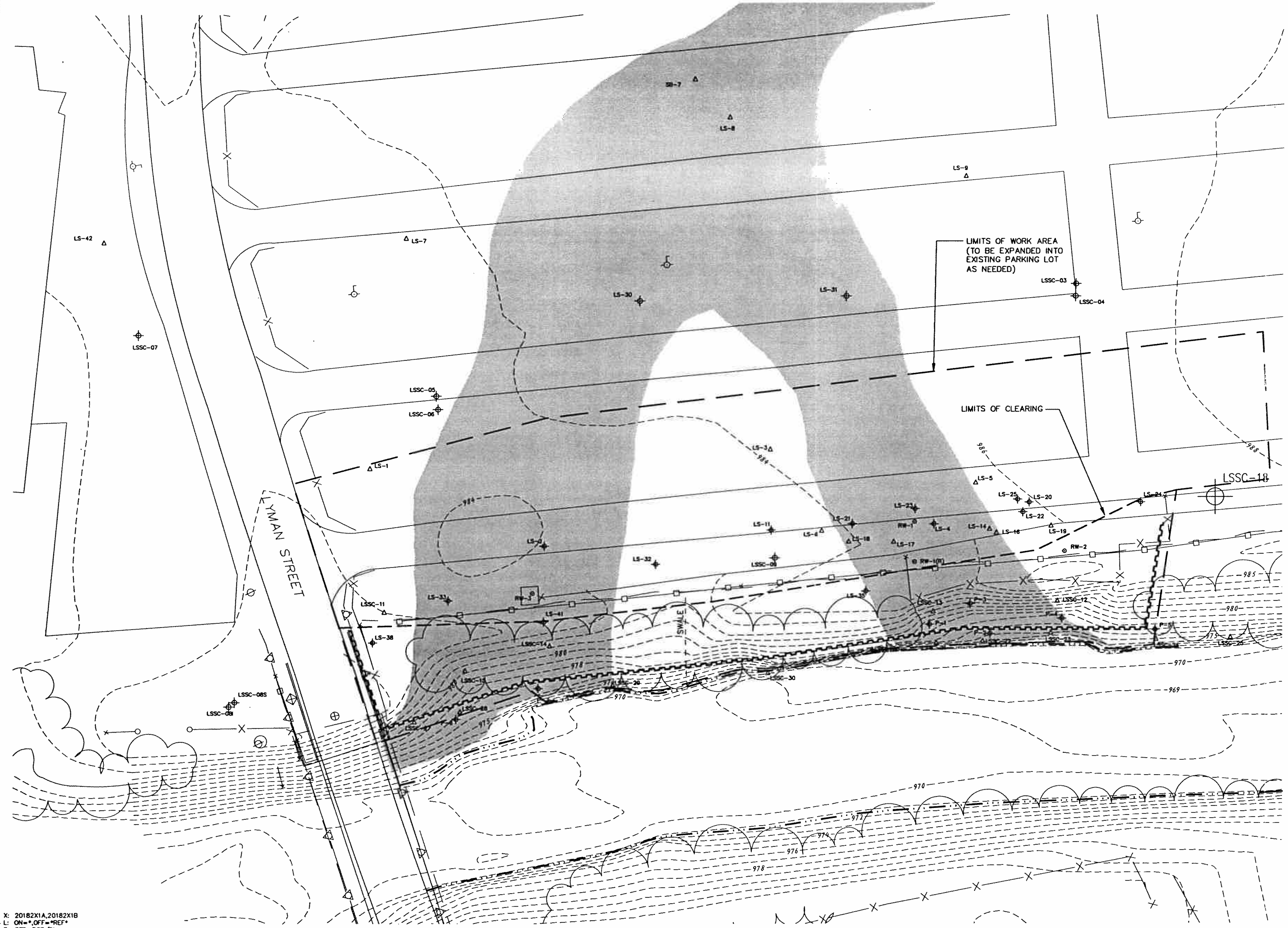
GENERAL

File Number
201.82. F

Date
JULY 1999

Blasland, Bouck & Lee, Inc.
Corporate Headquarters
6723 Towpath Road
Syracuse, NY 13214
315-446-9120

1



LEGEND:

- APPROXIMATE MEAN RIVER ELEVATION AND MEAN GROUNDWATER TABLE ELEVATION (971.5)
- APPROXIMATE LOCATION OF PORPOSED CONTAINMENT BARRIER
- 980 — EXISTING INDEX ELEVATION CONTOUR
- EXISTING INTERMEDIATE ELEVATION CONTOUR
- DECIDUOUS TREE
- CONIFEROUS TREE
- MANHOLE
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NOTES:

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2. COORDINATE GRID BASED ON 1927 STATE PLANE COORDINATES.
3. ELEVATION DATUM REFERENCED TO NGVD 1929.
4. ALL SAMPLING LOCATIONS ARE APPROXIMATE.

X: 20182X1A,20182X1B
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P: STD-PCP/DL
7/7/99 SYR-54-YCC
20182001/CONTRACT/20182G02.DWG

Graphic Scale
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GENERAL ELECTRIC COMPANY• PITTSFIELD, MASSACHUSETTS
LYMAN STREET SITE SUPPLEMENTAL SOURCE CONTROL CONTAINMENT

WORK LIMITS

GENERAL

File Number
201.82. F

Date
JULY 1999

Blasland, Bouck & Lee, Inc.
Corporate Headquarters
6723 Towpath Road
Syracuse, NY 13214
315-446-9120

2

X: 20182X1A,20182X1B
L: ON=*,OFF=*REF*
P: STD-PCP/DL
7/12/99 SYR-54-YCC
20182001/CONTRACT/20182G03.DWG

LEGEND:

- APPROXIMATE MEAN RIVER ELEVATION AND MEAN GROUNDWATER TABLE ELEVATION (971.5)
- APPROXIMATE LOCATION OF PORPOSED CONTAINMENT BARRIER
- 980 — EXISTING INDEX ELEVATION CONTOUR
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NOTES:

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2. COORDINATE GRID BASED ON 1927 STATE PLANE COORDINATES.
3. ELEVATION DATUM REFERENCED TO NGVD 1929.
4. ALL SAMPLING LOCATIONS ARE APPROXIMATE.

Graphic Scale
20' 0 20' 40'

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Project Mgr.
Designed by
Drawn by
Checked by
Prof. Eng.
PE License

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LYMAN STREET SITE SUPPLEMENTAL SOURCE CONTROL CONTAINMENT

CONTAINMENT BARRIER PLAN

GENERAL

File Number
201.82. F

Date
JULY 1999

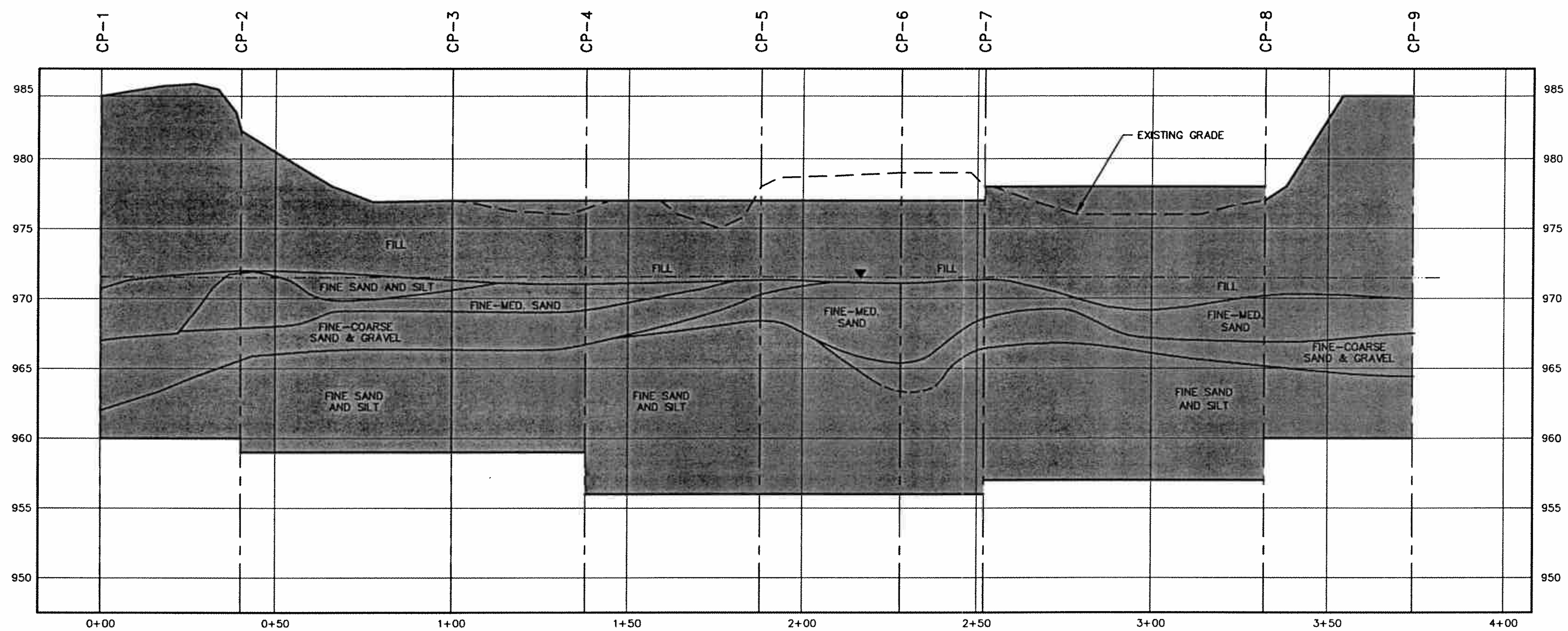
Blasland, Bouck & Lee, Inc.
Corporate Headquarters
6723 Towpath Road
Syracuse, NY 13214
315-446-9120

3

LEGEND:

APPROXIMATE WATER TABLE ELEVATION (BASED ON HISTORICAL MONITORING DATA)

DEPTH OF THE SHEET PILE



P: STD-PCP/DL
7/12/99 SYR-54-YCC
20182001/CONTRACT/20182G04.DWG

Graphic Scale
VERTICAL 5' 0 5' 10'
HORIZONTAL 20' 0 20' 40'

NO ALTERATIONS PERMITTED HEREON EXCEPT AS PROVIDED UNDER SECTION 7209 SUBDIVISION 2 OF THE NEW YORK STATE EDUCATION LAW

No.	Date	Revisions	Init

Project Mgr. __________
Designed by __________
Drawn by __________
Checked by __________
Prof. Eng. __________
PE License __________

BBL
BLASLAND, BOUCK & LEE, INC.
engineers & scientists

GENERAL ELECTRIC COMPANY• PITTSFIELD, MASSACHUSETTS

LYMAN STREET SITE SUPPLEMENTAL SOURCE CONTROL CONTAINMENT

CONTAINMENT BARRIER PROFILE

CONTRACT DRAWING

File Number 201.82. F

Date JULY 1999

Blasland, Bouck & Lee, Inc.
Corporate Headquarters
6723 Towpath Road
Syracuse, NY 13214
315-446-9120

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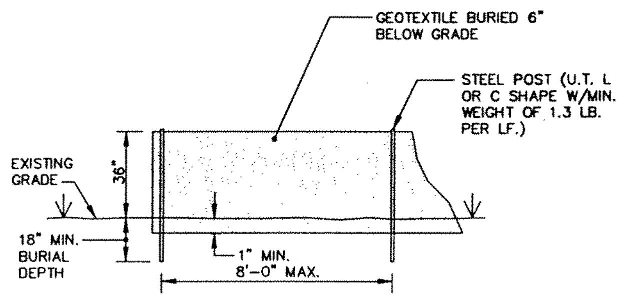
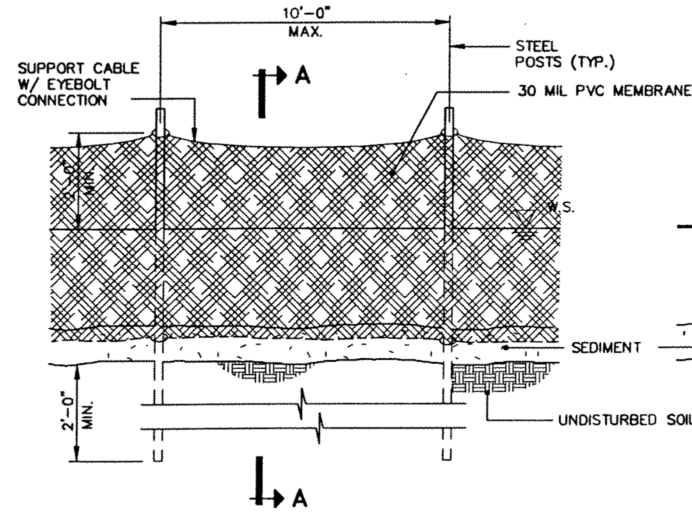


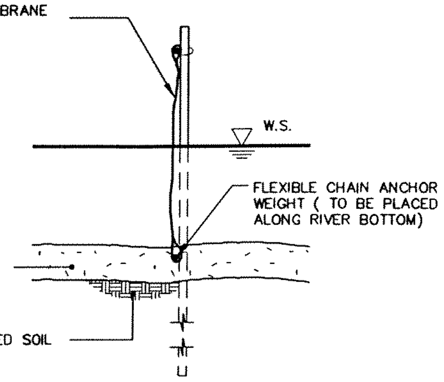
NOTES:

1. SEDIMENT DEPOSITS SHALL BE REMOVED WHEN THE DEPOSIT REACHES APPROX. 6 INCHES ABOVE GRADE LEVEL.
2. THE SILT FENCE WILL REMAIN IN PLACE UNTIL A STRONG VEGETATIVE STAND IS ESTABLISHED.
3. THE SILT FENCE WILL BE USED FOR TEMPORARY EROSION AND SEDIMENTATION CONTROL ONLY.
4. SILT FENCE TO BE CONSTRUCTED AND MAINTAINED IN ACCORDANCE WITH AASHTO M 288 APPENDIX A5.

SILT FENCE DETAIL 1

NOT TO SCALE





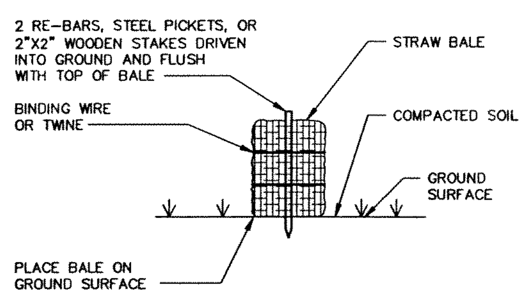
SECTION A-A

NOTE:

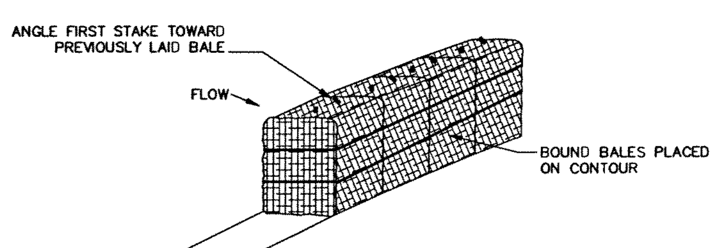
1. THE SILT CURTAIN WILL BE USED FOR TEMPORARY EROSION AND SEDIMENTATION CONTROL ONLY.

SILT CURTAIN DETAIL 3

NOT TO SCALE



BEDDING DETAIL



ANCHORING DETAIL

NOTE:

1. THE STRAW BALES WILL BE USED FOR TEMPORARY EROSION AND SEDIMENTATION CONTROL ONLY.

STRAW BALE 2

NOT TO SCALE

GENERAL NOTES:

1. ALL MATERIALS SHALL BE FURNISHED AND INSTALLED TO MEET THE APPROPRIATE REQUIREMENTS OF THE LATEST EDITION OF MASSACHUSETTS HIGHWAY DEPARTMENT (MHD) STANDARD SPECIFICATIONS FOR HIGHWAYS AND BRIDGES, AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS (AASHTO) STANDARD SPECIFICATIONS FOR TRANSPORTATION MATERIALS AND METHODS OF SAMPLING AND TESTING, AMERICAN CONCRETE INSTITUTE (ACI) CONCRETE CODES AND STANDARDS, ASSOCIATION FOR TESTING AND MATERIALS (ASTM) SPECIFICATIONS AND STANDARDS, CONCRETE REINFORCING STEEL INSTITUTE (CRSI), AND AMERICAN WELDING SOCIETY (AWS).

2. SILT FENCE FABRIC: FABRIC SHALL BE MANUFACTURED BY NICOLON MIRAFI, INC., AMOCO FABRICS, INC., OR OTHER APPROVED MANUFACTURER. THE FABRIC SHALL BE A WOVEN FABRIC WITH LESS THAN 50% ELONGATION (IN ACCORDANCE WITH ASTM D 4632) AND SHALL CONFORM TO THE TEMPORARY SILT FENCE REQUIREMENTS OF AASHTO M-288, SECTION 8, SUMMARIZED BELOW:

PROPERTY	TEST METHOD	REQUIREMENT
GRAB STRENGTH	ASTM D 4632	
MACHINE DIRECTION		> 550 N
X-MACHINE DIRECTION		> 450 N
PERMITTIVITY	ASTM D 4491	> 0.05 SEC -1
APPARENT OPENING SIZE	ASTM D 4751	< 0.60 MM
ULTRAVIOLET STABILITY (RETAINED STRENGTH)	ASTM D 4355	70% AFTER 500 HOURS EXPOSURE.

3. BALED STRAW: STRAW BALES SHALL BE OATS, WHEAT, RYE GRAIN, BROOMSAGE OR OTHER STRAW, OR NATIVE GRASS HAY.

4. STAKES: STAKES FOR BALED STRAW FILTER BARRIERS SHALL BE A MINIMUM OF 1" X 2" WOOD OR EQUIVALENT METAL WITH A MINIMUM LENGTH OF 3 FEET.

5. POSTS: STEEL OR WOODEN POSTS SHOULD BE USED FOR SILT FENCE CONSTRUCTION. THE MAXIMUM POST SPACING FOR SILT FENCING SHALL BE 4 FEET. PRE-FABRICATED SILT FENCE WITH ATTACHED WOODEN POSTS IS ACCEPTABLE PROVIDED THE FABRIC MEETS THE REQUIREMENTS. POSTS FOR SILT CURTAIN CONSTRUCTION SHALL BE MANUFACTURED USING ASTM A-36 STEEL OR EQUAL.

6. SILT CURTAIN: GEOMEMBRANE SHALL BE 30-MIL PVC HAVING THE MINIMUM PHYSICAL PROPERTIES INDICATED BELOW:

PROPERTY	TEST METHOD	REQUIREMENT
TENSILE STRENGTH	ASTM D-882	>60 LB/IN
ELONGATION AT BREAK	ASTM D-882	>300%
TEAR RESISTANCE	ASTM D-882	>9LB/IN

7. SEED: ALL SEEDS SHALL COMPLY WITH LAW OF THE STATE OF MASSACHUSETTS AND THE CURRENT REGULATIONS DULY PROMULGATED THEREUNDER. UNLESS OTHERWISE DIRECTED, GRASS SEED MIXES SHOULD MEET THE REQUIREMENTS OF MHD ITEM 6.03.0 SEED FOR SLOPES AND SHOULDERS. THE SEED SHALL BE DELIVERED IN BAGS WITH CERTIFIED TAGS OR LABELS ATTACHED TO EACH BAG SHOWING THE NAME (KIND AND VARIETY), PERCENT OF GERMINATION AND PURITY OF THE SEED, AND THE PERCENT OF OBNOXIOUS WEEDS AND INERT MATTER.

8. FERTILIZER: FERTILIZERS SHALL COMPLY WITH THE FERTILIZER LAWS OF THE STATE OF MASSACHUSETTS. UNLESS OTHERWISE DIRECTED, FERTILIZER SHOULD MEET THE REQUIREMENTS OF MHD ITEM 6.02.0. ALL FERTILIZERS SHALL BE TRANSPORTED IN CONTAINERS WHICH WILL ENSURE PROPER PROTECTION AND HANDLING.

9. MULCH: MULCHING MATERIALS SHALL BE OATS, WHEAT, RYE GRAIN, BROOMSAGE OR TO OTHER STRAW, NOR NATIVE GRASS HAY. IT SHALL BE REASONABLY FREE FROM JOHNSON GRASS AND OTHER OBNOXIOUS GRASSES AND WEEDS. VEGETATIVE MATERIAL THAT IS WET OR THAT HAS BEEN BALED GREEN SHALL NOT BE USED.

10. GEOTEXTILES: GEOTEXTILES USED FOR TEMPORARY OR PERMANENT EROSION CONTROL, INCLUDING PLACEMENT BELOW RIP RAP, WILL BE A WOVEN MONOFILAMENT GEOTEXTILE THAT MEETS THE REQUIREMENTS BELOW:

PROPERTY	TEST METHOD	REQUIREMENT
GRAB STRENGTH	ASTM D 4632	> 1100 N
TEAR STRENGTH	ASTM D 4533	> 250 N
PUNCTURE STRENGTH	ASTM D 4833	> 400 N
BURST STRENGTH	ASTM D 3786	> 2700 N
PERMITTIVITY	ASTM D 4491	> 0.2 SEC -1
APPARENT OPENING SIZE	ASTM D 4751	< 0.25 MM

11. ROLLED EROSION CONTROL PRODUCTS: ROLLED EROSION CONTROL PRODUCTS SHOULD BE SUITABLE FOR THEIR INTENDED PURPOSE. THEY SHOULD BE RATED FOR A FLOW VELOCITY OF AT LEAST 6 FEET PER SECOND AND BE CONSTRUCTED TO HAVE A LIFESPAN OF AT LEAST ONE YEAR. ROLLED EROSION CONTROL PRODUCTS THAT ARE PLACED BELOW ELEVATION 973 SHOULD BE RATED FOR A FLOW VELOCITY OF AT LEAST 10 FEET PER SECOND AND HAVE A LIFESPAN OF AT LEAST ONE YEAR UNDER WET CONDITIONS.

12. RIPRAP: UNLESS OTHERWISE DIRECTED OR SHOWN ON THE DRAWINGS, RIPRAP SHALL MEET THE REQUIREMENTS OF MHD ITEM M2.02.2 – DUMPED RIPRAP.

13. GEOTEXTILE AND ROLLED EROSION CONTROL PRODUCT PLACEMENT: GEOTEXTILES AND ROLLED EROSION CONTROL PRODUCTS WILL BE PLACED AND MAINTAINED IN ACCORDANCE WITH AASHTO M288 APPENDIX A4.

L: ON=*; OFF=*REF
P: STD-PCP/DL, D2BL
7/12/99 SYR-54-NES KLN YCC
20182001\CONTRACT\20182G05.DWG

GRAPHIC SCALE

No.	Date	Revisions	Init

Project Mgr. JMN
Designed by RDD
Drawn by RCA/GMS
Checked by RDD
Prof. Eng.
PE License

BBL
BLASLAND, BOUCK & LEE, INC.
engineers & scientists

GENERAL ELECTRIC COMPANY• PITTSFIELD, MASSACHUSETTS
LYMAN STREET SITE SUPPLEMENTAL SOURCE CONTROL CONTAINMENT

DETAILS

CONTRACT DRAWINGS

File Number 201.82. F
Date JULY 1999

Blasland, Bouck & Lee, Inc.
Corporate Headquarters
6723 Towpath Road
Syracuse, NY 13214
315-446-9120

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Appendix E

BLASLAND, BOUCK & LEE, INC.
engineers & scientists

Wetland Reconnaissance Report

Wetland Reconnaissance Report
Riverbank Area General Electric Company Parking Area
USEPA AREA 5/ MCP Lyman Street Site
Pittsfield, MA

The above mentioned area was reviewed for wetlands boundaries on June 29, 1999 by Shannon Lombardi of White Engineering, Inc. The resource area was delineated based on vegetation alone using the methods described in "Delineating Bordering Vegetated Wetlands Under the Massachusetts Wetlands Protection Act, A Handbook", March 1995 by MA Department of Environmental Protection. The property abuts the Housatonic River which has a bordering vegetated wetland (BVW) approximately 5 feet wide. The wetland boundary was flagged with orange and white-stripped survey flags numbered WL-1 through WL-6 end. Vegetation and topography were adequate to determine the wetland boundary.

The area consists of the riverine system including land under waterway, bank, bordering vegetated wetland, floodplain, upland and riverfront area. The land under waterway associated with the Housatonic River extends to the bottom of the bank. The associated bank is dominated by Red-Osier Dogwood (*Cornus stolonifera)*, Tartarian Honeysuckle (*Lonicera tartarica*), Eastern Cottonwood (*Polpulus deltoides*) and American Elm (Ulnus americana). A bordering vegetated wetland averaging 5 feet wide along the 400 feet stretch of river is dominated by American Elm (*Alnus americana*), Eastern Cottonwood (*Populus deltoides*) and Red-Osier Dogwood (*Cornus stolonifera*). At the top of the bank the land creates a "shelf" several feet wide along most of the 400 foot stretch of river then changes to an upward direction forming the upper bank until leveling off to the open lot. From the edge of the bordering vegetated wetland the 100-year floodplain extends well into the upland. The entire bank of the river is part of the 100-foot inner riparian zone of the riverfront area.

Wetland Indicator Categories:

OBL (Obligate Wetland): Occurs almost always (>99%) in wetlands
FACW (Facultative Wetland): Usually occurs in wetlands (67%-99%) but occasionally found in upland environments
FAC (Facultative): Equally likely to occur in wetland or uplands (34%-66%)
FACU (Facultative Upland): Usually occurs in uplands (67%-99%), but occasionally found in wetland environments
UPL (Obligate Upland): Occurs almost always (>99%) in uplands under natural conditions in this region. May occur in wetlands in other regions of the country.

The following resource areas present at the site are subject to the Massachusetts Wetlands Protection Act; land under waterway (Housatonic River), bank of the Housatonic River, bordering vegetated wetland adjacent to the bank, 100 ft. buffer zone from the bordering vegetated wetland, floodplain extending from the BVW boundary into the upland and 200 ft riparian zone from the Housatonic River bank under the Rivers Protection Act. This site is not included in an area of estimated wildlife habitat by the Natural Heritage and Endangered Species Program. The 400-foot stretch of riverbank is significantly less than the 10% allowable disturbance under the Wetlands Protection Act for wildlife habitat protection.

Shannon D. Lombardi

Shannon D. Lombardi
Environmental Analyst
White Engineering, Inc.

Site Location

The location of this proposed activity is designated as DEP Site No. 1-0856, USEPA Area 5/ MCP Lyman Street Site in the document entitled "Conceptual Containment Barrier Design for Lyman Street Site" prepared by BBL, Inc. (February, 1999). The work will occur just upstream of the Lyman Street bridge along the bank of the Housatonic River. The project site is adjacent to a General Electric Company parking lot which is secured with gates and fencing.

Proposed Project

As part of the ongoing activities identified in the source control work plans, the General Electric Company is proposing to implement supplemental containment measures. The activities subject to the Massachusetts Wetlands Protection Act (310 CMR 10.00) are outlined below with associated mitigating measures.

The proposed project will include installing sheet piling approximately five feet from the edge of the lower bank of the Housatonic River. The sheet piling shall have an upper elevation of 978-977 feet and the maximum vertical length shall be 20 to 21 feet. Erosion control silt fence shall be installed at waters edge, between the proposed sheet piling and the water edge. This silt fence shall prevent any soil from entering the river during the installation of the sheet piling. An existing containment boom adjacent to the work area will be extended to include the entire length of the proposed sheetpile wall. In addition, a silt curtain will be installed in the river along the entire length of the work area, prior to beginning the work activities. In order to install the sheetpile, the majority of the trees on the bank of the river will need to be cleared. The trees which occur along the proposed alignment of sheetpiling will be removed, including the roots. Other trees in the work area will be cut to ground level to facilitate use of a crane and excavator to place the sheets and remove some soil from the toe of the riverbank. The roots of these trees will not be removed at this time. In addition, any fence or guardrails along the top of the bank will be removed as necessary to allow access by equipment.

Areas Subject to Work Under the Jurisdiction of the Wetlands Protection Act

The proposed work is along the bank of the Housatonic River. In this area, a major portion of the riverbank has a shelf below the upper bank of the river. This shelf is essentially the boundary of a bordering vegetated wetland associated with the river. (See enclosed wetland report). Therefore, the following areas are identified as resource areas as delineated by White Engineering, Inc. on June 29, 1999:

Land Under Waterway: The only work being performed within the river is the installation of the silt curtain and extension of the existing absorbent boom system. These are temporary devices. This resource area extends from the edge of the bank under the river water for the entire 400 feet of proposed work area. There is a potential for a minimal short-term increase of silt in this portion of the resource area from the work activities. However, this potential will be mitigated by the placement of silt fencing along the shoreline and installation of a silt curtain approximatley 3-5 feet from the edge of the water.

Bordering Vegetated Wetland: A strip of bordering vegetated wetland (BVW) exists along the lower shelf of the riverbank. See attachments for vegetation analysis. The sheet piling and silt fence will be installed within this BVW. This area will also be cleared of trees in order to accommodate installation activities. Trees will be cut flush with the ground and roots will be removed along the proposed alignment of the sheetpile. Roots will not be removed from those trees which occur outside the alignment of the proposed barrier wall. Additionally, some soil may be excavated from the lower portion of the riverbank to prevent possible sloughing into the river during sheetpile installation. Precautions to minimize erosion into the river include the silt fence and silt curtain. The proposed work will disturb less than 5,000 SF of BVW. Temporary restoration will include the installation of geotextiles and placement of seed and mulch to stabilize the bank. These measures are expected to be temporary, less than a year in duration, since this area will be subject to further disturbance during GE's implementation of its proposed removal project for the upper ½ mile of the river. Final bank restoration will occur as part of that project.

Bank: The bank of the river is the first observable break in slope, which is essentially the BVW line. This activity will involve approximately 400 linear feet. The majority of the existing trees will be removed from the lower portion of the bank. Temporary restoration will include the installation of geotextiles, rolled erosion control products or mulch, to stabilize the bank. This area will be subject to further disturbance during GE's implementation of its proposed removal project for the upper ½ mile of the river and final bank restoration will occur as part of that project.

Bordering Land Subject to Flooding: This site is entirely within the 100-year floodplain of the Housatonic River according to the FEMA maps. Topographic surveys will be done before work begins and after to confirm and adjust any floodplain storage compensation as needed.

Riverfront Area: The installation of erosion controls, sheet piling and clearing of vegetation will occur within the 100 ft. inner riparian zone to the Housatonic River. Incidental work and storage of equipment and materials will occur within the 100-ft. outer riparian zone to the river although no disturbance is proposed in this area. Less than 10% of either zone will be disturbed. The final outcome, at the completion of this project and the removal project for the upper ½ mile of the river, will be essentially the same as the current physical conditions on the bank.

Riverbank of General Electric Company Property
USEPA Area 5/ MCP Lyman Street Site

Species List
as observed June 29, 1999

Trees (3" in dia. to 12" in dia.)	Scientific Name	Wetland Indicator Category	# of Trees
Northern Red Oak	*Quercus rubra*	FACU-	13
Eastern Cottonwood	*Populus deltoides*	FAC	2
American Elm	*Ulnus americana*	FACW-	16
Red Maple	*Acer rubrum*	FAC	5
Smooth Sumac	*Rhus glabra*	FAC	2
White Pine	*Pinus strobus*	FACU	1

Trees (over 12" in dia.)	Scientific Name	Wetland Indicator Category	
Eastern Cottonwood	*Populus deltoides*	FAC	18

Shrubs	Scientific Name	Wetland Indicator Category	
Tartarian Honeysuckle	*Lonicera tartarica*	NI	3
Red-Osier Dogwood	*Cornus stolonifera*	FACW	2

Appendix G

DEP Bordering Vegetated Wetland (310 CMR 10.55) Delineation Field Data Form

Applicant: General Electric Co. Prepared by: S. Lombardi, White Engineering Inc. Project location: Lyman Street Parking lot DEP File #:

Check all that apply:

☒ Vegetation alone presumed adequate to delineate BVW boundary: fill out Section I only

☐ Vegetation and other indicators of hydrology used to delineate BVW boundary: fill out Sections I and II

☐ Method other than dominance test used (attach additional information)

Section I. Vegetation

Observation Plot Number: A Transect Number: 1 Date of Delineation: 6/29/99

A. Sample Layer and Plant Species (by common/scientific name)	B. Percent Cover (or basal area)	C. Percent Dominance	D. Dominant Plant (yes or no)	E. Wetland Indicator Category*
Shrubs/saplings				
Red maple (Acer rubrum)	10%	15.4%	no	FAC *
American Elm (Ulmus americana)	30%	46.2%	yes	FACW- *
Tartarian Honeysuckle (Lonicera tartarica)	10%	15.4%	no	NI
Red osier Dogwood (Cornus sericea)	15%	23.1%	yes	FACW+ *
Groundcover				
Cow Vetch (Vicia cracca)	5%	19.2%	no	FAC
Canada Goldenrod (Solidago canadensis)	8%	30.8%	yes	FACU *
Canada mayflower (Maianthemum canadense)	4%	38.5%	yes	FAC- *
Birdsfoot Trefoil (Lotus corniculatus)	4%	38.5%	yes	FACU- *
Giant Solomon's Seal (Polygonatum biflorum)	5%	19.2%	no	FAC

* Use an asterisk to mark wetland indicator plants: plant species listed in the Wetlands Protection Act (MGL c.131, s.40); plants in the genus *Sphagnum*; plants listed as FAC, FAC+, FACW-, FACW, FACW+, or OBL; or plants with physiological or morphological adaptations. If any plants are identified as wetland indicator plants due to physiological or morphological adaptations, describe the adaptation next to the asterisk.

Vegetation conclusion:

Number of dominant wetland indicator plants: 3 **Number of dominant non-wetland indicator plants:** 2

Is the number of dominant wetland plants equal to or greater than the number of dominant non-wetland plants? yes no

If vegetation alone is presumed adequate to delineate the BVW boundary, submit this form with the Request for Determination of Applicability or Notice of Intent.

MA DEP: 3/95