BFI Waste Systems

BROWNING-FERRIS INDUSTRIES
OF ARIZONA, INC.

# RESPONSES TO ADEQ COMMENTS

## CHOLLA SANITARY LANDFILL

## EL MIRAGE, ARIZONA

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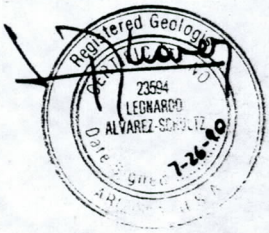


JULY 1990

A031.902

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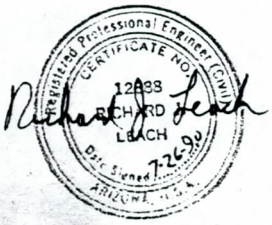


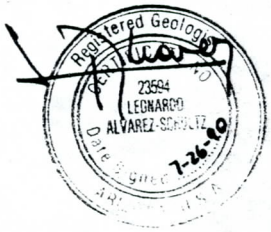


JULY 1990

# RESPONSES TO ADEQ'S COMMENTS ON APPLICATION FOR AQUIFER PROTECTION PERMIT AND APPROVAL OF SOLID WASTE DISPOSAL OPERATIONS PLAN FOR CHOLLA SANITARY LANDFILL, EL MIRAGE, ARIZONA

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# 2 RESPONSES TO JANUARY 23, 1990 REVIEW OF APPLICATION FOR SOLID WASTE OPERATIONS PLAN APPROVAL BY THE SOLID WASTE UNIT

## PART 1
## ENGINEERING MASTER PLAN

Section

2.2 **Pursuant to A.R.S Title 49 Section 767B., written verification by the applicant is required to verify that the entire site does not have grandfathered irrigation rights pursuant to A.R.S. Title 45, Chapter 2, Article 5.**

Attachment 1 includes three letters that establish that the site does not have grandfathered irrigation rights.

2.3 **The Conditional Use Permit issued by the City of El Mirage for the facility requires additional improvements for El Mirage Road. Is paving of the El Mirage Road required? If not, a description of methods used to provide an all weather access road which will not create a dust problem is needed.**

The additional improvements consist of widening the presently paved El Mirage Road as shown on the enclosed figure prepared by HNTB (Attachment 2). In addition, two lanes of El Mirage Road from Olive Avenue to Northern Avenue will be repaved with a new 3-inch asphalt concrete surface. The widened lanes



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will be paved. No dust problem will be created since the access road to the landfill will be paved through the entrance facilities. Roads from the entrance facilities to the disposal areas will be surfaced with crushed rock and watered daily to avoid dust problems.

2.4 **The location of the 100-year floodplain and the integrity of the flood control dike to protect the site from washout is required to be approved or permitted by the appropriate flood control agency prior to ADEQ approval. A copy of the formal approval or permit will be required as part of the application. Also, verification is needed regarding the appropriate local flood control agency designated by the Federal Emergency Management Agency (FEMA).**

**The plan to install bank protection features within the 100-year floodplain may result in the need to obtain approvals pursuant to Section 401 and 404 of the Clean Water Act. Written verification is needed regarding whether such approvals are not required or are required and, if so, a copy of required approvals is needed as part of the application prior to ADEQ approval.**

Arizona Revised Statutes §48-3601, et seq. (specifically A.R.S. §48-3610) provides that floodplain management duties "may be assumed by the governing body of an incorporated city or town within its area of jurisdiction if the incorporated city or town declares by resolution that it intends to assume the powers and duties, including the adoption of floodplain management regulations, pursuant to this article." The City of El Mirage has assumed floodplain jurisdiction. Attachment 3 includes copies of the relevant El Mirage resolution and a letter from the Flood

Control District of Maricopa County indicating that the County's flood control files are being transferred to the City of El Mirage.

A permit has been obtained for the site pursuant to Section 404 of the Clean Water Act (copy included in Attachment 3).

The Corps of Engineers, Los Angeles District, Regulatory Branch, has stated that a Section 401 Certification is not necessary in this instance.

3.2.1a **Information is needed on whether the soils at the on-site facilities (Union Rock & Material Corporation) have been tested for petroleum hydrocarbon content to determine if state action levels (Total Petroleum Hydrocarbons 100 ppm) have been exceeded. Information is needed on whether the facility/site has underground storage tanks abandoned or in use?**

The soils of the Union Rock & Materials Corporation (UR & MC) facilities were sampled and analyzed in February 1989 as reported in the Final Environmental Regulatory Compliance Evaluation Report prepared by SCS Engineers and submitted to UR & MC on April 3, 1989 (see Attachment 4). Ten boreholes adjacent to various buildings, tanks, and areas with soil staining were drilled and sampled. Soil samples from each borehole were analyzed for total petroleum hydrocarbons (TPH). One sample contained TPH exceeding the 100 mg/kg state action level. This sample, B-3-0, which contained 40,000 mg/kg, was obtained at the ground surface from a borehole located 5 feet north of the burner oil tank of the asphalt plant. UR & MC has developed and will implement a remediation plan for this area.



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Contaminated soils found on site could be used as part of the daily and intermediate cover materials, but will not be used as part of the soil component of the liner system.

The UR & MC facilities include the following underground tanks which are not in use and which will be removed and disposed of in accordance with applicable regulations:

- One 12,000-gallon diesel fuel fiberglass tank in the heater/dryer burner plant
- Two 10,000-gallon diesel fuel steel tanks in the fueling facility
- One 4,000-gallon gasoline steel tank in the fueling facility

3.2.1b **(1) Information is needed on whether the soils beneath the existing settling ponds or the abandoned settling ponds have been tested for moisture content or contamination at depth. Of concern is the question of whether the soils having a high moisture content, will create a problem when an impermeable liner is placed over them. (2) Could this result in the creation of a solar still, which may cause vapor entrapment and the ballooning of the liner material and require venting?**

No exploration was conducted to test the soils beneath the settling ponds. However, soils adjacent to them were explored by drilling several borings. The moisture content of soil samples collected in these borings was estimated visually and is recorded on the corresponding boring logs. No soil samples from these borings were tested for contamination since no evidence of deep contamination exists or is suspected.

Since the soils beneath the settling ponds will be removed during excavation for the disposal modules and the geologic environment of the site is not conducive to the occurrence of gases, ballooning of the liner material is not anticipated. The time needed to prepare the excavated surface for liner placement would allow soil moisture to evaporate in the arid Arizona climate. The moisture in the soil component of the composite liner will not be sufficient to cause that effect.

3.2.4 **The inactive well which will be within refuse model 1 will need to be abandoned per the Department of Water Resources requirements. ADEQ would recommend a continuous cement grouting when abandoning the well. The well (W-2) abandonment procedures and certification of abandonment will need to be submitted to ADEQ before approval to operate.**

The well will be abandoned by installing a continuous cement-bentonite slurry grout. This abandonment will occur shortly after permit approval along with the proposed monitoring well installations. The well abandonment notification, abandonment procedures, and certification of abandonment documentation according to Arizona Administrative Rules and Regulations Title 12, Chapter 15, Article 8 - Well Construction and Licensing of Well Drillers (specifically R12-15-816) will be submitted to the Arizona Department of Environmental Quality (ADEQ) as well as the Arizona Department of Water Resources (ADWR).



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3.5 **The values as stated in the application for the 100-year flood event in the Aqua Fria River, 99,000 cfs currently and 33,000 cfs with the new Waddell Dam in operation, need to be referenced as to the information source.**

Current 100-year Agua Fria flows in the vicinity of the proposed landfill were established by the 1989 Flood Insurance Study prepared by Jerry R. Jones & Associates for the Flood Control District of Maricopa County. This study indicates that the current 100-year flow at Olive Avenue is 98,780 cfs. The post New Waddell Dam flows have not been finally established. Information from a preliminary study prepared by the Corps of Engineers entitled "Hydrology for Evaluation of Flood Reduction by New Waddell Dam: Agua Fria River Below New Waddell Dam to the New River Confluence," September 1988, indicates that 100-year flows in the vicinity of the landfill are estimated to be 33,000 cfs. This reduced flow has not been used for design purposes, but is noted so that reviewing agencies will understand that the design proposed for current 100-year events will greatly exceed the required design for future 100-year events.

The following details flows at various locations along the Agua Fria river as computed for present conditions by the Corps of Engineers in 1984 and by Jerry R. Jones in 1989.

1. Corps of Engineers - 1984 - HEC-2 output data from computer printout obtained from Flood Control District of Maricopa County (FCDMC), dated 19 March 1984.

| Cross-Section No. | Location | $Q_{100}$ cfs |
|---|---|---|
| 16.58 | 1,056 ft. upstream (u.s.) from Grand | 108,205 |
| 16.291 | 1,340 ft. downstream (d.s.) from Grand (Main Channel Only) | 77,775 |
| 15.823 | 1,470 ft. u.s. from Cactus | 107,775 |
| 15.647 | 540 ft. u.s. from Cactus | 105,600 |
| 14.501 | 350 ft. u.s. from Peoria | 102,500 |
| 13.554 | 600 ft. u.s. from Olive | 99,000 |
| 12.470 | 270 ft. u.s. from Northern | 96,150 |
| 12.357 | 320 ft. d.s. from Northern | 95,800 |

2. Jerry R. Jones (JRJ) study for FCDMC (FEMA study) HEC-2 model output dated 29, January 1989, obtained from FCDMC via JRJ.

| Cross-Section No. | Location | $Q_{100}$ cfs |
|---|---|---|
| 16.72 | 1,800 ft. u.s. from Grand | 106,742 |
| 15.61 | 1,200 ft. u.s. from Cactus (Main Channel Only) | 83,181 |
| 15.42 | 200 ft. u.s. from Cactus | 102,650 |
| 14.47 | 600 ft. u.s. from Peoria | 98,780 |
| 13.34 | at u.s. side Olive | 98,780 |
| 12.38 | 175 ft. u.s. from Northern | 95,540 |
| 12.27 | 315 ft. d.s. from Northern | 95,540 |

**4.0a** **A detailed description of acceptable and unacceptable wastes for receipt at the landfill is required for the approval of the operation plan (similar to operation plan for BFI transfer station), especially for special wastes such as tires, asbestos, contaminated soils, incinerator ash, infectious waste, sewage sludge, household liquids and household hazardous wastes.**

The Cholla Landfill will accept the following wastes:

- Residential wastes consisting of household garbage and trash.



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- Commercial and Industrial wastes consisting of nonhazardous solid wastes such as packaging materials, trash, used products (tires, etc.) and manufacturing wastes.

- Yard wastes

- Construction and demolition debris such as wood, concrete, asphalt and soil.

- Dewatered sewage sludge

- Asbestos in DEQ approved plastic bags or containers and in properly moistened and covered bulk shipments. (Asbestos wastes will be immediately buried in prepared pits and covered with a layer of other wastes or soils).

- White goods

Unacceptable wastes are listed below.

- Commercial compressed gas cylinders

- RCRA hazardous waste

- Infectious waste

- Bulk liquid waste

- Oil-field drilling fluids

- Radioactive wastes

4.0b **A description of materials which will be recycled at the facility and methods used for proper sanitary controls so as not to create a public nuisance is required. Will the container for recyclables be protected from the weather and will recycling activities be managed by on-site personnel?**

Materials to be recycled at the citizen drop-off facility will be white goods, metals, paper (newsprint, high grade paper, mixed paper), cardboard, plastics, and glass. There will be weatherproof containers for specific recyclable materials.

Separate weatherproof containers will be provided for aluminum, ferric and other metals, newsprint, glass and plastics. The containers will be emptied when they are filled or at least once each week. The recycling area will be policed daily to control litter.

The recycling activities will be managed by the District Landfill Manager who is located on site.

4.0c **The concrete and asphalt which will be recycled from the waste stream will have to be stockpiled or incorporated into the landfill operations immediately. If the material is to be stockpiled, an area set aside for initial stockpiling will need to be included in the plans.**

The concrete and asphalt to be recycled will be stockpiled and used on interior roads. An area set aside for initial stockpiling is included in the plans (see revised Drawing No. 2, enclosed). The stockpile will not be larger than about 3/4 acre and will not exceed 15 feet in height.



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5.8 **More detail is required for the temporary sumps which are to be used to accumulate run-off from within the active disposal cells so the water does not come in contact refuse fill (working face). At what frequency will these drainage features be installed and maintained? How will they be designed and installed?**

The base of the landfill is 80 to 110 feet below the original ground surface. Until the landfill module has been filled to above the original ground surface the runoff from precipitation that falls directly on the fill module will have to be collected and pumped from the module. To collect the runoff temporary drainage sumps will be provided in each active module or on the floor of the adjacent partially excavated module. Runoff from excavation side slopes and the soil covered refuse, that has not been contaminated by contact with the refuse, will drain to a sump. Each sump will be designed with the capacity to contain the computed runoff from a 2-year, 24-hour storm. Each sump will be equipped with a pump sized to provide rapid discharge of the sump water to a sedimentation basin at the ground surface.

Where a sump is located over a soil covered lift of refuse any leakage from the sump that percolates through the underlying refuse will be collected in the leachate collection system above the base liner and drained to the double lined leachate sump. Any leachate collected in the sump will be recirculated through the landfilled wastes or disposed of at a publicly owned sewage treatment works (POTW).

The sumps will be inspected and necessary maintenance performed prior to the summer and winter peak rain periods, and after major rainstorms. Low berms situated to deflect surface runoff around the active working face also will be constructed at

this time. These drainage control berms will be extended or relocated as necessary to divert runoff around the wastes exposed on the working face.

A vector/mosquito abatement program will be implemented if water remains in a sump for a period longer than 3 days.

A small working face (about 200 feet wide) will be active at any given time. Rain water falling on the active working face will be collected by a continuously maintained berm within the working face area and will be treated as leachate, i.e., it will be recirculated through landfilled wastes or disposed of at a POTW.

5.10.2 **Will on-site materials be capable of obtaining a horizontal hydraulic conductivity of 1 cm/sec which is specified for the leachate collection drainage layer. Documentation needs to be provided that the hydraulic conductivity specified can be obtained.**

**The leachate collection system should not allow for leachate to obtain depths of greater than one foot on top of the geomembrane except in the double lined collection sumps. Documentation needs to be provided that the combination of the drainage layer hydraulic conductivity value; drainage layer slopes and distance between drain pipes, will not allow leachate to accumulate and exceed one foot depths on top of the geomembrane.**

Clean gravels or clean coarse sands from on-site material will be processed to have hydraulic conductivities of 1 cm/sec or greater. Attached is a figure from "Design Manual - Soil Mechanics, Foundations, and Earth Structures" NAVFAC DM-7, March 1971, documenting that clean fine gravels and clean



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coarse sand materials have hydraulic conductivities of 1 cm/sec or greater (see Attachment 5). Curves for materials currently produced by the El Mirage plant of Union Rock & Materials Corp. are plotted on the graph and show that appropriate materials are available on site. A QA/QC program will be developed to confirm the permeability of materials used in the leachate drainage layer.

The leachate collection system is normally designed with the capacity to carry twice the estimated annual leachate production rate as determined by the HELP-2 Model, with a maximum head build-up on the liner of one foot. Due to the low rainfall in the area of the Cholla landfill the HELP-2 Model indicates a very low rate of 0.0001 in/yr for percolation of precipitation through the 4-foot thick final landfill cover soil. For the operational phase, the model indicated a percolation rate of 0.0128 in/yr. Using twice the higher of these rates as a basis for the leachate collection and removal system (LCRS) design would indicate an unreasonably wide spacing of collection pipes. Therefore, a nominal collection pipe spacing of 400 feet was selected and analyzed for maximum head over the liner at twice the calculated operational phase percolation rate (0.0256 in/yr). The calculated maximum head was less than 0.1 foot indicating a very large excess capacity in the LCRS design. Calculations are presented in Attachment 6.

5.11.3 **With the facility design incorporating a composite liner and a low permeability soil final cover, the methane gas which may be generated at the facility will have no method of escape from the filled areas without affecting the integrity of the liner or final cover. A contingency plan needs to be developed to monitor explosive gas build-up within the filled areas to either show that the gas is not creating prob-**

**lems or giving detailed methods to be used for corrective actions if the gas presents a problem.**

Landfill gas produces low levels of pressure that will cause the gas to slowly permeate through cover soils. In wet climates, pressures of 12 to 20 inches may occur below the cap when the cap soils are wet clays. However, lower pressures will exist beneath the cap with lower moisture content and higher cap permeability. Testing of the clay-lined and capped Yolo County landfill (near Sacramento, California) which is located in an area having 16 inches or twice the Phoenix average annual rainfall, yielded static gas pressures beneath the clay cap of 1 to 6 inches[1], prior to installing a gas recovery system. In drier climates, such as Arizona, pressures would be less than those measured at the Yolo County landfill.

Ruptures of cap soils due to gas emissions have occurred only occasionally and have been due to high gas generation rates, where the refuse has a high moisture content, and where the cover consists of a wet low permeability clay (permeability less than $10^{-8}$ cm/sec.)[2].

Due to the dry Arizona climate, low initial moisture content of the placed refuse, and the moisture content of the soil cover of intermediate permeability, the landfill gas will not be capable of exerting high pressures. The gas will therefore be able to slowly permeate through the soil without rupturing the cover.

Methane monitoring probes will be installed at 1,000-foot centers along the site perimeter to detect the presence of methane if it

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1. Landfill Methane Recovery Assessment, Central Landfill, Yolo County, California, EMCON, November 1983.
2. Personal communication, St. Johns Landfill, Portland,, Oregon.



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escapes beneath the site. A methane control system consisting of extraction wells, collection and header piping, and a methane treatment/processing facility will be designed and installed. The treatment/processing facility will be either a flare or a recovery facility and will satisfy environmental protection standards as well as all state, federal and local air regulations applicable at that time. This system installation will start within three years after reaching final grade in Module 1, and will continue as the fill is completed.

5.12 **The actual calculations and assumptions used to arrive at the values used for $Q_r$ and $Q_a$ in the EPA risk-based algorithm equation need to be shown as the references cited do not contain the values used. In this section the values obtained from the risk-based algorithm method are not crucial for the review process, however, the values obtained are cited in the liner reliability section which is crucial in the review process.**

The proposed Cholla landfill has been designed with a liner and a leachate collection and removal system (LCRS) even though leachate generation is calculated to be low because the site is located in a dry desert environment with low annual precipitation. In addition, depth to the ground-water table under the site is in excess of 250 feet. The calculations using the HELP-2 model indicate that small amounts (0.0001 inch per year) of rainwater may infiltrate through the final cover and be potentially available to generate leachate. For leachate to be formed due to rainwater infiltration, the assumption would have to be made that the refuse has no absorptive capacity left, thus further having to assume that rainwater infiltrated through the final cover reaches the base of the landfill and is, by definition, leachate. The EPA-proposed risk-based algorithm was used to evaluate the impact

of this potentially generated leachate on the underlying aquifer if no liner and no LCRS were included in the design. This method, presented in the proposed Subtitle D regulations (Federal Register, Vol. 53, No. 168, August 30, 1988, pp. 33356-33365) involves calculating the ground-water carcinogenic risk level at the point of compliance (which is assumed to be at the site boundary). EPA proposed that a design would be acceptable if the excess lifetime risk cancer level due to continuous lifetime exposure was in the $1 \times 10^{-4}$ to $1 \times 10^{-7}$ range.

The calculated risk was $1 \times 10^{-10}$ or several orders of magnitude below the above values. The calculations and assumptions presented in the Application for the predicted rate of leachate release, $Q_R$, and of the ground-water flow rate under the site, $Q_A$, are presented below.

The values of $Q_R$ and $Q_A$ were calculated as follows.

$Q_R$, the predicted rate of leachate release to the uppermost aquifer, was calculated using the results of the Water Balance Calculations presented in Appendix I-D of Part 1 of the Application. The calculations were made using the EPA HELP-2 simulation, which predicts a percolation rate of 0.0001 in/yr through the final cover. It was then assumed that no attenuation of leachate occurs as a result of the absorptive capacity of the refuse or the 185-foot thick soils separating the base of the landfill from the aquifer. The area of the completed landfill, 175 acres, was used for this computation, as follows:

$$Q_R = 0.0001 \frac{\text{in}}{\text{yr}} \times 1 \frac{\text{ft}}{12 \text{ in}} \times 175 \text{ acre} \times 43{,}560 \frac{\text{ft}^2}{\text{acre}} \times$$

$$7.48 \frac{\text{gal}}{\text{ft}^3} = 475 \frac{\text{gal}}{\text{yr}}$$



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$$= 475 \frac{gal}{yr} \times 3.7854 \times 10^{-3} \frac{m^3}{gal} = 1.80 \frac{m^3}{yr}$$

Note 1: Due to an error in the conversion of units (using Imperial instead of U.S. gallons) the value in the Application was reported as

$$2.16 \frac{m^3}{yr}$$

Note 2: This analysis is conservative because the release rate calculated here assumes that all moisture passing through the final cover also enters the aquifer. In reality, this would not happen since the liner and leachate collection system would intercept any liquid.

$Q_A$, the ground-water flow rate of the uppermost aquifer under the site, is calculated on the basis of transmissivity data presented by Eaton et al. (1972) and Long et al. (1982). Long et al. data are presented on Table 2 of Part 2 of the Application. From this Table an average value of transmissivity of 50,000 gal/day/ft was calculated for the aquifer underlying the site area. This value was used as follows:

$$Q_A = T \times w \times i \qquad \text{Equation (1)}$$

Where

T = transmissivity

w = length of base of landfill excavation (assuming that the leachate is released at the downgradient edge of the property, coinciding with the point of compliance, POC)

i = hydraulic gradient

The length of the excavated base of the landfill is 4,400 feet and the hydraulic gradient of the ground-water table (as depicted on Figure 9 of Part 2 of the Application) is approximately 0.015 av-

eraged over a 1-mile distance through the center of the site (west half of Section 36 and east half of Section 35).

Substituting in Equation (1)

$$Q_A = 50{,}000 \frac{\text{gals}}{\text{day ft}} \times 3.7854 \times 10^{-3} \frac{m^3}{\text{gal}} \times 3.281 \frac{\text{ft}}{m} \times 4{,}400 \text{ ft} \times 0.3048 \frac{m}{\text{ft}} \times 0.015 \times 365 \frac{\text{day}}{\text{yr}}$$

$$= 4.56 \times 10^6 \frac{m^3}{\text{yr}}$$

Note 2: The value in the Application ($1.4 \times 10^6$ $m^3$/yr) was erroneously derived from a flatter hydraulic gradient which did not consider the impact of ground water overdraft west of the site.

These $Q_A$ and $Q_R$ values have a slight impact on the risk calculation, as shown in the following:

The EPA risk algorithm is

$$R = 4.5 \times 10^{-4} (Q_R/Q_A) \times e^{(TOT)(-0.029)}$$

where TOT, time of travel (in years) for leachate in the aquifer from the unit boundary to the compliance point (POC) is equal to 0 (since the POC and the unit boundary are assumed to be coincident in this analysis).

Substituting in the risk equation

$$R = 4.5 \times 10^{-4} (1.80/4.56 \times 10^6) \times 1 = 1.78 \times 10^{-10}$$

The calculated value for R presented in Section 5.12 of Part 1 of the Application is $6.9 \times 10^{-10}$. The new value is more conserva-



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tive than the value above. Therefore, this new value confirms the validity of the liner reliability section presented in Appendix II-E of Part 2 of the Application.

Although the calculated risk is much lower than what EPA considers acceptable for unlined sites, BFI considers it prudent to include a composite liner in the design.

6.1a **The inspection of incoming waste loads for hazardous materials by the gate/scale personnel needs to be further explained. Will there be several employees at the scale house with various job functions, or will there be a dedicated trained employee at the scale house? Will incoming loads be required to be covered? If so, how could scale house personnel visually inspect?**

BFI uses a system approach for monitoring incoming loads. This system consists of (1) visual inspection, (2) television camera surveillance (at the gatehouse), and (3) radiation detection scanning (at the gatehouse). This system is reinforced through long-established employee training programs aimed at identification of unauthorized waste. It is also reinforced by stiff warnings (see 6.5.7) to customers who use the site, required manifests for special wastes, and a reputation for denial of disposal privileges to those who attempt to circumvent disposal policies.

Visual inspections are carried out both at the entrance and at the working face. At the gatehouse, attendants inspect the surface of loads particularly for evidence of closed containers and liquid wastes. Because covered or enclosed loads (packer trucks) cannot be readily inspected in the truck they will be observed by spotters and equipment operators as the wastes are unloaded at the working faces.

All loads will be required to be covered. Loads coming in vehicles of familiar customers will typically be allowed to proceed to the working face for inspection. Loads of unfamiliar customers will be inspected both at the entrance and at the working face.

All vehicles carrying unacceptable waste will have the entire load rejected. Even if an unauthorized load is deposited at the working face, it will be removed by the operator and returned to the customer or taken to an authorized facility at the customer's expense.

6.1b **The public tipping containers should be unloaded at the end of the working day even if not full. The waste should be compacted and covered daily.**

Comment noted. The public containers will be unloaded daily at the end of the working day. The waste will be compacted and covered daily.

6.2 **When a refuse fill module has been completed and will not receive waste for at least 180 days, intermediate soil cover of 12 inches should be applied within 30 days of completing the module.**

Comment noted. Twelve inches of intermediate soil cover will be placed over refuse in any module within 30 days of completing the module, if the module will not receive waste for at least 180 days.

6.5.5 **A more detailed discussion of available fire fighting equipment is necessary. This includes information on the closest fire hydrant to the facility which is connected to the El Mirage water main and is the closest hydrant able to be uti-**



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**lized at the landfill facility by city fire fighting personnel. What is the amount of water to be stored at the facility available for the fire fighting and will city personnel be able to utilize it?**

The closest fire hydrant to the facility is located on El Mirage Road 1 mile north of the site. It will be available to the City of El Mirage volunteer Fire Department, who will service the site.

10,000 gallons of water stored permanently on site, primarily for dust control, will be available for fire fighting by site personnel as well as by the city's Fire Department.

Fires in landfill wastes will be controlled by smothering the flames with soil using a bulldozer. The burned wastes will then be separated from other wastes, spread on the soil-covered landfill or natural ground surface, and sprayed with water as necessary to extinguish any embers or smoldering materials. The burned materials will then be reburied in the landfill, after they are completely extinguished.

6.5.7 **Information needs to be provided on proposed language for entrance signs and warning signs. Placement and size of the signs shall conform to the requirements of the El Mirage Zoning Ordinance.**

The proposed language for the entrance sign is as follows:

- Cholla Sanitary Landfill
- Operated by Browning-Ferris Industries of Arizona
- Open to the public Monday - Friday from dawn to dusk and Saturday until noon.
- Closed Sunday, New Years Day, Thanksgiving and Christmas

Two warning signs are proposed with the first as follows:

WARNING

Transporting any unauthorized waste to this facility for disposal is prohibited by law. Persons violating this prohibition are subject to civil and criminal prosecution.

Examples of materials which are NOT permitted in the Cholla Sanitary Landfill include (but are not limited to): Automobile Batteries, Compressed Gas Cylinders, Hazardous Waste, Infectious Waste, Liquid Waste, Oil-Field Drilling Fluids, Pharmaceutical Wastes, Radioactive Wastes, Septic Tank Pumpings. These wastes require special handling.

The second warning sign will read as follows:

ABSOLUTELY NO SALVAGING

ALL CHILDREN AND ANIMALS MUST REMAIN IN VEHICLE



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Placement and size of the signs will conform to the requirements of the El Mirage Sign Code Ordinance 35-003.

6.9 **A proposed organizational chart needs to be provided regarding the employees responsible for on-site activities along with a brief job description for each job classification.**

A proposed organizational is included as Figure 1.

The Regional Manager, headquartered in San Jose, California, is responsible for all aspects of the operation. He is assisted by an Environmental Compliance Manager, also headquartered in San Jose, California, who is responsible for all technical aspects of environmental monitoring and compliance.

The following lists the initial operations minimum and probable maximum number of employees responsible for on-site activities and describes their duties.

District Landfill Manager (1) - responsible for all of the operations; main contact with regulatory agencies; has supervisory authority over all site employees.

Heavy Equipment Operators (3-6) - responsible for operating the landfill equipment (dozer, compactors, scraper, grader, loader)

Site Mechanics (1-2) - responsible for servicing the landfill equipment

General Laborers (2-5) - responsible for spotting loads, controlling landfill traffic, providing litter control and inspecting for hazardous materials at the working face.

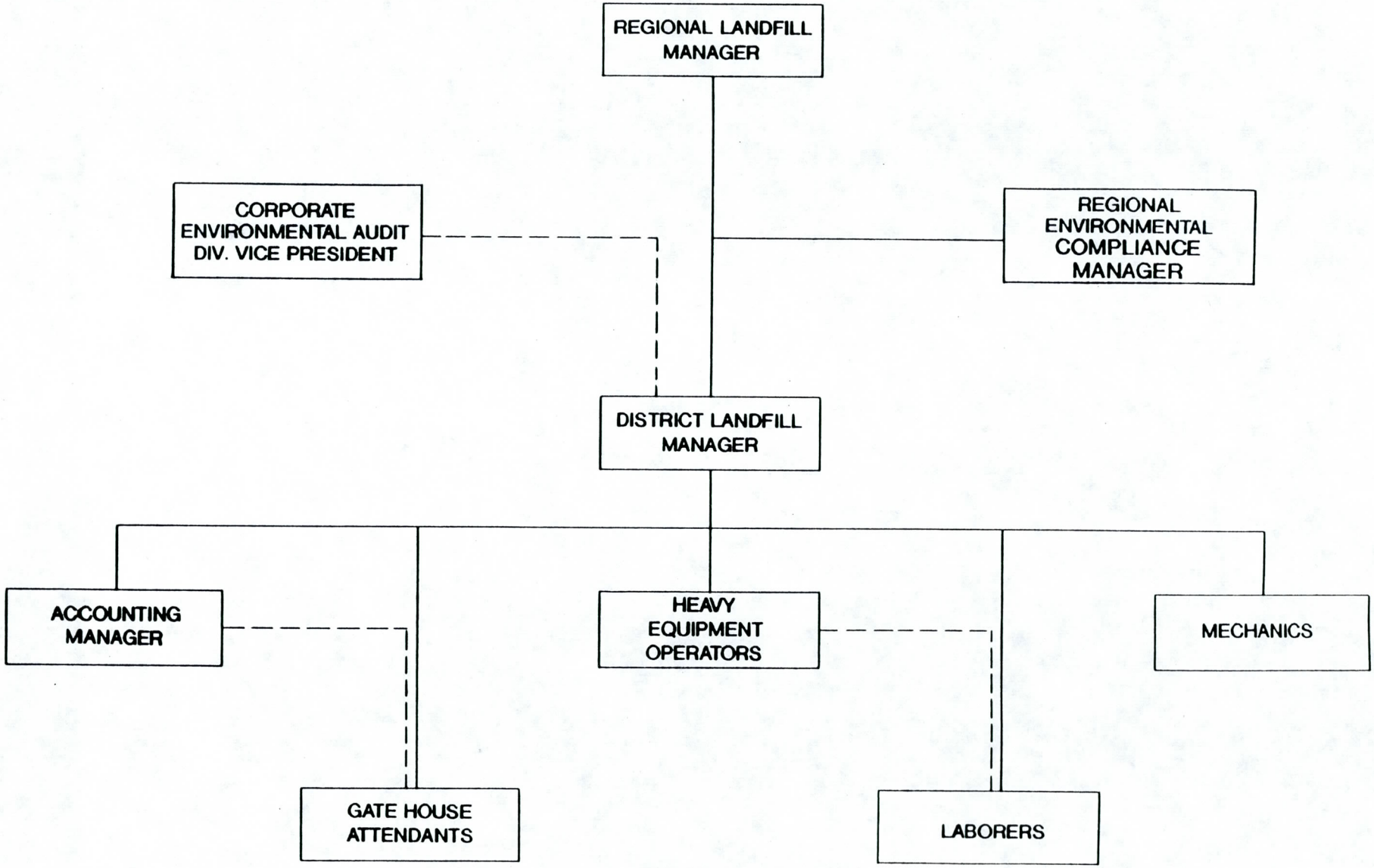


FIGURE 1
ORGANIZATIONAL CHART

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Gate House Attendants (1-3) - responsible for collecting fees, visually inspecting loads for hazardous materials through a video system, recording unacceptable loads

Accounting Manager (1) - responsible for keeping books, paying bills

6.11 **Any change, modifications or other deviation from approved plans will require written approval by ADEQ before implementation.**

Comment noted. BFI will submit any substantial changes, modifications or deviations from approved plans to ADEQ for approval before implementation.

7.3 **It appears that the gas monitoring probes will be outside the perimeter 6 foot chain link perimeter fence. What method(s) will be used to protect these monitoring probes from being struck by a vehicle or vandalized, besides the locking cap.**

**Could the gas monitoring probes to be installed along the flood control dike also be used as neutron monitoring wells to determine if wetting fronts may be moving through the dike towards the landfill? If not, some type of vadose zone monitoring should be established along the east side of the facility to determine if surface water from the Agua Fria River is seeping through the west bank which may affect the facility.**

The gas probes will be located adjacent to landscaped berms and out of the perimeter maintenance road where vehicular damage is unlikely. The 8-inch steel surface casing set in concrete and lockable steel cap is reasonable protection. In any



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case, if damage to the gas probes occurs, it will be promptly repaired.

The gas monitoring probes to be installed along the west bank of the Agua Fria River could not be used as neutron monitoring wells for several reasons: (1) neutron probes require casing with a minimum of 2 inches in diameter and the gas monitoring probes are 1 inch in diameter; (2) the neutron probes would be measuring the hydrogen content of the 1-inch PVC casing rather than the soil moisture nuclei; (3) neutron probe casing has to be aluminum for calibration purposes; (4) the presence of the 12-volt detector within a possible methane gas environment; (5) neutron probes should not be installed within a backfill since under this installation only moisture content of the backfill is measured.

As indicated in the unsaturated ground-water flow modeling for the river bank area and the subsequent confirmation laboratory hydraulic conductivity and moisture retention data (see Attachments 7 and 8), an advancing moisture front or saturated conditions would not impact the proposed landfill excavation area under the most conservative conditions as indicated by cross-section No. 9 in the northern part of the site. The modeled moisture front does not come closer than 100 feet (horizontally) to the proposed excavation lined slope.

8.3 **Will the wastewater from the ancillary facilities be connected to city sewer or will a septic system be installed? If a septic system is to be installed, please provide details of the septic system including size of septic tank(s), locations of tank(s) and leach lines or seepage pits and estimated wastewater flows. Will the maintenance building have floor drains and will these drains be connected to the septic sys-**

**tem/city sewer and will an oil-water separator be incorporated.**

A septic system will be installed to handle wastewater from the ancillary facilities. Wastewater will be generated at two locations: the landfill office and the equipment maintenance building. Proposed staffing and related estimated wastewater discharges are as follows (Table 1, Engineering Bulletin No. 12, ADEQ, June 1989):

| | |
|---|---|
| • Landfill office - 6 employees at 25 gpd = | 150 gpd |
| • Equipment maintenance building (w/showers) - 15 employees at 35 gpd = | 525 gpd |
| Total Estimated Wastewater Discharge | 675 gpd |

Using a design safety factor of 1.6, the total design flow will be 1,080 gpd. Consequently, the septic tanks will be sized to accommodate this flow. If more than one septic tank is used the minimum tank capacity will be 960 gallons (as recommended in Engineering Bulletin No. 12, Part IV, page 24 [ADEQ, June 1989]).

The septic tank and leach field will be located immediately east of the maintenance building and designed to the requirements of the City of El Mirage and Maricopa County Health Department. Borings and percolation tests will be made to confirm the suitability of the selected leach field area and to establish design parameters as specified in Engineering Bulletin No. 12, ADEQ, June 1989. A copy of the design report and permit application to



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the City of El Mirage and Maricopa County Health Department will be forwarded to the ADEQ.

The septic tank and leach field will be located away from the edge of the landfill modules and be at least 100 feet from the composite liner system.

The maintenance building will have floor drains and will incorporate an oil-water separator.

10.4 **Depending on the sampling results of the first 30 years of post-closure monitoring of groundwater and methane, the state may extend post-closure monitoring past the initial 30-year period. Perpetual care maintenance may be required for the site to keep it from causing a public nuisance or environmental hazards. A notation on the deed to property shall be required which shall state that the land has been used as a landfill for municipal refuse.**

Comment noted.

10.5 **Closure and post-closure costs will need to be updated yearly to reflect cost in current dollars (inflation adjustments) and changes to facility plans or operations which may increase the closure and post-closure cost estimates.**

Comment noted. Closure and post-closure cost estimates will be revised yearly.

**Appendix I-B Geotechnical Engineering and Slope Stability**

**A Quality Assurance and Quality Control (QA/QC) Plan will need to be developed for installation of the composite liner, leachate collec-**

**tion system and operational soil layer placement. The QA/QC plan should include the following at a minimum:**

- **Sampling frequency and testing methods used to determine that the soil liner hydraulic conductivity in place will meet the $1 \times 10^{-6}$ cm/sec criteria.**
- **HDPE liner field seaming methods and testing requirements.**
- **Methods for applying the drainage blanket over the HDPE liner.**
- **Methods for seaming the geotextile.**
- **Methods for applying the one foot thick soil operations layer cover.**

**Will a geotextile be used between drainage layer material (gravel) and the HDPE liner to protect the liner from rupturing by the gravel material?**

**How and when will the 2 foot thick soil operations layer be applied directly over the HDPE liner on the side slopes? Will this soil layer, (on the side slopes), exceed the friction angle between the soil and liner interface especially on the 2:1 (H:V) slopes?**

**Will a geotextile material be used between the primary soil liner and leak detection sump gravel layer (see Drawing 7 Detail 13)?**

The QA Procedures for Earthwork Construction at Cholla Sanitary Landfill are presented as Attachment 10. QA/QC testing frequencies for geosynthetic materials are described in the QA Procedures for Synthetic Containment System Construction presented as Attachment 11.

If material larger then 1/2 inch is used in the drainage layer, a cushion geotextile, or HDPE rub sheet will be used under the gravel drainage material. If fine-rounded gravel (smaller than 1/2 inch) or coarse sand is used for the drainage layer, a geotextile will not be needed to protect the liner from rupture by the drainage material.

The 2-foot-thick soil operations layer will be applied to the slopes immediately ahead of refuse placement adjacent to the slope. As filling progresses in the cell, the side slope buffer will be placed so that it is a minimum of 5 feet above the waste.



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Soil will be deposited by scrapers on top of the refuse lift. The scraper will not get closer than 10 or 15 feet from the side slope to prevent damage to the HDPE liner. Dozers will then carefully push the soil up the lined slope so that it is a minimum of 5 feet vertically above the lift (15 feet slope length). The thickness of the operations layer will be monitored to be no less than 2 feet (measured perpendicular to the slope).

The friction angle between the operations layer material and the HDPE liner is greater than the friction angle between the HDPE and underlying low permeability soil. The operations layer placement method negates potential concern for liner-operations layer stability.

A geotextile filter layer will be used between the primary soil liner and the leak detection sump gravel.

# PART 2
# HYDROGEOLOGIC STUDY

3.2.1 **Probablistic calculation of ground acceleration stated in this section indicates that there is a 10 % chance of exceeding 0.004g in a 50-year period. While in Part I it states 10% chance of exceeding 0.04g in a 50-year period. See comments for Part I Section 3.6.4.**

As stated in Section 3.6.4 of Part 1 of the application, the site lies in an area with a 90 percent probability that the acceleration in rock will not exceed 0.04 g in a 50-year period (Algermissen and Perkins, 1976). Algermissen et al. (1982) indicate that the rock acceleration in the site area also has a 90 percent probability of not exceeding approximately 0.1 to 0.11 g in a 250-year period (which is the same as a 10 percent probability of being exceeded in that period). The site is located immediately adjacent to the 10 percent of gravity contour on Plate 3 of Algermissen et al. (1982).

Since Arizona does not have a requirement that stipulates acceptable seismic design methods, the performance of the facility under earthquake loading conditions was also evaluated on the basis of the pending EPA landfill regulations. The proposed 40 CFR Part 258.14 (Subtitle D regulations) specify that "all containment structures...must be designed to resist the maximum horizontal acceleration in lithified material for the site."

These regulations are proposed for "seismic impact zones," which are defined as areas "with a 10 percent or greater probability that the maximum horizontal acceleration in hard rock,



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expressed as a percentage of the earth's gravitational pull (g), will exceed 0.10 g in 250 years." The proposed landfill site is in a "seismic impact zone" as defined by the proposed regulations.

The tectonic model used by Algermissen et al. (1982) includes a magnitude 7.3 earthquake to arrive at a horizontal peak rock acceleration of about 0.11 g. Due to the site conditions the peak ground acceleration would also be 0.11 g based on relationships derived by Seed and Idriss (1982). Using these parameters (magnitude 7.3 and 0.11 g peak ground acceleration), an analysis using procedures described by Newmark (1965) and Makdisi and Seed (1977) was performed to estimate the effects of the regulatory earthquake on the facility's slopes and its consequences on the containment structures.

The Newmark procedure is based on the assumption that a slope will move and permanently deform by ground displacements during an earthquake when the yield acceleration is exceeded. The yield acceleration is defined as the average acceleration that imparts a horizontal inertial force on a potential sliding mass so as to produce a safety factor of unity (1.0) against sliding. The yield acceleration was computed for the critical sliding surfaces shown on Figures 1B-14 through 1B-18 of Appendix B, Volume I of the Application for Aquifer Protection Permit and Approval of Solid Waste Disposal Operations Plan. The permanent displacements were then estimated by comparing the ratio of the computed yield accelerations to the peak ground acceleration for a 7.5 magnitude earthquake using the methodology presented in Makdisi and Seed (1977).

The deformation analyses indicated that there would be no permanent displacements due to earthquake shaking for the landfill excavation slopes and final landfill slopes, and that there would

be a total displacement of 1 foot or less for the temporary operating refuse slopes. Deformations of this magnitude are not significant and are not expected to be detrimental to the performance of the landfill's liners and leachate collection system.

References

Makdisi, F.I. and H.B. Seed. 1977. *A Simplified Procedure for Estimated Earthquake-Induced Deformations in Dams and Embankments.* Report No. UCB/EERC-77/19. University of California, Berkeley, California.

Newmark, N.M. 1965. *The 5th Rankine Lecture - Effects of Earthquakes on Dams and Embankments.* Geotechnique, Vol. 5, No. 2.

Algermissen, S.T., Perkins, D.M., Thenhaus, P.C., Hanson, S.L., and Bender, B.L. 1982. *Probabilistic Estimates of Maximum Acceleration and Velocity in Rock in the Contiguous United States.* Open-File Report 82-1033, United States Department of the Interior Geological Survey.

## Appendix II-A UNSAT-2 Computer Model

**This computer model assumes flow is to occur in the vertical plane and hydraulic conductivities are based on vertical flow. Consequently, the hydraulic conductivity (Kz) in a vertical direction will be less than in a horizontal direction (Kx). It is not unusual to find Kz values that are only one-fifth or one-tenth of Kx values and is the rule rather than the exception for undisturbed alluvial deposits (Bouwer 1978). Aquifers and ground-water basins deposited by flowing water may also exhibit anisotropy in the horizontal plane itself, because Kx tends to be greater in the downstream direction than perpendicular there to. Such medias then have three-dimensional anisotropy with principal K axes in the vertical direction, horizontal direction parallel**



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**to past prevailing stream flows, and the horizontal direction at a right angle to these flows (Bouwer 1978). Please address how these concerns were or were not incorporated into the computer model and how they may change the results.**

**Two of the three sections (7 and 8) that were used to model flows through the vadose zone beneath the site incorporated a soil/cement region in the west bank of the Agua Fria River. Will the soil/cement region be incorporated into the design of the flood protection structures? If so, it should be included in the application. If not, it should not be included in the computer model.**

At the proper modeling scale, the hydraulic conductivity within each layer can be assumed to be homogeneous and isotropic. However, on the scale of the thickness of several hundred feet, the hydraulic conductivity is anisotropic. To see this, consider a square cross-sectional area of a hypothetical aquifer composed of isotropic and homogeneous horizontal layers of different hydraulic conductivities, where the $i^{the}$ layer is $d_i$ units thick with hydraulic conductivity, $K_i$ and,

$$\sum_{i=1}^{n} d_1 = d$$

where d is the length of the side of the square cross-sectional area. To the scale of the length d, the anisotropy is calculated as follows.

For a discharge in the vertical direction,

$$K_z = \frac{d}{\frac{d_1}{K_1} + \frac{d_2}{K_2} + \cdots + \frac{d_n}{K_n}}$$

For a discharge in the horizontal direction,

$$K_x = \frac{K_1 d_1 + K_2 d_2 + \cdots K_n d_n}{d}$$

As can be seen $K_z \neq K_x$, and so to the scale of length d, the system is anisotropic.

The process of layering by lithological units produces anisotropy, but within those lithological units, the hydraulic conductivity is considered isotropic and homogeneous. It should be noted that the anisotropy is also a function of the wetting front and changes as more and more layers are exposed to flow. Therefore, the model treats anisotropy in an implicit but exact manner.

The grids for each section modeled were constructed to account for the various layers of material properties encountered. In the simulation, the hydraulic properties of the materials were assigned from three sources: (1) laboratory permeability measurements, (2) in-situ tests in selected boreholes, and (3) published ranges of saturated hydraulic conductivity based on soil texture (Maddock et al., 1989a). Subsequently, 14 undisturbed cores taken from boreholes SHC-5 and SHC-22 near Section 8 and from borehole SHC-9 near Section 7 were analyzed by Daniel B. Stephens and Associates to determine values of hydraulic conductivity and the moisture retention characteristics. These data indicated that the estimated values used for the simulation were conservative (Maddock et al., 1989b, see Attachments 8 and 9). By assigning hydraulic properties to individual layers in the model, anisotropy induced by layer heterogeneity has been incorporated in the model. The degree of anisotropy within a homogeneous unit is minor and was not considered in the model. The ability of the model to account for vertical variations of hydraulic properties is demonstrated in Figures 35 and 46 of Maddock et al. (1989a). Figure 35 shows the pressure head distribution at 0.5 day, and indicates that the wetting front is moving downward through uniform material with little lateral movement. Figure 46 shows the influence of the clay layer on the movement of the wetting front. The clay layer impedes flow in the vertical direction while enhancing lateral movement of the wetting front.

Within a lithologic unit, anisotropy can result from imbrication (Bouwer, 1978). In this situation, the hydraulic conductivity in the vertical direction will be less than that in the horizontal direction because of the manner in which the alluvial materials were laid down. The type of measurement technique used determines whether the measurement is representative of the horizontal or vertical hydraulic conductivity. Analysis of in-situ borehole tests demonstrates that these tests are most representative of the horizontal hydraulic conductivity as they are weighted towards lateral flow away from the borehole (Philip, 1986, 1987). Results from vertically ori-



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ented cores placed in a permeameter in the laboratory provide estimates of hydraulic conductivity in the vertical direction. The differences in results arising from the choice of measurement technique can be seen from the data obtained from two of the boreholes at the proposed Cholla Landfill Site. In the table below, values of saturated hydraulic conductivity determined from in-situ tests and vertical cores measured in the laboratory are compared.

| Borehole | Depth (ft) | USCS | In-Situ K (cm/s) | Laboratory K (cm/s) |
|---|---|---|---|---|
| SHC-5 | 79.5 | SM-SC | | $2.32 \times 10^{-4}$ |
| | 78.9 to 80.9 | SC | $1.31 \times 10^{-3}$ | |
| SHC-22 | 20.3 to 23.9 | SW | $3.51 \times 10^{-3}$ | |
| | 23.5 | SW | | $1.94 \times 10^{-3}$ |

At all tested locations, the value of the saturated hydraulic conductivity measured with the in-situ borehole test was larger than that determined from the laboratory cores.

An examination of Tables 1-3 in Maddock et al. (1989a) shows that the sources of the values of the saturated hydraulic conductivity used in the model were mainly from in-situ borehole tests. Because the in-situ tests are more representative of horizontal flow, these values are representative of the horizontal hydraulic conductivities within the layers. A comparison between the values of saturated hydraulic conductivity used in the model and those obtained from the laboratory cores is contained in Maddock et al. (1989b). These comparisons show that the values of the hydraulic properties used lead to a conservative estimate of the advance of the wetting front during the course of the flood event used in the simulation.

These results demonstrate that the model effectively accounts for anisotropy of the hydraulic properties at this site.

Sections 7 and 8 incorporate a soil-cement region in the west bank of the Agua Fria River since it had been considered earlier as a potential erosion protection scheme. The soil-cement erosion protection scheme is not being proposed; instead, a rip-rap erosion protection blanket would be placed on the river bank, as presented in the Application. Section 9 does not incorporate soil-cement and presents a worst-case scenario in terms of time of travel. Inspection of Section 9 UNSAT 2 computer model results indicates that the saturation front does not reach the proposed excavation

either. Therefore, it is not necessary to rerun the model for Sections 7 and 8 to demonstrate the lack of flood impact on the proposed excavation.

**References**


Bouwer, H. *Groundwater Hydrology*, McGraw Hill, New York, 480p, 1978.

Philip, J.R. Approximate analysis of the borehole permeameter in unsaturated soil. Water Resour. Res. 21:1025-1033, 1985.

Philip, J.R. Steady infiltration from buried discs and other sources. Water Resour. Res. 22:1058-1066, 1986.

Philip, J.R. The quasilinear analysis, the scattering analog, and other aspects of infiltration and seepage. In Infiltration Development and Application, edited by Y.-S. Fok, pp. 1-27. Water Resour. Res. Center, Honolulu, Hawaii, 1987.

Maddock III, T., Sully, M.G., Guzman, A., Wittmeyer, G., and Paleologos, E. 1989a. Unsaturated Ground-water Flow Modeling for Proposed Cholla Landfill. Dept. of Hydrology and Water Resources, University of Arizona, Tucson, Arizona. July, 1989. (Included as Appendix II-A of Part 2: Hydrogeologic Study).

Maddock III, T., Sully, M.G. and Wittmeyer, G. 1989b. Unsaturated Ground-water Flow Modeling for Proposed Cholla Landfill-Part 2. Dept. of Hydrology and Water Resources, University of Arizona, Tucson, Arizona. October, 1989. (Included as Attachment 7).





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## Appendix II-C Exploration for Earth Fissures

**Please provide information as to why shear wave energy generation devices were not used, as this method has been documented as capable of identifying earth fissures in their early stages of development. Conventional refraction-reflection seismic methods are only documented in the application (by personnel conversations) as identifying fissures along irrigation canals.**

To determine whether earth fissuring was occurring or had occurred at the proposed solid waste disposal site, a seismic investigation using refraction methods was undertaken in August, 1988. The results of that investigation have been reported in previous documents (Terrametrics Associates, November 1988; EMCON Associates, February and September 1989). Prior to conducting the seismic work, considerable time and effort was spent confirming that the seismic refraction method/devices utilized would detect non-visible and visible earth fissuring. The following describes in detail the preliminary work done prior to choosing seismic refraction as the preferred method to detect earth fissures.

### Preliminary Work

Wrege, Hasbrouck, and Schumann (1985) presented a paper describing seismic work conducted in 1983 which indicated that both shear and surface waves are attenuated as they cross visible and non-visible earth fissures in the alluvium. In June and August 1988, prior to beginning the seismic work at the proposed waste disposal site, Mr. Herbert Schumann of the U.S. Geological Survey and one of the coauthors of the paper, was contacted by EMCON and Terrametrics to discuss the application of the method to search for fissures at the site, as well as to obtain information regarding documented earth fissures in the proposed waste site vicinity. Mr. Schumann confirmed his belief in the technique and suggested that Mr. Gus Harrell, U.S. Bureau of Reclamation (USBR), Denver, be contacted, as the USBR had worked with similar techniques to detect non-visible and visible earth fissures.

Mr. Harrell was then contacted in early August, 1988. Mr. Harrell described the original USBR work which had been undertaken for the Central Arizona Project and which included the work conducted by Wrege, Hasbrouck, and Schumann in 1983. The objective of the USBR work was to evaluate which geophysical method was the best for discovering earth

fissures before they became detectable at the ground surface by visual examination. The Central Arizona Project engineer subsequently requested of the USBR in January 1988 a more suitable method than visual examination to identify incipient earth fissure development at regular intervals of time along their system of canals. Since the canals are shotcrete lined, it was feared that the lining could be damaged by earth fissures.

The more recent USBR study was conducted in June 1988, in two areas of known fissuring (Arizona State-owned land northwest of the intersection of Baseline and Meridian Roads, and Reach 2 of the Salt-Gila Aqueduct in the area of the "Junkers" fissure). Common offset seismic refraction methods were used in a series of cross profiles transverse to the known earth fissuring. This method was used because the shot-to-geophone distance is kept constant and is minimal. Some of the seismic lines (transects) were located across the extension of known fissures into areas where earth fissuring was not visually apparent. The conclusion reached was that this seismic method was successful in detecting visible and non-visible earth fissuring (fissures in their incipient stages). The USBR used standard energy sources used in normal refraction work which generate primarily compressional or P waves. They found measurement of attenuation and/or arrival time delays of P waves to give results equally satisfactory if not better than shear and surface wave analysis from work done by the USGS in 1983.

Following the August 1988 conversations with Messrs. Schuman and Harrell, it was decided that standard seismic refraction profiling techniques (surface and P-wave analysis) would be the most accurate and effective method to identify non-visible incipient earth fissures at the proposed waste disposal site.

In April 1990, Mr. Harrell was contacted again. He confirmed the earlier conversations and noted that a preliminary draft report on the work conducted for the Central Arizona Project had been prepared in December, 1988 (Sirles, 1988). He sent a copy of the report for inspection. The USBR work confirmed (in support of the seismic work done for the Cholla site) that both P and surface waves can be used to detect non-visible and visible earth fissures. Mr. Harrell also noted that the overall USBR project work showed that seismic refraction techniques (P and surface wave analysis) were more accurate, faster, and more cost effective than several



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other geophysical methods evaluated, including shear wave analysis, electrical resistivity and ground penetrating radar.

**Conclusions**

The following summarizes the conclusions arrived at from the results of the work performed by the USBR and USGS for the CAP and by Terrametrics for the Cholla site.

- Conventional refraction - reflection seismic methods (P and surface wave analysis) have not been documented as having been performed along irrigation canals; instead, they have been performed by the USBR along the Central Arizona Project Aqueduct utilizing common offset refraction seismic surveys. Shear wave analysis was also conducted by the USGS along the Central Arizona Project Aqueduct. The natural terrain explored with both of these techniques is similar to that of the proposed waste site area.

- Seismic records of the work conducted by Wrege et al., indicate that surface waves were as attenuated as shear (S) waves where they crossed known earth fissures.

- The USBR concluded from their seismic records that: (1) If fissure-caused surface voids are present and fissure infilling with loosely compacted soils has occurred, then surface waves are attenuated and body waves (P and S) will be significantly delayed; (2) Where fissures are present at depth with no surface expression, both body and surface waves are attenuated; (3) Body and surface wave arrival times are delayed where tensional fractures or loosened soil areas due to tensional stresses occur.

- Body and surface waves have been identified as capable of identifying non-visible and visible earth fissures through either delay in wave arrival times or decrease in wave amplitude (attenuation of wave energy).

- Surface waves are generated by either shear wave or primary wave energy sources. The use of shear wave

flows lasting longer than the 100-year flood flows in the Agua Fria River, the following paragraphs discuss theoretical scenarios of water contacting the liners.

A conservative modeling of subsurface seepage flood waters from the Agua Fria River has shown that the water produced by a 100-year flood will not contact the landfill liner. Nevertheless, this response will address the potential consequences and actions that would be taken to mitigate such contact under the following three scenarios.

1. Landfill lined but no wastes placed in the landfill.

   If subsurface seepage from the Agua Fria River were to reach the landfill slope and rise above the level of the base liner, leachate collection system and protective soil layer the possible consequences would be bulging and displacement at the toe of the slope. The low permeability soil component of the liner would be displaced and water might collect behind the HDPE liner and "float" the liner. The liner would float (if not held down by the LCRS system or soil), because the HDPE liner specific gravity is less than the specific gravity of water and because of the water pressure behind the HDPE liner. This condition (floating or lifting of the HDPE liner) could be readily (visually) observed.

   Past observations with water collecting under HDPE liners indicates that the liner would not be ruptured unless the water pressure exceeded the liner seam strength. This condition was observed by D. Buranek (EMCON Associates) during the construction of Pond P-9 at the Kettleman Hills Hazardous Waste Management Facility near Kettleman City, California in March/April 1986. Water from heavy precipitation collected under the HDPE liner of the partially lined pond. The water collected at the pond low point and lifted the HDPE liner.

   If this were to occur, BFI would (1) drain the collected water from behind the HDPE liner, (2) remove the protective soil cover, leachate collection and removal system (LCRS) material, and the HDPE liner from the affected area, (3) remove, dry, and recompact the low permeability layer material from the affected area, and (4) reinstall the HDPE liner, LCRS, and protective soil cover. The repair work would be performed under the original installation QA/QC procedures.



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2. Partially Filled Module

Once waste is placed in the module (against the east slope) the height or level of seepage against the landfill slope needed to cause deformation or damage to the lining system would increase. Thus the level of seepage against the lined landfill slope would have to be above the level of the refuse in order for there to be a problem. This is because the hydrostatic pressure exerted by the water behind the lining system would be opposed by the weight and strength of the protective soil cover and the refuse.

If this condition were to occur, bulging or deformation of the lined slope above the level of the refuse could result. Water would collect behind the HDPE liner and "float" the liner. Again, this condition could be readily visually observed.

To remediate the condition, BFI would first pump the water from behind the HDPE liner to limit contact of this water with the refuse. This would be accomplished by cutting a small round or oval hole in the liner, inserting a hose (pump stinger) and pumping the water to a Baker tank or water truck. This step may have to be performed in stages at different locations to remove the trapped water. BFI would then remove the refuse from the area of the affected slope and follow the same repair sequence as described above.

3. Completed Landfill

The completed landfill will extend above the original ground surface. The resulting refuse weight will be much greater than the hydrostatic head from subsurface flow from the Agua Fria river. Therefore, there will be no liner displacement or damage due to hydrostatic forces from Agua Fria River flood water seepage.

## II-1 Submit copies of final determinations by appropriate agencies dealing with floodplain and river bank issues.

The City of El Mirage assumed floodplain management authority for those portions of the Agua Fria River within the corporate limits of the city on September 14, 1989. A copy of resolution No. 89-09-039 documenting this is included in Attachment 3.

A copy of a September 19, 1989 letter from the Flood Control district of Maricopa County to the Mayor of the City of El Mirage acknowledging the assumption of floodplain management is also included in Attachment 3.

The Los Angeles District, Corps of Engineers of the Department of the Army issued nationwide permit No. 26 dated March 30, 1990 for BFI's application No. 90-112-CL dated March 2, 1990 for the construction of the proposed rip-rapped bank stabilization of the Agua Fria western bank between Olive Avenue and Northern Avenue. A copy of the permit is included in Attachment 3 also. The City of El Mirage approved on June 26, 1990 BFI's application for a floodplain development permit for the bank stabilization and levee repair (see correspondence in Attachment 3).

## II-2 Clarify which is the amount of precipitation that would be directly collected by an active cell and whether this water will be discharged.

Using the Maricopa County Flood Control District Uniform Drainage Policies and Standards (UDPS) for Maricopa County, Arizona (February 25, 1987), EMCON estimates that approximately 94,000 cubic feet of runoff from a 2-year 24-hour storm would drain to the Module 1 or 2 temporary sump. This water would be pumped to the sedimentation basin immediately north of Module 1.

Until the landfill module has been filled to above the original ground surface, the runoff from precipitation that falls directly on the fill module will have to be collected and pumped from the module. To collect the runoff, temporary drainage sumps will be provided in each active module or on the floor of the adjacent, partially excavated module. Runoff from



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excavation side slopes and the soil-covered refuse, that has not been contaminated by contact with the refuse, will drain to a sump.

A small working face (about 200 feet wide) will be active at any given time. Most of the rain water falling on the working face will be absorbed by the exposed refuse. Any runoff from the active working face will be collected by a continuously maintained berm within the working face area and will be treated as leachate, i.e., it will be recirculated through landfilled wastes or disposed of at a POTW.

Assuming 20 percent of the incident precipitation runs off the active working face, approximately 200 cubic feet of potentially contaminated water would be generated as a result of a 24-hour, 2-year storm.

## II-3 Explain how the existing sedimentation ponds will be treated before the construction of the landfill.

No treatment of the existing sedimentation ponds before construction of the landfill is planned. The sedimentation ponds will remain for continued use by Union Rock and Materials Corporation until landfill construction advances to the areas occupied by them. At that time, they will be excavated and removed.

## II-4 Submit analysis of water and soil in the proximity of the ponds.

The analyses are requested to ascertain the extent of any existing contamination caused by existing and abandoned ponds. An analysis of the potential contamination of the site was performed by SCS Engineers in 1989 and is included here as Attachment 4.

No water samples have been analyzed from the ponds vicinity. Borings drilled in the pond's vicinity did not encounter free standing water that could be sampled.

## II-5 Clarify the status of the dike of soil and cement that appears only in cross-sections 7 and 8, and discuss alternative locations.

Sections 7 and 8 originally incorporated a soil-cement region in the west bank of the Agua Fria River since it had been considered earlier as a potential erosion protection scheme. The soil-cement erosion protection

scheme is not being proposed; instead, a rip-rap erosion protection blanket would be placed on the river bank, as presented in the Application. Section 9 does not incorporate soil-cement and presents a worst-case scenario. Inspection of Section 9 indicates that the saturation front does not reach the proposed excavation (see Attachments 7 and 9).

## II-6 Submit a detailed plan for Quality Assurance/Quality Control (QA/QC) of the composite liner and geosynthetics during construction; reference methods for prevention/remediation of environmental stress cracking of the geomembrane.

Two documents are attached and provide the required information:

Quality Assurance Procedures for Earthwork Construction at Cholla Sanitary Landfill (Attachment 10)

Quality Assurance Procedures for Synthetic Containment System Construction, Cholla Sanitary Landfill (Attachment 11).

## II-7 Redesign the landfill - cells considering site-specific geologic and hydrogeologic hazards.

The Cholla Landfill Design described and contained in the application to the DEQ (DEQ Application) is conservative and appropriate for the geologic, hydrogeologic, hydrologic, and climatic conditions of the landfill site as detailed below.

## Site Conditions

### Hydrogeology and Geology

EMCON's hydrogeologic and geotechnical investigations found no geologic hazards that preclude developing the site as a landfill (DEQ Application, Appendix II B&C). The potential for the occurrence of fissures at the site was studied by EMCON and by Richard H. Raymond, an acknowledged specialist in the field of fissure formation. Mr. Raymond's independent study is documented in his report (DEQ Application, Appendix B). In his report, Mr. Raymond concludes as follows:

"The proposed Cholla Sanitary Landfill site is on a gravel terrace where the piping that commonly causes damage from fissures would be unlikely,



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even if a fissure were to form at the site. The site is over the broadly rounded top of the Luke salt body. No irregularities are known there that might cause a fissure to form. The Luke fissures are the closest existing fissures. Their location apparently is controlled by the shape of the western side of the salt body. They have been forming within a narrow band since 1959 and no evidence exists to indicate that they will migrate out of that band in the future. Water-level declines of as much as 15 feet per year have caused subsidence and earth fissures between the Luke salt body and the White Tank Mountains in the past, but that rate has been greatly reduced. Future activity of the Luke fissures should also diminish correspondingly. Water levels on the east side of the Luke salt body essentially have been stable since about 1964. Previous water-level declines did not produce earth fissures on the east side of the salt body, and no reason exists for expecting fissures to form there in the future. With the Central Arizona Project (CAP) and the new ground-water law, rapid declines in the ground-water level in the future are unlikely. Therefore, the formation of earth fissures under the proposed site of the Cholla Sanitary Landfill are also unlikely."

At the DEQ's request the proposed liner design has been analyzed to determine its performance if a fissure were to form beneath it. The analysis, performed by GeoServices, Inc. Consulting Engineers shows that the proposed composite liner would not fail if located over an eroded fissure with a horizontal separation of up to 1.44 feet. GeoServices analysis is presented in Attachment No. 14.

**Hydrology**

Unsaturated flow modeling by Maddock, et al. (1990) shows that water percolating through the soil from the 100-year flood in the adjacent Agua Fria River will not migrate far enough to penetrate waste cells (see Attachments 7 and 9).

Plans prepared by Mathews, Kessler and Associates for improvement of the existing broad soil berm that separates the Agua Fria River from the future landfill waste cells have been approved by the Army Corps of Engineers who has issued a Nationwide 404 permit (No. 26, for application No. 90-112-CL) for the improvements (see Attachment 3). The plans include installation of rip rap on the west bank of the Agua Fria River that will be adequately keyed into the river bed to prevent undercutting (scour) by 100-year flood waters. The analysis that accompanies the plans

demonstrates that the river bank improvements will prevent wash out of the landfill in the event of a 100-year flood.

**Climate**

Because the site is in an arid climate, evaporation significantly exceeds precipitation. A water balance analysis (DEQ Application, Appendix ID) indicates that little precipitation will infiltrate the refuse. This, in conjunction with the capacity of the soil and waste to absorb moisture, indicates a minimal potential for leachate generation at the site. Thus, the potential for discharge of leachate to ground water is low (even if the landfill were not lined).

The inclusion of a composite liner and leachate collection and removal system (LCRS), lower to an insignificant level the potential for leachate releases from the site.

In summary, the landfill environment presents no hydrogeologic, geologic or hydrologic conditions that would affect the integrity of the lined landfill as designed.



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This section compares single composite and double liner systems and describes the design and the components of the proposed Cholla Landfill composite liner system which was tailored to the site-specific conditions previously described.

## Comparison of Single Composite Liner and Double Liner

Composite liners (geomembrane in close contact with compacted low permeability soil) are very effective in limiting leachate migration. The geomembrane component has an equivalent hydraulic conductivity of $1 \times 10^{-12}$ cm/sec to $1 \times 10^{-14}$ cm/sec, or less. The low permeability soil component, serves to limit flow through defects or holes in the geomembrane. The geomembrane component also increases the efficiency of the overlying leachate collection and removal system (LCRS). LCRS collection efficiency can be defined as the ratio of the quantity of leachate entering the LCRS to the quantity of leachate collected in the LCRS sumps. EPA-sponsored studies for hazardous waste disposal facilities (U.S. EPA, 1987, 1988) have indicated that composite liners increase the collection efficiency of the overlying LCRS to nearly 100 percent, even if the geomembrane has one or two small holes per acre. Thus, any leachate that may be generated will drain to the LCRS sumps where it can be removed for proper treatment and disposal.

As shown on Figure IB-19, the range of anticipated flow rates through a composite liner (geomembrane and 3 feet of soil with a hydraulic conductivity of $1 \times 10^{-6}$ cm/sec) with a geomembrane defect (hole size 1 $cm^2$) and a liquid depth above the geomembrane of 0.1 foot is estimated to be 0.2 to 2 gallons per acre per day. This amount of leakage is extremely small and could only be reached on the assumption that there will be a constant depth or quantity of leachate above the liner and a hole in the liner (an unlikely condition).

It is instructive to compare this quantity of leakage with the quantity of leakage that would occur through a double liner system, with primary and secondary leachate collection; the upper liner is a flexible membrane liner and lower liner is compacted soil. We have assumed that the lower liner is

3-feet thick and has a hydraulic conductivity of $1 \times 10^{-6}$ cm/sec. The top liner is a geomembrane with one small hole per acre. Research documented by the EPA (April 1987 and May 1987), and in unpublished EPA documents, indicated that even with excellent Construction Quality Control (CQC) and Construction Quality Assurance (CQA) one or two small holes or defects per acre will go undetected in an installed geomembrane. The flow through one of these defects (noncomposite liner) can be computed using the orifice equation:

$$Q = CA (2gh)^{1/2}$$

Where

Q = flow rate

C = coefficient generally assumed to be 0.6

A = area of the hole

g = acceleration due to gravity

h = head on the geomembrane

Using the orifice equation and assuming a 1 $cm^2$ hole, unrestricted flow to and through the hole, and 0.1 foot of head above the top liner, the flow through the top liner hole would be about 1,060 gallons per day. This computed flow rate represents an upper bound, since the presence of granular LCRS systems above and below the top liner may significantly reduce the flow rate. A sand LCRS would reduce the flow rate through the hole, while a clean coarse gravel LCRS would have little effect on flow through a geomembrane hole.

Flow through a top liner defect would then enter the secondary LCRS between the liners. If we assume that the concentrated top liner leakage spreads out uniformly in the secondary LCRS system, the concentrated leak would be like a uniform top liner leak of 1,060 gallons per day per acre. For this top liner leakage to reach the LCRS sumps a head must build up in the LCRS system and on the bottom liner. Assuming that only a very small head builds up on the bottom liner such that the hydraulic gradient is 1, the flow through the bottom liner can be computed using Darcy's equation.

$$Q = k\ i\ A$$

Where



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Q = flow rate

k = coefficient of permeability (hydraulic conductivity)

i = hydraulic gradient

A = area (use 1 acre as a basis)

Using Darcy's equation, a hydraulic gradient of 1, an area of 1 acre, and a bottom liner hydraulic conductivity of $1 \times 10^{-6}$ cm/sec, flow through the bottom liner would be 925 gallons per day, or nearly the same as the top liner leakage. However, the top liner leak is a concentrated leak rather then a uniform one. The analysis of concentrated leaks in a double liner system is a three dimensional problem and extremely complex. The EPA (December 1987) suggested using a flow concentration factor (fc) for an approximate 1-dimensional solution to the three dimensional problem. It is the ratio of the bottom liner plan area wetted by a top liner leak to the total bottom liner plan area. The factor accounts for several parameters including (1) size and number of top liner leaks, (2) type of LCRS drainage medium (sand, gravel, geonets), (3) LCRS drainage medium capillary tensions, (4) LCRS system slope, (5) top liner leak location with respect to LCRS sump, (6) bottom liner type, and (7) bottom liner surface regularity. For example, more liquid would be collected in the LCRS sump from a top liner leak located close to the sump than one located at a large distance from the sump. The EPA has suggested values of fc between 0.05 and 0.25. Using an fc of 0.2, leakage through a bottom soil liner due to a concentrated top liner leak of 1,060 gallons per day would be about 200 gallons per acre per day. This value (200 gallons, per acre, per day) is two to three orders of magnitude greater than the leakage through the composite liner proposed for the Cholla Landfill calculated using the above stated conservative assumptions.

Similar computations can be made for liner options where both top and bottom liners are geomembranes. However, to evaluate the performance of these systems a probabilistic assessment would be needed to assess the likelihood of a bottom liner leak or defect being located beneath or in the LCRS drainage path of a top liner leak.

The advantages offered by a composite liner over other liner configurations in minimizing leakage are substantial. Apparently, the EPA has come to this same conclusion in proposing the single composite liner as the required liner system for non-hazardous solid waste landfills in its draft Subtitle D regulations (40 CFR 258.28(b), proposed August 30, 1988).

## Proposed Cholla Landfill Containment System

### Low Permeability Soil Component

The composite liner proposed for the Cholla Landfill consists of the following components (from top to bottom): (1) a 1-foot-thick operations layer, (2) a 1-foot gravel LCRS (hydraulic conductivity of 1 cm/sec), (3) a 60-mil-thick HDPE geomembrane, and (4) a 3-foot thickness of compacted low permeability soil with a hydraulic conductivity of $1 \times 10^{-6}$ cm/sec, or less. A non-woven geotextile filter/separator is provided between the operations layer and the LCRS. The LCRS is provided on the landfill bottom surface, but not on the side slopes. On the landfill's side slopes the operations layer is 2-foot thick.

The proposed composite lining system was selected for the Cholla Sanitary Landfill rather than the double liner systems because (1) there is about 180 feet vertical separation between the proposed landfill bottom and ground water, (2) the Phoenix area receives very little precipitation, minimizing the potential for leachate generation, (3) only nonhazardous solid wastes, (no liquid waste or non-dewatered sludge) will be accepted at the landfill, again minimizing the potential for leachate generation, and (4) the proposed landfill final cover system will greatly minimize or eliminate infiltration, keeping the refuse well below its field capacity so that leachate will not form after closure.

The composite liner's low permeability soil component will be constructed using the site's near surface silty sand and sandy silt with an admixture of about 5 percent by weight of bentonite. These materials when thoroughly mixed, water conditioned and compacted have permeabilities between $3.7 \times 10^{-8}$ cm/sec and $6 \times 10^{-9}$ cm/sec.

The materials for the low permeability soil will have 100 percent of the particles finer than the 3/4-inch sieve and at least 30 percent (before bentonite addition) by weight finer than the No. 200 U.S. Standard Sieve. After bentonite addition, the material will be water conditioned to between optimum moisture content and 6 percent above optimum moisture content and compacted to at least 90 percent of maximum dry density as determined by ASTM D1557. The material will be placed in approximately 8-inch-thick (maximum) loose lifts and compacted with tamping type or sheeps-foot type compactors. The placed material will be protected from drying and desiccating.



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### Geomembrane Component

The top component of the composite liner will be a 60 mil HDPE geomembrane. The geomembrane will be installed above the low permeability soil component and will serve as an effective barrier to leachate migration from the landfill and will increase the efficiency of the LCRS to nearly 100 percent in directing leachate to the sumps.

### LCRS and Operations Layer

A blanket type LCRS will be constructed on top of the geomembrane on the landfill bottom. The landfill base of excavation will be graded to drain towards a module corner where a sump will be located. Each layer of the composite liner system will follow this excavation grade.

The LCRS will consist of

- a 1-foot drainage layer having a 2 percent slope and a hydraulic conductivity of 1 cm/sec.
- 6-inch diameter leachate collection pipes at a 1 percent or greater slope
- leachate collection sumps and extraction pumps

The LCRS was designed to prevent buildup of leachate in excess of 0.1 foot over the geomembrane.

A leak detection system beneath the leachate collection sump will provide added protection by allowing BFI to identify leaks in the system. The leak detection system provides the capability to remove any fluids detected by collecting any leakage from the LCRS sump, the only location where the depth of leachate could potentially exceed 1-foot.

A 1-foot-thick operations layer will overlie the LCRS to protect it from damage when the first lift of refuse is placed. A nonwoven geotextile filter/separator will be provided between the LCRS gravel and the operations layer. On the landfill's excavation side slopes, the operations layer (2-foot-thick) will be placed directly above the geomembrane liner.

Operations layer material will consist of soils obtained from landfill excavations with 100 percent of the particles finer than the 3/4 inch sieve.

## Conclusion

The composite liner and LCRS represent a sate-of-practice solution for leachate containment in the Cholla landfill site environment.



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## SURFACE WATER DIVERSION

The on-site and off-site drainage system is designed for the 100-year, 24-hour storm. Overland flow will drain to sedimentation/detention basins separated from the solid waste disposal area by earthen berms. The basins will discharge to the Agua Fria River.

The proposed river bank improvements (see Attachment 3) wil protect the site from the 100-year flood in the Agua Fria River.

## REFERENCES


Arizona Department of Environmental Quality. (1987). *BADCT Guidance Document for the Landfill Category.* Draft

Environmental Protection Agency. (April 1987). *Background Document on Bottom Liner Performance in Double-Lined Landfills and Surface Impoundments.* EPA/520-SW-87-013.

Environmental Protection Agency. (May 1987). *Background Document on Proposed Liner and Leak Detection Rule.* EPA/530-SW-015.

Environmental Protection Agency. (December 1987). *Background Document on Minimum Technical Requirements for Compacted-Soil Component of Composite Bottom Liners.* Draft Report.

Environmental Protection Agency. (January 1988). *Draft Background Document on the Final Double Liner and Leachate Collection System Rule.*





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**II-8 Submit, for our approval, a thorough protocol for experimental testing and characterization of the geomembrane material (plane test and fissure-box test).**

The validity of the liner testing performed to simulate the effects of an earth fissure occurring under the site was questioned. From the discussions held on March 13 with the ADEQ staff, it was clear that the staff was concerned about cavities developing under the liner as a result of fissure erosion due to surface water flow.

Instead of conducting additional physical testing of HDPE liner materials under the various conditions stated in the request for clarification, we discussed the following alternative which meets ADEQ objectives and provides independent verification of the physical tests already performed.

The ability of a composite soil/geosynthetic liner system to support loads due to the removal of underlying support can be evaluated using recently developed analytical models. We proposed to use this technique to calculate the largest void size that the proposed liner system may bridge, as well as the maximum load the liner system can carry over such a void. This analysis, described in GeoServices, Inc. letter dated May 8, 1990 (see Attachment 1), was conducted under the direction of Dr. R. Bonaparte to supplement and validate the testing already conducted.

Results of the analysis are included in the report by GeoServices, Inc. titled Analysis of the Effect of an Earth Fissure on Lining System Integrity, Cholla Sanitary Landfill (see Attachment 14).

**II-9 Perform the approved membrane characterization tests. Submit documentation and results.**

As discussed above no additional membrane testing was performed. The results of the analytical work performed instead are included in Attachment 14.

**II-10 Submit Plans for monitoring geologic and hydrogeologic hazards and contingency plans for such occurrences. Plans should pertain to the entire facility life: construction, active phase, and closure. Plans should include remediation measures.**

A Fissure Monitoring Plan is included as Attachment 12.

As discussed with Ms. Haney and Messrs. Lerman and Abbott during the April 27 meeting, vadose zone monitoring may not be necessary at the site if the results of the unsaturated ground-water flow model showed that no subsurface flow would reach the landfill, even when the horizontal permeability was increased by a factor of 2 and the initial moisture content of the naturally dry soils was increased to a value just short of saturation. Since the model run under these conditions showed that subsurface flow does not reach the landfill (see Attachment 8), no vadose zone monitoring is being proposed.



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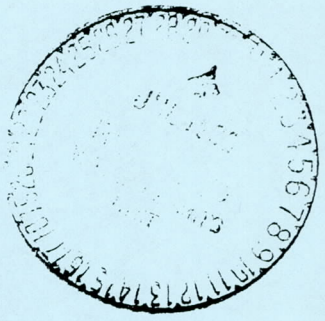

Attachment 1

LETTERS ADDRESSING RETIREMENT OF GRANDFATHERED IRRIGATION RIGHTS

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WRITER'S DIRECT LINE
257-5428

February 28, 1989

HAND DELIVERED

Mr. Dennis Kimberlin
Arizona Department of Water Resources
Phoenix Active Management Area
15 South Fifteenth Avenue
Phoenix, Arizona 85007

Re: Extinguishment of Irrigation Grandfathered Right No. 58-105297

Gentlemen:

Enclosed on behalf of Union Rock and Materials Corporation ("Union") is the original Irrigation Grandfathered Right Certificate No. 58-105297. Union has not exercised this right since 1977, and Union does not foresee that the property will ever be irrigated again. Union is surrendering the enclosed certificate for permanent extinguishment. Union disclaims any further entitlement to irrigate with groundwater any portion of the irrigation acres described in the certificate. Accordingly, Union also asserts that it has no further obligation to prepare annual reports respecting the right.

Also enclosed for your convenience is a form letter to Union, for execution on behalf of the Department, confirming that the Irrigation Grandfathered Right has been extinguished. Your prompt attention in confirming the extinguishment would be greatly appreciated.

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Please let me know if you have any questions.

Sincerely

Philip H. Darrow
For the Firm

PHD:jes

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RECEIVED PHD

MAR 06 1989

March 3, 1989

ARIZONA
DEPARTMENT
OF WATER
RESOURCES

Rose Mofford, Governor
N. W. Plummer
Director

Phoenix Active Management Area
15 South 15th Avenue
Phoenix, Arizona 85007

Mr. Robert K. Gjere
President/General Manager
Union Rock and Materials Corporation
2800 S. Central Avenue
Phoenix, Arizona 85040

RE: Irrigation Grandfathered Right Number 58-105297.0000

Dear Mr. Gjere;

We have received the original Certificate of Irrigation Grandfathered Right Number 58-105297.0000, with a written request on behalf of Union Rock and Materials Corporation to inactivate the Irrigation Grandfathered Right described in the certificate. This will confirm that the right has been Inactive-Withdrawn and is no longer appurtenant to the acreage described in the certificate. Since the effective date of the inactivation of this right is February 28, 1989, you are required to file the 1988 Annual Water Withdrawal and Use Report. Annual Reports for 1989 and future years will no longer be necessary.

If you have questions regarding this matter, please contact me at 542-1512.

Sincerely,

Dennis Kimberlin
Chief, Field Services/Operations
Phoenix Active Management Area

DK/jlc

cc: Phillip H. Darrow,
Fennemore Craig

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NICOLAS T. NIKAS
JEFFREY A. GRABOWSKI
MARTIN P. SLARE
REN R. MAYMURGY
MICHELLE MILLS
ROBERT D. ANDERSON
JOHN J. SALITIS, JR.
JEANINE A. MOSERLY
DAVID B. SALTZMAN
MARK E. HAMELBURG

KENT A. BLAKE
ARTHUR B. SHAENNEION
WILLIAM T. BOUTELL, JR.
ROBERT P. ROBINSON
JOHN D. EVERROAD
DONALD R. GILBERT
JEROME B. SHULTZ
ROGER T. HARGROVE
RUTH V. McGREGOR
JOHN G. RYAN
TIMOTHY BERG
MICHAEL A. GROHIN
CHARLES M. KING
ROBERT B. RIDE
J. GREGORY OSBORNE
PAUL J. MOONEY
RITA A. EISENFELD
MARGARET R. GALLOGLY
SCOTT M. PINGAL
JOHN DAVID TORRES
GREGG HANES
CHRISTOPHER B. HAMILL
BRADLEY P. FORST
JAMES J. TRIMBLE
PHILIP H. DARROW
JIM L. WRIGHT
BRENDA R. SHUREN
STEVEN P. KRAMER
CHRISTOPHER P. STARING
GARY H. ARNDT
ELLEN KEEGER
MARC T. STEARMAN
SHARON GOLDEN GOMEN
LUIS J. DIAZ
MARK H. GRAMI
DEBRA L. RUNDECK
SCOTT M. THOMAS

PHOENIX OFFICE
SUITE 2200
TWO NORTH CENTRAL AVENUE
PHOENIX, ARIZONA 85004-2393
(602) 257-8700
TELECOPIER: 602-257-8327
TWX: 910-950-4609

SCOTTSDALE OFFICE
SUITE A201
8991 EAST CAMELBACK ROAD
SCOTTSDALE, ARIZONA 85251-2466
(602) 257-8400
TELECOPIER: 602-945-4832

WRITER'S DIRECT LINE

257-5428

March 7, 1989

Mr. Dennis Kimberlin
Arizona Department of Water Resources
Phoenix Active Management Area
15 South Fifteenth Avenue
Phoenix, Arizona 85007

Re: Union Rock and Materials -- Former Irrigation Grandfathered Right No. 58-105297

Dear Dennis:

In reference to your letter to Mr. Robert Gjere of March 3, 1989, this will confirm our conversation this afternoon in which you explained that the status "Inactive-Withdrawn" means that the Irrigation Grandfathered Right discussed in the letter no longer exists and can never be reactivated. This is consistent with the intent of the owner in surrendering the certificate.

Thank you very much for clarifying this matter.

Sincerely,

Philip H. Darrow
For the Firm

PHD:jes
cc: Mr. Robert Gjere

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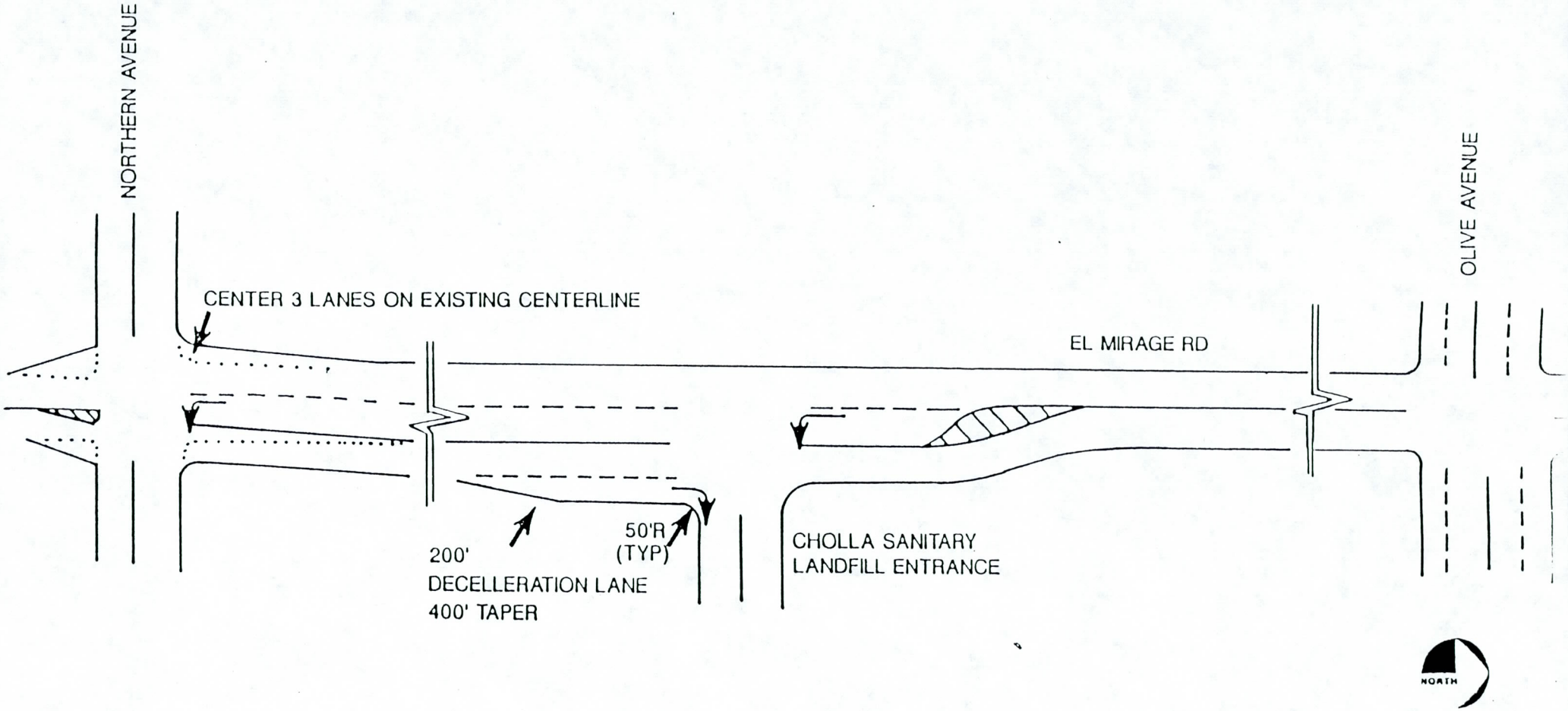
Attachment 2

PROPOSED ROADWAY IMPROVEMENTS,

EL MIRAGE ROAD




PROPOSED ROADWAY IMPROVEMENTS
EL MIRAGE ROAD

HNTB

3/16/1989

**ATTACHMENT 2**
372-15.01

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Attachment 3

DOCUMENTS REGARDING FLOODPLAIN MANAGEMENT AND BANK STABILIZATION

CITY HALL
[illegible] N. Palm Street
El Mirage, AZ 85335
(602) 972-8116

ROSARIO YALENZUELA
Mayor

JOHN GARZA
Vice Mayor

CAROLINA HERNANDEZ
MARY IZAGUIRRE
GLENN KELTGEN
ARTHUR MORENO
ROBERT ROBLES
City Councilors

February 26, 1990

Arizona Department of Environmental Quality
Attn: Mr. Barry Abbott
Solid Waste Unit
2005 North Central Avenue
Phoenix, Arizona 85004

Re: Floodplain Management

Gentlemen:

Enclosed is a copy of Resolution No. R89-09-039 that was adopted by the El Mirage City Council on September 14, 1989. Pursuant to that resolution and applicable state statutes, the City of El Mirage has assumed floodplain management authority for those portions of the Agua Fria within the corporate limits of the City of El Mirage. A copy of this resolution was sent to the Federal Emergency Management Agency, the Arizona Department of Water Resources and the Flood Control District of Maricopa County.

The Flood Control District of Maricopa County has transferred its files to El Mirage. On October 26, 1989, FEMA requested that El Mirage enact some minor revisions to El Mirage's floodplain management ordinance; those changes have been made. The City of El Mirage, with the assistance of Wood & Associates and the Arizona Department of Water Resources, is currently reviewing submittals relating to the Cholla Landfill. We will advise you of our action with respect to these submittals when completed.

Sincerely,

Scott Lind
City Manager

encl.

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CITY HALL
14405 N. Palm Street
El Mirage AZ 85335
(602) 972-8116

ROSARIO VALENZUELA
Mayor

JOHN GARZA
Vice Mayor

CAROLINA HERNANDEZ
MARY IZAGUIRRE
GLENN KELTEEN
ARTHUR MORENO
ROBERT ROBLES
City Councilors

# RESOLUTION NO. R89-09-039

A RESOLUTION OF THE CITY OF EL MIRAGE, MARICOPA COUNTY, ARIZONA, ASSUMING THE RESPONSIBILITY FOR THE MANAGEMENT OF FLOODPLAIN ACTIVITIES TO REDUCE FUTURE FLOOD LOSSES PURSUANT TO THE NATIONAL FLOOD INSURANCE ACT OF 1968 AND TITLE 48, CHAPTER 21, ARTICLE 1 OF THE ARIZONA REVISED STATUTES AND AMENDMENTS THERETO; REPEALING CONFLICTING RESOLUTIONS; AND DECLARING AND EMERGENCY.

WHEREAS, certain areas of the City of El Mirage are subject to periodic flooding from streams and rivers causing serious damages to properties within these areas;

WHEREAS, it is the intent of this Council to require the recognition and evaluation of flood hazards in all official actions relating to land use in the floodplain areas having special flood hazards; and

WHEREAS, relief is available in the form of flood insurance as authorized by the National Flood Insurance Act of 1968; and

WHEREAS, this body has the legal authority to adopt and enforce land use and control measures to reduce future flood losses pursuant to 48-3610, Arizona Revised Statutes and amendments and supplements thereto;

NOW, THEREFORE, BE IT RESOLVED BY THE COUNCIL OF THE CITY OF EL MIRAGE AS FOLLOWS:

SECTION 1. That the City Council of the City of El Mirage assures the Federal Insurance Administration that it will enact as necessary, and maintain in force for those areas having flood hazards, adequate land use and control measures with effective enforcement provisions consistent with the criteria set forth in Part 60 of the National Flood Insurance Program Regulations; and

SECTION 2. That the duly appointed Floodplain Administrator or his authorized representative, is hereby vested with the responsibility, authority and means to: (a) delineate or assist the Federal Insurance Administrator, at his request, in delineating the limits of areas having special flood hazards on available local maps of sufficient scale to identify the location of building sites; (b) provide such information as the Federal Insurance Administrator may request concerning present uses and occupancy of the floodplain; (c)

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cooperate with Federal, State and local agencies and private firms which undertake to study, survey, map and identify floodplain areas, and cooperate with neighboring communities with respect to management of adjoining floodplain areas in order to prevent aggravation of existing hazards; and (d) submit an annual report to the Federal Insurance Administrator on the progress made during the past year within the community in the development and implementation of floodplain management measures.

SECTION 3. That the duly appointed Floodplain Administrator, or his authorized representative, is hereby appointed to maintain for public inspection and to furnish upon request a record of elections (in relation to mean sea level) of the lowest floor (including basement) of all new or substantially improved structures located in the special flood hazard areas. If the lowest floor is below grade on one or more sides, the elevation of the floor immediately above must also be recorded.

SECTION 4. That the City Council agrees to take such other official action as may be reasonably necessary to carry out the objectives of the program.

SECTION 5. All resolutions of parts of resolutions in conflict herewith are hereby repealed.

SECTION 6. WHEREAS the immediate operation of the provisions of this Resolution is necessary for the preservation of public peace, health and safety of the City, an emergency is hereby declared to exist, and this Resolution shall be in full force and effect from the after its passage, adoption and approval by the Mayor and Council of the City, and it is hereby exempt from the referendum provisions of the Constitution and laws of the State of Arizona.

PASSED, ADOPTED AND APPROVED by the Mayor and Council of the City of El Mirage, Maricopa County, Arizona, this 14th day of September, 1989.

Rosario Valenzuela
ROSARIO VALENZUELA, Mayor

ATTEST:

Rosalinda Herrera
ROSALINDA HERRERA, Clerk

APPROVED AS TO FORM:

PAUL FAITH, City Attorney

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# FLOOD CONTROL DISTRICT
of
Maricopa County

3335 West Durango Street • Phoenix, Arizona 85009
Telephone (602) 262-1501

D. E. Sagramoso, P.E., Chief Engineer and General Manager



BOARD of DIRECTORS
James D. Bruner
Carole Carpenter
Tom Freestone
Fred Koory, Jr.
Ed Pastor


September 19, 1989

The Honorable Rosario Valenzuela
Mayor, City of El Mirage
City Hall
14405 North Palm Street
El Mirage, AZ 85335

Subject: Floodplain Mangement

Dear Mayor Valenzuela:

I have received your letter datad September 15, 1989 to the Federal Emergengy Management Agency (FEMA) concerning your assumption of the responsibility for floodplain management within the City of El Mirage.

Mr. Ron Nevitt, Floodplain Representative of my staff, will be contacting City staff with respect to transitioning files and permit applications in progress.

If you have any questions, please call.

Sincerely,

D. E. Sagramoso, P. E.

Copy to: Jim Morris, Arizona Department of Water Resources
Terri Miller, Arizona Department of Water Resources
Johnny Taylor, FEMA Region IX


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DEPARTMENT OF THE ARMY
LOS ANGELES DISTRICT, CORPS OF ENGINEERS
P.O. BOX 2711
LOS ANGELES, CALIFORNIA 90053-2325

REPLY TO
ATTENTION OF

MAR 30 1990

Office of the Chief
Regulatory Branch

Browning-Ferris Industries of Arizona, Inc.
ATTN: Brett Frazier
1580 E. Elwood
Phoenix, Arizona 85040

Gentlemen:

This is in reply to your application (No. 90-112-CL) dated March 2, 1990 for a Department of the Army Permit to discharge dredged or fill material for the construction of 4200 feet of rock rip-rapped bank stabilization in the Agua Fria River (the west bank between Olive Avenue to Northern Avenue) at El Mirage, Maricopa County, Arizona.

Regulations for our permit program, published in the Federal Register, include Part 330 - Nationwide Permits (see the enclosure). The Corps of Engineers has determined that your proposed activity complies with the terms of the nationwide permit at Part 330.5(a)(26) for discharges of dredged or fill material into waters of the United States, including wetlands, that are located above the headwaters or are isolated waters and which would cause the loss or substantial adverse modification of one to 10 acres of such waters. Furthermore, you must comply with the attached special conditions, the nationwide permit conditions described in Part 330.5(b), and the management practices described in Part 330.6.

A nationwide permit does not grant any property rights or exclusive privileges. Also, it does not authorize any injury to the property or rights of others or authorize interference with any existing or proposed Federal project. Furthermore, it does not obviate the need to obtain other Federal, state, or local authorizations required by law.

This verification will be valid until the nationwide permit is modified, reissued, or revoked. All the nationwide permits are scheduled to be modified, reissued, or revoked prior to 13 January 1992. It is incumbent upon you to remain informed of changes to the nationwide permits. We will issue a public notice announcing the changes when they occur. Furthermore, if you commence or are under contract to commence this activity before the date the nationwide permit is modified or revoked, you will have twelve months from the date of the modification or revocation to complete the activity under the present terms and conditions of this nationwide permit.

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If you have any questions please contact Cindy Lester of my staff at (602) 640-5385.

Sincerely,

David J. Castanon

David J. Castanon
Chief, Northern Section

Enclosures

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SPECIAL CONDITIONS APPLICABLE TO NATIONWIDE PERMIT NO. 26 FOR
THE GENERAL PERMITTEE'S APPLICATION NUMBER 90-112-CL

1. The applicant shall obtain the Floodplain Development Permit from the City of El Mirage, for both the streambank stabilization and the landfill prior to the construction of the streambank protection.

2. The applicant shall obtain the "Aquifer Protection Permit" and the "Approval to Construct and Operate Landfills" from the Arizona Department of Environmental Quality prior to the construction of the streambank protection.

3. The applicant shall revegetate a 50 foot wide buffer zone at the top of the bank stabilization with endemic vegetation at the conclusion of the streambank stabilization project.

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## PART 330—NATIONWIDE PERMITS

Sec.
330.1 General.
330.2 Definitions.
330.3 Activities occuring before certain dates.
330.4 Public notice.
330.5 Nationwide permits.
330.6 Management practices.
330.7 Notification procedures.
330.8 Discretionary Authority.
330.9 State water quality certification.
330.10 Coastal Zone Management consistency determination.
330.11 Nationwide permit verification.
330.12 Expiration of nationwide permits.

**Authority: 33 U.S.C. 401 et seq.; 33 U.S.C. 1344; 33 U.S.C. 1413.**

### § 330.1 General.

The purpose of this regulation is to describe the Department of the Army's (DA) nationwide permit program and to list all current nationwide permits which have been issued by publication herein. A nationwide permit is a form of general permit which may authorize activities throughout the nation. (Another type of general permit is a "regional permit" and is issued by division or district engineers on a regional basis in accordance with 33 CFR Part 325). Copies of regional conditions and modifications, if any, to the nationwide permits can be obtained from the appropriate district engineer. Nationwide permits are designed to allow certain activities to occur with little, if any, delay or paperwork. Nationwide permits are valid only if the conditions applicable to the nationwide permits are met. Failure to comply with a condition does not necessarily mean the activity cannot be authorized but rather that the activity can only be authorized by an individual or regional permit. Several of the nationwide permits require notification to the district engineer prior to commencement of the authorized activity. The procedures for this notification are located at § 330.7 of this Part. Nationwide permits can be issued to satisfy the requirements of section 10 of the Rivers and Harbors Act of 1899, section 404 of the Clean Water Act, and/or section 103 of the Marine Protection, Research and Sanctuaries Act. The applicable authority is indicated at the end of each nationwide permit.

### § 330.2 Definitions.

(a) The definitions of 33 CFR Parts 321–329 are applicable to the terms used in this Part.

(b) The term "headwaters" means the point on a non-tidal stream above which the average annual flow is less than five cubic feet per second. The district engineer may estimate this point from available data by using the mean annual area precipitation, area drainage basin maps, and the average runoff coefficient, or by similar means. For streams that are dry for long periods of the year, district engineers may establish the "headwaters" as that point on the stream where a flow of five cubic feet per second is equaled or exceeded 50 percent of the time.

(c) Discretionary authority means the authority delegated to division engineers in § 330.8 of this part to override provisions of nationwide permits, to add regional conditions, or to require individual permit application.

### § 330.3 Activities occurring before certain dates.

The following activities were permitted by nationwide permits issued on July 19, 1977, and unless modified do not require further permitting:

(a) Discharges of dredged or fill material into waters of the United States outside the limits of navigable waters of the United States that occurred before the phase-in dates which began July 25, 1975, and extended section 404 jurisdiction to all waters of the United

States. (These phase-in dates are: After July 25, 1975, discharges into navigable waters of the United States and adjacent wetlands; after September 1, 1976, discharges into navigable waters of the United States and their primary tributaries, including adjacent wetlands, and into natural lakes, greater than 5 acres in surface area; and after July 1, 1977, discharges into all waters of the United States.) (Section 404)

(b) Structures or work completed before December 18, 1968, or in waterbodies over which the district engineer had not asserted jurisdiction at the time the activity occurred provided, in both instances, there is no interference with navigation. (Section 10)

**§ 330.4 Public notice.**

(a) *Chief of Engineers.* Upon proposed issuance of new nationwide permits, modification to, or reissuance of, existing nationwide permits, the Chief of Engineers will publish a notice in the **Federal Register** seeking public comments and including the opportunity for a public hearing. This notice will state the availability of information at the Office of the Chief of Engineers and at all district offices which reveals the Corps' provisional determination that the proposed activities comply with the requirements for issuance under general permit authority. The Chief of Engineers will prepare this information which will be supplemented, if appropriate, by division engineers.

(b) *District engineers.* Concurrent with publication in the **Federal Register** of proposed, new, or reissued nationwide permits by the Chief of Engineers, district engineers will so notify the known interested public by an appropriate notice. The notice will include regional conditions, if any, developed by the division engineer.

**§ 330.5 Nationwide permits.**

(a) *Authorized activities.* The following activities are hereby permitted provided they meet the conditions listed in paragraph (b) of this section and, where required, comply with the notification procedures, of § 330.7.

(1) The placement of aids to navigation and regulatory markers which are approved by and installed in accordance with the requirements of the U.S. Coast Guard (33 CFR Part 66, Subchapter C). (Section 10)

(2) Structures constructed in artificial canals within principally residential developments where the connection of the canal to a navigable water of the United States has been previously authorized (see 33 CFR Part 322.5(g)). (Section 10)

(3) The repair, rehabilitation, or replacement of any previously authorized, currently serviceable, structure or fill, or of any currently serviceable structure or fill constructed prior to the requirement for authorization, provided such repair, rehabilitation, or replacement does not result in a deviation from the plans of the original structure or fill, and further provided that the structure or fill has not been put to uses differing from uses specified for it in any permit authorizing its original construction. Minor deviations due to changes in materials or construction techniques and which are necessary to make repair, rehabilitation, or replacement are permitted. Maintenance dredging and beach restoration are not authorized by this nationwide permit. (Section 10 and 404)

(4) Fish and wildlife harvesting devices and activities such as pound nets, crab traps, eel pots, lobster traps, duck blinds, and clam and oyster digging. (Section 10)

(5) Staff gages, tide gages, water recording devices, water quality testing and improvement devices, and similar scientific structures. (Section 10)

(6) Survey activities including core sampling, seismic exploratory operations, and plugging of seismic shot holes and other exploratory-type bore holes. Drilling of exploration-type bore holes for oil and gas exploration is not authorized by this nationwide permit; the plugging of such holes is authorized. (Sections 10 and 404).

(7) Outfall structures and associated intake structures where the effluent from that outfall has been permitted under the National Pollutant Discharge Elimination System program (Section 402 of the Clean Water Act) (see 40 CFR Part 122) provided that the district or division engineer makes a determination that the individual and cumulative adverse environmental effects of the structure itself are minimal in accordance with § 330.7 (c)(2) and (d). Intake structures per se are not included—only those directly associated with an outfall structure are covered by this nationwide permit. This permit includes minor excavation, filling and other work associated with installation of the intake and outfall structures. (Sections 10 and 404)

(8) Structures for the exploration, production, and transportation of oil, gas, and minerals on the outer continental shelf within areas leased for such purposes by the Department of Interior, Mineral Management Service, provided those structures are not placed within the limits of any designated shipping safety fairway or traffic separation scheme (where such limits have not been designated or where changes are anticipated, district engineers will consider recommending the discretionary authority provided by 330.8 of this Part, and further subject to the provisions of the fairway regulations in 33 CFR 322.5(1) (Section 10).

(9) Structures placed within anchorage or fleeting areas to facilitate moorage of vessels where such areas have been established for that purpose by the U.S. Coast Guard. (Section 10)

(10) Non-commercial, single-boat, mooring buoys. (Section 10)

(11) Temporary buoys and markers placed for recreational use such as water skiing and boat racing provided that the buoy or marker is removed within 30 days after its use has been discontinued. At Corps of Engineers reservoirs, the reservoir manager must approve each buoy or marker individually. (Section 10)

(12) Discharge of material for backfill or bedding for utility lines, including outfall and intake structures, provided there is no change in preconstruction bottom contours (excess material must be removed to an upland disposal area). A "utility line" is defined as any pipe or pipeline for the transportation of any gaseous, liquid, liquifiable, or slurry substance, for any purpose, and any cable, line, or wire for the transmission for any purpose of electrical energy, telephone and telegraph messages, and radio and television communication. (The utility line and outfall and intake structures will require a Section 10 permit if in navigable waters of the United States. See 33 CFR Part 322. See also paragraph (a)(7) of this section). (Section 404)

(13) Bank stabilization activities provided:

(i) The bank stabilization activity is less than 500 feet in length;

(ii) The activity is necessary for erosion prevention;

(iii) The activity is limited to less than an average of one cubic yard per running foot placed along the bank within waters of the United States;

(iv) No material is placed in excess of the minimum needed for erosion protection;

(v) No material is placed in any wetland area;

(vi) No material is placed in any location or in any manner so as to impair surface water flow into or out of any wetland area;

(vii) Only clean material free of waste metal products, organic materials, unsightly debris, etc. is used; and

(viii) The activity is a single and complete project. (Sections 10 and 404)

(14) Minor road crossing fills including all attendant features, both temporary and permanent, that are part of a single and complete project for crossing of a non-tidal waterbody, provided that the crossing is culverted, bridged or otherwise designed to prevent the restriction of, and to withstand, expected high flows and provided further that discharges into any wetlands adjacent to the waterbody do not extend beyond 100 feet on either side of the ordinary high water mark of that waterbody. A "minor road crossing fill" is defined as a crossing that involves the discharge of less than 200 cubic yards of fill material below the plane of ordinary high water. The crossing may require a permit from the US Coast Guard if located in navigable waters of the United States. Some road fills may be eligible for an exemption from the need for a Section 404 permit altogether (see 33 CFR 323.4). District engineers are authorized, where local circumstances indicate the need, to define the term "expected high flows" for the purpose of establishing applicability of this nationwide permit. (Sections 10 and 404)

(15) Discharges of dredged or fill material incidental to the construction of bridges across navigable waters of the United States, including cofferdams, abutments, foundation seals, piers, and temporary construction and access fills provided such discharge has been authorized by the US Coast Guard as part of the bridge permit. Causeways and approach fills are not included in this nationwide permit and will require an individual or regional Section 404 permit. (Section 404)

(16) Return water from an upland, contained dredged material disposal area (see 33 CFR 323.2(d)) provided the state has issued a site specific or generic certification under section 401 of the Clean Water Act (see also 33 CFR 325.2(b)(1)). The dredging itself requires a Section 10 permit if located in navigable waters of the United States. The return water or runoff from a contained disposal area is administratively defined as a discharge of dredged material by 33 CFR 323.2(d) even though the disposal itself occurs on the upland and thus does not require a section 404 permit. This nationwide permit satisfies the technical requirement for a section 404 permit for the return water where the quality of the return water is controlled by the state through the section 401 certification procedures. (Section 404)

(17) Fills associated with small hydropower projects at existing reservoirs where the project which includes the fill is licensed by the Federal Energy Regulatory Commission (FERC) under the Federal Power Act of 1920, as amended; has a total generating capacity of not more than 1500 kw (2,000 horsepower); qualifies for the short-form licensing procedures of the FERC (see 18 CFR 4.61); and the district or division engineer makes a determination that the individual and cumulative adverse effects on the environment are minimal in accordance with § 330.7 (c)(2) and (d). (Section 404)

(18) Discharges of dredged or fill material into all waters of the United States other than wetlands that do not exceed ten cubic yards as part of a single and complete project provided the material is not placed for the purpose of stream diversion. (Sections 10 and 404)

(19) Dredging of no more than ten cubic yards from navigable waters of the United States as part of a single and complete project. This permit does not authorize the connection of canals or other artificial waterways to navigable waters of the United States (see Section 33 CFR 322.5(g)). (Section 10)

(20) Structures, work, and discharges for the containment and cleanup of oil and hazardous substances which are subject to the National Oil and Hazardous Substances Pollution Contingency Plan, (40 CFR Part 300), provided the Regional Response Team which is activated under the Plan concurs with the proposed containment and cleanup action. (Sections 10 and 404)

(21) Structures, work, discharges associated with surface coal mining activities provided they were authorized by the Department of the Interior, Office of Surface Mining, or by states with approved programs under Title V of the Surface Mining Control and Reclamation Act of 1977; the appropriate district engineer is given the opportunity to review the Title V permit application and all relevant Office of Surface Mining or state (as the case may be) documentation prior to any decision on that application; and the district or division engineer makes a determination that the individual and cumulative adverse effects on the environment from such structures, work, or discharges are minimal in accordance with §§ 330.7 (c) (2) and (3) and (d). (Sections 10 and 404)

(22) Minor work, fills, or temporary structures required for the removal of wrecked, abandoned, or disabled vessels, or the removal of man-made obstructions to navigation. This permit does not authorize maintenance dredging, shoal removal, or river bank snagging. (Sections 10 and 404)

(23) Activities, work, and discharges undertaken, assisted, authorized, regulated, funded, or financed, in whole or in part, by another federal agency or department where that agency or department has determined, pursuant to the CEQ Regulation for Implementing the Procedural Provisions of the National Environmental Policy Act (40 CFR Part 1500 et seq.), that the activity, work, or discharge is categorically excluded from environmental documentation because it is included within a category of actions which neither individually nor cumulatively have a significant effect on the human environment, and the Office of the Chief of Engineers (ATTN: DAEN–CWO–N) has been furnished notice of the agency's or department's application for the categorical exclusion and concurs with that determination. Prior to approval for purposes of this nationwide permit of any agency's categorical exlcusions, the Chief of Engineers will solicit comments through publication in the **Federal Register**. (Sections 10 and 404)

(24) Any activity permitted by a state administering its own Section 404 permit program for the discharge of dredged or fill material authorized at 33 U.S.C. 1344(g)–(l) is permitted pursuant to section 10 of the Rivers and Harbors Act of 1899. Those activities which do not involve a section 404 state permit are not included in this nationwide permit but many will be exempted by section 154 of Pub. L. 94–587. (See 33 CFR 322.3(a)(2)). (Section 10)

(25) Discharge of concrete into tightly sealed forms or cells where the concrete is used as a structural member which would not otherwise be subject to Clean Water Act jurisdiction. (Section 404)

(26) Discharges of dredged or fill material into the waters listed in paragraphs (a)(26) (i) and (ii) of this section except those which cause the loss or substantial adverse modification of 10 acres or more of such waters of the United States, including wetlands. For discharges which cause the loss or substantial adverse modification of 1 to 10 acres of such waters, including wetlands, notification to the district engineer is required in accordance with section 330.7 of this section. (Section 404).

(i) Non-tidal rivers, streams, and their lakes and impoundments, including adjacent wetlands, that are located above the headwaters.

(ii) Other non-tidal waters of the United States, including adjacent wetlands, that are not part of a surface tributary system to interstate waters or

navigable waters of the United States (i.e., isolated waters).

(b) *Conditions.* The following special conditions must be followed in order for the nationwide permits identified in paragraph (a) of this section to be valid:

(1) That any discharge of dredged or fill material will not occur in the proximity of a public water supply intake.

(2) That any discharge of dredged or fill material will not occur in areas of concentrated shellfish production unless the discharge is directly related to a shellfish harvesting activity authorized by paragraph (a)(4) of this section.

(3) That the activity will not jeopardize a threatened or endangered species as identified under the Endangered Species Act (ESA), or destroy or adversely modify the critical habitat of such species. In the case of federal agencies, it is the agencies' responsibility to comply with the requirements of the ESA. If the activity may adversely affect any listed species or critical habitat, the district engineer must initiate Section 7 consultation in accordance with the ESA. In such cases, the district engineer may:

(i) Initiate section 7 consultation and then, upon completion, authorize the activity under the nationwide permit by adding, if appropriate, activity specific conditions, or

(ii) Prior to or concurrent with section 7 consultation he may recommend discretionary authority (See section 330.8) or use modification, suspension, or revocation procedures (See 33 CFR 325.7).

(4) That the activity shall not significantly disrupt the movement of those species of aquatic life indigenous to the waterbody (unless the primary purpose of the fill is to impound water);

(5) That any discharge of dredged or fill material shall consist of suitable material free from toxic pollutants (see section 307 of the Clean Water Act) in toxic amounts;

(6) That any structure or fill authorized shall be properly maintained.

(7) That the activity will not occur in a component of the National Wild and Scenic River System; nor in a river officially designated by Congress as a "study river" for possible inclusion in the system, while the river is in an official study status;

(8) That the activity shall not cause an unacceptable interference with navigation;

(9) That, if the activity may adversely affect historic properties which the National Park Service has listed on, or determined eligible for listing on, the National Register of Historic Places, the permittee will notify the district engineer. If the district engineer determines that such historic properties may be adversely affected, he will provide the Advisory Council on Historic Preservation an opportunity to comment on the effects on such historic properties or he will consider modification, suspension, or revocation in accordance with 33 CFR 325.7. Furthermore, that, if the permittee before or during prosecution of the work authorized, encounters a historic property that has not been listed or determined eligible for listing on the National Register, but which may be eligible for listing in the National Register, he shall immediately notify the district engineer;

(10) That the construction or operation of the activity will not impair reserved tribal rights, including, but not limited to, reserved water rights and treaty fishing and hunting rights;

(11) That in certain states, an individual state water quality certification must be obtained or waived (See § 330.9);

(12) That in certain states, an individual state coastal zone management consistency concurrence must be obtained or waived (See § 330.10);

(13) That the activity will comply with regional conditions which may have been added by the division engineer (See § 330.8(a)); and

(14) That the management practices listed in § 330.6 of this part shall be followed to the maximum extent practicable.

(c) *Further information.* (1) District engineers are authorized to determine if an activity complies with the terms and conditions of a nationwide permit unless that decision must be made by the division engineer in accordance with § 330.7.

(2) Nationwide permits do not obviate the need to obtain other Federal, state or local authorizations required by law.

(3) Nationwide permits do not grant any property rights or exclusive privileges.

(4) Nationwide permits do not authorize any injury to the property or rights of others.

(5) Nationwide permits do not authorize interference with any existing or proposed Federal project.

(d) *Modification, Suspension or Revocation of Nationwide Permits.* The Chief of Engineers may modify, suspend, or revoke nationwide permits in accordance with the relevant procedures of 33 CFR 325.7. Such authority includes, but is not limited to: adding individual, regional, or nationwide conditions; revoking authorization for a category of activities or a category of waters by requiring individual or regional permits; or revoking an authorization on a case-by-case basis. This authority is not limited to concerns for the aquatic environment as is the discretionary authority in § 330.8.

**§ 330.6 Management practices.**

(a) In addition to the conditions specified in § 330.5 of this Part, the following management practices shall be followed, to the maximum extent practicable, in order to minimize the adverse effects of these discharges on the aquatic environment. Failure to comply with these practices may be cause for the district engineer to recommend, or the division engineer to take, discretionary authority to regulate the activity on an individual or regional basis pursuant to § 330.8 of this Part.

(1) Discharges of dredged or fill material into waters of the United States shall be avoided or minimized through the use of other practical alternatives.

(2) Discharges in spawning areas during spawning seasons shall be avoided.

(3) Discharges shall not restrict or impede the movement of aquatic species indigenous to the waters or the passage of normal or expected high flows or cause the relocation of the water (unless the primary purpose of the fill is to impound waters).

(4) If the discharge creates an impoundment of water, adverse impacts on the aquatic system caused by the accelerated passage of water and/or the restriction of its flow shall be minimized.

(5) Discharge in wetlands areas shall be avoided.

(6) Heavy equipment working in wetlands shall be placed on mats.

(7) Discharges into breeding areas for migratory waterfowl shall be avoided.

(8) All temporary fills shall be removed in their entirety.

**§ 330.7 Notification procedures.**

(a) The general permittee shall not begin discharges requiring pre-discharge notification pursuant to the nationwide permit at § 330.5(a)(26):

(1) Until notified by the district engineer that the work may proceed under the nationwide permit with any special conditions imposed by the district or division engineer; or

(2) If notified by the district or division engineer that an individual permit may be required; or

(3) Unless 20 days have passed from receipt of the notification by the district engineer and no notice has been

received from the district or division engineer.

(b) Notification pursuant to the nationwide permit at § 330.5(a)(26) must be in writing and include the information listed below. Notification is not an admission that the proposed work would result in more than minimal impacts to waters of the United States; it simply allows the district or division engineer to evaluate specific activities for compliance with general permit criteria.

(1) Name, address, and phone number of the general permittee;

(2) Location of the planned work;

(3) Brief description of the proposed work, its purpose, and the approximate size of the waters, including wetlands, which would be lost or substantially adversely modified as a result of the work; and

(4) Any specific information required by the nationwide permit and any other information that the permittee believes is appropriate.

(c) *District engineer review of notification.* Upon receipt of notification, the district engineer will promptly review the general permittee's notification to determine which of the following procedures should be followed:

(1) If the nationwide permit at § 330.5(a)(26) is involved and the district engineer determines either, (i) the proposed activity falls within a class of discharges or will occur in a category of waters which has been previously identified by the Regional Administrator, Environmental Protection Agency; the Regional Director, Fish and Wildlife Service; the Regional Director, National Marine Fisheries Service; or the heads of the appropriate state natural resource agencies as being of particular interest to those agencies; or (ii) the particular discharge has not been previously identified but he believes it may be of importance to those agencies, he will promptly forward the notification to the division engineer and the head and appropriate staff officials of those agencies to afford those agencies an adequate opportunity before such discharge occurs to consider such notification and express their views, if any, to the district engineer concerning whether individual permits should be required.

(2) If the nationwide permits at § 330.5(a) (7), (17), or (21) are involved and the Environmental Protection Agency, the Fish and Wildlife Service, the National Marine Fisheries Service or the appropriate state natural resource or water quality agencies forward concerns to the district engineer, he will forward those concerns to the division engineer together with a statement of the factors pertinent to a determination of the environmental effects of the proposed discharges, including those set forth in the 404(b)(1) guidelines, and his views on the specific points raised by those agencies.

(3) If the nationwide permit at § 330.5(a)(21) is involved the district engineer will give notice to the Environmental Protection Agency and the appropriate state water quality agency. This notice will include as a minimum the information required by paragraph (b) of this section.

(d) *Division engineer review of notification.* The division engineer will review all notifications referred to him in accordance with paragraph (c)(1) or (c)(2) of this section. The division engineer will require an individual permit when he determines that an activity does not comply with the terms or conditions of a nationwide permit or does not meet the definition of a general permit (see 33 CFR 322.2(f) and 323.2(n)) including discharges under the nationwide permit at § 330.5(a)(26) which have more than minimal adverse environmental effects on the aquatic environment when viewed either cumulatively or separately. In reaching his decision, he will review factors pertinent to a determination of the environmental effects of the proposed discharge, including those set forth in the 404(b)(1) guidelines, and will give full consideration to the views, if any, of the federal and state natural resource agencies identified in paragraph (c) of this section. If the division engineer decides that an individual permit is not required, and a federal or appropriate state natural resource agency has indicated in writing that an activity may result in more than minimal adverse environmental impacts, he will prepare a written statement, available to the public on request, which sets forth his response to the specific points raised by the commenting agency. When the division engineer reaches his decision he will notify the district engineer, who will immediately notify the general permittee of the division engineer's decision.

**§ 330.8 Discretionary authority.**

Except as provided in paragraphs (c)(2) and (d) of this section, division engineers on their own initiative or upon recommendation of a district engineer are authorized to modify nationwide permits by adding regional conditions or to override nationwide permits by requiring individual permit applications on a case-by-case basis, for a category of activities, or in specific geographic areas. Discretionary authority will be based on concerns for the aquatic environment as expressed in the guidelines published by EPA pursuant to section 404(b)(1). (40 CFR Part 230)

(a) *Activity Specific conditions.* Division engineers are authorized to modify nationwide permits by adding individual conditions on a case-by-case basis applicable to certain activities within their division. Activity specific conditions may be added by the District Engineer in instances where there is mutual agreement between the district engineer and the permittee. Furthermore, district engineers will condition NWPs with conditions which have been imposed on a state section 401 water quality certification issued pursuant to § 330.9 of this Part.

(b) *Regional conditions.* Division engineers are authorized to modify nationwide permits by adding conditions on a generic basis applicable to certain activities or specific geographic areas within their divisions. In developing regional conditions, division and district engineers will follow standard permit processing procedures as prescribed in 33 CFR Part 325 applying the evaluation criteria of 33 CFR Part 320 and appropriate parts of 33 CFR Parts 321, 322, 323, and 324. Division and district engineers will take appropriate measures to inform the public of the additional conditions.

(c) *Individual permits*—(1) *Case-by-Case.* In nationwide permit cases where additional individual or regional conditioning may not be sufficient to address concerns for the aquatic environment or where there is not sufficient time to develop such conditions under paragraphs (a) or (b) of this section, the division engineer may suspend use of the nationwide permit and require an individual permit application on a case-by-case basis. The district engineer will evaluate the application and will either issue or deny a permit. However, if at any time the reason for taking discretionary authority is satisfied, then the division engineer may remove the suspension, reactivating authority under the nationwide permit. Where time is of the essence, the district engineer may telephonically recommend that the division engineer assert discretionary authority to require an individual permit application for a specific activity. If the division engineer concurs, he may orally authorize the district engineer to implement that authority. Oral authorization should be followed by written confirmation.

(2) *Category.* Additionally, after notice and opportunity for public hearing, division engineers may decide that individual permit applications

should be required for categories of activities, or in specific geographic areas. However, only the Chief of Engineers may modify, suspend, or revoke nationwide permits on a statewide or nationwide basis. The division engineer will announce the decision to persons affected by the action. The district engineer will then regulate the activity or activities by processing an application(s) for an individual permit(s) pursuant to 33 CFR Part 325.

(d) For the nationwide permit found at § 330.5(a)(26), after the applicable provisions of § 330.7(a) (1) and (3) have been satisfied, the permittee's right to proceed under the general permit may be modified, suspended, or revoked only in accordance with the procedure set forth in 33 CFR 325.7.

(e) A copy of all modifications or revocations of activities covered by nationwide permits will be forwarded to the Office of the Chief of Engineers, ATTN: DAEN–CWO–N.

**§ 330.9 State water quality certification.**

(a) State water quality certification is required for nationwide permits which may result in any discharge into waters of the United States. If a state issues a water quality certification which includes special conditions, the district engineer will add these conditions as conditions of the nationwide permit in that state. However, if such conditions do not comply with the provisions of 33 CFR 325.4 or if a state denies a required 401 certification for a particular nationwide permit, authorization for all discharges covered by the nationwide permit within the state is denied without prejudice until the state issues an individual or generic water quality certification or waives its right to do so. A district engineer will not process an individual permit application for an activity for which authorization has been denied without prejudice under the nationwide permit program. However, if the division engineer determines that it would otherwise be appropriate to exercise his discretionary authority, pursuant to § 330.8, to override the nationwide permit or permits in question, he may do so, and the district engineer may proceed with the processing of individual permit applications. In instances where a state has denied the 401 water quality certification for discharges under a particular nationwide permit, applicants must furnish the district engineer with an individual or generic 401 certification or a copy of the application to the state for the certification. If a state fails to act within a reasonable period of time (see § 325.2(b)(1)(ii)), a waiver will be presumed. Upon receipt of an individual or generic certification or a waiver of certification, the proposed work is authorized under the nationwide permit. If a state issues a conditioned individual certification, the district engineer will include those conditions that comply with 33 CFR 325.4 as special conditions of the nationwide permit (see 33 CFR Part 330.8(a)) and notify the applicant that the work is authorized under the nationwide permit provided all conditions are met.

(b) Certification requirements for nationwide permits fall into the following general categories:

(1) *No certification required.* Nationwide permits numbered 1, 2, 4, 5, 8, 9, 10, 11, and 19 do not involve activities which may result in a discharge and therefore 401 certification is not applicable.

(2) *Certification sometimes required.* Nationwide permits numbered 3, 6, 7, 13, 20, 21, 22, and 23 each involve various activities, some of which may result in a discharge and require certification, and others of which do not. State denial of certification for any specific nationwide permit in this category affects only those activities involving discharges. Those not involving discharges remain in effect.

(3) *Certification required.* Nationwide permits numbered 12, 14, 15, 16, 17, 18, 24, 25, and 26 involve activities which would result in discharges and therefore 401 certification is required.

(c) District engineers will take appropriate measures to inform the public of which waterbodies or regions within the state, and for which nationwide permits, an individual 401 water quality certification is required.

**§ 330.10 Coastal zone management consistency determination.**

In instances where a state has not concurred that a particular nationwide permit is consistent with an approved coastal zone management plan, authorization for all activities subject to such nationwide permit within or affecting the state coastal zone agency's area of authority is denied without prejudice until the applicant has furnished to the district engineer a coastal zone management consistency determination pursuant to section 307 of the Coastal Zone Management Act and the state has concurred in it. If a state does not act on an applicant's consistency statement within six months after receipt by the state, consistency shall be presumed. District engineers will take appropriate measures to inform the public of which waterbodies or regions within the state, and for which nationwide permits, such individual consistency determination is required. District engineers will not process any permit application for an activity which has been denied without prejudice under the nationwide permit program. However, if the division engineer determines that it would otherwise be appropriate to exercise his discretionary authority, pursuant to § 330.8, to override the nationwide permit or permits in question, he may do so, and the district engineer may proceed with the processing of individual permit applications.

**§ 330.11 Nationwide permit verification.**

(a) General permittees may, and in some cases must, request from a district engineer confirmation that an activity complies with the terms and conditions of a nationwide permit. District engineers will respond promptly to such requests. The response will state that the verification is valid for a period of no more than two years or a lesser period of time if deemed appropriate. Section 330.12 takes precedence over this section, therefore, it is incumbent upon the permittee to remain informed of changes to nationwide permits.

(b) If the district engineer decides that an activity does not comply with the terms or conditions of a nationwide permit, he will so notify the person desiring to do the work and indicate that an individual permit is required (unless covered by a regional permit).

(c) If the district engineer decides that an activity does comply with the terms and conditions of a nationwide permit he will so notify the general permittee. In such cases, as with any activity which qualifies under a nationwide permit, the general permittee's right to proceed with the activities under the nationwide permit may be modified, suspended, or revoked only in accordance with the procedures of 33 CFR 325.7.

**§ 330.12 Expiration of nationwide permits.**

The Chief of Engineers will review nationwide permits on a continual basis, and will decide to either modify, reissue (extend) or revoke the permits at least every five years. If a nationwide permit is not modified or reissued within five years of publication in the **Federal Register**, it automatically expires and becomes null and void. Authorization of activities which have commenced or are under contract to commence in reliance upon a nationwide permit will remain in effect provided the activity is completed within twelve months of the date a nationwide permit has expired or was revoked unless discretionary permit authority has been exercised in

accordance with § 330.8 of this Part or modification, suspension, or revocation procedures are initiated in accordance with the relevant provisions of 33 CFR 325.7. Activities completed under the authorization of a nationwide permit which was in effect at the time the activity was completed continue to be authorized by that nationwide permit.

[FR Doc. 86-25301 Filed 11-12-86; 8:45 am]
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CITY HALL
14405 N. Palm Street
El Mirage, AZ 85335
(602) 972 8116

ROSARIO VALENZUELA
Mayor

JOHN GARZA
Vice Mayor

CAROLINA HERNANDEZ
MARY IZAGUIRRE
GLENN KELTGEN
ARTHUR MORENO
ROBERT ROBLES
City Councilors

June 26, 1990

Rick Acuña
Browning-Ferris Industries of Arizona, Inc.
1580 E. Elwood
Phoenix, AZ 85040

RE: Cholla Landfill Floodplain Review [Landfill]

Dear Rick:

In accordance with the provisions of the Floodplain Management Ordinance of the City of El Mirage, the City has reviewed the application of Browning-Ferris Industries of Arizona, Inc. (BFI) for a floodplain development permit for the Cholla Landfill. Pursuant to the Floodplain Management Ordinance, your application for a floodplain development permit for the Cholla Landfill is hereby approved, subject to the conditions set forth in this letter.

In the application, you incorporated by reference the floodplain development permit application for the Cholla Landfill bank stabilization. This approval likewise incorporates the floodplain development permit approval for the Cholla landfill bank stabilization as set forth in my letter of June 26, 1990. The landfill shall not accept refuse until the construction of the bank stabilization is complete.

Although the area of the Cholla Landfill is not currently included on the official Flood Insurance Rate Maps (FIRM) for this site, preliminary indications are that a portion of the subject area may be included in the next official map. Pursuant to the Floodplain Management Ordinance of the City of El Mirage, I deem this area to be subject to jurisdiction of the City of El Mirage with respect to floodplain matters.

Applications have been submitted to the Arizona Department of Environmental Quality (ADEQ) for the Cholla Landfill. No construction on the actual landfill shall commence until the aquifer protection permit and the solid waste disposal operations plan have been approved by ADEQ.

Sincerely,

Scott Lind
Floodplain Administrator

cc: Wood & Associates and Mathews, Kessler & Associates

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CITY HALL
14405 N. Palm Street
El Mirage, AZ 85335
(602) 972 8116

ROSARIO VALENZUELA
Mayor

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Vice Mayor

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GLENN KELTGEN
ARTHUR MORENO
ROBERT ROBLES
City Councilors

June 26, 1990

Rick Acuña
Browning-Ferris Industries of Arizona, Inc.
1580 E. Elwood
Phoenix, AZ 85040

RE: Cholla Landfill Floodplain Review [Bank Stabilization]

Dear Rick:

In accordance with the provisions of the Floodplain Management Ordinance of the City of El Mirage, the City has reviewed the application of Browning-Ferris Industries of Arizona, Inc. (BFI) for a floodplain development permit for the Cholla Landfill Bank Stabilization and Levee Repair. Pursuant to the Floodplain Management Ordinance, your application for a floodplain development permit for the Cholla Landfill Bank Stabilization and Levee Repair is hereby approved, subject to the conditions set forth in this letter.

All construction and other activities shall be in accordance with the Engineering Design, Calculations and Studies for Cholla Landfill Bank Stabilization and Levee Repair, prepared by Mathews, Kessler & Associates dated December, 1989 (the "Report"), as modified by a letter from Mathews, Kessler & Associates to Mr. Scott Lind, Floodplain Administrator, dated March 27, 1990, and as amende by the following paragraph.

The first paragraph on page 31 of the Report is amended to read:

> To insure the continued integrity of the rip-rap bank stabilization project, the landfill owner will commit to semiannual field inspections, as well as field inspections after a flooding event which equals or exceeds 20,000 cfs, for the entire stabilized bank between Olive Avenue and Northern Avenue. The City will be provided a certification by a registered professional engineer that the system is structurally intact and adequately maintained. In the event that structural defects or need for maintenance should be noted, the City of El Mirage will be notified within 10 days of the certification of further action to be taken. That notification shall include a specific plan of action to implement maintenance and repairs as necessary to insure the integrity of the system.

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The City of El Mirage will require that as-built drawings be submitted along with a certification from a registered professional engineer that site improvements were constructed in accordance with the construction drawings.

Sincerely

Scott Lind
Floodplain Administrator

cc: Wood & Associates
Mathews, Kessler & Associates

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Attachment 4

FINAL ENVIRONMENTAL REGULATORY COMPLIANCE EVALUATION REPORT, UNION ROCK & MATERIALS CORPORATION, PLANT NUMBER SIX, EL MIRAGE, ARIZONA

# FINAL
# ENVIRONMENTAL REGULATORY COMPLIANCE EVALUATION REPORT

## UNION ROCK & MATERIALS CORPORATION
Plant Number Six
El Mirage, Arizona

**Prepared By:**

SCS ENGINEERS
2702 North 44th Street
Suite 105 B
Phoenix, Arizona 85008
(602) 840-2596

File No. 10.88011.00
April 3, 1989

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# SECTION 1
# INTRODUCTION

## BACKGROUND

Union Rock and Materials Corporation (Union Rock) has requested that a Real Estate Transfer Environmental Compliance Audit be performed at its Plant Number Six in El Mirage, Arizona. Operations at the site include aggregate mining, equipment maintenance and storage, and concrete and asphalt production.

## SCOPE OF WORK

SCS Engineers (SCS) was retained by Union Rock to perform a Real Estate Transfer Environmental Compliance Audit to evaluate environmental conditions at the subject property. The audit was performed in accordance with our proposal dated December 19, 1988, and consisted of a site reconnaissance, site history search, review of regulatory records, soil sampling and laboratory analysis, and this report. To assist SCS, Mr. Bill Peck of Union Rock supplied the following materials:

- aerial photograph of El Mirage property, dated 12/20/88;
- geotechnical boring logs from Olive Avenue Bridge area, dated 12/3/84 through 1/22/85;
- aggregate test worksheets and sieve analyses, dated 2/26/75;
- pump test data sheet for Union Rock and Material Well #1, dated 12/2/88;
- Policy of Title Insurance issued by Arizona Title Insurance and Trust Company, Policy No. S-347170-T;
- survey plat of W1/2 of Section 36, T3N, R1W, dated 1/17/89;
- Bureau of Air Pollution Control Operating Permit No. A8600158 and three citations, dated 10/27/88;
- Arizona Department of Environmental Quality Registration for Underground Storage Tanks form, filed 4/30/86; and
- water sample laboratory report for water sample collected on 2/2/86.



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## LIMITATIONS

This report has been specifically prepared for Union Rock with regard to the assessment of environmental conditions at the subject site. The report has been prepared within the limits prescribed by Union Rock in a manner consistent with the level of care and skill ordinarily exercised by other professional consultants, under similar circumstances at the time the services were performed, in this or similar localities. No other representation, either expressed or implied, and no warranty or guarantee is made as to the professional advice presented herein. Because regulatory criteria are constantly changing, contaminant levels presently considered low may, in the future, fall under different regulatory standards that require remediation. Positive identification of asbestos-containing materials can only be accomplished through laboratory analysis; no such analyses were made as part of this investigation.

# SECTION 2
# SITE INFORMATION

## LOCATION

The site is located at 8641 North El Mirage Road in El Mirage, Arizona, and occupies approximately 320 acres in the west half of Section 36, Township 3 North, Range 1 West of the Gila and Salt River Base Line Meridian. A site location map is provided in Figure 1.

## DESCRIPTION

On January 31, 1989, SCS personnel conducted a visual reconnaissance of the site in order to gain an appreciation of site conditions and current uses. Surface and subsurface soil samples were subsequently collected on February 8 and 9, 1989.

Union Rock Plant No. 6, occupying approximately 75 acres in the northern portion of the property, consists of an aggregate mining operation, a concrete batch mixing plant, an asphalt batch mixing plant, and several buildings including an office, residence, and shops for maintenance of trucks and equipment. Kiewit Western Company (Kiewit) maintains an equipment and materials storage area on the southern portion of the property which occupies approximately seven acres. The eastern portion of the property is comprised of the Agua Fria River bed, and the northern, western, and southern portions of the site include approximately 90 acres of idle agricultural land. A site map is provided in Figure 2.

## SITE USE

### Union Rock & Materials Corporation

The active sand and gravel pit in the north-central portion of the property currently supplies material for the plant. This pit is located west of the present bed of the Agua Fria River and ranges from approximately 50 to 80 feet deep. Mr. Marshall Jurn, plant superintendent for Union Rock Plant No. 6, reported that materials were formerly mined from the old pit in the river bed. A portion of the eastern edge of the active pit is covered by asphalt and concrete debris, reportedly from the removal of old roadbeds



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prior to resurfacing. Sand and gravel is transported from the active pit via a series of electrically-driven conveyors to the crusher, located 200 feet northeast of the northeast corner of the fenced area. Maintenance of the crusher and associated equipment includes the application of oil and grease. Minor staining, apparently the result of spillage during routine maintenance procedures, was observed in several areas around the crusher. Four drums of used gear oil were observed in various locations near the crusher during the site reconnaissance; these containers were stored on bare soil and identified as used gear oil by Non-Regulated Waste Labels. Drums of new oil were stored in a similar manner. A transformer, identified by Arizona Public Service (APS) as No. W9718, is located adjacent to the crusher on a concrete pad imprinted with the date September, 1980. No evidence of fluid leakage from this transformer was observed.

Crushed material is then transported to the wash plant, located approximately 500 feet southwest of the crusher within the fenced area, where it is washed, sieved and segregated. The wash plant, which requires a large volume of water for efficient operation, uses fresh water from an on-site well and recycled water from adjacent ponds containing used wash water. Used water flows from the wash plant into the first of a series of two settling ponds, each currently approximately 1.5 acres in size. Water is returned to the wash plant from the second settling pond. Three abandoned settling ponds, dry at the time of our site reconnaissance, are also located in the vicinity of the wash plant. Used lubricating oil is stored at the wash plant in the same manner as at the crusher, but some drums of lube oil and antifreeze were placed horizontally on a rack. Minor staining was observed beneath these drums. An APS transformer (No. W9719) is located adjacent to the wash plant. No visible leakage from this transformer was observed.

The concrete batch plant consists of mixing facilities and various aboveground tanks used to store admixes, dry cement and fly ash (Figure 3). The admix tanks are located on a concrete slab (without containment berms) and include three 1,500 gallon polyethylene tanks, one 500 gallon polyethylene tank, and one insulated tank with an estimated capacity of 2,000 gallons. Mobile tanker trucks also dispense special admixes for some batches of concrete, and one was parked near the polyethylene tanks during the site reconnaissance. An on-site review of Material Safety Data Sheets pertaining to admixes stored on the site

indicated that some of these materials contain compounds such as formaldehyde. Because formaldehyde is a listed hazardous waste, spillage of this material must be treated as such. A refrigerator truck trailer was also located in this area; during warmer months, it contains ice which is used to prevent elevated temperatures from occurring in concrete during the mixing process. During the site inspection, soil staining could not be evaluated because soil surrounding the concrete plant was wet from water used in the mixing process. Mr. Jurn reported that the concrete plant is approximately two years old. A small, two-story administrative office building is located south of the concrete plant.

Trucks used in the transport of concrete are rinsed of concrete residue at the truck wash area. The wash area consists of a concrete pad which directs wash water to an unlined ditch located east of the pad. This ditch also receives runoff from the steam cleaning area adjacent to the truck maintenance shop. Once water flows north through this ditch, it apparently infiltrates the soil north of the wash area. As this ditch accumulates concrete and sediment, it is periodically excavated. Piles of the excavated material were observed adjacent to the northeast corner of the wash area pad. Unused concrete from returning trucks has apparently been dumped along the west bank of the Agua Fria River, in the area east of the settling ponds.

According to Mr. Jurn, the asphalt batch plant was placed at its present location in the mid-1970s. The asphalt batch plant basically consists of aggregate storage bins, aggregate heater/dryer, mixer, and loading bin. Aggregate is transported from the storage bins into the heater/dryer via a conveyor. As the aggregate is heated and dried, air is drawn off and filtered through the bag house. Dust which accumulates in the bag house is periodically emptied into an unlined pit located east of the dryer.

The heater/dryer burner is fueled by burner fuel, which is stored in two aboveground tanks located south of the asphalt batch plant. These tanks, approximately 10,000 and 8,000 gallons in size, were reportedly installed in November 1988, and are situated on bare soil. One of the burner fuel tanks is insulated with what appears to be fiberglass and is reportedly kept warm by circulating warm heating oil through this insulating jacket. Diesel fuel is used as a backup fuel source for the aggregate heater/dryer burner. This diesel fuel is stored in two tanks; one aboveground 12,000 gallon steel tank, and one underground 12,000 gallon fiberglass tank. Mr. Jurn reported that the aboveground tank



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was installed in the early 1980s, and registration forms for the fiberglass tank indicate an installation date of 1976. It was noted that, during the site reconnaissance, there was no cap on the fill port of the underground tank. Underground piping carries both fuel sources to the aggregate heater burner.

After the aggregate is heated, it is mixed with asphalt oil. This oil is stored in two heated aboveground tanks with capacities of 20,000 and 25,000 gallons. Mr. Jurn reported the age of these tanks to be approximately 15 years. Due to the high viscosity of the asphalt oil, these tanks, associated plumbing and the mixing portion of the plant are heated by circulating warm oil around these areas. This heating oil is circulated from a 1,000 gallon aboveground tank which is warmed by a natural gas burner.

Areas of soil staining were observed between the burner oil tanks. Mr. Jurn reported that this staining occurred during installation of these tanks and their associated plumbing. Staining was also observed beneath the fill port of the eastern burner oil tank. Significant soil staining was noted beneath the south end of the aggregate dryer, where underground piping, which carries fuel to the burner, surfaces. Soil around this piping had apparently been previously excavated to a depth of approximately one foot, apparently in an effort to locate a leak in the line. It was noted during the site reconnaissance that a new section of galvanized pipe had been installed at this location. The soil around the new section of pipe was saturated with black oily liquid and an estimated one liter of this liquid was observed standing in the bottom of the excavation. The depth of potential contamination could not be evaluated at this location due to inadequate drill rig access caused by the proximity of equipment associated with the aggregate dryer, electrical transformers, and overhead structures.

Another area of significant surface soil staining was observed at the north end of the asphalt oil tanks. This staining is reportedly the result of spillage during filling of the tanks. A pit measuring approximately eight feet in length and width, and six feet in depth, had been excavated in an area roughly ten feet north of these tanks to capture this spillage. Stained soil from this pit is periodically excavated and disposed of in an area dedicated to storage of old asphalt, located south of the fenced area. The pit had been recently excavated to a depth of approximately seven feet at the time of our site reconnaissance, and little staining was evident below a depth of six feet. Asphalt oil was

also present in puddles directly beneath the north end of the tanks. None of the aboveground tanks in the asphalt batch plant area are surrounded by containment structures.

A transformer, APS No. W1815, is located near the asphalt batch plant. No leakage from this unit was observed during our site reconnaissance. Several partially full and empty drums were observed in various locations around the asphalt plant. These were labeled turbine oil, gear oil, and soap-based asphalt release compound.

Prior to loading, the beds of trucks which carry asphalt must be lubricated to enable the material to flow smoothly during dumping. A truck bed spray rack is located approximately 100 feet west of the asphalt batch plant. As trucks enter the plant, the beds are sprayed with a soap compound which is stored in a pressurized 550 gallon aboveground tank. Mr. Jurn reported that prior to late 1988, diesel fuel was used as the lubricating compound. This fuel was also stored in an aboveground 500 gallon tank, apparently filled from the larger underground diesel fuel tank at the asphalt plant via an underground supply line. Significant staining of soil was noted beneath and on both sides of the spray rack, extending south approximately 100 feet (excess lubricant drips from the truck bed as it is driven away from the spray rack). Several inches of soil are periodically scraped from the west side of the spray rack and stockpiled adjacent to the entrance road, at the location shown in Figure 3. Mr. Jurn reported that, during soil removal, stained soils were seldom observed beneath a depth of approximately one to two feet. After this soil has been stockpiled for an unspecified period of time, it may be used in certain aggregate/asphalt mixtures. Soil has not been removed from the east side of the spray rack due to the presence of utility lines in this area. Also observed near the truck bed spray area was a stack of old railroad ties, reportedly removed from beneath the asphalt batch plant when its foundation was replaced with concrete.

Cold mix asphalt is stockpiled in an area southeast of the truck bed spray area. This material is placed on bare soil and is laterally contained within concrete barriers. Old asphalt, generated by removal of old roadbeds and when new roadbeds are shaved, is stockpiled on bare soil in an area located approximately 200 feet south of the asphalt plant. A portion of this material is reused in certain asphalt mixtures. This stockpile area occupies approximately 1.5 acres.



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An automobile and truck fueling facility is located near the main entrance to the plant. According to registration forms filed with the Arizona Department of Environmental Quality, underground fuel storage tanks at this facility include two 10,000 gallon steel tanks of unknown age storing diesel fuel, and one 4,000 gallon steel tank of unknown age storing gasoline. Staining was noted adjacent to the diesel dispenser and around the fill ports of both diesel tanks. The dispenser nozzle is stored between uses in an upright four inch diameter pipe, around which significant staining was observed.

The equipment maintenance garage consists of service bays, parts storage, an office, steam cleaning area, and storage areas for new and used oils, etc. The service bays do not have floor drains or grease traps. Fluids drained from equipment include oil, which is disposed of in a 500 gallon underground waste oil tank, and antifreeze, which is reportedly reused after addition of new rust inhibitor. Batteries are stored in a locked storage room; six were in storage at the time of our site reconnaissance. No apparent acid staining was observed in this storage space. Approximately twelve 55 gallon drums of oil and grease are stored inside and adjacent to the south wall of the garage. No staining was noted on the concrete slab in this area. Other liquids stored inside the garage include new antifreeze (one drum), alkaline corrosion inhibitor (one drum), and used antifreeze to be reused (one drum). A 30 gallon drum of granular absorbent is located in a central area for use in controlling spills of these materials.

Used materials are stored outside and adjacent to the west wall for eventual disposal. One 55 gallon drum is dedicated to storage of spent solvent, which reportedly accumulates at a rate of approximately one drum every six months. This drum is labeled with a hazardous waste label, although the lettering has become illegible. As previously mentioned, waste oil is stored in the underground tank, also located outside and adjacent to the west wall of the garage. This tank is reportedly emptied two times per month by B S & W Energy Corporation, who also reportedly collect the spent solvent on an as-needed basis. An empty fiberglass tank with an estimated capacity of 500 gallons is also located in this area. This tank reportedly formerly held dilute acid, which was used in cleaning residue from concrete trucks. This method of cleaning has reportedly not taken place at this site for approximately five years. Remodeling of the garage is presently occurring, and includes a new contained storage area for aboveground waste oil, oil, grease, and antifreeze tanks.

The steam cleaning area is located adjacent to the drum storage area outside the west wall of the maintenance shop. Equipment to be washed is parked on a concrete slab adjacent to the steam cleaner. Runoff is directed from this slab to the nearby unlined ditch, which also accepts runoff from the concrete truck wash area. A soap-based detergent solution is occasionally used in the steam cleaner.

Kiewit Western Company

Kiewit maintains an equipment storage yard at the south end of the property along Northern Avenue. During a site reconnaissance conducted on February 8, 1989, this area contained a steam cleaning area, vehicle maintenance area with an underground lube pit, portable restrooms, drum storage area, at least 14 portable fuel tanks, two removed underground storage tanks, and a residence.

The steam cleaning area is located in the southeast corner of the yard and consists of a steam cleaner mounted on a concrete pad. An area of significant oily stains, occupying approximately 25 square feet, was observed in soils adjacent to the west edge of the pad, apparently related to cleaning operations. Based on field observation, depth of this staining is at least one foot.

The vehicle maintenance area includes a drum storage rack, upon which were placed, in a horizontal position, six 55 gallon drums of oil, antifreeze, and unidentified solvent. Soil staining, apparently the result of leaking bung spouts and spillage during dispensing of these products, was noted beneath these drums. The bottom of the underground lube pit was covered with sheets of plywood, and it could not be ascertained if the bottom of the pit was soil or concrete. No visible staining was noted on the plywood in the bottom of the pit.

The drum storage area is comprised of a concrete pad surrounded by a six inch high berm. Ten 55 gallon drums (four of which were laid in a horizontal position) and ten 5 gallon cans were located on this pad at the time of our site reconnaissance. Contents of these containers included oil, petroleum naphtha solvent, grease, and antifreeze. Minor staining was observed on the concrete pad. Approximately 20 empty 55 gallon drums were stored outside the west edge of the pad.



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Fourteen portable fuel storage tanks, with estimated sizes of 1,000 to 10,000 gallons, were located along the west edge of the storage yard. These skid-mounted tanks were labeled oil, diesel fuel and gasoline, and appeared to contain only small amounts of product. Localized staining of soils beneath the tanks was noted. Two removed underground tanks with estimated capacities of 4,000 and 10,000 gallons were also stored in the yard. According to Mr. Jurn, these tanks would probably be converted to portable fuel tanks such as those described above. Minor spillage had occurred from the fill and pump ports of these tanks.

## TOPOGRAPHY AND DRAINAGE

The site is located on the west bank of the Agua Fria River, an ephemeral stream which flows from north to south. The old Union Rock pit is located in and topographically lower than the river bed. A levee approximately 20 feet high separates the river bed from adjacent wash water holding ponds to the west. Historical aerial photographs indicate that aggregate mining in this and adjacent areas has been conducted in the river bed and associated floodplain. Road and bridge construction has occurred both up and down stream from the site.

Various areas on the site have been leveled to facilitate agricultural, and mining and processing activities, producing an average topographic gradient across the property of approximately one percent. Several lined irrigation ditches were observed traversing the portion of the property formerly dedicated to irrigated agriculture; these are currently inactive. No areas of significant runoff accumulation were noted during the site reconnaissance. The wash water ponds do not appear to collect a significant amount of runoff from the site. As noted earlier, water accumulates in the drainage ditch adjacent to the concrete truck wash area.

## HYDROGEOLOGY

The site is located within the West Salt River sub-basin of the Phoenix Active Management Area. Three main water-bearing units comprise the basin-fill deposits. These units are, in descending order, the Quaternary upper alluvial unit, the Quaternary and Tertiary middle fine-grained unit, and the Tertiary lower conglomerate unit (U.S.

Bureau of Reclamation, 1976). Beneath the site, the depth to groundwater is approximately 300 feet (Reeter and Remick, 1986). This was verified by a pump test conducted by National Pump Service in December 1988 at the Union Rock well located in the northwest portion of the site (Figure 2). The direction of regional ground water flow appears to be generally to the west (Reeter and Remick, 1986).

A water sample, reportedly from this well, was collected on December 2, 1986, and analyzed by Western Technologies, Inc. (WTI). Volatile organic compounds were not detected, and concentrations of inorganic primary drinking water constituents met Safe Drinking Water Act standards.



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# SECTION 3
# ADJACENT LAND USE

Properties surrounding the site are presently either vacant, active and idle agricultural land, or dedicated to residential or light industrial/storage uses. Adjacent land uses are described below and shown on Figure 2.

## North - Vacant and Idle Agricultural Land

The site is bordered to the north by Olive Avenue. Idle agricultural land is present north and northwest of the property. A vacant plot of land occupies an area between this agricultural land and the Agua Fria River. Aerial photographs indicate that this vacant area has been graded within the past year. No visible indications of contamination were observed during a drive-by inspection, however, pesticide residues may be found on agricultural land in this and other agricultural areas adjacent to the site.

## East - Agua Fria River and Vacant Land

The eastern boundary of the site approximately coincides with the eastern bank of the Agua Fria River bed. To the east of the river bed lies the currently undeveloped Floodplain of the Agua Fria River. An area occupying approximately 1.5 acres, adjacent to the east bank of the river bed, contains fill material of unknown origin and type.

## South - Residential and Light Industrial

The site is bordered to the south by Northern Avenue. South of Northern Avenue are several parcels of land occupied by residences, stored trailers and equipment, an automobile salvage yard, and a small livestock corral. In the rear of one parcel, two polyethylene tanks labeled Armor Termite and Pest Control were observed. It is assumed that these tanks have been used for storage of pesticides. Because access to this site was restricted, contents and condition of these tanks could not be evaluated. The listed business address for Armor Termite and Pest Control is not at this location.



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West - Active and Idle Agricultural Land

The site is bordered on the west by El Mirage Road. West of El Mirage Road is agricultural land which is currently under cultivation. South of this cultivated area is a parcel of vacant land which was apparently cultivated in the past. Also present on this southern parcel is an abandoned residence which has miscellaneous solid waste dumped around the area including furniture, soil, and tree clippings. No obvious sources of significant contamination were observed during a drive-by reconnaissance of this area.

# SECTION 4
# HISTORICAL LAND USE

According to information obtained by interview with Mr. Marshall Jurn, Plant Superintendent for Union Rock Plant No. 6, the property was purchased in approximately 1975. Prior to that time, the facility was also operated as an aggregate mining concern.

A review of 11 aerial photographs, taken between 1967 and 1987, was conducted at Landis Aerial Survey to evaluate the historic uses of the site. It should be noted that no aerial photographs taken between the years of 1969 and 1978 were available for the site.

The 1967 photograph shows that the site, presently occupied by Union Rock, was predominantly cultivated agricultural land. A residence and small pond were also observed. Some grading and/or removal of aggregate material had occurred to the north and south of the site. A large lined drainage ditch, originating from Luke Air Force Base, intersects the Agua Fria River approximately one-half mile south of the site.

Subsequent photographs indicate land use remained relatively unchanged. Exceptions to this include the recent development of homes and a golf course one mile north and approximately one-half mile east of the property; aggregate mining at the Union Rock property (apparently commencing between 1969 and 1978), and southeast of Northern Avenue and the Agua Fria River; and construction of the City of Glendale Landfill one mile to the southeast between the years of 1969 and 1978. The 1987 photograph indicates that aggregate mining at Union Rock Plant No. 6, was being conducted from the old pit located in the Agua Fria River bed.



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# SECTION 5
# REGULATORY RECORDS REVIEW

## UNITED STATES ENVIRONMENTAL PROTECTION AGENCY (EPA)

The CERCLIS list is an EPA database that identifies properties that have known or suspected contamination problems. It includes all National Priority List (NPL) and proposed NPL sites under EPA's Superfund Program. CERCLIS data is cross-referenced by zip code. The site lies within the zip code 85345 and is adjacent to the zip codes 85351, 85363, and 85307. The site is not currently in this database. The City of Glendale Landfill, located approximately one mile southeast of the site, is on the CERCLIS list. No other listed properties are located within one mile of the site.

The RCRA database was reviewed to ascertain if there are known RCRA-regulated facilities near the site. The RCRA database is a computer-generated list, maintained by the EPA, of registered hazardous waste facilities. The database indicates whether a site is a generator of hazardous waste, or is a hazardous waste treatment, storage, or disposal facility (TSDF). The database also indicates whether the facility transports hazardous waste. A review of this database, dated October 21, 1987, did not reveal the presence of any such facilities located within one mile of the site.

## ARIZONA DEPARTMENT OF ENVIRONMENTAL QUALITY (ADEQ)

Available information reviewed from ADEQ includes the following:

- Registered and Leaking Underground Storage Tanks
- Solid Waste Landfill Directory
- Water Quality Assurance Revolving Fund (WQARF)
- Arizona CERCLA Information and Data System (ACIDS)
- Groundwater Quality Database for Arizona, 1987
- Arizona Hazardous Waste Treatment, Storage, and Disposal Facilities (TSD)
- Inventory of Registered Dry Wells
- Groundwater Quality Protection Permits



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Under state and federal law, persons who own or have owned underground storage tanks containing "regulated substances" are required to complete a notification form and submit it to the state. The assembled list (current through July 1988) is referred to by ADEQ as the UST list. The UST list was reviewed and five tanks have been registered for Union Rock Plant No. 6. These include the three tanks at the fueling facility, the diesel tank adjacent to the asphalt batch plant, and the waste oil tank adjacent to the garage. As of August, 1988, no releases from these tanks or those on adjacent properties had been reported to ADEQ.

ADEQ maintains a state-wide list of active landfills current through 1982. This directory was reviewed to ascertain if any active solid waste landfills are located within one mile of the site. ADEQ records indicate no active landfills are located near the site; however, according to Mr. Robert Larsen of the ADEQ Solid Waste Unit, the City of Glendale Landfill, located approximately one mile southeast of the site, should be on this list.

The State of Arizona has established a program to remedy sites which may have an actual or potential impact from hazardous substances upon waters of the state. The Water Quality Assurance Revolving Fund (WQARF) program allows the state to identify the extent and impact of contamination, and to identify responsible parties. The site is not currently located within any WQARF areas.

The Arizona CERCLA Information and Data System (ACIDS), generated by ADEQ, is a computer database list of locations subject to investigation concerning possible contamination of soil, surface or groundwater. Inclusion of any facility or site on this list does not mean that the site is contaminated, causing contamination, or in violation of state or federal statutes or regulations. There are no listed sites within one-half mile of the Union Rock site.

ADEQ maintains a list of contaminated wells within the State of Arizona, compiled into a groundwater quality database. This list, current through 1987, was reviewed to assess whether any of these wells are located near the site; none are located within one mile.

ADEQ maintains a list of hazardous waste treatment, storage, and disposal facilities in the State of Arizona. According to this list, no such facilities are located within one-half mile of the site.

ADEQ maintains a list of registered dry wells in the state. The list, current through October 1988, was reviewed and indicated that no registered dry wells are located on the site.

Any person disposing of any material that may affect groundwater quality is required to complete a Notice of Disposal (NOD) as the first step in applying for a Groundwater Quality Protection Permit (R9-20-201 et.seq.). Review of an ADEQ list of filed NODs, current through February 1988, indicated that no NOD had been filed for the site. No NODs have been submitted for facilities located within one mile of the site.

## MARICOPA COUNTY

Maricopa County maintains a local Emergency Planning District which has a Hazardous Materials Emergency Response Plan, as required by the Superfund Amendments and Reauthorization Act of 1986 (SARA). The site does not have any hazardous chemicals listed in this plan. No other facilities located within one mile of the site are listed in this plan.

Maricopa County Bureau of Air Pollution Control regulates air emissions from facilities within the county. Union Rock Plant No. 6 possesses a current annual operating permit issued by the Bureau. Copies of three citations were provided by Union Rock; these citations, issued on October 27, 1988, cited Plant No. 6 for violation of visible air emissions standards at the asphalt batch plant bag house, shaker, and concrete plant dump boot. According to Mr. Tony Jones, a supervisor from the Bureau, no other citations have been issued to Plant No. 6.



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# SECTION 6
# SURFACE SOIL SAMPLING

## SAMPLING PROCEDURES

Surface soil samples G-1 through -5 were collected from five random locations at the site formerly used for irrigated agricultural purposes as shown on Figure 2. The surface soil sampling program was developed from preliminary information obtained by inspection of historical aerial photographs and a visual inspection of the site.

Soil samples were collected on February 8, 1989, from a depth of 6 to 12 inches below the ground surface using a hand auger. The hand auger was decontaminated prior to and between collection of each sample by washing with detergent, followed by a deionized water double rinse. Samples were placed in glass jars, sealed, labeled, and chilled prior to delivery to the SCS Analytical Laboratory under chain-of-custody procedures.

## LABORATORY ANALYSES

Surface soil samples G-1 through -5 were analyzed for the presence of organochlorine pesticides in accordance with EPA Method 8080; laboratory reports are attached as Appendix A. All five samples exhibited the presence of p,p' -DDT, -DDE, and -DDD; the results of these analyses are presented in Table 1. Total concentrations of these three compounds ranged from 1.17 to 8.87 mg/Kg. According to the Arizona Department of Health Services (ADHS) Task Assignment No. 18 (February 6, 1989), the soil clean-up level for the sum of these three compounds is 5.0 mg/Kg. This value is exceeded in sample G-1, which exhibited a total concentration of 8.87 mg/Kg.



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TABLE 1

SUMMARY OF P,P', -DDT, -DDE, AND -DDD ANALYSES (mg/Kg)

| COMPOUND | G-1 | G-2 | G-3 | G-4 | G-5 |
|---|---|---|---|---|---|
| p,p' -DDE | 2.36 | 1.54 | 0.76 | 1.05 | 1.55 |
| p,p' -DDT | 6.07 | 0.82 | 0.40 | 0.90 | 2.27 |
| p,p' -DDD | 0.44 | 0.06 | 0.01 | 0.07 | 0.03 |
| Total Concentration | 8.87 | 2.42 | 1.17 | 2.02 | 3.85 |

# SECTION 7
# SUBSURFACE SOIL SAMPLING

## DRILLING AND SAMPLING

Ten boreholes, B-1 through B-4, B-4A, and B-5 through B-9 were drilled to depths of 16 to 30 feet on February 8 and 9, 1989, by Western Technologies Inc. with a CME 45 hollow stem auger drilling rig. Borehole locations are shown on Figure 3. Boring logs are presented in Appendix B.

Soil samples were collected at ten foot depth intervals, except where difficult drilling required sampling at intervals of five feet. Surface drive samples were collected at borings B-3, -4, and -5 to evaluate surficial contamination at these locations. Samples were collected using a California modified split spoon sampler sleeved with brass tubes. The sampler was driven into the soil below the auger with a 140-pound drop hammer falling 30 inches. The sampler was decontaminated prior to collection of each sample by washing with Alconox detergent, followed by a deionized water double rinse. Augers were steam cleaned between borings. Soil samples collected in the brass tubes were labeled, and sealed with teflon sheets, plastic caps, and tape before being preserved by chilling for delivery to the SCS Analytical Laboratory under chain-of-custody procedures.

Soil samples were evaluated for organic vapors in the field using an HNU Systems, Inc. Model PI-101 photoionization detector. This instrument measures organic vapor concentrations on a relative scale calibrated to parts per million benzene. Field measurements of collected samples were accomplished using a headspace technique, which involves placing soil into a sealable plastic bag and allowing vapors from the soil to equilibrate with the air inside the bag. The organic vapor concentration in the head-space is then measured by inserting the HNU probe into the bag. HNU readings obtained in this manner are shown on the boring logs adjacent to their respective sample depth.

Drill cuttings were placed adjacent to each boring. At the completion of drilling, the boreholes were backfilled to the ground surface with a sand-cement grout. Borings B-8 and B-9 were backfilled to approximately 10 feet below grade with drill cuttings from



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their respective boreholes at the request of Mr. Marshall Jurn. Drill cuttings, rather than grout, were used to ease anticipated excavation of the adjacent waste oil tank.

## STRATIGRAPHY

Geologic logging of the ten borings indicated differences in the lithologic sequences between boring locations. However, similarities exist between several sets of borings where borings were located in close proximity to one another. This situation occurs at borings B-1 and B-2, B-4 and B-4A, B-6 and B-7, and B-8 and B-9. Because the site borders the Agua Fria River, numerous depositional episodes account for areal irregularities in stratigraphy. Generally, each of the borings (except B-5) encountered surficial sand and gravel fill ranging in thickness of approximately 0.5 to 4 feet. Additionally, gravel and cobbles were encountered in the borings, except B-5 and B-6, at depths of 13 to 21.5 feet, resulting in auger refusal. Boring logs are attached as Appendix B.

Borings B-1 and B-2 were drilled to total depths of 20 and 23 feet, respectively. Clayey silt and silty clay was encountered in both borings from a depth of approximately 0.5 to 10 feet, underlain by silty sand to depths of 14 and 18 feet. Two to 4 feet of silty clay was then encountered, underlain by sandy gravel to auger refusal.

The upper nine feet of boring B-3 were similar to the surficial soils encountered in borings B-1 and B-2. However, from 9 to 16 feet, dry gravelly silt was encountered, underlain by one foot of silty fine-grained sand, and then sandy gravel to auger refusal at 20 feet.

Borings B-4 and B-4A were advanced to auger refusal at depths of 17 and 19.5 feet, respectively. Below three feet of fill, medium to fine-grained sand was penetrated to a depth of seven feet, underlain by clayey silt to 15 feet. Three feet of medium-grained sand was then encountered, underlain by gravel with cobbles to auger refusal.

Boring B-5 penetrated silty and gravelly sand from the ground surface to a depth of 11 feet, underlain by sandy clay with gravel to a depth of 18.5 feet. Fine-grained silty sand was then encountered to the designed depth of 30 feet.

After penetrating four feet of fill, boring B-6 encountered silty clay to a depth of eight feet, underlain by fine and medium-grained sand with gravel to the designed depth of 30 feet. Boring B-7 exhibited identical lithology, but encountered auger refusal in sandy gravel at a depth of 21.5 feet.

Boring B-8 penetrated sandy clay to a depth of 7.5 feet, underlain by clayey sand to a depth of 15 feet, and coarse to medium-grained sand with gravel to auger refusal at 16.5 feet. The upper 7.5 feet of boring B-9 were similar to boring B-8. Below this depth, to 13 feet, medium to fine-grained silty sand was encountered, underlain by gravel with cobbles to auger refusal at 20 feet.

## LABORATORY ANALYSES

Soil samples from borings B-1 and B-2, located adjacent to the underground diesel fuel tank at the asphalt batch plant, were analyzed for total petroleum hydrocarbons in accordance with EPA Method 8015 (modified). Samples from borings B-3, B-4 and B-4A, located adjacent to the aboveground oil tanks at the asphalt plant, were analyzed for waste oil in accordance with EPA Method 418.1. Samples collected from borings B-5, B-6, and B-7, located adjacent to the truck bed spray rack and fuel facility, were analyzed for total petroleum hydrocarbons in accordance with EPA Method 8015 (modified). Soil samples collected from borings B-8 and B-9, located adjacent to the waste oil tank at the garage, were analyzed for total petroleum hydrocarbons and solvents in accordance with EPA Methods 418.1 and 8010, respectively. Of the 27 samples collected, sample numbers B-2-20, B-6-15, and B-6-25 were archived. Laboratory reports appear in Appendix A.

Laboratory results of the 24 subsurface samples analyzed for total hydrocarbons are summarized in Table 2. Hydrocarbons were not detected (detection limit 10 mg/Kg) in soil samples collected from borings B-1, B-2, B-4, B-5, and B-7. Analyses of surface drive samples collected at borings B-3, B-4, and B-5 indicate that only sample B-3-0, located five feet north of the burner oil tanks, exhibits surficial hydrocarbon contamination. Hydrocarbon concentrations at this location decrease dramatically with depth, from 40,000 mg/Kg at the ground surface to 50 mg/Kg at a depth of 10 feet and 30 mg/Kg at a depth of 20 feet. Sample B-3-0 is the only analyzed sample to exceed the state action level for total hydrocarbons of 100 mg/Kg.



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Sample B-4A-15, collected just south of the asphalt oil overflow pit at a depth of 15 feet, exhibited 20 mg/Kg total hydrocarbons. This is the only sample at this location (B-4, B-4A) to exhibit detectable hydrocarbons.

Sample B-6-10, collected three feet west of the refueling area at a depth of 10 feet, exhibited 11 mg/Kg total petroleum hydrocarbons. This is the only sample at this location (B-6, B-7) to exhibit detectable hydrocarbons.

Samples B-8-10 and B-8-15 were collected just south of the underground waste oil tank at depths of 10 and 15 feet, respectively. Hydrocarbons were detected in both samples at 10 mg/Kg. Samples B-9-15 and B-9-20 were collected just west of the tank at depths of 15 and 20 feet, respectively. Hydrocarbons were detected in both samples at 30 mg/Kg. These four samples, along with sample B-9-10, were analyzed for purgeable halocarbon compounds in accordance with EPA Method 8010. The only compound detected in each of the five samples was 1,1-dichloroethene, ranging from 53.3 to 206 ug/Kg. The state action level for 1,1-dichloroethene is 700 ug/Kg (ADHS Task Assignment No. 18).

TABLE 2

SUMMARY OF TOTAL PETROLEUM HYDROCARBON ANALYSES

| BOREHOLE NO.* | SAMPLE NO. | DEPTH (Ft.) | TOTAL HYDROCARBONS (mg/Kg) |
|---|---|---|---|
| B-1 | B-1-10 | 10 | ND |
| B-2 | B-2-10 | 10 | ND |
| B-2 | B-2-23 | 23 | ND |
| B-3 | B-3-0 | 0 | 40,000 |
| B-3 | B-3-10 | 10 | 50 |
| B-3 | B-3-20 | 20 | 30 |
| B-4 | B-4-0 | 0 | ND |
| B-4 | B-4-10 | 10 | ND |
| B-4A | B-4A-15 | 15 | ND |
| B-5 | B-5-0 | 0 | 20 |
| B-5 | B-5-10 | 10 | ND |
| B-5 | B-5-20 | 20 | ND |
| B-5 | B-5-29 | 29 | ND |
| B-6 | B-6-10 | 10 | ND |
| B-6 | B-6-20 | 20 | 11 |
| B-6 | B-6-30 | 30 | ND |
| B-7 | B-7-10 | 10 | ND |
| B-7 | B-7-15 | 15 | ND |
| B-7 | B-7-20 | 20 | ND |
| B-8 | B-8-10 | 10 | ND |
| B-8 | B-8-15 | 15 | 10 |
| B-9 | B-9-10 | 10 | ND |
| B-9 | B-9-15 | 15 | 30 |
| B-9 | B-9-20 | 20 | 30 |

* Samples from boreholes B-1, B-2, B-5, B-6, and B-7 analyzed in accordance with EPA Method 8015 (modified); boreholes B-3, B-4, B-8, and B-9 analyzed in accordance with EPA Method 418.1 (modified).

ND - Not Detected (Detection Limit 10 mg/Kg)



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# SECTION 8

## REGULATORY COMPLIANCE ISSUES

The site is regulated by numerous federal, state, and county environmental statutes and regulations. The following is a brief description of regulatory compliance issues we have identified as possibly applicable to the site, along with their respective enforcement agencies:

- Resource Conservation and Recovery Act (RCRA) - ADEQ
- Clean Water Act (CWA) - EPA, U.S. Army Corps of Engineers
- Clean Air Act (CAA) - Maricopa County Bureau of Air Pollution Control
- Environmental Quality Act - ADEQ

The federal Resource Conservation and Recovery Act (RCRA) regulates both petroleum and chemical underground storage tanks, and hazardous and solid wastes. ADEQ has primary responsibility for the RCRA program in Arizona. RCRA specifically regulates the generation, transportation, and disposal of hazardous waste. Regulations concerning hazardous waste can be found in Title 40 of the Code of Federal Regulations (CFR) Parts 260-270. The regulations define a hazardous waste, and require that when hazardous wastes are generated, proper methods of handling, transporting, and disposing of these wastes be followed. Formaldehyde is a listed waste (40 CFR Part 261.33), and must be handled accordingly. Therefore, any spillage of this material must be treated as a hazardous waste.



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Solvents currently used in the garage for cleaning of parts, etc. may also contain constituents listed as hazardous waste. Storage, handling, and transportation of such wastes must be performed in accordance with 40 CFR Parts 262 and 263. According to the shop foreman, Mr. Bill Allen, waste solvent accumulates at a rate of approximately 55 gallons every six months. This rate of accumulation (approximately nine gallons or 33 kilograms per month) classifies the facility as a conditionally-exempt small quantity generator (40 CFR Part 261.5) which requires identification of all hazardous waste generated by the facility, disposal of this waste at a hazardous waste facility or landfill approved by the state for such wastes, and accumulation of no more than 1,000 kilograms of hazardous waste on the property.

Requirements for underground storage tanks are found in 40 CFR Part 280. Requirements for the upgrading of existing underground storage tank systems are present in 40 CFR 280.21. General operating requirements pursuant to 40 CFR Subpart C must be met, and release detection pursuant to 40 CFR Subpart D must be performed. Suspected releases must be reported pursuant to 40 CFR 280.50 and Arizona Revised Statutes (ARS) 49-1004. Closure and out-of-service requirements pursuant to 40 CFR Subpart G must be met.

The federal Clean Water Act (CWA) regulates the discharge of pollutants into navigable waters. With respect to the site, two programs under the CWA may be applicable. A Spill Prevention Control and Countermeasure plan is required by EPA pursuant to 40 CFR Part 112 to prevent the discharge of oil from aboveground storage tanks into navigable waters. This plan must be approved by a professional engineeer, and be available on site.

Section 404 of the CWA controls discharges of dredge or fill material into navigable waters. This section authorizes the U.S. Army Corps of Engineers (Corps) to issue permits for such discharges. According to Mr. Robert Dummer of the Corps, a Section 404 Permit is required for certain earth moving activities which are conducted in river bottoms, defined in this case as that portion of the floodplain devoid of vegetation. Placement of material on river banks (such as waste concrete or rip-rap) is also regulated in certain circumstances. Mr. Dummer reported that such activities affecting less than ten acres are covered by a nationwide permit, which authorized the activities outlined in 40 CFR 330.5(a)(26). If these activities, which include the placement of concrete on the west bank of the river and stockpiling of aggregate in the old pit, affect less than one acre, no action is necessary. If these activities affect between one and ten acres, notification to the district engineer is required; unless, according to Mr. Dummer, they occurred prior to October 1984. Activities affecting more than ten acres are not covered by the nationwide permit, and may require an individual Section 404 permit.

The federal Clean Air Act regulates stationary sources of air pollution by setting limits on air emissions from specific sources, such as those found at the site. The Maricopa County Bureau of Air Pollution Control has primacy for controlling emissions in Maricopa County, and has issued an operating permit to Union Rock Plant No. 6.

Arizona's Environmental Quality Act was passed in 1986. Two provisions of this act are applicable to the site. ARS 49-241 requires that ADEQ issue Aquifer Protection Permits for surface impoundments; therefore, such a permit is required for the ponds located on the site. Because aquifer protection permit regulations have not yet been promulgated, surface impoundments are regulated by existing groundwater quality permit regulations,



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A.A.C. Title 9, Chapter 20, Article 2. These regulations require that a Notice of Disposal be submitted to ADEQ for the active ponds at the site.

Of the five samples analyzed for organochlorine pesticides, one has exceeded the soil clean-up level for DDT of 5.0 mg/Kg. Provided that the application of a pesticide product registered under the federal Insecticide, Fungicide, and Rodenticide Act (FIFRA) was performed according to label requirements, the liability associated with the presence of a pesticide product is minimal. This is because a person deemed a responsible party pursuant to ARS 49-283 is not liable under Article 5, Chapter 35, Title 49 of the Arizona Revised Statutes, if it can be established that a release of a hazardous substance was caused solely by the application of a pesticide product registered under FIFRA according to label requirements (ARS 49-283.D.5). Because samples were collected from random locations on formerly cultivated land, the presence of DDT is most likely due to normal application associated with agricultural activities, as opposed to a release due to spilling or leakage.

# SECTION 9

# CONCLUSIONS

Based on the findings of our site reconnaissance, site history search, review of regulatory agency records, surface and subsurface soil sampling, and laboratory analysis, we conclude the following:

- Surface staining associated with spillage was observed adjacent to the aboveground burner and asphalt oil tanks adjacent to the asphalt batch plant. In cases where soil is in direct contact with the bottom of these tanks, early detection of tank leakage may not be readily accomplished. No spill containment structures are present around these tanks.

- The aboveground tanks, which have the potential to discharge oil into the Agua Fria River, are required to have a Spill Prevention Control and Countermeasure plan pursuant to 40 CFR Part 112.

- Surface staining was also observed in the vicinity of the truck bed spray area and the steam cleaning area at the Kiewit yard. However, soil sampling near the truck bed spray area did not detect the presence of total petroleum hydrocarbons. No samples were collected from the steam cleaning area at the Kiewit yard, because this area was not accessible for inspection until after the drilling program was in place. Additional sampling is not recommended in the Kiewit yard due to the limited extent of observable contamination. However, steps should be taken to minimize spillage in this area and in the vicinity of the vehicle maintenance area.



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- Total petroleum hydrocarbon and purgeable halocarbon compound analyses of soil samples collected at the site indicate that only one sample exceeded the state action level for analyzed compounds. The surface sample collected at boring B-3, located in the stained area between and adjacent to the burner oil tanks, exceeded the state action level of 100 mg/Kg for total hydrocarbons.

- Underground storage tanks on the property are currently registered with the state, in accordance with 40 CFR 280.22 and ARS 49-1002. Subsurface soil sampling adjacent to the five underground storage tanks on the site did not detect the presence of total petroleum hydrocarbons in excess of the state action level of 100 mg/Kg. Therefore, it appears that no releases have occurred from these tanks. However, the soil sampling program performed was not designed to detect releases from underground piping.

- One potential piping release was observed beneath the south end of the aggregate dryer. Samples were not collected at this location due to inadequate drill rig access caused by the proximity of the aggregate dryer, electrical transformers, and overhead structures.

- Laboratory analyses of surface soil samples collected from the undeveloped portions of the property formerly used for cultivated agriculture indicate that these areas contain p,p'-DDT, p,p'-DDE, and p,p'-DDD. One sample exceeded the state soil clean-up level of 5.0 mg/Kg. It is our opinion that, because samples were collected from random locations on formerly cultivated land, these levels represent background levels for this particular site. However, the presence of these

compounds in on-site surface soils should be considered if future land utilization involves residential and associated uses.

- Other than some relatively minor violations, the site is generally in compliance with the operating permit issued by the Maricopa County Bureau of Air Pollution Control.

- The submittal of a Notice of Disposal to ADEQ pursuant to R9-20-205 is required for the storage of aggregate wash water in the ponds on the property.

- The facility appears to be in compliance with 40 CFR Part 261.5, requirements for conditionally-exempt small quantity generators of hazardous waste, assuming that the current waste transporter is properly transporting and disposing of these wastes.

- Notification to the U. S. Army Corps of Engineers is required for earth moving and disposal activities which are conducted in the bottom or along the banks of the Agua Fria River, and affect more than one acre.

- Under existing operating conditions, adjacent properties do not appear to be a potential source of hazardous substances which are likely to have contaminated the site.



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# SECTION 10

# RECOMMENDATIONS

Based on our conclusions, we recommend the following:

- The underground storage tanks should be operated in compliance with 40 CFR Part 280. Piping associated with the tanks should be tightness tested to evaluate piping integrity.

- The extent of subsurface contamination associated with the piping release at the south end of the aggregate dryer should be evaluated. This could be accomplished by hand augering of the soil immediately adjacent to the replaced section of pipe. If the vertical and horizontal extent of soil contamination cannot be evaluated in this manner, the use of portable powered boring equipment may be necessary. If a quantity of greater than 100 pounds of burner oil or diesel fuel was released, a release must be reported to ADEQ pursuant to ARS 49-1004.

- A Spill Prevention Control and Countermeasure Plan pursuant to 40 CFR Part 112 should be prepared. The aboveground fuel storage tank systems and drum storage areas should be upgraded to include impermeable containment structures for spill control. In conjunction with this, existing surface spillage should be removed from the ground surface. This material should be disposed of at an authorized landfill or, if possible, used in asphalt mixtures.



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- Areas of contaminated soil storage, such as that adjacent to the truck bed spray rack, should have an impermeable surface such as concrete or asphalt upon which material may be stored without potentially contaminating the underlying soil.

- A Notice of Disposal for the wash water settling ponds should be submitted to ADEQ.

- The U. S. Army Corps of Engineers should be notified regarding the storage of aggregate in the old pit and the placement of concrete on the west bank of the Agua Fria River, provided these areas occupy more than one acre.

- Proper disposal of the waste solvent generated by the garage, in accordance with 40 CFR Part 261.5 requirements, should be confirmed by contacting the current waste transporter.

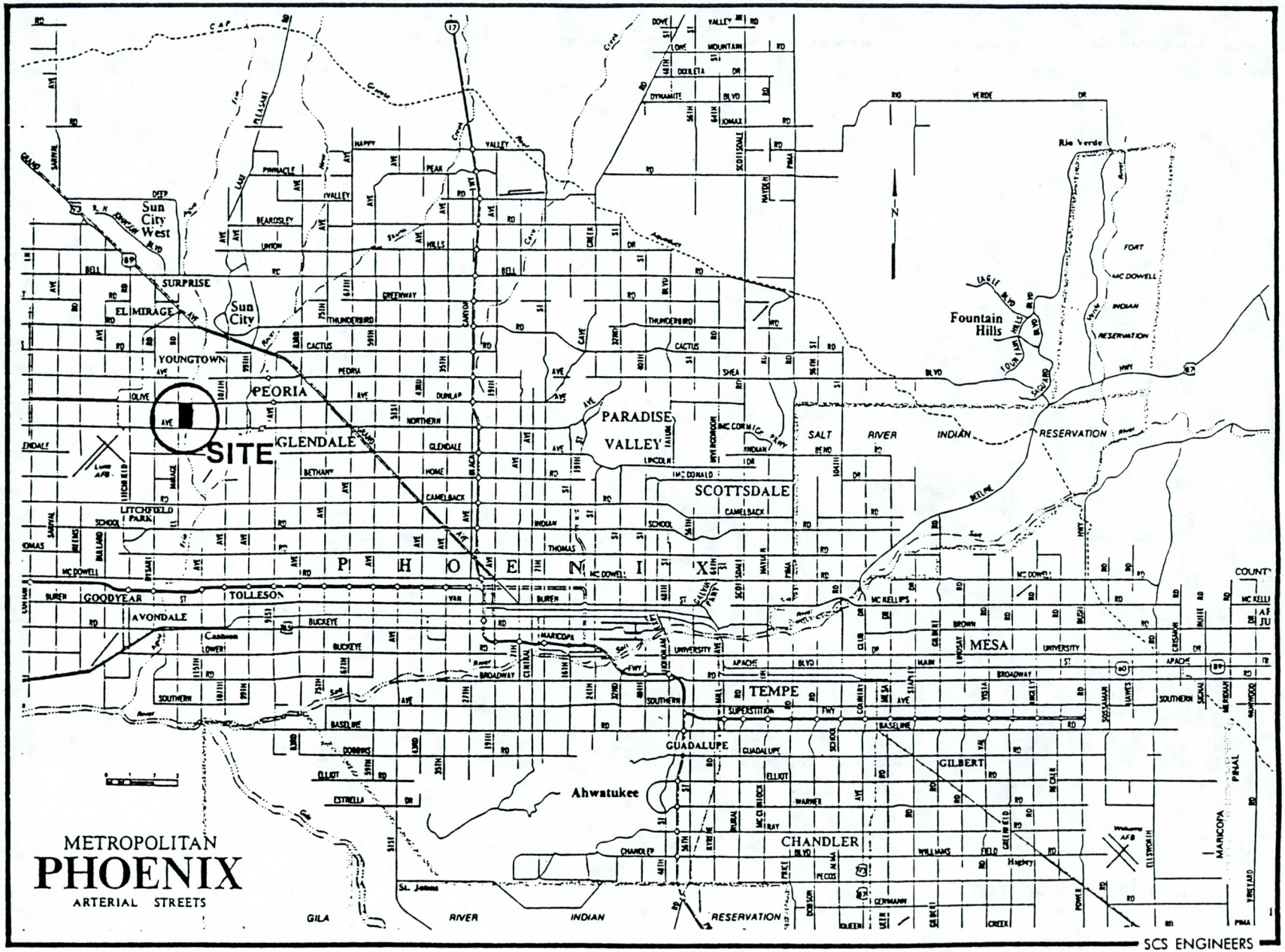


FIGURE 1 SITE LOCATION MAP

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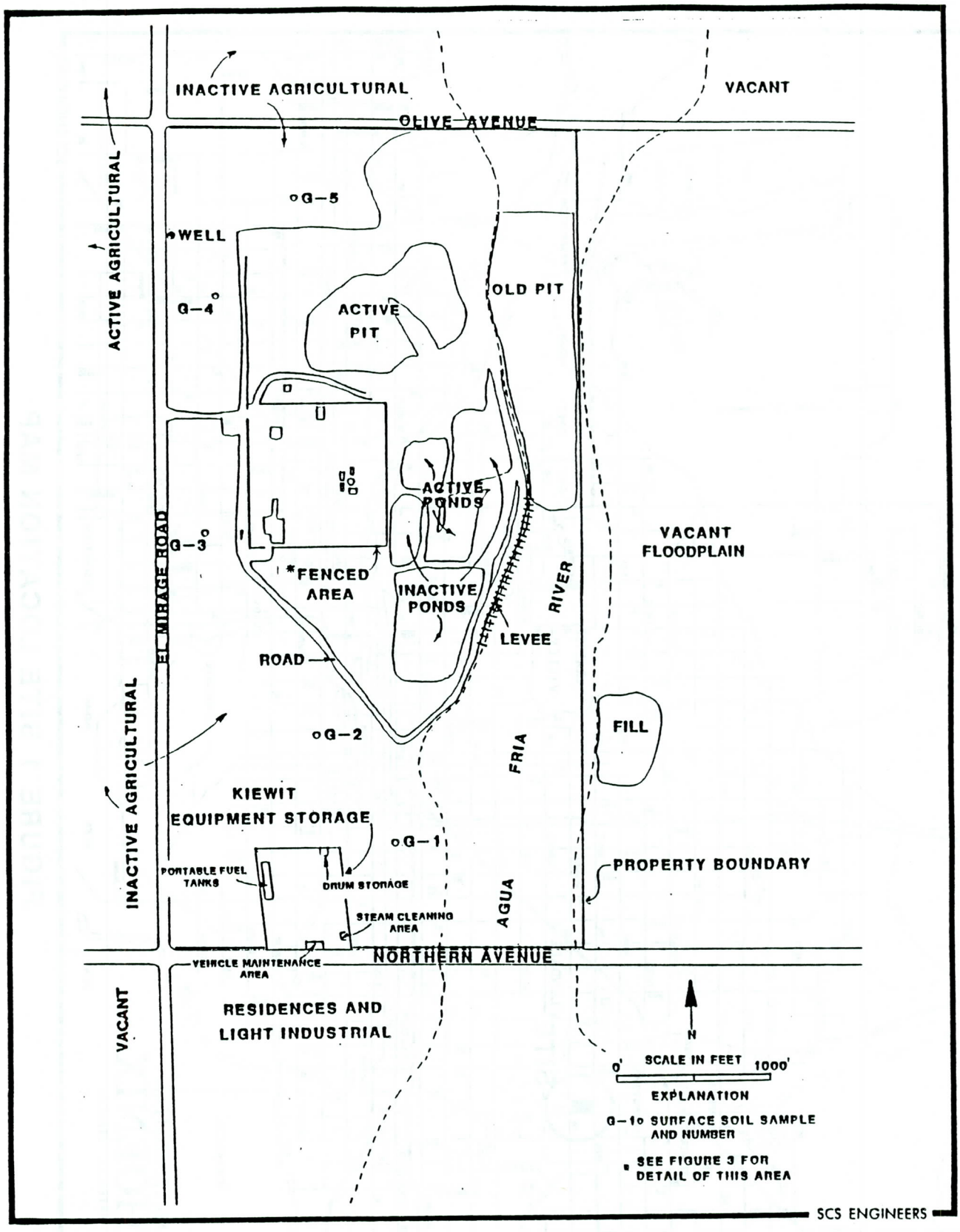


**FIGURE 2. SITE MAP**

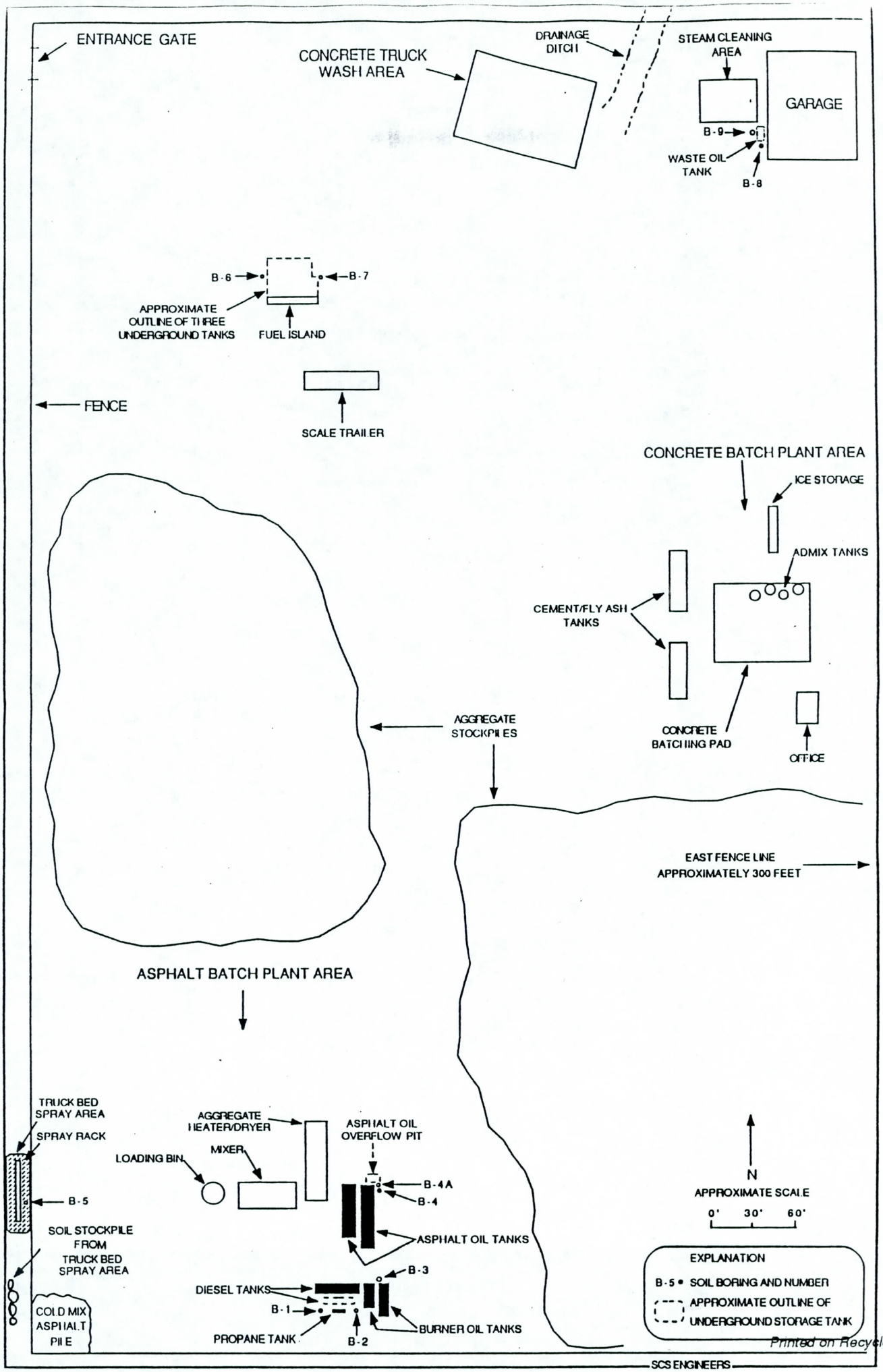


FIGURE 3. SITE PLAN

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# APPENDICES

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# APPENDIX A

# LABORATORY REPORTS

SCS ANALYTICAL LABORATORY
2860 WALNUT AVENUE
LONG BEACH, CALIFORNIA 90806
(213) 595-9324
FAX (213) 595-6709

MEMO

To: Brad Johnston/Chris Miller

From: Curtis B. Jenkins

February 28, 1989

Job No.: 1088011.00



## LABORATORY REPORT

Samples: Thirty two (32) soil samples from Union Rock, received 2/10/89, analyzed 2/24/89.

| Sample ID | EPA 418.1 -----mg/kg----- |
|---|---|
| B-3-0' | 40,000 |
| B-3-10' | 50 |
| B-3-20' | 30 |
| B-4-0' | ND |
| B-4-10' | ND |
| B-4A-15' | 20 |
| B-8-10' | 10 |
| B-8-15' | 10 |
| B-9-10' | ND |
| B-9-15' | 30 |
| B-9-20' | 30 |

| Sample ID | EPA 8015 ---mg/kg--- | Sample ID | EPA 8015 ---mg/kg--- |
|---|---|---|---|
| B-1-10' | ND | B-6-10' | 11 |
| B-2-10' | ND | B-6-20' | ND |
| B-2-23' | ND | B-6-30' | ND |
| B-5-0' | ND | B-7-10' | ND |
| B-5-10' | ND | B-7-15' | ND |
| B-5-20' | ND | B-7-20' | ND |
| B-5-29' | ND | | |

ND - Not Detected (<10)

EPA 8010 and EPA 8080 - see attached sheets

David Sincerbeaux
Chemist

Curtis B. Jenkins
Laboratory Director

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SCS ANALYTICAL LABORATORY
2860 WALNUT AVENUE
LONG BEACH, CALIFORNIA 90806
(213) 595-9324
FAX (213) 595-6709



Sample I.D.: B-8-10'
Date Received: 2/10/89
Date Analyzed: 2/24/89
Matrix: Soil
Project #: 1088011.00
File #: union1.rep

| Compound | Result ----ug/kg (ppb)---- | D.L. |
|---|---|---|
| Bromomethane | ND | 50 |
| Bromodichloromethane | ND | 5 |
| Bromoform | ND | 5 |
| Carbon Tetrachloride | ND | 5 |
| Chlorobenzene | ND | 5 |
| Chloroethane | ND | 50 |
| 2-Chloroethylvinyl Ether | ND | 50 |
| Chloroform | ND | 5 |
| Chloromethane | ND | 50 |
| Dibromochloromethane | ND | 5 |
| 1,1-Dichloroethane | ND | 5 |
| 1,2-Dichloroethane | ND | 5 |
| 1,1-Dichloroethene | 206 | 5 |
| trans-1,2-Dichloroethene | ND | 5 |
| 1,2-Dichloropropane | ND | 5 |
| cis-1,3-Dichloropropene | ND | 5 |
| trans-1,3-Dichloropropene | ND | 5 |
| Methylene Chloride | ND | 50 |
| 1,1,2,2-Tetrachloroethane | ND | 5 |
| Tetrachloroethene | ND | 5 |
| 1,1,1-Trichloroethane | ND | 5 |
| 1,1,2-Trichloroethane | ND | 5 |
| Trichloroethene | ND | 5 |
| Trichlorofluoromethane | ND | 5 |
| Vinyl Chloride | ND | 50 |

D.L. = Detection Limit
ND = Not Detected

SCS
ANALYTICAL
LABORATORY

2860 WALNUT AVENUE
LONG BEACH, CALIFORNIA 90806
(213) 595-9324
FAX (213) 595-6709



Sample I.D.: B-8-15'
Date Received: 2/10/89
Date Analyzed: 2/24/89
Matrix: Soil
Project #: 1088011.00
File #: union1.rep

| Compound | Result ----ug/kg (ppb)---- | D.L. |
|---|---|---|
| Bromomethane | ND | 50 |
| Bromodichloromethane | ND | 5 |
| Bromoform | ND | 5 |
| Carbon Tetrachloride | ND | 5 |
| Chlorobenzene | ND | 5 |
| Chloroethane | ND | 50 |
| 2-Chloroethylvinyl Ether | ND | 50 |
| Chloroform | ND | 5 |
| Chloromethane | ND | 50 |
| Dibromochloromethane | ND | 5 |
| 1,1-Dichloroethane | ND | 5 |
| 1,2-Dichloroethane | ND | 5 |
| 1,1-Dichloroethene | 53.3 | 5 |
| trans-1,2-Dichloroethene | ND | 5 |
| 1,2-Dichloropropane | ND | 5 |
| cis-1,3-Dichloropropene | ND | 5 |
| trans-1,3-Dichloropropene | ND | 5 |
| Methylene Chloride | ND | 50 |
| 1,1,2,2-Tetrachloroethane | ND | 5 |
| Tetrachloroethene | ND | 5 |
| 1,1,1-Trichloroethane | ND | 5 |
| 1,1,2-Trichloroethane | ND | 5 |
| Trichloroethene | ND | 5 |
| Trichlorofluoromethane | ND | 5 |
| Vinyl Chloride | ND | 50 |

D.L. = Detection Limit
ND = Not Detected

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2860 WALNUT AVENUE
LONG BEACH, CALIFORNIA 90806
(213) 595-9324
FAX (213) 595-6709



Sample I.D.: B-9-10'
Date Received: 2/10/89
Date Analyzed: 2/24/89
Matrix: Soil
Project #: 1088011.00
File #: union1.rep

| Compound | Result ----ug/kg (ppb)---- | D.L. |
|---|---|---|
| Bromomethane | ND | 50 |
| Bromodichloromethane | ND | 5 |
| Bromoform | ND | 5 |
| Carbon Tetrachloride | ND | 5 |
| Chlorobenzene | ND | 5 |
| Chloroethane | ND | 50 |
| 2-Chloroethylvinyl Ether | ND | 50 |
| Chloroform | ND | 5 |
| Chloromethane | ND | 50 |
| Dibromochloromethane | ND | 5 |
| 1,1-Dichloroethane | ND | 5 |
| 1,2-Dichloroethane | ND | 5 |
| 1,1-Dichloroethene | 133 | 5 |
| trans-1,2-Dichloroethene | ND | 5 |
| 1,2-Dichloropropane | ND | 5 |
| cis-1,3-Dichloropropene | ND | 5 |
| trans-1,3-Dichloropropene | ND | 5 |
| Methylene Chloride | ND | 50 |
| 1,1,2,2-Tetrachloroethane | ND | 5 |
| Tetrachloroethene | ND | 5 |
| 1,1,1-Trichloroethane | ND | 5 |
| 1,1,2-Trichloroethane | ND | 5 |
| Trichloroethene | ND | 5 |
| Trichlorofluoromethane | ND | 5 |
| Vinyl Chloride | ND | 50 |

D.L. = Detection Limit
ND = Not Detected

SCS ANALYTICAL LABORATORY
2860 WALNUT AVENUE
LONG BEACH, CALIFORNIA 90806
(213) 595-9324
FAX (213) 595-6709



Sample I.D.: B-9-15'
Date Received: 2/10/89
Date Analyzed: 2/24/89
Matrix: Soil
Project #: 1088011.00
File #: union1.rep

| Compound | Result ----ug/kg (ppb)---- | D.L. |
|---|---|---|
| Bromomethane | ND | 50 |
| Bromodichloromethane | ND | 5 |
| Bromoform | ND | 5 |
| Carbon Tetrachloride | ND | 5 |
| Chlorobenzene | ND | 5 |
| Chloroethane | ND | 50 |
| 2-Chloroethylvinyl Ether | ND | 50 |
| Chloroform | ND | 5 |
| Chloromethane | ND | 50 |
| Dibromochloromethane | ND | 5 |
| 1,1-Dichloroethane | ND | 5 |
| 1,2-Dichloroethane | ND | 5 |
| 1,1-Dichloroethene | 175 | 5 |
| trans-1,2-Dichloroethene | ND | 5 |
| 1,2-Dichloropropane | ND | 5 |
| cis-1,3-Dichloropropene | ND | 5 |
| trans-1,3-Dichloropropene | ND | 5 |
| Methylene Chloride | ND | 50 |
| 1,1,2,2-Tetrachloroethane | ND | 5 |
| Tetrachloroethene | ND | 5 |
| 1,1,1-Trichloroethane | ND | 5 |
| 1,1,2-Trichloroethane | ND | 5 |
| Trichloroethene | ND | 5 |
| Trichlorofluoromethane | ND | 5 |
| Vinyl Chloride | ND | 50 |

D.L. = Detection Limit
ND = Not Detected

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2860 WALNUT AVENUE
LONG BEACH, CALIFORNIA 90806
(213) 595-9324
FAX (213) 595-6709



Sample I.D.: B-9-20'
Date Received: 2/10/89
Date Analyzed: 2/24/89
Matrix: Soil
Project #: 1088011.00
File #: union1.rep

| Compound | Result ----ug/kg (ppb)---- | D.L. |
|---|---|---|
| Bromomethane | ND | 50 |
| Bromodichloromethane | ND | 5 |
| Bromoform | ND | 5 |
| Carbon Tetrachloride | ND | 5 |
| Chlorobenzene | ND | 5 |
| Chloroethane | ND | 50 |
| 2-Chloroethylvinyl Ether | ND | 50 |
| Chloroform | ND | 5 |
| Chloromethane | ND | 50 |
| Dibromochloromethane | ND | 5 |
| 1,1-Dichloroethane | ND | 5 |
| 1,2-Dichloroethane | ND | 5 |
| 1,1-Dichloroethene | 108 | 5 |
| trans-1,2-Dichloroethene | ND | 5 |
| 1,2-Dichloropropane | ND | 5 |
| cis-1,3-Dichloropropene | ND | 5 |
| trans-1,3-Dichloropropene | ND | 5 |
| Methylene Chloride | ND | 50 |
| 1,1,2,2-Tetrachloroethane | ND | 5 |
| Tetrachloroethene | ND | 5 |
| 1,1,1-Trichloroethane | ND | 5 |
| 1,1,2-Trichloroethane | ND | 5 |
| Trichloroethene | ND | 5 |
| Trichlorofluoromethane | ND | 5 |
| Vinyl Chloride | ND | 50 |

D.L. = Detection Limit
ND = Not Detected

SCS ANALYTICAL LABORATORY
2860 WALNUT AVENUE
LONG BEACH, CALIFORNIA 90806
(213) 595-9324
FAX (213) 595-6709



Sample I.D.: G-1
Date Received: 2/10/89
Date Analyzed: 2/24/89
Matrix: Soil
Project #: 1088011.00
File #: union1.rep

| Compound | Result | D.L. |
|---|---|---|
| | ----mg/kg (ppm)---- | |
| p,p'-DDE | 2.36 | 0.01 |
| Endosulfan I | ND | 0.02 |
| Aldrin | ND | 0.01 |
| Endosulfan II | ND | 0.02 |
| p,p'-DDT | 6.07 | 0.04 |
| Endrin Aldehyde | ND | 0.01 |
| Heptachlor Epoxide | ND | 0.01 |
| Endrin | ND | 0.01 |
| Dieldrin | ND | 0.01 |
| p,p'-DDD | 0.44 | 0.01 |
| Beta-BHC | ND | 0.01 |
| Delta-BHC | ND | 0.01 |
| Endosulfan Sulfate | ND | 0.02 |
| Heptachlor | ND | 0.01 |
| Alpha-BHC | ND | 0.01 |
| Lindane | ND | 0.01 |
| Toxaphene | ND | 2 |
| Chlordane | ND | 2 |
| Methoxychlor | ND | 1 |
| PCB-1016 | ND | 1 |
| PCB-1221 | ND | 1 |
| PCB-1232 | ND | 1 |
| PCB-1242 | ND | 1 |
| PCB-1248 | ND | 1 |
| PCB-1254 | ND | 1 |
| PCB-1260 | ND | 1 |

D.L. = Detection Limit
ND = Not Detected

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(213) 595-9324
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Sample I.D.: G-2
Date Received: 2/10/89
Date Analyzed: 2/24/89
Matrix: Soil
Project #: 1088011.00
File #: union1.rep

| Compound | Result ----mg/kg (ppm)---- | D.L. |
|---|---|---|
| p,p'-DDE | 1.54 | 0.01 |
| Endosulfan I | ND | 0.02 |
| Aldrin | ND | 0.01 |
| Endosulfan II | ND | 0.02 |
| p,p'-DDT | 0.82 | 0.04 |
| Endrin Aldehyde | ND | 0.01 |
| Heptachlor Epoxide | ND | 0.01 |
| Endrin | ND | 0.01 |
| Dieldrin | ND | 0.01 |
| p,p'-DDD | 0.06 | 0.01 |
| Beta-BHC | ND | 0.01 |
| Delta-BHC | ND | 0.01 |
| Endosulfan Sulfate | ND | 0.02 |
| Heptachlor | ND | 0.01 |
| Alpha-BHC | ND | 0.01 |
| Lindane | ND | 0.01 |
| Toxaphene | ND | 2 |
| Chlordane | ND | 2 |
| Methoxychlor | ND | 1 |
| PCB-1016 | ND | 1 |
| PCB-1221 | ND | 1 |
| PCB-1232 | ND | 1 |
| PCB-1242 | ND | 1 |
| PCB-1248 | ND | 1 |
| PCB-1254 | ND | 1 |
| PCB-1260 | ND | 1 |

D.L. = Detection Limit
ND = Not Detected

SCS ANALYTICAL LABORATORY
2860 WALNUT AVENUE
LONG BEACH, CALIFORNIA 90806
(213) 595-9324
FAX (213) 595-6709



Sample I.D.: G-3
Date Received: 2/10/89
Date Analyzed: 2/24/89
Matrix: Soil
Project #: 1088011.00
File #: union1.rep

| Compound | Result ----mg/kg (ppm)---- | D.L. ---- |
|---|---|---|
| p,p'-DDE | 0.76 | 0.01 |
| Endosulfan I | ND | 0.02 |
| Aldrin | ND | 0.01 |
| Endosulfan II | ND | 0.02 |
| p,p'-DDT | 0.40 | 0.04 |
| Endrin Aldehyde | ND | 0.01 |
| Heptachlor Epoxide | ND | 0.01 |
| Endrin | ND | 0.01 |
| Dieldrin | ND | 0.01 |
| p,p'-DDD | 0.01 | 0.01 |
| Beta-BHC | ND | 0.01 |
| Delta-BHC | ND | 0.01 |
| Endosulfan Sulfate | ND | 0.02 |
| Heptachlor | ND | 0.01 |
| Alpha-BHC | ND | 0.01 |
| Lindane | ND | 0.01 |
| Toxaphene | ND | 2 |
| Chlordane | ND | 2 |
| Methoxychlor | ND | 1 |
| PCB-1016 | ND | 1 |
| PCB-1221 | ND | 1 |
| PCB-1232 | ND | 1 |
| PCB-1242 | ND | 1 |
| PCB-1248 | ND | 1 |
| PCB-1254 | ND | 1 |
| PCB-1260 | ND | 1 |

D.L. = Detection Limit
ND = Not Detected

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SCS ANALYTICAL LABORATORY
2860 WALNUT AVENUE
LONG BEACH, CALIFORNIA 90806
(213) 595-9324
FAX (213) 595-6709



Sample I.D.: G-4
Date Received: 2/10/89
Date Analyzed: 2/24/89
Matrix: Soil
Project #: 1088011.00
File #: union1.rep

| Compound | Result | D.L. |
|---|---|---|
| | ----mg/kg (ppm)---- | |
| p,p'-DDE | 1.05 | 0.01 |
| Endosulfan I | ND | 0.02 |
| Aldrin | ND | 0.01 |
| Endosulfan II | ND | 0.02 |
| p,p'-DDT | 0.90 | 0.04 |
| Endrin Aldehyde | ND | 0.01 |
| Heptachlor Epoxide | ND | 0.01 |
| Endrin | ND | 0.01 |
| Dieldrin | ND | 0.01 |
| p,p'-DDD | 0.07 | 0.01 |
| Beta-BHC | ND | 0.01 |
| Delta-BHC | ND | 0.01 |
| Endosulfan Sulfate | ND | 0.02 |
| Heptachlor | ND | 0.01 |
| Alpha-BHC | ND | 0.01 |
| Lindane | ND | 0.01 |
| Toxaphene | ND | 2 |
| Chlordane | ND | 2 |
| Methoxychlor | ND | 1 |
| PCB-1016 | ND | 1 |
| PCB-1221 | ND | 1 |
| PCB-1232 | ND | 1 |
| PCB-1242 | ND | 1 |
| PCB-1248 | ND | 1 |
| PCB-1254 | ND | 1 |
| PCB-1260 | ND | 1 |

D.L. = Detection Limit
ND = Not Detected

SCS ANALYTICAL LABORATORY
2860 WALNUT AVENUE
LONG BEACH, CALIFORNIA 90806
(213) 595-9324
FAX (213) 595-6709



Sample I.D.: G-5
Date Received: 2/10/89
Date Analyzed: 2/24/89
Matrix: Soil
Project #: 1088011.00
File #: union1.rep

| Compound | Result ----mg/kg (ppm)---- | D.L. |
|---|---|---|
| p,p'-DDE | 1.55 | 0.01 |
| Endosulfan I | ND | 0.02 |
| Aldrin | ND | 0.01 |
| Endosulfan II | ND | 0.02 |
| p,p'-DDT | 2.27 | 0.04 |
| Endrin Aldehyde | ND | 0.01 |
| Heptachlor Epoxide | ND | 0.01 |
| Endrin | ND | 0.01 |
| Dieldrin | ND | 0.01 |
| p,p'-DDD | 0.03 | 0.01 |
| Beta-BHC | ND | 0.01 |
| Delta-BHC | ND | 0.01 |
| Endosulfan Sulfate | ND | 0.02 |
| Heptachlor | ND | 0.01 |
| Alpha-BHC | ND | 0.01 |
| Lindane | ND | 0.01 |
| Toxaphene | ND | 2 |
| Chlordane | ND | 2 |
| Methoxychlor | ND | 1 |
| PCB-1016 | ND | 1 |
| PCB-1221 | ND | 1 |
| PCB-1232 | ND | 1 |
| PCB-1242 | ND | 1 |
| PCB-1248 | ND | 1 |
| PCB-1254 | ND | 1 |
| PCB-1260 | ND | 1 |

D.L. = Detection Limit
ND = Not Detected

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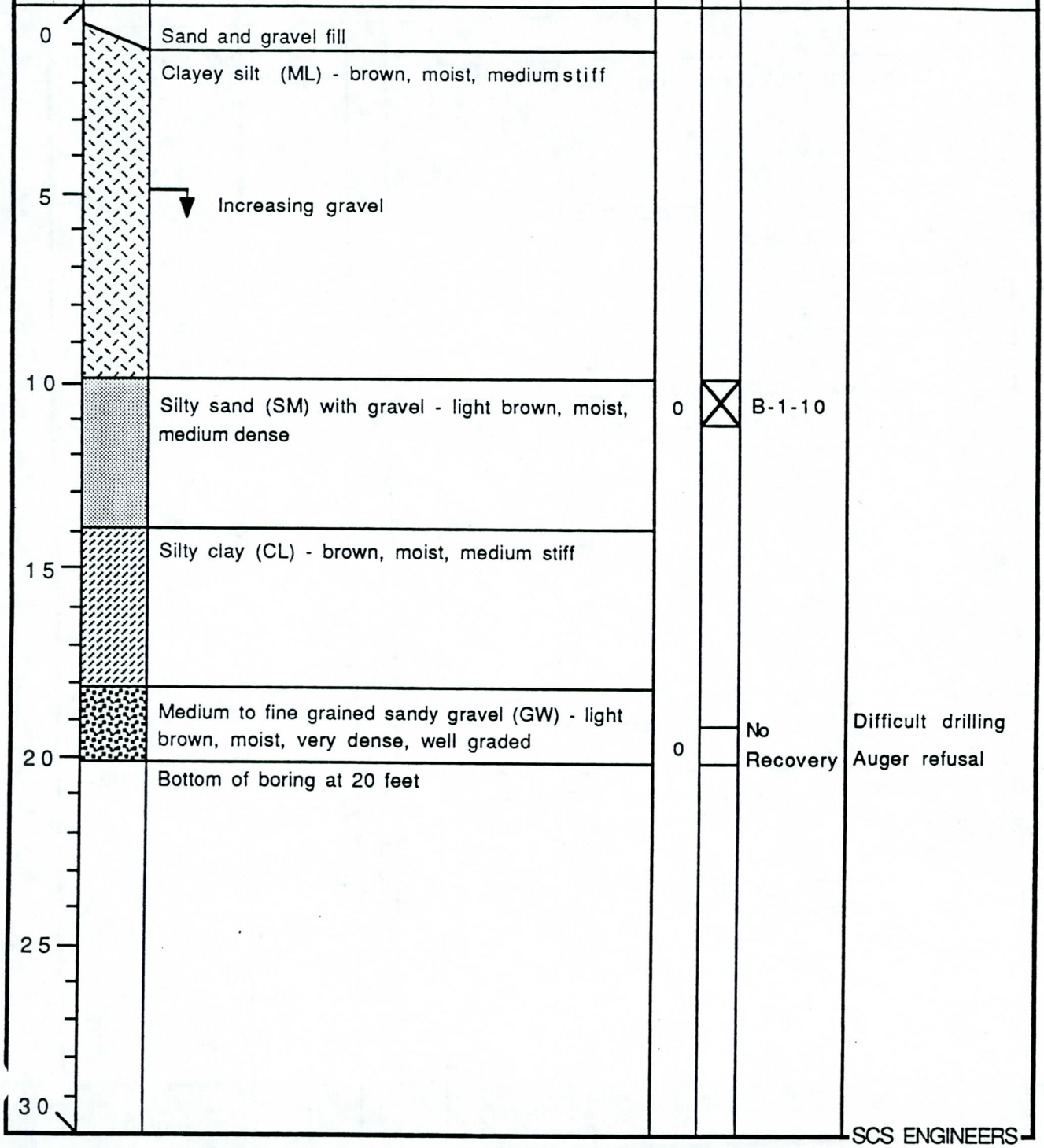
# APPENDIX B

# BORING LOGS

| Project Name: El Mirage (Plant No. 6) | | Log of Boring No.: B-1 |
|---|---|---|
| Date Started: 2-8-89 | WL Depth: Dry | Drilling Equipment: CME 45 |
| Date Completed: 2-8-89 | Total Depth: 20 feet | Logged/Checked by: C.M./A.G. |

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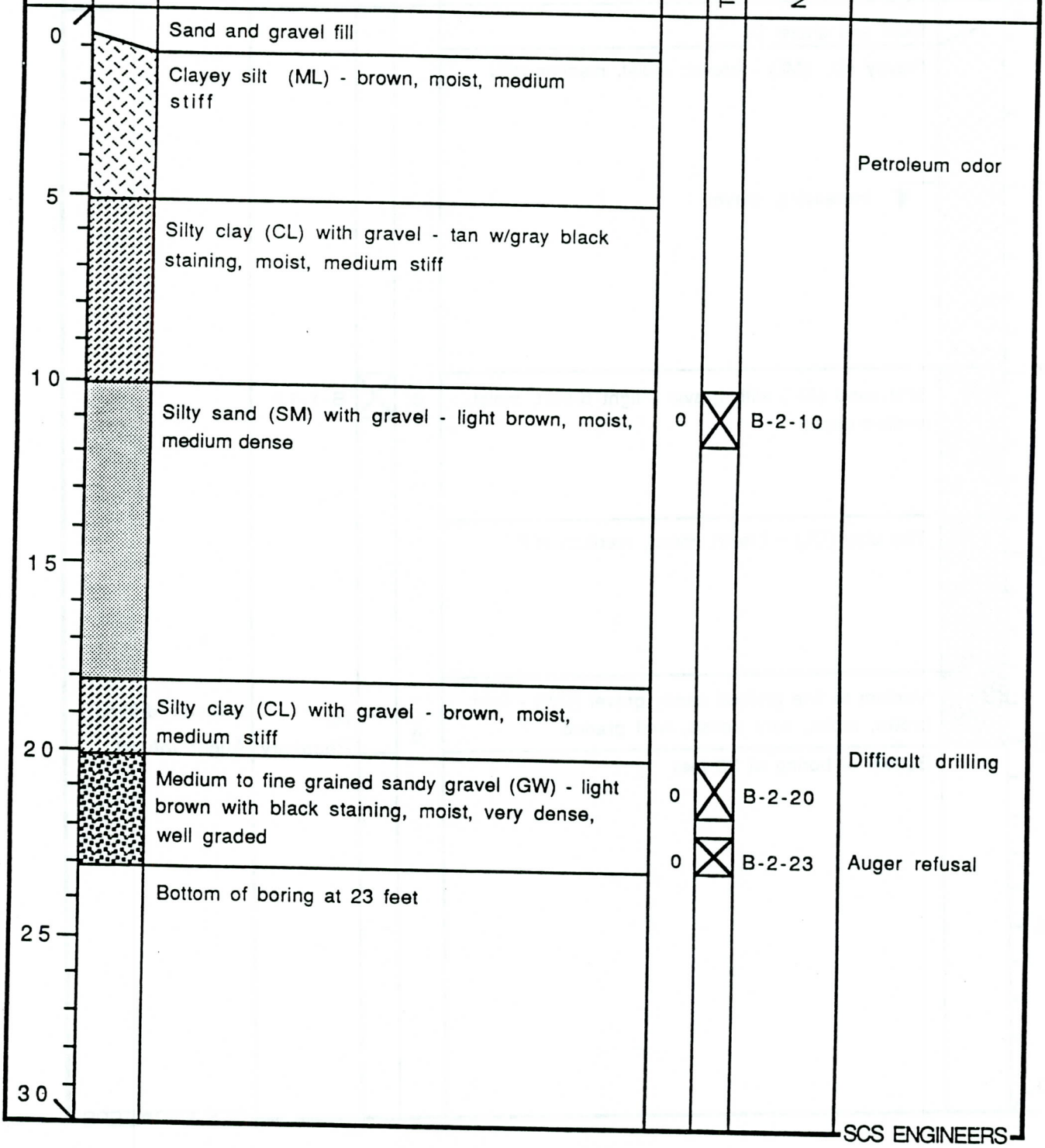
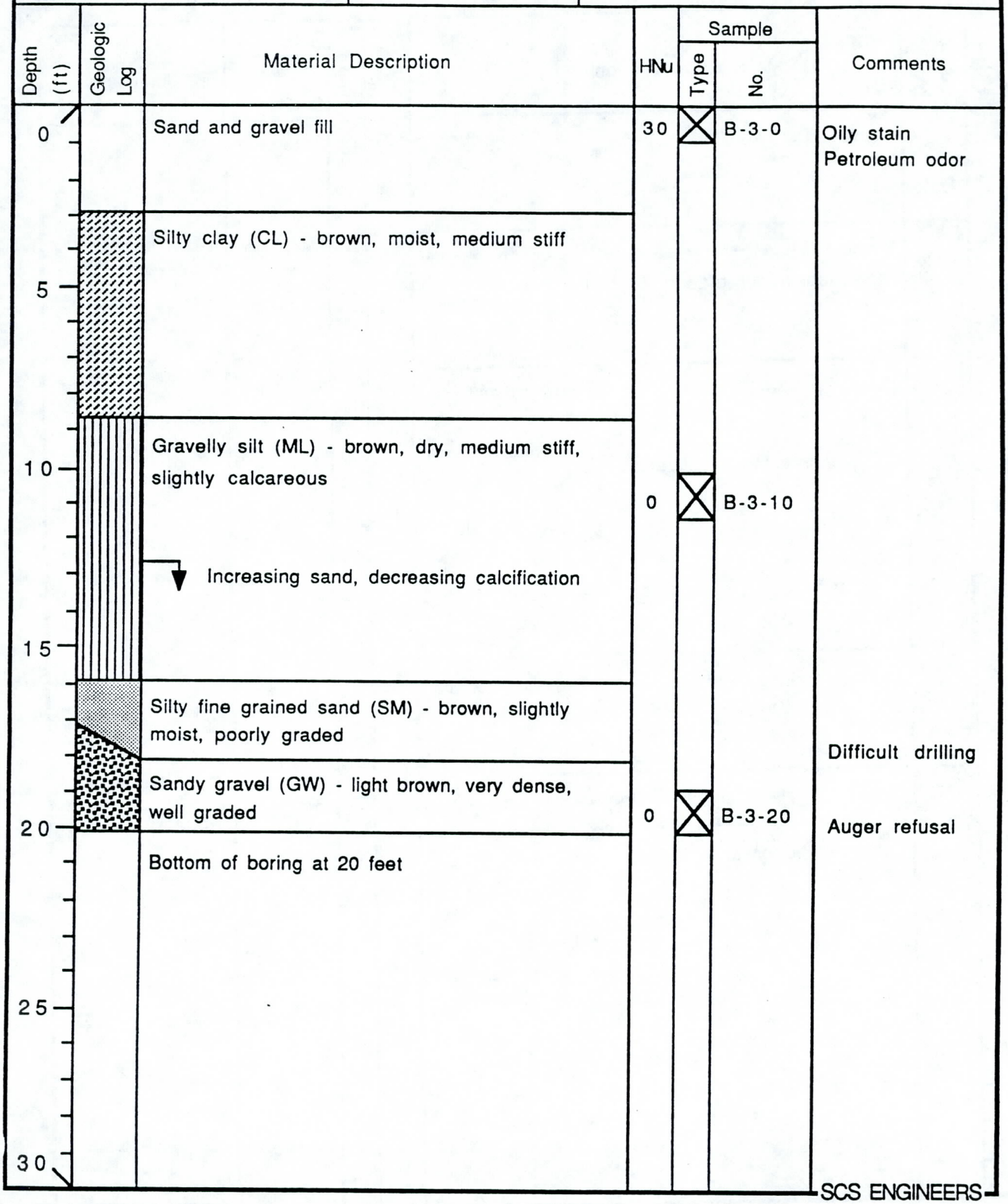
| Project Name: El Mirage (Plant No. 6) | | Log of Boring No.: B-2 |
|---|---|---|
| Date Started: 2-8-89 | WL Depth: Dry | Drilling Equipment: CME 45 |
| Date Completed: 2-8-89 | Total Depth: 23 feet | Logged/Checked by: C.M./A.G. |



| Project Name: El Mirage (Plant No. 6) | | Log of Boring No.: B-3 |
|---|---|---|
| Date Started: 2-8-89 | WL Depth: Dry | Drilling Equipment: CME 45 |
| Date Completed: 2-8-89 | Total Depth: 20 feet | Logged/Checked by: C.M./A.G. |

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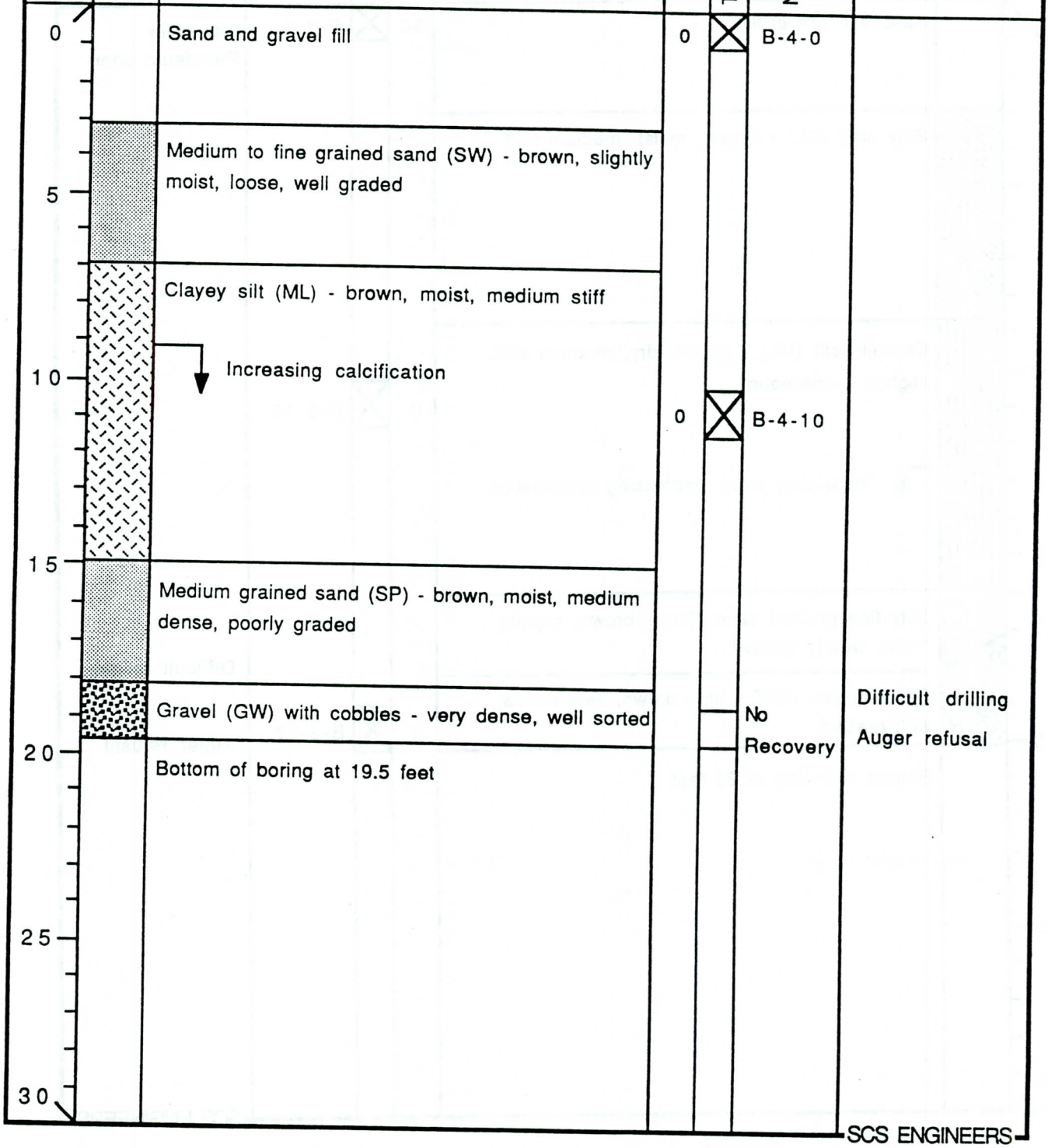
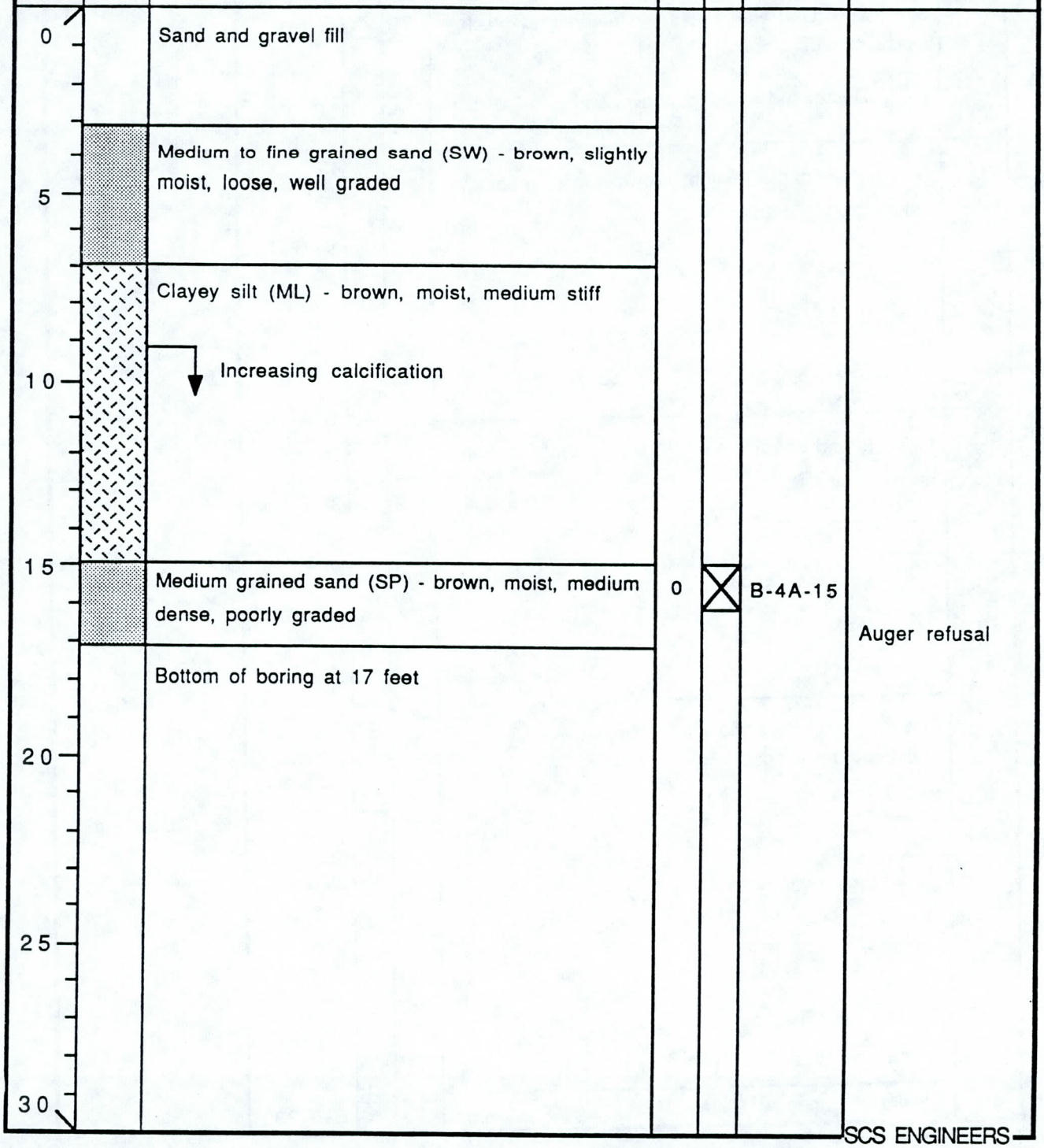
| Project Name: El Mirage (Plant No. 6) | | Log of Boring No.: B-4 |
|---|---|---|
| Date Started: 2-8-89 | WL Depth: Dry | Drilling Equipment: CME 45 |
| Date Completed: 2-8-89 | Total Depth: 19.5 feet | Logged/Checked by: C.M./A.G. |



| Project Name: El Mirage (Plant No. 6) | | Log of Boring No.: B-4A |
|---|---|---|
| Date Started: 2-8-89 | WL Depth: Dry | Drilling Equipment: CME 45 |
| Date Completed: 2-8-89 | Total Depth: 17 feet | Logged/Checked by: C.M./A.G. |

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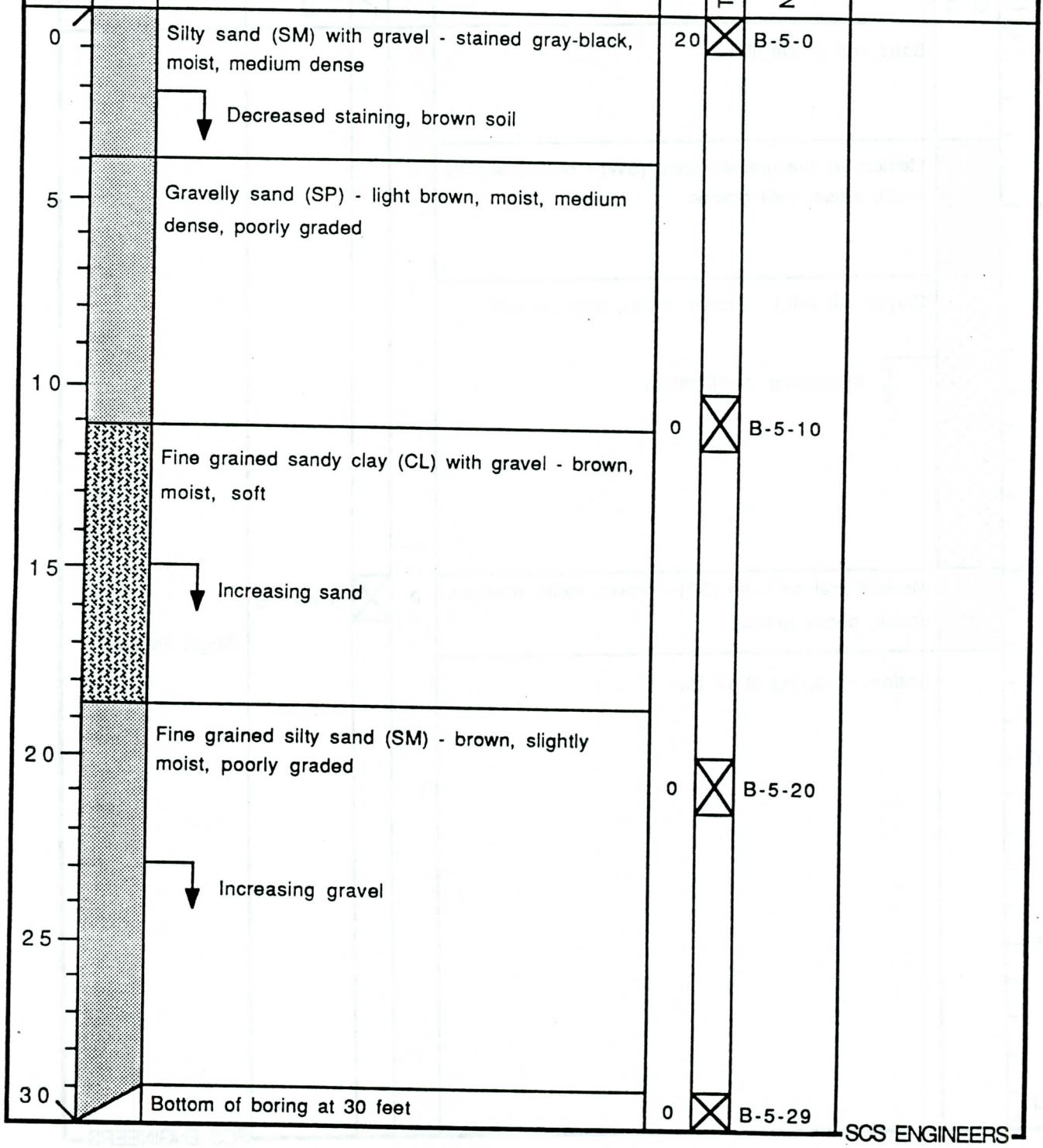
| Project Name: El Mirage (Plant No. 6) | | Log of Boring No.: B-5 |
|---|---|---|
| Date Started: 2-8-89 | WL Depth: Dry | Drilling Equipment: CME 45 |
| Date Completed: 2-8-89 | Total Depth: 30 feet | Logged/Checked by: C.M./A.G. |

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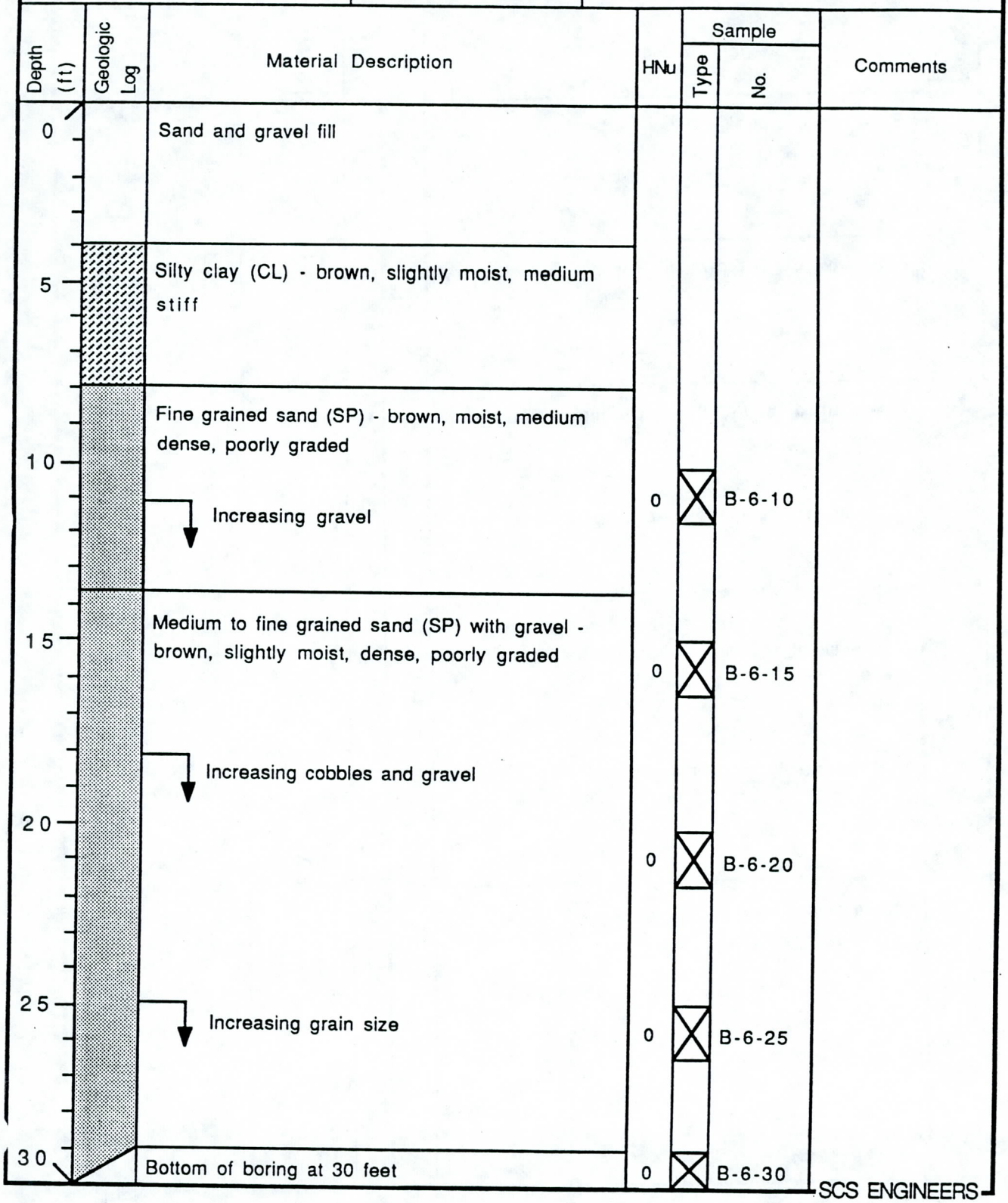
| Project Name: El Mirage (Plant No. 6) | | Log of Boring No.: B-6 |
|---|---|---|
| Date Started: 2-8-89 | WL Depth: Dry | Drilling Equipment: CME 45 |
| Date Completed: 2-8-89 | Total Depth: 30 feet | Logged/Checked by: C.M./A.G. |

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| Project Name: El Mirage (Plant No. 6) | | Log of Boring No.: B-7 |
|---|---|---|
| Date Started: 2-9-89 | WL Depth: Dry | Drilling Equipment: CME 45 |
| Date Completed: 2-9-89 | Total Depth: 21.5 feet | Logged/Checked by: C.M./A.G. |

| Depth (ft) | Geologic Log | Material Description | HNu | Sample Type | Sample No. | Comments |
|---|---|---|---|---|---|---|
| 0 | | Sand and gravel fill | | | | |
| 5 | | Silty clay (CL) - brown, slightly moist, medium stiff | | | | |
| 10 | | Fine grained sand (SP) - brown, moist, medium dense, poorly graded<br>Increasing gravel | 0 | X | B-7-10 | |
| 15 | | Medium to fine grained sand (SP) with gravel - brown, slightly moist, medium dense, poorly graded<br>Increasing cobbles and gravel | 0 | X | B-7-15 | |
| 20 | | | 0 | X | B-7-20 | Auger refusal |
| 25 | | Bottom of boring at 21.5 feet | | | | |
| 30 | | | | | | |

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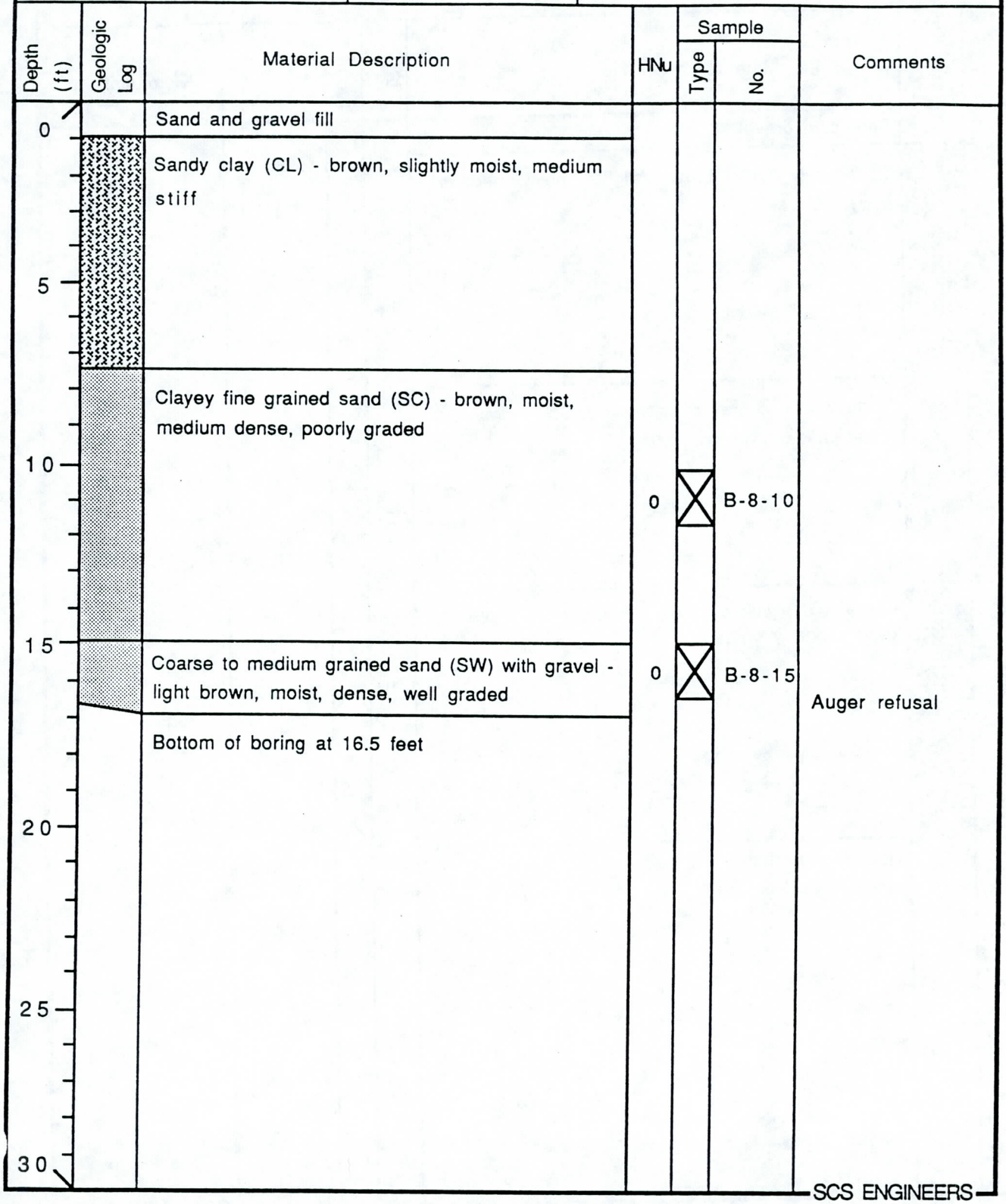
| Project Name: El Mirage (Plant No. 6) | | Log of Boring No.: B-8 |
|---|---|---|
| Date Started: 2-9-89 | WL Depth: Dry | Drilling Equipment: CME 45 |
| Date Completed: 2-9-89 | Total Depth: 16.5 feet | Logged/Checked by: C.M./A.G. |

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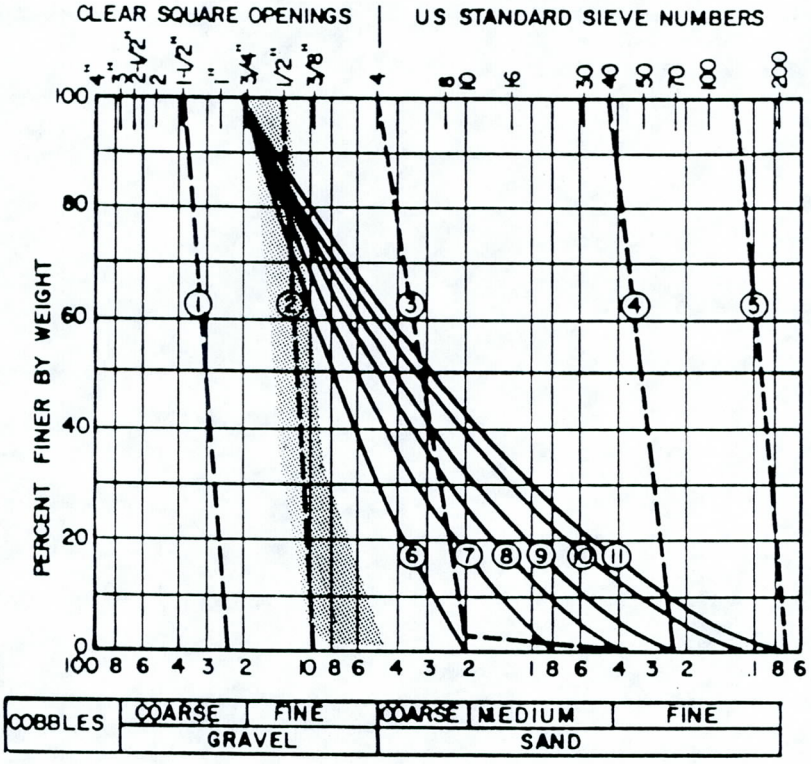
| Project Name: El Mirage (Plant No. 6) | | Log of Boring No.: B-9 |
|---|---|---|
| Date Started: 2-9-89 | WL Depth: Dry | Drilling Equipment: CME 45 |
| Date Completed: 2-9-89 | Total Depth: 20 feet | Logged/Checked by: C.M./A.G. |

| Depth (ft) | Geologic Log | Material Description | HNu | Sample Type | Sample No. | Comments |
|---|---|---|---|---|---|---|
| 0 | | Sand and gravel fill | | | | |
| | | Sandy clay (CL) - brown, slightly moist, medium stiff | | | | |
| 5 | | | | | | |
| | | Medium to fine grained silty sand (SM) - brown, moist, medium dense | | | | |
| 10 | | | 0 | X | B-9-10 | |
| | | Gravel (GP) with cobbles - light brown, very dense, poorly graded | | | | |
| 15 | | | 0 | X | B-9-15 | |
| | | | 0 | X | B-9-20 | Auger refusal |
| 20 | | Bottom of boring at 20 feet | | | | |
| 25 | | | | | | |
| 30 | | | | | | |

SCS ENGINEERS

Attachment 5

PERMEABILITY OF DRAINAGE MATERIALS




COEFFICIENT OF PERMEABILITY FOR CLEAN COARSE-GRAINED DRAINAGE MATERIAL

| CURVE | K, FT/MIN. |
|---|---|
| (1) | 73.7 |
| (2) | 56.9 |
| (3) | 5.41 |
| (4) | 0.13 |
| (5) | 0.01 |
| (6) | 2.08 |
| (7) | 1.81 |
| (8) | 0.70 |
| (9) | 0.22 |
| (10) | 0.08 |
| (11) | 0.01 |

NOTE: Shaded area shows materials currently produced by the El Mirage plant of Union Rock and Materials Corp.

*SOURCE: Dept. of the Navy, Naval Facilities Engineering Command: Soil Mechanics, Design Manual 7.1, May 1982*

EMCON Associates

BROWNING-FERRIS INDUSTRIES OF ARIZONA, INC.
CHOLLA SANITARY LANDFILL
EL MIRAGE, ARIZONA

PERMEABILITY OF DRAINAGE MATERIALS

ATTACHMENT 5

PROJECT NO. 372-15.01

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Attachment 6

LEACHATE PIPE SPACING CALCULATIONS

# COMPUTATION SHEET

PROJECT TITLE: CHOLLA PROJECT NO. 372-15.01

DESCRIPTION: LEACHATE COLLECTION SYSTEM DESIGN SHEET 2 OF 2

PREP. BY: AE DATE: 4/25/90 CHKD BY: R. Leach DATE: 4/25/90

A HELP-II ANALYSIS WAS RUN FOR THE OPERATIONAL PHASE OF THE LANDFILL RESULTING IN A PERCOLATION RATE THROUGH THE FIRST 5 FOOT THICK LIFT OF REFUSE OF 0.0128 in/YR. ✓

FOR DESIGN PURPOSES, ASSUME THE PERCOLATION RATE IS TWICE THIS VALUE.

$$q = 2(0.0128 \text{ in/YR}) = 0.0256 \text{ in/YR.}$$ ✓

CONVERTING TO FT/SEC,

$$q = (0.0256 \text{ in/YR})\left(\frac{\text{FT}}{12 \text{ in}}\right)\left(\frac{1 \text{ YR}}{365 \text{ d}}\right)\left(\frac{1 \text{ d}}{24 \text{ hR}}\right)\left(\frac{1 \text{ hR}}{3600 \text{ sec}}\right)$$ ✓

$$q = 6.76 \times 10^{-11} \text{ FT/sec.}$$ ✓

PERMEABILITY OF THE DRAINAGE LAYER IS

$$K_1 = 1 \text{ cm/sec}$$ ✓

CONVERTING TO FT/SEC,

$$K_1 = 1 \text{ cm/sec}\left(\frac{\text{in}}{2.54 \text{ cm}}\right)\left(\frac{\text{FT}}{12 \text{ in}}\right)$$

$$K_1 = 0.033 \text{ FT/SEC}$$ ✓

TRY NOMINAL PIPE SPACING OF L = 400 FT AND DETERMINE MAXIMUM HEAD, $h_{max}$:

$$h_{max} = \frac{L}{2}\sqrt{\frac{q}{K_1}} = \frac{400 \text{ FT}}{2}\sqrt{\frac{6.76 \times 10^{-11} \text{ FT/sec}}{0.033 \text{ FT/sec}}}$$ ✓

$$h_{max} = 0.009 \text{ FT} \leq 1 \text{ FOOT} \quad \text{OK.}$$ ✓

THE RESULT SHOWS 400 FT SPACING OF LEACHATE COLLECTION PIPES IS CONSERVATIVE.

# COMPUTATION SHEET

PROJECT TITLE: CHOLLA PROJECT NO. 372-15.01

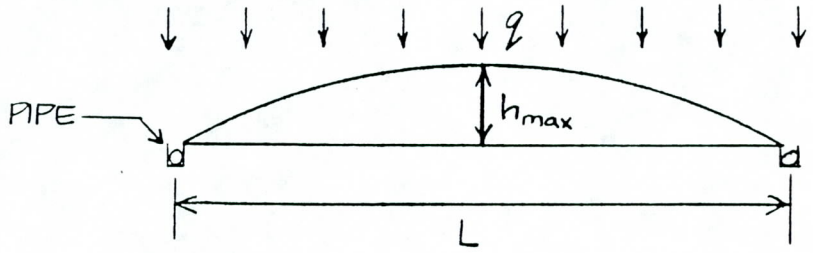
DESCRIPTION: LEACHATE COLLECTION SYSTEM DESIGN SHEET 1 OF 2

PREP. BY: JE DATE: 4/25/96 CHKD BY: R Leach DATE: 4/25/96

OBJECTIVE: DETERMINE LEACHATE COLLECTION PIPE SPACING FOR LEACHATE COLLECTION AND REMOVAL SYSTEM.

DESIGN CRITERIA: THE MAXIMUM HEAD BUILD UP OVER THE LINER MUST BE LESS THAN OR EQUAL TO 1 FOOT.

METHOD: "MODELING FOR DESIGN OF LANDFILL BOTTOM LINERS" BY ALEXANDER C. DEMETRACOPOULOUS, GEORGE P. KORFIATIS, et. al. ASCE JOURNAL OF ENVIRONMENTAL ENGINEERING VOL 110, No 6, DECEMBER 1984.




GOVERNING EQUATION ✓

$$h_{max} = \frac{L}{2}\sqrt{\frac{q}{K_1}}$$

WHERE: $h_{max}$ = MAXIMUM HEAD BUILD UP ON LINER ≤ 1 FT.

$L$ = COLLECTION PIPE SPACING.

$q$ = STEADY STATE RATE OF PERCOLATION TO THE DRAINAGE LAYER.

$K_1$ = HYDRAULIC CONDUCTIVITY OF THE DRAINAGE LAYER.

ASSUMPTION: THE LANDFILL BOTTOM IS MODELED AS BEING FLAT. ✓ THIS ASSUMPTION SIMPLIFIES THE ANALYSIS AND RESULTS IN A HIGHER, MORE CONSERVATIVE $h_{max}$ VALUE THAN FOR THE 1.4 % SLOPE SHOWN ON THE DESIGN DRAWINGS.

Attachment 7

# UNSATURATED GROUND-WATER FLOW MODELING FOR PROPOSED CHOLLA LANDFILL - PART 2

# Unsaturated Ground-water Flow Modeling for Proposed Cholla Landfill - Part 2


Work Performed By:

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Dr. Michael J. Sully

Gordon Wittmeyer

Department of Hydrology and Water Resources

University of Arizona

Tucson, Arizona 85721


October, 1989

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3 Equivalency of Model and Laboratory Material Properties

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6 Moisture Retention Curve for Sample SHC-9-59.5

7 Moisture Retention Curve for Sample SHC-9-69.5

8 Moisture Retention Curve for Sample SHC-9-103.0

9 Moisture Retention Curve for Sample SHC-22-23.5

10 Moisture Retention Curve for Sample SHC-22-52.5

11 Moisture Retention Curve for Sample SHC-22-54.0

12 Moisture Retention Curve for Sample SHC-22-59.0

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20 K-Suction Comparison -- Section 7, Material 4, SC with SHC-9-30.0

21 K-Suction Comparison -- Section 7, Material 5, SW with SHC-22-23.5

22 K-Suction Comparison -- Section 7, Material 8, CL with SHC-9-49.5 and SHC-9-103.0

23 K-Suction Comparison -- Section 8, Material 2, SM with SHC-5-79.5

24 K-Suction Comparison -- Section 8, Material 3, CL with SHC-22-77.5

25 K-Suction Comparison -- Section 8, Material 4, SM-SC with SHC-5-79.5

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## 1.0 Introduction

As specified in Section 5.4 of Unsaturated Ground-Water Flow Modeling of Proposed Cholla Landfill, (Maddock et al., 1989), saturated hydraulic conductivity and moisture retention data on fourteen undisturbed cores taken from boreholes SHC-5 and SHC-22 near Section 8 and from borehole SHC-9 near Section 7 were determined by Daniel B. Stephens and Associates (Appendix I). The purpose of this report is to compare the hydraulic properties obtained from the laboratory analysis with the properties used in flow modeling described in Maddock et al (1989) and to determine whether or not the estimated hydraulic properties used for the model adequately represented the hyraulic properties of the site. Due to the absence of samples near Section 9 (EMCON and Assoc., 1989), no comparisons were made with the values used for Section 9 in the model.

## 2.0 Soil Hydraulic Properties

Estimates of van Genuchten parameters and the unsaturated hydraulic conductivity - suction relations were determined from moisture retention data obtained from samples taken at 14 locations.

### 2.1 Estimation of van Genuchten Parameters

The relative hydraulic conductivity is given by:

$$K_r = \frac{K(h)}{K_s} \qquad \text{with } 0 \leq K_r \leq 1 \tag{1}$$

where K(h) is hydraulic conductivity as a function of pressure head. The dependence of $K_r$ on water content can be measured directly or estimated by using the closed-form expression for $K_r$ of van Genuchten (1980):

$$\Theta = \begin{cases} \dfrac{1}{[1 + |\alpha h|^n]^{1-1/n}} & h < 0 \\ 1 & h \geq 0 \end{cases} \tag{2}$$



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$$K_r = \Theta^{\frac{1}{2}}\left[1 - \left(1 - \Theta^{\frac{n}{n-1}}\right)^{1-\frac{1}{n}}\right]^2 \quad (3)$$

and

$$K(h) = K_r K_s \quad (4)$$

where;

$\Theta$ = reduced water content, $\Theta = \dfrac{(\theta - \theta_r)}{(\theta_s - \theta_r)}$

and where;

$K_s$ = saturated hydraulic conductivity,

h = pressure head,

$\theta$ = volumetric water content,

$\theta_r$ = residual volumetric water content,

$\theta_s$ = saturated volumetric water content,

$n$ = fitting parameter inversely related to pore size distribution, and

$\alpha$ = fitting parameter inversely related to air-entry tension.

The volumetric water content, $\theta$, is the volume of water per bulk volume of porous material. The parameters $\theta_r$, $\alpha$ and $n$ were estimated by fitting the van Genuchten (van Genuchten, 1980) model to the moisture retention data by nonlinear least squares. This procedure yields an analytical expression for the relationship between hydraulic conductivity and moisture content required by UNSAT2. Residual volumetric water content ($\theta_r$) was estimated by extrapolating the laboratory information to high suction values. A standard Marquardt-Levenberg algorithm (Press et al., 1986) was used to solve the non-linear least squares problem. Numerical algorithms for obtaining these parameters from moisture retention data are readily available (i.e., van Genuchten, 1980). The

parameter values are shown in Table 1. The fitted moisture retention curves for the 14 samples are shown in Figures 1-14 along with the experimental suction-volumetric water content data values. Suction is defined as minus pressure head , $-h$.

## 2.2 Unsaturated Hydraulic Conductivity Relations

Values of saturated hydraulic conductivity, $K_s$, and saturated volumetric water content, $\theta_s$, determined from the laboratory procedures are listed in Table 1. Using equation (3), an estimated hydraulic conductivity as a function of suction was determined for each of the samples. The curves of these functions are shown in Figs 15-17.

## 3.0 Comparison of Model Hydraulic Properties with Laboratory Results

For the purpose of modeling flow, the subsurface profile was divided into layers estimated from geologic cross-sections provided by EMCON and Associates (Maddock, et al., 1989). The USCS classification of material types provided by Stephens (1989) at sample locations listed in Table 2 were compared with the material types and locations of layers used in the model. Since samples were not obtained from each material used in the model, comparisons can be made only for those samples which correspond to the depths and material types used in the model. Table 3 shows the correspondence between the material properties used for the layers in each section and the laboratory results based on the samples.

## 3.1 Comparison of Sample Hydraulic Properties to Hydraulic Properties Used in the Model

1. Section 7, Material 1 compared with sample SHC-9-34.5: The saturated hydraulic conductivity used in the model is less than the value of $K_s$ measured in the laboratory, (0.283 ft./d vs 0.81 ft./d). The hydraulic conductivity used in the model was less than that measured over all suctions, however, the two curves are quite similar in shape (Figure 18).



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2. Section 7, Material 3 compared with samples SHC-9-49.5 and SHC-9-103.0: Parameter values used in modeling result in the K-Suction relation having a higher value of conductivity at all suctions than either of the K-Suction relations based on the laboratory data ( Figure 19).

3. Section 7, Material 4 compared with sample SHC-9-39.0: The saturated hydraulic conductivity used in the model is a factor of 2 smaller than the value of $K_s$ measured by the laboratory ( 0.045 ft/d vs 0.0961 ft/d). Unsaturated hydraulic conductivity used for the model was less than the unsaturated K obtained from the sample for suction less than 10 ft. However, over the entire range of suction the K-Suction relations are remarkably similar (Figure 20).

4. Section 7, Material 5 compared with sample SHC-22-23.5: The saturated hydraulic conductivity used in the model is a factor of 2 smaller than the value of $K_s$ measured in the laboratory (2.72 ft/d vs 5.5 ft/d). The unsaturated hydraulic conductivity used for the model was less than that determined from the sample for suction less than 0.1 ft and greater than that determined from the sample for suction greater than 0.1 ft (Figure 21).

5. Section 7, Material 8 compared with samples SHC-9-49.5 and SHC-9-103.0: Parameter values used in modeling result in the K-Suction relation having a higher value of conductivity at all suctions than either of the K-Suction relations based on the laboratory data ( Figure 22).

6. Section 8, Material 2 compared with sample SHC-5-79.5: The saturated hydraulic conductivity used in the model is greater than the value of $K_s$ measured in the laboratory (2.72 ft/d vs 0.657 ft/d). Hydraulic conductivity of the model exceeded that of the sample over the entire range of suction. However, the shapes of the curves are similar (Figure 23).

7. Section 8, Material 3 compared with sample SHC-22-77.5: The saturated hydraulic conductivity used in the model is much greater than the value of $K_s$ measured in the laboratory (0.49 ft/d vs 0.00731 ft/d). The unsaturated hydraulic conductivity used in the model is greater than that determined from the sample for suction less than 30 ft (Figure 24).

8. Section 8, Material 4 compared with sample SHC-5-79.5: The saturated hydraulic conductivity used in the model is greater than the value of $K_s$ measured in the laboratory (3.69 ft/d vs .657 ft/d). The hydraulic conductivity used in the model exceeded the measured K over the entire range of suctions (Figure 25).

## 4.0 Conclusions and Recommendations

The simulation for Section 7 showed that the movement of the wetting front was confined to Material 1. From item 1 of 3.1 above, the saturated hydraulic conductivity measured in the laboratory was approximately 2.5 times that measured *in situ* by a falling head test. As can be seen from Figure 18, however, the K-Suction relations show good agreement. For a suction corresponding to the effective pore pressure at the wetting front, Figure 18 shows the hydraulic conductivities to be the same. For Materials 4 and 5, although the values of saturated hydraulic conductivity used in the model were smaller than the measured values by a factor of two, the wetting front never reached these materials during the simulation. Estimates of saturated hydraulic conductivity for Materials 3 and 8 were conservative, although these data were not necessary for the simulated time period.

For Section 8, the laboratory results indicate that the values of saturated hydraulic conductivities used in the model were conservative.

On the basis of these comparisons, the results of the simulation indicate that the wetting front would not reach the landfill liner during the course of the modeled flood.



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## 5.0 References


Maddock, Thomas III, M. J. Sully, A. Guzman, G. Wittmeyer, and E. Paleologos, *Unsaturated Ground-water Flow Modeling for Proposed Cholla Landfill,* Dept of Hydrology and Water Resources, University of Arizona, 1989.

Press, William H., Brian P. Flannery, Saul A. Teukolsky, and William T. Vetterling, *Numerical Recipes, the Art of Scientific Computing*, Cambridge University Press, 1986.

Stephens, Daniel B., *Laboratory Analysis of Hydraulic Properties for the Browning and Ferris Industries Proposed Cholla Landfill Site,* Daniel B. Stephens & Associates, Inc., Albuquerque, N.M., 1989.

van Genuchten, M. Th; *A Closed-form Equation for Predicting the Hydraulic Conductivity of Unsaturated Soils*, Soil Science Society of America Journal, vol. 44, pp. 892-898, 1980.

**Figure 1**
Moisture Retention Curve for Sample SHC-5-79.5

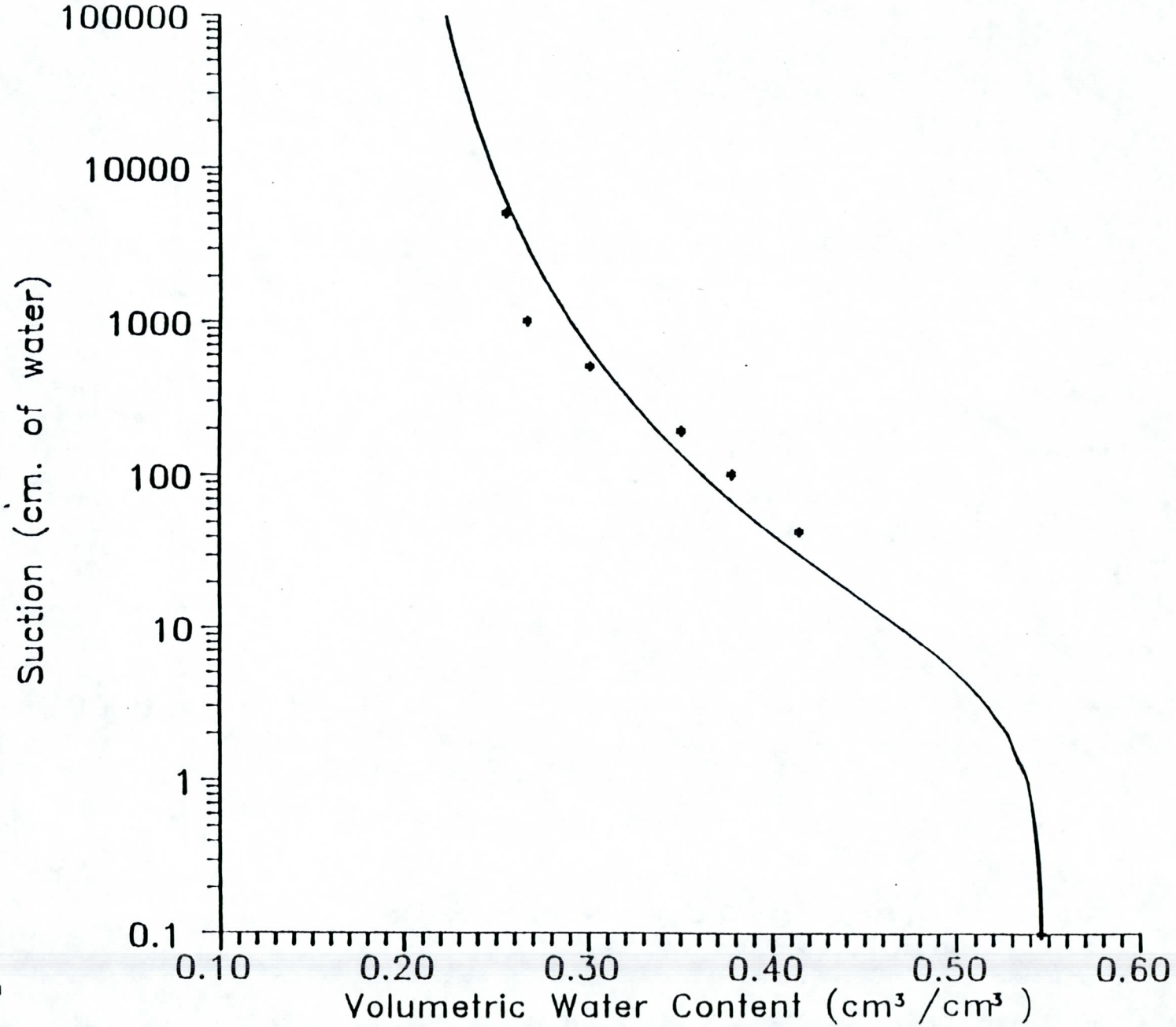


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**Figure 2**
Moisture Retention Curve for Sample SHC-9-30.5

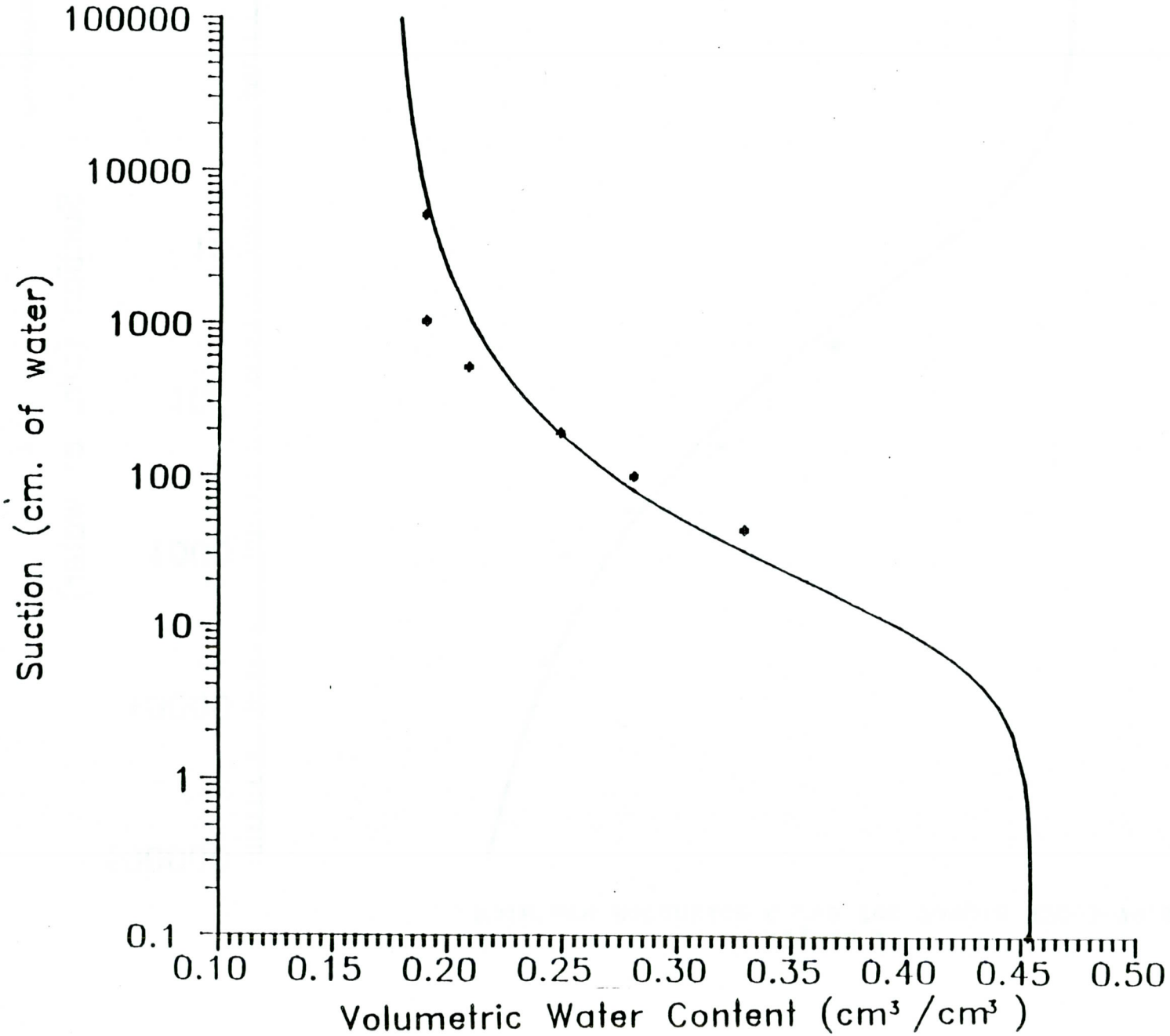

**Figure 3**
**Moisture Retention Curve for Sample SHC-9-34.5**

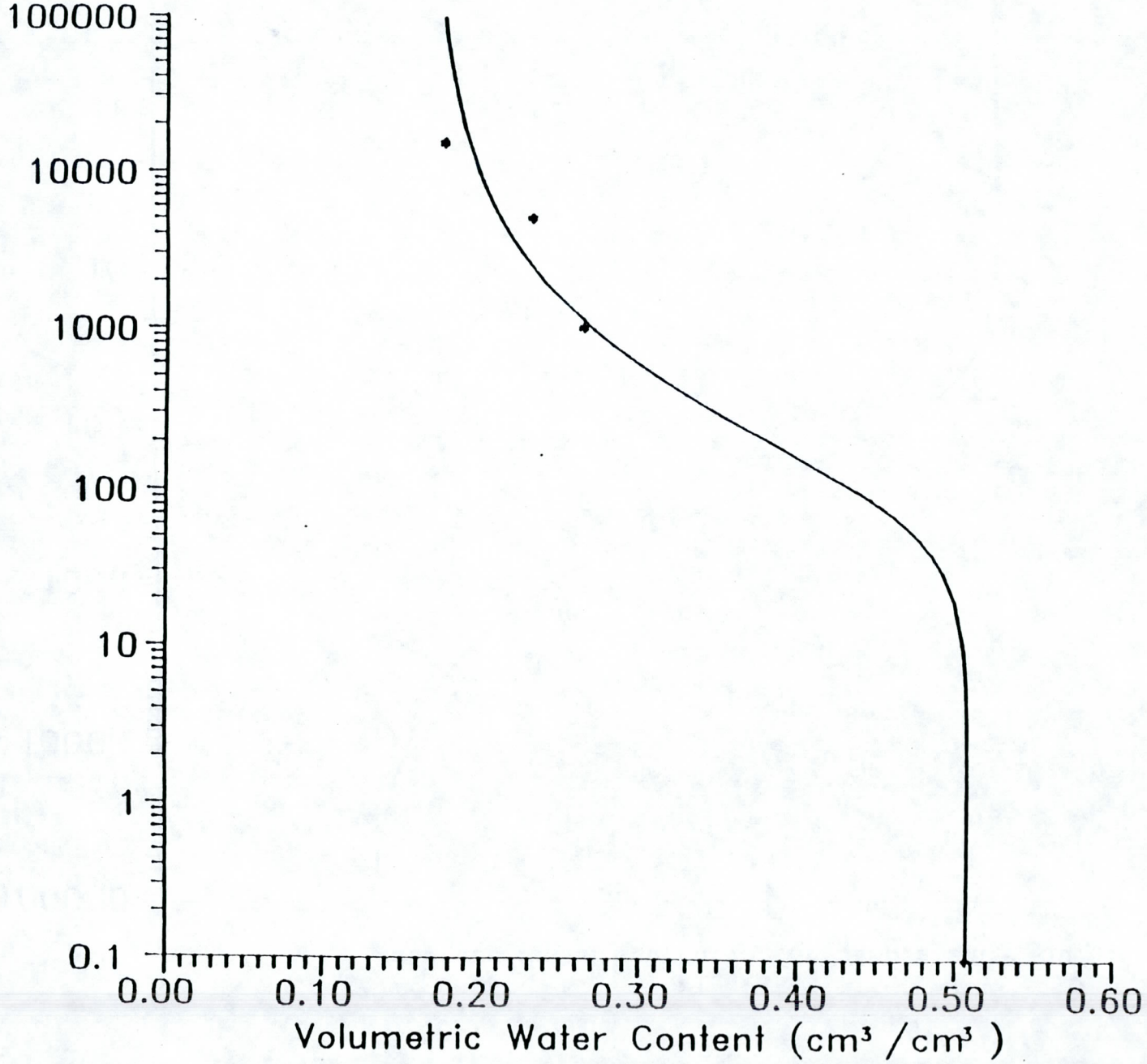




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**Figure 4**
Moisture Retention Curve for Sample SHC-9-39.0

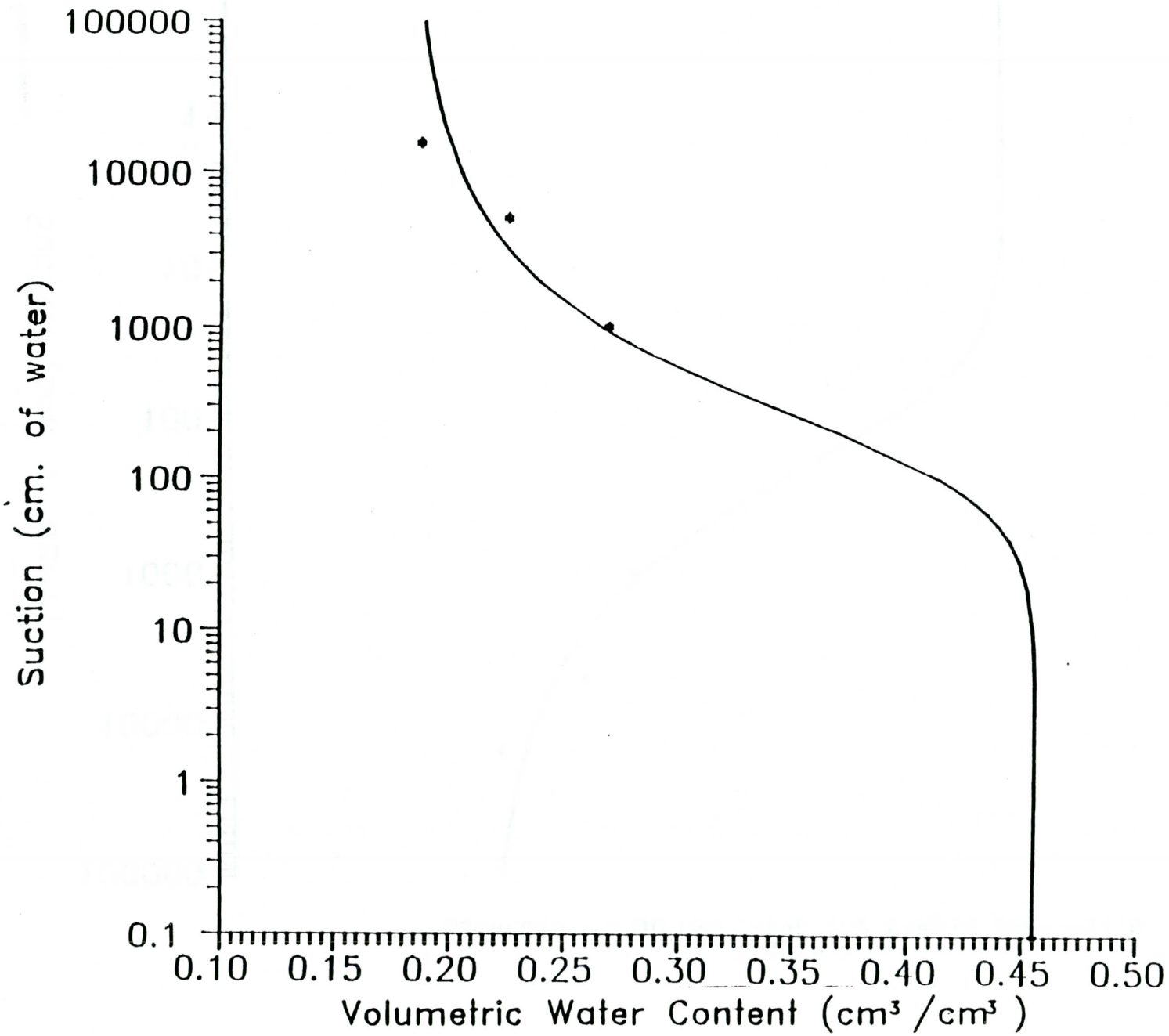

**Figure 5**
**Moisture Retention Curve for Sample SHC-9-49.5**

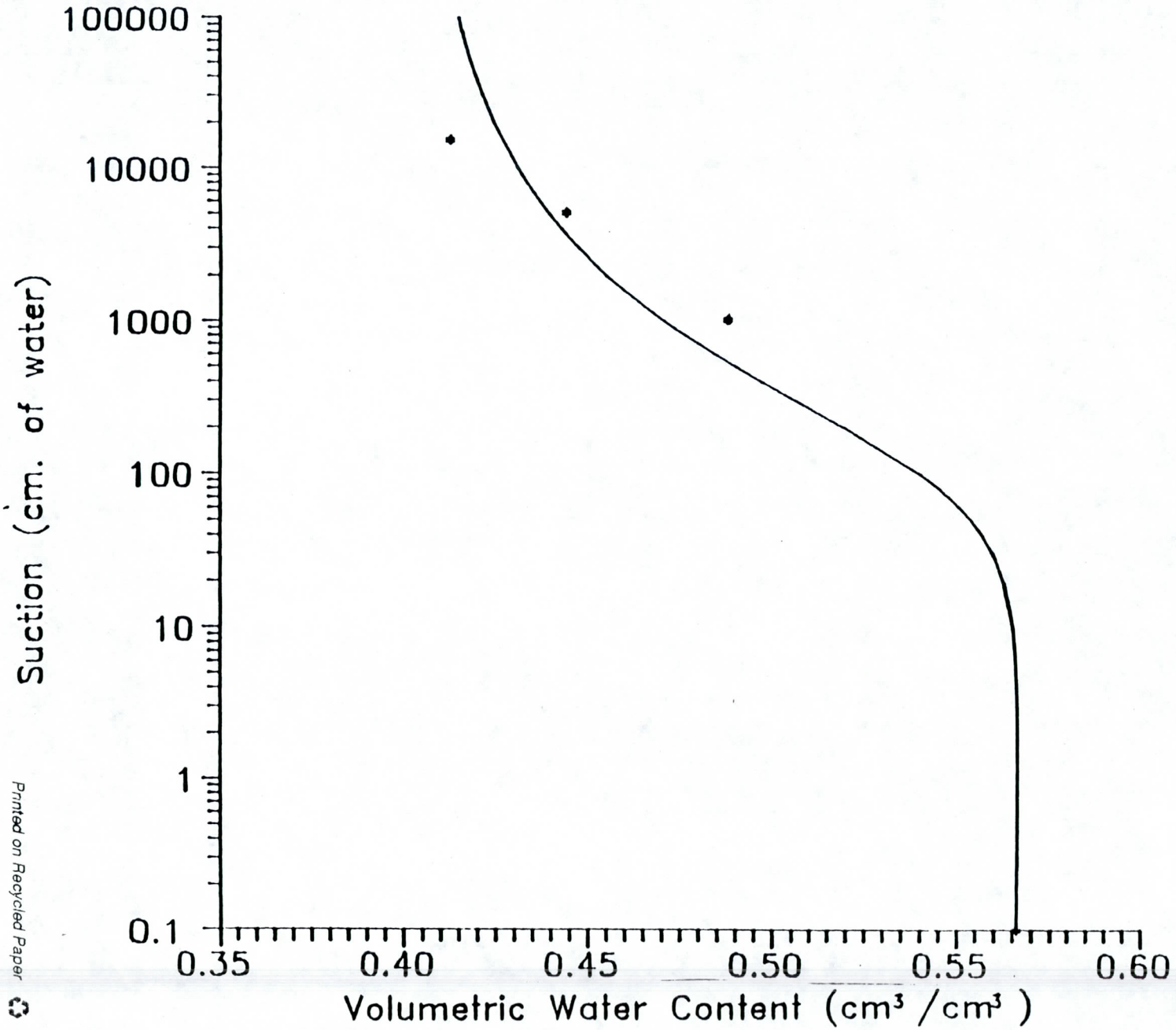




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**Figure 6**
**Moisture Retention Curve for Sample SHC-9-59.5**

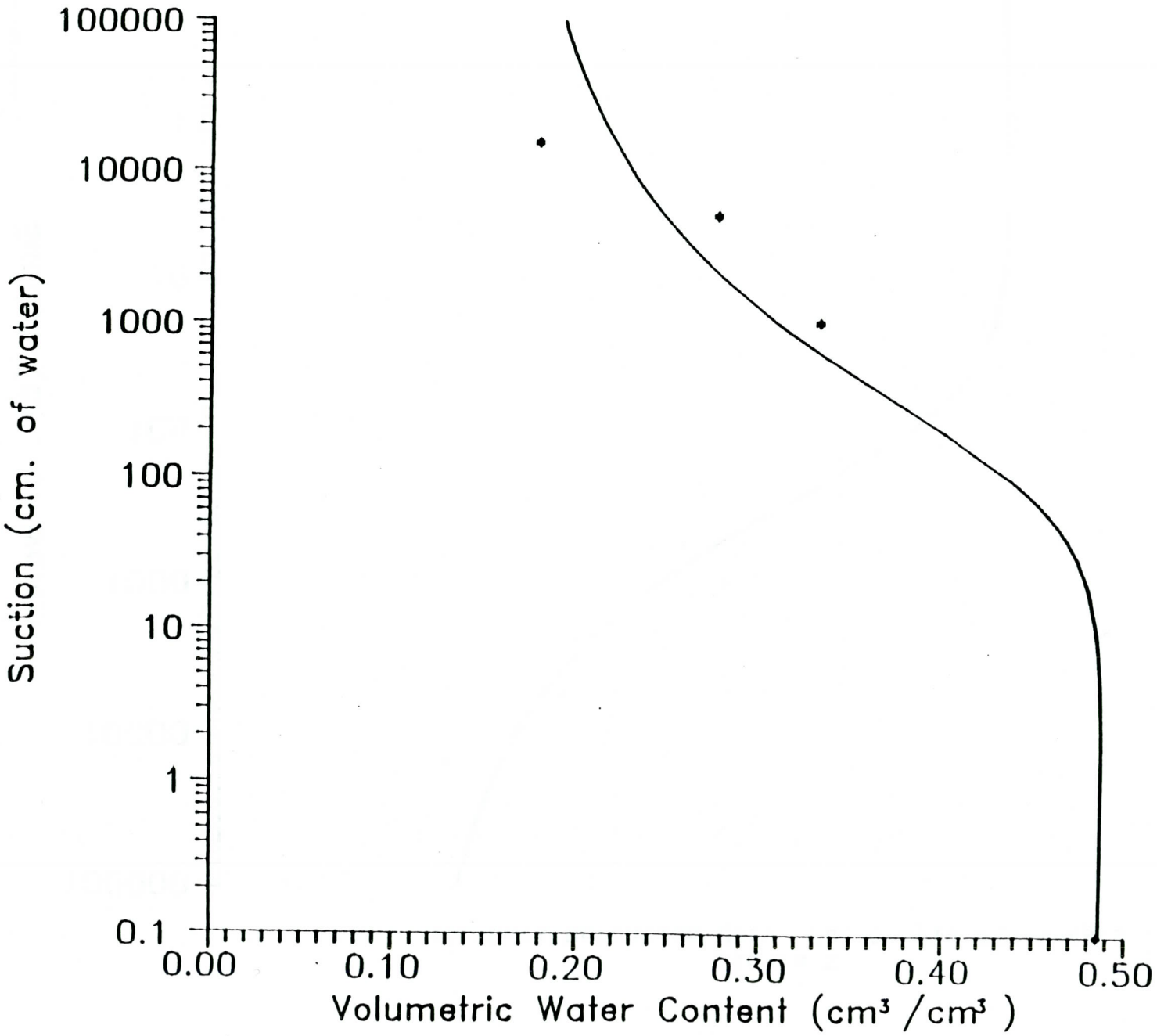

**Figure 7**
**Moisture Retention Curve for Sample SHC-9-69.5**

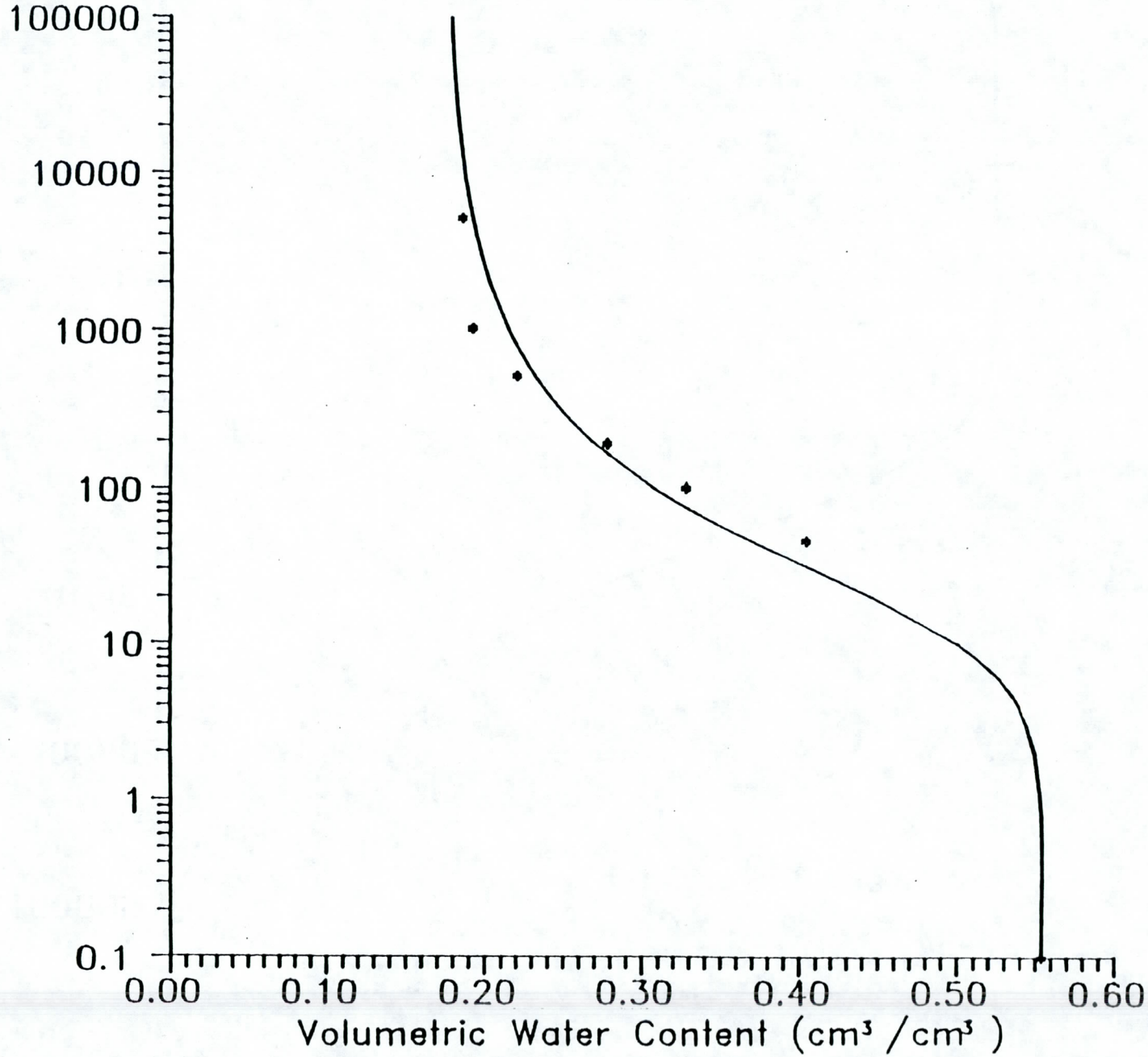




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**Figure 8**
**Moisture Retention Curve for Sample SHC-9-103.0**

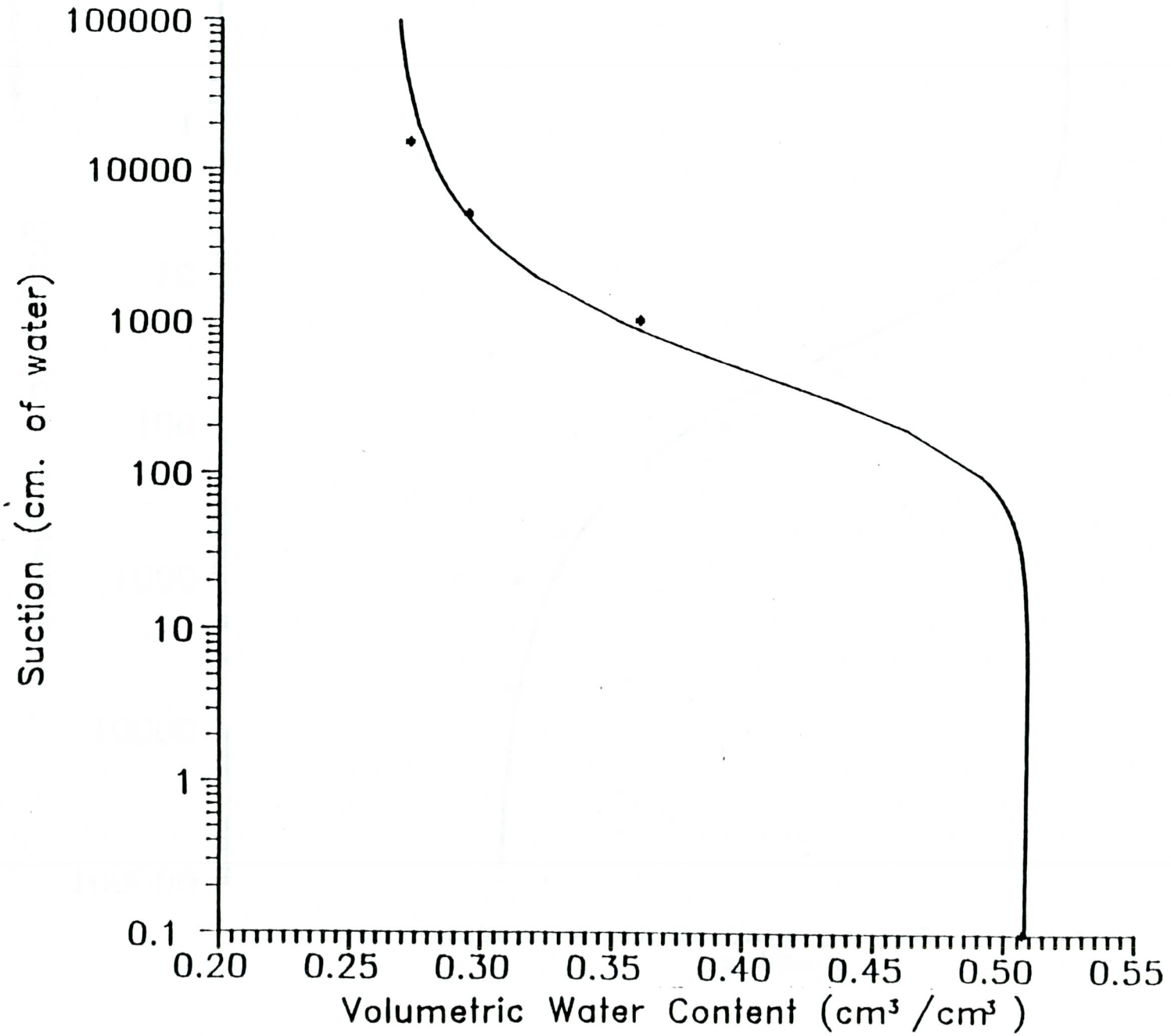

**Figure 9**
**Moisture Retention Curve for Sample SHC-22-23.5**

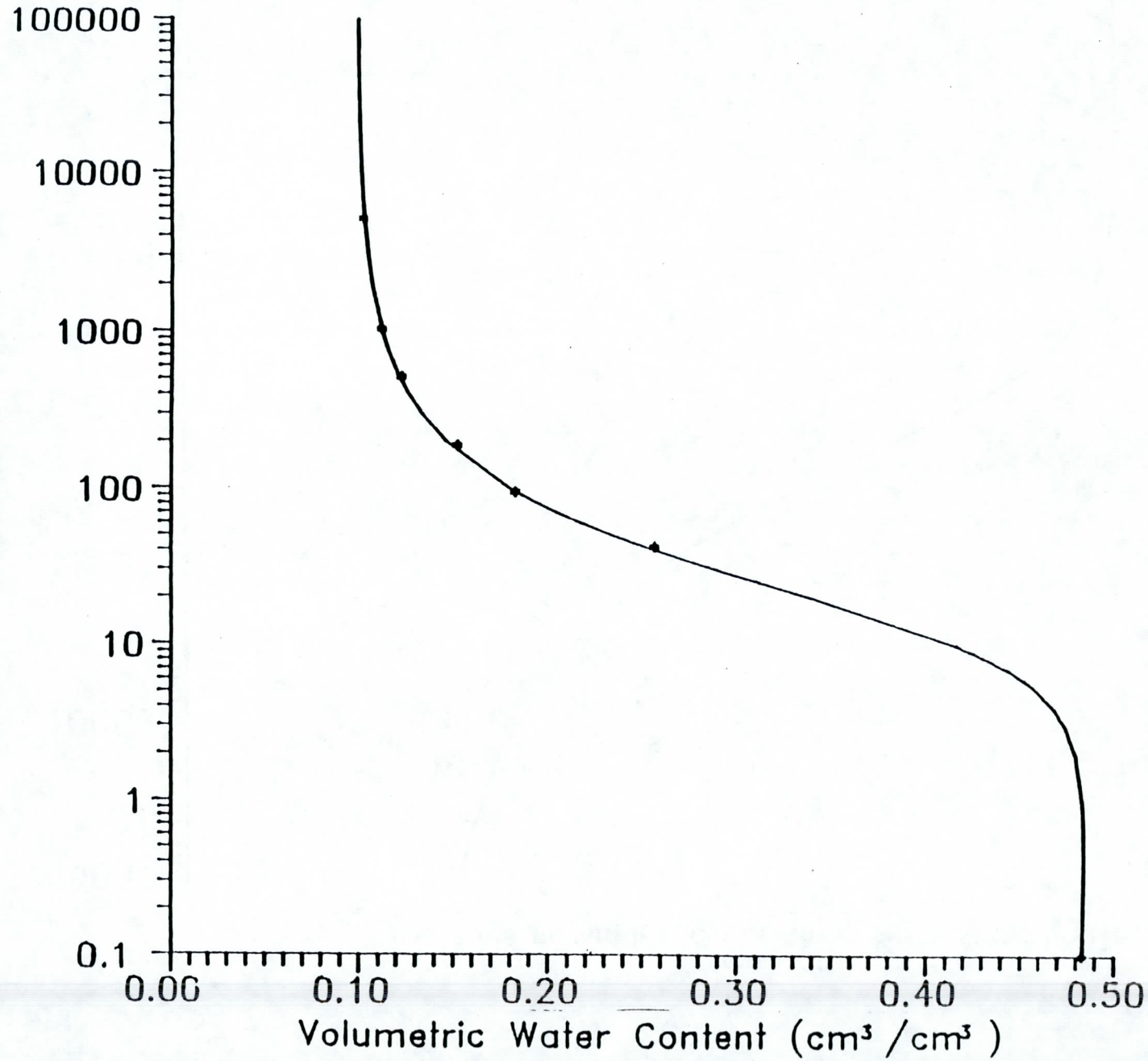




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**Figure 10**
**Moisture Retention Curve for Sample SHC-22-52.5**

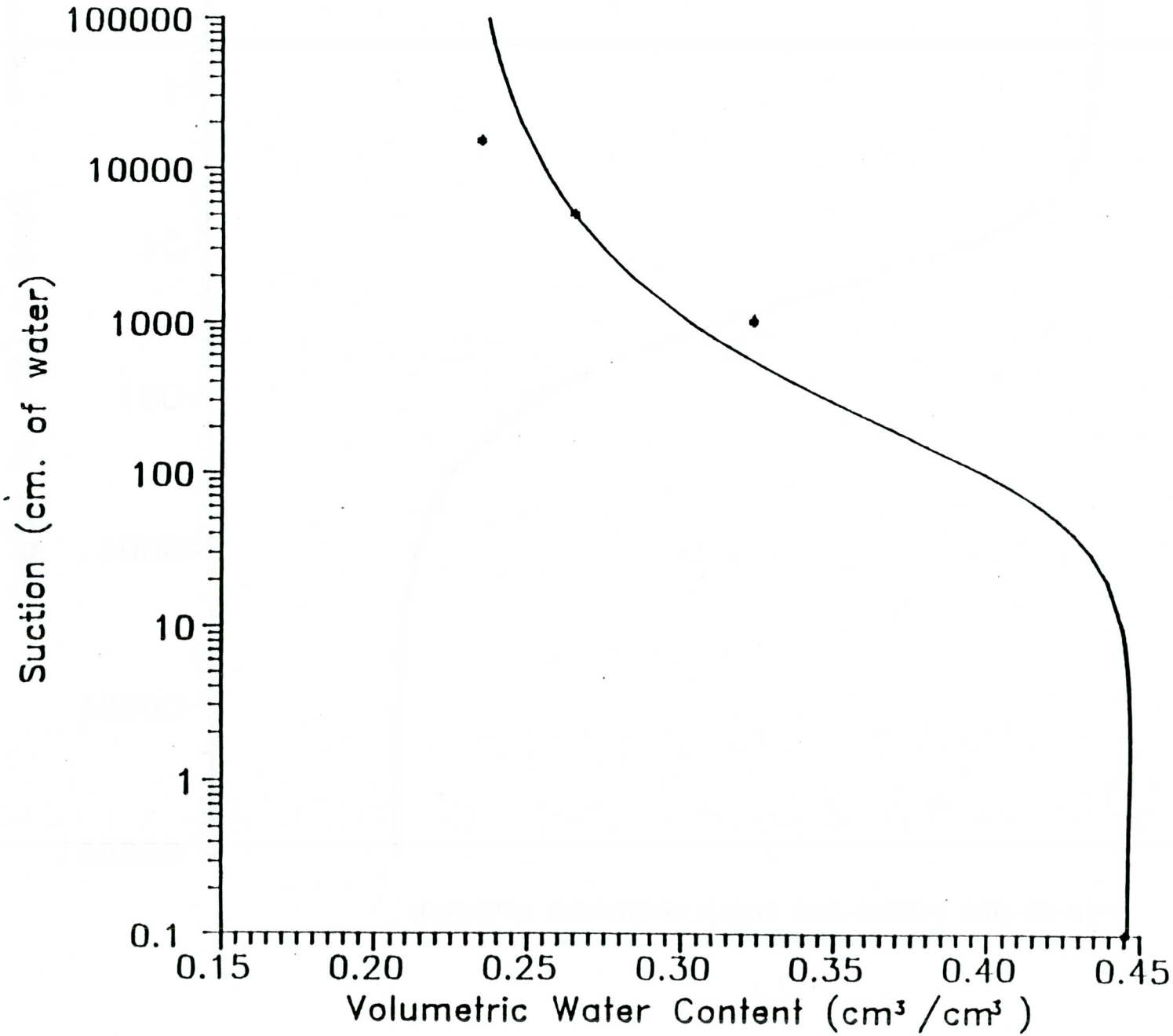

**Figure 11**
**Moisture Retention Curve for Sample SHC-22-54.0**

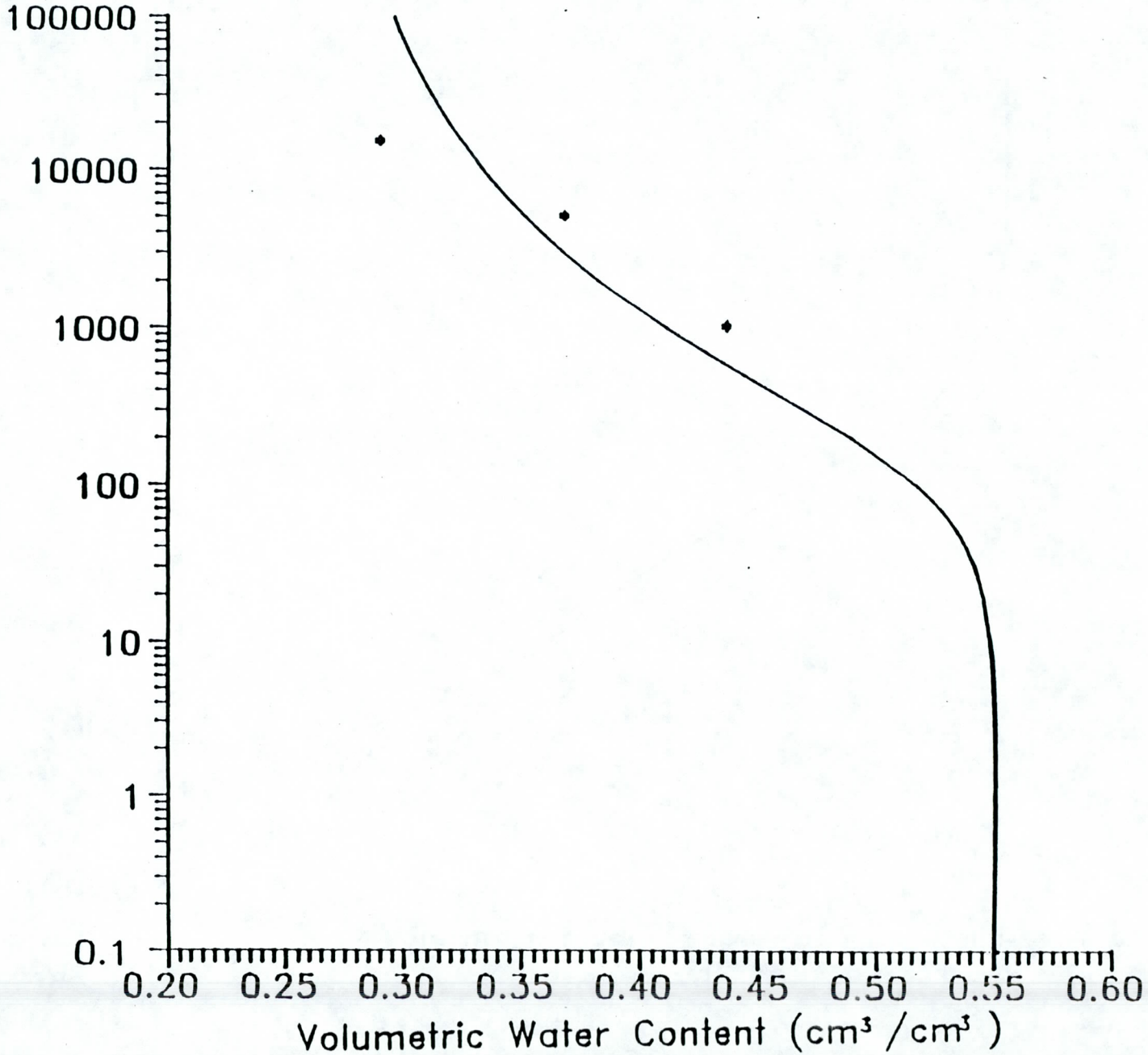




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**Figure 12**
**Moisture Retention Curve for Sample SHC-22-59.0**

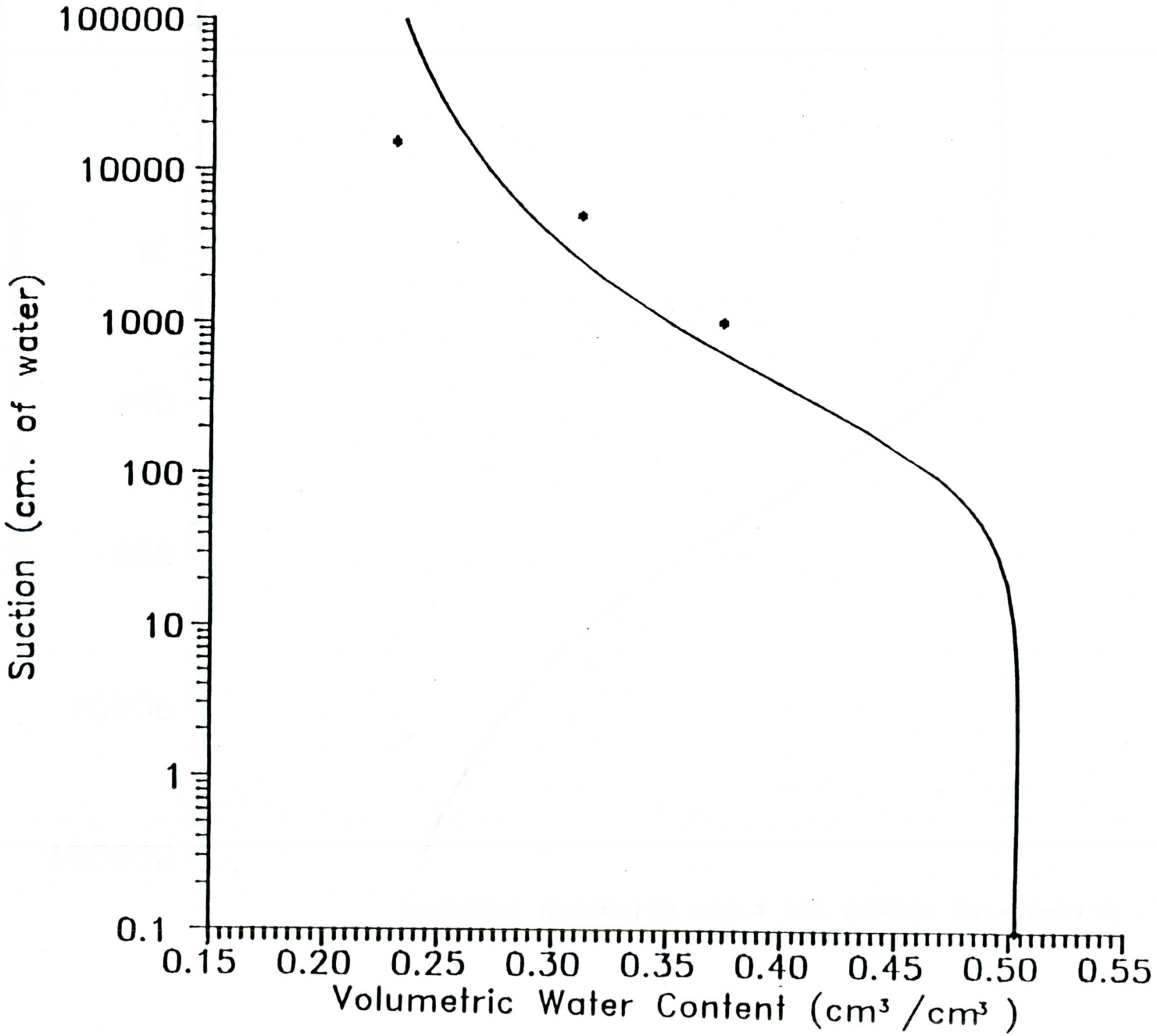

**Figure 13**
**Moisture Retention Curve for Sample SHC-22-68.0**

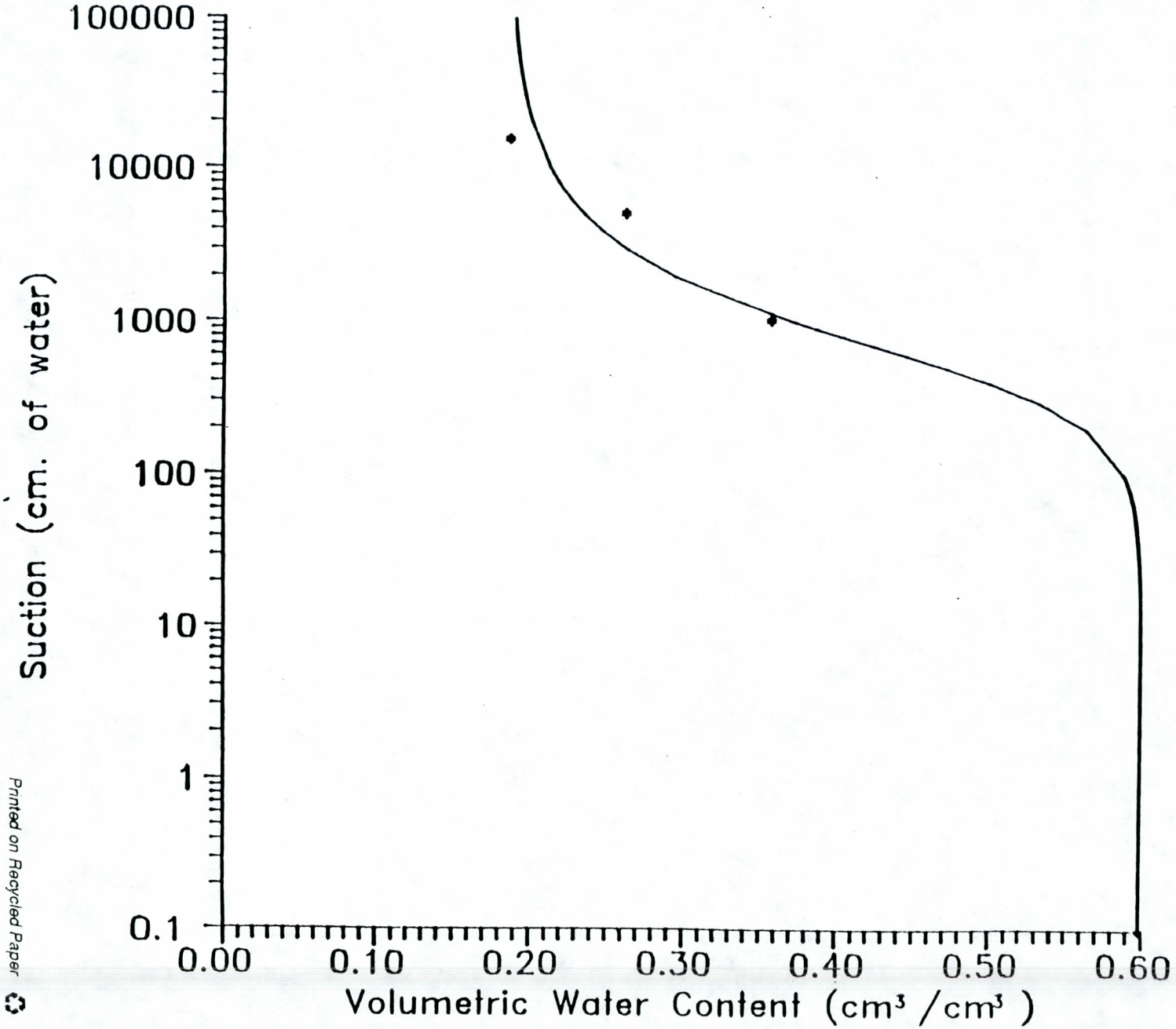




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**Figure 14**
**Moisture Retention Curve for Sample SHC-22-77.5**

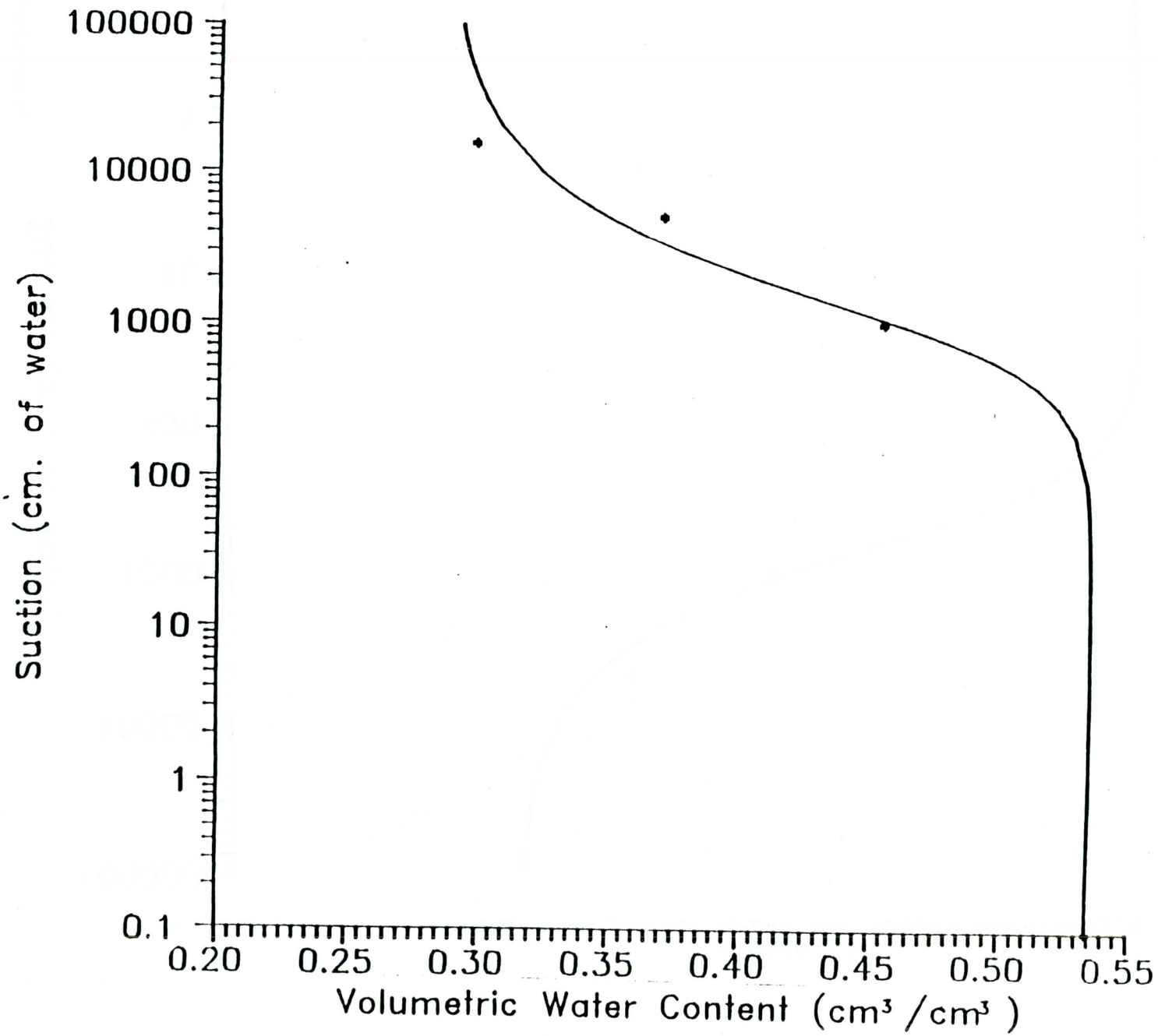

**Figure 15**
Hydraulic Conductivity as a Function of Suction -- Borehole 22

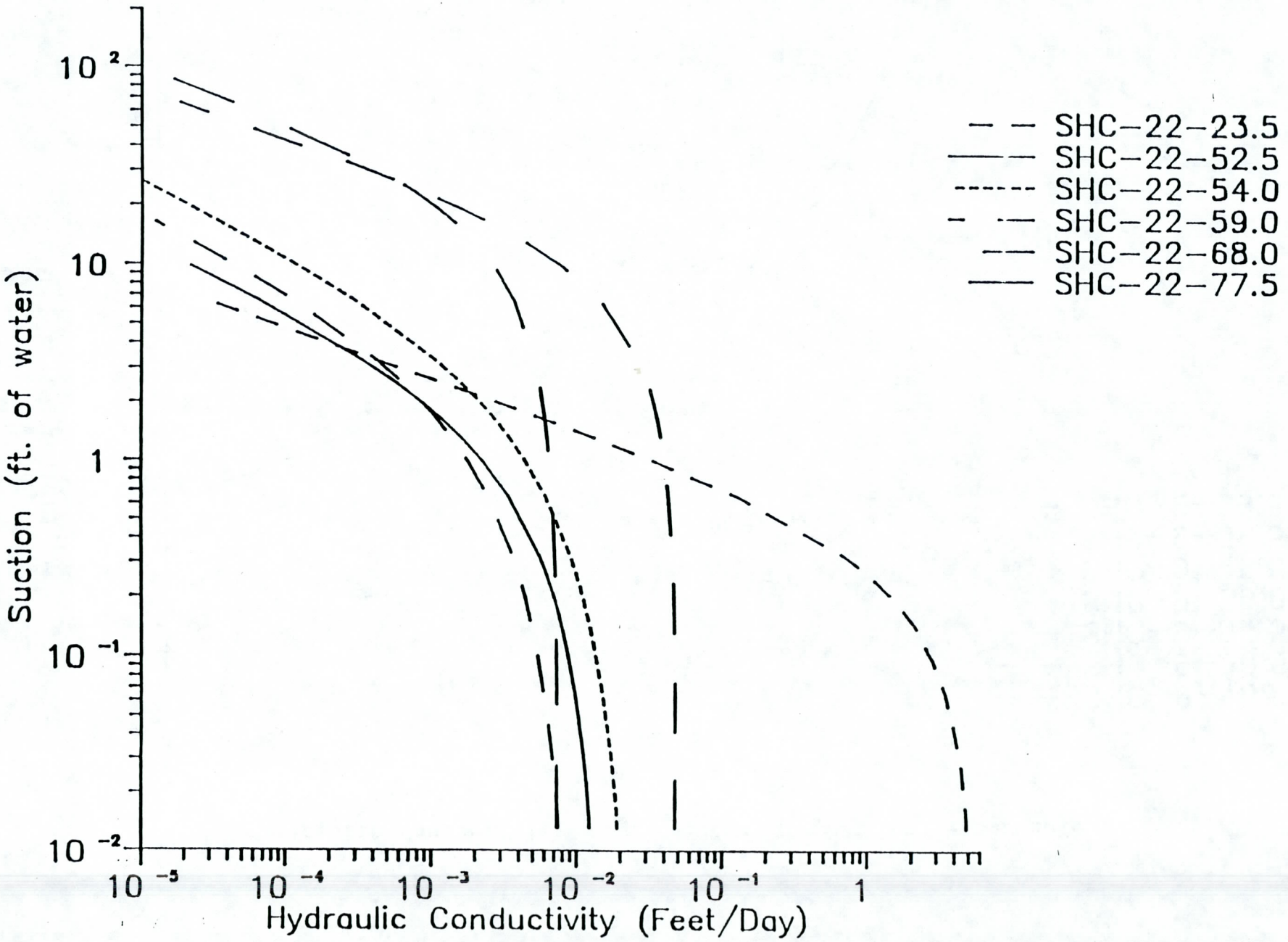




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**Figure 16**
Hydraulic Conductivity as a Function of Suction -- Borehole 9

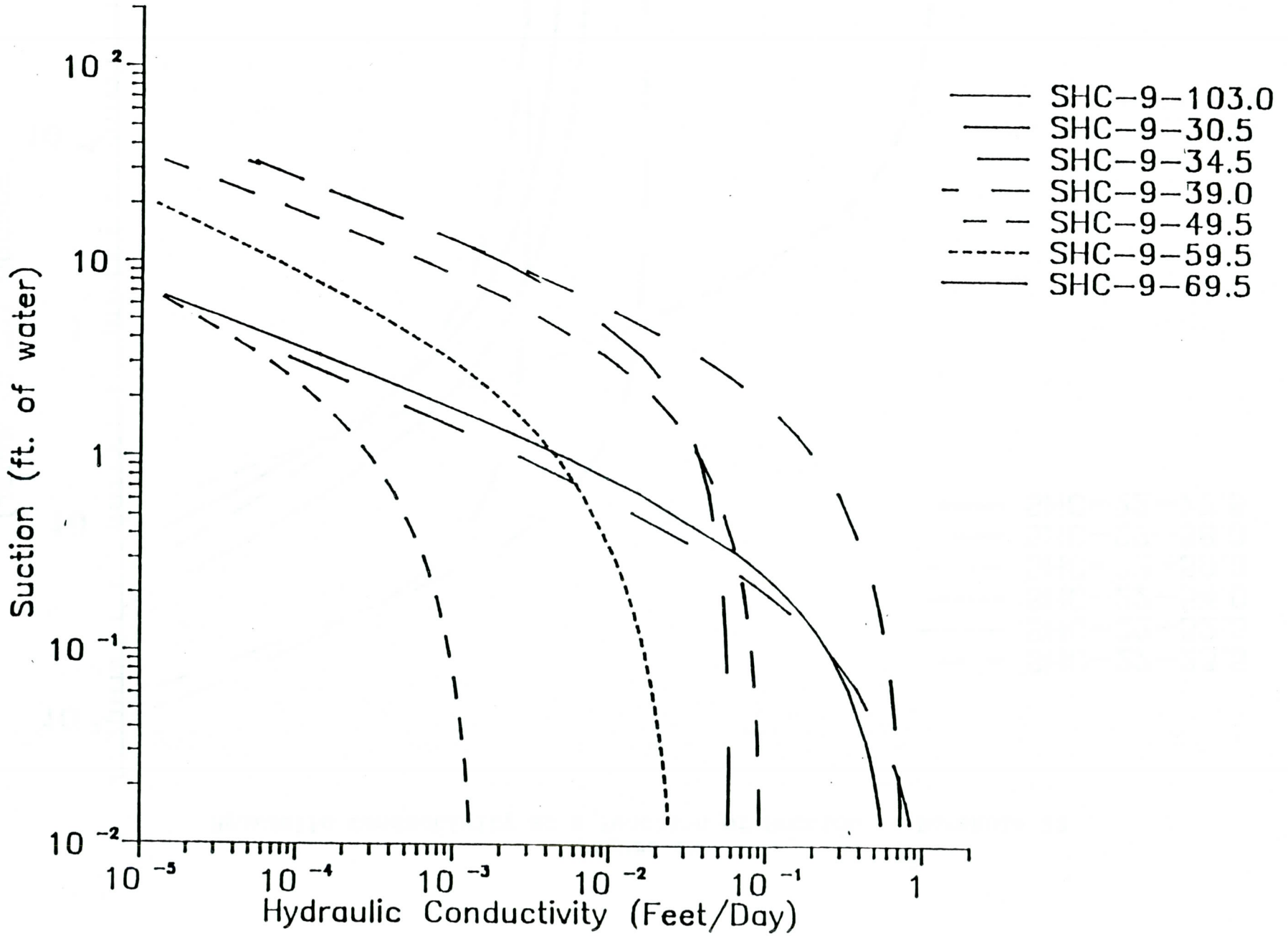

**Figure 17**
Hydraulic Conductivity as a Function of Suction -- Borehole 5

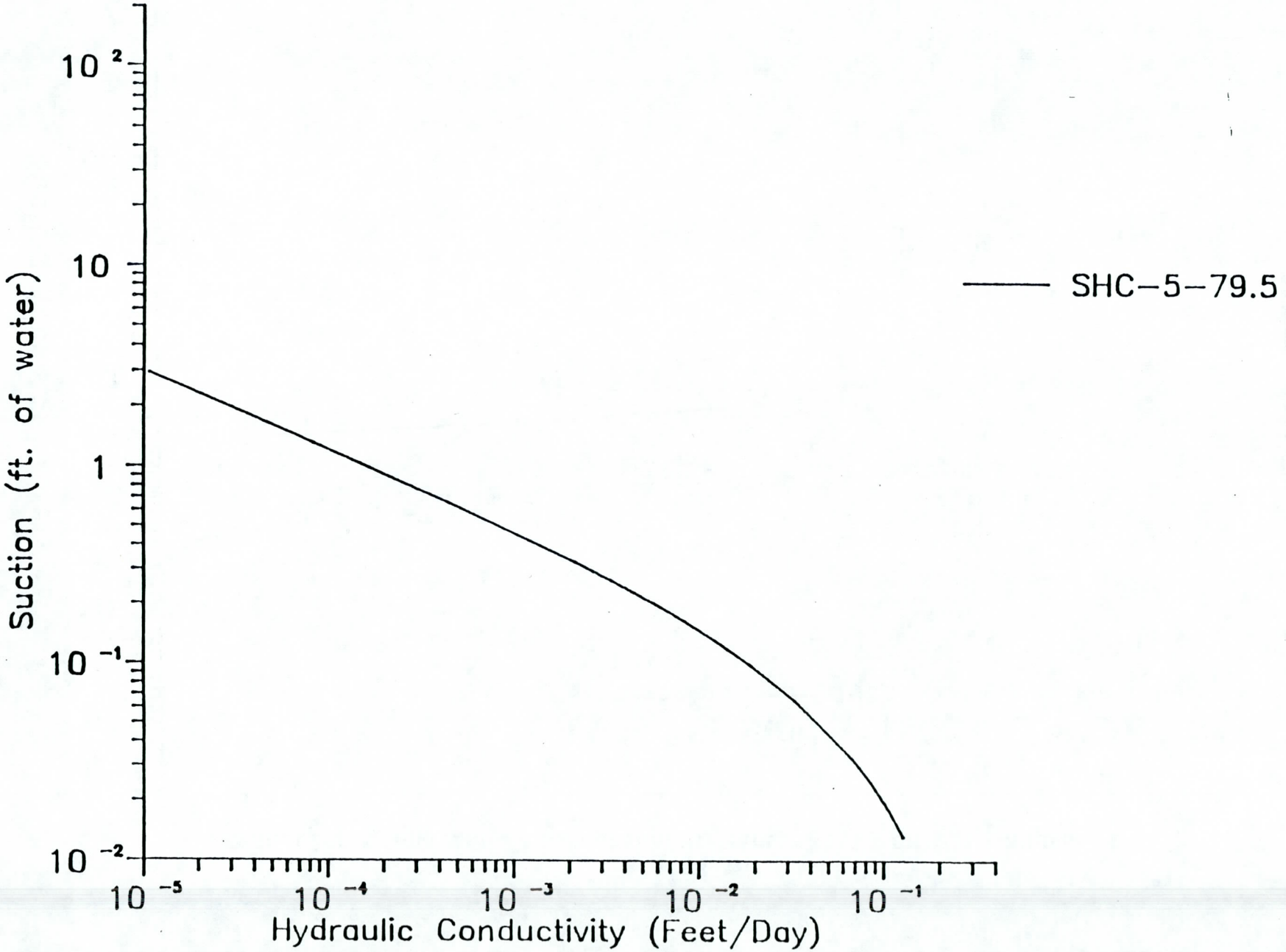




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**Figure 18**
K-Suction Comparison -- Section 7, Material 1, SM with SHC-9-34.5

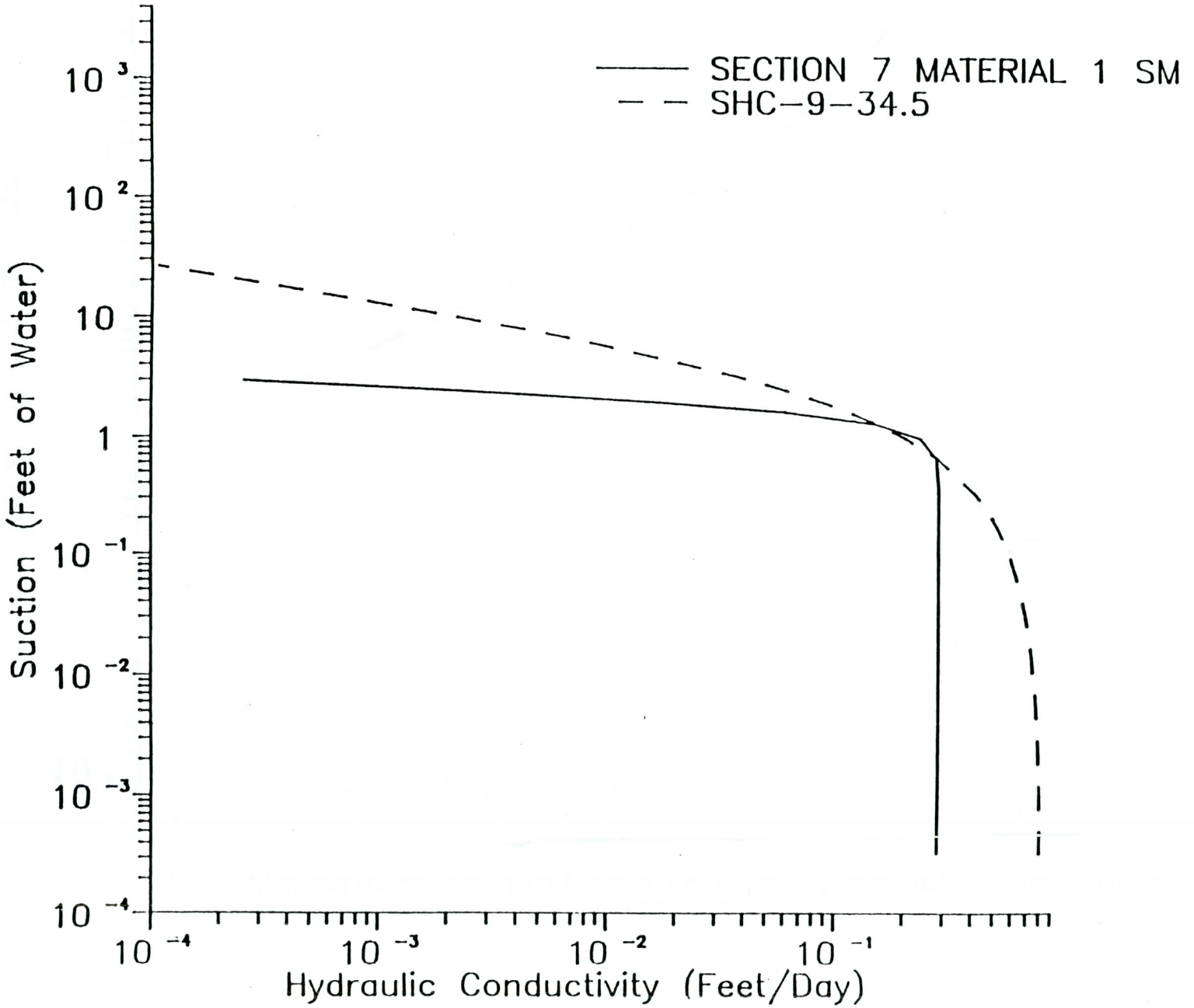

**Figure 19**
K-Suction Comparison -- Section 7, Material 3, CL-ML with SHC-9-49.5 and SHC-9-103.0

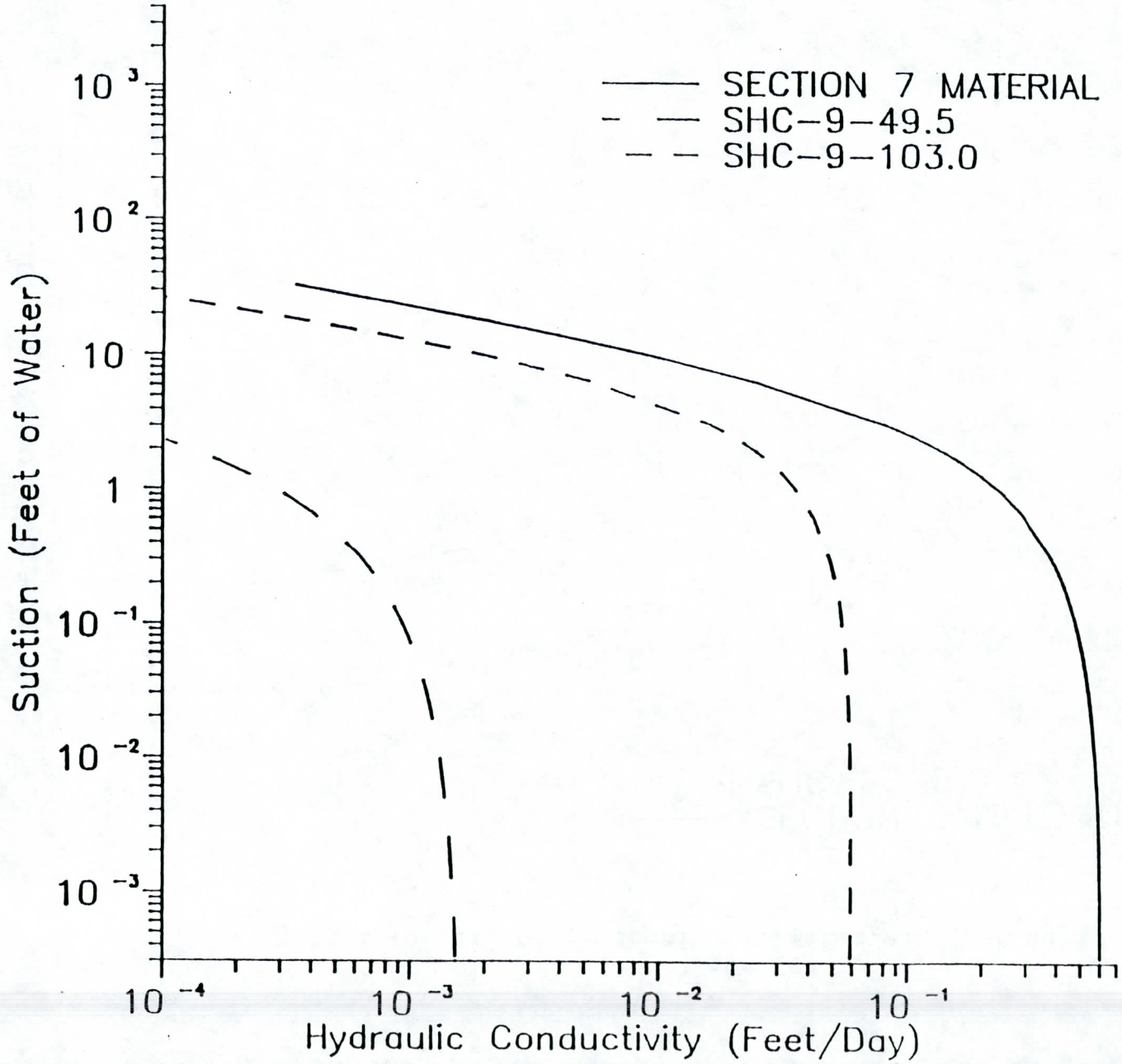




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**Figure 20**
K-Suction Comparison -- Section 7, Material 4, SC with SHC-9-30.0

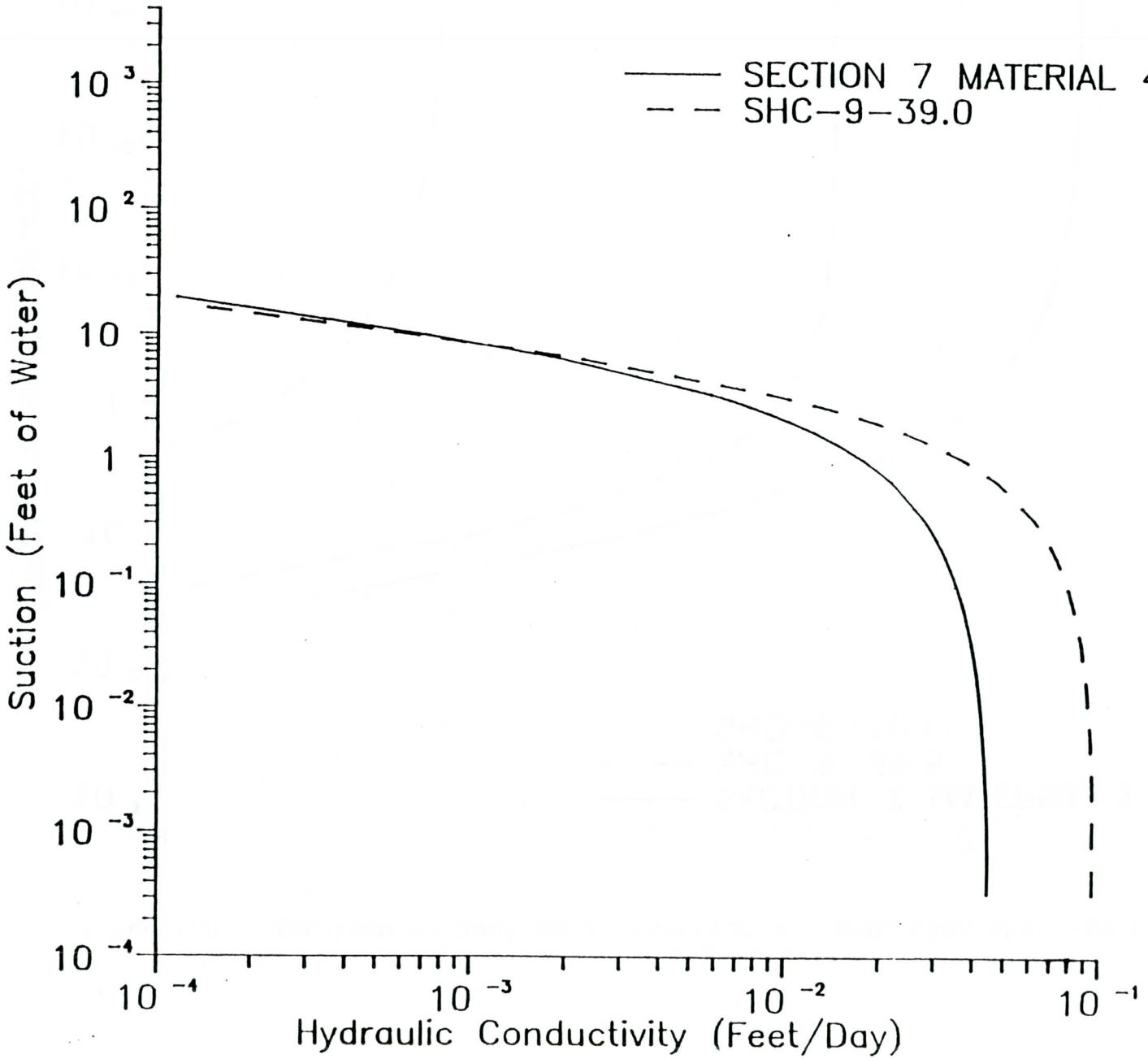

**Figure 21**
K-Suction Comparison -- Section 7, Material 5, SW with SHC-22-23.5

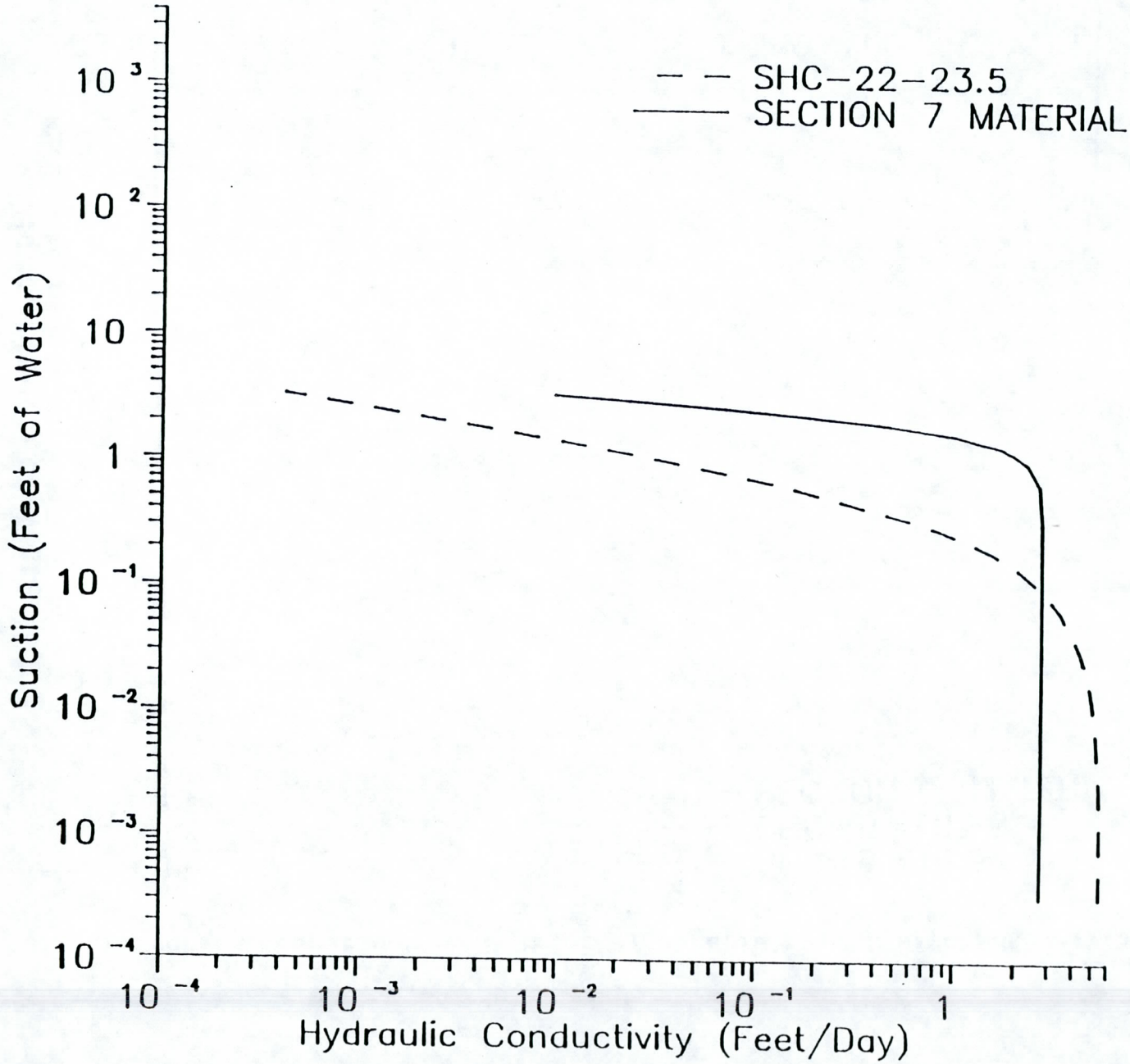




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**Figure 22**
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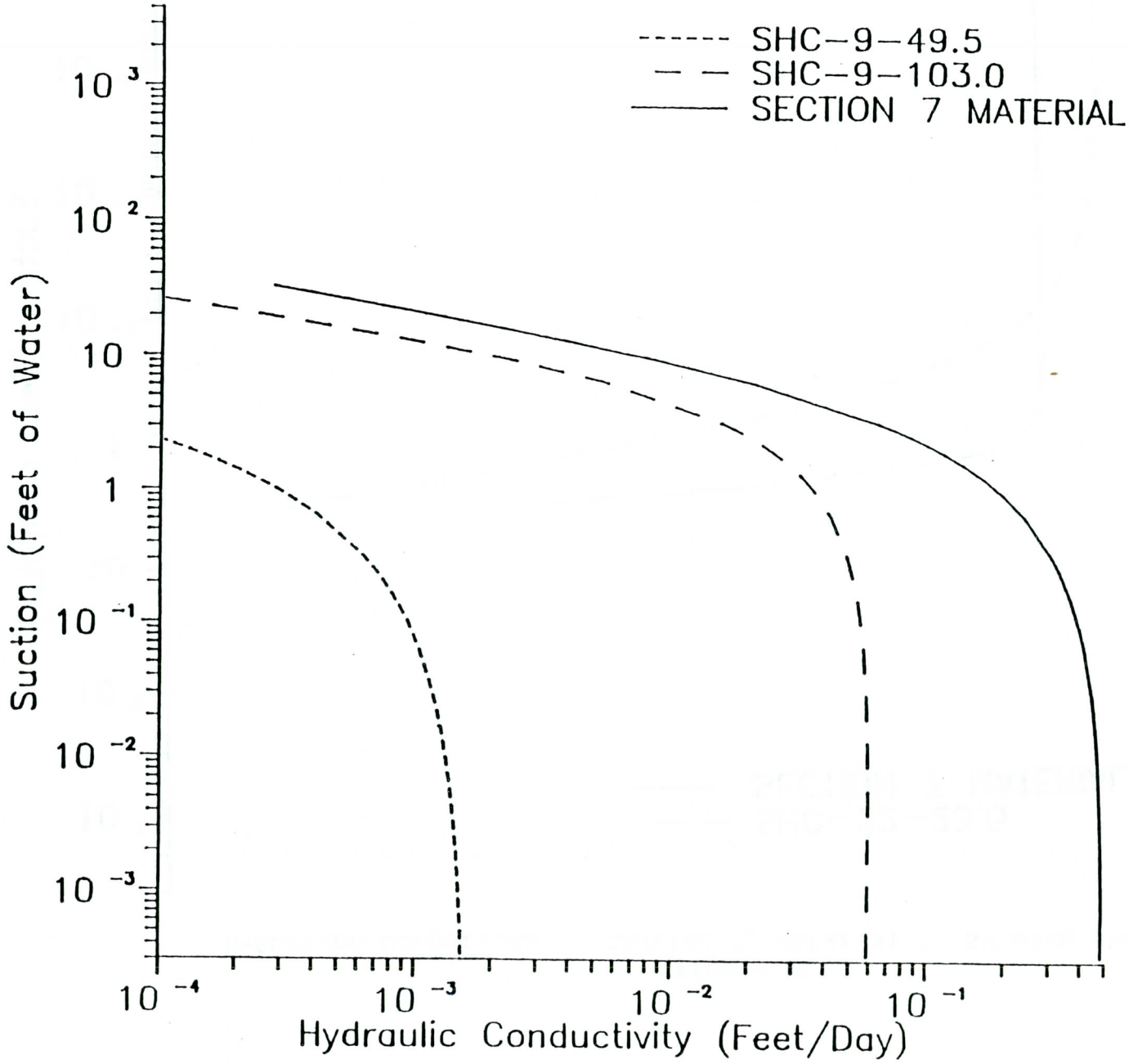

**Figure 23**
K-Suction Comparison -- Section 8, Material 2, SM with SHC-5-79.5

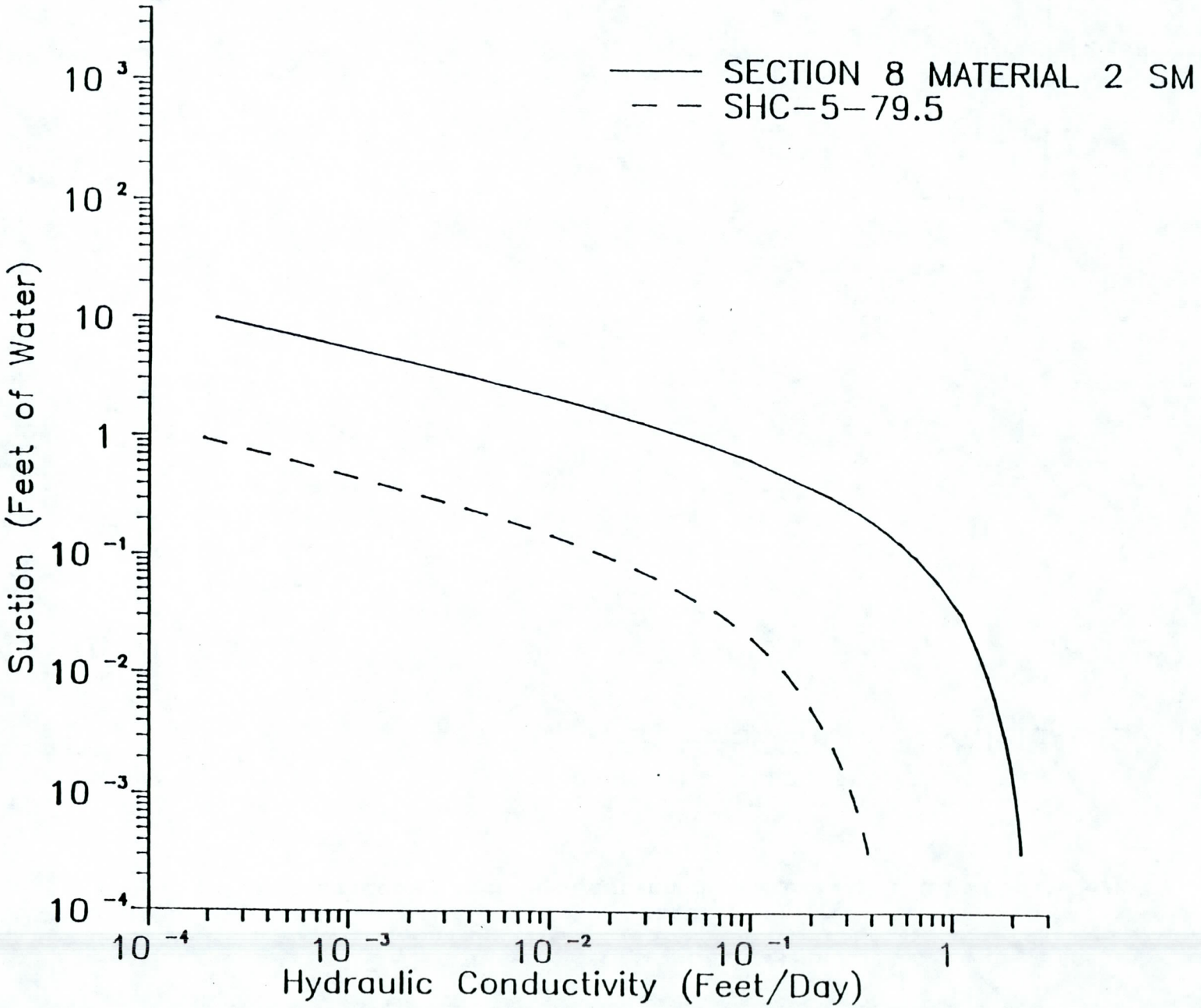




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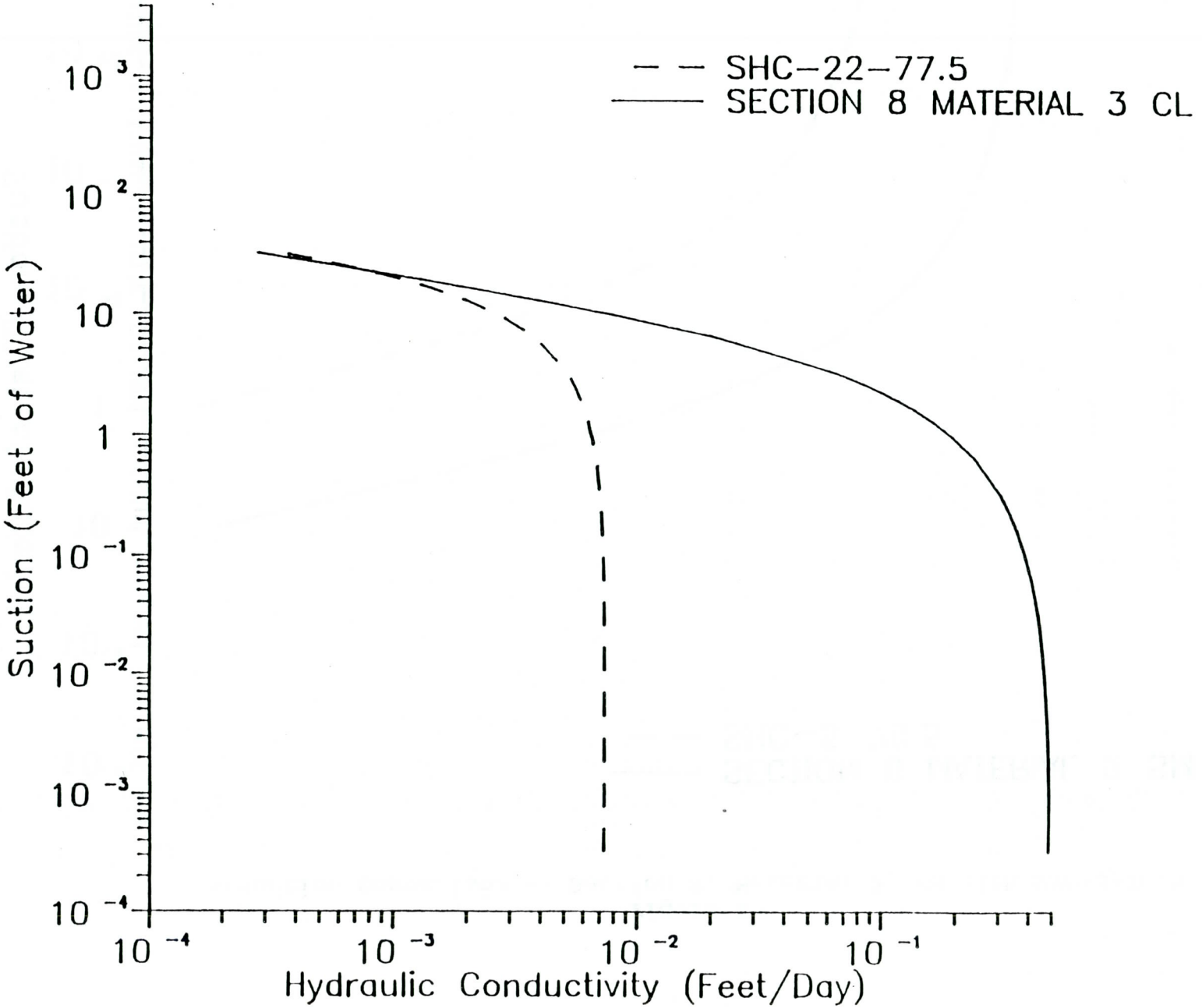
**Figure 24**
K-Suction Comparison -- Section 8, Material 3, CL with SHC-22-77.5



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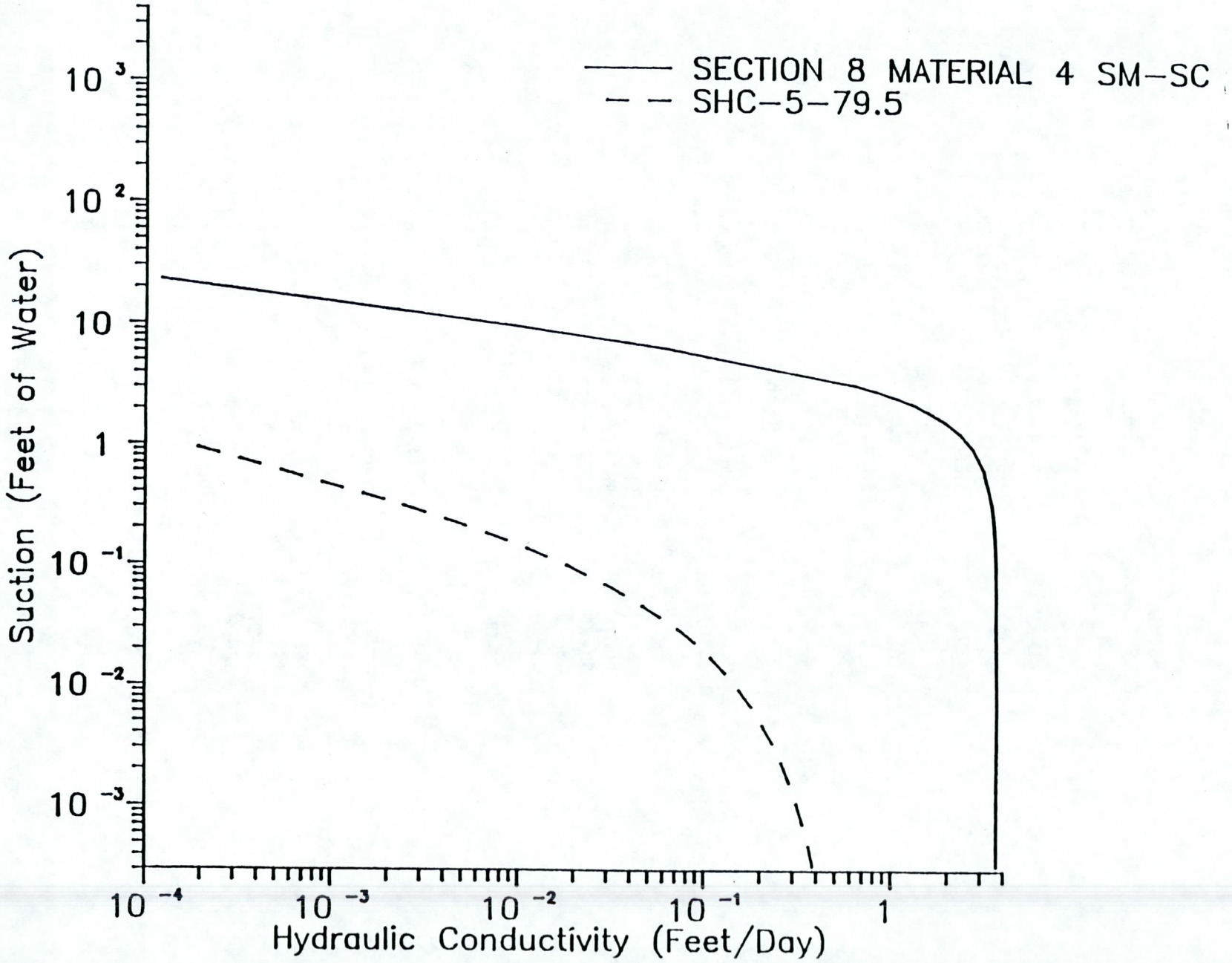




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**Table 3**

Equivalency of Model and Laboratory Material Classification

Section 7

| Mat. No. | Model USCS Class. | Comparable Sample | Laboratory USCS Class. | Depth (ft) |
|---|---|---|---|---|
| 1 | SM | SHC-9-34.5 | SM-SC | 34.5 |
| 2 | GW | | | |
| 3 | CL-ML | SHC-9-49.5 | CL-ML | 49.5 |
| | | SHC-9-103.0 | CL-ML | 103.0 |
| 4 | SC | SHC-9-39.0 | SM-SC | 39.0 |
| 5 | SW | SHC-22-23.5 | SW | 23.5 |
| 6 | GM-GW | | | |
| 7 | GC-GM | | | |
| 8 | CL | SHC-9-49.5 | CL-ML | 49.5 |
| | | SHC-9-103.0 | CL-ML | 103.0 |

Section 8

| Mat. No. | Model USCS Class. | Comparable Sample | Laboratory USCS Class. | Depth (ft) |
|---|---|---|---|---|
| 1 | GM | | | |
| 2 | SM | SHC-5-79.5 | SM-SC | 79.5 |
| 3 | CL | SHC-22-77.5 | CL-ML | 77.5 |
| 4 | SM-SC | SHC-5-79.5 | SM-SC | 79.5 |
| 5 | GM-GC | | | |
| 6 | GM-SM | | | |
| 7 | GW | | | |
| 8 | GM-GW | | | |

Section 9

| Mat. No. | Model USCS Class. | Comparable Sample | Laboratory USCS Class. | Depth (ft) |
|---|---|---|---|---|
| 1 | SP | | | |
| 2 | GP | | | |
| 3 | GW-GM | | | |
| 4 | SP-SW | | | |
| 5 | CL | SHC-9-49.5 | CL-ML | 49.5 |
| | | SHC-9-103.0 | CL-ML | 103.0 |

Attachment 8

LABORATORY ANALYSIS OF HYDRAULIC PROPERTIES FOR THE BROWNING-FERRIS INDUSTRIES PROPOSED CHOLLA LANDFILL SITE

DANIEL B. STEPHENS & ASSOCIATES, INC.

CONSULTANTS IN GROUND-WATER HYDROLOGY ALBUQUERQUE, NEW MEXICO

# LABORATORY ANALYSIS OF HYDRAULIC PROPERTIES FOR THE BROWN FERRIS INDUSTRIES PROPOSED CHOLLA LANDFILL SITE

PREPARED FOR

EMCON ASSOCIATES

PHOENIX, ARIZONA

AUGUST, 1989



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## INTRODUCTION

Daniel B. Stephens & Associates, Inc. (DBS&A) was requested by Mr. Michael Green of Emcon Associates in Phoenix, Arizona to perform laboratory analysis for physical and hydraulic properties of soil samples, as outlined in the written communication of June 7, 1989. The scope of work included conducting the following tasks:

1. Sample Preparation
2. Saturated Hydraulic Conductivity
3. Initial Moisture Content, Dry Bulk Density and Porosity
4. Moisture Characteristics
5. Particle Size Distribution

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## SUMMARY

DBS&A has completed the laboratory analysis as summarized in Table 1. Tables 2 through 6 summarize the soil physical and hydraulic properties. Raw laboratory data are contained in Appendices A through D. Appendix E contains descriptions of the laboratory methods used for this suite of analyses.

Fourteen soil samples were sub-sampled from the soil cores delivered to the DBS&A Laboratory. Soil core SHC-9-30.5(a) required repacking of the soil due to poor sample integrity. The remaining soil cores were delivered generally well intact. Table 2 contains soil core comments.

The results were evaluated subjectively for consistency and reasonableness. Most of the results appear to be reasonably representative of the material tested. However, calculated porosities are less than the saturated moisture content achieved after permeability testing. This may be due to an incorrect assumption that the particle density is equal to 2.65 g/cc and/or slight swelling of the soil after water imbibition.

We also note that the moisture characteristic curves for some clay samples do not approach the typical asymptotic value at 15 bars of tension. For example, sample numbers SHC-22-54.0, SCH-22-59.0, SCH-22-68.0 and SCH-22-77.5 loose a significant amount of water between 7 and 15 bars of tension.

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DBS&A does not assume any responsibility for interpretations or analysis based on these data, nor can we guarantee that these results are fully representative of the undisturbed materials at the field scale. We recommend that careful evaluation of these laboratory results be made for your particular application.

Table 1. Summary of Tests Performed

| | Saturated Hydraulic Conductivity | Moisture Characteristics | | | Initial Moisture Content | Dry Bulk Density | Porosity | Particle Size Distribution | |
|---|---|---|---|---|---|---|---|---|---|
| | | Hanging Column | Pressure Plate | Pressure Membrane | | | | Sieve | Hydrometer |
| SHC-5-79.5 | X | X | X | | X | X | X | X | X |
| SHC-9-30.5(a) | X | X | X | | X | X | X | X | |
| SHC-9-34.5(b) | X | | | X | X | X | X | X | X |
| SHC-9-39.0(a) | X | | | X | X | X | X | X | X |
| SHC-9-49.5 | X | | | X | X | X | X | X | X |
| SHC-9-59.5(a) | X | | | X | X | X | X | X | X |
| SHC-9-69.5(b) | X | X | X | | X | X | X | X | |
| SHC-9-103.0(a) | X | | | X | X | X | X | | X |
| SHC-22-23.5 | X | X | X | | X | X | X | X | |
| SHC-22-52.5 | X | | | X | X | X | X | X | X |
| SHC-22-54.0 | X | | | X | X | X | X | | X |
| SHC-22-59.0 | X | | | X | X | X | X | | X |
| SHC-22-68.0 | X | | | X | X | X | X | | X |
| SHC-22-77.5 | X | | | X | X | X | X | | X |

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Table 2. Summary of Sample Characteristics

| Sample Number | Depth (ft) | Color | Texture | ASTM D 2488-84 Classification | Comments |
|---|---|---|---|---|---|
| SHC-5-79.5 | 79.5 | brown | silt | SM-SC | slightly moist, moderately loose compaction, sample contained some large particles |
| SHC-9-30.5(a) | 30.5 | brown | silty sand w/gravel | SW-SC | dry to slightly mosit, moderately loose compaction, gravel particles were 0.5-1.0 cm in diameter |
| SHC-9-34.5(b) | 34.5 | brown | silty sand | SM-SC | dry to slightly moist, moderately loose compaction, high silt content |
| SHC-9-39.0(a) | 39.0 | brown | silty sand w/gravel | SM-SC | slightly moist, moderately dense compaction gravel particles were 0.5-1.0 cm in diameter, high silt content |
| SHC-9-49.5 | 49.5 | brown | clay | CL-ML | moist, dense compaction, sample contained an area of angular particles |
| SHC-9-59.5(a) | 59.5 | brown | silty sand w/gravel | SM-SC | moist, moderately loose compaction, high silt content |
| SHC-9-69.5(b) | 69.5 | brown | sand | SM-SC | slightly moist to moist, moderately dense compaction |
| SHC-9-103.0(a) | 103.0 | brown | clay | CL-ML | moist, dense compaction |
| SHC-22-23.5 | 23.5 | brown | silty sand w/gravel | SW | dry, loose compaction |
| SHC-22-52.5 | 52.5 | brown | silt | CL-ML | moist, moderately dense compaction |

Table 2. Summary of Sample Characteristics (Continued)

| Sample Number | Depth (ft) | Color | Texture | ASTM D 2488-84 Classification | Comments |
|---|---|---|---|---|---|
| SHC-22-54.0 | 54.0 | brown | clay | CL-ML | moist, dense compaction |
| SHC-22-59.0 | 59.0 | brown | clay | CL-ML | moist, dense compaction |
| SHC-22-68.0 | 68.0 | brown | silt | CL-ML | moist, moderately loose compaction, sample contained a large cobble 5 cm in diameter |
| SHC-22-77.5 | 77.5 | brown | clay | CL-ML | moist, dense compaction |

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Table 3. Summary of Initial Moisture Content, Dry Bulk Density and Porosity

| Sample No. | Initial Moisture Content Gravimetric (% g/g) | Initial Moisture Content Volumetric (% $cm^3/cm^3$) | Dry Bulk Density (g/cc) | Porosity (%) |
|---|---|---|---|---|
| SHC-5-79.5 | 25.69 | 35.58 | 1.39 | 47.73 |
| SHC-9-30.5(a) | 12.15 | 20.44 | 1.68 | 36.48 |
| SHC-9-34.5(b) | 16.42 | 26.73 | 1.63 | 38.57 |
| SHC-9-39.0(a) | 19.66 | 31.39 | 1.60 | 39.75 |
| SHC-9-49.5 | 34.16 | 45.00 | 1.32 | 50.29 |
| SHC-9-59.5(a) | 18.91 | 31.37 | 1.66 | 37.40 |
| SHC-9-69.5(b) | 12.97 | 20.97 | 1.62 | 38.99 |
| SHC-9-103.0(a) | 26.34 | 38.38 | 1.46 | 45.02 |
| SHC-22-23.5 | 8.72 | 13.91 | 1.59 | 39.83 |
| SHC-22-52.5 | 21.85 | 35.92 | 1.64 | 37.98 |
| SHC-22-54.0 | 31.94 | 42.42 | 1.33 | 49.89 |
| SHC-22-59.0 | 28.34 | 42.08 | 1.48 | 43.97 |
| SHC-22-68.0 | 28.69 | 42.63 | 1.49 | 43.92 |
| SHC-22-77.5 | 32.83 | 47.26 | 1.44 | 45.67 |

Table 4. Summary of Saturated Hydraulic Conductivity Tests

| Sample No. | $K_s$(cm/sec) | Method of Analysis: Constant Head | Method of Analysis: Falling Head |
|---|---|---|---|
| SHC-5-79.5 | 2.32E-04 | X | |
| SHC-9-30.5(a) | 5.37E-04 | X | |
| SHC-9-34.5(b) | 2.87E-04 | X | |
| SHC-9-39.0(a) | 3.39E-05 | | X |
| SHC-9-49.5 | 5.62E-07 | | X |
| SHC-9-59.5(a) | 1.17E-05 | X | |
| SHC-9-69.5(b) | 2.66E-04 | X | |
| SHC-9-103.0(a) | 2.08E-05 | | X |
| SHC-22-23.5 | 1.94E-03 | X | |
| SHC-22-52.5 | 5.40E-06 | | X |
| SHC-22-54.0 | 9.29E-06 | | X |
| SHC-22-59.0 | 3.50E-06 | | X |
| SHC-22-68.0 | 1.68E-05 | | X |
| SHC-22-77.5 | 2.58E-06 | | X |

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Table 5. Summary of Moisture Characteristics

| Sample No. | Pressure Head (-cm of water) | Moisture Content (% $cm^3/cm^3$) |
|---|---|---|
| SHC-5-79.5 | 0.0 | 54.64 |
| | 43.5 | 41.41 |
| | 102.0 | 37.73 |
| | 195.5 | 34.99 |
| | 509.9 | 30.01 |
| | 1019.8 | 26.61 |
| | 5099.0 | 25.49 |
| SHC-9-30.5(a) | 0.0 | 45.40 |
| | 44.0 | 32.80 |
| | 99.0 | 27.98 |
| | 191.5 | 24.78 |
| | 509.9 | 20.77 |
| | 1019.8 | 18.92 |
| | 5099.0 | 18.93 |
| SHC-9-34.5(b) | 0.0 | 50.67 |
| | 1019.8 | 26.27 |
| | 5099.0 | 23.02 |
| | 15297.0 | 17.51 |
| SHC-9-39.0(a) | 0.0 | 45.50 |
| | 1019.8 | 26.83 |
| | 5099.0 | 22.49 |
| | 15297.0 | 18.70 |
| SHC-9-49.5 | 0.0 | 56.62 |
| | 1019.8 | 48.77 |
| | 5099.0 | 44.39 |
| | 15297.0 | 41.24 |
| SHC-9-59.5(a) | 0.0 | 48.50 |
| | 1019.8 | 33.01 |
| | 5099.0 | 27.47 |
| | 15297.0 | 17.73 |
| SHC-9-69.5(b) | 0.0 | 55.37 |
| | 45.5 | 40.37 |
| | 100.0 | 32.75 |
| | 192.5 | 27.74 |
| | 509.9 | 21.98 |
| | 1019.8 | 19.95 |
| | 5099.0 | 18.52 |

Table 5. Summary of Moisture Characteristics (Continued)

| Sample No. | Pressure Head (-cm of water) | Moisture Content (% $cm^3/cm^3$) |
|---|---|---|
| SHC-9-103.0(a) | 0.0 | 50.71 |
| | 1019.8 | 35.90 |
| | 5099.0 | 29.39 |
| | 15297.0 | 27.17 |
| SHC-22-23.5 | 0.0 | 48.35 |
| | 41.5 | 25.60 |
| | 94.0 | 18.22 |
| | 186.0 | 15.16 |
| | 509.9 | 12.18 |
| | 1019.8 | 11.13 |
| | 5099.0 | 10.12 |
| SHC-22-52.5 | 0.0 | 44.51 |
| | 1019.8 | 32.25 |
| | 5099.0 | 26.43 |
| | 15297.0 | 23.41 |
| SHC-22-54.0 | 0.0 | 55.00 |
| | 1019.8 | 43.61 |
| | 5099.0 | 36.74 |
| | 15297.0 | 28.95 |
| SHC-22-59.0 | 0.0 | 50.26 |
| | 1019.8 | 37.23 |
| | 5099.0 | 31.03 |
| | 15297.0 | 22.95 |
| SHC-22-68.0 | 0.0 | 59.80 |
| | 1019.8 | 35.68 |
| | 5099.0 | 26.19 |
| | 15297.0 | 18.64 |
| SHC-22-77.5 | 0.0 | 53.41 |
| | 1019.8 | 45.36 |
| | 5099.0 | 36.93 |
| | 15297.0 | 29.77 |

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Table 6. Summary of Particle Size Characteristics

| Sample No. | $d_{10}$ (mm) | $d_{50}$ (mm) | $d_{60}$ (mm) | $C_u$ | $C_c$ |
|---|---|---|---|---|---|
| SHC-5-79.5 | * | 0.22 | 0.34 | -- | -- |
| SHC-9-30.5(a) | 0.13 | 0.80 | 1.10 | 8.46 | 0.0 |
| SHC-9-34.5(b) | 0.011 | 0.30 | 0.38 | 34.55 | 4.04 |
| SHC-9-39.0(a) | 0.02 | 0.20 | 0.36 | 18.00 | 0.93 |
| SHC-9-49.5 | * | 0.0054 | 0.012 | -- | -- |
| SHC-9-59.5(a) | 0.0152 | 0.26 | 0.38 | 25.00 | 1.40 |
| SHC-9-69.5(b) | 0.08 | 0.38 | 0.51 | 6.38 | 1.19 |
| SHC-9-103.0(a) | * | 0.018 | 0.027 | -- | -- |
| SHC-22-23.5 | 0.15 | 0.73 | 1.00 | 6.67 | 0.96 |
| SHC-22-52.5 | 0.0044 | 0.07 | 0.08 | 18.18 | 1.92 |
| SHC-22-54.0 | * | 0.01 | 0.17 | -- | -- |
| SHC-22-59.0 | 0.0022 | 0.029 | 0.037 | 16.82 | 1.77 |
| SHC-22-68.0 | 0.003 | 0.03 | 0.04 | 13.33 | 1.88 |
| SHC-22-77.5 | * | 0.018 | 0.024 | -- | -- |

* diameter was not reached with test(s) specified
- values are dependent on diameters that were not reached

# Appendix A: INITIAL MOISTURE CONTENT, DRY BULK DENSITY AND POROSITY

## Summary of Initial Moisture Content, Dry Bulk Density and Porosity

| Sample No. | Initial Moisture Content Gravimetric (% g/g) | Initial Moisture Content Volumetric (% $cm^3/cm^3$) | Dry Bulk Density (g/cc) | Porosity (%) |
|---|---|---|---|---|
| SHC-5-79.5 | 25.69 | 35.58 | 1.39 | 47.73 |
| SHC-9-30.5(a) | 12.15 | 20.44 | 1.68 | 36.48 |
| SHC-9-34.5(b) | 16.42 | 26.73 | 1.63 | 38.57 |
| SHC-9-39.0(a) | 19.66 | 31.39 | 1.60 | 39.75 |
| SHC-9-49.5 | 34.16 | 45.00 | 1.32 | 50.29 |
| SHC-9-59.5(a) | 18.91 | 31.37 | 1.66 | 37.40 |
| SHC-9-69.5(b) | 12.97 | 20.97 | 1.62 | 38.99 |
| SHC-9-103.0(a) | 26.34 | 38.38 | 1.46 | 45.02 |
| SHC-22-23.5 | 8.72 | 13.91 | 1.59 | 39.83 |
| SHC-22-52.5 | 21.85 | 35.92 | 1.64 | 37.98 |
| SHC-22-54.0 | 31.94 | 42.42 | 1.33 | 49.89 |
| SHC-22-59.0 | 28.34 | 42.08 | 1.48 | 43.97 |
| SHC-22-68.0 | 28.69 | 42.63 | 1.49 | 43.92 |
| SHC-22-77.5 | 32.83 | 47.26 | 1.44 | 45.67 |

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DATA FOR INITIAL MOISTURE CONTENT,
BULK DENSITY, AND POROSITY

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-5-79.5 |
| RING NUMBER: | P27 |
| DEPTH: | 79.5 FT. |

| | |
|---|---|
| FIELD WEIGHT OF SAMPLE (W/CAP AND RING): | 120.79 (g) |
| TARE WEIGHT, RING: | 17.89 (g) |
| TARE WEIGHT, PAN: | 0.00 (g) |
| SAMPLE VOLUME: | 59.11 (cc) |
| DATE AND TIME INTO OVEN: | 8/14/89 @ 830 |
| DATE AND TIME OUT OF OVEN: | 8/16/89 @ 1200 |
| DRY WEIGHT OF SAMPLE: | 81.87 (g) |
| DRY BULK DENSITY: | 1.39 (g/cc) |
| PARTICLE DENSITY: | 2.65 (g/cc) |
| (METHOD: ASSUME MEAN PARTICLE DENSITY = 2.65 g/cc) | |
| CALCULATED POROSITY: | 47.73 (% vol) |
| INITIAL MOISTURE CONTENT (VOLUMETRIC): | 35.58 (% vol) |
| INITIAL MOISTURE CONTENT (GRAVIMETRIC): | 25.69 (%) |

COMMENTS:

| | |
|---|---|
| LABORATORY ANALYSIS PERFORMED BY: | W. Strong |
| CALCULATIONS MADE BY: | L. Simpson |
| CHECKED BY: | E. Mattson |

DATA FOR INITIAL MOISTURE CONTENT,
BULK DENSITY, AND POROSITY

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-9-30.5 (a) |
| RING NUMBER: | P11 |
| DEPTH: | 30.5 FT. |

| | |
|---|---|
| FIELD WEIGHT OF SAMPLE (W/CAP AND RING): | 134.67 (g) |
| TARE WEIGHT, RING: | 18.42 (g) |
| TARE WEIGHT, PAN: | 0.00 (g) |
| SAMPLE VOLUME: | 61.58 (cc) |
| DATE AND TIME INTO OVEN: | 8/14/89 @ 830 |
| DATE AND TIME OUT OF OVEN: | 8/16/89 @ 1200 |
| DRY WEIGHT OF SAMPLE: | 103.66 (g) |
| DRY BULK DENSITY: | 1.68 (g/cc) |
| PARTICLE DENSITY: | 2.65 (g/cc) |
| (METHOD: ASSUME MEAN PARTICLE DENSITY = 2.65 g/cc) | |
| CALCULATED POROSITY: | 36.48 (% vol) |
| INITIAL MOISTURE CONTENT (VOLUMETRIC): | 20.44 (% vol) |
| INITIAL MOISTURE CONTENT (GRAVIMETRIC): | 12.15 (%) |

COMMENTS:

| | |
|---|---|
| LABORATORY ANALYSIS PERFORMED BY: | W. Strong |
| CALCULATIONS MADE BY: | L. Simpson |
| CHECKED BY: | E. Mattson |

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DATA FOR INITIAL MOISTURE CONTENT,
BULK DENSITY, AND POROSITY

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-9-34.5 (b) |
| RING NUMBER: | P21 |
| DEPTH: | 34.5 FT. |

| | |
|---|---|
| FIELD WEIGHT OF SAMPLE (W/CAP AND RING): | 129.98 (g) |
| TARE WEIGHT, RING: | 17.96 (g) |
| TARE WEIGHT, PAN: | 0.00 (g) |
| SAMPLE VOLUME: | 59.11 (cc) |
| DATE AND TIME INTO OVEN: | 8/7/89 @ 1600 |
| DATE AND TIME OUT OF OVEN: | 8/9/89 @ 1500 |
| DRY WEIGHT OF SAMPLE: | 96.22 (g) |
| DRY BULK DENSITY: | 1.63 (g/cc) |
| PARTICLE DENSITY: | 2.65 (g/cc) |
| (METHOD: ASSUME MEAN PARTICLE DENSITY = 2.65 g/cc) | |
| CALCULATED POROSITY: | 38.57 (% vol) |
| INITIAL MOISTURE CONTENT (VOLUMETRIC): | 26.73 (% vol) |
| INITIAL MOISTURE CONTENT (GRAVIMETRIC): | 16.42 (%) |

COMMENTS:

| | |
|---|---|
| LABORATORY ANALYSIS PERFORMED BY: | W. Strong |
| CALCULATIONS MADE BY: | L. Simpson |
| CHECKED BY: | E. Mattson |

DATA FOR INITIAL MOISTURE CONTENT,
BULK DENSITY, AND POROSITY

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-9-39.0 (a) |
| RING NUMBER: | X7 |
| DEPTH: | 39.0 FT. |

| | |
|---|---|
| FIELD WEIGHT OF SAMPLE (W/CAP AND RING): | 181.01 (g) |
| TARE WEIGHT, RING: | 41.43 (g) |
| TARE WEIGHT, PAN: | 0.00 (g) |
| SAMPLE VOLUME: | 73.06 (cc) |
| DATE AND TIME INTO OVEN: | 8/7/89 @ 1600 |
| DATE AND TIME OUT OF OVEN: | 8/9/89 @ 1500 |
| DRY WEIGHT OF SAMPLE: | 116.65 (g) |
| DRY BULK DENSITY: | 1.60 (g/cc) |
| PARTICLE DENSITY: | 2.65 (g/cc) |
| (METHOD: ASSUME MEAN PARTICLE DENSITY = 2.65 g/cc) | |
| CALCULATED POROSITY: | 39.75 (% vol) |
| INITIAL MOISTURE CONTENT (VOLUMETRIC): | 31.39 (% vol) |
| INITIAL MOISTURE CONTENT (GRAVIMETRIC): | 19.66 (%) |

COMMENTS:

| | |
|---|---|
| LABORATORY ANALYSIS PERFORMED BY: | W. Strong |
| CALCULATIONS MADE BY: | L. Simpson |
| CHECKED BY: | E. Mattson |

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DATA FOR INITIAL MOISTURE CONTENT,
BULK DENSITY, AND POROSITY

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-9-49.5 |
| RING NUMBER: | X5 |
| DEPTH: | 49.5 FT. |

| | |
|---|---|
| FIELD WEIGHT OF SAMPLE (W/CAP AND RING): | 169.86 (g) |
| TARE WEIGHT, RING: | 40.74 (g) |
| TARE WEIGHT, PAN: | 0.00 (g) |
| SAMPLE VOLUME: | 73.06 (cc) |
| DATE AND TIME INTO OVEN: | 8/7/89 @ 1600 |
| DATE AND TIME OUT OF OVEN: | 8/9/89 @ 1500 |
| DRY WEIGHT OF SAMPLE: | 96.24 (g) |
| DRY BULK DENSITY: | 1.32 (g/cc) |
| PARTICLE DENSITY: | 2.65 (g/cc) |
| (METHOD: ASSUME MEAN PARTICLE DENSITY = 2.65 g/cc) | |
| CALCULATED POROSITY: | 50.29 (% vol) |
| INITIAL MOISTURE CONTENT (VOLUMETRIC): | 45.00 (% vol) |
| INITIAL MOISTURE CONTENT (GRAVIMETRIC): | 34.16 (%) |

COMMENTS:

| | |
|---|---|
| LABORATORY ANALYSIS PERFORMED BY: | W. Strong |
| CALCULATIONS MADE BY: | L. Simpson |
| CHECKED BY: | E. Mattson |

DATA FOR INITIAL MOISTURE CONTENT,
BULK DENSITY, AND POROSITY

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-9-59.5 (a) |
| RING NUMBER: | P20 |
| DEPTH: | 59.5 FT. |

| | |
|---|---|
| FIELD WEIGHT OF SAMPLE (W/CAP AND RING): | 134.52 (g) |
| TARE WEIGHT, RING: | 17.92 (g) |
| TARE WEIGHT, PAN: | 0.00 (g) |
| SAMPLE VOLUME: | 59.11 (cc) |
| DATE AND TIME INTO OVEN: | 8/7/89 @ 1600 |
| DATE AND TIME OUT OF OVEN: | 8/9/89 @ 1500 |
| DRY WEIGHT OF SAMPLE: | 98.06 (g) |
| DRY BULK DENSITY: | 1.66 (g/cc) |
| PARTICLE DENSITY: | 2.65 (g/cc) |
| (METHOD: ASSUME MEAN PARTICLE DENSITY = 2.65 g/cc) | |
| CALCULATED POROSITY: | 37.40 (% vol) |
| INITIAL MOISTURE CONTENT (VOLUMETRIC): | 31.37 (% vol) |
| INITIAL MOISTURE CONTENT (GRAVIMETRIC): | 18.91 (%) |

COMMENTS:

| | |
|---|---|
| LABORATORY ANALYSIS PERFORMED BY: | W. Strong |
| CALCULATIONS MADE BY: | L. Simpson |
| CHECKED BY: | E. Mattson |

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DATA FOR INITIAL MOISTURE CONTENT,
BULK DENSITY, AND POROSITY

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-9-69.5 (b) |
| RING NUMBER: | X8 |
| DEPTH: | 69.5 FT. |

| | |
|---|---|
| FIELD WEIGHT OF SAMPLE (W/CAP AND RING): | 175.43 (g) |
| TARE WEIGHT, RING: | 41.98 (g) |
| TARE WEIGHT, PAN: | 0.00 (g) |
| SAMPLE VOLUME: | 73.06 (cc) |
| DATE AND TIME INTO OVEN: | 8/14/89 @ 830 |
| DATE AND TIME OUT OF OVEN: | 8/16/89 @ 1200 |
| DRY WEIGHT OF SAMPLE: | 118.13 (g) |
| DRY BULK DENSITY: | 1.62 (g/cc) |
| PARTICLE DENSITY: | 2.65 (g/cc) |
| (METHOD: ASSUME MEAN PARTICLE DENSITY = 2.65 g/cc) | |
| CALCULATED POROSITY: | 38.99 (% vol) |
| INITIAL MOISTURE CONTENT (VOLUMETRIC): | 20.97 (% vol) |
| INITIAL MOISTURE CONTENT (GRAVIMETRIC): | 12.97 (%) |

COMMENTS:

| | |
|---|---|
| LABORATORY ANALYSIS PERFORMED BY: | W. Strong |
| CALCULATIONS MADE BY: | L. Simpson |
| CHECKED BY: | E. Mattson |

DATA FOR INITIAL MOISTURE CONTENT,
BULK DENSITY, AND POROSITY

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-9-103 (a) |
| RING NUMBER: | X1 |
| DEPTH: | 103.0 FT. |

| | |
|---|---|
| FIELD WEIGHT OF SAMPLE (W/CAP AND RING): | 175.15 (g) |
| TARE WEIGHT, RING: | 40.66 (g) |
| TARE WEIGHT, PAN: | 0.00 (g) |
| SAMPLE VOLUME: | 73.06 (cc) |
| DATE AND TIME INTO OVEN: | 8/7/89 @ 1600 |
| DATE AND TIME OUT OF OVEN: | 8/9/89 @ 1500 |
| DRY WEIGHT OF SAMPLE: | 106.45 (g) |
| DRY BULK DENSITY: | 1.46 (g/cc) |
| PARTICLE DENSITY: | 2.65 (g/cc) |
| (METHOD: ASSUME MEAN PARTICLE DENSITY = 2.65 g/cc) | |
| CALCULATED POROSITY: | 45.02 (% vol) |
| INITIAL MOISTURE CONTENT (VOLUMETRIC): | 38.38 (% vol) |
| INITIAL MOISTURE CONTENT (GRAVIMETRIC): | 26.34 (%) |

COMMENTS:

| | |
|---|---|
| LABORATORY ANALYSIS PERFORMED BY: | W. Strong |
| CALCULATIONS MADE BY: | L. Simpson |
| CHECKED BY: | E. Mattson |

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DATA FOR INITIAL MOISTURE CONTENT,
BULK DENSITY, AND POROSITY

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-22-23.5 |
| RING NUMBER: | P23 |
| DEPTH: | 23.5 FT. |

| | |
|---|---|
| FIELD WEIGHT OF SAMPLE (W/CAP AND RING): | 120.52 (g) |
| TARE WEIGHT, RING: | 18.05 (g) |
| TARE WEIGHT, PAN: | 0.00 (g) |
| SAMPLE VOLUME: | 59.11 (cc) |
| DATE AND TIME INTO OVEN: | 8/14/89 @ 830 |
| DATE AND TIME OUT OF OVEN: | 8/16/89 @ 1200 |
| DRY WEIGHT OF SAMPLE: | 94.25 (g) |
| DRY BULK DENSITY: | 1.59 (g/cc) |
| PARTICLE DENSITY: | 2.65 (g/cc) |
| (METHOD: ASSUME MEAN PARTICLE DENSITY = 2.65 g/cc) | |
| CALCULATED POROSITY: | 39.83 (% vol) |
| INITIAL MOISTURE CONTENT (VOLUMETRIC): | 13.91 (% vol) |
| INITIAL MOISTURE CONTENT (GRAVIMETRIC): | 8.72 (%) |

COMMENTS:

| | |
|---|---|
| LABORATORY ANALYSIS PERFORMED BY: | W. Strong |
| CALCULATIONS MADE BY: | L. Simpson |
| CHECKED BY: | E. Mattson |

DATA FOR INITIAL MOISTURE CONTENT,
BULK DENSITY, AND POROSITY

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-22-52.5 |
| RING NUMBER: | X6 |
| DEPTH: | 52.5 FT. |

| | |
|---|---|
| FIELD WEIGHT OF SAMPLE (W/CAP AND RING): | 187.05 (g) |
| TARE WEIGHT, RING: | 40.73 (g) |
| TARE WEIGHT, PAN: | 0.00 (g) |
| SAMPLE VOLUME: | 73.06 (cc) |
| DATE AND TIME INTO OVEN: | 8/7/89 @ 1600 |
| DATE AND TIME OUT OF OVEN: | 8/9/89 @ 1500 |
| DRY WEIGHT OF SAMPLE: | 120.08 (g) |
| DRY BULK DENSITY: | 1.64 (g/cc) |
| PARTICLE DENSITY: | 2.65 (g/cc) |
| (METHOD: ASSUME MEAN PARTICLE DENSITY = 2.65 g/cc) | |
| CALCULATED POROSITY: | 37.98 (% vol) |
| INITIAL MOISTURE CONTENT (VOLUMETRIC): | 35.92 (% vol) |
| INITIAL MOISTURE CONTENT (GRAVIMETRIC): | 21.85 (%) |

COMMENTS:

| | |
|---|---|
| LABORATORY ANALYSIS PERFORMED BY: | W. Strong |
| CALCULATIONS MADE BY: | L. Simpson |
| CHECKED BY: | E. Mattson |

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DATA FOR INITIAL MOISTURE CONTENT,
BULK DENSITY, AND POROSITY

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-22-54.0 |
| RING NUMBER: | X3 |
| DEPTH: | 54.0 FT. |

| | |
|---|---|
| FIELD WEIGHT OF SAMPLE (W/CAP AND RING): | 169.18 (g) |
| TARE WEIGHT, RING: | 41.17 (g) |
| TARE WEIGHT, PAN: | 0.00 (g) |
| SAMPLE VOLUME: | 73.06 (cc) |
| DATE AND TIME INTO OVEN: | 8/7/89 @ 1600 |
| DATE AND TIME OUT OF OVEN: | 8/9/89 @ 1500 |
| DRY WEIGHT OF SAMPLE: | 97.02 (g) |
| DRY BULK DENSITY: | 1.33 (g/cc) |
| PARTICLE DENSITY: | 2.65 (g/cc) |
| (METHOD: ASSUME MEAN PARTICLE DENSITY = 2.65 g/cc) | |
| CALCULATED POROSITY: | 49.89 (% vol) |
| INITIAL MOISTURE CONTENT (VOLUMETRIC): | 42.42 (% vol) |
| INITIAL MOISTURE CONTENT (GRAVIMETRIC): | 31.94 (%) |

COMMENTS:

| | |
|---|---|
| LABORATORY ANALYSIS PERFORMED BY: | W. Strong |
| CALCULATIONS MADE BY: | L. Simpson |
| CHECKED BY: | E. Mattson |

DATA FOR INITIAL MOISTURE CONTENT,
BULK DENSITY, AND POROSITY

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-22-59.0 |
| RING NUMBER: | X2 |
| DEPTH: | 59.0 FT. |

| | |
|---|---|
| FIELD WEIGHT OF SAMPLE (W/CAP AND RING): | 180.53 (g) |
| TARE WEIGHT, RING: | 41.32 (g) |
| TARE WEIGHT, PAN: | 0.00 (g) |
| SAMPLE VOLUME: | 73.06 (cc) |
| DATE AND TIME INTO OVEN: | 8/7/89 @ 1600 |
| DATE AND TIME OUT OF OVEN: | 8/9/89 @ 1500 |
| DRY WEIGHT OF SAMPLE: | 108.47 (g) |
| DRY BULK DENSITY: | 1.48 (g/cc) |
| PARTICLE DENSITY: | 2.65 (g/cc) |
| (METHOD: ASSUME MEAN PARTICLE DENSITY = 2.65 g/cc) | |
| CALCULATED POROSITY: | 43.97 (% vol) |
| INITIAL MOISTURE CONTENT (VOLUMETRIC): | 42.08 (% vol) |
| INITIAL MOISTURE CONTENT (GRAVIMETRIC): | 28.34 (%) |

COMMENTS:

| | |
|---|---|
| LABORATORY ANALYSIS PERFORMED BY: | W. Strong |
| CALCULATIONS MADE BY: | L. Simpson |
| CHECKED BY: | E. Mattson |

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DATA FOR INITIAL MOISTURE CONTENT,
BULK DENSITY, AND POROSITY

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-22-68.0 |
| RING NUMBER: | P12 |
| DEPTH: | 68.0 FT. |

| | |
|---|---|
| FIELD WEIGHT OF SAMPLE (W/CAP AND RING): | 131.40 (g) |
| TARE WEIGHT, RING: | 18.36 (g) |
| TARE WEIGHT, PAN: | 0.00 (g) |
| SAMPLE VOLUME: | 59.11 (cc) |
| DATE AND TIME INTO OVEN: | 8/7/89 @ 1600 |
| DATE AND TIME OUT OF OVEN: | 8/9/89 @ 1500 |
| DRY WEIGHT OF SAMPLE: | 87.84 (g) |
| DRY BULK DENSITY: | 1.49 (g/cc) |
| PARTICLE DENSITY: | 2.65 (g/cc) |
| (METHOD: ASSUME MEAN PARTICLE DENSITY = 2.65 g/cc) | |
| CALCULATED POROSITY: | 43.92 (% vol) |
| INITIAL MOISTURE CONTENT (VOLUMETRIC): | 42.63 (% vol) |
| INITIAL MOISTURE CONTENT (GRAVIMETRIC): | 28.69 (%) |

COMMENTS:

| | |
|---|---|
| LABORATORY ANALYSIS PERFORMED BY: | W. Strong |
| CALCULATIONS MADE BY: | L. Simpson |
| CHECKED BY: | E. Mattson |

DATA FOR_INITIAL MOISTURE CONTENT,
BULK DENSITY, AND POROSITY

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-22-77.5 |
| RING NUMBER: | X4 |
| DEPTH: | 77.5 FT. |

| | |
|---|---|
| FIELD WEIGHT OF SAMPLE (W/CAP AND RING): | 181.74 (g) |
| TARE WEIGHT, RING: | 42.03 (g) |
| TARE WEIGHT, PAN: | 0.00 (g) |
| SAMPLE VOLUME: | 73.06 (cc) |
| DATE AND TIME INTO OVEN: | 8/7/89 @ 1600 |
| DATE AND TIME OUT OF OVEN: | 8/9/89 @ 1500 |
| DRY WEIGHT OF SAMPLE: | 105.18 (g) |
| DRY BULK DENSITY: | 1.44 (g/cc) |
| PARTICLE DENSITY: | 2.65 (g/cc) |
| (METHOD: ASSUME MEAN PARTICLE DENSITY = 2.65 g/cc) | |
| CALCULATED POROSITY: | 45.67 (% vol) |
| INITIAL MOISTURE CONTENT (VOLUMETRIC): | 47.26 (% vol) |
| INITIAL MOISTURE CONTENT (GRAVIMETRIC): | 32.83 (%) |

COMMENTS:

| | |
|---|---|
| LABORATORY ANALYSIS PERFORMED BY: | W. Strong |
| CALCULATIONS MADE BY: | L. Simpson |
| CHECKED BY: | E. Mattson |

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# Appendix B: SATURATED HYDRAULIC CONDUCTIVITY

## Summary of Saturated Hydraulic Conductivity Tests

| Sample No. | $K_s$(cm/sec) | Method of Analysis: Constant Head | Method of Analysis: Falling Head |
|---|---|---|---|
| SHC-5-79.5 | 2.32E-04 | X | |
| SHC-9-30.5(a) | 5.37E-04 | X | |
| SHC-9-34.5(b) | 2.87E-04 | X | |
| SHC-9-39.0(a) | 3.39E-05 | | X |
| SHC-9-49.5 | 5.62E-07 | | X |
| SHC-9-59.5(a) | 1.17E-05 | X | |
| SHC-9-69.5(b) | 2.66E-04 | X | |
| SHC-9-103.0(a) | 2.08E-05 | | X |
| SHC-22-23.5 | 1.94E-03 | X | |
| SHC-22-52.5 | 5.40E-06 | | X |
| SHC-22-54.0 | 9.29E-06 | | X |
| SHC-22-59.0 | 3.50E-06 | | X |
| SHC-22-68.0 | 1.68E-05 | | X |
| SHC-22-77.5 | 2.58E-06 | | X |

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CONSTANT HEAD PERMEAMETER DATA

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-5-79.5 |
| RING NUMBER: | P27 |
| DEPTH: | 79.5 FT. |
| TYPE OF WATER USED: | 0.01N CaCl2 SOLUTION |
| SAMPLE LENGTH: | 2.4 (cm) |
| SAMPLE RADIUS: | 2.8 (cm) |
| SAMPLE X-SECTIONAL AREA: | 24.63 (sq. cm) |

| DATE (1989) | TIME (DAY) | TEMP ( C) | HEAD CHANGE (CM) | FLOW VOL(CC) | ELAPSED TIME(SEC) | K SAT (CM/SEC) | K SAT @ 20 (CM/SEC) |
|---|---|---|---|---|---|---|---|
| 7/12 | 1154 | 23.0 | 2.4 | 4.0 | 931 | 1.74E-04 | 1.63E-04 |
| 7/13 | 1021 | 23.0 | 2.5 | 3.4 | 547 | 2.42E-04 | 2.26E-04 |
| 7/14 | 1417 | 24.0 | 2.4 | 2.1 | 336 | 2.54E-04 | 2.32E-04 |
| 7/17 | 1218 | 23.0 | 2.4 | 18.5 | 445 | 1.69E-03 | 1.58E-03 |

COMMENTS:

| | |
|---|---|
| LABORATORY ANALYSIS BY: | S. Stoller |
| CALCULATIONS MADE BY: | S. Stoller |
| CHECKED BY: | E. Mattson |

CONSTANT HEAD PERMEAMETER DATA

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-9-30.5 (b) |
| RING NUMBER: | P11 |
| DEPTH: | 30.5 FT. |
| TYPE OF WATER USED: | 0.01N CaCl2 SOLUTION |
| SAMPLE LENGTH: | 2.5 (cm) |
| SAMPLE RADIUS: | 2.8 (cm) |
| SAMPLE X-SECTIONAL AREA: | 24.63 (sq. cm) |

| DATE (1989) | TIME (DAY) | TEMP ( C) | HEAD CHANGE (CM) | FLOW VOL(CC) | ELAPSED TIME(SEC) | K SAT (CM/SEC) | K SAT @ 20 (CM/SEC) |
|---|---|---|---|---|---|---|---|
| 7/12 | 1152 | 23.0 | 1.9 | 7.5 | 1120 | 3.58E-04 | 3.34E-04 |
| 7/13 | 1020 | 23.0 | 1.9 | 8.0 | 573 | 7.46E-04 | 6.97E-04 |
| 7/14 | 1416 | 24.0 | 1.9 | 3.8 | 345 | 5.88E-04 | 5.37E-04 |
| 7/17 | 1217 | 23.0 | 1.9 | 5.5 | 7702 | 3.81E-05 | 3.56E-05 |

COMMENTS:

| | |
|---|---|
| LABORATORY ANALYSIS BY: | S. Stoller |
| CALCULATIONS MADE BY: | S. Stoller |
| CHECKED BY: | E. Mattson |

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CONSTANT HEAD PERMEAMETER DATA

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-9-34.5 (b) |
| RING NUMBER: | P21 |
| DEPTH: | 34.5 FT. |
| TYPE OF WATER USED: | 0.01N CaCl2 SOLUTION |
| SAMPLE LENGTH: | 2.4 (cm) |
| SAMPLE RADIUS: | 2.8 (cm) |
| SAMPLE X-SECTIONAL AREA: | 24.63 (sq. cm) |

| DATE (1989) | TIME (DAY) | TEMP ( C) | HEAD CHANGE (CM) | FLOW VOL(CC) | ELAPSED TIME(SEC) | K SAT (CM/SEC) | K SAT @ 20 (CM/SEC) |
|---|---|---|---|---|---|---|---|
| 7/12 | 1201 | 23.0 | 2.2 | 4.6 | 8260 | 2.47E-05 | 2.30E-05 |
| 7/13 | 1020 | 23.0 | 2.3 | 3.0 | 8260 | 1.54E-05 | 1.44E-05 |
| 7/14 | 1416 | 24.0 | 2.2 | 3.8 | 5346 | 3.15E-05 | 2.87E-05 |
| 7/17 | 1217 | 23.0 | 2.2 | 11.0 | 18924 | 2.57E-05 | 2.41E-05 |

COMMENTS:

| | |
|---|---|
| LABORATORY ANALYSIS BY: | S. Stoller |
| CALCULATIONS MADE BY: | S. Stoller |
| CHECKED BY: | E. Mattson |

FALLING HEAD TEST DATA

JOB NAME: EMCON
JOB NUMBER: 89-L-060
SAMPLE NUMBER: SHC-9-39.0 (a)
RING NUMBER: X7
DEPTH: 39.0 FT.
TYPE OF WATER USED: 0.01N CaCl2 SOLUTION
SAMPLE X-SECTION AREA: 29.22 (sq. cm)
STANDPIPE X-SECTION AREA: 0.709 (sq. cm)
SAMPLE LENGTH: 2.5 (cm)

| DATE (1989) | TIME (DAY) | DEL T (SEC) | TEMP ( C) | RESERVOIR HEAD(CM) | SAMPLE HEAD(CM) | K SAT (CM/SEC) | K SAT @ 20 C (CM/SEC) |
|---|---|---|---|---|---|---|---|
| TEST # 1: | | | | | | | |
| 7/12 | 1236.55 | | 21.0 | 0.0 | 86.2 | | |
| 7/12 | 1418.20 | 6085 | 22.0 | 0.0 | 5.8 | 2.69E-05 | 2.60E-05 |
| TEST # 2: | | | | | | | |
| 7/13 | 1027.17 | | 21.5 | 0.0 | 87.0 | | |
| 7/13 | 1131.36 | 3859 | 21.5 | 0.0 | 11.2 | 3.22E-05 | 3.11E-05 |
| TEST # 3: | | | | | | | |
| 7/14 | 1427.55 | | 23.0 | 0.0 | 86.5 | | |
| 7/14 | 1542.33 | 4478 | 23.0 | 0.0 | 4.8 | 3.92E-05 | 3.66E-05 |

AVERAGE K SAT: 3.39E-05 (CM/SEC)

COMMENTS: Average Ksat was taken to be the average of the last 2 readings.

LABORATORY ANALYSIS BY: S. Stoller
CALCULATIONS MADE BY: S. Stoller
CHECKED BY: E. Mattson

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FALLING HEAD TEST DATA

JOB NAME: EMCON
JOB NUMBER: 89-L-060
SAMPLE NUMBER: SHC-9-49.5
RING NUMBER: X5
DEPTH: 49.5 FT.
TYPE OF WATER USED: 0.01N CaCl2 SOLUTION
SAMPLE X-SECTION AREA: 29.22 (sq. cm)
STANDPIPE X-SECTION AREA: 0.709 (sq. cm)
SAMPLE LENGTH: 2.5 (cm)

| DATE (1989) | TIME (DAY) | DEL T (SEC) | TEMP ( C) | RESERVOIR HEAD(CM) | SAMPLE HEAD(CM) | K SAT (CM/SEC) | K SAT @ 20 C (CM/SEC) |
|---|---|---|---|---|---|---|---|
| TEST # 1: | | | | | | | |
| 7/12 | 1238.30 | | 21.0 | 0.0 | 87.1 | | |
| 7/13 | 800.55 | 69745 | 21.0 | 0.0 | 21.1 | 1.23E-06 | 1.20E-06 |
| TEST # 2: | | | | | | | |
| 7/13 | 1028.40 | | 21.5 | 0.0 | 92.2 | | |
| 7/14 | 1423.55 | 100515 | 23.0 | 0.0 | 32.7 | 6.26E-07 | 5.94E-07 |
| TEST # 3: | | | | | | | |
| 7/17 | 1224.00 | | 21.5 | 0.0 | 63.1 | | |
| 7/18 | 909.00 | 74700 | 21.0 | 0.0 | 32.2 | 5.46E-07 | 5.31E-07 |

AVERAGE K SAT: 5.62E-07 (CM/SEC)

COMMENTS: Average Ksat was taken to be the average of the last 2 readings.

LABORATORY ANALYSIS BY: S. Stoller
CALCULATIONS MADE BY: S. Stoller
CHECKED BY: E. Mattson

CONSTANT HEAD PERMEAMETER DATA

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-9-59.5 (a) |
| RING NUMBER: | P20 |
| DEPTH: | 59.5 FT. |
| TYPE OF WATER USED: | 0.01N CaCl2 SOLUTION |
| SAMPLE LENGTH: | 2.4 (cm) |
| SAMPLE RADIUS: | 2.8 (cm) |
| SAMPLE X-SECTIONAL AREA: | 24.63 (sq. cm) |

| DATE (1989) | TIME (DAY) | TEMP ( C) | HEAD CHANGE (CM) | FLOW VOL(CC) | ELAPSED TIME(SEC) | K SAT (CM/SEC) | K SAT @ 20 (CM/SEC) |
|---|---|---|---|---|---|---|---|
| 7/12 | 1153 | 23.0 | 2.0 | 1.6 | 8750 | 8.91E-06 | 8.32E-06 |
| 7/13 | 1021 | 23.0 | 2.0 | 1.4 | 7513 | 9.08E-06 | 8.48E-06 |
| 7/14 | 1417 | 24.0 | 2.0 | 1.4 | 5342 | 1.28E-05 | 1.17E-05 |
| 7/17 | 1218 | 23.0 | 2.0 | 3.6 | 18927 | 9.27E-06 | 8.66E-06 |

COMMENTS:

| | |
|---|---|
| LABORATORY ANALYSIS BY: | S. Stoller |
| CALCULATIONS MADE BY: | S. Stoller |
| CHECKED BY: | E. Mattson |

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CONSTANT HEAD PERMEAMETER DATA

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-9-69.5 (b) |
| RING NUMBER: | X8 |
| DEPTH: | 69.5 FT. |
| TYPE OF WATER USED: | 0.01N CaCl2 SOLUTION |
| SAMPLE LENGTH: | 2.5 (cm) |
| SAMPLE RADIUS: | 3.05 (cm) |
| SAMPLE X-SECTIONAL AREA: | 29.22 (sq. cm) |

| DATE (1989) | TIME (DAY) | TEMP ( C) | HEAD CHANGE (CM) | FLOW VOL(CC) | ELAPSED TIME(SEC) | K SAT (CM/SEC) | K SAT @ 20 (CM/SEC) |
|---|---|---|---|---|---|---|---|
| 7/12 | 1152 | 23.0 | 2.4 | 6.7 | 1056 | 2.26E-04 | 2.11E-04 |
| 7/13 | 1021 | 23.0 | 2.4 | 4.6 | 607 | 2.70E-04 | 2.52E-04 |
| 7/14 | 1417 | 24.0 | 2.4 | 2.8 | 343 | 2.91E-04 | 2.66E-04 |
| 7/17 | 1217 | 23.0 | 2.4 | 3.4 | 474 | 2.56E-04 | 2.39E-04 |

COMMENTS:

| | |
|---|---|
| LABORATORY ANALYSIS BY: | S. Stoller |
| CALCULATIONS MADE BY: | S. Stoller |
| CHECKED BY: | E. Mattson |

FALLING HEAD TEST DATA

JOB NAME: EMCON
JOB NUMBER: 89-L-060
SAMPLE NUMBER: SHC-9-103.0 (a)
RING NUMBER: X1
DEPTH: 103.0 FT.
TYPE OF WATER USED: 0.01N CaCl2 SOLUTION
SAMPLE X-SECTION AREA: 29.22 (sq. cm)
STANDPIPE X-SECTION AREA: 0.709 (sq. cm)
SAMPLE LENGTH: 2.5 (cm)

| DATE (1989) | TIME (DAY) | DEL T (SEC) | TEMP ( C) | RESERVOIR HEAD(CM) | SAMPLE HEAD(CM) | K SAT (CM/SEC) | K SAT @ 20 C (CM/SEC) |
|---|---|---|---|---|---|---|---|
| TEST # 1: | | | | | | | |
| 7/12 | 1238.00 | | 21.0 | 0.0 | 87.0 | | |
| 7/12 | 1500.18 | 8538 | 22.0 | 0.0 | 15.6 | 1.22E-05 | 1.18E-05 |
| TEST # 2: | | | | | | | |
| 7/13 | 1028.05 | | 21.5 | 0.0 | 84.0 | | |
| 7/13 | 1227.24 | 7159 | 21.5 | 0.0 | 7.6 | 2.04E-05 | 1.97E-05 |
| TEST # 3: | | | | | | | |
| 7/14 | 1428.32 | | 23.0 | 0.0 | 86.1 | | |
| 7/14 | 1542.59 | 4467 | 23.0 | 0.0 | 15.3 | 2.35E-05 | 2.19E-05 |

AVERAGE K SAT: 2.08E-05 (CM/SEC)

COMMENTS: Average Ksat was taken to be the average of the last 2 readings.

LABORATORY ANALYSIS BY: S. Stoller
CALCULATIONS MADE BY: S. Stoller
CHECKED BY: E. Mattson

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CONSTANT HEAD_PERMEAMETER DATA

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-22-23.5 |
| RING NUMBER: | P23 |
| DEPTH: | 23.5 FT. |
| TYPE OF WATER USED: | 0.01N CaCl2 SOLUTION |
| SAMPLE LENGTH: | 2.4 (cm) |
| SAMPLE RADIUS: | 2.8 (cm) |
| SAMPLE X-SECTIONAL AREA: | 24.63 (sq. cm) |

| DATE (1989) | TIME (DAY) | TEMP ( C) | HEAD CHANGE (CM) | FLOW VOL(CC) | ELAPSED TIME(SEC) | K SAT (CM/SEC) | K SAT @ 20 (CM/SEC) |
|---|---|---|---|---|---|---|---|
| 7/12 | 1154 | 23.0 | 2.3 | 14.6 | 168 | 3.68E-03 | 3.44E-03 |
| 7/13 | 1021 | 23.0 | 2.3 | 8.6 | 133 | 2.74E-03 | 2.56E-03 |
| 7/14 | 1421 | 24.0 | 2.2 | 10.2 | 213 | 2.12E-03 | 1.94E-03 |
| 7/17 | 1218 | 23.0 | 2.3 | 15.5 | 464 | 1.40E-03 | 1.31E-03 |

COMMENTS:

| | |
|---|---|
| LABORATORY ANALYSIS BY: | S. Stoller |
| CALCULATIONS MADE BY: | S. Stoller |
| CHECKED BY: | E. Mattson |

FALLING HEAD TEST DATA

JOB NAME: EMCON
JOB NUMBER: 89-L-060
SAMPLE NUMBER: SHC-22-52.5
RING NUMBER: X6
DEPTH: 52.5 FT.
TYPE OF WATER USED: 0.01N CaCl2 SOLUTION
SAMPLE X-SECTION AREA: 29.22 (sq. cm)
STANDPIPE X-SECTION AREA: 0.709 (sq. cm)
SAMPLE LENGTH: 2.5 (cm)

| DATE (1989) | TIME (DAY) | DEL T (SEC) | TEMP ( C) | RESERVOIR HEAD(CM) | SAMPLE HEAD(CM) | K SAT (CM/SEC) | K SAT @ 20 C (CM/SEC) |
|---|---|---|---|---|---|---|---|
| TEST # 1: | | | | | | | |
| 7/12 | 1238.57 | | 21.0 | 0.0 | 90.9 | | |
| 7/12 | 1708.20 | 16163 | 22.0 | 0.0 | 30.1 | 4.15E-06 | 4.01E-06 |
| TEST # 2: | | | | | | | |
| 7/13 | 1028.59 | | 21.5 | 0.0 | 88.0 | | |
| 7/13 | 1613.10 | 20651 | 22.0 | 0.0 | 12.9 | 5.64E-06 | 5.42E-06 |
| TEST # 3: | | | | | | | |
| 7/14 | 1429.30 | | 23.0 | 0.0 | 93.0 | | |
| 7/14 | 1704.25 | 9475 | 23.0 | 0.0 | 37.8 | 5.76E-06 | 5.38E-06 |

AVERAGE K SAT: 5.40E-06 (CM/SEC)

COMMENTS: Average Ksat was taken to be the average of the last 2 readings.

LABORATORY ANALYSIS BY: S. Stoller
CALCULATIONS MADE BY: S. Stoller
CHECKED BY: E. Mattson

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FALLING HEAD TEST DATA

JOB NAME: EMCON
JOB NUMBER: 89-L-060
SAMPLE NUMBER: SHC-22-54.0
RING NUMBER: X3
DEPTH: 54.0 FT.
TYPE OF WATER USED: 0.01N CaCl2 SOLUTION
SAMPLE X-SECTION AREA: 29.22 (sq. cm)
STANDPIPE X-SECTION AREA: 0.709 (sq. cm)
SAMPLE LENGTH: 2.5 (cm)

| DATE (1989) | TIME (DAY) | DEL T (SEC) | TEMP ( C) | RESERVOIR HEAD(CM) | SAMPLE HEAD(CM) | K SAT (CM/SEC) | K SAT @ 20 C (CM/SEC) |
|---|---|---|---|---|---|---|---|
| TEST # 1: | | | | | | | |
| 7/12 | 1239.57 | | 21.0 | 0.0 | 85.4 | | |
| 7/12 | 1708.40 | 16123 | 22.0 | 0.0 | 31.0 | 3.81E-06 | 3.68E-06 |
| TEST # 2: | | | | | | | |
| 7/13 | 1029.42 | | 21.5 | 0.0 | 83.4 | | |
| 7/13 | 1612.20 | 20588 | 22.0 | 0.0 | 3.0 | 9.80E-06 | 9.41E-06 |
| TEST # 3: | | | | | | | |
| 7/14 | 1430.30 | | 23.0 | 0.0 | 78.5 | | |
| 7/14 | 1705.02 | 9272 | 23.0 | 0.0 | 17.5 | 9.82E-06 | 9.17E-06 |

AVERAGE K SAT: 9.29E-06 (CM/SEC)

COMMENTS: Average Ksat was taken to be the average of the last 2 readings.

LABORATORY ANALYSIS BY: S. Stoller
CALCULATIONS MADE BY: S. Stoller
CHECKED BY: E. Mattson

FALLING HEAD TEST DATA

JOB NAME: EMCON
JOB NUMBER: 89-L-060
SAMPLE NUMBER: SHC-22-59.0
RING NUMBER: X2
DEPTH: 59.0 FT.
TYPE OF WATER USED: 0.01N CaCl2 SOLUTION
SAMPLE X-SECTION AREA: 29.22 (sq. cm)
STANDPIPE X-SECTION AREA: 0.709 (sq. cm)
SAMPLE LENGTH: 2.5 (cm)

| DATE (1989) | TIME (DAY) | DEL T (SEC) | TEMP ( C) | RESERVOIR HEAD(CM) | SAMPLE HEAD(CM) | K SAT (CM/SEC) | K SAT @ 20 C (CM/SEC) |
|---|---|---|---|---|---|---|---|
| TEST # 1: | | | | | | | |
| 7/12 | 1239.32 | | 21.0 | 0.0 | 91.2 | | |
| 7/13 | 800.27 | 69655 | 21.0 | 0.0 | 8.2 | 2.10E-06 | 2.05E-06 |
| TEST # 2: | | | | | | | |
| 7/13 | 1029.20 | | 21.5 | 0.0 | 93.0 | | |
| 7/13 | 1612.50 | 20610 | 22.0 | 0.0 | 29.2 | 3.41E-06 | 3.28E-06 |
| TEST # 3: | | | | | | | |
| 7/14 | 1430.06 | | 23.0 | 0.0 | 84.5 | | |
| 7/14 | 1704.45 | 9279 | 23.0 | 0.0 | 45.9 | 3.99E-06 | 3.73E-06 |

AVERAGE K SAT: 3.50E-06 (CM/SEC)

COMMENTS: Average Ksat was taken to be the average of the last 2 readings.

LABORATORY ANALYSIS BY: S. Stoller
CALCULATIONS MADE BY: S. Stoller
CHECKED BY: E. Mattson

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FALLING HEAD TEST DATA

JOB NAME: EMCON
JOB NUMBER: 89-L-060
SAMPLE NUMBER: SHC-22-68.0
RING NUMBER: P12
DEPTH: 68.0 FT.
TYPE OF WATER USED: 0.01N CaCl2 SOLUTION
SAMPLE X-SECTION AREA: 24.63 (sq. cm)
STANDPIPE X-SECTION AREA: 0.709 (sq. cm)
SAMPLE LENGTH: 2.4 (cm)

| DATE (1989) | TIME (DAY) | DEL T (SEC) | TEMP ( C) | RESERVOIR HEAD(CM) | SAMPLE HEAD(CM) | K SAT (CM/SEC) | K SAT @ 20 C (CM/SEC) |
|---|---|---|---|---|---|---|---|
| TEST # 1: | | | | | | | |
| 7/12 | 1237.30 | | 21.0 | 0.0 | 86.6 | | |
| 7/12 | 1459.54 | 8544 | 22.0 | 0.0 | 11.0 | 1.67E-05 | 1.61E-05 |
| TEST # 2: | | | | | | | |
| 7/13 | 1027.47 | | 21.5 | 0.0 | 88.0 | | |
| 7/13 | 1227.02 | 7155 | 21.5 | 0.0 | 13.6 | 1.80E-05 | 1.74E-05 |

AVERAGE K SAT: 1.68E-05 (CM/SEC)

COMMENTS:

LABORATORY ANALYSIS BY: S. Stoller
CALCULATIONS MADE BY: S. Stoller
CHECKED BY: E. Mattson

FALLING HEAD TEST DATA

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-22-77.5 |
| RING NUMBER: | X4 |
| DEPTH: | 77.5 FT. |
| TYPE OF WATER USED: | 0.01N CaCl2 SOLUTION |
| SAMPLE X-SECTION AREA: | 29.22 (sq. cm) |
| STANDPIPE X-SECTION AREA: | 0.709 (sq. cm) |
| SAMPLE LENGTH: | 2.5 (cm) |

| DATE (1989) | TIME (DAY) | DEL T (SEC) | TEMP ( C) | RESERVOIR HEAD(CM) | SAMPLE HEAD(CM) | K SAT (CM/SEC) | K SAT @ 20 C (CM/SEC) |
|---|---|---|---|---|---|---|---|
| TEST # 1: | | | | | | | |
| 7/12 | 1240.27 | | 21.0 | 0.0 | 86.7 | | |
| 7/13 | 800.00 | 69573 | 21.0 | 0.0 | 5.0 | 2.49E-06 | 2.43E-06 |
| TEST # 2: | | | | | | | |
| 7/13 | 1029.57 | | 21.5 | 0.0 | 87.5 | | |
| 7/13 | 1740.36 | 25835 | 22.0 | 0.0 | 26.0 | 2.85E-06 | 2.74E-06 |

AVERAGE K SAT: 2.58E-06 (CM/SEC)

COMMENTS:

| | |
|---|---|
| LABORATORY ANALYSIS BY: | S. Stoller |
| CALCULATIONS MADE BY: | S. Stoller |
| CHECKED BY: | E. Mattson |

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# Appendix C: MOISTURE CHARACTERISTICS

## Summary of Moisture Characteristics

| Sample No. | Pressure Head (-cm of water) | Moisture Content (% $cm^3/cm^3$) |
|---|---|---|
| SHC-5-79.5 | 0.0 | 54.64 |
| | 43.5 | 41.41 |
| | 102.0 | 37.73 |
| | 195.5 | 34.99 |
| | 509.9 | 30.01 |
| | 1019.8 | 26.61 |
| | 5099.0 | 25.49 |
| SHC-9-30.5(a) | 0.0 | 45.40 |
| | 44.0 | 32.80 |
| | 99.0 | 27.98 |
| | 191.5 | 24.78 |
| | 509.9 | 20.77 |
| | 1019.8 | 18.92 |
| | 5099.0 | 18.93 |
| SHC-9-34.5(b) | 0.0 | 50.67 |
| | 1019.8 | 26.27 |
| | 5099.0 | 23.02 |
| | 15297.0 | 17.51 |
| SHC-9-39.0(a) | 0.0 | 45.50 |
| | 1019.8 | 26.83 |
| | 5099.0 | 22.49 |
| | 15297.0 | 18.70 |
| SHC-9-49.5 | 0.0 | 56.62 |
| | 1019.8 | 48.77 |
| | 5099.0 | 44.39 |
| | 15297.0 | 41.24 |
| SHC-9-59.5(a) | 0.0 | 48.50 |
| | 1019.8 | 33.01 |
| | 5099.0 | 27.47 |
| | 15297.0 | 17.73 |
| SHC-9-69.5(b) | 0.0 | 55.37 |
| | 45.5 | 40.37 |
| | 100.0 | 32.75 |
| | 192.5 | 27.74 |
| | 509.9 | 21.98 |
| | 1019.8 | 19.95 |
| | 5099.0 | 18.52 |

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Summary of Moisture Characteristics (Continued)

| Sample No. | Pressure Head (-cm of water) | Moisture Content (% $cm^3/cm^3$) |
|---|---|---|
| SHC-9-103.0(a) | 0.0 | 50.71 |
| | 1019.8 | 35.90 |
| | 5099.0 | 29.39 |
| | 15297.0 | 27.17 |
| SHC-22-23.5 | 0.0 | 48.35 |
| | 41.5 | 25.60 |
| | 94.0 | 18.22 |
| | 186.0 | 15.16 |
| | 509.9 | 12.18 |
| | 1019.8 | 11.13 |
| | 5099.0 | 10.12 |
| SHC-22-52.5 | 0.0 | 44.51 |
| | 1019.8 | 32.25 |
| | 5099.0 | 26.43 |
| | 15297.0 | 23.41 |
| SHC-22-54.0 | 0.0 | 55.00 |
| | 1019.8 | 43.61 |
| | 5099.0 | 36.74 |
| | 15297.0 | 28.95 |
| SHC-22-59.0 | 0.0 | 50.26 |
| | 1019.8 | 37.23 |
| | 5099.0 | 31.03 |
| | 15297.0 | 22.95 |
| SHC-22-68.0 | 0.0 | 59.80 |
| | 1019.8 | 35.68 |
| | 5099.0 | 26.19 |
| | 15297.0 | 18.64 |
| SHC-22-77.5 | 0.0 | 53.41 |
| | 1019.8 | 45.36 |
| | 5099.0 | 36.93 |
| | 15297.0 | 29.77 |

MOISTURE RETENTION DATA - HANGING COLUMN
(PORE SIZE DISTRIBUTION)

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-5-79.5 |
| RING NUMBER: | P27 |
| DEPTH: | 79.5 FT. |
| SAMPLE VOLUME: | 59.11 (cc) |
| SATURATED WEIGHT AT 0 CM TENSION (WITH CAP AND RING): | 132.06 (g) |
| TARE RING: | 17.89 (g) |
| TARE CAP: | 0.00 (g) |
| DRY WEIGHT OF SAMPLE: | 81.87 (g) |
| SATURATED MOISTURE CONTENT: | 54.64 (% vol) |
| INITIAL VOLUME OF WATER IN SAMPLE: | 32.30 (cc) |

| DATE (1989) | TIME | SUCTION (CM) | SAMPLE WT (G) | CHANGE WT (G) | CHANGES WT (G) | MOISTURE CONTENT(% VOL) DRYING | WETTING |
|---|---|---|---|---|---|---|---|
| 7/8 | 1515 | 0.0 | 132.06 | -- | -- | 54.64 | 0.00 |
| 7/23 | 1715 | 43.5 | 124.24 | 7.82 | 7.82 | 41.41 | 0.00 |
| 7/27 | 1600 | 102.0 | 122.06 | 2.18 | 10.00 | 37.73 | 0.00 |
| 7/31 | 1130 | 195.5 | 120.44 | 1.62 | 11.62 | 34.99 | 0.00 |

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: W. Strong
CALCULATION MADE BY: L. Simpson
CHECKED BY: E. Mattson

Printed on Recycled Paper

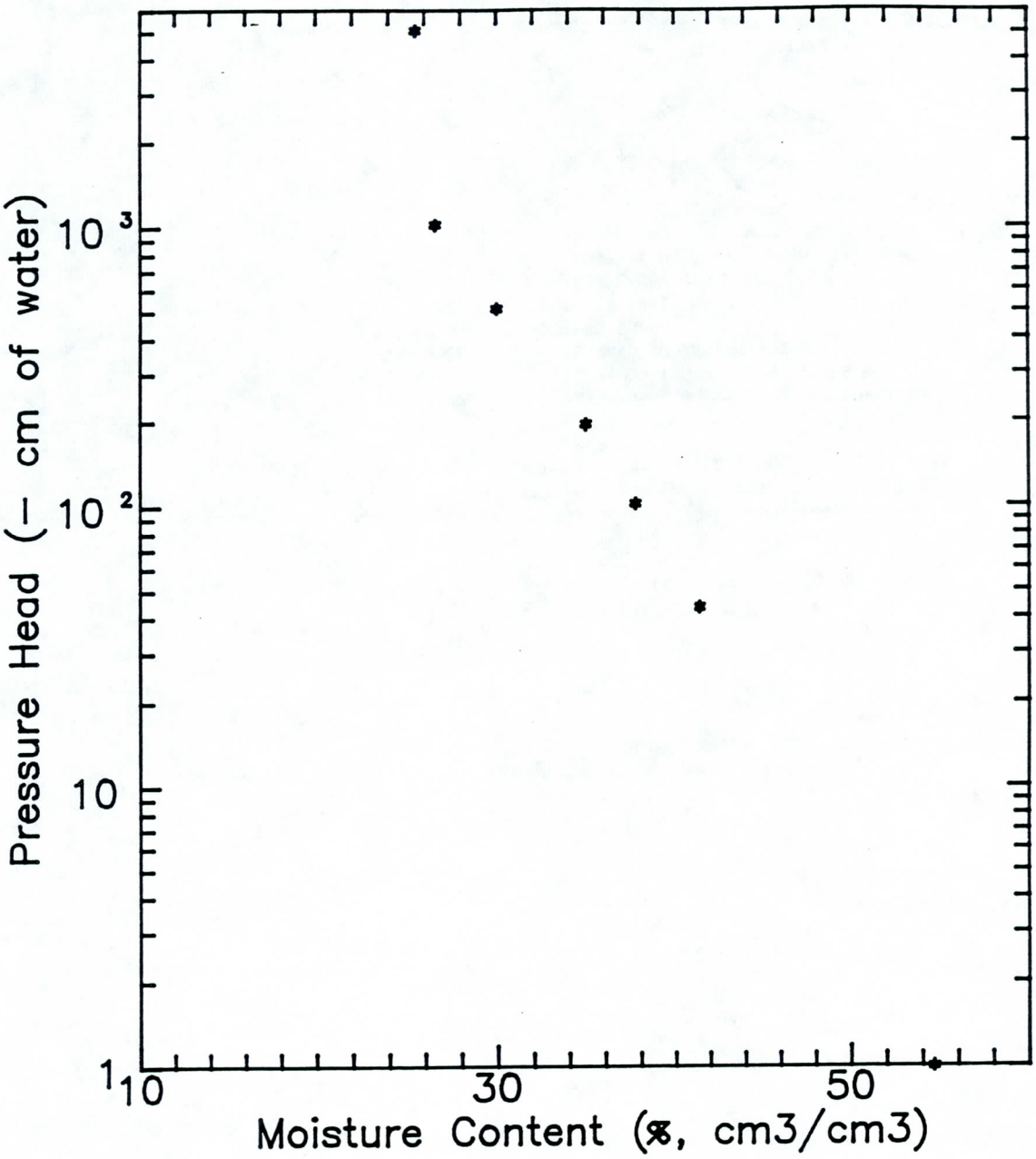
MOISTURE RETENTION DATA - 15 BAR PRESSURE PLATE
(PORE SIZE DISTRIBUTION)

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-5-79.5 |
| RING NUMBER: | P27 |
| DEPTH: | 79.5 FT. |
| SAMPLE VOLUME: | 59.11 (cc) |
| SATURATED WEIGHT AT 0 CM TENSION (WITH CAP AND RING): | 132.06 (g) |
| TARE RING: | 17.89 (g) |
| TARE CAP: | 0.00 (g) |
| DRY WEIGHT OF SAMPLE: | 81.87 (g) |
| SATURATED MOISTURE CONTENT: | 54.64 (% vol) |
| INITIAL VOLUME OF WATER IN SAMPLE: | 20.68 (cc) |
| WEIGHT FROM HANGING COLUMN, W/O CAP: | 120.44 (g) |
| FINAL TENSION ON HANGING COLUMN: | 195.5 (cm) |

| DATE (1988) | TIME | PRESSURE (BAR) | WEIGHT W/RING(G) | CHANGE WT (G) | CHANGES WT (G) | MOISTURE CONTENT (% VOL) |
|---|---|---|---|---|---|---|
| 7/31 | 1200 | 0.0 | 120.44 | -- | -- | -- |
| 8/4 | 1310 | 0.5 | 117.50 | 2.94 | 2.94 | 30.01 |
| 8/9 | 1615 | 1.0 | 115.49 | 2.01 | 4.95 | 26.61 |
| 8/14 | 800 | 5.0 | 114.83 | 0.66 | 5.61 | 25.49 |

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: W. Strong
CALCULATION MADE BY: L. Simpson
CHECKED BY: E. Mattson




**Pressure Head vs. Moisture Content, Sample No. SHC-5-79.5**

Printed on Recycled Paper

MOISTURE RETENTION DATA - HANGING COLUMN
(PORE SIZE DISTRIBUTION)

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-9-30.5 (a) |
| RING NUMBER: | P11 |
| DEPTH: | 30.5 FT. |
| SAMPLE VOLUME: | 61.58 (cc) |
| SATURATED WEIGHT AT 0 CM TENSION (WITH CAP AND RING): | 150.04 (g) |
| TARE RING: | 18.42 (g) |
| TARE CAP: | 0.00 (g) |
| DRY WEIGHT OF SAMPLE: | 103.66 (g) |
| SATURATED MOISTURE CONTENT: | 45.40 (% vol) |
| INITIAL VOLUME OF WATER IN SAMPLE: | 27.96 (cc) |

| DATE (1989) | TIME | SUCTION (CM) | SAMPLE WT (G) | CHANGE WT (G) | CHANGES WT (G) | MOISTURE CONTENT(% VO DRYING | WETTING |
|---|---|---|---|---|---|---|---|
| 7/8 | 1515 | 0.0 | 150.04 | -- | -- | 45.40 | 0.00 |
| 7/23 | 1715 | 44.0 | 142.28 | 7.76 | 7.76 | 32.80 | 0.00 |
| 7/27 | 1600 | 99.0 | 139.31 | 2.97 | 10.73 | 27.98 | 0.00 |
| 7/31 | 1130 | 191.5 | 137.34 | 1.97 | 12.70 | 24.78 | 0.00 |

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: W. Strong
CALCULATION MADE BY: L. Simpson
CHECKED BY: E. Mattson

MOISTURE RETENTION DATA - 15 BAR PRESSURE PLATE
(PORE SIZE DISTRIBUTION)

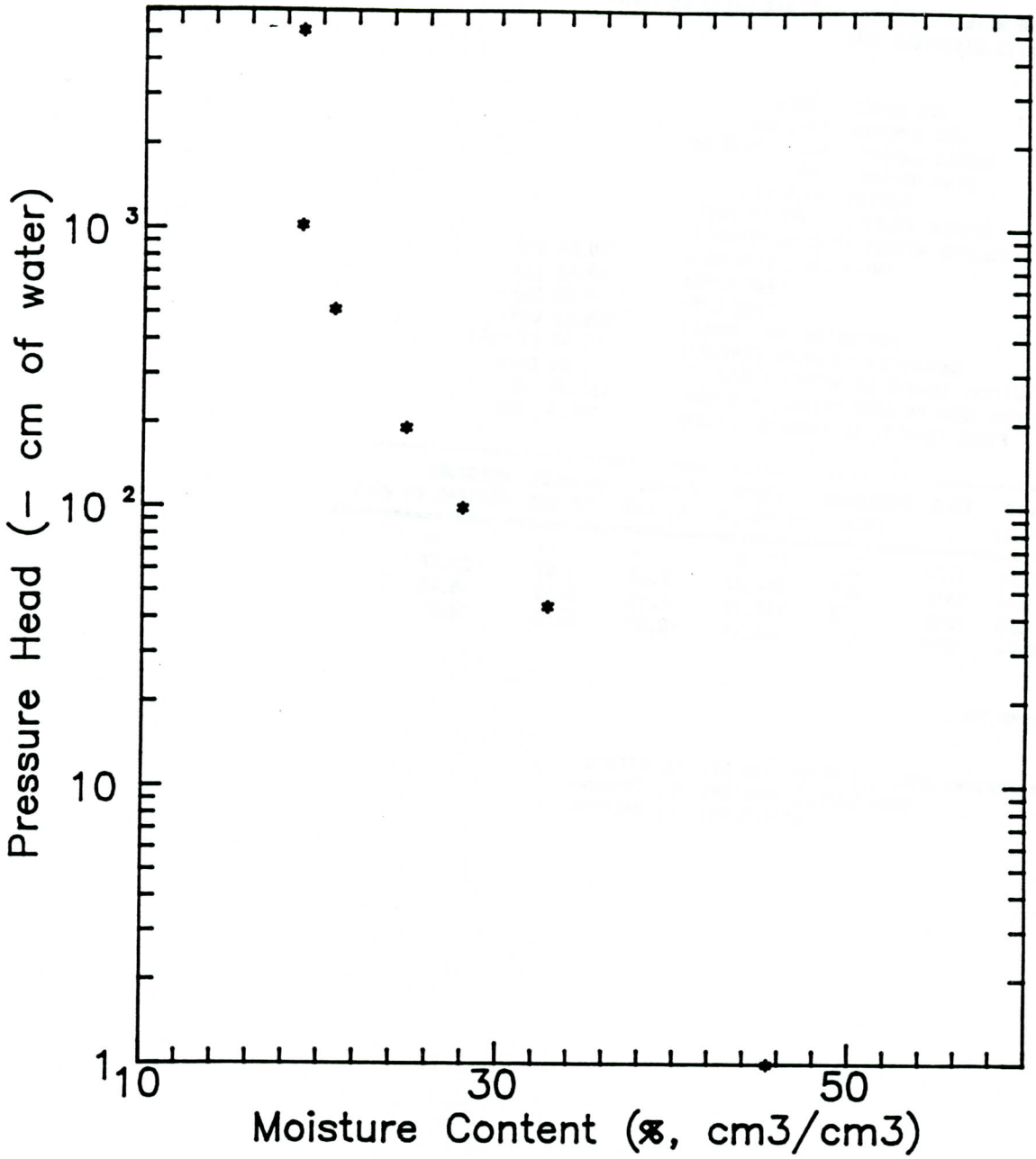
JOB NAME: EMCON
JOB NUMBER: 89-L-060
SAMPLE NUMBER: SHC-9-30.5 (a)
RING NUMBER: P11
DEPTH: 30.5 FT.
SAMPLE VOLUME: 61.58 (cc)
SATURATED WEIGHT AT 0 CM TENSION (WITH CAP AND RING): 150.04 (g)
TARE RING: 18.42 (g)
TARE CAP: 0.00 (g)
DRY WEIGHT OF SAMPLE: 103.66 (g)
SATURATED MOISTURE CONTENT: 45.40 (% vol)
INITIAL VOLUME OF WATER IN SAMPLE: 15.26 (cc)
WEIGHT FROM HANGING COLUMN, W/O CAP: 137.34 (g)
FINAL TENSION ON HANGING COLUMN: 191.5 (cm)

| DATE (1988) | TIME | PRESSURE (BAR) | WEIGHT W/RING(G) | CHANGE WT (G) | CHANGES WT (G) | MOISTURE CONTENT (% VOL) |
|---|---|---|---|---|---|---|
| 7/31 | 1200 | 0.0 | 137.34 | -- | -- | -- |
| 8/4 | 1310 | 0.5 | 134.87 | 2.47 | 2.47 | 20.77 |
| 8/9 | 1615 | 1.0 | 133.73 | 1.14 | 3.61 | 18.92 |
| 8/14 | 800 | 5.0 | 133.74 | -0.01 | 3.60 | 18.93 |

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: W. Strong
CALCULATION MADE BY: L. Simpson
CHECKED BY: E. Mattson

Printed on Recycled Paper




**Pressure Head vs. Moisture Content, Sample No. SHC-9-30.5 (a).**

MOISTURE RETENTION DATA - PRESSURE MEMBRANE
(PORE SIZE DISTRIBUTION)

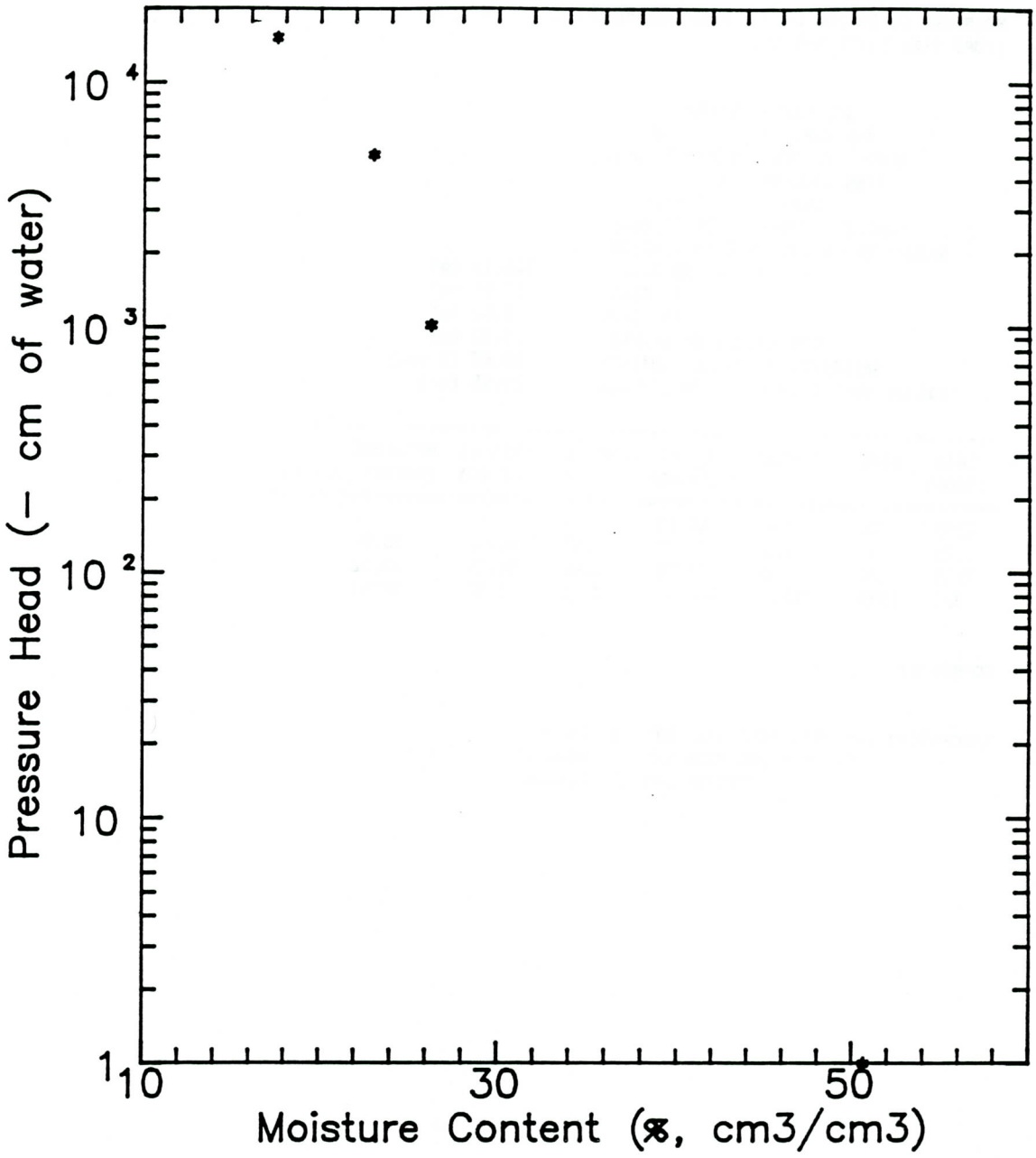
| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-9-34.5 (b) |
| RING NUMBER: | P21 |
| DEPTH: | 34.5 FT. |
| SAMPLE VOLUME: | 59.11 (cc) |
| SATURATED WEIGHT AT 0 CM TENSION (WITH CAP AND RING): | 144.13 (g) |
| TARE RING: | 17.96 (g) |
| TARE CAP: | 0.00 (g) |
| DRY WEIGHT OF SAMPLE: | 96.22 (g) |
| SATURATED MOISTURE CONTENT: | 50.67 (% vol) |
| INITIAL VOLUME OF WATER IN SAMPLE: | 29.95 (cc) |

| DATE (1988) | TIME | PRESSURE (BAR) | WEIGHT W/RING(G) | CHANGE WT (G) | CHANGES WT (G) | MOISTURE CONTENT (% VOL) |
|---|---|---|---|---|---|---|
| 7/18 | 1000 | 0.0 | 144.13 | -- | -- | -- |
| 7/23 | 1645 | 1.0 | 129.71 | 14.42 | 14.42 | 26.27 |
| 7/31 | 1000 | 5.0 | 127.79 | 1.92 | 16.34 | 23.02 |
| 8/7 | 1515 | 15.0 | 124.53 | 3.26 | 19.60 | 17.51 |

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: W. Strong
CALCULATION MADE BY: L. Simpson
CHECKED BY: E. Mattson

Printed on Recycled Paper




**Pressure Head vs. Moisture Content, Sample No. SHC-9-34.5 (b)**

MOISTURE RETENTION DATA - PRESSURE MEMBRANE
(PORE SIZE DISTRIBUTION)

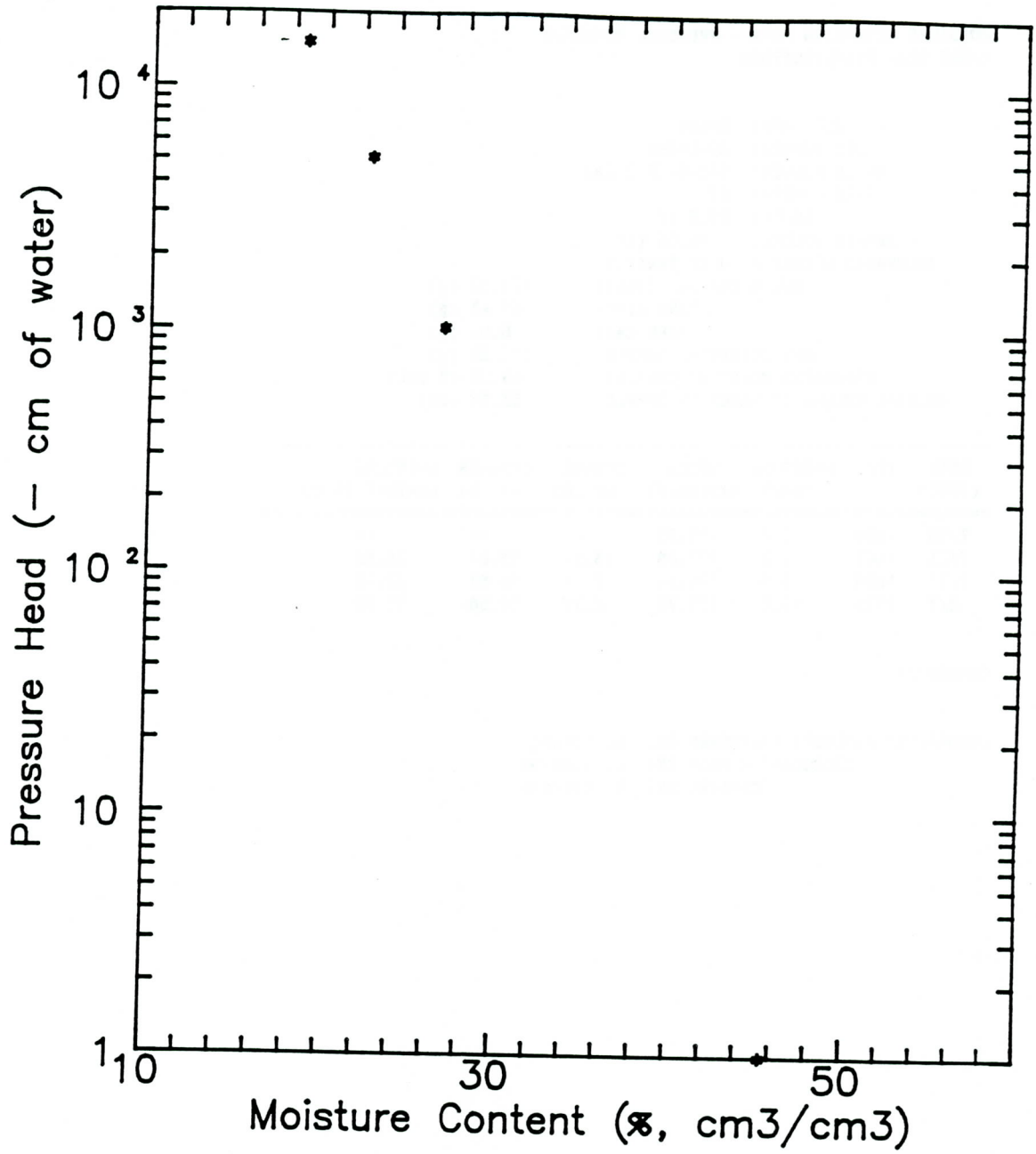
| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-9-39.0 (a) |
| RING NUMBER: | X7 |
| DEPTH: | 39.0 FT. |
| SAMPLE VOLUME: | 73.06 (cc) |
| SATURATED WEIGHT AT 0 CM TENSION (WITH CAP AND RING): | 191.32 (g) |
| TARE RING: | 41.43 (g) |
| TARE CAP: | 0.00 (g) |
| DRY WEIGHT OF SAMPLE: | 116.65 (g) |
| SATURATED MOISTURE CONTENT: | 45.50 (% vol) |
| INITIAL VOLUME OF WATER IN SAMPLE: | 33.24 (cc) |

| DATE (1988) | TIME | PRESSURE (BAR) | WEIGHT W/RING(G) | CHANGE WT (G) | CHANGES WT (G) | MOISTURE CONTENT (% VOL) |
|---|---|---|---|---|---|---|
| 7/18 | 1000 | 0.0 | 191.32 | -- | -- | -- |
| 7/23 | 1645 | 1.0 | 177.68 | 13.64 | 13.64 | 26.83 |
| 7/31 | 1000 | 5.0 | 174.51 | 3.17 | 16.81 | 22.49 |
| 8/7 | 1515 | 15.0 | 171.74 | 2.77 | 19.58 | 18.70 |

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: W. Strong
CALCULATION MADE BY: L. Simpson
CHECKED BY: E. Mattson

Printed on Recycled Paper




Pressure Head vs. Moisture Content, Sample No. SHC-9-39.0 (a)

MOISTURE RETENTION DATA - PRESSURE MEMBRANE
(PORE SIZE DISTRIBUTION)

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-9-49.5 |
| RING NUMBER: | X5 |
| DEPTH: | 49.5 FT. |
| SAMPLE VOLUME: | 73.06 (cc) |
| SATURATED WEIGHT AT 0 CM TENSION (WITH CAP AND RING): | 178.35 (g) |
| TARE RING: | 40.74 (g) |
| TARE CAP: | 0.00 (g) |
| DRY WEIGHT OF SAMPLE: | 96.24 (g) |
| SATURATED MOISTURE CONTENT: | 56.62 (% vol) |
| INITIAL VOLUME OF WATER IN SAMPLE: | 41.37 (cc) |

| DATE (1988) | TIME | PRESSURE (BAR) | WEIGHT W/RING(G) | CHANGE WT (G) | CHANGES WT (G) | MOISTURE CONTENT (% VOL) |
|---|---|---|---|---|---|---|
| 7/18 | 1000 | 0.0 | 178.35 | -- | -- | -- |
| 7/23 | 1645 | 1.0 | 172.61 | 5.74 | 5.74 | 48.77 |
| 7/31 | 1000 | 5.0 | 169.41 | 3.20 | 8.94 | 44.39 |
| 8/7 | 1515 | 15.0 | 167.11 | 2.30 | 11.24 | 41.24 |

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: W. Strong
CALCULATION MADE BY: L. Simpson
CHECKED BY: E. Mattson

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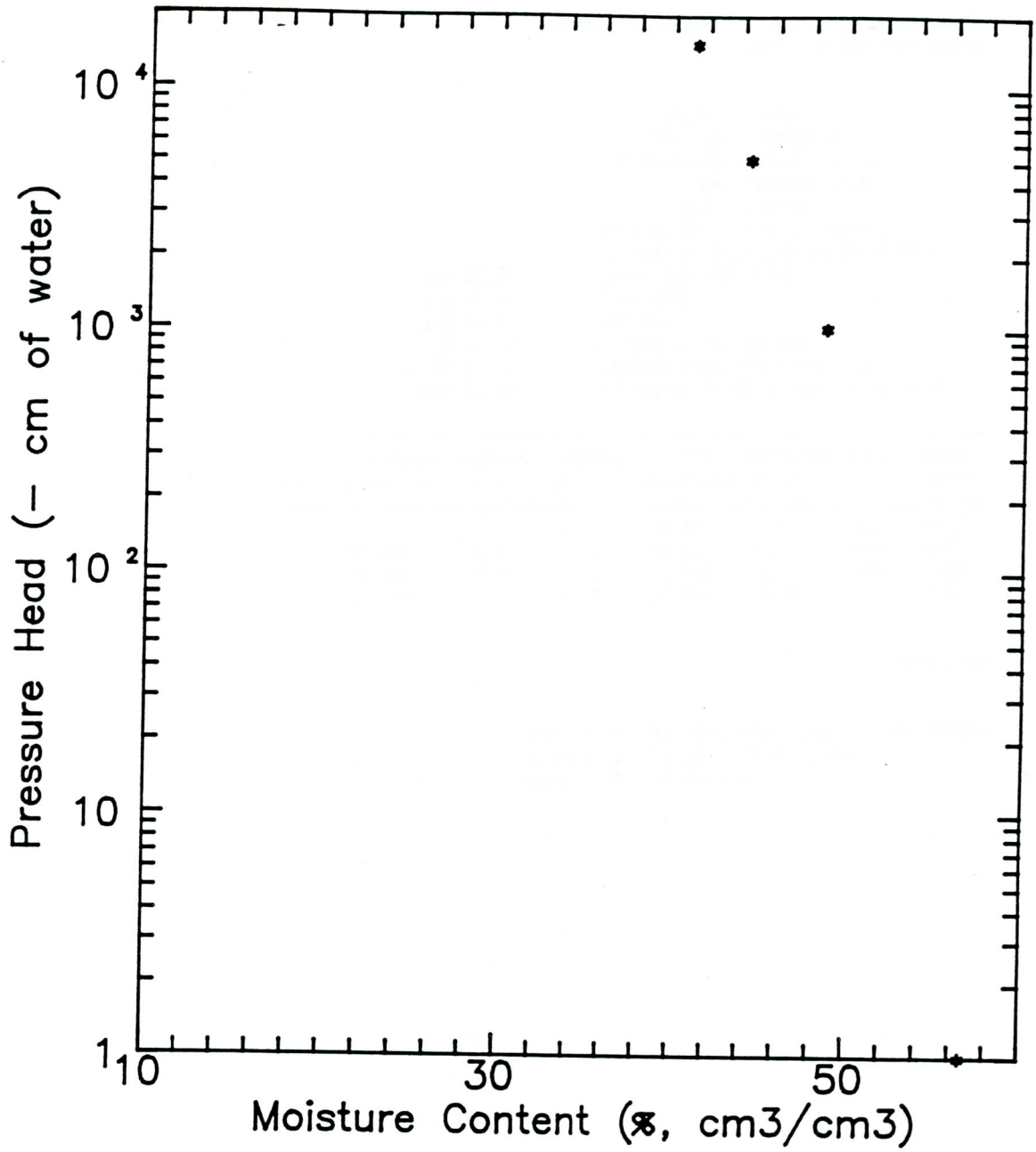



Pressure Head vs. Moisture Content, Sample No. SHC-9-49.5

MOISTURE RETENTION DATA - PRESSURE MEMBRANE
(PORE SIZE DISTRIBUTION)

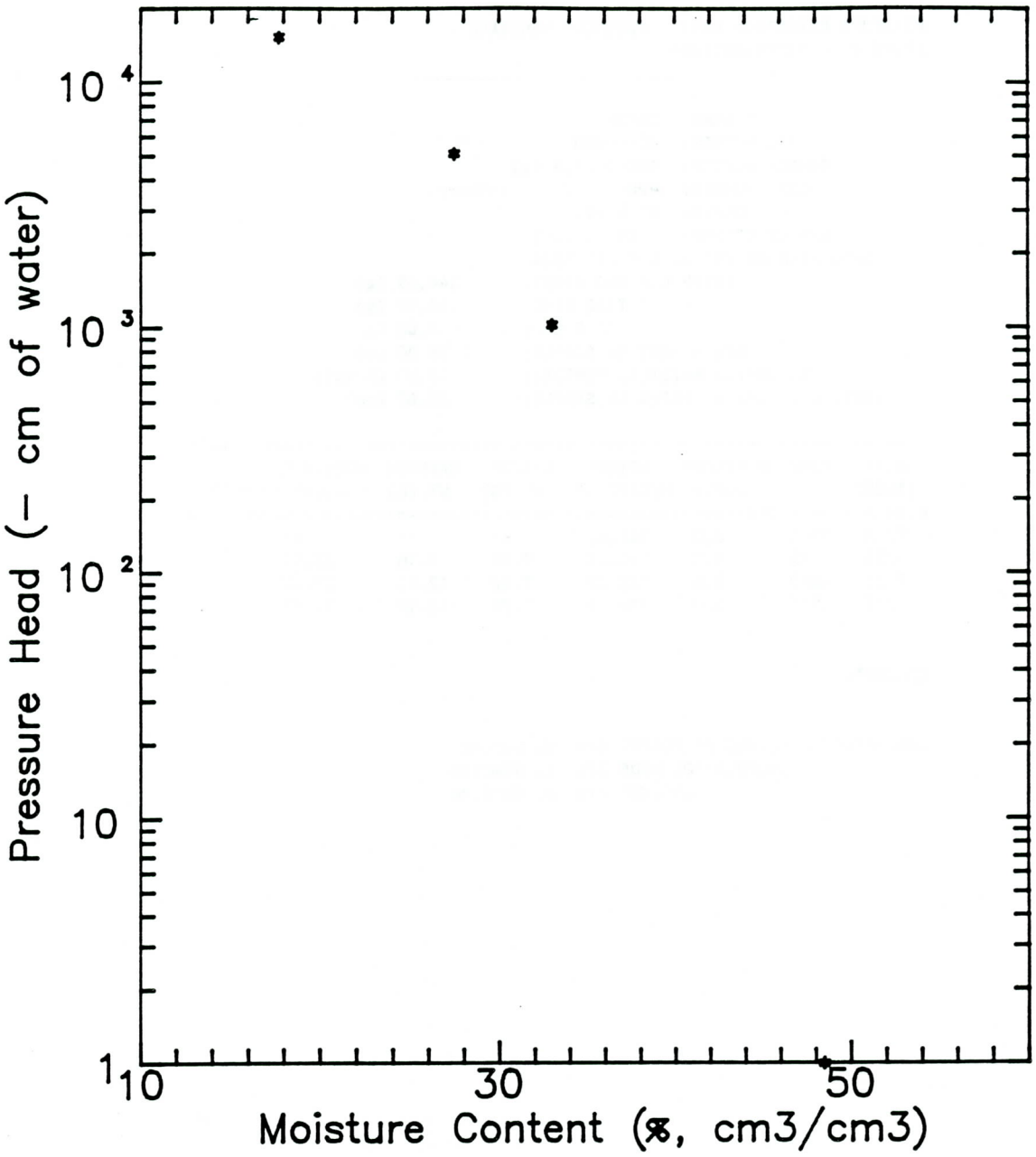
JOB NAME: EMCON
JOB NUMBER: 89-L-060
SAMPLE NUMBER: SHC-9-59.5 (a)
RING NUMBER: P20
DEPTH: 59.5 FT.
SAMPLE VOLUME: 59.11 (cc)
SATURATED WEIGHT AT 0 CM TENSION
(WITH CAP AND RING): 144.65 (g)
TARE RING: 17.92 (g)
TARE CAP: 0.00 (g)
DRY WEIGHT OF SAMPLE: 98.06 (g)
SATURATED MOISTURE CONTENT: 48.50 (% vol)
INITIAL VOLUME OF WATER IN SAMPLE: 28.67 (cc)

| DATE (1988) | TIME | PRESSURE (BAR) | WEIGHT W/RING(G) | CHANGE WT (G) | CHANGES WT (G) | MOISTURE CONTENT (% VOL) |
|---|---|---|---|---|---|---|
| 7/18 | 1000 | 0.0 | 144.65 | -- | -- | -- |
| 7/23 | 1645 | 1.0 | 135.49 | 9.16 | 9.16 | 33.01 |
| 7/31 | 1000 | 5.0 | 132.22 | 3.27 | 12.43 | 27.47 |
| 8/7 | 1515 | 15.0 | 126.46 | 5.76 | 18.19 | 17.73 |

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: W. Strong
CALCULATION MADE BY: L. Simpson
CHECKED BY: E. Mattson

Printed on Recycled Paper




**Pressure Head vs. Moisture Content, Sample No. SHC-9-59.5 (a)**

MOISTURE RETENTION DATA - HANGING COLUMN
(PORE SIZE DISTRIBUTION)

JOB NAME: EMCON
JOB NUMBER: 89-L-060
SAMPLE NUMBER: SHC-9-69.5 (b)
RING NUMBER: X8
DEPTH: 69.5 FT.
SAMPLE VOLUME: 59.11 (cc)
SATURATED WEIGHT AT 0 CM TENSION (WITH CAP AND RING): 192.84 (g)
TARE RING: 41.98 (g)
TARE CAP: 0.00 (g)
DRY WEIGHT OF SAMPLE: 118.13 (g)
SATURATED MOISTURE CONTENT: 55.37 (% vol)
INITIAL VOLUME OF WATER IN SAMPLE: 32.73 (cc)

| DATE (1989) | TIME | SUCTION (CM) | SAMPLE WT (G) | CHANGE WT (G) | CHANGES WT (G) | MOISTURE CONTENT(% VO DRYING | WETTING |
|---|---|---|---|---|---|---|---|
| 7/8 | 1515 | 0.0 | 192.84 | -- | -- | 55.37 | 0.00 |
| 7/23 | 1715 | 45.5 | 183.97 | 8.87 | 8.87 | 40.37 | 0.00 |
| 7/27 | 1600 | 100.0 | 179.47 | 4.50 | 13.37 | 32.75 | 0.00 |
| 7/31 | 1130 | 192.5 | 176.51 | 2.96 | 16.33 | 27.74 | 0.00 |

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: W. Strong
CALCULATION MADE BY: L. Simpson
CHECKED BY: E. Mattson

Printed on Recycled Paper

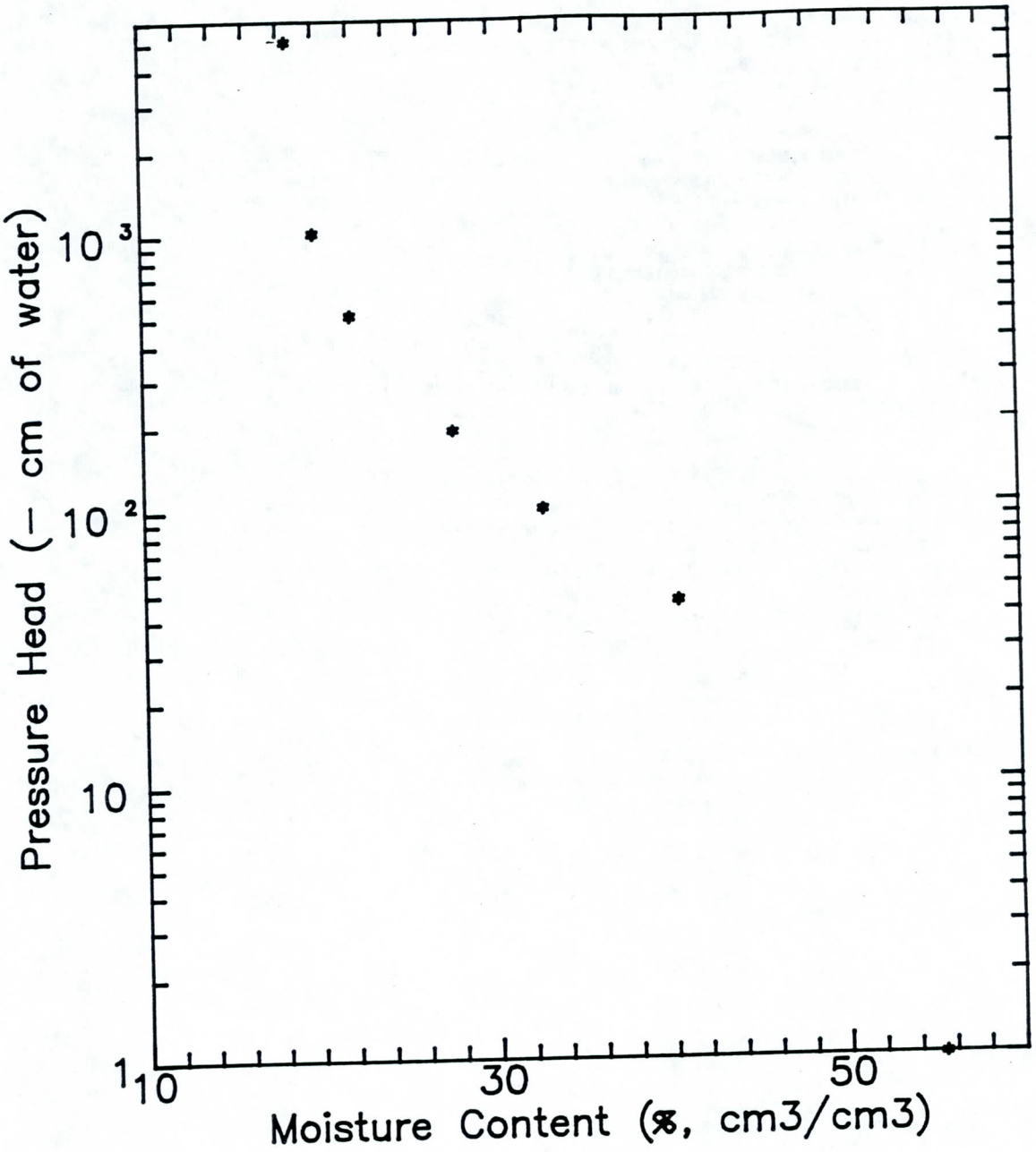
MOISTURE RETENTION DATA - 15 BAR PRESSURE PLATE
(PORE SIZE DISTRIBUTION)

| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-9-69.5 (b) |
| RING NUMBER: | X8 |
| DEPTH: | 69.5 FT. |
| SAMPLE VOLUME: | 59.11 (cc) |
| SATURATED WEIGHT AT 0 CM TENSION (WITH CAP AND RING): | 192.84 (g) |
| TARE RING: | 41.98 (g) |
| TARE CAP: | 0.00 (g) |
| DRY WEIGHT OF SAMPLE: | 118.13 (g) |
| SATURATED MOISTURE CONTENT: | 55.37 (% vol) |
| INITIAL VOLUME OF WATER IN SAMPLE: | 16.40 (cc) |
| WEIGHT FROM HANGING COLUMN, W/O CAP: | 176.51 (g) |
| FINAL TENSION ON HANGING COLUMN: | 192.5 (cm) |

| DATE (1988) | TIME | PRESSURE (BAR) | WEIGHT W/RING(G) | CHANGE WT (G) | CHANGES WT (G) | MOISTURE CONTENT (% VOL) |
|---|---|---|---|---|---|---|
| 7/31 | 1200 | 0.0 | 176.51 | -- | -- | -- |
| 8/4 | 1310 | 0.5 | 173.10 | 3.41 | 3.41 | 21.98 |
| 8/9 | 1615 | 1.0 | 171.90 | 1.20 | 4.61 | 19.95 |
| 8/14 | 800 | 5.0 | 171.06 | 0.84 | 5.45 | 18.52 |

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: W. Strong
CALCULATION MADE BY: L. Simpson
CHECKED BY: E. Mattson




**Pressure Head vs. Moisture Content, Sample No. SHC-9-69.5 (b)**

Printed on Recycled Paper

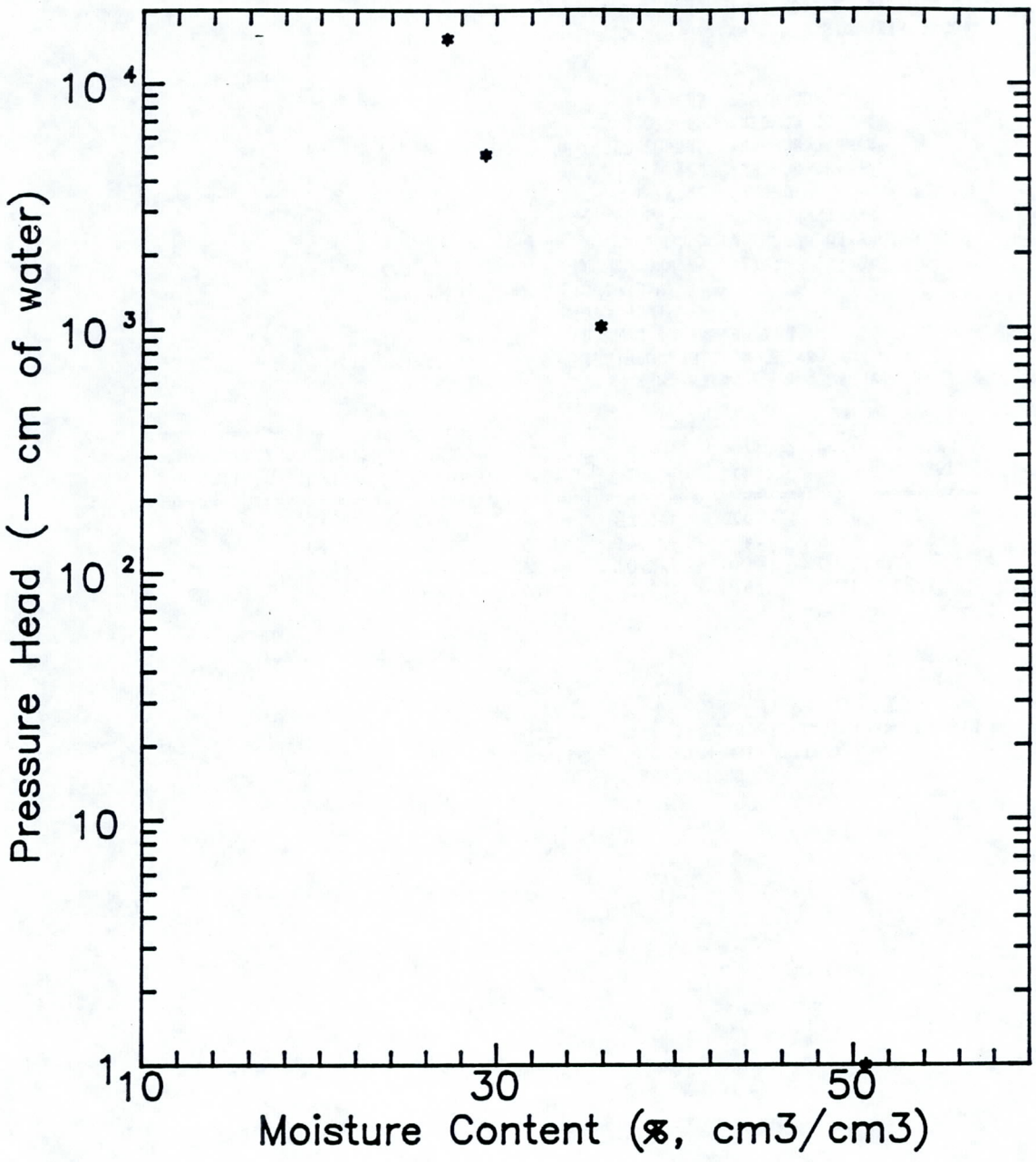
MOISTURE RETENTION DATA - PRESSURE MEMBRANE
(PORE SIZE DISTRIBUTION)

JOB NAME: EMCON
JOB NUMBER: 89-L-060
SAMPLE NUMBER: SHC-9-103 (a)
RING NUMBER: X1
DEPTH: 103.0 FT.
SAMPLE VOLUME: 73.06 (cc)
SATURATED WEIGHT AT 0 CM TENSION
(WITH CAP AND RING): 184.16 (g)
TARE RING: 40.66 (g)
TARE CAP: 0.00 (g)
DRY WEIGHT OF SAMPLE: 106.45 (g)
SATURATED MOISTURE CONTENT: 50.71 (% vol)
INITIAL VOLUME OF WATER IN SAMPLE: 37.05 (cc)

| DATE (1988) | TIME | PRESSURE (BAR) | WEIGHT W/RING(G) | CHANGE WT (G) | CHANGES WT (G) | MOISTURE CONTENT (% VOL) |
|---|---|---|---|---|---|---|
| 7/18 | 1000 | 0.0 | 184.16 | -- | -- | -- |
| 7/23 | 1645 | 1.0 | 173.34 | 10.82 | 10.82 | 35.90 |
| 7/31 | 1000 | 5.0 | 168.58 | 4.76 | 15.58 | 29.39 |
| 8/7 | 1515 | 15.0 | 166.96 | 1.62 | 17.20 | 27.17 |

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: W. Strong
CALCULATION MADE BY: L. Simpson
CHECKED BY: E. Mattson




**Pressure Head vs. Moisture Content, Sample No. SHC-9-103.0 (a)**

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MOISTURE RETENTION DATA - HANGING COLUMN
(PORE SIZE-DISTRIBUTION)

JOB NAME: EMCON
JOB NUMBER: 89-L-060
SAMPLE NUMBER: SHC-22-23.5
RING NUMBER: P23
DEPTH: 23.5 FT.
SAMPLE VOLUME: 59.11 (cc)
SATURATED WEIGHT AT 0 CM TENSION
(WITH CAP AND RING): 140.88 (g)
TARE RING: 18.05 (g)
TARE CAP: 0.00 (g)
DRY WEIGHT OF SAMPLE: 94.25 (g)
SATURATED MOISTURE CONTENT: 48.35 (% vol)
INITIAL VOLUME OF WATER IN SAMPLE: 28.58 (cc)

| DATE (1989) | TIME | SUCTION (CM) | SAMPLE WT (G) | CHANGE WT (G) | CHANGES WT (G) | MOISTURE CONTENT(% VOL) DRYING | WETTING |
|---|---|---|---|---|---|---|---|
| 7/8 | 1515 | 0.0 | 140.88 | -- | -- | 48.35 | 0.00 |
| 7/23 | 1715 | 41.5 | 127.43 | 13.45 | 13.45 | 25.60 | 0.00 |
| 7/27 | 1600 | 94.0 | 123.07 | 4.36 | 17.81 | 18.22 | 0.00 |
| 7/31 | 1130 | 186.0 | 121.26 | 1.81 | 19.62 | 15.16 | 0.00 |

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: W. Strong
CALCULATION MADE BY: L. Simpson
CHECKED BY: E. Mattson

MOISTURE RETENTION DATA - 15 BAR PRESSURE PLATE
(PORE SIZE DISTRIBUTION)

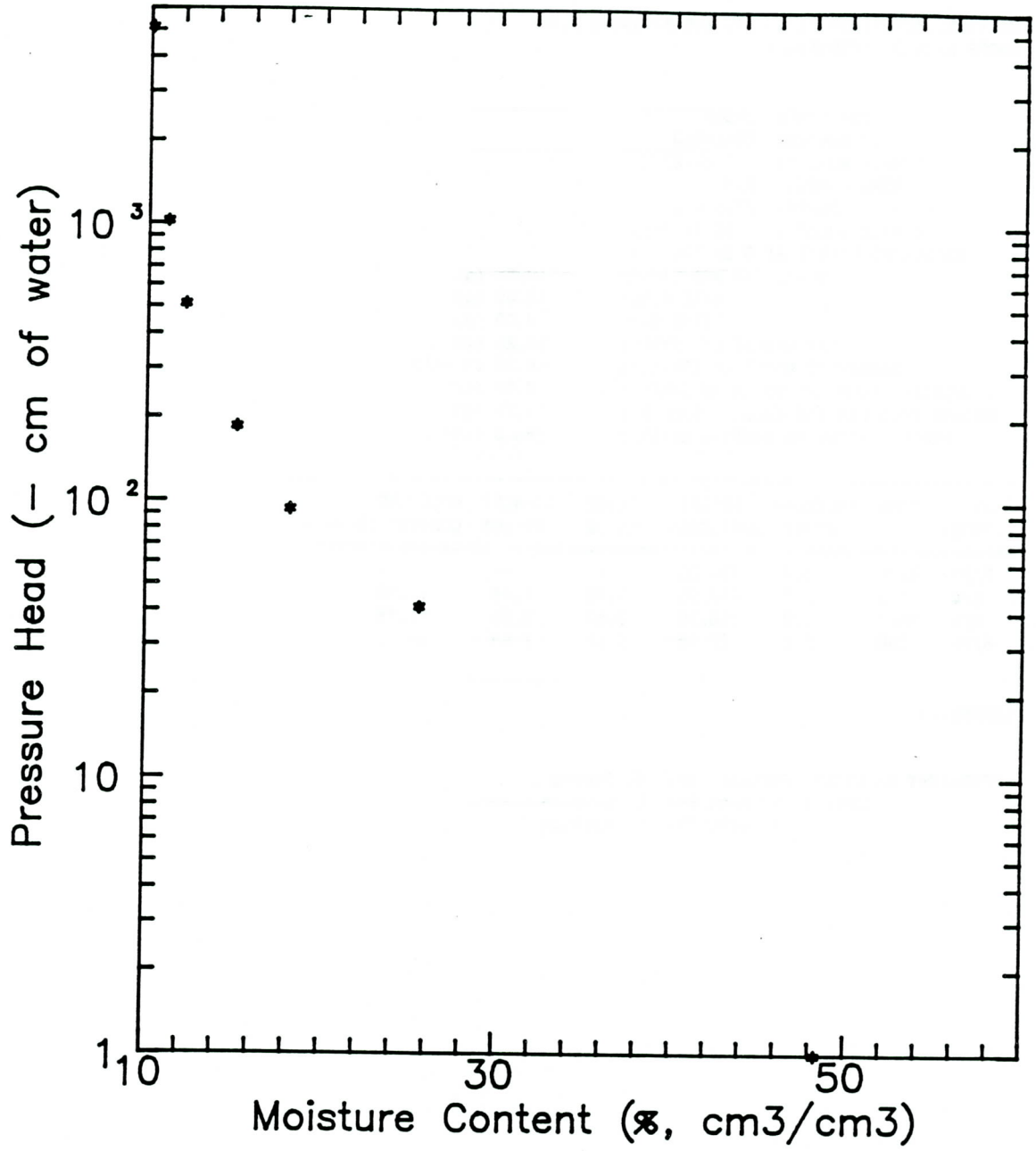
| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-22-23.5 |
| RING NUMBER: | P23 |
| DEPTH: | 23.5 FT. |
| SAMPLE VOLUME: | 59.11 (cc) |
| SATURATED WEIGHT AT 0 CM TENSION (WITH CAP AND RING): | 140.88 (g) |
| TARE RING: | 18.05 (g) |
| TARE CAP: | 0.00 (g) |
| DRY WEIGHT OF SAMPLE: | 94.25 (g) |
| SATURATED MOISTURE CONTENT: | 48.35 (% vol) |
| INITIAL VOLUME OF WATER IN SAMPLE: | 8.96 (cc) |
| WEIGHT FROM HANGING COLUMN, W/O CAP: | 121.26 (g) |
| FINAL TENSION ON HANGING COLUMN: | 186.0 (cm) |

| DATE (1988) | TIME | PRESSURE (BAR) | WEIGHT W/RING(G) | CHANGE WT (G) | CHANGES WT (G) | MOISTURE CONTENT (% VOL) |
|---|---|---|---|---|---|---|
| 7/31 | 1200 | 0.0 | 121.26 | -- | -- | -- |
| 8/4 | 1310 | 0.5 | 119.50 | 1.76 | 1.76 | 12.18 |
| 8/9 | 1615 | 1.0 | 118.88 | 0.62 | 2.38 | 11.13 |
| 8/14 | 800 | 5.0 | 118.28 | 0.60 | 2.98 | 10.12 |

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: W. Strong
CALCULATION MADE BY: L. Simpson
CHECKED BY: E. Mattson

Printed on Recycled Paper




Pressure Head vs. Moisture Content, Sample No. SHC-22-23.5

MOISTURE RETENTION DATA - PRESSURE MEMBRANE
(PORE SIZE DISTRIBUTION)

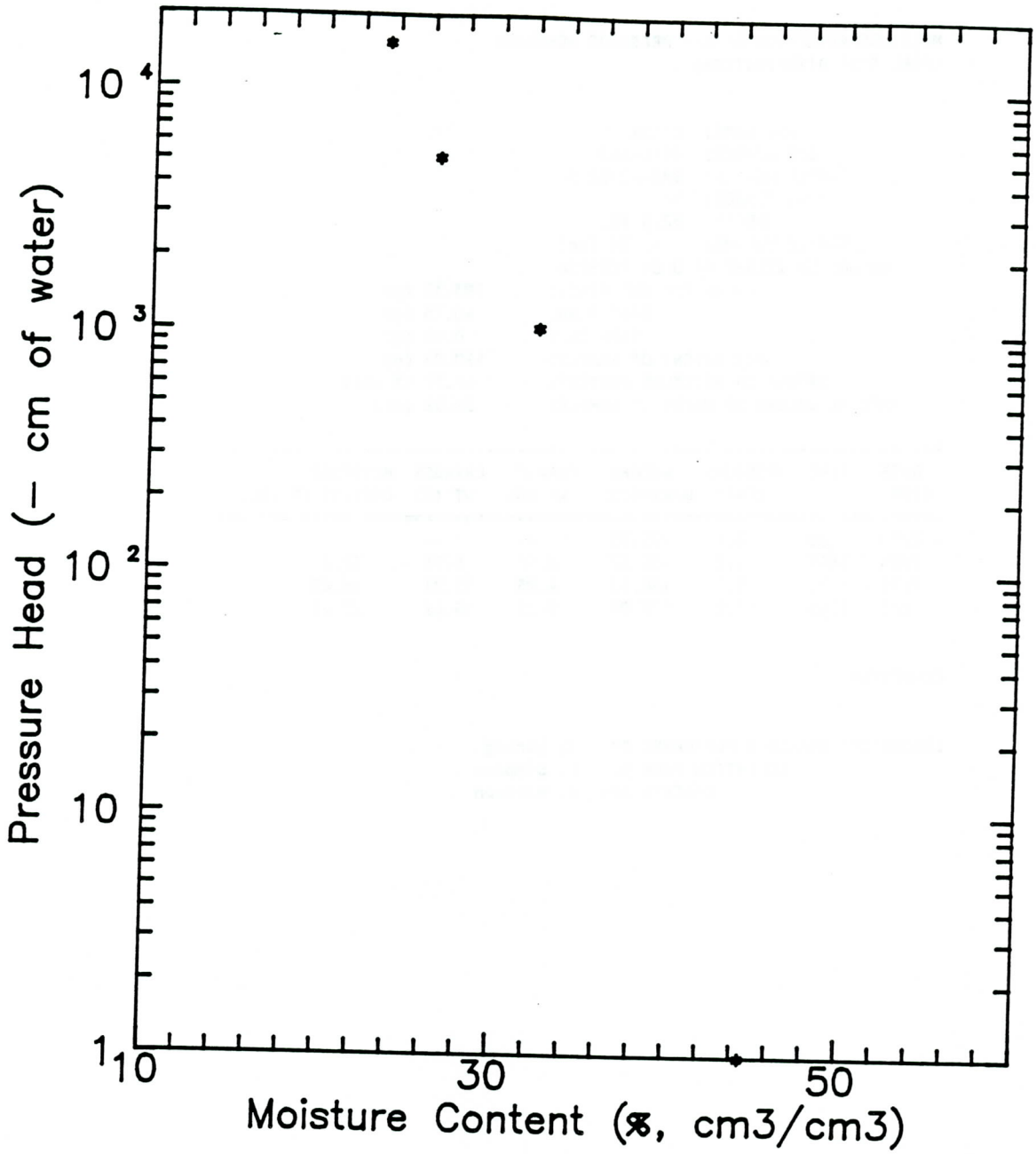
JOB NAME: EMCON
JOB NUMBER: 89-L-060
SAMPLE NUMBER: SHC-22-52.5
RING NUMBER: X6
DEPTH: 52.5 FT.
SAMPLE VOLUME: 73.06 (cc)
SATURATED WEIGHT AT 0 CM TENSION
(WITH CAP AND RING): 193.33 (g)
TARE RING: 40.73 (g)
TARE CAP: 0.00 (g)
DRY WEIGHT OF SAMPLE: 120.08 (g)
SATURATED MOISTURE CONTENT: 44.51 (% vol)
INITIAL VOLUME OF WATER IN SAMPLE: 32.52 (cc)

| DATE (1988) | TIME | PRESSURE (BAR) | WEIGHT W/RING(G) | CHANGE WT (G) | CHANGES WT (G) | MOISTURE CONTENT (% VOL) |
|---|---|---|---|---|---|---|
| 7/18 | 1000 | 0.0 | 193.33 | -- | -- | -- |
| 7/23 | 1645 | 1.0 | 184.37 | 8.96 | 8.96 | 32.25 |
| 7/31 | 1000 | 5.0 | 180.12 | 4.25 | 13.21 | 26.43 |
| 8/7 | 1515 | 15.0 | 177.91 | 2.21 | 15.42 | 23.41 |

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: W. Strong
CALCULATION MADE BY: L. Simpson
CHECKED BY: E. Mattson

Printed on Recycled Paper




**Pressure Head vs. Moisture Content, Sample No. SHC-22-52.5**

MOISTURE RETENTION DATA - PRESSURE MEMBRANE
(PORE SIZE DISTRIBUTION)

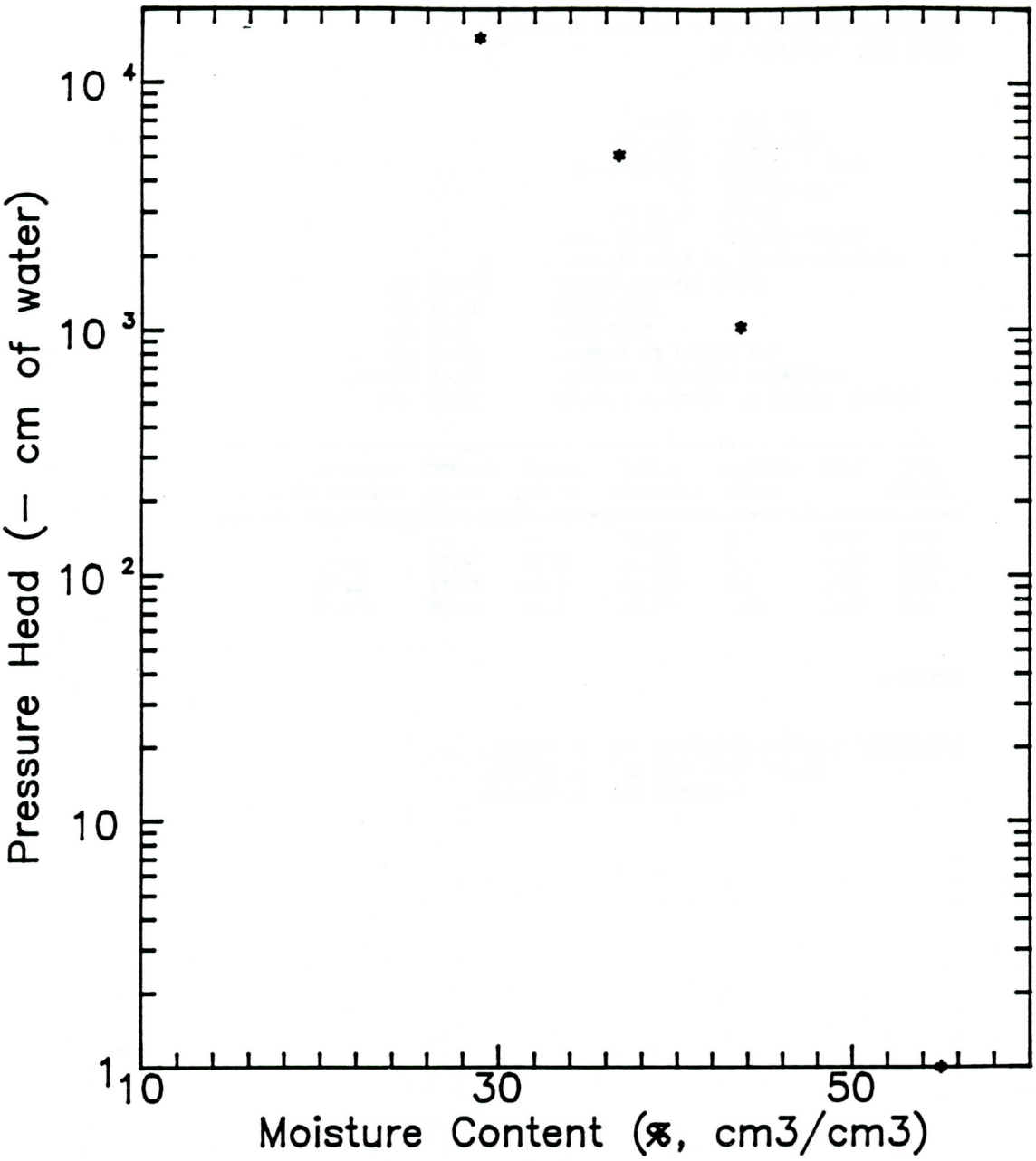
| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-22-54.0 |
| RING NUMBER: | X3 |
| DEPTH: | 54.0 FT. |
| SAMPLE VOLUME: | 73.06 (cc) |
| SATURATED WEIGHT AT 0 CM TENSION (WITH CAP AND RING): | 178.37 (g) |
| TARE RING: | 41.17 (g) |
| TARE CAP: | 0.00 (g) |
| DRY WEIGHT OF SAMPLE: | 97.02 (g) |
| SATURATED MOISTURE CONTENT: | 55.00 (% vol) |
| INITIAL VOLUME OF WATER IN SAMPLE: | 40.18 (cc) |

| DATE (1988) | TIME | PRESSURE (BAR) | WEIGHT W/RING(G) | CHANGE WT (G) | CHANGES WT (G) | MOISTURE CONTENT (% VOL) |
|---|---|---|---|---|---|---|
| 7/18 | 1000 | 0.0 | 178.37 | -- | -- | -- |
| 7/23 | 1645 | 1.0 | 170.05 | 8.32 | 8.32 | 43.61 |
| 7/31 | 1000 | 5.0 | 165.03 | 5.02 | 13.34 | 36.74 |
| 8/7 | 1515 | 15.0 | 159.34 | 5.69 | 19.03 | 28.95 |

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: W. Strong
CALCULATION MADE BY: L. Simpson
CHECKED BY: E. Mattson

Printed on Recycled Paper




**Pressure Head vs. Moisture Content, Sample No. SHC-22-54.0**

MOISTURE RETENTION DATA - PRESSURE MEMBRANE
(PORE SIZE DISTRIBUTION)

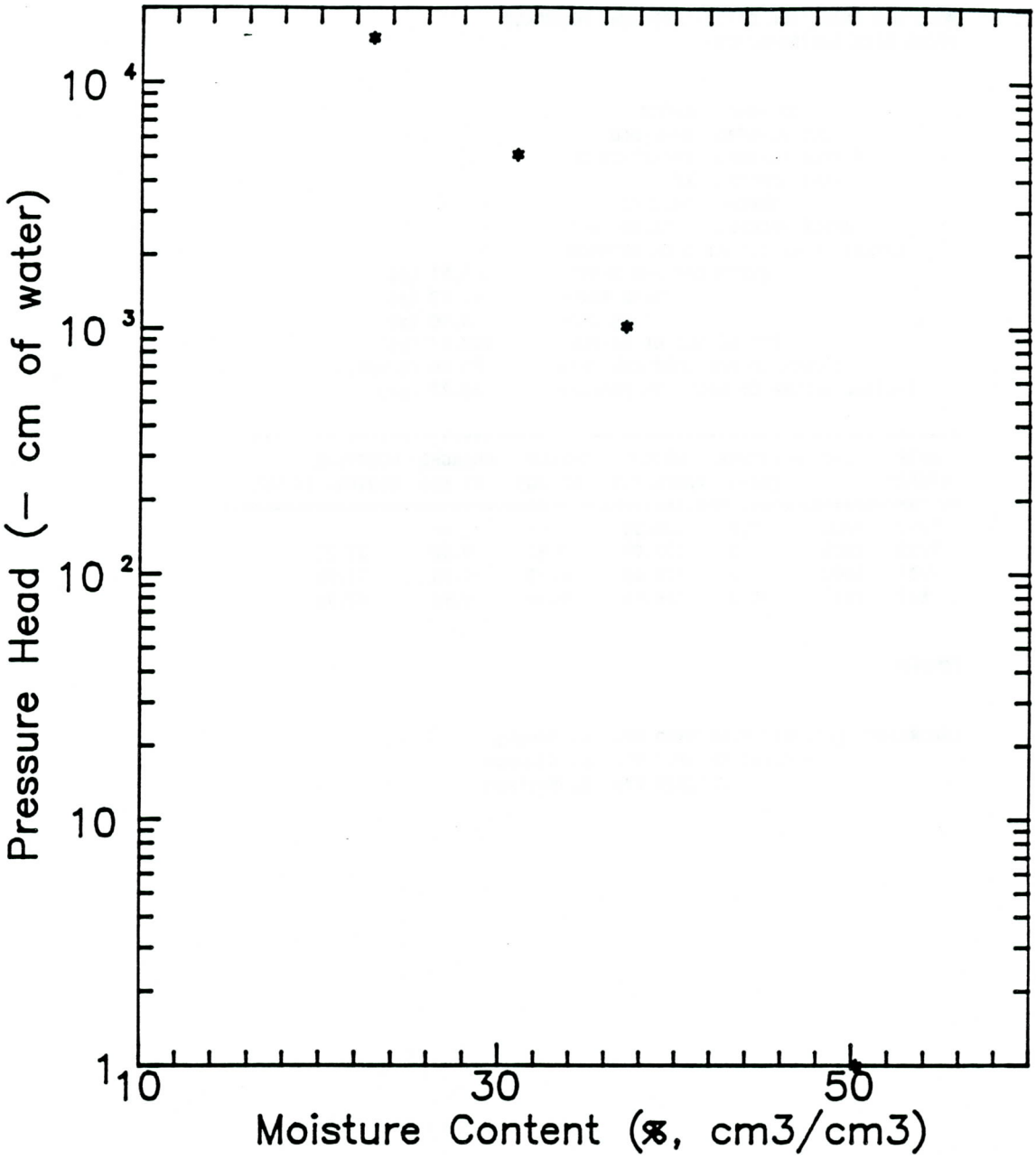
JOB NAME: EMCON
JOB NUMBER: 89-L-060
SAMPLE NUMBER: SHC-22-59.0
RING NUMBER: X2
DEPTH: 59.0 FT.
SAMPLE VOLUME: 73.06 (cc)
SATURATED WEIGHT AT 0 CM TENSION
(WITH CAP AND RING): 186.51 (g)
TARE RING: 41.32 (g)
TARE CAP: 0.00 (g)
DRY WEIGHT OF SAMPLE: 108.47 (g)
SATURATED MOISTURE CONTENT: 50.26 (% vol)
INITIAL VOLUME OF WATER IN SAMPLE: 36.72 (cc)

| DATE (1988) | TIME | PRESSURE (BAR) | WEIGHT W/RING(G) | CHANGE WT (G) | CHANGES WT (G) | MOISTURE CONTENT (% VOL) |
|---|---|---|---|---|---|---|
| 7/18 | 1000 | 0.0 | 186.51 | -- | -- | -- |
| 7/23 | 1645 | 1.0 | 176.99 | 9.52 | 9.52 | 37.23 |
| 7/31 | 1000 | 5.0 | 172.46 | 4.53 | 14.05 | 31.03 |
| 8/7 | 1515 | 15.0 | 166.56 | 5.90 | 19.95 | 22.95 |

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: W. Strong
CALCULATION MADE BY: L. Simpson
CHECKED BY: E. Mattson

Printed on Recycled Paper




**Pressure Head vs. Moisture Content, Sample No. SHC-22-59.0**

MOISTURE RETENTION DATA - PRESSURE MEMBRANE
(PORE SIZE DISTRIBUTION)

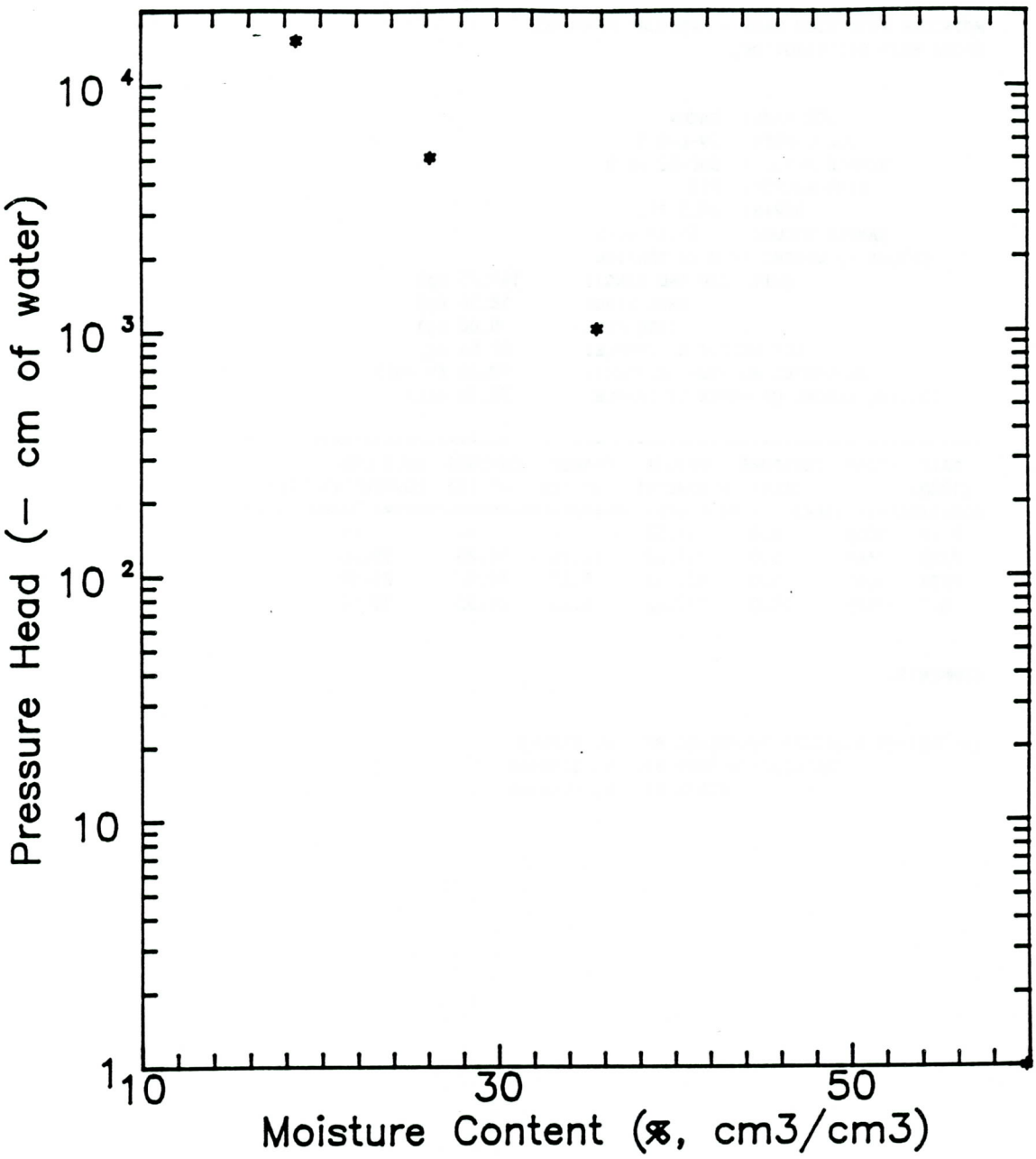
| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-22-68.0 |
| RING NUMBER: | P12 |
| DEPTH: | 68.0 FT. |
| SAMPLE VOLUME: | 59.11 (cc) |
| SATURATED WEIGHT AT 0 CM TENSION (WITH CAP AND RING): | 141.55 (g) |
| TARE RING: | 18.36 (g) |
| TARE CAP: | 0.00 (g) |
| DRY WEIGHT OF SAMPLE: | 87.84 (g) |
| SATURATED MOISTURE CONTENT: | 59.80 (% vol) |
| INITIAL VOLUME OF WATER IN SAMPLE: | 35.35 (cc) |

| DATE (1988) | TIME | PRESSURE (BAR) | WEIGHT W/RING(G) | CHANGE WT (G) | CHANGES WT (G) | MOISTURE CONTENT (% VOL) |
|---|---|---|---|---|---|---|
| 7/18 | 1000 | 0.0 | 141.55 | -- | -- | -- |
| 7/23 | 1645 | 1.0 | 127.29 | 14.26 | 14.26 | 35.68 |
| 7/31 | 1000 | 5.0 | 121.68 | 5.61 | 19.87 | 26.19 |
| 8/7 | 1515 | 15.0 | 117.22 | 4.46 | 24.33 | 18.64 |

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: W. Strong
CALCULATION MADE BY: L. Simpson
CHECKED BY: E. Mattson

Printed on Recycled Paper




Pressure Head vs. Moisture Content, Sample No. SHC-22-68.0

MOISTURE RETENTION DATA - PRESSURE MEMBRANE
(PORE SIZE DISTRIBUTION)

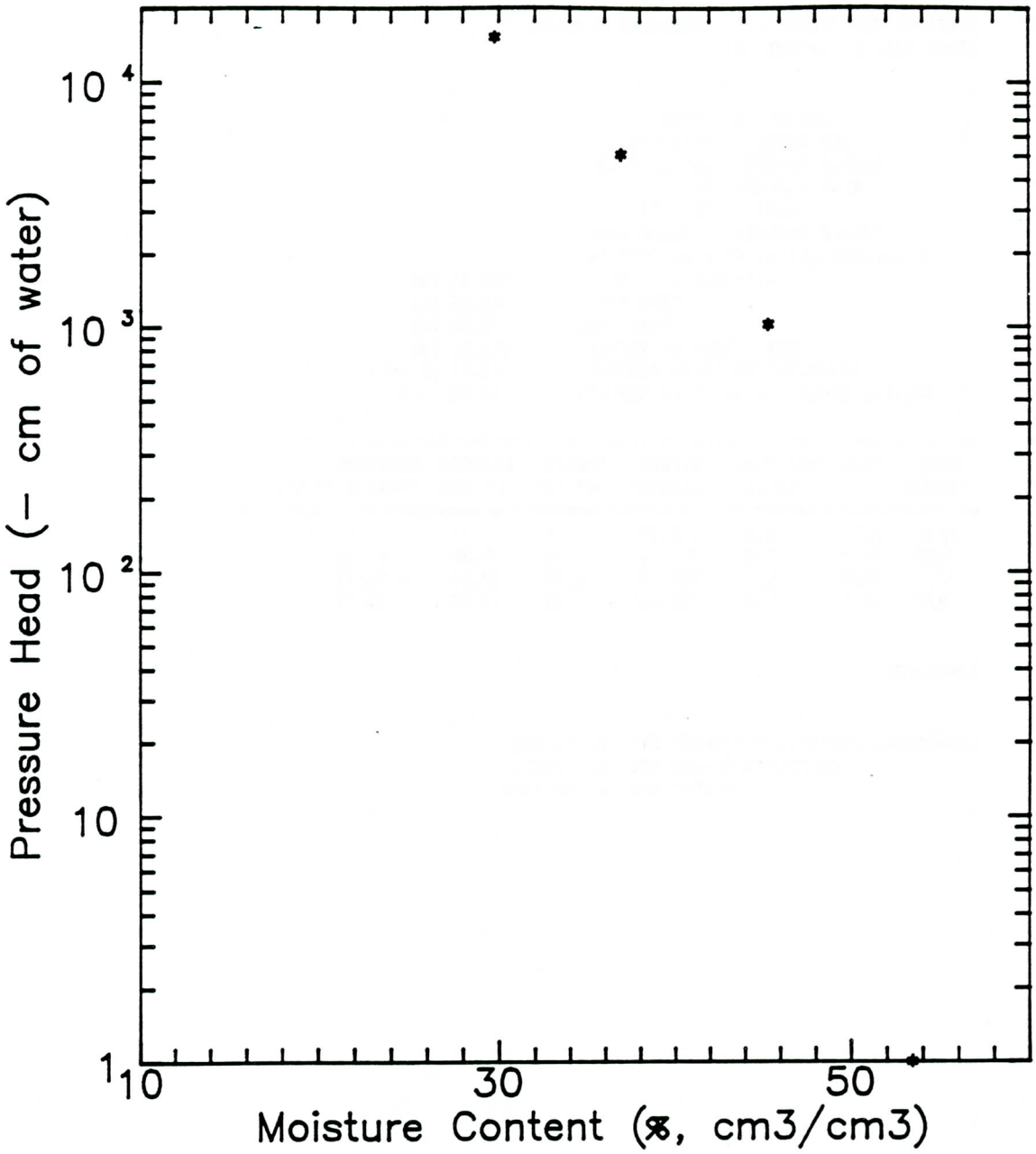
| | |
|---|---|
| JOB NAME: | EMCON |
| JOB NUMBER: | 89-L-060 |
| SAMPLE NUMBER: | SHC-22-77.5 |
| RING NUMBER: | X4 |
| DEPTH: | 77.5 FT. |
| SAMPLE VOLUME: | 73.06 (cc) |
| SATURATED WEIGHT AT 0 CM TENSION (WITH CAP AND RING): | 186.23 (g) |
| TARE RING: | 42.03 (g) |
| TARE CAP: | 0.00 (g) |
| DRY WEIGHT OF SAMPLE: | 105.18 (g) |
| SATURATED MOISTURE CONTENT: | 53.41 (% vol) |
| INITIAL VOLUME OF WATER IN SAMPLE: | 39.02 (cc) |

| DATE (1988) | TIME | PRESSURE (BAR) | WEIGHT W/RING(G) | CHANGE WT (G) | CHANGES WT (G) | MOISTURE CONTENT (% VOL) |
|---|---|---|---|---|---|---|
| 7/18 | 1000 | 0.0 | 186.23 | -- | -- | -- |
| 7/23 | 1645 | 1.0 | 180.35 | 5.88 | 5.88 | 45.36 |
| 7/31 | 1000 | 5.0 | 174.19 | 6.16 | 12.04 | 36.93 |
| 8/7 | 1515 | 15.0 | 168.96 | 5.23 | 17.27 | 29.77 |

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: W. Strong
CALCULATION MADE BY: L. Simpson
CHECKED BY: E. Mattson

Printed on Recycled Paper




**Pressure Head vs. Moisture Content, Sample No. SHC-22-77.5**

# Appendix D: PARTICLE SIZE DISTRIBUTION

## Summary of Particle Size Characteristics

| Sample No. | $d_{10}$ (mm) | $d_{50}$ (mm) | $d_{60}$ (mm) | $C_u$ | $C_c$ |
|---|---|---|---|---|---|
| SHC-5-79.5 | * | 0.22 | 0.34 | -- | -- |
| SHC-9-30.5(a) | 0.13 | 0.80 | 1.10 | 8.46 | 0.0 |
| SHC-9-34.5(b) | 0.011 | 0.30 | 0.38 | 34.55 | 4.04 |
| SHC-9-39.0(a) | 0.02 | 0.20 | 0.36 | 18.00 | 0.93 |
| SHC-9-49.5 | * | 0.0054 | 0.012 | -- | -- |
| SHC-9-59.5(a) | 0.0152 | 0.26 | 0.38 | 25.00 | 1.40 |
| SHC-9-69.5(b) | 0.08 | 0.38 | 0.51 | 6.38 | 1.19 |
| SHC-9-103.0(a) | * | 0.018 | 0.027 | -- | -- |
| SHC-22-23.5 | 0.15 | 0.73 | 1.00 | 6.67 | 0.96 |
| SHC-22-52.5 | 0.0044 | 0.07 | 0.08 | 18.18 | 1.92 |
| SHC-22-54.0 | * | 0.01 | 0.17 | -- | -- |
| SHC-22-59.0 | 0.0022 | 0.029 | 0.037 | 16.82 | 1.77 |
| SHC-22-68.0 | 0.003 | 0.03 | 0.04 | 13.33 | 1.88 |
| SHC-22-77.5 | * | 0.018 | 0.024 | -- | -- |

* diameter was not reached with test(s) specified
- values are dependent on diameters that were not reached

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SIEVE ANALYSIS DATA

JOB NAME: EMCON
JOB NUMBER: 89-L-060
TEST DATE: 8/2/89
SAMPLE NUMBER: SHC-5-79.5
DEPTH: 79.5 FT.
TOTAL SAMPLE WEIGHT (WET): 116.63 (g)
TOTAL SAMPLE WEIGHT (DRY): 116.63 (g)
WATER WEIGHT: 0.00 (g)
CONTAINER WEIGHT: 0.00 (g)
TOTAL WEIGHT OF SAMPLE: 116.63 (g)

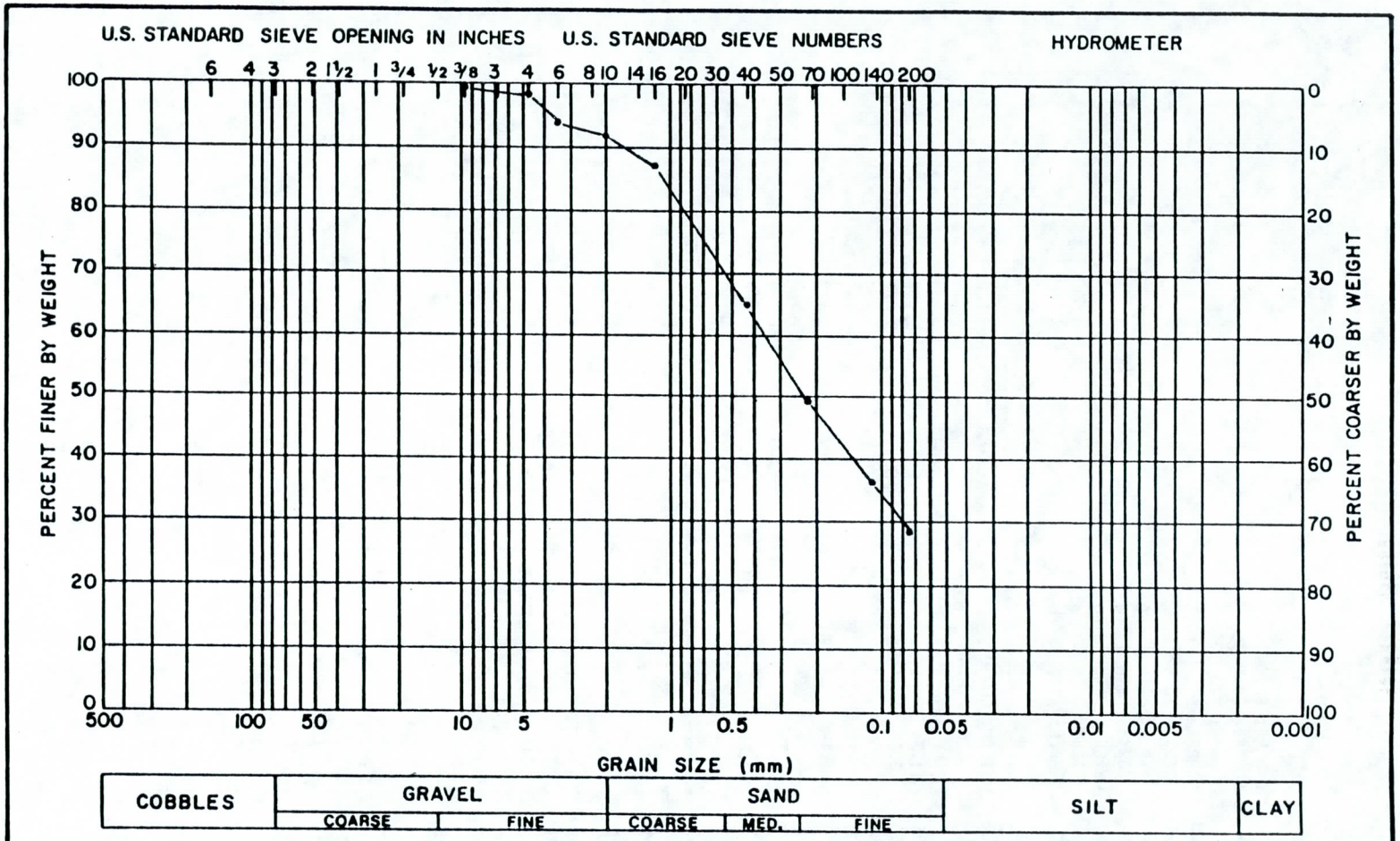
| SIEVE NUMBER | DIAMETER (mm) | WT. RETAINED | CUM WT. RETAINED | WT. PASSING | % PASSING | LOG DIAMETER |
|---|---|---|---|---|---|---|
| 3/8" | 9.525 | 1.89 | 1.89 | 114.74 | 98.38 | 0.979 |
| 4 | 4.750 | 1.53 | 3.42 | 113.21 | 97.07 | 0.677 |
| 6 | 3.350 | 2.50 | 5.92 | 110.71 | 94.92 | 0.525 |
| 10 | 2.000 | 2.46 | 8.38 | 108.25 | 92.81 | 0.301 |
| 16 | 1.180 | 6.66 | 15.04 | 101.59 | 87.10 | 0.072 |
| 40 | 0.425 | 24.80 | 39.84 | 76.79 | 65.84 | -0.372 |
| 70 | 0.212 | 19.33 | 59.17 | 57.46 | 49.27 | -0.674 |
| 140 | 0.106 | 15.53 | 74.70 | 41.93 | 35.95 | -0.975 |
| 200 | 0.075 | 8.92 | 83.62 | 33.01 | 28.30 | -1.125 |
| pan | | 33.87 | 117.49 | -0.86 | | |

d10: -- (mm) d50: 0.22 (mm)
d16: -- (mm) d60: 0.34 (mm)
d30: 0.081 (mm) d84: 1.00 (mm)

MEDIAN PARTICLE DIAMETER (d50): 0.22
UNIFORMITY COEFFICIENT, Cu (d60/d10): --
COEFFICIENT OF CURVATURE, Cc, [(d30)**2/(d10*d60)]: --
MEAN PARTICLE DIAMETER, [(d16+d50+d84)/3]: --

COMMENTS: d(16) was not reached with sieve analysis.

LABORATORY ANALYSIS PERFORMED BY: L. Simpson
CALCULATIONS MADE BY: S. Stoller
CHECKED BY: E. Mattson




USDA PARTICLE SIZE LIMIT CLASSIFICATION

| $\theta_i$ = 35.58 | $d_{10}$ = * | $d_{50}$ = 0.22 | $d_{60}$ = 0.34 | $C_u$ = -- | $C_c$ = -- |
|---|---|---|---|---|---|
| SAMPLE NUMBER | | DEPTH | CLASSIFICATION (USDA TEXTURE TRIANGLE) | | |
| SHC-5-79.5 | | 79.5 ft | Sandy loam | | |

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SIEVE ANALYSIS DATA

JOB NAME: EMCON
JOB NUMBER: 89-L-060
TEST DATE: 8/2/89
SAMPLE NUMBER: SHC-9-30.5 (a)
DEPTH: 30.5 FT.
TOTAL SAMPLE WEIGHT (WET): 116.83 (g)
TOTAL SAMPLE WEIGHT (DRY): 116.83 (g)
WATER WEIGHT: 0.00 (g)
CONTAINER WEIGHT: 0.00 (g)
TOTAL WEIGHT OF SAMPLE: 116.83 (g)

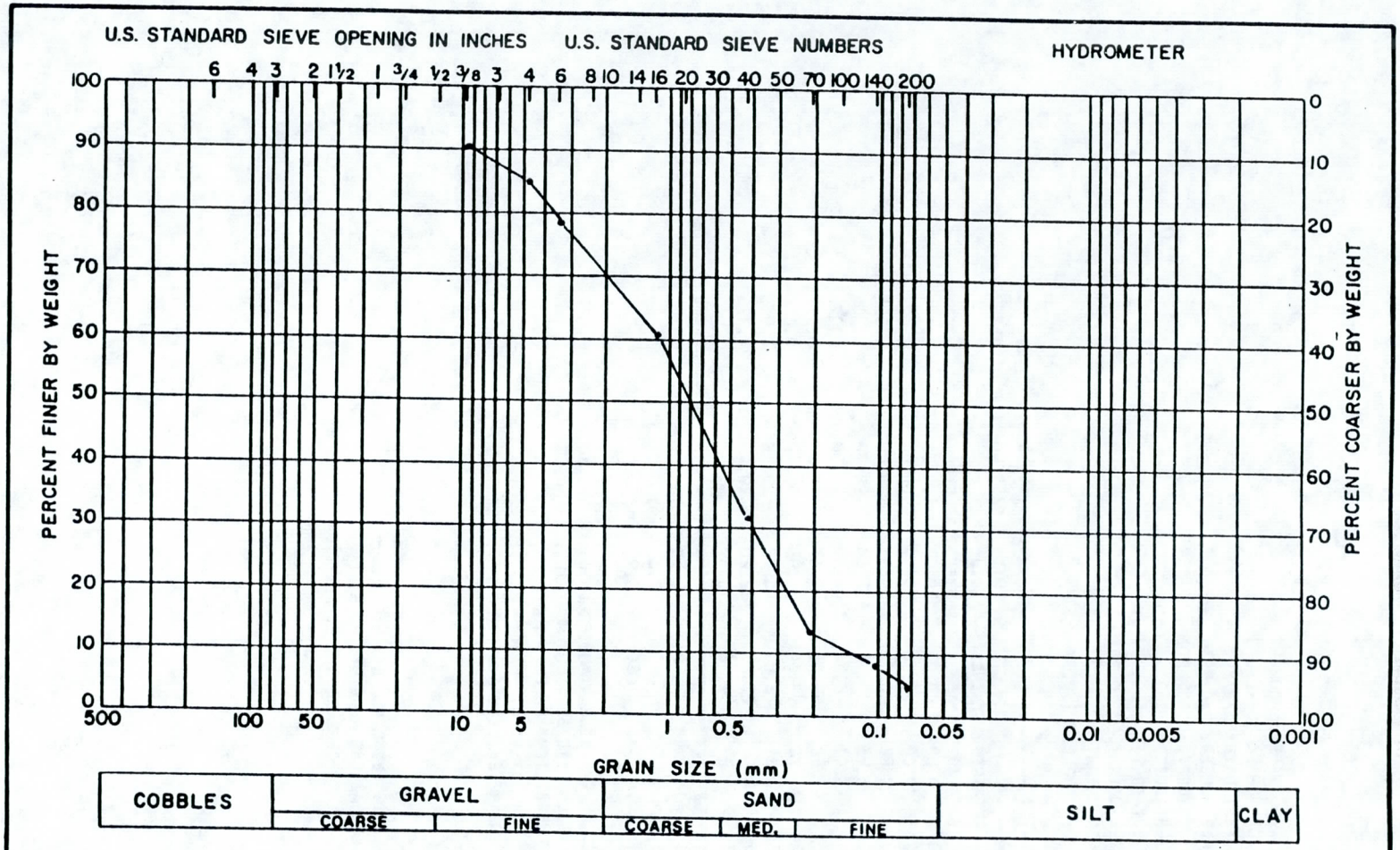
| SIEVE NUMBER | DIAMETER (mm) | WT. RETAINED | CUM WT. RETAINED | WT. PASSING | % PASSING | LOG DIAMETER |
|---|---|---|---|---|---|---|
| 3/8" | 9.525 | 11.34 | 11.34 | 105.49 | 90.29 | 0.979 |
| 4 | 4.750 | 5.29 | 16.63 | 100.20 | 85.77 | 0.677 |
| 6 | 3.350 | 7.07 | 23.70 | 93.13 | 79.71 | 0.525 |
| 10 | 2.000 | 9.33 | 33.03 | 83.80 | 71.73 | 0.301 |
| 16 | 1.180 | 12.19 | 45.22 | 71.61 | 61.29 | 0.072 |
| 40 | 0.425 | 34.13 | 79.35 | 37.48 | 32.08 | -0.372 |
| 70 | 0.212 | 21.35 | 100.70 | 16.13 | 13.81 | -0.674 |
| 140 | 0.106 | 6.78 | 107.48 | 9.35 | 8.00 | -0.975 |
| 200 | 0.075 | 2.50 | 109.98 | 6.85 | 5.86 | -1.125 |
| pan | | 7.33 | 117.31 | -0.48 | | |

d10: 0.13 (mm) d50: 0.80 (mm)
d16: 0.22 (mm) d60: 1.10 (mm)
d30: 0.04 (mm) d84: 4.60 (mm)

MEDIAN PARTICLE DIAMETER (d50): 0.80
UNIFORMITY COEFFICIENT, Cu (d60/d10): 8.46
COEFFICIENT OF CURVATURE, Cc, [(d30)**2/(d10*d60)]: 0.01
MEAN PARTICLE DIAMETER, [(d16+d50+d84)/3]: 1.87

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: L. Simpson
CALCULATIONS MADE BY: S. Stoller
CHECKED BY: E. Mattson




## USDA PARTICLE SIZE LIMIT CLASSIFICATION

| $\theta_i$ = 20.44 | $d_{10}$ = 0.13 | $d_{50}$ = 0.80 | $d_{60}$ = 1.1 | $C_u$ = 8.46 | $C_c$ = 0.01 |
|---|---|---|---|---|---|
| SAMPLE NUMBER | | DEPTH | CLASSIFICATION (USDA TEXTURE TRIANGLE) | | |
| SHC-9-30.5(a) | | 30.5 ft | Sand | | |

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SIEVE ANALYSIS DATA

JOB NAME: EMCON
JOB NUMBER: 89-L-060
TEST DATE: 8/2/89
SAMPLE NUMBER: SHC-9-34.5 (b)
DEPTH: 34.5 FT.
TOTAL SAMPLE WEIGHT (WET): 159.53 (g)
TOTAL SAMPLE WEIGHT (DRY): 159.53 (g)
WATER WEIGHT: 0.00 (g)
CONTAINER WEIGHT: 0.00 (g)
TOTAL WEIGHT OF SAMPLE: 159.53 (g)

| SIEVE NUMBER | DIAMETER (mm) | WT. RETAINED | CUM WT. RETAINED | WT. PASSING | % PASSING | LOG DIAMETER |
|---|---|---|---|---|---|---|
| 3/8" | 9.53 | 0.00 | 0.00 | 159.53 | 100.00 | 0.979 |
| 4 | 4.750 | 0.00 | 0.00 | 159.53 | 100.00 | 0.677 |
| 6 | 3.350 | 0.00 | 0.00 | 159.53 | 100.00 | 0.525 |
| 10.00 | 2.000 | 1.93 | 1.93 | 157.60 | 98.79 | 0.301 |
| 16.00 | 1.180 | 13.41 | 15.34 | 144.19 | 90.38 | 0.072 |
| 40.00 | 0.425 | 41.52 | 56.86 | 102.67 | 64.36 | -0.372 |
| 70.00 | 0.212 | 41.74 | 98.60 | 60.93 | 38.19 | -0.674 |
| 140.00 | 0.106 | 20.21 | 118.81 | 40.72 | 25.52 | -0.975 |
| 200.00 | 0.075 | 8.44 | 127.25 | 32.28 | 20.23 | -1.125 |
| pan | | 31.89 | 159.14 | 0.39 | | |

d10: 0.011 (mm) d50: 0.30 (mm)
d16: 0.031 (mm) d60: 0.38 (mm)
d30: 0.130 (mm) d84: 0.95 (mm)

MEDIAN PARTICLE DIAMETER (d50): 0.30
UNIFORMITY COEFFICIENT, Cu (d60/d10): 34.55
COEFFICIENT OF CURVATURE, Cc, [(d30)**2/(d10*d60)]: 4.04
MEAN PARTICLE DIAMETER, [(d16+d50+d84)/3]: 0.43

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: L. Simpson
CALCULATIONS MADE BY: S. Stoller
CHECKED BY: E. Mattson

HYDROMETER DATA

| | | | |
|---|---|---|---|
| JOB NAME: | EMCON | TEST DATE: | 8/10/89 |
| JOB NUMBER: | 89-L-060 | DEPTH: | 34.5 FT. |
| SAMPLE NUMBER: | SHC-9-34.5 (b) | INITIAL WT: | 31.23 (g) |
| TYPE OF WATER USED: | DISTILLED | TOTAL SAMPLE WT: | 159.53 (g) |
| REACTION WITH H2O2: | NONE | START TIME: | 8:43:00 |
| DISPERSANT: | (NaPO3)6 | | |

| DATE (1989) | TIME (MIN) | TEMP ( C) | R (g/l) | Rl (g/l) | Rcorr (g/l) | L (CM) | D (MM) | P (%) | %FINER |
|---|---|---|---|---|---|---|---|---|---|
| 8/10 | 0.25 | 21.0 | 33.5 | 3.0 | 30.5 | 10.8 | 0.08896 | 97.66250 | 19.11866 |
| | 0.50 | 21.0 | 32.0 | 3.0 | 29.0 | 11.1 | 0.06362 | 92.85943 | 18.17840 |
| | 1.00 | 21.0 | 31.5 | 3.0 | 28.5 | 11.1 | 0.04515 | 91.25841 | 17.86498 |
| | 2.00 | 21.0 | 29.0 | 3.0 | 26.0 | 11.5 | 0.03251 | 83.25328 | 16.29788 |
| | 5.00 | 21.0 | 24.5 | 3.0 | 21.5 | 12.3 | 0.02121 | 68.84406 | 13.47709 |
| | 10.00 | 21.0 | 21.5 | 3.0 | 18.5 | 12.8 | 0.01529 | 59.23791 | 11.59656 |
| | 20.00 | 21.0 | 18.5 | 3.0 | 15.5 | 13.3 | 0.01102 | 49.63176 | 9.71604 |
| | 60.00 | 21.0 | 14.0 | 3.0 | 11.0 | 14.0 | 0.00654 | 35.22254 | 6.89525 |
| | 120.00 | 21.0 | 12.0 | 3.0 | 9.0 | 14.3 | 0.00468 | 28.81844 | 5.64157 |
| | 240.00 | 21.0 | 11.0 | 3.0 | 8.0 | 14.5 | 0.00333 | 25.61639 | 5.01473 |
| | 473.00 | 22.0 | 10.0 | 3.0 | 7.0 | 14.7 | 0.00235 | 22.41435 | 4.38789 |
| | 704.00 | 22.0 | 9.0 | 3.0 | 6.0 | 14.8 | 0.00194 | 19.21230 | 3.76105 |
| 8/11 | 1426.00 | 21.0 | 8.0 | 3.0 | 5.0 | 15.0 | 0.00139 | 16.01025 | 3.13421 |

LABORATORY ANALYSIS PERFORMED BY: L. Simpson
CALCULATIONS MADE BY: S. Stoller
CHECKED BY: E. Mattson

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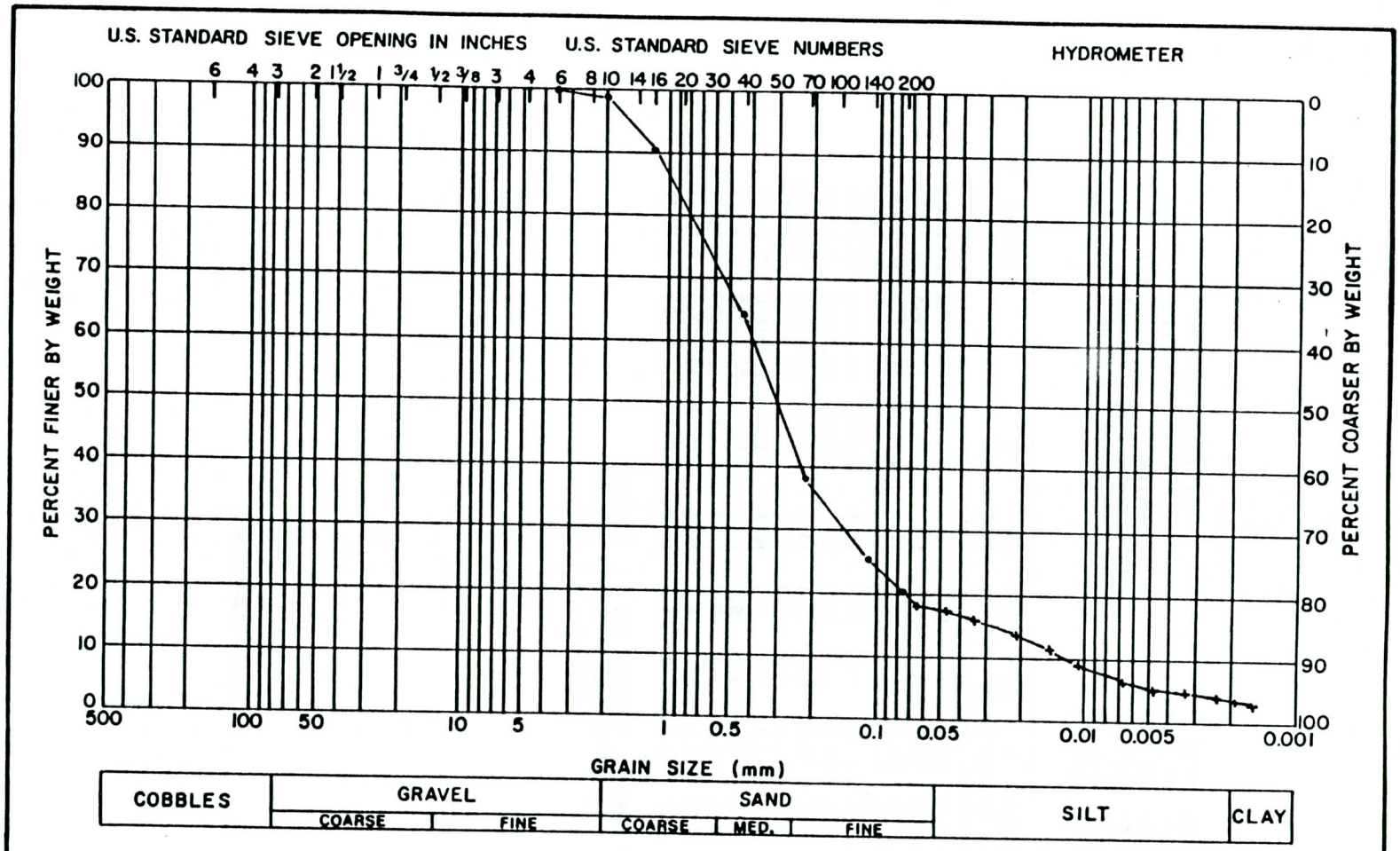



## USDA PARTICLE SIZE LIMIT CLASSIFICATION

| $\theta_i$ = 26.73 | $d_{10}$ = 0.011 | $d_{50}$ = 0.30 | $d_{60}$ = 0.38 | $C_u$ = 34.55 | $C_c$ = 4.04 |
|---|---|---|---|---|---|
| SAMPLE NUMBER | | DEPTH | CLASSIFICATION (USDA TEXTURE TRIANGLE) | | |
| SHC-9-34.5(b) | | 34.5 ft. | loamy sand | | |

SIEVE ANALYSIS-DATA

JOB NAME: EMCON
JOB NUMBER: 89-L-060
TEST DATE: 8/2/89
SAMPLE NUMBER: SHC-9-39.0 (a)
DEPTH: 39.0 FT.
TOTAL SAMPLE WEIGHT (WET): 153.92 (g)
TOTAL SAMPLE WEIGHT (DRY): 153.92 (g)
WATER WEIGHT: 0.00 (g)
CONTAINER WEIGHT: 0.00 (g)
TOTAL WEIGHT OF SAMPLE: 153.92 (g)

| SIEVE NUMBER | DIAMETER (mm) | WT. RETAINED | CUM WT. RETAINED | WT. PASSING | % PASSING | LOG DIAMETER |
|---|---|---|---|---|---|---|
| 4 | 4.750 | 0.00 | 0.00 | 153.92 | 100.00 | 0.677 |
| 6 | 3.350 | 0.11 | 0.11 | 153.81 | 99.93 | 0.525 |
| 10 | 2.000 | 4.85 | 4.96 | 148.96 | 96.78 | 0.301 |
| 16 | 1.180 | 13.23 | 18.19 | 135.73 | 88.18 | 0.072 |
| 40 | 0.425 | 37.62 | 55.81 | 98.11 | 63.74 | -0.372 |
| 70 | 0.212 | 17.83 | 73.64 | 80.28 | 52.16 | -0.674 |
| 140 | 0.106 | 20.97 | 94.61 | 59.31 | 38.53 | -0.975 |
| 200 | 0.075 | 16.82 | 111.43 | 42.49 | 27.61 | -1.125 |
| pan | | 42.27 | 153.70 | 0.22 | | |

d10: 0.020 (mm) d50: 0.20 (mm)
d16: 0.040 (mm) d60: 0.36 (mm)
d30: 0.082 (mm) d84: 0.98 (mm)

MEDIAN PARTICLE DIAMETER (d50): 0.20
UNIFORMITY COEFFICIENT, Cu (d60/d10): 18.00
COEFFICIENT OF CURVATURE, Cc, [(d30)**2/(d10*d60)]: 0.93
MEAN PARTICLE DIAMETER, [(d16+d50+d84)/3]: 0.41

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: L. Simpson
CALCULATIONS MADE BY: S. Stoller
CHECKED BY: E. Mattson

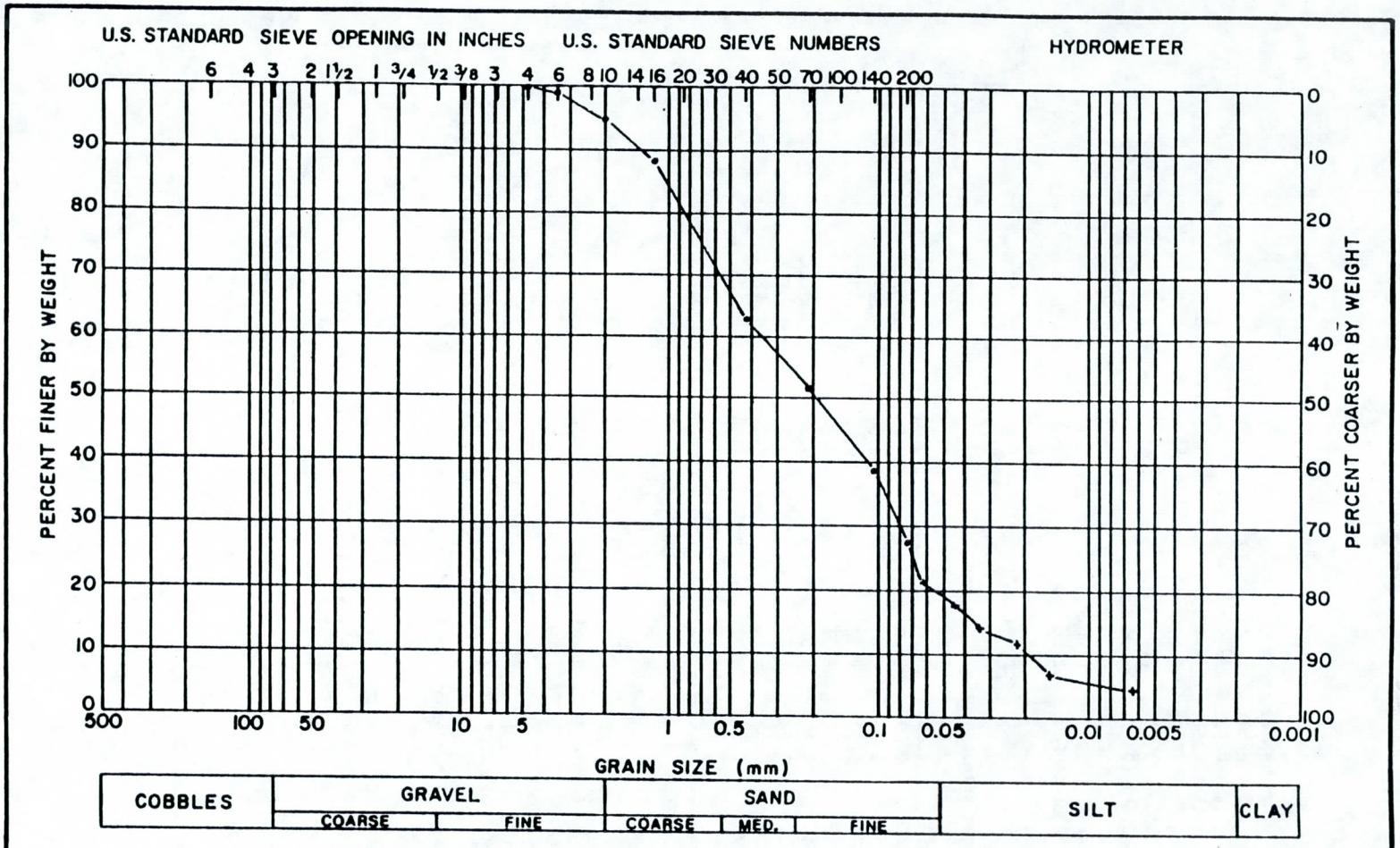
Printed on Recycled Paper

HYDROMETER DATA

| | | | |
|---|---|---|---|
| JOB NAME: | EMCON | TEST DATE: | 8/10/89 |
| JOB NUMBER: | 89-L-060 | DEPTH: | 39.0 FT. |
| SAMPLE NUMBER: | SHC-9-39.0 (a) | INITIAL WT: | 41.56 (g) |
| TYPE OF WATER USED: | DISTILLED | TOTAL SAMPLE WT: | 153.92 (g) |
| REACTION WITH H202: | NONE | START TIME: | 8:28:00 |
| DISPERSANT: | (NaPO3)6 | | |

| DATE (1989) | TIME (MIN) | TEMP ( C) | R (g/l) | Rl (g/l) | Rcorr (g/l) | L (CM) | D (MM) | P (%) | %FINER |
|---|---|---|---|---|---|---|---|---|---|
| 8/10 | 0.25 | 21.0 | 37.0 | 3.0 | 34.0 | 10.2 | 0.08657 | 81.80943 | 22.08940 |
| | 0.50 | 21.0 | 36.0 | 3.0 | 33.0 | 10.4 | 0.06170 | 79.40327 | 21.43971 |
| | 1.00 | 21.0 | 32.0 | 3.0 | 29.0 | 11.1 | 0.04498 | 69.77863 | 18.84096 |
| | 2.00 | 21.0 | 26.5 | 3.0 | 23.5 | 12.0 | 0.03308 | 56.54475 | 15.26767 |
| | 5.00 | 21.0 | 20.5 | 3.0 | 17.5 | 12.9 | 0.02177 | 42.10780 | 11.36954 |
| | 10.00 | 21.0 | 16.5 | 3.0 | 13.5 | 13.6 | 0.01578 | 32.48316 | 8.77079 |
| | 21.00 | 21.0 | 14.0 | 3.0 | 11.0 | 14.0 | 0.01105 | 26.46776 | 7.14657 |
| | 60.00 | 21.0 | 11.5 | 3.0 | 8.5 | 14.4 | 0.00663 | 20.45236 | 5.52235 |
| | 121.00 | 21.0 | 10.0 | 3.0 | 7.0 | 14.7 | 0.00471 | 16.84312 | 4.54782 |
| | 240.00 | 21.0 | 9.0 | 3.0 | 6.0 | 14.8 | 0.00336 | 14.43696 | 3.89813 |
| | 487.00 | 22.0 | 8.0 | 3.0 | 5.0 | 15.0 | 0.00235 | 12.03080 | 3.24844 |
| | 718.00 | 22.0 | 8.0 | 3.0 | 5.0 | 15.0 | 0.00193 | 12.03080 | 3.24844 |
| 8/11 | 1440.00 | 21.0 | 7.5 | 3.0 | 4.5 | 15.1 | 0.00138 | 10.82772 | 2.92360 |

LABORATORY ANALYSIS PERFORMED BY: M. Burkhard
CALCULATIONS MADE BY: M. Burkhard
CHECKED BY: E. Mattson




## USDA PARTICLE SIZE LIMIT CLASSIFICATION

| $\theta_1$ = 31.39 | $d_{10}$ = 0.02 | $d_{50}$ = 0.20 | $d_{60}$ = 0.36 | $C_u$ = 18.00 | $C_c$ = 0.93 |
|---|---|---|---|---|---|
| SAMPLE NUMBER | | DEPTH | CLASSIFICATION (USDA TEXTURE TRIANGLE) | | |
| SHC-9-39.0(a) | | 39.0 ft. | loamy sand | | |

Printed on Recycled Paper

SIEVE ANALYSIS DATA

JOB NAME: EMCON
JOB NUMBER: 89-L-060
TEST DATE: 8/22/89
SAMPLE NUMBER: SHC-9-49.5
DEPTH: 49.5 FT.
TOTAL SAMPLE WEIGHT (WET): 52.86 (g)
TOTAL SAMPLE WEIGHT (DRY): 52.86 (g)
WATER WEIGHT: 0.00 (g)
CONTAINER WEIGHT: 0.00 (g)
TOTAL WEIGHT OF SAMPLE: 52.86 (g)

| SIEVE NUMBER | DIAMETER (mm) | WT. RETAINED | CUM WT. RETAINED | WT. PASSING | % PASSING | LOG DIAMETER |
|---|---|---|---|---|---|---|
| 3/8" | 9.525 | 0.00 | 0.00 | 52.86 | 100.00 | 0.979 |
| 4 | 4.750 | 0.00 | 0.00 | 52.86 | 100.00 | 0.677 |
| 6 | 3.350 | 0.00 | 0.00 | 52.86 | 100.00 | 0.525 |
| 10 | 2.000 | 0.02 | 0.02 | 52.84 | 99.96 | 0.301 |
| 16 | 1.180 | 1.21 | 1.23 | 51.63 | 97.67 | 0.072 |
| 40 | 0.425 | 4.16 | 5.39 | 47.47 | 89.80 | -0.372 |
| 70 | 0.212 | 1.92 | 7.31 | 45.55 | 86.17 | -0.674 |
| 140 | 0.106 | 1.63 | 8.94 | 43.92 | 83.09 | -0.975 |
| 200 | 0.075 | 0.77 | 9.71 | 43.15 | 81.63 | -1.125 |
| pan | | 43.20 | 52.91 | -0.05 | | |

d10: -- (mm) d50: 0.0054 (mm)
d16: -- (mm) d60: 0.012 (mm)
d30: 0.0022 (mm) d84: 0.20 (mm)

MEDIAN PARTICLE DIAMETER (d50): 0.0054
UNIFORMITY COEFFICIENT, Cu (d60/d10): --
COEFFICIENT OF CURVATURE, Cc, [(d30)**2/(d10*d60)]: --
MEAN PARTICLE DIAMETER, [(d16+d50+d84)/3]: --

COMMENTS: d16 was not reached during hydrometer analysis.

LABORATORY ANALYSIS PERFORMED BY: S. Stoller
CALCULATIONS MADE BY: S. Stoller
CHECKED BY: E. Mattson

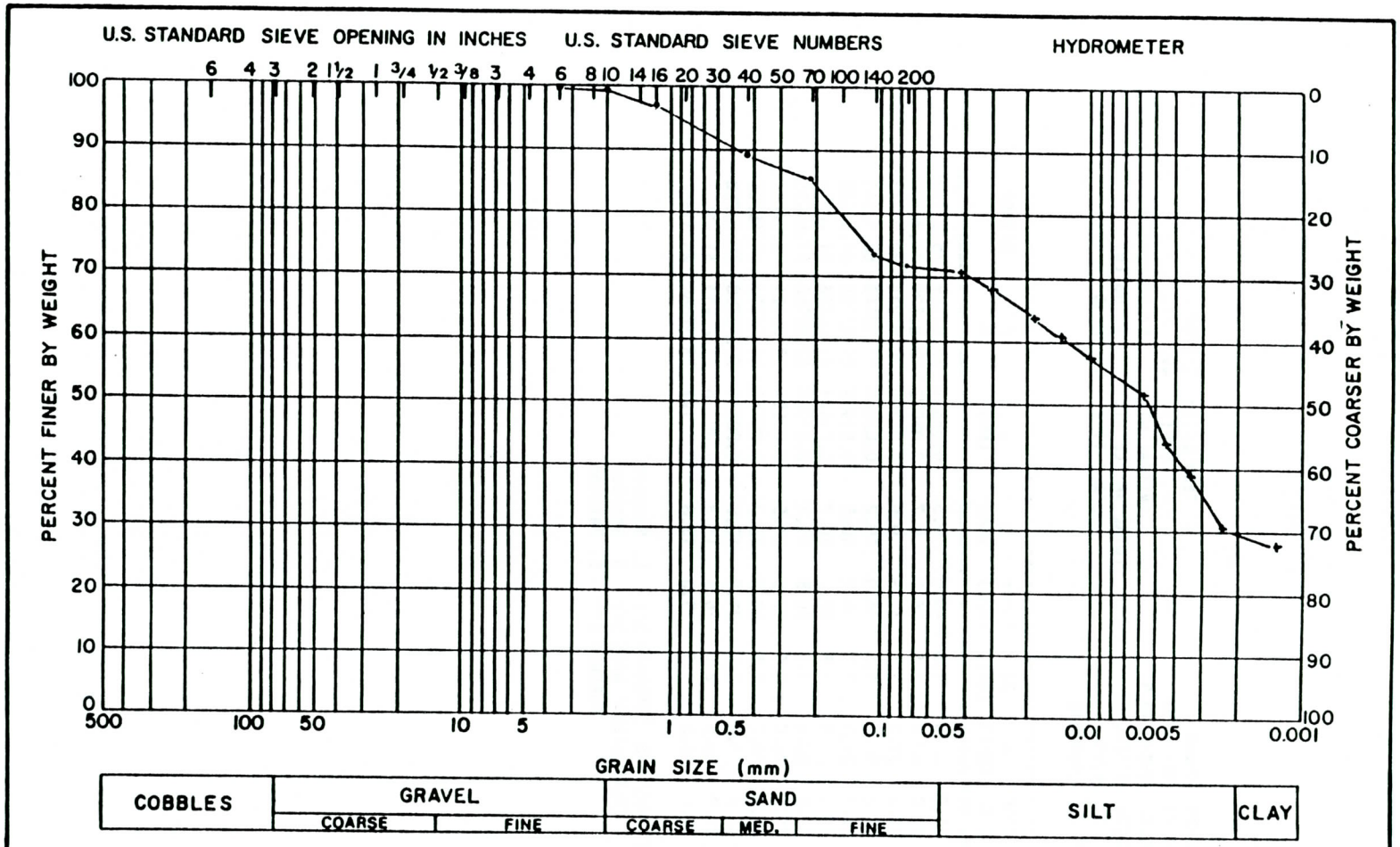
HYDROMETER DATA

| | | | |
|---|---|---|---|
| JOB NAME: | EMCON | TEST DATE: | 8/3/89 |
| JOB NUMBER: | 89-L-060 | DEPTH: | 49.5 FT. |
| SAMPLE NUMBER: | SHC-9-49.5 | INITIAL WT: | 52.86 (g) |
| TYPE OF WATER USED: | DISTILLED | TOTAL SAMPLE WT: | 52.86 (g) |
| REACTION WITH H202: | MILD | START TIME: | 11:18:00 |
| DISPERSANT: | (NaPO3)6 | | |

| DATE (1989) | TIME (MIN) | TEMP ( C) | R (g/l) | Rl (g/l) | Rcorr (g/l) | L (CM) | D (MM) | P (%) | %FINER |
|---|---|---|---|---|---|---|---|---|---|
| 8/3 | 0.25 | 21.0 | 41.5 | 1.5 | 40.0 | 9.5 | 0.08339 | 75.67159 | 75.67159 |
| | 0.50 | 21.0 | 40.0 | 1.5 | 38.5 | 9.7 | 0.05972 | 72.83390 | 72.83390 |
| | 1.00 | 21.0 | 39.0 | 1.5 | 37.5 | 9.9 | 0.04259 | 70.94211 | 70.94211 |
| | 2.00 | 21.0 | 37.5 | 1.5 | 36.0 | 10.2 | 0.03048 | 68.10443 | 68.10443 |
| | 5.00 | 21.0 | 35.5 | 1.5 | 34.0 | 10.5 | 0.01959 | 64.32085 | 64.32085 |
| | 10.00 | 21.0 | 34.0 | 1.5 | 32.5 | 10.7 | 0.01401 | 61.48316 | 61.48316 |
| | 20.00 | 21.0 | 32.0 | 1.5 | 30.5 | 11.1 | 0.01006 | 57.69958 | 57.69958 |
| | 60.00 | 21.0 | 29.0 | 1.5 | 27.5 | 11.5 | 0.00594 | 52.02421 | 52.02421 |
| | 120.00 | 21.0 | 25.0 | 1.5 | 23.5 | 12.2 | 0.00431 | 44.45706 | 44.45706 |
| | 206.00 | 21.0 | 23.0 | 2.0 | 21.0 | 12.5 | 0.00334 | 39.72758 | 39.72758 |
| | 477.00 | 21.0 | 18.5 | 2.0 | 16.5 | 13.3 | 0.00226 | 31.21453 | 31.21453 |
| 8/4 | 1242.00 | 21.0 | 16.5 | 1.5 | 15.0 | 13.6 | 0.00142 | 28.37684 | 28.37684 |

LABORATORY ANALYSIS PERFORMED BY: L. Simpson
CALCULATIONS MADE BY: S. Stoller
CHECKED BY: E. Mattson

Printed on Recycled Paper




## USDA PARTICLE SIZE LIMIT CLASSIFICATION

| $\theta_i$ = 45.00 | $d_{10}$ = * | $d_{50}$ = 0.0054 | $d_{60}$ = 0.012 | $C_u$ = -- | $C_c$ = -- |
|---|---|---|---|---|---|
| SAMPLE NUMBER | | DEPTH | CLASSIFICATION (USDA TEXTURE TRIANGLE) | | |
| SHC-9-49.5 | | 49.5 ft | Clay loam | | |

SIEVE ANALYSIS DATA

JOB NAME: EMCON
JOB NUMBER: 89-L-060
TEST DATE: 8/9/89
SAMPLE NUMBER: SHC-9-59.5 (a)
DEPTH: 59.5 FT.
TOTAL SAMPLE WEIGHT (WET): 164.25 (g)
TOTAL SAMPLE WEIGHT (DRY): 164.25 (g)
WATER WEIGHT: 0.00 (g)
CONTAINER WEIGHT: 0.00 (g)
TOTAL WEIGHT OF SAMPLE: 164.25 (g)

| SIEVE NUMBER | DIAMETER (mm) | WT. RETAINED | CUM WT. RETAINED | WT. PASSING | % PASSING | LOG DIAMETER |
|---|---|---|---|---|---|---|
| 3/8" | 9.525 | 3.33 | 3.33 | 160.92 | 97.97 | 0.979 |
| 4 | 4.750 | 0.00 | 3.33 | 160.92 | 97.97 | 0.677 |
| 6 | 3.350 | 0.12 | 3.45 | 160.80 | 97.90 | 0.525 |
| 10 | 2.000 | 0.04 | 3.49 | 160.76 | 97.88 | 0.301 |
| 16 | 1.180 | 2.98 | 6.47 | 157.78 | 96.06 | 0.072 |
| 40 | 0.425 | 53.12 | 59.59 | 104.66 | 63.72 | -0.372 |
| 70 | 0.212 | 28.27 | 87.86 | 76.39 | 46.51 | -0.674 |
| 140 | 0.106 | 19.50 | 107.36 | 56.89 | 34.64 | -0.975 |
| 200 | 0.075 | 11.84 | 119.20 | 45.05 | 27.43 | -1.125 |
| pan | | 44.49 | 163.69 | 0.56 | | |

d10: 0.0152 (mm) d50: 0.26 (mm)
d16: 0.032 (mm) d60: 0.38 (mm)
d30: 0.09 (mm) d84: 0.8 (mm)

MEDIAN PARTICLE DIAMETER (d50): 0.26
UNIFORMITY COEFFICIENT, Cu (d60/d10): 25.00
COEFFICIENT OF CURVATURE, Cc, [(d30)**2/(d10*d60)]: 1.40
MEAN PARTICLE DIAMETER, [(d16+d50+d84)/3]: 0.36

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: L. Simpson
CALCULATIONS MADE BY: S. Stoller
CHECKED BY: E. Mattson

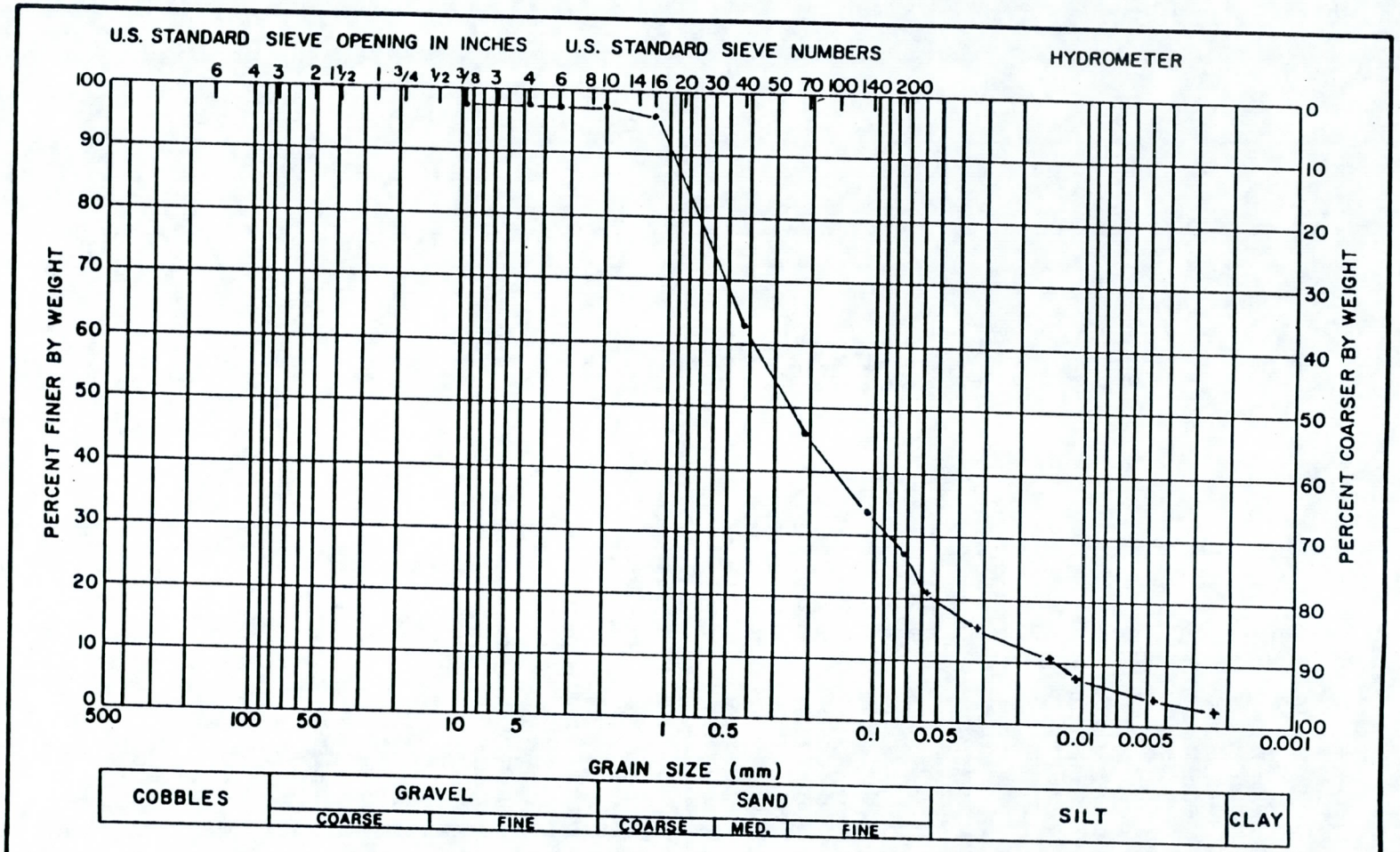
Printed on Recycled Paper

HYDROMETER DATA

| | | | |
|---|---|---|---|
| JOB NAME: | EMCON | TEST DATE: | 8/10/89 |
| JOB NUMBER: | 89-L-060 | DEPTH: | 59.5 FT. |
| SAMPLE NUMBER: | SHC-9-59.5 (a) | INITIAL WT: | 48.08 (g) |
| TYPE OF WATER USED: | DISTILLED | TOTAL SAMPLE WT: | 164.25 (g) |
| REACTION WITH H2O2: | VERY MILD | START TIME: | 8:22:00 |
| DISPERSANT: | (NaPO3)6 | | |

| DATE (1989) | TIME (MIN) | TEMP ( C) | R (g/l) | Rl (g/l) | Rcorr (g/l) | L (CM) | D (MM) | P (%) | %FINER |
|---|---|---|---|---|---|---|---|---|---|
| 8/10 | 0.25 | 21.0 | 40.5 | 3.0 | 37.5 | 9.7 | 0.08411 | 77.99501 | 22.83105 |
| | 0.50 | 21.0 | 39.0 | 3.0 | 36.0 | 9.9 | 0.06023 | 74.87521 | 21.91781 |
| | 1.00 | 21.0 | 35.0 | 3.0 | 32.0 | 10.6 | 0.04397 | 66.55574 | 19.48250 |
| | 2.00 | 21.0 | 30.0 | 3.0 | 27.0 | 11.4 | 0.03228 | 56.15641 | 16.43836 |
| | 5.00 | 21.0 | 24.0 | 3.0 | 21.0 | 12.4 | 0.02128 | 43.67720 | 12.78539 |
| | 10.00 | 21.0 | 20.5 | 3.0 | 17.5 | 12.9 | 0.01539 | 36.39767 | 10.65449 |
| | 20.00 | 21.0 | 17.0 | 3.0 | 14.0 | 13.5 | 0.01112 | 29.11814 | 8.52359 |
| | 60.00 | 21.0 | 13.0 | 3.0 | 10.0 | 14.2 | 0.00657 | 20.79867 | 6.08828 |
| | 121.00 | 21.0 | 11.5 | 3.0 | 8.5 | 14.4 | 0.00467 | 17.67887 | 5.17504 |
| | 240.00 | 21.0 | 10.0 | 3.0 | 7.0 | 14.7 | 0.00334 | 14.55907 | 4.26180 |
| | 492.00 | 22.0 | 9.0 | 3.0 | 6.0 | 14.8 | 0.00232 | 12.47920 | 3.65297 |
| | 722.00 | 22.0 | 8.0 | 3.0 | 5.0 | 15.0 | 0.00193 | 10.39933 | 3.04414 |
| 8/11 | 1443.00 | 21.0 | 7.5 | 3.0 | 4.5 | 15.1 | 0.00138 | 9.35940 | 2.73973 |

LABORATORY ANALYSIS PERFORMED BY: M. BURKHARD
CALCULATIONS MADE BY: M. BURKHARD
CHECKED BY: E. Mattson




## USDA PARTICLE SIZE LIMIT CLASSIFICATION

| $\theta_1$ = 31.37 | $d_{10}$ = 0.0152 | $d_{50}$ = 0.26 | $d_{60}$ = 0.38 | $C_u$ = 25.00 | $C_c$ = 1.40 |
|---|---|---|---|---|---|
| SAMPLE NUMBER | | DEPTH | CLASSIFICATION (USDA TEXTURE TRIANGLE) | | |
| SHC-9-59.5(a) | | 59.5 ft. | loamy sand | | |

Printed on Recycled Paper

SIEVE ANALYSIS DATA

JOB NAME: EMCON
JOB NUMBER: 89-L-060
TEST DATE: 8/2/89
SAMPLE NUMBER: SHC-9-69.5 (b)
DEPTH: 69.5 FT.
TOTAL SAMPLE WEIGHT (WET): 108.41 (g)
TOTAL SAMPLE WEIGHT (DRY): 108.41 (g)
WATER WEIGHT: 0.00 (g)
CONTAINER WEIGHT: 0.00 (g)
TOTAL WEIGHT OF SAMPLE: 108.41 (g)

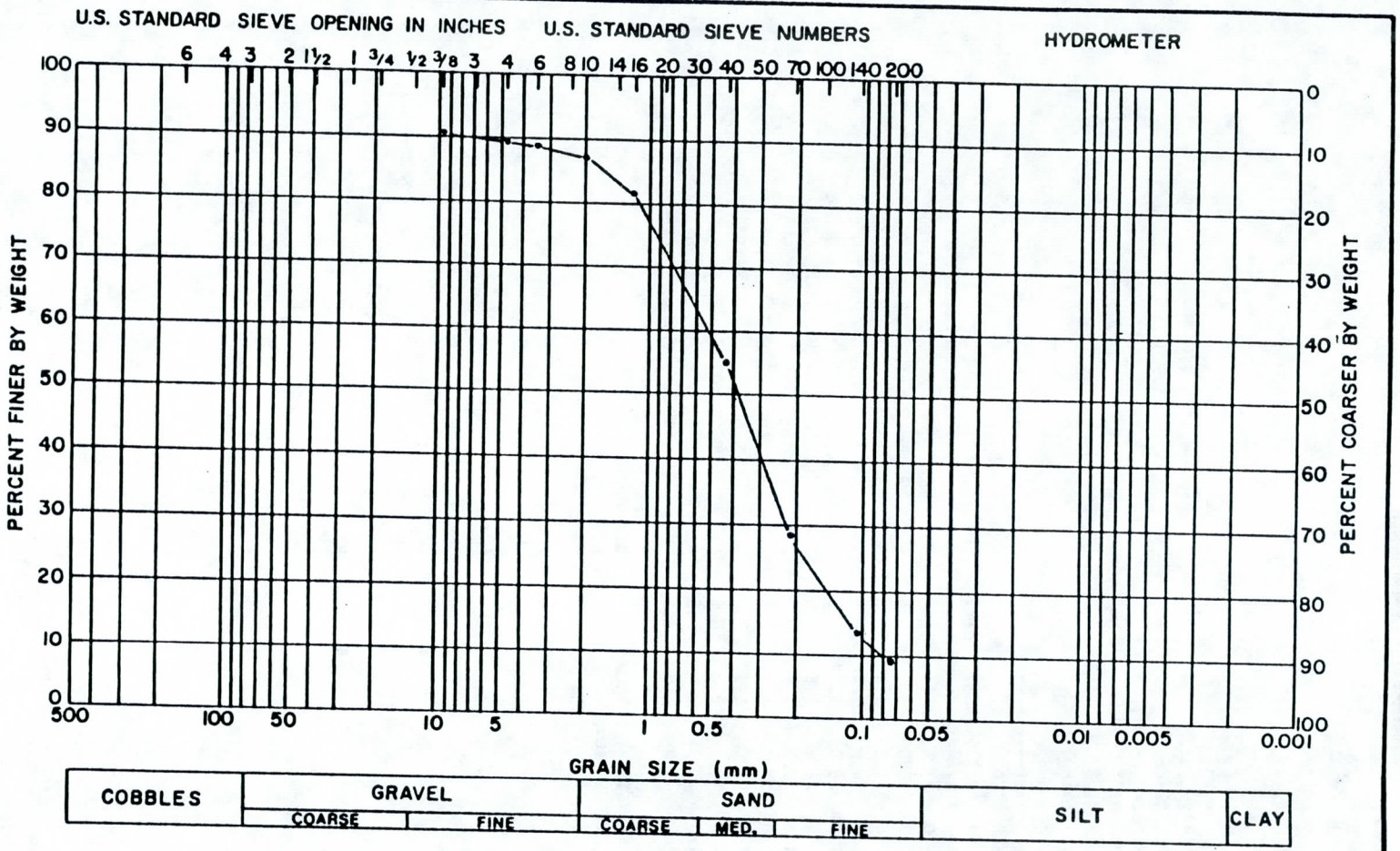
| SIEVE NUMBER | DIAMETER (mm) | WT. RETAINED | CUM WT. RETAINED | WT. PASSING | % PASSING | LOG DIAMETER |
|---|---|---|---|---|---|---|
| 3/8" | 9.525 | 10.18 | 10.18 | 98.23 | 90.61 | 0.979 |
| 4 | 4.750 | 0.70 | 10.88 | 97.53 | 89.96 | 0.677 |
| 6 | 3.350 | 1.71 | 12.59 | 95.82 | 88.39 | 0.525 |
| 10 | 2.000 | 1.42 | 14.01 | 94.40 | 87.08 | 0.301 |
| 16 | 1.180 | 5.15 | 19.16 | 89.25 | 82.33 | 0.072 |
| 40 | 0.425 | 29.65 | 48.81 | 59.60 | 54.98 | -0.372 |
| 70 | 0.212 | 29.48 | 78.29 | 30.12 | 27.78 | -0.674 |
| 140 | 0.106 | 15.48 | 93.77 | 14.64 | 13.50 | -0.975 |
| 200 | 0.075 | 5.02 | 98.79 | 9.62 | 8.87 | -1.125 |
| pan | | 10.57 | 109.36 | -0.95 | | |

d10: 0.08 (mm) d50: 0.38 (mm)
d16: 0.12 (mm) d60: 0.51 (mm)
d30: 0.22 (mm) d84: 1.50 (mm)

MEDIAN PARTICLE DIAMETER (d50): 0.38
UNIFORMITY COEFFICIENT, Cu (d60/d10): 6.38
COEFFICIENT OF CURVATURE, Cc, [(d30)**2/(d10*d60)]: 1.19
MEAN PARTICLE DIAMETER, [(d16+d50+d84)/3]: 0.67

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: L. Simpson
CALCULATIONS MADE BY: S. Stoller
CHECKED BY: E. Mattson




## USDA PARTICLE SIZE LIMIT CLASSIFICATION

| $\Theta_i$ = 20.97 | $d_{10}$ = 0.08 | $d_{50}$ = 0.38 | $d_{60}$ = 0.51 | $C_u$ = 6.38 | $C_c$ = 1.19 |
|---|---|---|---|---|---|
| SAMPLE NUMBER | | DEPTH | CLASSIFICATION (USDA TEXTURE TRIANGLE) | | |
| SHC-9-69.5(b) | | 69.5 ft. | Sand | | |

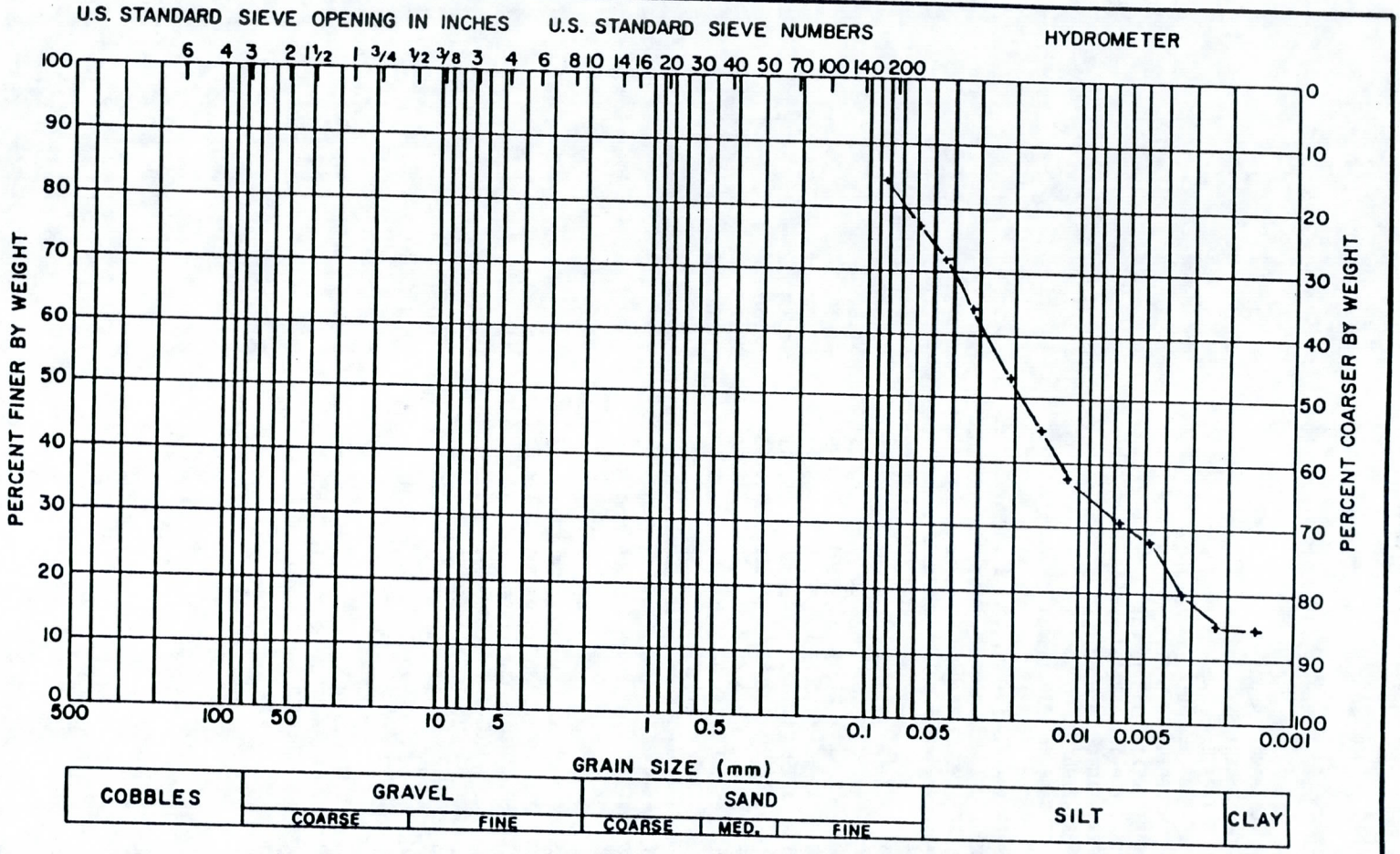
Printed on Recycled Paper

HYDROMETER DATA

| | | | |
|---|---|---|---|
| JOB NAME: | EMCON | TEST DATE: | 8/3/89 |
| JOB NUMBER: | 89-L-060 | DEPTH: | 103.0 FT. |
| SAMPLE NUMBER: | SHC-9-103.0 (a) | INITIAL WT: | 49.94 (g) |
| TYPE OF WATER USED: | DISTILLED | TOTAL SAMPLE WT: | 49.94 (g) |
| REACTION WITH H202: | MILD | START TIME: | 10:17:30 |
| DISPERSANT: | (NaPO3)6 | | |

| DATE (1989) | TIME (MIN) | TEMP ( C) | R (g/l) | Rl (g/l) | Rcorr (g/l) | L (CM) | D (MM) | P (%) | %FINER |
|---|---|---|---|---|---|---|---|---|---|
| 8/3 | 0.25 | 21.0 | 43.5 | 1.5 | 42.0 | 9.2 | 0.08194 | 84.10092 | 84.10092 |
| | 0.50 | 21.0 | 40.0 | 1.5 | 38.5 | 9.7 | 0.05972 | 77.09251 | 77.09251 |
| | 1.00 | 21.0 | 37.5 | 1.5 | 36.0 | 10.2 | 0.04311 | 72.08650 | 72.08650 |
| | 2.00 | 21.0 | 34.0 | 1.5 | 32.5 | 10.7 | 0.03133 | 65.07809 | 65.07809 |
| | 5.00 | 21.0 | 28.0 | 1.5 | 26.5 | 11.7 | 0.02071 | 53.06368 | 53.06368 |
| | 10.00 | 21.0 | 24.0 | 1.5 | 22.5 | 12.4 | 0.01505 | 45.05406 | 45.05406 |
| | 20.00 | 21.0 | 20.5 | 1.5 | 19.0 | 12.9 | 0.01088 | 38.04565 | 38.04565 |
| | 60.00 | 21.0 | 17.0 | 1.5 | 15.5 | 13.5 | 0.00642 | 31.03724 | 31.03724 |
| | 120.00 | 21.0 | 15.0 | 1.5 | 13.5 | 13.8 | 0.00460 | 27.03244 | 27.03244 |
| | 261.00 | 21.0 | 12.0 | 2.0 | 10.0 | 14.3 | 0.00317 | 20.02403 | 20.02403 |
| | 532.50 | 21.0 | 9.5 | 2.0 | 7.5 | 14.7 | 0.00225 | 15.01802 | 15.01802 |
| 8/4 | 1297.50 | 21.0 | 9.0 | 1.5 | 7.5 | 14.8 | 0.00145 | 15.01802 | 15.01802 |

LABORATORY ANALYSIS PERFORMED BY: L. Simpson
CALCULATIONS MADE BY: S. Stoller
CHECKED BY: E. Mattson




## USDA PARTICLE SIZE LIMIT CLASSIFICATION

| $\theta_i$ = 38.38 | $d_{10}$ = * | $d_{50}$ = 0.018 | $d_{60}$ = 0.027 | $C_u$ = -- | $C_c$ = -- |
|---|---|---|---|---|---|
| SAMPLE NUMBER | | DEPTH | CLASSIFICATION (USDA TEXTURE TRIANGLE) | | |
| SHC-9-103.0(a) | | 103 ft. | Silt loam | | |

Printed on Recycled Paper

SIEVE ANALYSIS DATA

JOB NAME: EMCON
JOB NUMBER: 89-L-060
TEST DATE: 8/2/89
SAMPLE NUMBER: SHC-22-23.5
DEPTH: 23.5 FT.
TOTAL SAMPLE WEIGHT (WET): 117.10 (g)
TOTAL SAMPLE WEIGHT (DRY): 117.10 (g)
WATER WEIGHT: 0.00 (g)
CONTAINER WEIGHT: 0.00 (g)
TOTAL WEIGHT OF SAMPLE: 117.10 (g)

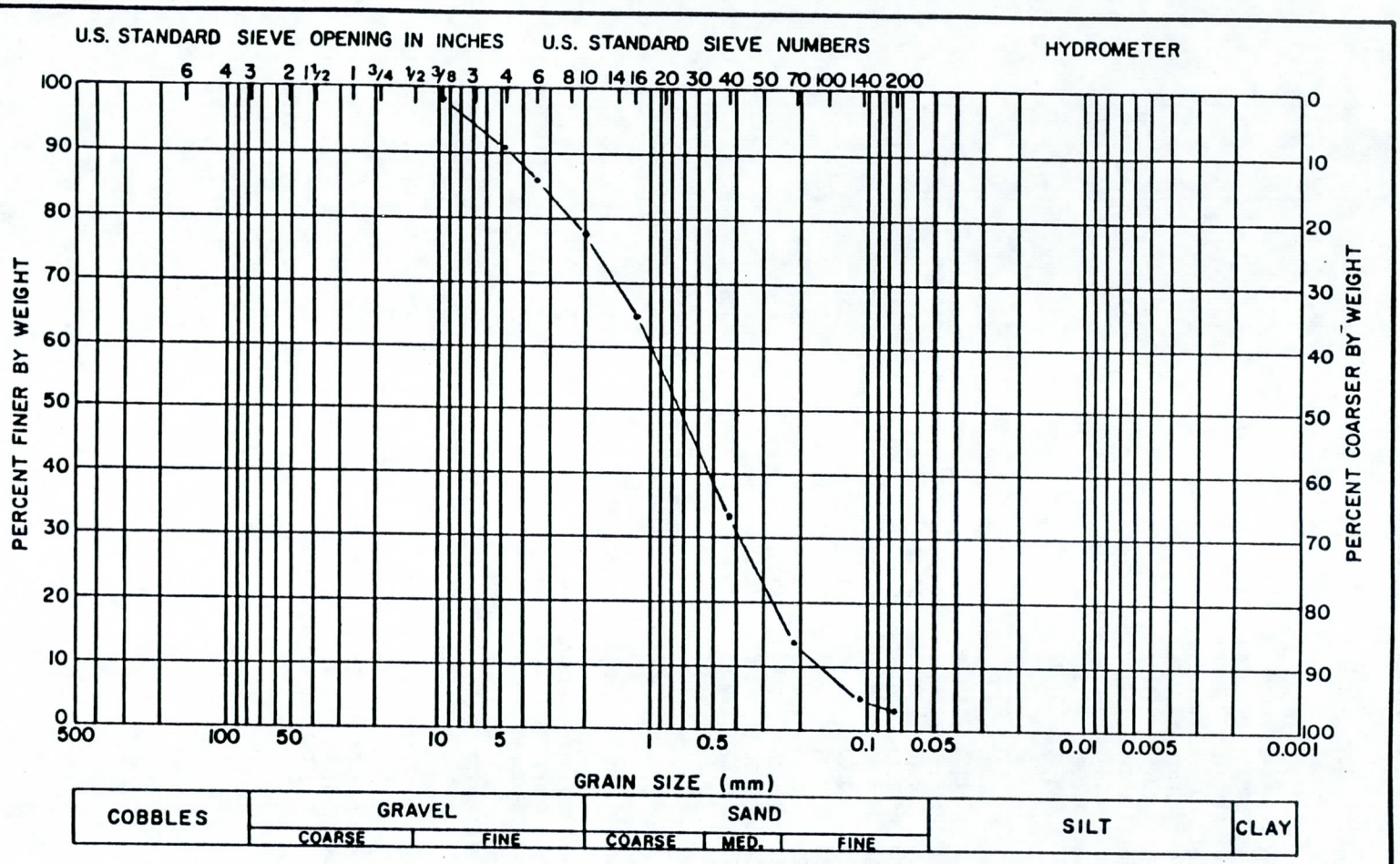
| SIEVE NUMBER | DIAMETER (mm) | WT. RETAINED | CUM WT. RETAINED | WT. PASSING | % PASSING | LOG DIAMETER |
|---|---|---|---|---|---|---|
| 3/8" | 9.525 | 2.60 | 2.60 | 114.50 | 97.78 | 0.979 |
| 4 | 4.750 | 7.43 | 10.03 | 107.07 | 91.43 | 0.677 |
| 6 | 3.350 | 6.30 | 16.33 | 100.77 | 86.05 | 0.525 |
| 10 | 2.000 | 8.86 | 25.19 | 91.91 | 78.49 | 0.301 |
| 16 | 1.180 | 15.56 | 40.75 | 76.35 | 65.20 | 0.072 |
| 40 | 0.425 | 37.55 | 78.30 | 38.80 | 33.13 | -0.372 |
| 70 | 0.212 | 22.33 | 100.63 | 16.47 | 14.06 | -0.674 |
| 140 | 0.106 | 10.08 | 110.71 | 6.39 | 5.46 | -0.975 |
| 200 | 0.075 | 2.31 | 113.02 | 4.08 | 3.48 | -1.125 |
| pan | | 3.72 | 116.74 | 0.36 | | |

d10: 0.15 (mm) d50: 0.73 (mm)
d16: 0.22 (mm) d60: 1.00 (mm)
d30: 0.38 (mm) d84: 3.20 (mm)

MEDIAN PARTICLE DIAMETER (d50): 0.73
UNIFORMITY COEFFICIENT, Cu (d60/d10): 6.67
COEFFICIENT OF CURVATURE, Cc, [(d30)**2/(d10*d60)]: 0.96
MEAN PARTICLE DIAMETER, [(d16+d50+d84)/3]: 1.38

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: L. Simpson
CALCULATIONS MADE BY: S. Stoller
CHECKED BY: E. Mattson




## USDA PARTICLE SIZE LIMIT CLASSIFICATION

| $\theta_i$ = 13.91 | $d_{10}$ = 0.15 | $d_{50}$ = 0.73 | $d_{60}$ = 1.0 | $C_u$ = 6.67 | $C_c$ = 0.96 |
|---|---|---|---|---|---|
| SAMPLE NUMBER | | DEPTH | CLASSIFICATION (USDA TEXTURE TRIANGLE) | | |
| SHC-22-23.5 | | 23.5 ft. | Sand | | |

Printed on Recycled Paper

SIEVE ANALYSIS DATA

JOB NAME: EMCON
JOB NUMBER: 89-L-060
TEST DATE: 8/22/89
SAMPLE NUMBER: SHC-22-52.5
DEPTH: 52.5 FT.
TOTAL SAMPLE WEIGHT (WET): 51.22 (g)
TOTAL SAMPLE WEIGHT (DRY): 51.22 (g)
WATER WEIGHT: 0.00 (g)
CONTAINER WEIGHT: 0.00 (g)
TOTAL WEIGHT OF SAMPLE: 51.22 (g)

| SIEVE NUMBER | DIAMETER (mm) | WT. RETAINED | CUM WT. RETAINED | WT. PASSING | % PASSING | LOG DIAMETER |
|---|---|---|---|---|---|---|
| 3/8" | 9.525 | 0.00 | 0.00 | 51.22 | 100.00 | 0.979 |
| 4 | 4.750 | 0.00 | 0.00 | 51.22 | 100.00 | 0.677 |
| 6 | 3.350 | 0.00 | 0.00 | 51.22 | 100.00 | 0.525 |
| 10 | 2.000 | 0.00 | 0.00 | 51.22 | 100.00 | 0.301 |
| 16 | 1.180 | 0.03 | 0.03 | 51.19 | 99.94 | 0.072 |
| 40 | 0.425 | 2.40 | 2.43 | 48.79 | 95.26 | -0.372 |
| 70 | 0.212 | 6.48 | 8.91 | 42.31 | 82.60 | -0.674 |
| 140 | 0.106 | 8.18 | 17.09 | 34.13 | 66.63 | -0.975 |
| 200 | 0.075 | 3.44 | 20.53 | 30.69 | 59.92 | -1.125 |
| pan | | 30.61 | 51.14 | 0.08 | | |

d10: 0.0044 (mm) d50: 0.07 (mm)
d16: 0.0072 (mm) d60: 0.08 (mm)
d30: 0.026 (mm) d84: 0.24 (mm)

MEDIAN PARTICLE DIAMETER (d50): 0.07
UNIFORMITY COEFFICIENT, Cu (d60/d10): 18.18
COEFFICIENT OF CURVATURE, Cc, [(d30)**2/(d10*d60)]: 1.92
MEAN PARTICLE DIAMETER, [(d16+d50+d84)/3]: 0.11

COMMENTS:

LABORATORY ANALYSIS PERFORMED BY: S. Stoller
CALCULATIONS MADE BY: S. Stoller
CHECKED BY: E. Mattson

HYDROMETER DATA

| | | | |
|---|---|---|---|
| JOB NAME: | EMCON | TEST DATE: | 8/3/89 |
| JOB NUMBER: | 89-L-060 | DEPTH: | 52.5 FT. |
| SAMPLE NUMBER: | SHC-22-52.5 | INITIAL WT: | 51.22 (g) |
| TYPE OF WATER USED: | DISTILLED | TOTAL SAMPLE WT: | 51.22 (g) |
| REACTION WITH H202: | NONE | START TIME: | 10:50:00 |
| DISPERSANT: | (NaPO3)6 | | |

| DATE (1989) | TIME (MIN) | TEMP ( C) | R (g/l) | Rl (g/l) | Rcorr (g/l) | L (CM) | D (MM) | P (%) | %FINER |
|---|---|---|---|---|---|---|---|---|---|
| 8/3 | 0.25 | 21.0 | 30.0 | 1.5 | 28.5 | 11.4 | 0.09129 | 55.64233 | 55.64233 |
| | 0.50 | 21.0 | 26.0 | 1.5 | 24.5 | 12.0 | 0.06639 | 47.83288 | 47.83288 |
| | 1.00 | 21.0 | 23.0 | 1.5 | 21.5 | 12.5 | 0.04789 | 41.97579 | 41.97579 |
| | 2.00 | 21.0 | 20.0 | 1.5 | 18.5 | 13.0 | 0.03452 | 36.11870 | 36.11870 |
| | 5.00 | 21.0 | 16.0 | 1.5 | 14.5 | 13.7 | 0.02238 | 28.30925 | 28.30925 |
| | 10.00 | 21.0 | 14.0 | 1.5 | 12.5 | 14.0 | 0.01601 | 24.40453 | 24.40453 |
| | 20.00 | 21.0 | 12.0 | 1.5 | 10.5 | 14.3 | 0.01145 | 20.49980 | 20.49980 |
| | 60.00 | 21.0 | 9.0 | 1.5 | 7.5 | 14.8 | 0.00673 | 14.64272 | 14.64272 |
| | 120.00 | 21.0 | 7.0 | 1.5 | 5.5 | 15.2 | 0.00481 | 10.73799 | 10.73799 |
| | 232.00 | 21.0 | 6.0 | 2.0 | 4.0 | 15.3 | 0.00348 | 7.80945 | 7.80945 |
| | 503.00 | 21.0 | 4.5 | 2.0 | 2.5 | 15.6 | 0.00238 | 4.88091 | 4.88091 |

LABORATORY ANALYSIS PERFORMED BY: L. Simpson
CALCULATIONS MADE BY: S. Stoller
CHECKED BY: E. Mattson

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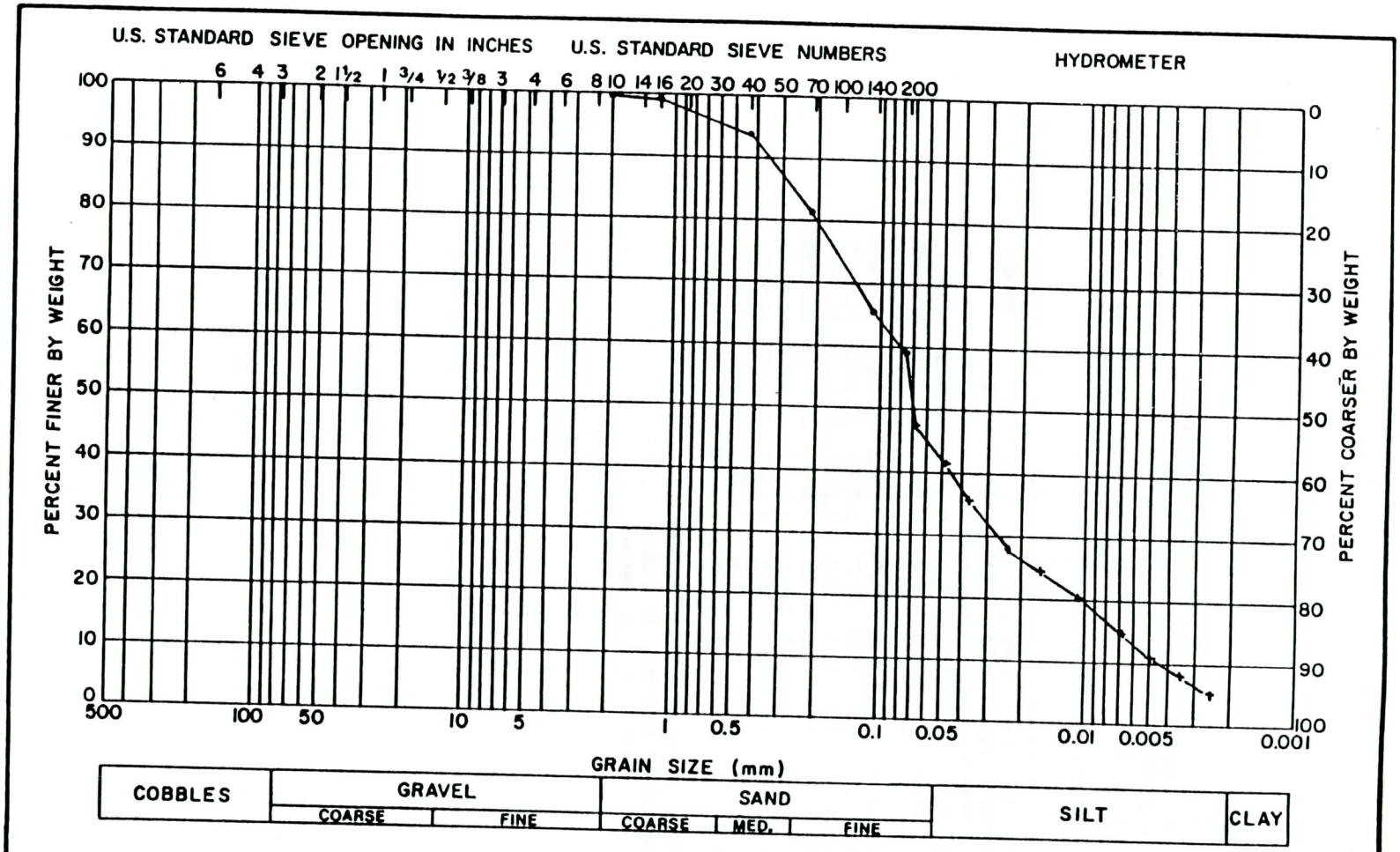


## USDA PARTICLE SIZE LIMIT CLASSIFICATION

| $\theta_i$ = 35.92 | $d_{10}$ = 0.0044 | $d_{50}$ = 0.07 | $d_{60}$ = 0.08 | $C_u$ = 18.18 | $C_c$ = 1.92 |
|---|---|---|---|---|---|
| SAMPLE NUMBER | | DEPTH | CLASSIFICATION (USDA TEXTURE TRIANGLE) | | |
| SHC-22-52.5 | | 52.5 ft. | Sandy loam | | |

HYDROMETER DATA

| | | | |
|---|---|---|---|
| JOB NAME: | EMCON | TEST DATE: | 8/3/89 |
| JOB NUMBER: | 89-L-060 | DEPTH: | 54 FT. |
| SAMPLE NUMBER: | SHC-22-54 | INITIAL WT: | 51.19 (g) |
| TYPE OF WATER USED: | DISTILLED | TOTAL SAMPLE WT: | 51.19 (g) |
| REACTION WITH H2O2: | VERY MILD | START TIME: | 10:39:00 |
| DISPERSANT: | (NaPO3)6 | | |

| DATE (1989) | TIME (MIN) | TEMP ( C) | R * (g/l) | Rl (g/l) | Rcorr (g/l) | L (CM) | D (MM) | P (%) | %FINER |
|---|---|---|---|---|---|---|---|---|---|
| 8/3 | 0.25 | 21.0 | 45.0 | 1.5 | 43.5 | 8.9 | 0.08083 | 84.97753 | 84.97753 |
| | 0.50 | 21.0 | 44.0 | 1.5 | 42.5 | 9.1 | 0.05768 | 83.02403 | 83.02403 |
| | 1.00 | 21.0 | 41.5 | 1.5 | 40.0 | 9.5 | 0.04170 | 78.14026 | 78.14026 |
| | 2.00 | 21.0 | 38.5 | 1.5 | 37.0 | 10.0 | 0.03024 | 72.27974 | 72.27974 |
| | 5.00 | 21.0 | 34.0 | 1.5 | 32.5 | 10.7 | 0.01982 | 63.48896 | 63.48896 |
| | 10.00 | 21.0 | 31.0 | 1.5 | 29.5 | 11.2 | 0.01433 | 57.62844 | 57.62844 |
| | 20.00 | 21.0 | 27.5 | 1.5 | 26.0 | 11.8 | 0.01039 | 50.79117 | 50.79117 |
| | 60.00 | 21.0 | 21.0 | 1.5 | 19.5 | 12.9 | 0.00626 | 38.09338 | 38.09338 |
| | 120.00 | 21.0 | 17.0 | 1.5 | 15.5 | 13.5 | 0.00454 | 30.27935 | 30.27935 |
| | 242.00 | 21.0 | 14.0 | 2.0 | 12.0 | 14.0 | 0.00325 | 23.44208 | 23.44208 |
| | 513.00 | 21.0 | 11.0 | 2.0 | 9.0 | 14.5 | 0.00227 | 17.58156 | 17.58156 |
| 8/4 | 1278.00 | 21.0 | 10.0 | 1.5 | 8.5 | 14.7 | 0.00145 | 16.60481 | 16.60481 |

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CALCULATIONS MADE BY: S. Stoller
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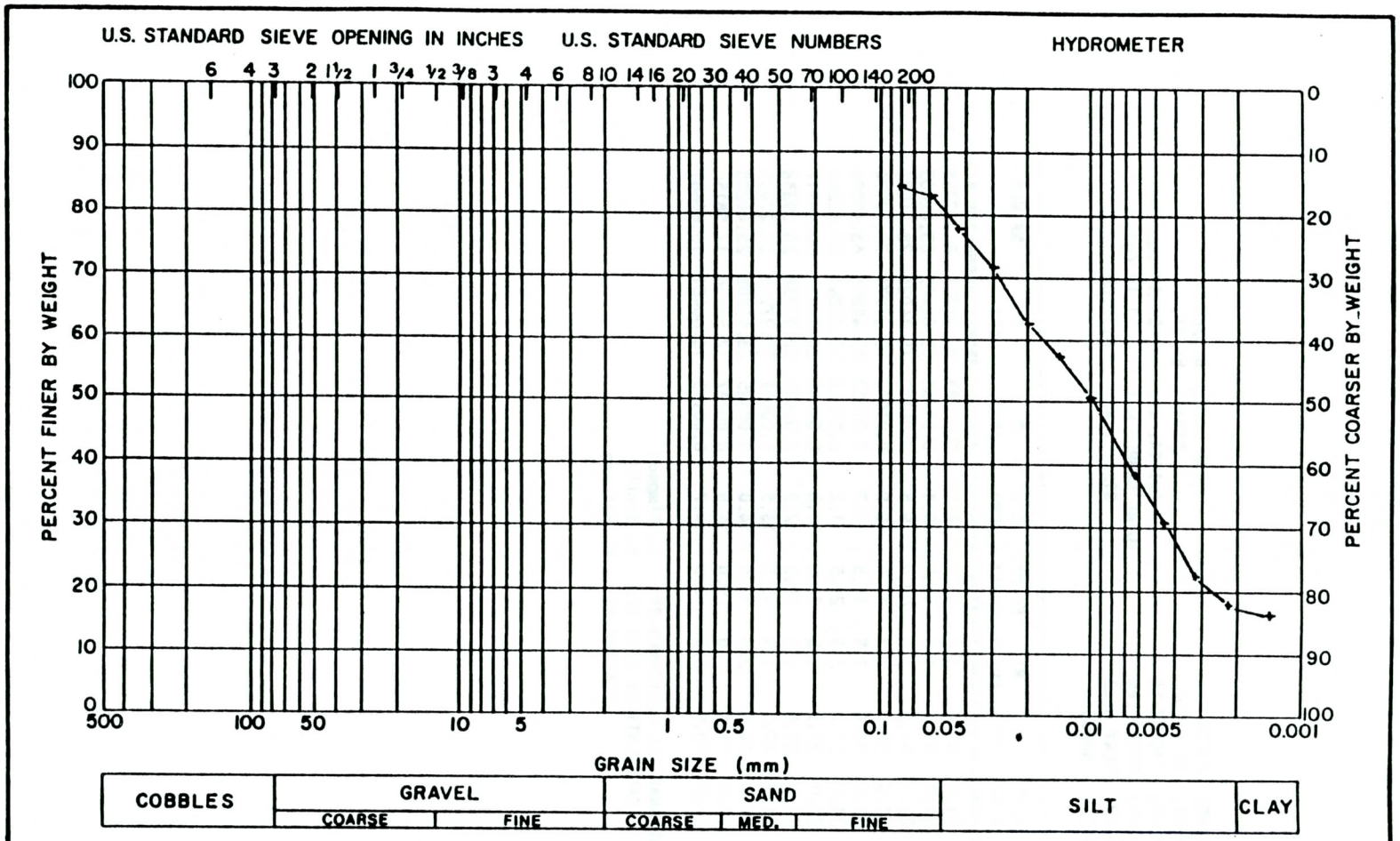


## USDA PARTICLE SIZE LIMIT CLASSIFICATION

| $\theta_i$ = 42.42 | $d_{10}$ = * | $d_{50}$ = 0.01 | $d_{60}$ = 0.017 | $C_u$ = -- | $C_c$ = -- |
|---|---|---|---|---|---|
| SAMPLE NUMBER | | DEPTH | CLASSIFICATION (USDA TEXTURE TRIANGLE) | | |
| SHC-22-54.0 | | 54 ft. | Silt loam | | |

HYDROMETER DATA

| | | | |
|---|---|---|---|
| JOB NAME: | EMCON | TEST DATE: | 8/3/89 |
| JOB NUMBER: | 89-L-060 | DEPTH: | 59 FT. |
| SAMPLE NUMBER: | SHC-22-59 | INITIAL WT: | 49.99 (g) |
| TYPE OF WATER USED: | DISTILLED | TOTAL SAMPLE WT: | 49.99 (g) |
| REACTION WITH H2O2: | NONE | START TIME: | 10:11:00 |
| DISPERSANT: | (NaPO3)6 | | |

| DATE (1989) | TIME (MIN) | TEMP ( C) | R (g/l) | Rl (g/l) | Rcorr (g/l) | L (CM) | D (MM) | P (%) | %FINER |
|---|---|---|---|---|---|---|---|---|---|
| 8/3 | 0.25 | 21.0 | 41.0 | 1.5 | 39.5 | 9.6 | 0.08375 | 79.01580 | 79.01580 |
| | 0.50 | 21.0 | 39.0 | 1.5 | 37.5 | 9.9 | 0.06023 | 75.01500 | 75.01500 |
| | 1.00 | 21.0 | 35.0 | 1.5 | 33.5 | 10.6 | 0.04397 | 67.01340 | 67.01340 |
| | 2.00 | 21.0 | 29.5 | 1.5 | 28.0 | 11.5 | 0.03239 | 56.01120 | 56.01120 |
| | 5.00 | 21.0 | 22.0 | 1.5 | 20.5 | 12.7 | 0.02156 | 41.00820 | 41.00820 |
| | 10.00 | 21.0 | 20.0 | 1.5 | 18.5 | 13.0 | 0.01544 | 37.00740 | 37.00740 |
| | 20.00 | 21.0 | 15.5 | 1.5 | 14.0 | 13.8 | 0.01122 | 28.00560 | 28.00560 |
| | 60.00 | 21.0 | 12.5 | 1.5 | 11.0 | 14.3 | 0.00659 | 22.00440 | 22.00440 |
| | 120.00 | 21.0 | 11.0 | 1.5 | 9.5 | 14.5 | 0.00470 | 19.00380 | 19.00380 |
| | 265.00 | 21.0 | 9.0 | 2.0 | 7.0 | 14.8 | 0.00320 | 14.00280 | 14.00280 |
| | 538.00 | 21.0 | 7.0 | 2.0 | 5.0 | 15.2 | 0.00227 | 10.00200 | 10.00200 |

LABORATORY ANALYSIS PERFORMED BY: L. Simpson
CALCULATIONS MADE BY: S. Stoller
CHECKED BY: E. Mattson

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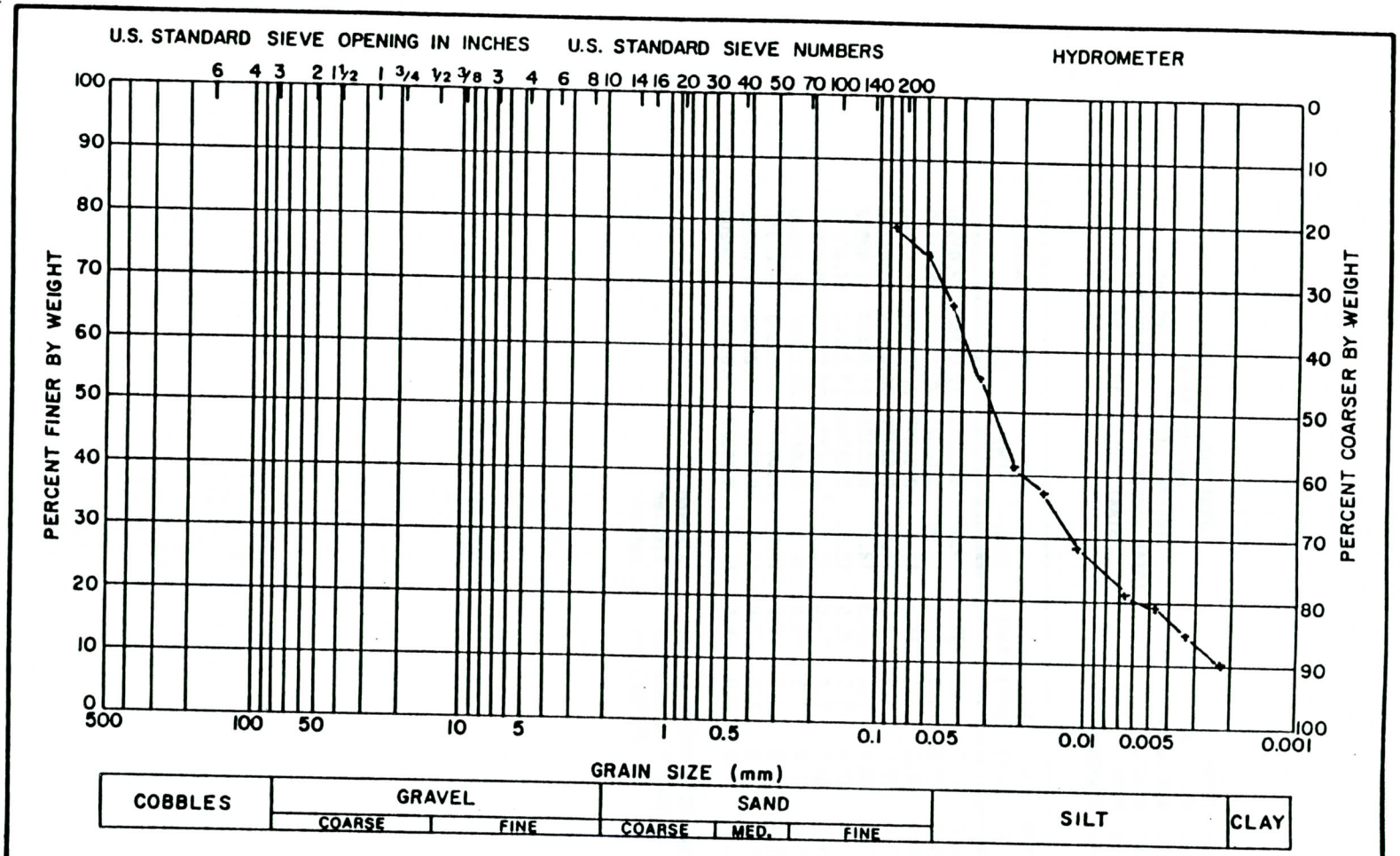


## USDA PARTICLE SIZE LIMIT CLASSIFICATION

| $\theta_i$ = 42.08 | $d_{10}$ = 0.0022 | $d_{50}$ = 0.029 | $d_{60}$ = 0.037 | $C_u$ = 16.82 | $C_c$ = 1.77 |
|---|---|---|---|---|---|
| SAMPLE NUMBER | | DEPTH | CLASSIFICATION (USDA TEXTURE TRIANGLE) | | |
| SHC-22-59.0 | | 59 ft. | Silt loam | | |

HYDROMETER DATA

| | | | |
|---|---|---|---|
| JOB NAME: | EMCON | TEST DATE: | 8/3/89 |
| JOB NUMBER: | 89-L-060 | DEPTH: | 68 FT. |
| SAMPLE NUMBER: | SHC-22-68 | INITIAL WT: | 50.39 (g) |
| TYPE OF WATER USED: | DISTILLED | TOTAL SAMPLE WT: | 50.39 (g) |
| REACTION WITH H202: | VERY MILD | START TIME: | 10:24:00 |
| DISPERSANT: | (NaPO3)6 | | |

| DATE (1989) | TIME (MIN) | TEMP ( C) | R (g/l) | Rl (g/l) | Rcorr (g/l) | L (CM) | D (MM) | P (%) | %FINER |
|---|---|---|---|---|---|---|---|---|---|
| 8/3 | 0.25 | 21.0 | 42.0 | 1.5 | 40.5 | 9.4 | 0.08303 | 80.37309 | 80.37309 |
| | 0.50 | 21.0 | 39.0 | 1.5 | 37.5 | 9.9 | 0.06023 | 74.41953 | 74.41953 |
| | 1.00 | 21.0 | 34.0 | 1.5 | 32.5 | 10.7 | 0.04431 | 64.49692 | 64.49692 |
| | 2.00 | 21.0 | 29.0 | 1.5 | 27.5 | 11.5 | 0.03251 | 54.57432 | 54.57432 |
| | 5.00 | 21.0 | 21.5 | 1.5 | 20.0 | 12.8 | 0.02163 | 39.69041 | 39.69041 |
| | 10.00 | 21.0 | 16.5 | 1.5 | 15.0 | 13.6 | 0.01578 | 29.76781 | 29.76781 |
| | 20.00 | 21.0 | 14.5 | 1.5 | 13.0 | 13.9 | 0.01129 | 25.79877 | 25.79877 |
| | 60.00 | 21.0 | 12.0 | 1.5 | 10.5 | 14.3 | 0.00661 | 20.83747 | 20.83747 |
| | 120.50 | 21.0 | 10.0 | 1.5 | 8.5 | 14.7 | 0.00472 | 16.86843 | 16.86843 |
| | 256.00 | 21.0 | 8.0 | 2.0 | 6.0 | 15.0 | 0.00327 | 11.90712 | 11.90712 |
| | 527.00 | 21.0 | 6.5 | 2.0 | 4.5 | 15.2 | 0.00230 | 8.93034 | 8.93034 |

LABORATORY ANALYSIS PERFORMED BY: L. Simpson
CALCULATIONS MADE BY: S. Stoller
CHECKED BY: E. Mattson

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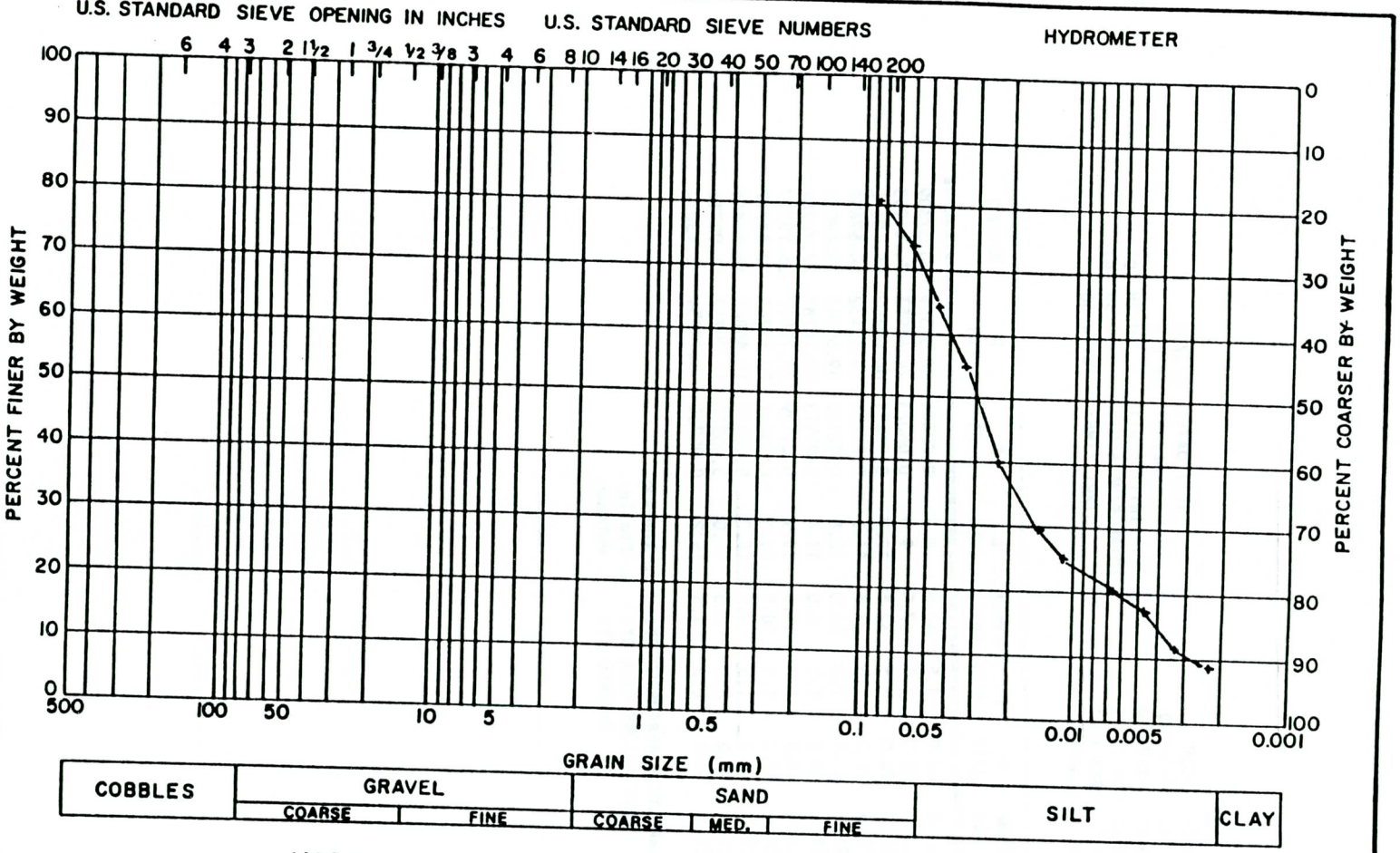


## USDA PARTICLE SIZE LIMIT CLASSIFICATION

| $\theta_i$ = 42.63 | $d_{10}$ = 0.003 | $d_{50}$ = 0.03 | $d_{60}$ = 0.04 | $C_u$ = 13.33 | $C_c$ = 1.88 |
|---|---|---|---|---|---|
| SAMPLE NUMBER | | DEPTH | CLASSIFICATION (USDA TEXTURE TRIANGLE) | | |
| SHC-22-68.0 | | 68 ft. | Silt loam | | |

HYDROMETER DATA

| | | | |
|---|---|---|---|
| JOB NAME: | EMCON | TEST DATE: | 8/3/89 |
| JOB NUMBER: | 89-L-060 | DEPTH: | 77.5 FT. |
| SAMPLE NUMBER: | SHC-22-77.5 | INITIAL WT: | 49.94 (g) |
| TYPE OF WATER USED: | DISTILLED | TOTAL SAMPLE WT: | 49.94 (g) |
| REACTION WITH H202: | VERY MILD | START TIME: | 10:56:00 |
| DISPERSANT: | (NaPO3)6 | | |

| DATE (1989) | TIME (MIN) | TEMP ( C) | R (g/l) | Rl (g/l) | Rcorr (g/l) | L (CM) | D (MM) | P (%) | %FINER |
|---|---|---|---|---|---|---|---|---|---|
| 8/3 | 0.25 | 21.0 | 44.0 | 1.5 | 42.5 | 9.1 | 0.08157 | 85.10212 | 85.10212 |
| | 0.50 | 21.0 | 42.0 | 1.5 | 40.5 | 9.4 | 0.05871 | 81.09732 | 81.09732 |
| | 1.00 | 21.0 | 40.5 | 1.5 | 39.0 | 9.7 | 0.04205 | 78.09371 | 78.09371 |
| | 2.00 | 21.0 | 35.0 | 1.5 | 33.5 | 10.6 | 0.03109 | 67.08050 | 67.08050 |
| | 5.00 | 21.0 | 29.0 | 1.5 | 27.5 | 11.5 | 0.02056 | 55.06608 | 55.06608 |
| | 10.00 | 21.0 | 23.5 | 1.5 | 22.0 | 12.4 | 0.01510 | 44.05286 | 44.05286 |
| | 20.00 | 21.0 | 20.0 | 1.5 | 18.5 | 13.0 | 0.01092 | 37.04445 | 37.04445 |
| | 60.00 | 21.0 | 16.0 | 1.5 | 14.5 | 13.7 | 0.00646 | 29.03484 | 29.03484 |
| | 120.00 | 21.0 | 13.0 | 1.5 | 11.5 | 14.2 | 0.00465 | 23.02763 | 23.02763 |
| | 227.00 | 21.0 | 11.5 | 2.0 | 9.5 | 14.4 | 0.00341 | 19.02283 | 19.02283 |
| | 498.00 | 21.0 | 10.0 | 2.0 | 8.0 | 14.7 | 0.00232 | 16.01922 | 16.01922 |
| 8/4 | 1258.00 | 21.0 | 8.0 | 1.5 | 6.5 | 15.0 | 0.00148 | 13.01562 | 13.01562 |

LABORATORY ANALYSIS PERFORMED BY: L. Simpson
CALCULATIONS MADE BY: S. Stoller
CHECKED BY: E. Mattson

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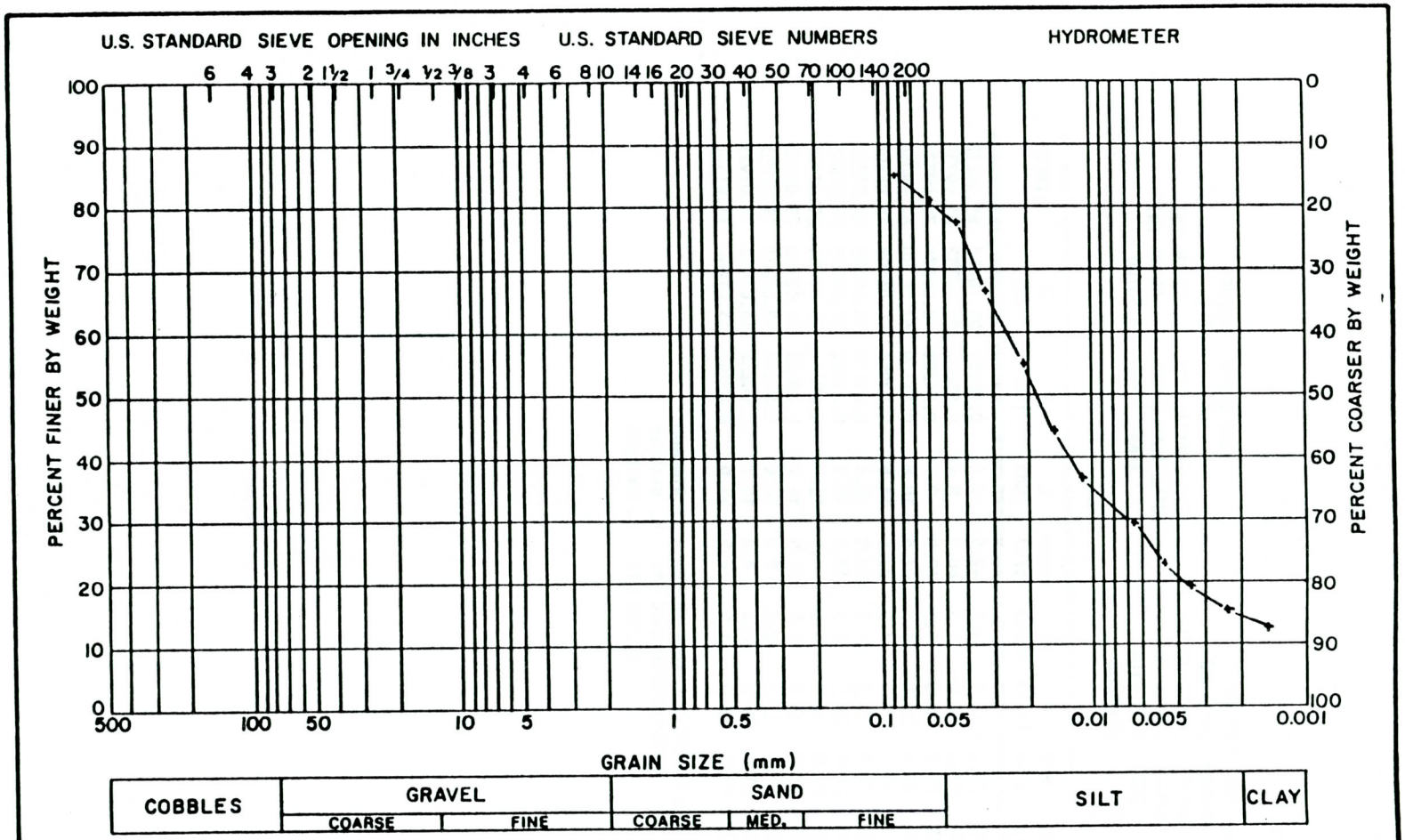


## USDA PARTICLE SIZE LIMIT CLASSIFICATION

| $\theta_i$ = 47.26 | $d_{10}$ = * | $d_{50}$ = 0.018 | $d_{60}$ = 0.024 | $C_u$ = -- | $C_c$ = -- |
|---|---|---|---|---|---|
| SAMPLE NUMBER | | DEPTH | CLASSIFICATION (USDA TEXTURE TRIANGLE) | | |
| SHC-22-77.5 | | 77.5 ft. | Silt loam | | |

# Appendix E: LABORATORY METHODS

# INITIAL MOISTURE CONTENT

## (Microwave Oven Drying Method)

### Methods

Methods and procedures outlined under ASTM standard D4643-87 are followed to determine the moisture content of a soil by the microwave oven method. This test method is not intended as a replacement for ASTM D2216-80, but rather as a supplement when more rapid results are required or desired to expedite other phases of testing. This test method is applicable for most soil types, however for soils containing significant amounts of halloysite, mica, montmorillonite, gypsum, or other hydrated materials; highly organic soils; or soils in which the pore water contains dissolved solids; this test method may not yield reliable moisture content values.

### Laboratory Procedures

To prepare disturbed samples, a sample is selected from the material after it has been thoroughly mixed. The mass of the selected sample follows the guidelines in Table 1.

To prepare core samples, different procedures for cohesionless and cohesive soils must be followed. For cohesionless soils, the material is mixed thoroughly and a sample with a mass in accordance with Table 1 is selected. For cohesive soils, about 3mm of material is removed from the exposed ends,

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and the remaining sample is sliced lengthwise to check if the sample is layered. If the sample is layered, then an average portion, is selected.

TABLE 1. Test Specimen Masses

| Sieve Retaining Not More Than About 10% of Sample | Recommended Mass of Moist Specimen (g) |
|---|---|
| 2.00 mm (No. 10) | 100 to 200 |
| 4.75 mm (No. 4) | 300 to 500 |
| 19.00 mm (3/4 in.) | 500 to 1000 |

The mass of a clean, dry dish is determined and recorded. The soil sample is placed on the dish and its mass is immediately determined and recorded. The soil and dish are cooked in the microwave oven for 3 minutes. The soil and dish are removed and allowed to cool. The mass is determined and recorded. The soil is carefully mixed with a spatula and returned to the oven and reheated for 1 minute. This procedure of cooling, mixing, and reheating is repeated until the change in mass is 0.1% or less of the initial mass. The final mass is then recorded.

Calculations

The initial moisture content on a percent volume basis is calculated as follows:[2]

$$\theta_i = \frac{(M_i - M_f)}{(V_T \times \rho)} * 100$$

where

$\theta_i$ = initial moisture content (% volume)

$M_i$ = initial mass of soil & water (g)

$M_f$ = final mass of soil (g)

$V_T$ = total volume of sample (cc)

$\rho$ = density of pores fluid in the soil when initial mass was determined (g/cc). The density of the pore fluid initially present in the sample is assumed to be 1.0 g/cc

The initial moisture content determined on a percent weight basis is according to:

$$w = \frac{(M_i - M_f)}{M_f} * 100$$

where

$w$ = initial moisture content (%)

$M_i$ = initial mass of soil (g)

$M_f$ = final mass of soil (g)

---

[2] Gardner, Walter H. 1986. Water Content. Methods of Soil Analysis, Part 1, ed. A. Klute. American Society of Agronomy, Madison Wis., pp 493-545.

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# INITIAL MOISTURE CONTENT

## (Oven Drying Method)

### Method

Methods and procedures outlined under ASTM standard D2216-80 are followed to determine the moisture content of a soil by the oven drying method. The oven drying method does not give true representative results for materials containing significant amounts of halloysite, montmorillonite, or gypsum minerals; highly organic soils; or materials in which the pore water contains dissolved solids.

### Laboratory Procedure

To prepare disturbed samples, a sample is selected from the material after it has been thoroughly mixed. The mass of the selected sample follows the guidelines in Table 1.

To prepare core samples, different procedures for cohesionless and cohesive soils must be followed. For cohesionless soils, the material is mixed thoroughly and a sample with a mass in accordance with Table 1 is selected. For cohesive soils, about 3mm of material is removed from the exposed ends, and the remaining sample is sliced lengthwise to check if the sample is layered. If the sample is layered, then an average portion, is selected.

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TABLE 1. Test Specimen Masses

| Sieve Retaining Not More Than About 10% of Sample | Recommended Mass of Moist Specimen (g) |
|---|---|
| 2.00 mm (No. 10) | 100 to 200 |
| 4.75 mm (No. 4) | 300 to 500 |
| 19.00 mm (3/4 in.) | 500 to 1000 |

The moist sample is placed in a dry container of known mass. The masses of the sample and of the container are determined and recorded. The sample and the container are placed in a drying oven maintained at 110° ± 5° C and dried to a constant mass. The time required to obtain a constant mass will vary depending on the type of material, the size of the specimen, and the oven type and capacity. Weights are recorded on a daily basis, but, in most cases, drying a test specimen over night (about 24 hours) is sufficient.

## Calculations

The initial moisture content on a percent volume basis is calculated as follows: [1]

$$\theta_i = \frac{(M_i - M_f)}{(V_T \times q)} * 100$$

---

[1] Gardner, Walter H. 1986. Water Content. Methods of Soil Analysis, Part 1, ed. A. Klute. American Society of Agronomy, Madison Wis., pp 493-545.

where

$\theta_i$ = initial moisture content (% volume)

$M_i$ = initial mass of soil & water (g)

$M_f$ = final mass of soil (g)

$V_T$ = total volume of sample (cc)

$\rho$ = density of pores fluid in the soil when initial mass was determined (g/cc). The density of the pore fluid initially present in the sample is assumed to be 1.0 g/cc

The initial moisture content determined on a percent weight basis is according to:

$$w = \frac{(M_i - M_f)}{M_f} * 100$$

where

$w$ = initial moisture content (%)

$M_i$ = initial mass of soil only (g)

$M_f$ = final mass of soil only (g)

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# BULK DENSITY

## Method

Bulk density is calculated from the initial soil sample volume and oven dried mass of the soil sample.

## Laboratory Procedures

The volume of the soil sample is calculated from geometric measurements of the sample. The sample mass is determined from methods outlined in ASTM D2216-80 (oven drying) or ASTM D4643-87 (microwave oven drying).

## Calculations

The bulk density is calculated as follows:

$$\rho_b = M_D/V_T$$

where

$\rho_b$ = dry bulk density (g/cc)

$M_D$ = mass of oven dried soil sample (g)

$V_T$ = total volume of soil sample (cc)

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# POROSITY

## (Particle Density Method)

### Method

Porosity can be calculated from dry bulk density and particle density. The particle density method is based on sample geometry and mass relationships.

### Laboratory Procedures

Bulk density is calculated by the sample geometry and sample mass determined by oven drying, as described in the section outlining the bulk density determination. Particle density is determined from measurements following the procedures outlined in the particle density principles and methods.

### Calculations

Porosity is calculated as follows:

$$n = [1 - (\rho_b/\rho_s)] \times 100$$

where

$n$ = porosity (%)

$\rho_b$ = bulk density (g/cc)

$\rho_s$ = particle density (g/cc)

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# PARTICLE SIZE DISTRIBUTION

## (Sieve Analysis)

### Method

Methods and procedures outlined under ASTM standard D422-63(72) are followed to determine the particle size distribution of particles larger than 75 $\mu$m using the mechanical sieve technique. Distribution of particles smaller than 75 $\mu$m are determined using the hydrometer sedimentation analysis.

### Laboratory Procedure

A soil sample is separated into a series of fractions from 4.75 mm (No. 4) to 0.075 mm (No. 200) by mechanical sieve procedures. The sieve operates by means of lateral and vertical jarring motions shaking the soil sample through a series of finer sieves. Mechanical sieving is considered complete when less than 1% of the mass fraction passes a sieve during a one minute hand sieving test.

### Calculations

A plot of the particle size distribution curve is developed from the mass retained on each sieve and data from the hydrometer analysis. This plot is used to estimate the $d_{10}$, $d_{16}$, $d_{30}$, $d_{50}$, $d_{60}$, and $d_{84}$ diameters ($d_x$ is the diameter of a particle of which

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x percent of the sample mass is finer). These soil particles diameters are used to calculate the uniformity coefficient, Cu:

$$Cu = d_{60}/d_{10}$$

the coefficient of curvature, $C_c$:

$$C_c = (d_{30})^2/(d_{10} * d_{60})$$

and the mean particle diameter, $\bar{d}$:

$$\bar{d} = (d_{16} + d_{50} + d_{84})/3$$

# PARTICLE SIZE DISTRIBUTION

## (Hydrometer Analysis)

### Method

Methods and procedures outlined under ASTM standard D422-63(72) are followed to determine the particle size distribution of particles smaller than 75 $\mu$m using the hydrometer sedimentation analysis. Distribution of particles larger than 75 $\mu$m are determined using the sieve analysis.

### Laboratory Procedures

A soil sample of approximately 50 grams for silts and clays, or 100 grams for sands, is soaked for a minimum of 16 hours in a solution of sodium hexametaphospate. At the end of the soaking period, the sample is dispersed further in a mechanically operated stirring device for one minute. Immediately after dispersion, the soil-water slurry is transferred to a glass sedimentation cylinder, and distilled water is added until the total volume is 1000 ml. The glass cylinder is turned upside down and back for one minute to complete agitation. Hydrometer readings are taken at ASTM recommended times for a period of 24 hours.

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Calculations

The percentage of soil remaining in solution at the level at which the hydrometer measures, P, is calculated as follows:

$$P = (Ra/W) \times 100$$

where

P = percentage of soil remaining in suspension at the level at which the hydrometer measures the density of the suspension

R = hydrometer reading with composite correction applied

a = correction factor applied to the readings of the 1524 hydrometer

W = oven-dry mass of the soil sample

The diameter of a particle corresponding to the above calculated percentage is calculated as follows:

$$D = K \sqrt{L/T}$$

where

D = diameter of the particle (mm)

K = a constant depending on temperature and the specific gravity of the soil particles

L = distance from the suspension surface to the level where the suspension is being measured (cm)

T = time since the beginning of sedimentation (min)

A plot of the particle size distribution curve is developed from the above data using the sieve analysis. This plot is used to estimate the $d_{10}$, $d_{16}$, $d_{30}$, $d_{50}$, $d_{60}$, and $d_{84}$ diameters ($d_x$ is the

diameter of a particle of which x percent of the sample mass is finer). These soil particles diameters are used to calculate the uniformity coefficient, Cu:

$$Cu = d_{60}/d_{10}$$

the coefficient of curvature, $C_c$:

$$C_c = (d_{30})^2/(d_{10} \times d_{60})$$

and the mean particle diameter, $\bar{d}$:

$$\bar{d} = (d_{16} + d_{50} + d_{84})/3$$

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# SATURATED HYDRAULIC CONDUCTIVITY

## (Falling Head Method)

### Method

Methods and procedures outlined under Methods of Soil Analysis (1986) are followed to determine the saturated hydraulic conductivity by the falling head method. Saturated hydraulic conductivity determined by the falling head method is based on a simple boundary valve problem that describes one-dimensional transient flow across a soil sample. The falling head method is best suited for soil with a saturated hydraulic conductivity in the range of $10^{-3}$ to $10^{-7}$ cm/sec.

### Laboratory Procedures

A soil sample, of length L and cross-sectional area A, is placed in a sample holder, which prevents soil loss or volume change. The soil sample is saturated using vacuum flooding techniques. After saturation, a stand pipe is connected, and the rate of water drop in the stand pipe is recorded. Figure 1 shows the falling head apparatus used.

The temperature of the water is measured with a thermometer. Corrections to 20°C of the kinematic viscosity of the fluid are applied to the reported saturated hydraulic conductivity.

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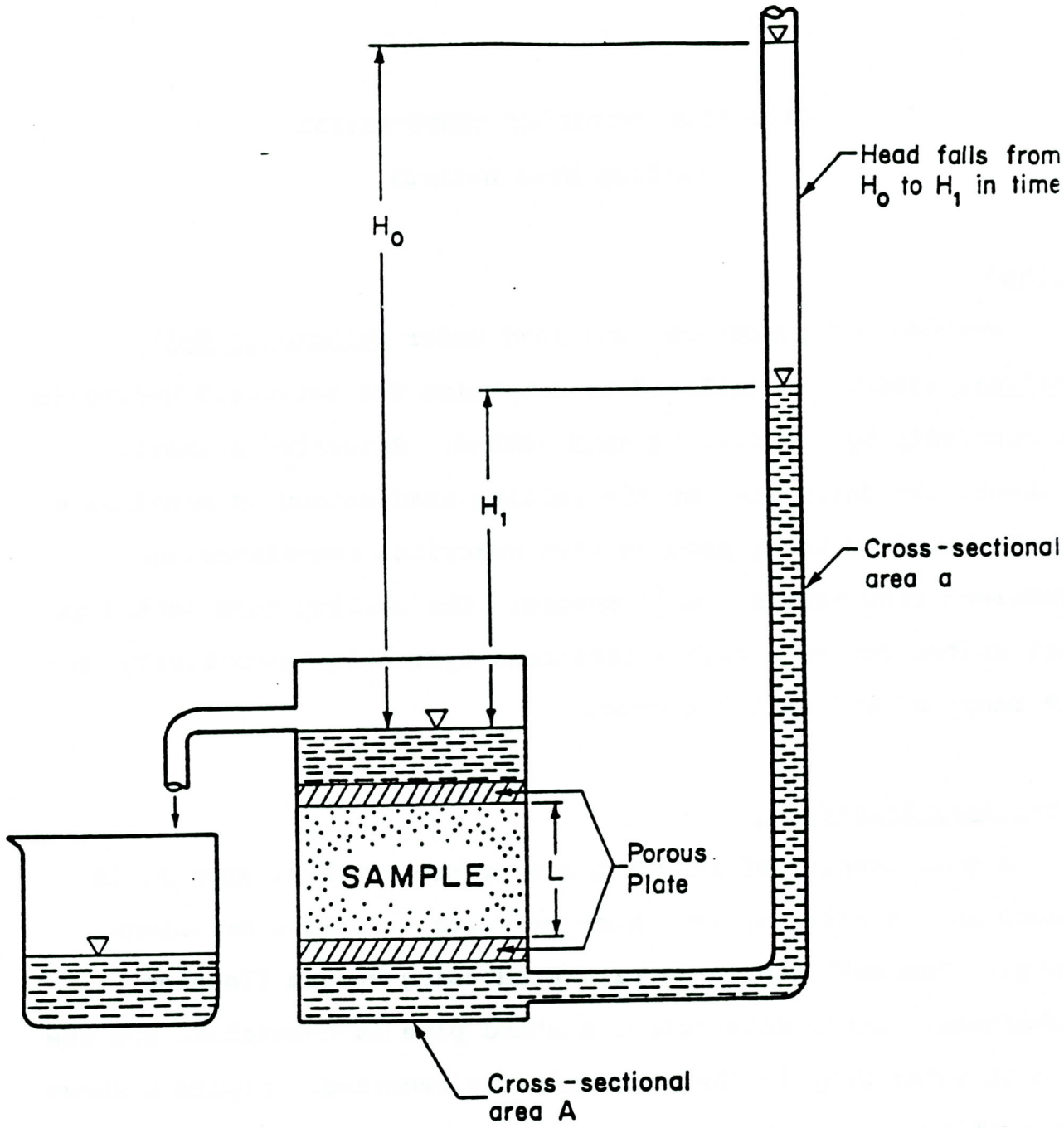


Figure 1. Falling Head Apparatus

## Calculations

The head measured in the stand pipe, of cross-sectional area a, is allowed to fall from $H_0$ to $H_1$ during time t. The saturated hydraulic conductivity is calculated as follows:

$$K_s = [aL/At] \ln[H_0/H_1][\nu_T/\nu_{20}]$$

where

$K_s$ = saturated hydraulic conductivity at 20°C (cm/sec)

a = cross-sectional area of the stand pipe ($cm^2$)

L = length of the soil sample (cm)

A = cross-sectional area of the soil sample ($cm^2$)

t = time for head to fall from $H_0$ to $H_1$ (sec)

$H_0$ = head at experiment start (cm)

$H_1$ = head at experiment end (cm)

$\nu_T$ = kinematic viscosity of water at the measured temperature ($m^2$/sec)

$\nu_{20}$ = kinematic viscosity of water at 20°C ($m^2$/sec)

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# SATURATED HYDRAULIC CONDUCTIVITY

## (Constant Head Method)

### Method

Methods and procedures outlined under ASTM standard D2434-68 are followed to determine the saturated hydraulic conductivity by the constant head method. Measurements of hydraulic conductivity of saturated soils are based on the direct application of Darcy's equation. The constant heads permeameter is best suited for materials with a saturated hydraulic conductivity in the range of 1 to $10^{-5}$ cm/sec.

### Laboratory Procedures

A soil sample, of length L and cross-sectional area A, is placed in a sample holder, which prevents soil loss or volume change. The soil sample is saturated using vacuum flooding techniques. After saturation, a constant head differential is maintained across the sample. Periodic readings of the volumetric outflow are taken until stable values for saturated volumetric conductivity, $K_s$, are obtained. Figure 1 shows the constant head apparatus used.

The temperature of the water is measured with a thermometer. Correction to 20°C of the kinematic viscosity of the fluid are applied to the reported saturated hydraulic conductivity.

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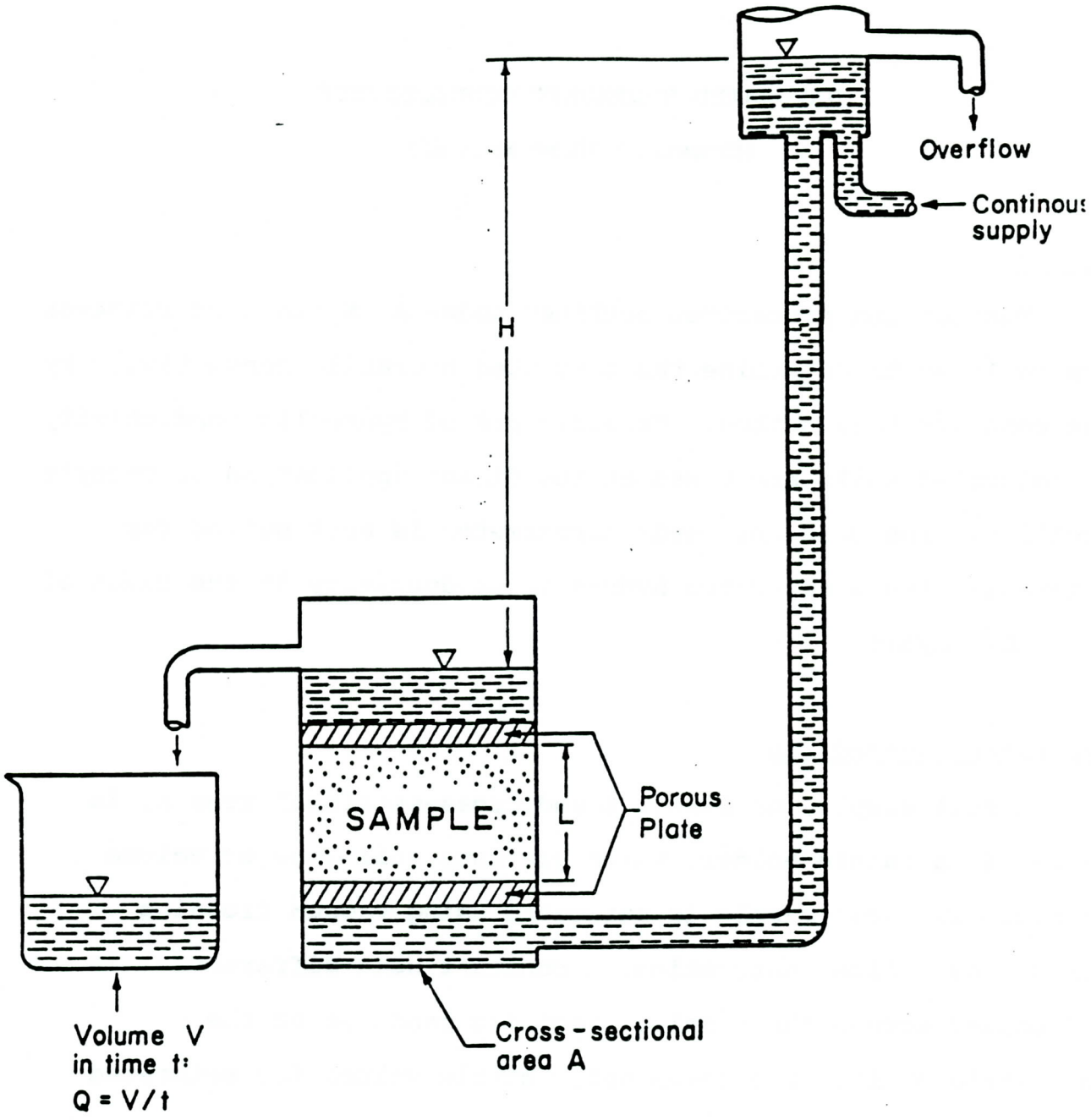


Figure 1. Constant Head Apparatus

## Calculations

Darcy's equation is used to calculate the saturated hydraulic conductivity as follows:

$$K_s = [Q/A][\Delta L/\Delta H][\nu_T/\nu_{20}]$$

where

$K_s$ = saturated hydraulic conductivity @ 20°C (cm/sec)

$Q$ = volumetric outflow from soil sample ($cm^3$/sec)

$A$ = cross-sectional area of the soil sample ($cm^2$)

$\Delta L$ = length of the soil sample (cm)

$\Delta H$ = head differential across the soil sample (cm)

$\nu_T$ = kinematic viscosity of water at the measured temperature ($m^2$/sec)

$\nu_{20}$ = kinematic viscosity of water at 20°C ($m^2$/sec)

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# MOISTURE RETENTION CHARACTERISTIC

## (Hanging Column Method)

### Method

The key component of the apparatus for measuring the retention of moisture at different pressure heads or pore size distribution is a fritted glass porous plate that conducts water, but, when wet, the plate is impermeable to air. Fritted glass plates have an air-entry pressure of about 300 to 400 cm of water. These plates are affixed to a glass funnel which is connected to a buret with stopcock by means of flexible tubing. A diagram of the apparatus is shown in Figure 1. A soil sample is placed on the plate and tension (h') is applied to the sample by positioning the fluid in the buret at different levels below the center of the sample. Water flows out of the sample into the buret until equilibrium is achieved. The tension is again increased or decreased to obtain another state of equilibrium between moisture held by capillary forces in the sample and the applied tension.

### Laboratory Procedures

Air is first removed from the porous plate by allowing de-aired water to pass continuously through it for 24 hours. The funnel with the porous plate, and the buret are supported on vertical rods by means of clamps. A saturated sample within its

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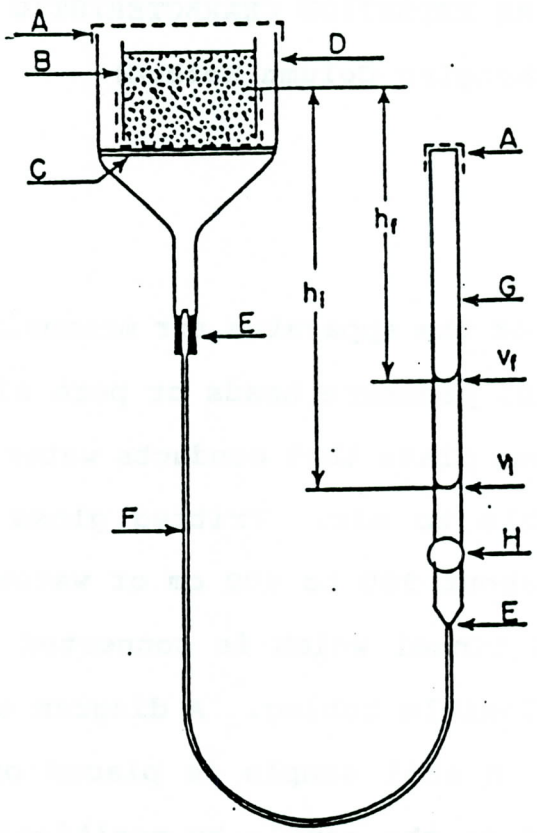


A Aluminum foil covers
B Sample in cylinder
C Fritted glass porous plate (part of D)
D Büchner funnel with porous plate
E Joints must be secure
F Flexible. tubing
G Burette, least division not more than 0.1% sample volume
H Stopcock of burette
$h_i$ cm of water suction, initial
$h_f$ cm of water suction, final
$v_i$ Burette reading, initial
$v_f$ Burette reading, final

Figure 1. Hanging Column Apparatus

sample ring is then placed on the porous plate, assuring that good hydraulic contact is established between the soil particles and the plate. With the stopcock of the buret closed, the initial level of the water in the buret is recorded.

The buret is then lowered a small increment to about 10 to 15 cm below the center of the soil sample. When the stopcock is opened, the soil may begin to desaturate, and the drainage will flow into the buret. When drainage has ceased, the stopcock is closed and the water level in the buret is recorded along with the vertical distance from the bottom of the meniscus of the water in the buret to the middle of the soil sample. The procedure is repeated in a stepwise manner until the maximum tension desired is reached. A reversal of the process is used to gather data on the wetting behavior of the sample.

## Calculation

Saturated moisture content (volume percent) is calculated as follows:

$$\theta_{sat} = [M_{sat} - M_{dry}]/[V_T \times \rho_w] \times 100 \qquad (1)$$

where

$\theta_{sat}$ = saturated volumetric water content (%cc/cc)

$M_{sat}$ = mass of sample saturated (g)

$M_{dry}$ = mass of sample, oven dried to a constant weight (g)

$V_T$ = volume of the sample (cc)

$\rho_w$ = density of the water at temperature when saturated mass was determined (g/cc)

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The quantity $[M_{sat} - M_{dry}]/\rho_w$ is the volume, in cubic centimeters, of water initially contained in the sample volume. The drainage is subtracted from the initial volume of water and then divided by the sample volume to arrive at the moisture content in percent volume at the given value of tension.

$$\theta_{h'} = [V_i - V_D]/V_T \times 100 \qquad (2)$$

where

$V_i$ = initial volume of water (cc)

$V_D$ = cumulative volume of water drained from sample (cc)

$V_T$ = volume of sample (cc)

$\theta_{h'}$ = moisture content at the tension value h' (%cc/cc)

This gives then a paired set of values of tension, or pressure head, versus volumetric moisture content.

## MOISTURE RETENTION CHARACTERISTICS

## (Pressure Plate Method)

### Method

Methods and procedures outlined under ASTM standard D2325-68 (81) are followed to determine the moisture retention characteristics in the 1 to 15 bar suction range. Moisture retention characteristics are obtained using a pressure plate extractor (Soil Moisture Inc., Santa Barbara, CA, Model 1500), with a 1, 3, or 15 bar ceramic plate. Pressure is provided by high pressure nitrogen from cylinders.

### Laboratory Procedure

The porous ceramic plate is placed in a shallow pan with deaired distilled water and allowed to stand overnight. The plate is then removed from the pan and placed in the extractor. De-aired distilled water is poured over the plate to the limit allowed by the rubber skirt, which generally just submerges the plate. The pressure plate is sealed and pressure brought to 50% of the plate's maximum rated pressure. This pressure is maintained until outflow ceases. The extractor is opened and any excess water around the plate is removed.

The soil samples in their sample rings are then placed on the plate, assuring that good hydraulic contact is established. The extractor is then sealed and the pressure brought to the level

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desired. The pressure is maintained until outflow ceases. The extractor is then opened and the samples weighed quickly on an electronic top-loading balance. Subsequently, the samples are returned to the extractor, and the pressure is increased to the next increment.

Calculations

The decrease in mass of the water in the sample during a period of applied pressure is converted to an equivalent decrease in volume of water according to:

$$V_w = m_w/\rho_w \quad (1)$$

where

$V_w$ = equivalent volume of water (cc)

$m_w$ = mass of water loss (g)

$\rho_w$ = density of water at temperature of experiment (g/cc)

Volumes of water calculated from equation 1 are then used to calculate the moisture content at that pressure as follows:

$$\theta_p = (V_i - \Sigma V_w)/V_T \times 100 \quad (2)$$

where

$\theta_p$ = moisture content at pressure p (% vol)

$V_i$ = initial volume of water in the sample (cc)

$\Sigma V_w$ = cumulative water volume change (cc)

$V_T$ = total volume of the sample (cc)

Attachment 9

UNSATURATED GROUND-WATER FLOW MODELING FOR
PROPOSED CHOLLA LANDFILL - PART 3

# Unsaturated Ground-water Flow Modeling for Proposed Cholla Landfill - Part 3


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## 1.0 Introduction

The initial report, *Unsaturated Ground-water Flow Modeling for the Proposed Cholla Landfill* (Maddock et al, 1989a) is hereafter referred to as Part 1. In Part 1, the unsaturated flow model, UNSAT2, was applied to three representative cross-sections of the proposed Cholla Sanitary Landfill to determine the distributions of pressure head, total head and moisture content; and the cumulative infiltration in response to an extreme hydrologic event in the Agua Fria River. The distributions indicated the potential for lateral migration of the ground water. The three cross-sections, referred to as Section 7, Section 8 and Section 9 in Part 1, were representative of geologic and lithologic conditions at the site. The extreme hydrologic event was modeled as a stage hydrograph (Maddock et al., 1989a, Figure 6) whose maximum stage reached 10 feet (the approximate 100 year flood boundary) for 1 hour. The duration of the hydrograph used for Part 1 was 8 days (Figure 6, Maddock et al, 1989a).

The cross-sections were oriented in an east-west direction (Figure 1), with Section 9 in the north, Section 8 in the center and Section 7 in the south. On the average, Section 9 was mainly composed of coarse grain materials such as sands and gravels, Section 8 was composed of sands and silts and Section 7 was composed of silts and clays with intermittent sands, EMCON and Assoc., 1989.

Model parameter estimates for Part 1 were obtained from three sources: laboratory permeability measurements, in-situ testing in selected boreholes, and published ranges of parameters based on soil texture. In Part 1, any estimates made on soil texture were biased toward the conservative side and thus permitted more cumulative infiltration than might actually occur.

Part 1 model simulations showed that the minimum distance from the wetting front to the landfill excavation was never less than 80 feet (approximately 225 feet in the horizontal direction). Section 9 had the largest cumulative infiltration with nearly 100 $ft^3/ft^2$ of water infiltrating over the 8 day period and with 60 $ft^3/ft^2$ of that occurring within the first 5 days. Section 8 had a total of 50 $ft^3/ft^2$ cumulative infiltration over the 8 day period. Section 7 had little if any infiltration as the result of a low permeability layer directly under the streambed. The total cumulative inflow for Section 7



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was 3.5 $ft^3/ft^2$ over the 8 day period.

The report *Unsaturated Ground-water Flow Modeling for the Proposed Cholla Landfill - Part 2* (Maddock et al, 1989b) is hereafter referred to as Part 2. In Part 2, the hydraulic properties (saturated hydraulic conductivity and moisture retention) based on soil textures were compared with the same hydraulic properties obtained from laboratory analysis. Laboratory values for saturated hydraulic conductivity and moisture retention were determined by Daniel B. Stephens and Associates (Stephens, 1989) on fourteen undisturbed cores taken from boreholes in Section 7 and Section 8. On the basis of these comparisons, Part 2 concluded that the hydraulic properties estimated by soil textures conservatively represented the field conditions found at the proposed Cholla Landfill site.

This report presents the results of three tasks. Task 1 reviewed pertinent records on the Agua Fria River to document flow conditions and to determine the largest flood sequence of record. Task 2 summarized representative applications of UNSAT2, assembled and summarized available case histories, and described a modified UNSAT2 where nonconvergence for incremental time steps and varying boundary conditions did not require manual restart. Task 3 augmented the natural sensitivity analysis provided by the intrinsic variation in parameters amongst sections 7, 8, and 9 to include analysis of anisotropy, moisture content effects, and extended flow effects.

## 2.0 Review of Agua Fria River flow conditions

Historically, three U. S. Geological Survey surface water gages have measured continuous flow on the Agua Fria River downstream of the Waddell Dam (USGS, 1988). The Avondale gage (#0951397, approximately 8 miles south of the site) recorded a maximum of 13 consecutive days of flow for the years 1965 to 1982. The 13 day flow event occurred in February of 1980 with a maximum mean daily flow of 19,400 cubic-feet per second (cfs) on February 20th. The Avondale gage also recorded the second longest flow period, 9 days, in March of 1978. The maximum mean daily flow for that event was 7000 cfs on March 2nd.

For the Youngtown gage (#0951397, approximately 1 mile north of the site), 3.7 miles west of Peoria, there is a continuous record from 1965 to 1968. This gage recorded a maximum of 6 consecutive days of flow in December, 1968. The maximum mean daily flow for this event occurred on December 19 and measured 1.5 cfs.

Although the El Mirage gage (#0951365, approximately 3 miles north of the site) operated continuously on the Agua Fria from 1982 to 1985, the USGS has withheld publication of these records. Furthermore, all recording at this gage was discontinued in 1985.

Release records for Waddell Dam during the period 1983 to 1988 provide some relevant flow information. Measurable spills from the Waddell Dam occurred in three months of 1983: 25,940 ac-ft. in January, 50,390 ac-ft. in February, and 6,200 ac-ft. in March. The only measurable spill from Waddell Dam during the period of 1984 to 1988 was 1 ac-ft. in 1985.

Personal communications with Wynn Hjalmerson (May, 1990) at the USGS district office in Tucson and with Robert Wallace at the USGS subdistrict office in Tempe indicated the existence of unpublished continuous flow records for the El Mirage gaging station. These records indicate that flow on the Agua Fria below Waddell Dam never extended to 30 consecutive days.

## 3.0 UNSAT2 applications, case histories, and modification.

The theoretical basis was developed for the application of the UNSAT2 model to the problem of two-dimensional nonsteady flow of water in unsaturated and partly saturated porous media in *Finite Element Analysis of Two-Dimensional Flow in Soils Considering Water Uptake by Roots: I. Theory* (Neuman et al, 1975a). Part II of their study (*Finite Element Analysis of Two-Dimensional Flow in Soils Considering Water Uptake by Roots: II. Field Applications)* applied the model to two field tests. The first test involved one-dimensional flow and compared numerical results with those obtained experimentally from water balance studies on red cabbage grown in the presence of a water table. The second application examined two-dimensional flow in a complex field situation in the Netherlands



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where flow takes place under cropped field conditions through five anisotropic layers (Neuman et al. 1975b).

In *Saturated-Unsaturated Seepage by Finite Elements*, Neuman (1973) described the development of the UNSAT2 program as a tool to solve the quasilinear partial differential equations of transient seepage in saturated-unsaturated porous media. Neuman demonstrated the application of the model with two examples: one of seepage through an earth dam with a sloping core and horizontal drainage blanket, and the other of seepage though a layered medium cut by a complex topography. The two examples illustrated the model's ability to handle nonuniform flow regions having complex boundaries and arbitrary degrees of local anisotropy as well as flow in a vertical or horizontal plane or in a three-dimensional system with radial symmetry. Neuman noted that for the case of transient seepage through soils the concept of a free surface is not always applicable.

Wei and Shieh (1979) used the UNSAT2 model to analyze the transient seepage of the 95-meter high Guri earthfill dam in their investigation of the saturation of earthfill and foundation during and after staged reservoir filling. The authors used the movements of the saturation (wetting) fronts and the pore pressure distributions in the earthfill and its foundation to evaluate the deformation and stability of the Guri dam under staged construction and reservoir filling.

UNSAT2 was used to study the effect of soil cementing and channel widening on infiltration and recharge in two idealized profiles along the Rillito River of southern Arizona in *Simulating Effects of Channel Changes on Stream Infiltration*, (Guzman et al, 1989). Their study utilized a synthetic 10-year runoff in the river to provide head values for generating infiltration before and after bank protection in the two sections. Results from the two base simulations combined to produce four additional cases. The study examined the effects of various combinations of soil cementing in the banks and widening the river channel on infiltration and recharge.

The original form of the program would iterate to find a solution until the specified error tolerance is satisfied (convergent) or the maximum number of iterations is reached (non-convergent).

When convergence is achieved the simulation proceeds with the next time step. Otherwise, the program would stop and manual restart would be required. UNSAT2 was modified to solve the non-convergence problems encountered when dealing with extreme non-linearities during the solution of the partial differential equation. The modication was done by one of the authors of the present study to facilitate the sensitivity analysis discussed in the next section. In this modified version of the program if non-convergent conditions are encountered the time step is automatically decreased and the simulation is continued from the last convergent time step. By using this version of the program, operator interaction is minimized.

## 4.0 Sensitivity analysis

The three cross-sections represented a diversity in hydrologic properties, and so model results provide an initial sensitivity analysis. The wetting front movement through the fine texture materials of Sections 7 and 8, reported in Part 1 (Maddock et al, 1989a) indicated the effects of smaller hydraulic conductivities in comparison with Section 9. Section 9 was the only cross-section to which a sensitivity analysis was applied because geologic cross sections and in situ permeability tests indicated that these materials were of the highest permeability and thus afforded the greatest chance for the wetting front to contact the liner. Sensitivity to the hydrograph, saturated hydraulic conductivity and moisture content were evaluated and are described below.

### 4.1 Extended flow hydrograph

The hydrograph was extended by maintaining a 3 feet stage condition from day 5.833 through day 30 (Figure 2). The purpose of this extension was to simulate an extreme flow event at the site.

The finite element grid was modified to accommodate the water table in an attempt to eliminate artificial boundary effects. This was necessary because the extended hydrograph would result in a greatly increased infiltration volume. In previous simulations this was not necessary because the



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wetting front never reached the depth of the water table which is located at approximately 868 feet above mean sea level (207 feet below the river bed in Section 9, EMCON and Assoc., 1989). The finite element grid consisted of 2464 nodes and 2358 elements (Figure 3).

Description of materials below the regional water table were obtained from well log B-3-1-36BBC (EMCON and Assoc., 1989). Using these descriptions hydraulic properties of these materials (CL, SP-SW, and SP) were assigned the same values as those of similar texture in the overlying layers of the section (Table 1). The hydraulic properties of the materials were identical to those used in previous simulations (Maddock et al., 1989). The hydraulic conductivities were assumed to be isotropic and the initial suction was set to 15 feet. The cross-section used is shown in Figure 4. The zero elevation in this cross-section corresponds to an elevation of 773 feet above mean sea level (95 feet below the water table).

The results of this simulation are shown in Figures 5 to 22. These figures show the simulated pressure head distribution at selected times from day 1 through 30. Results for days 1 through 8 are identical to those reported in Maddock et al., (1989) for Section 9. Cumulative infiltration is shown in Figures 23 and 24. Cumulative infiltration is the total volume of water which has entered the cross-section per unit surface area of river bed at a given time. The cumulative infiltration at 14 days was 150 $ft^3/ft^2$ and the total cumulative infiltration at 30 days was 212 $ft^3/ft^2$. The slope of this curve is controlled by the hydraulic properties of the materials. The initial steep slope is reduced at day one when the wetting front reaches the relative low permeability clay layer (Figure 5). From then on, the slope of the cumulative infiltration curve is controlled by the permeability of this clay layer. Note that between days 5 and 6 when the river stage changes from 1 feet to a maximum of 10 feet and returns to 3 feet (Figure 2) the effect on cumulative infiltration is negligible. This indicates that the infiltration rate is unaffected by the stage in the river.

Between 11 and 12 days (Figure 18 and 19) the wetting front is shown to reach the water table directly below the center of the river channel. Although the definition of the wetting front is somewhat

arbitrary, it is approximated by the zero pressure isobar. In Figure 18, the westward protrusion of the wetting front at 240 feet above the datum (1013 feet above mean sea level) and 325 feet from the center of the stream appears to be due to the lower permeability of the clay layer. At day 30 (Figure 22), the wetting front approached to within approximately 50 feet radially (110 feet measured in the horizontal direction) of the liner. For the hydraulic properties assigned to the materials, the prescribed initial suction and the hydrograph used, the results of the simulation indicate that the wetting front would not contact the base of the liner before 30 days. Continuing the simulation on this grid would only show the artificial effect of the imposed no-flow condition on the west boundary. The total head distributions for days 14 and 30 are shown in Figures 25 and 26. These figures provide an indication of the direction of flow. In a homogeneous isotropic flow domain the stream lines, the instantaneous direction of water particles, would be perpendicular to the total head contour. With this in mind, Figures 25 and 26 would indicate vertically downward flow below the proposed landfill and the side liner. There is a slight horizontal flow component below the Agua Fria river bed which becomes more pronounced at day 30.

## 4.2 Anisotropic hydraulic conductivity

The material properties were changed in all layers in order to reflect an anisotropy ratio ($K_x/K_z$) of 2/1 with the vertical saturated hydraulic conductivity remaining the same as in the initial simulations but the horizontal saturated hydraulic conductivity being increased by a factor of 2. The hydrograph up to 14 days, the initial suction, the position of the water table and the finite element discretization were identical to those described in 4.1 above. The model was run to simulate 14 days of infiltration.

The pressure head distribution for selected days between 1 and 14 are presented in Figures 27-41. Comparing the results for day two from this and the previous simulation (Figures 6 and 28) clearly shows the effect of anisotropy. For the isotropic case the wetting front has reached a distance of approximately 235 feet from the center of the river (Figure 6) whereas for the anisotropic case the



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wetting front has moved as far as 260 feet from the center of the river (Figure 28). Under the conditions for this simulation the wetting front reached the water table directly below the center of the river channel at ten days (Figure 38).

For this anisotropic case at 14 days (Figure 41) the most westward protrusion of the wetting front occurs in the poorly sorted gravel layer at an elevation of 198 feet (971 feet above mean sea level). At this point the wetting front has moved 207 feet west of the intersection of river bank and the river bed (located 188 feet from the center of the river). In contrast, for the isotropic case (Figure 20) the wetting front has moved only 157 feet west of this point. The effect of the anisotropy is an increase of 32% in the lateral movement of the wetting front. This percentage represents the relative difference at 14 days. With increasing time, this relative difference will decrease. Based on the results of the simulation described in 4.1 above, the pressure head distribution of this simulation extended to 30 days would show the artificial effects of the boundary condition on the west.

The shape of the cumulative infiltration curve (Figure 42) is similar to that of the simulation described in 4.1 (Figure 23). However, the total cumulative infiltration at 14 days is 160 $ft^3/ft^2$ as compared to 150 $ft^3/ft^2$ for simulation in 4.1. Total head distribution at 14 days is presented in Figure 43. The contours indicate a slight increase in horizontal flow component as compared to the isotropic case (Figure 25). The horizontal contours of total head indicate that flow is vertically downward below the excavation of the proposed landfill.

## 4.3 Wet initial conditions

To examine the effect of higher initial soil water content, the initial suction in the unsaturated portion of the simulated cross-section was decreased from 15 feet to 2 feet. For the materials in this section this reduction produces the changes in initial water content, $\theta_r$, shown in the chart below.

| | SP | GP | GW-GM | SP-SP | CL |
|---|---|---|---|---|---|
| $\theta_{r15}$ | 0.05 | 0.02 | 0.10 | 0.05 | 0.33 |
| $\theta_{r02}$ | 0.06 | 0.02 | 0.14 | 0.085 | 0.41 |

All other conditions and parameter values are identical to those described in 4.2.

The pressure head distributions for selected days between 1 and 14 are presented in Figures 44-57. Comparing the results for day 2 from this and the previous simulation in 4.2, (Figures 28 and 45) clearly shows the significant effect of increased background water content. For the dry case the wetting front has reached a distance of approximately 260 feet from the center of the river (Figure 28) whereas for the wetter case the wetting front has moved as far as 308 feet from the center of the river (Figure 45). Under the conditions for this simulation the wetting front reached the water table directly below the center of the river channel at 6 days (Figure 53). For the case where the initial suction is 2 feet the most westward protrusion of the wetting front at 14 days (Figure 57) occurs at the boundary of the poorly graded gravel layer and the poorly graded sand layer (Figure 4) at an elevation of 198 feet (971 feet above mean sea level). At this point the wetting front has moved 370 feet west of the intersection of the river bank and the river bed. In contrast, for the drier anisotropic case (Figure 41) the wetting front has moved only 207 feet west of this position. The effect of the increased background water content is an increase of 79% in the lateral movement of the wetting front. This percentage represents the relative difference at 14 days.

The cumulative infiltration curve is shown in Figure 58. The rate of infiltration for the first 0.6 day is the same as in the previous simulations (Figures 23 and 42). At about 0.6 day the wetting front reaches the low permeability clay layer (Figure 44). From then on, the rate of infiltration decreases unlike the previous cases in 4.1 and 4.2. The total cumulative infiltration at 14 days is 130 $ft^3/ft^2$ as compared to 160 and 150 $ft^3/ft^2$ for simulations in 4.1 and 4.2, respectively. Total head distribution for 14 days is presented in Figure 59. The contours indicate a slight increase in horizontal flow component as compared to the isotropic case (Figure 25). The horizontal contours of total head



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indicate that flow is vertically downward below the excavation of the proposed landfill.

## 5.0 Summary

Examination of the historical records of flow in the Agua Fria River demonstrates that the extended hydrograph used for these simulations represents an extreme flow event. The natural sensitivity analysis provided by soil variations among the different sections was augmented to investigate the effects of anisotropy within homogeneous layers, high initial soil moisture content and extended flow in the river on the movement of the wetting front. These conditions all contribute to an enhancement of the lateral movement of the wetting front. Results of the simulation of the extended hydrograph under isotropic conditions showed that the wetting front only approached to within 50 feet radially (110 feet measured in the horizontal direction) of the liner at 30 days (Figure 22). The effect of anisotropy of saturated hydraulic conductivity and higher initial water content was to increase the lateral movement of the wetting front and slightly increase the total cumulative infiltration. However, even for the anisotropic case with high initial water content the wetting front only approached to within 50 feet radially (110 feet measured in the horizontal direction) of the liner at 14 days (Figure 57). Examination of the total head distribution (Figure 59) at 14 days indicates vertical flow paths. These flow paths imply that little, if any, further lateral movement of the wetting front is likely, that near steady-state flow conditions exist, and thus, no impact to the liner would occur at later times. Therefore, it was not necessary to simulate the flow system beyond 14 days.

## 6.0 References


EMCON and Associates, *Application for aquifer protection permit and approval of solid waste disposal operations plan Cholla Sanitary Landfill, El Mirage, Arizona. Part II: hydrogeologic study*, Sept 1989.

Guzman, A. G., L. G. Wilson, S. P. Neuman, and M. D. Osborn, *Simulating Effects of Channel Changes on Stream Infiltration*, in Journal of Hydraulic Engineering, vol. 115, no. 12, Paper no. 24160, December 1989, pp. 1631-1645.

Maddock, Thomas III, M. J. Sully, A. G. Guzman, G. Wittmeyer, E. Paleologos, *Unsaturated ground-water flow modeling for proposed Cholla landfill*, July 1989a.

Maddock, Thomas III, M. J. Sully, G. Wittmeyer, *Unsaturated ground-water flow modeling for proposed Cholla landfill - Part 2*, October 1989b.

Neuman, S. P., *Saturated-Unsaturated Seepage by Finite Elements*, in Journal of the Hydraulics Division, ASCE, vol. 99, no. HY12, Proceedings Paper 10201, December 1973, pp. 2233-2250.

Neuman, S. P., R. A. Feddes, and E. Bresler, *Finite Element Analysis of Two-Dimensional Flow in Soils Considering Water Uptake by Roots: I. Theory*, in Soil Science Society of America Proceedings, vol. 39, no. 2, March-April, 1975a.

Neuman, S. P., R. A. Feddes, and E. Bresler, *Finite Element Analysis of Two-Dimensional Flow in Soils Considering Water Uptake by Roots: II. Field Applications*, in Soil Science Society of America Proceedings, vol. 39, no. 2, March-April, 1975b.

USGS, *Water Resources Data for Arizona, Part I. Surface Water Records*, 1965-1988.




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Figure 1

Location of Three Modeled Cross-sections

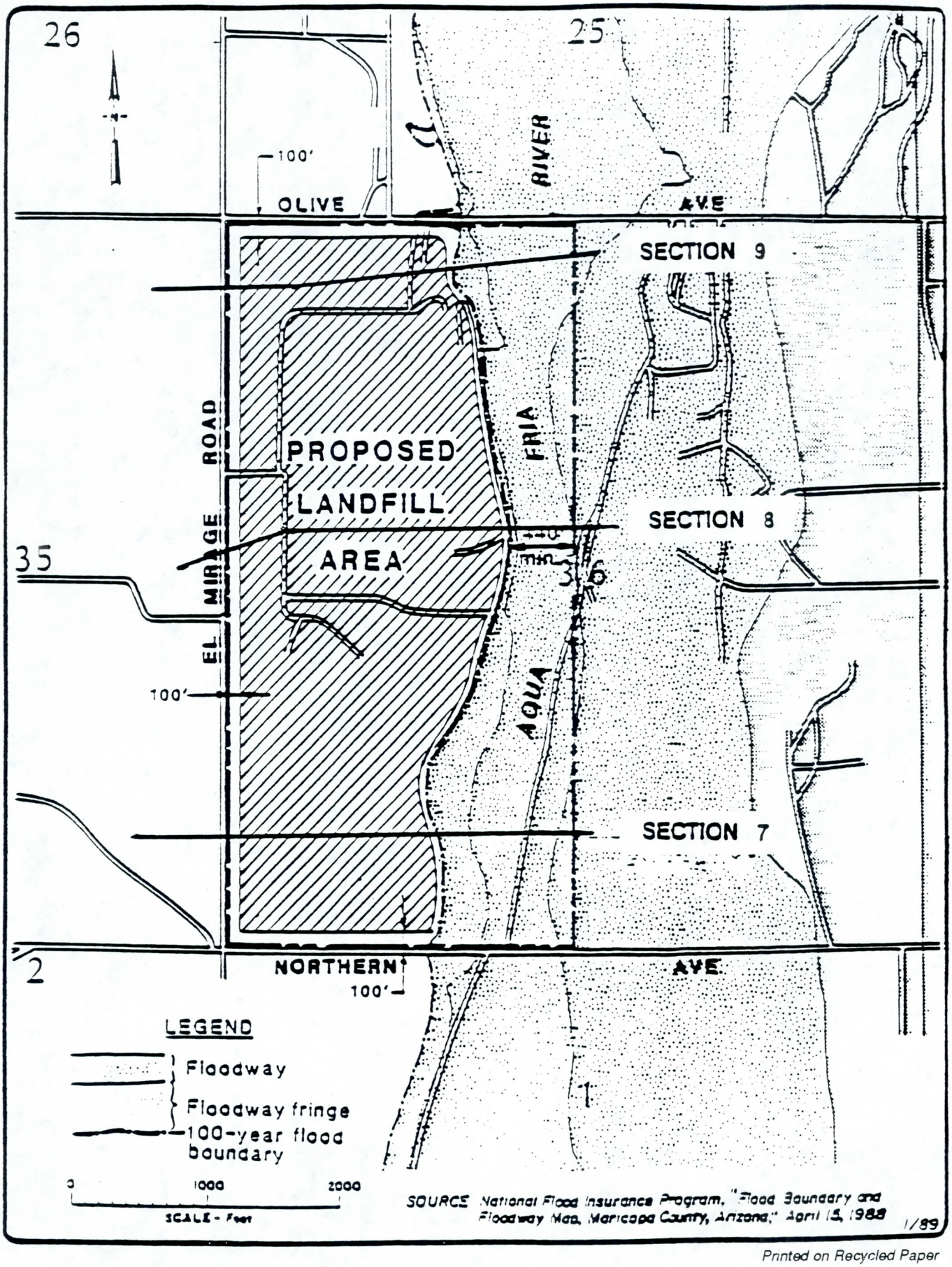


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**Figure 2**

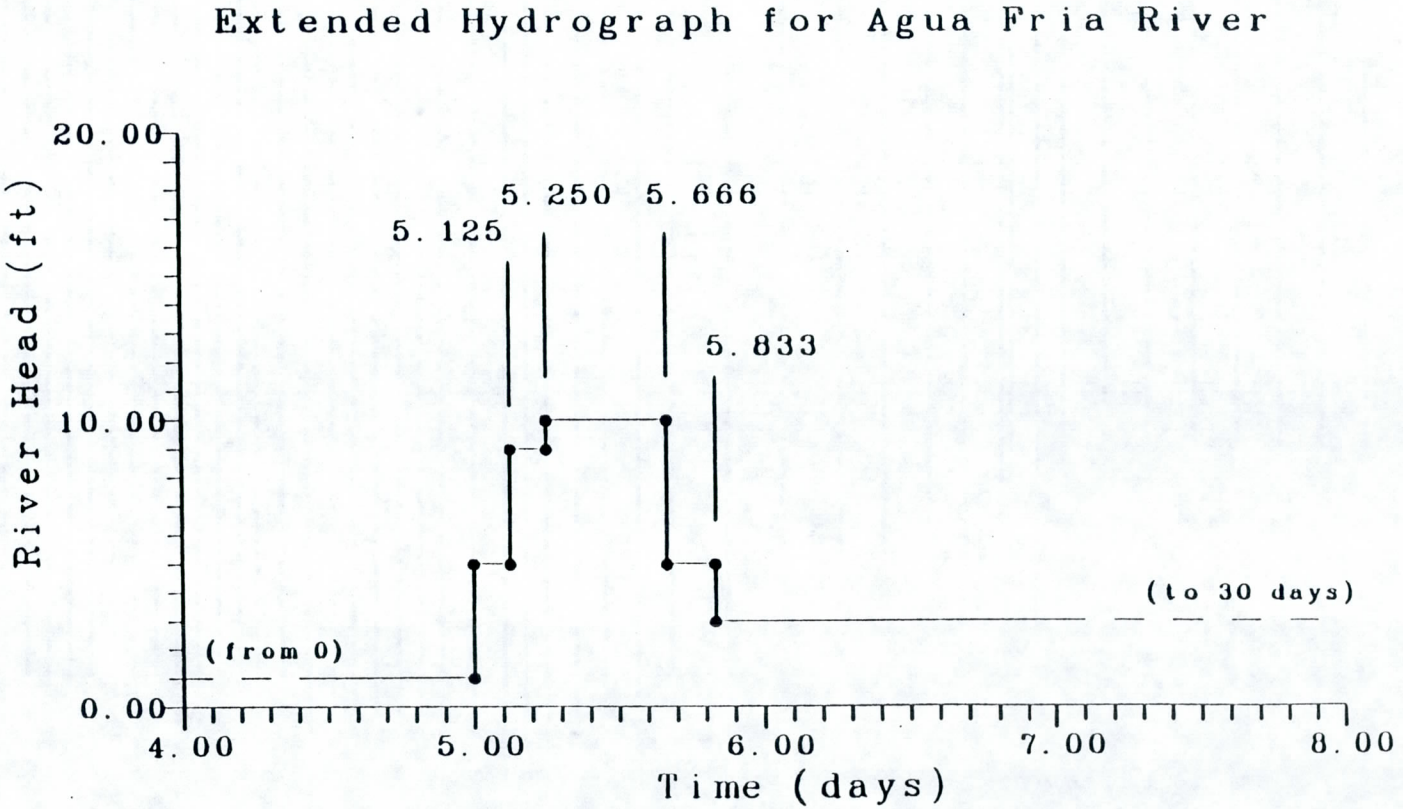




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Figure 3

**Finite Element Discretization**

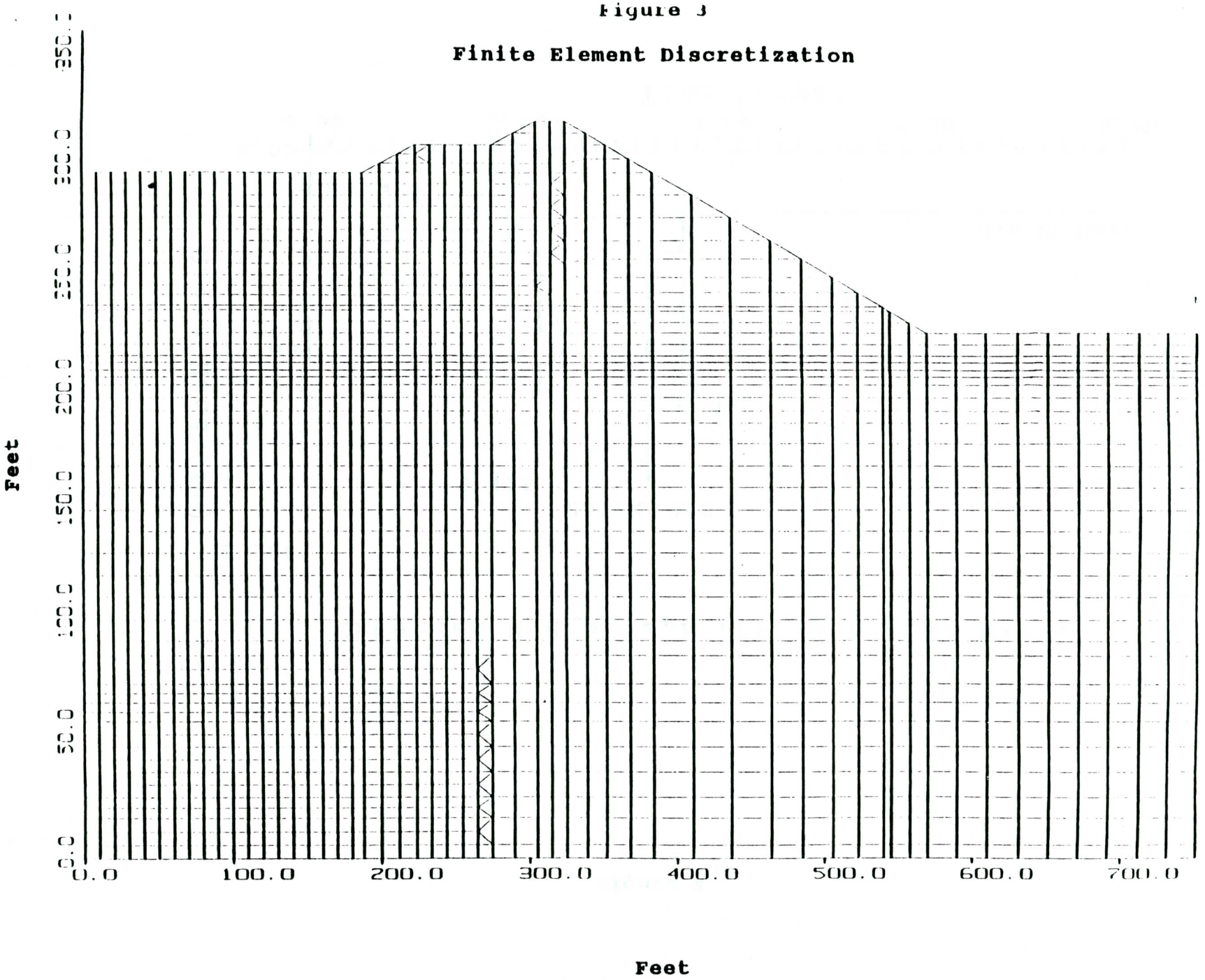
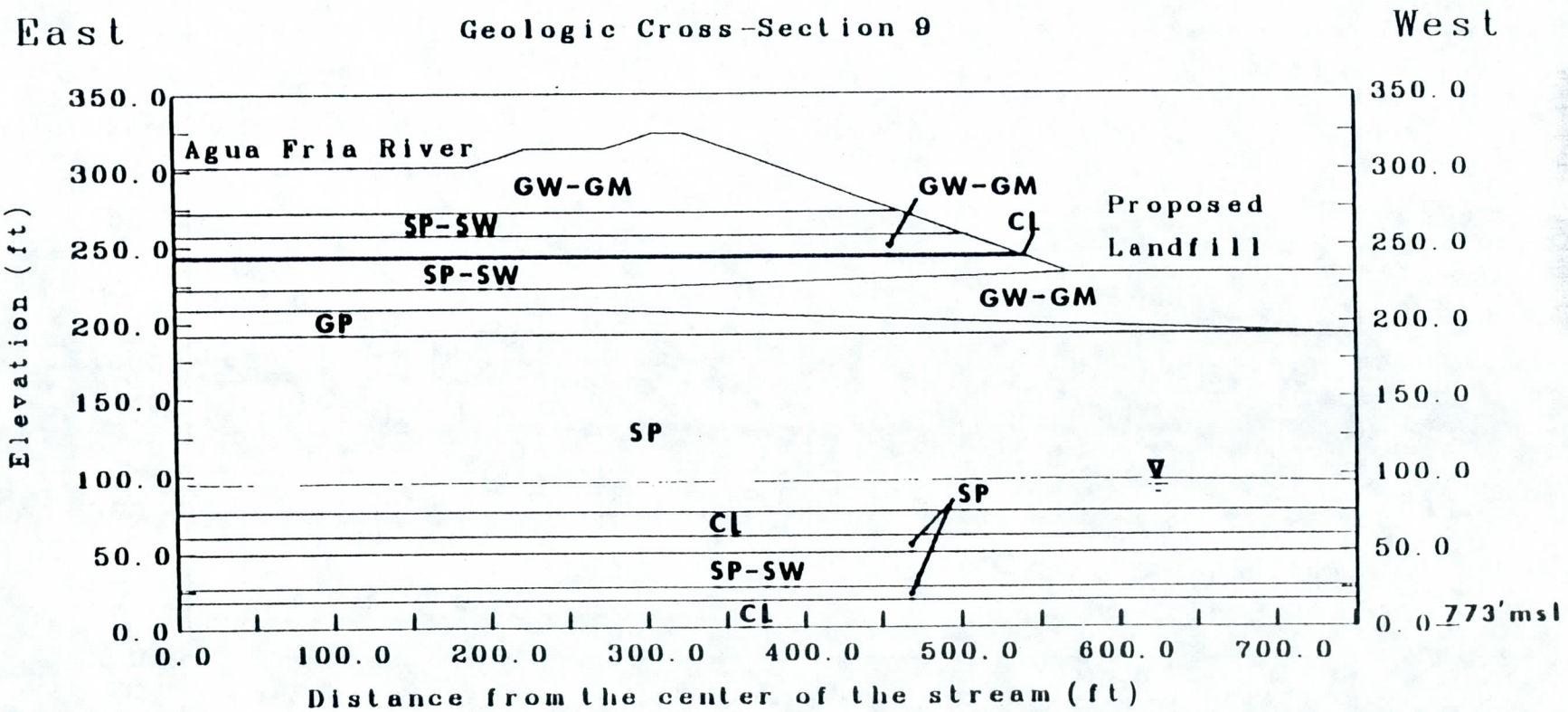


Figure 4



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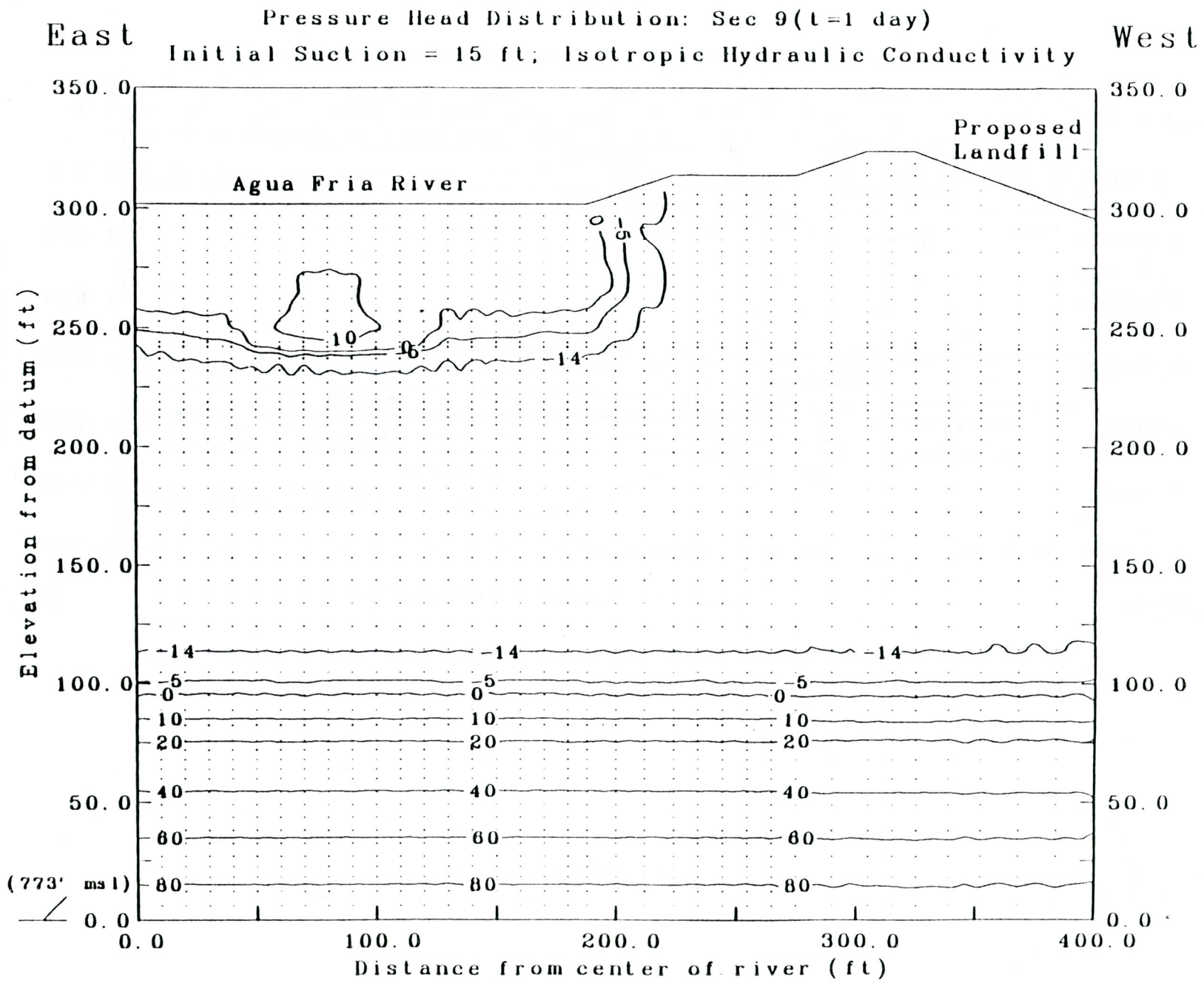
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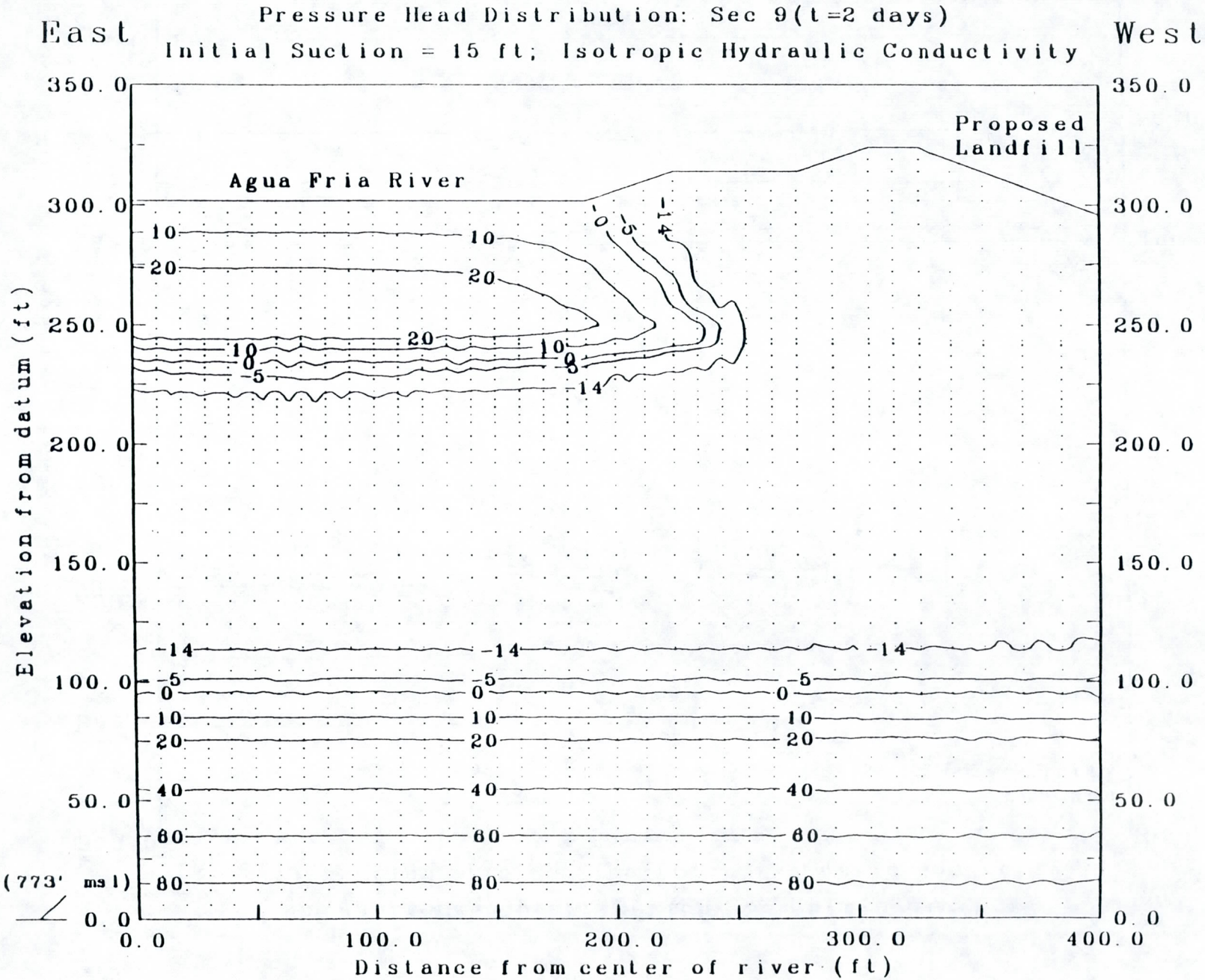
Initial Suction = 15 ft; Isotropic Hydraulic Conductivity
East
West
Agua Fria River
Proposed Landfill
Elevation from datum (ft)
(773' msl)
Distance from center of river (ft)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
0.0
100.0
200.0
300.0
400.0
0
-5
10
-14
-14
-5
0
10
20
40
60
80

Figur



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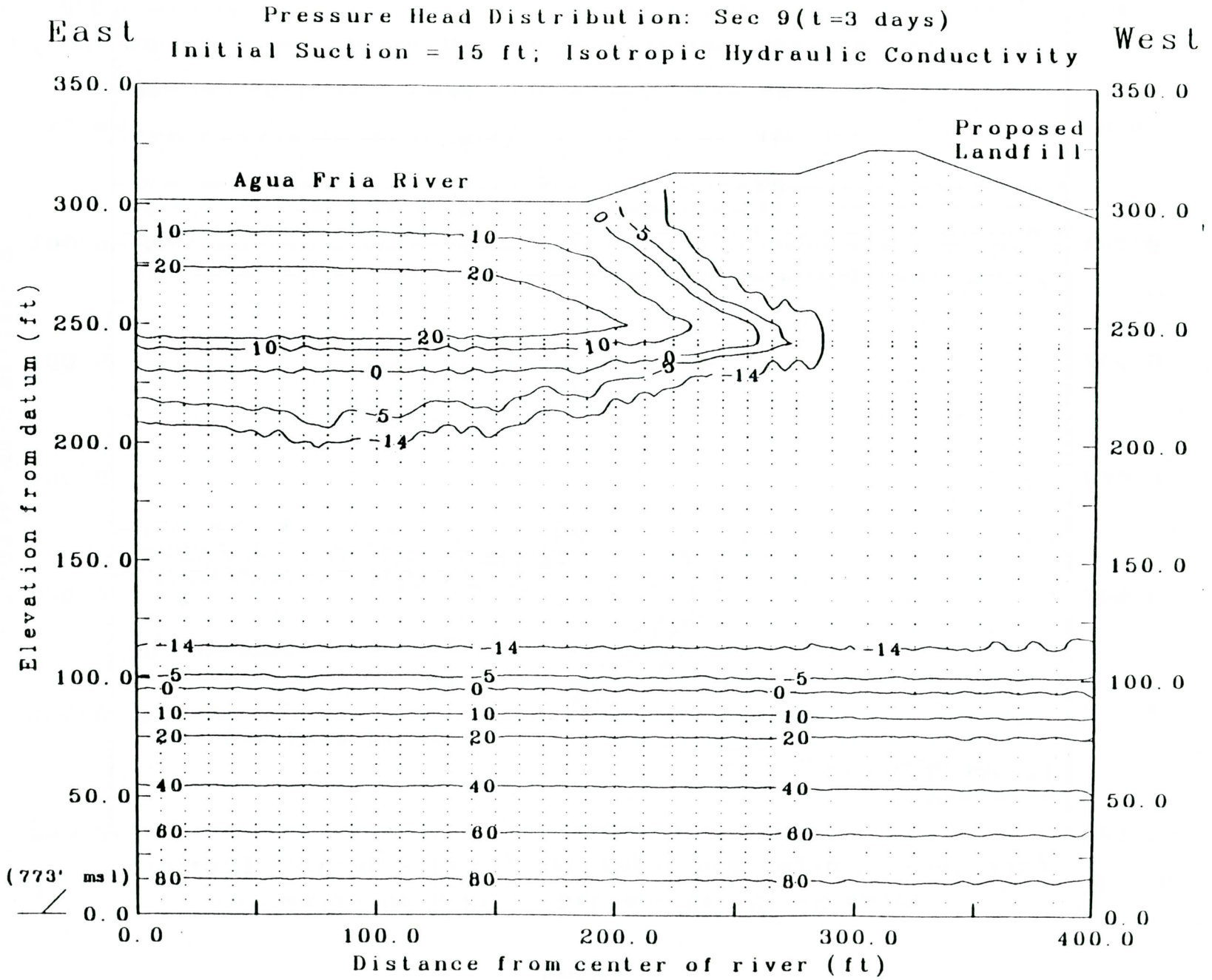
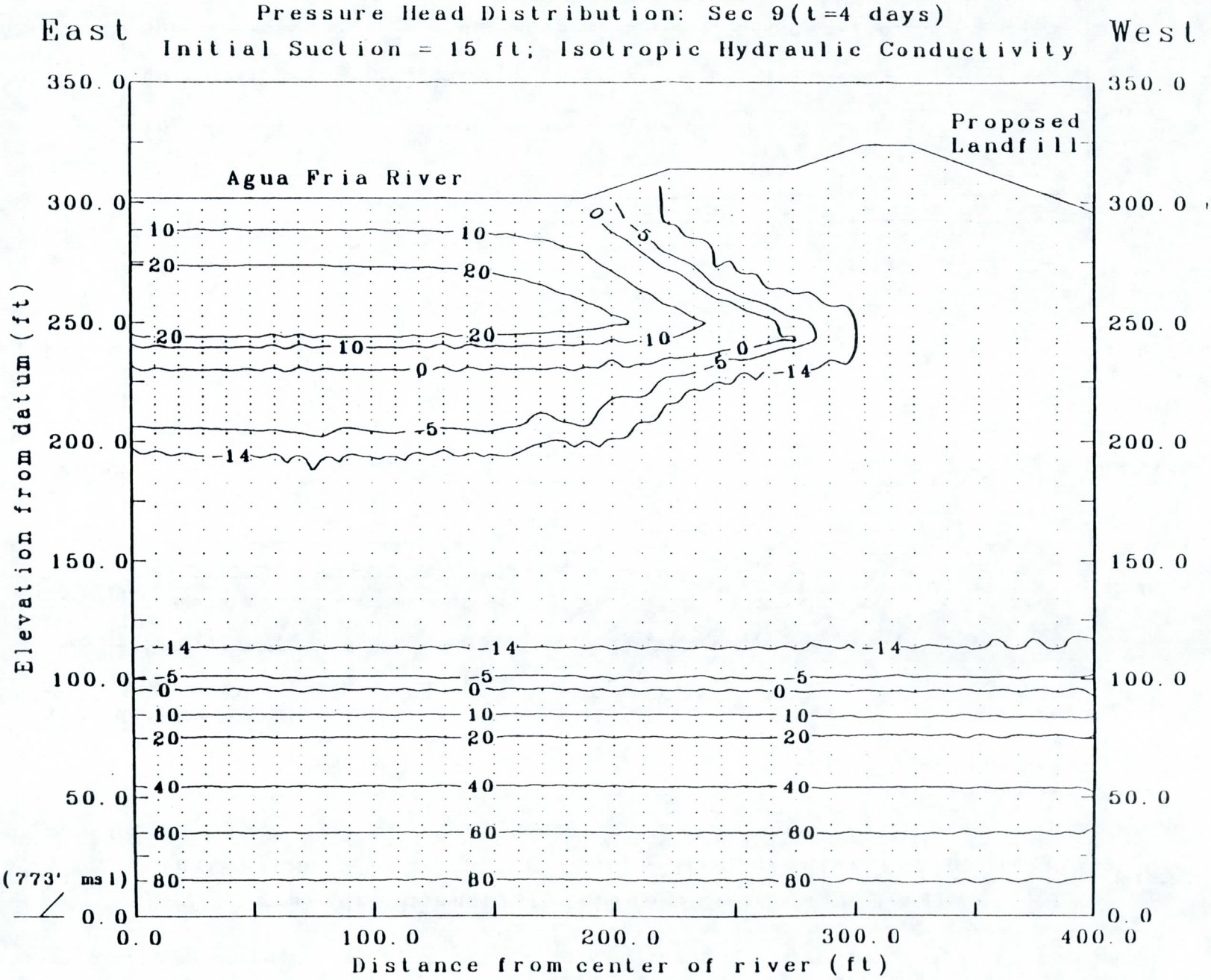

Pressure Head Distribution: Sec 9(t=3 days)
Initial Suction = 15 ft; Isotropic Hydraulic Conductivity
East
West
Agua Fria River
Proposed Landfill
Elevation from datum (ft)
(773' msl)
Distance from center of river (ft)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
100.0
200.0
300.0
400.0

**Figure 8**



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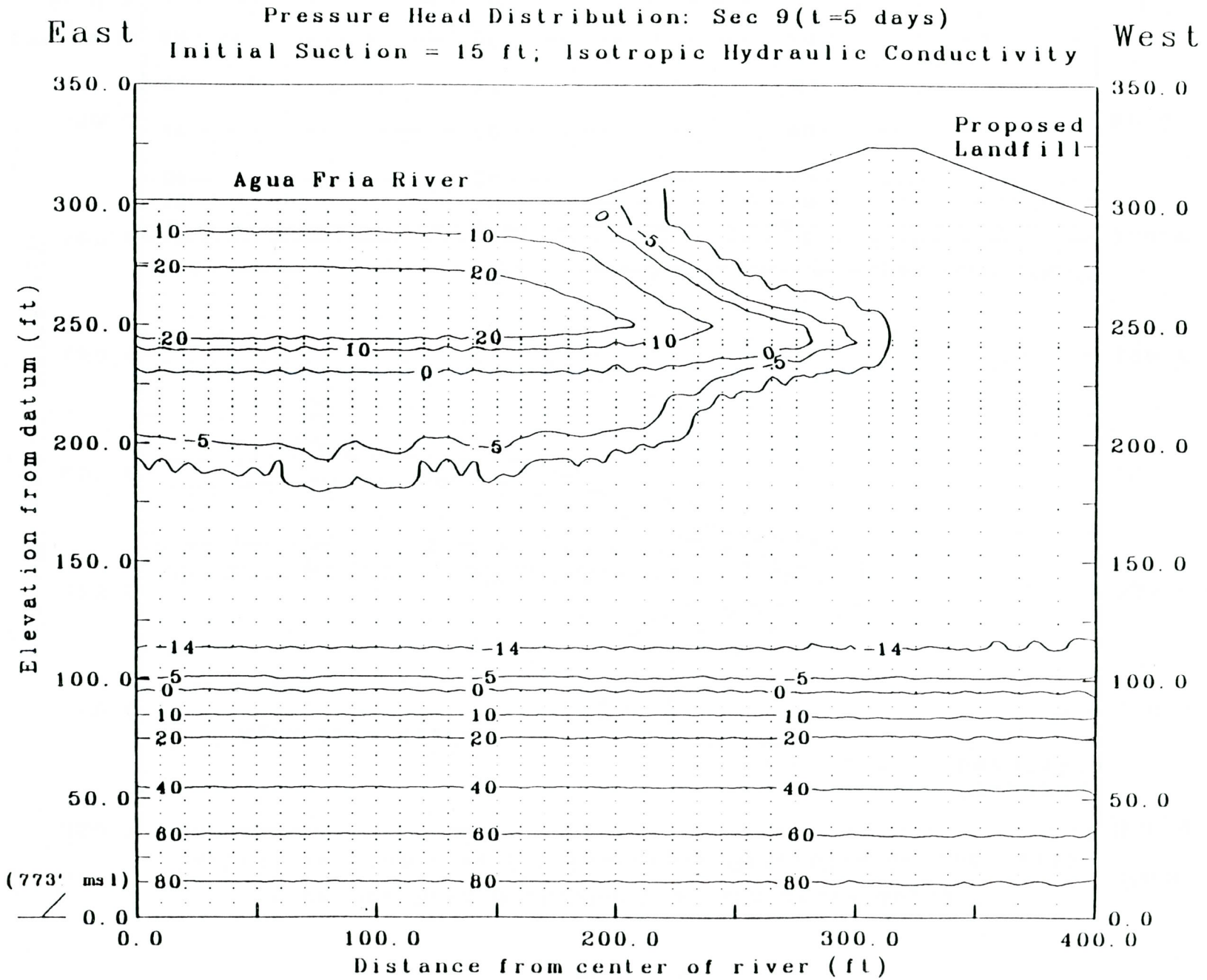
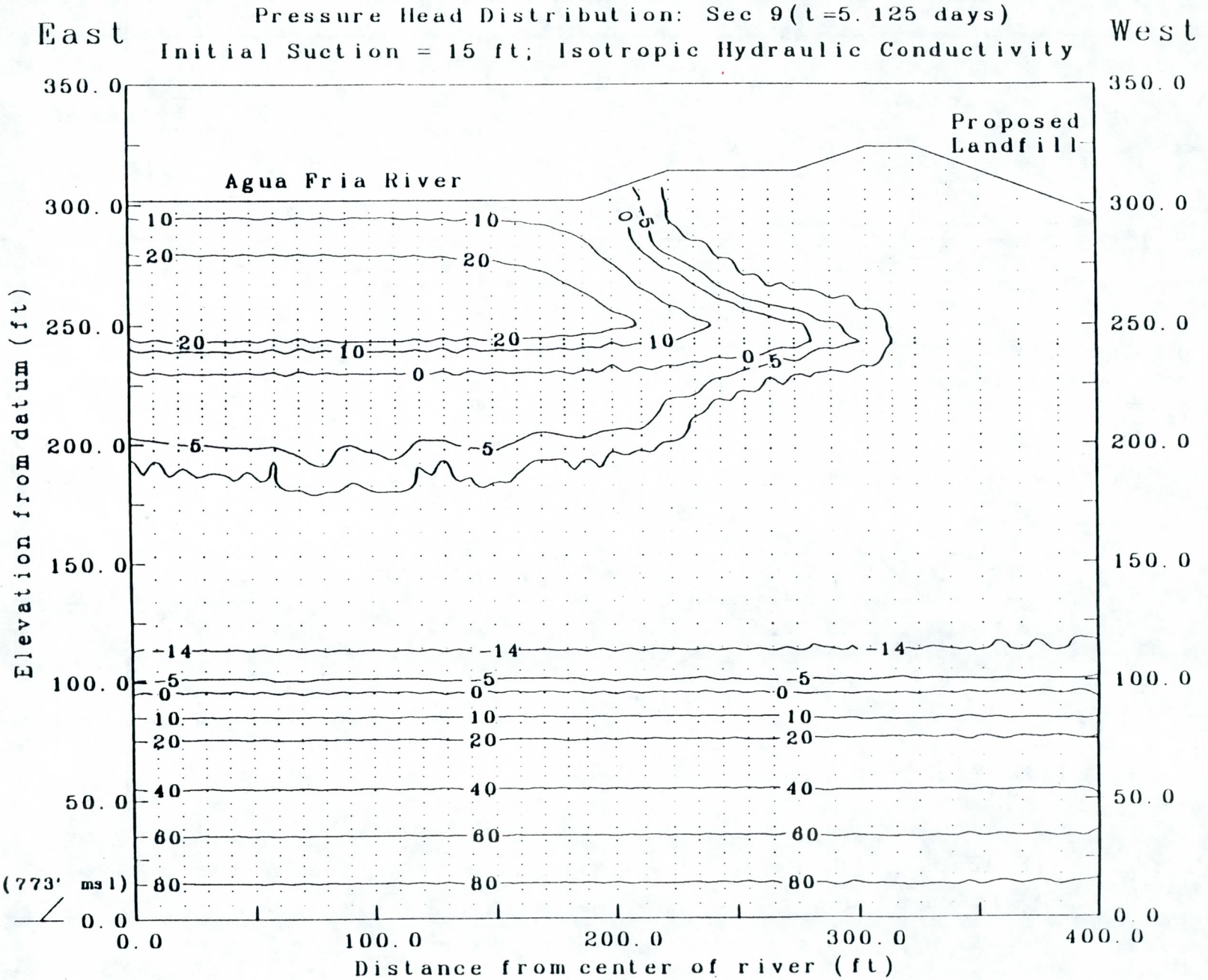

Pressure Head Distribution: Sec 9(t=5 days)
Initial Suction = 15 ft; Isotropic Hydraulic Conductivity
East
West
Proposed Landfill
Agua Fria River
Elevation from datum (ft)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
(773' msl)
0.0
100.0
200.0
300.0
400.0
Distance from center of river (ft)

**Figure 10**



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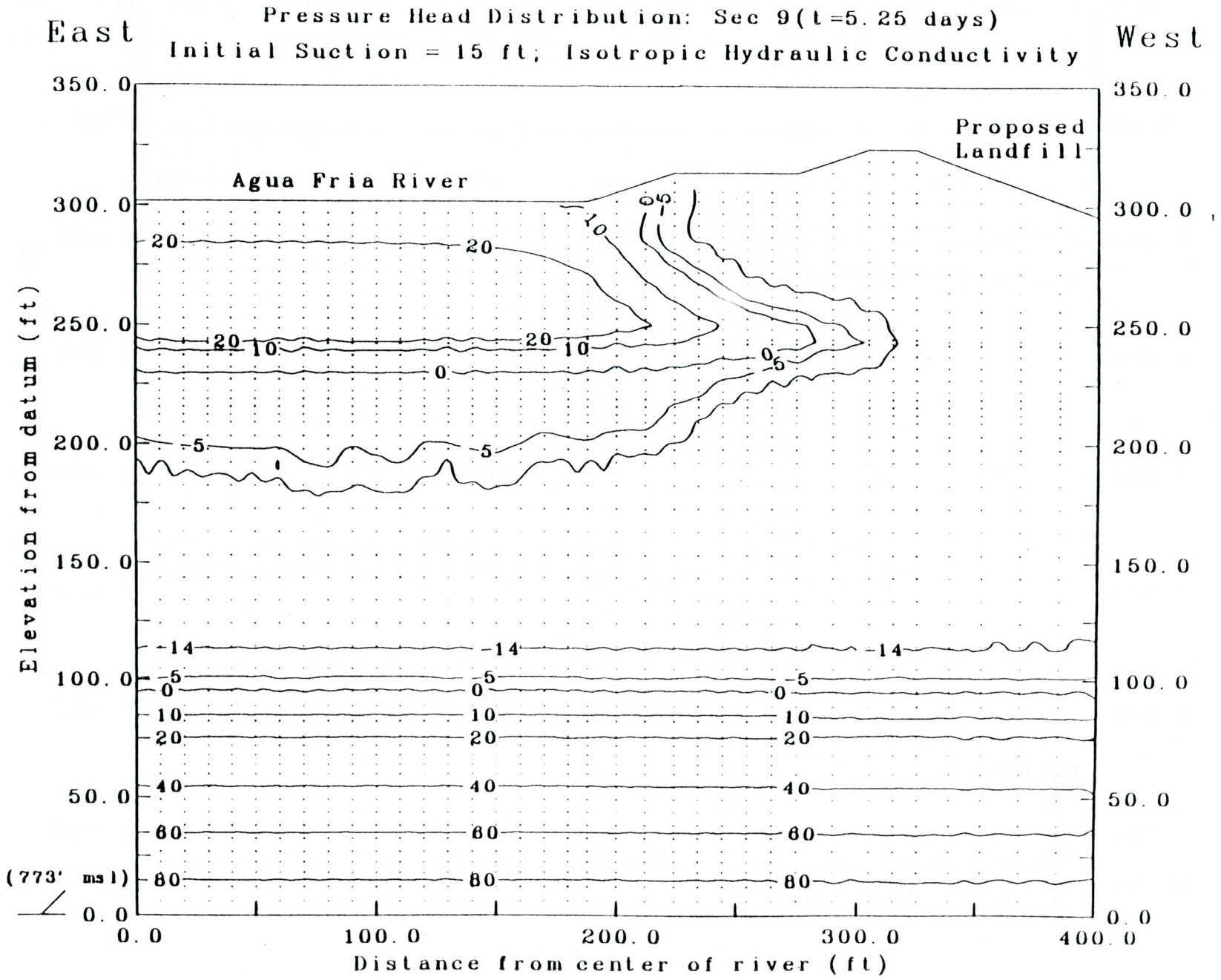
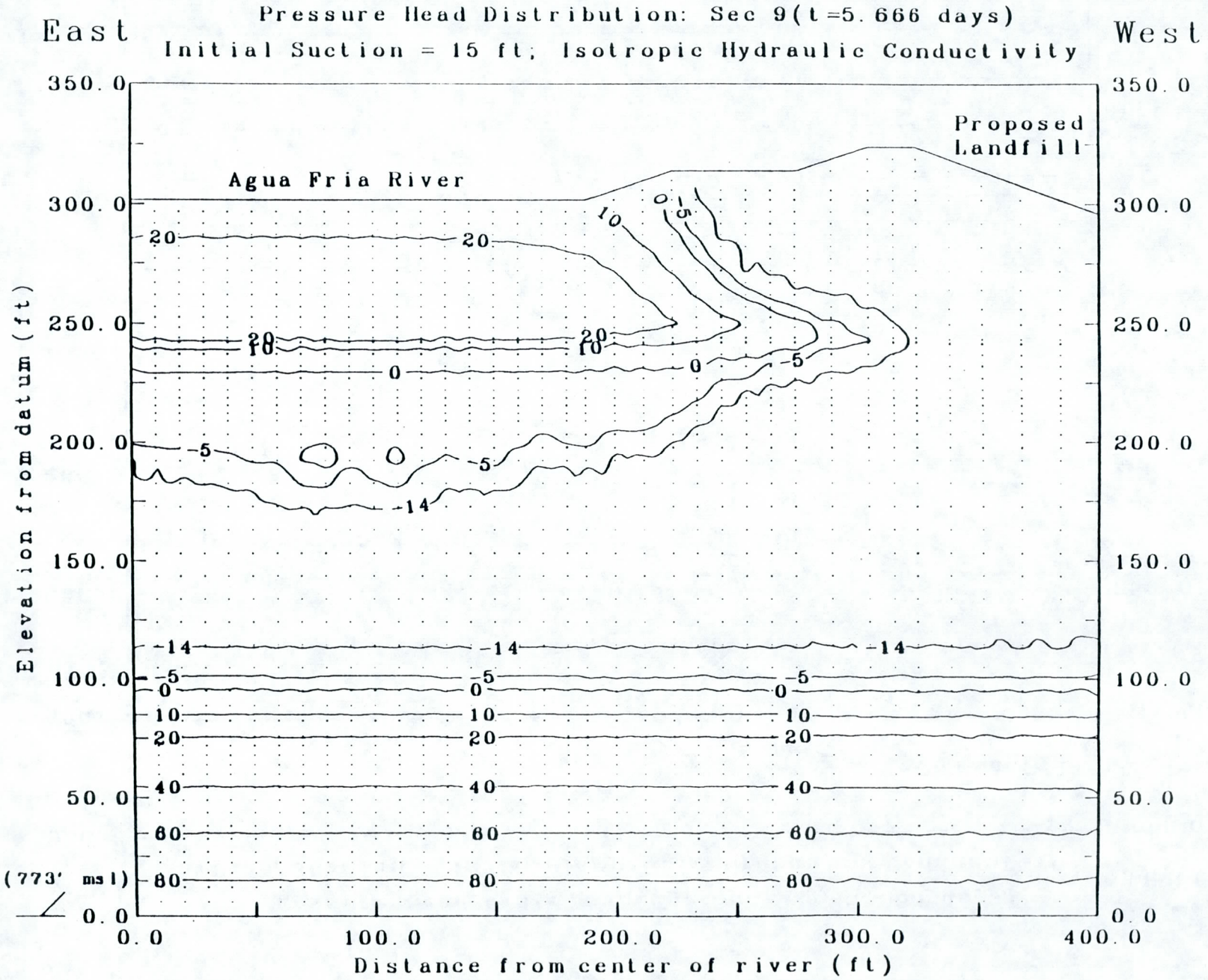

Pressure Head Distribution: Sec 9(t=5.25 days)
Initial Suction = 15 ft; Isotropic Hydraulic Conductivity
East
West
Agua Fria River
Proposed Landfill
Elevation from datum (ft)
(773' msl)
Distance from center of river (ft)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
0.0
100.0
200.0
300.0
400.0

**Figure 12**



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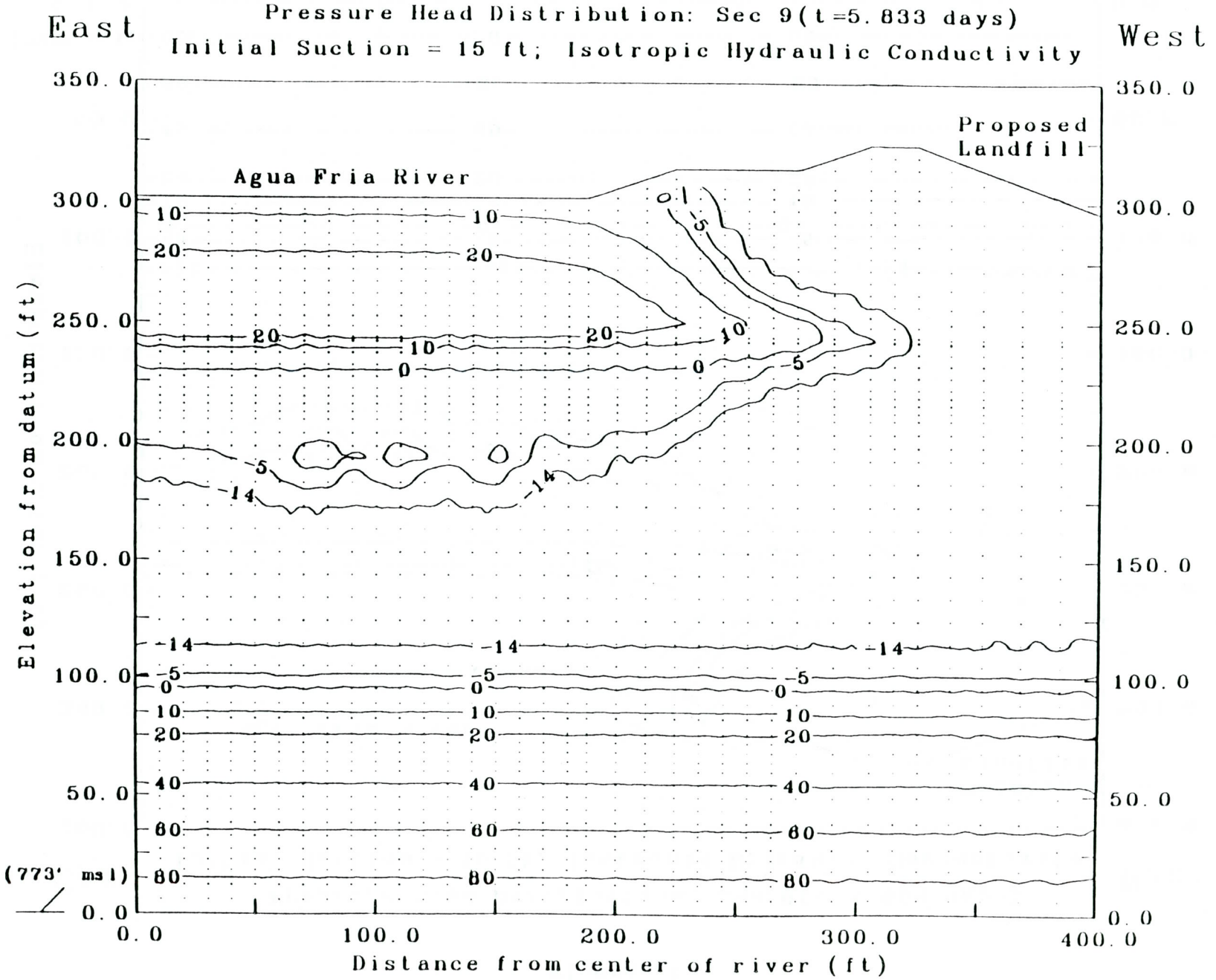
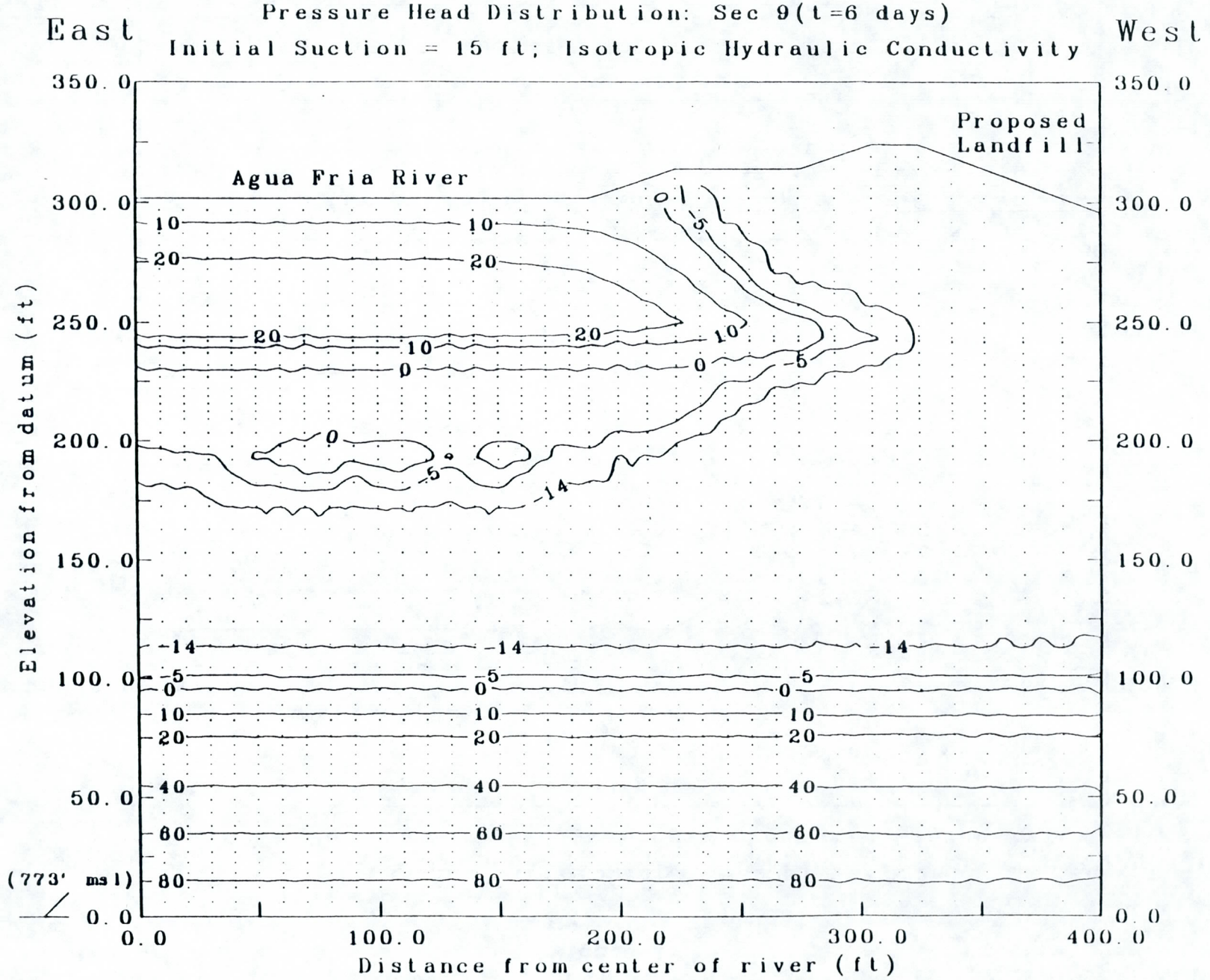

East
West
Pressure Head Distribution: Sec 9(t=5.833 days)
Initial Suction = 15 ft; Isotropic Hydraulic Conductivity
Proposed Landfill
Agua Fria River
Elevation from datum (ft)
Distance from center of river (ft)
(773' msl)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
100.0
200.0
300.0
400.0
10
20
0
-5
-14
40
60
80

**Figure 14**



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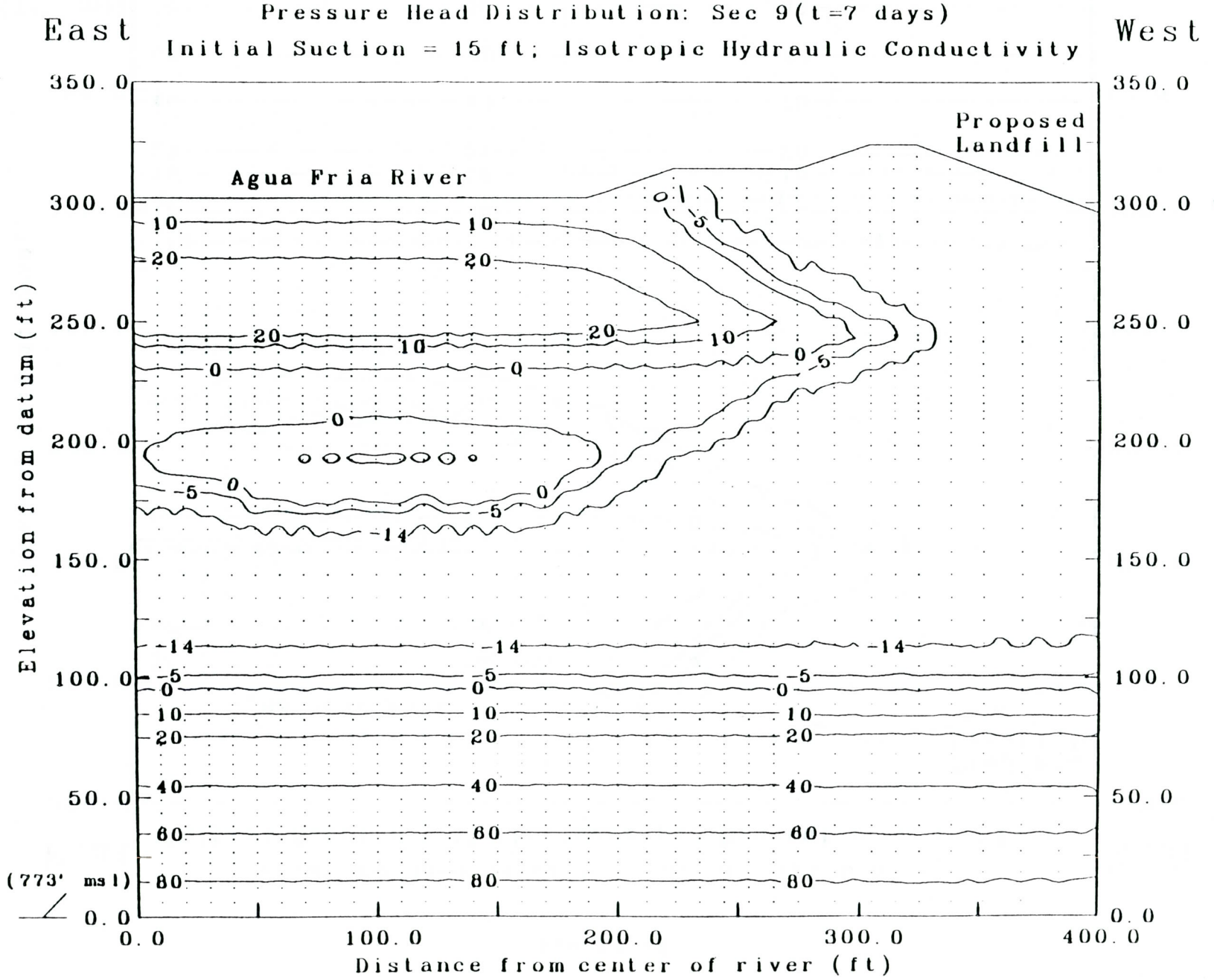
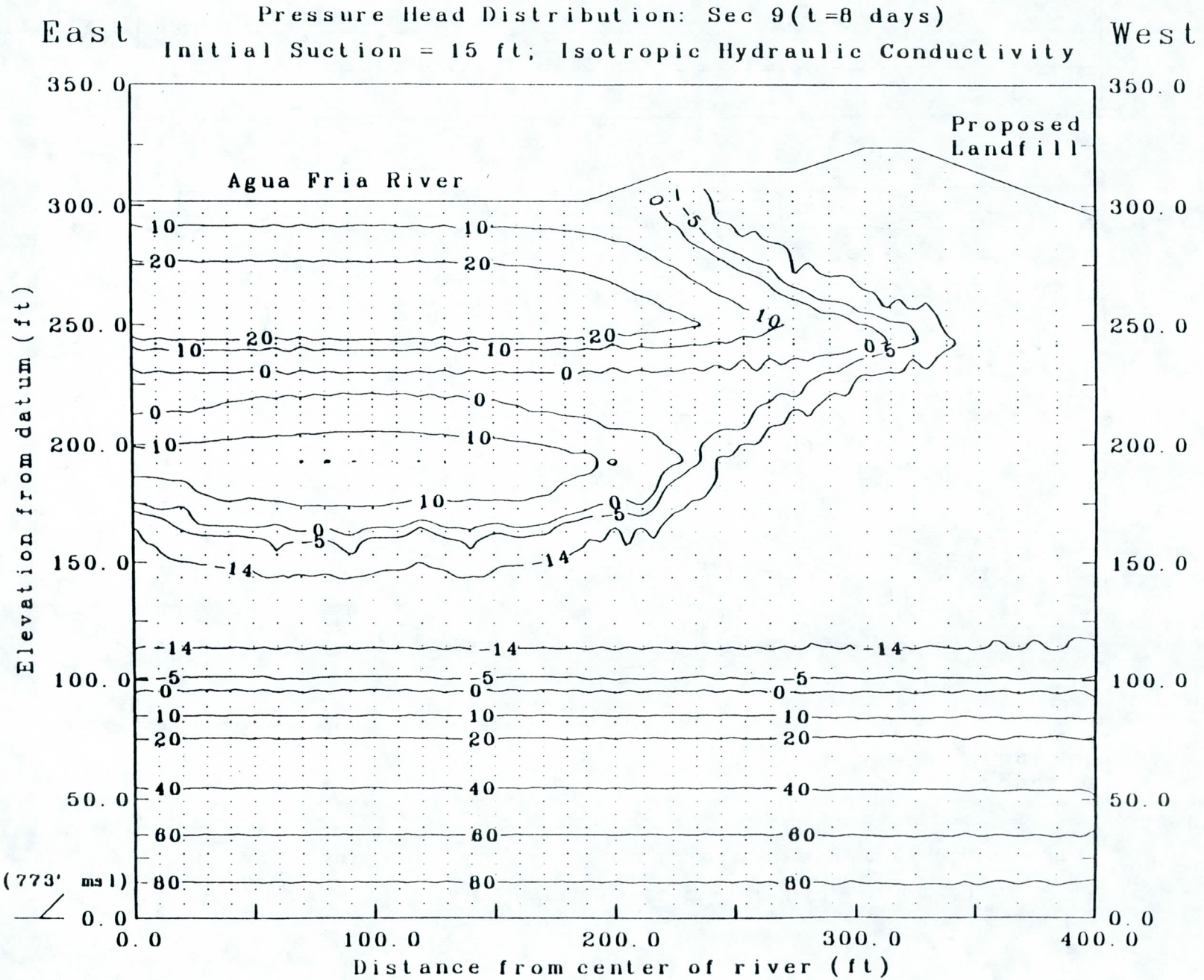

Pressure Head Distribution: Sec 9(t=7 days)
Initial Suction = 15 ft; Isotropic Hydraulic Conductivity
East
West
Proposed Landfill
Agua Fria River
Elevation from datum (ft)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
(773' msl)
0.0
100.0
200.0
300.0
400.0
Distance from center of river (ft)

**Figure 6**



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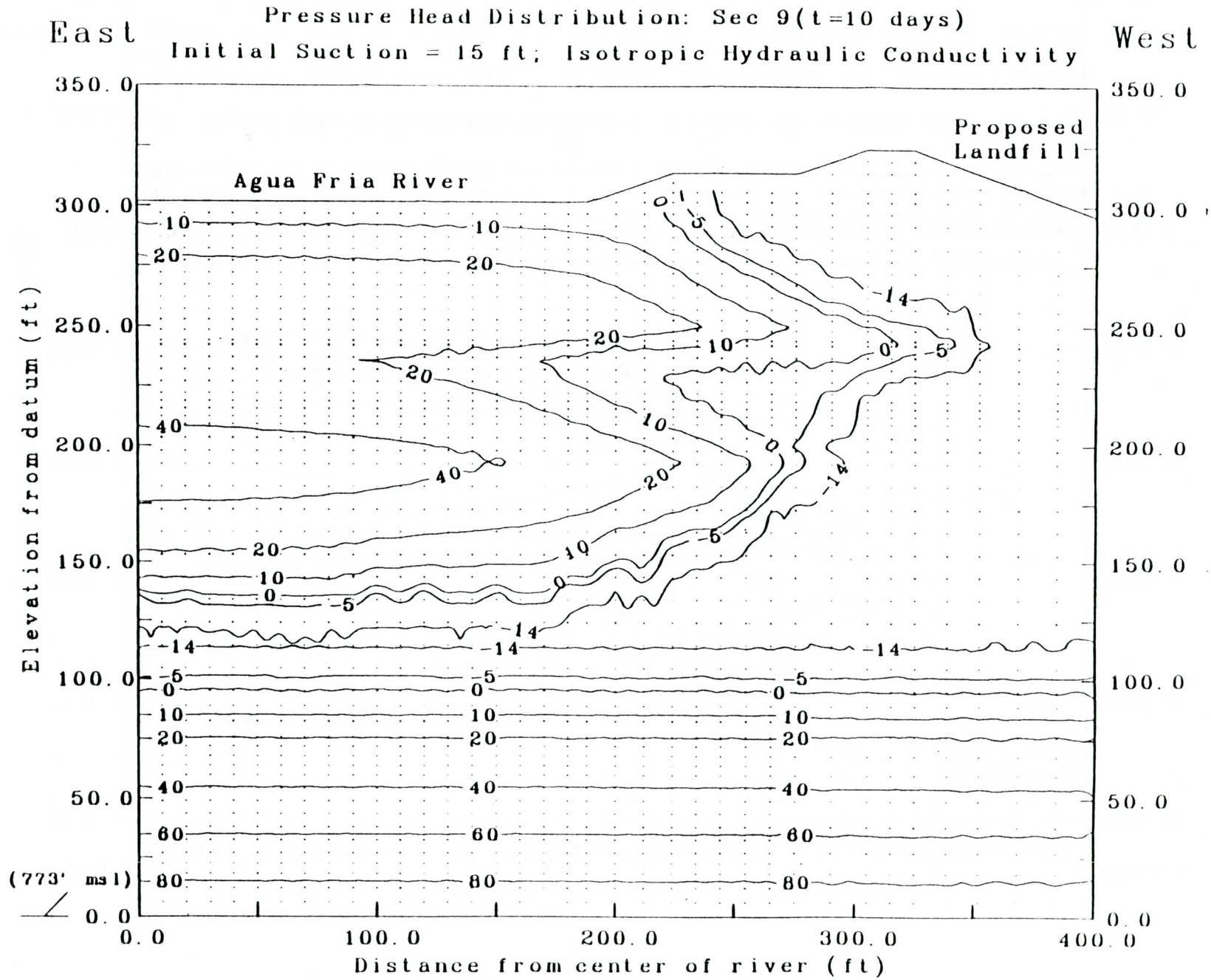
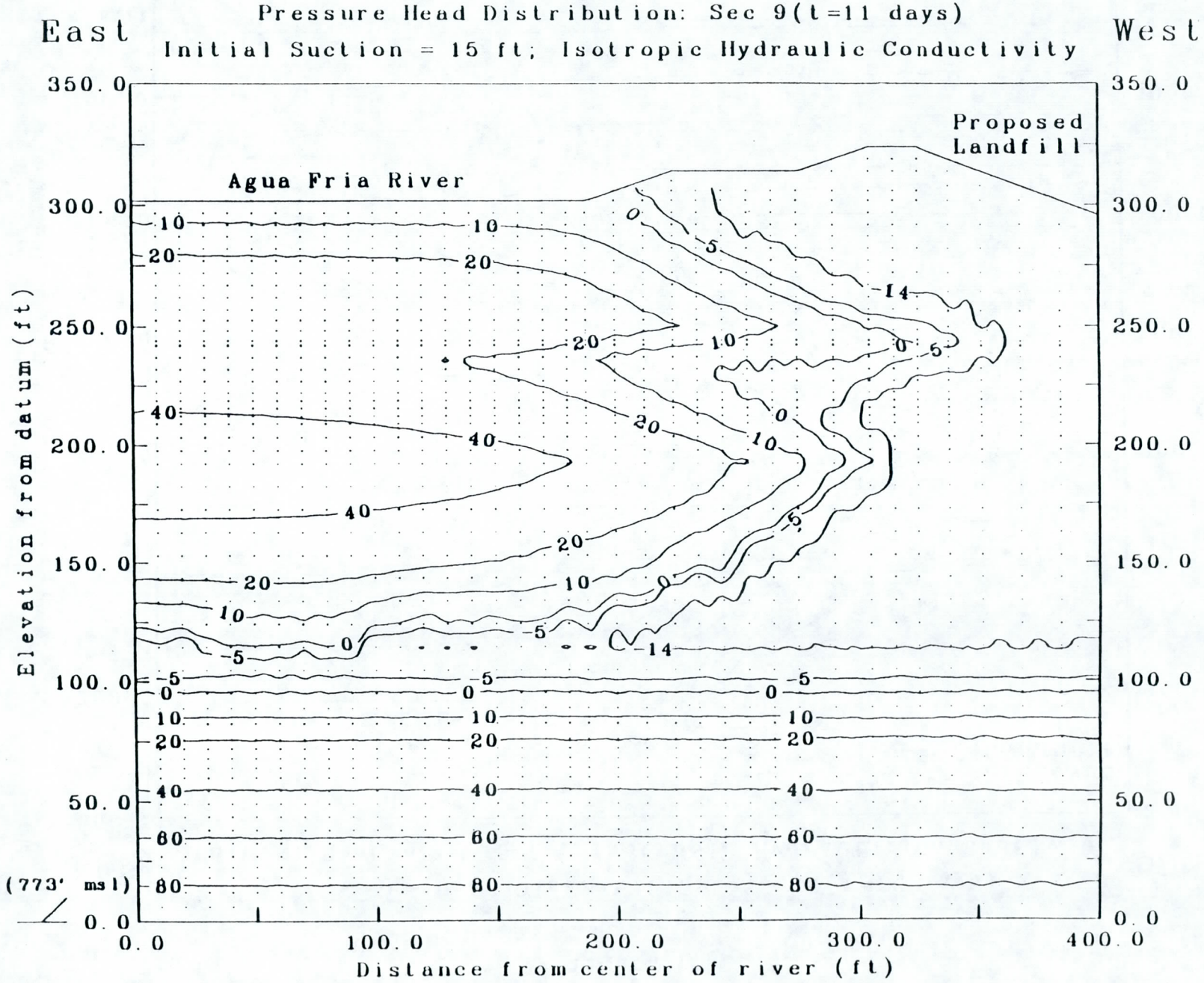

Pressure Head Distribution: Sec 9(t=10 days)
Initial Suction = 15 ft; Isotropic Hydraulic Conductivity
East
West
Agua Fria River
Proposed Landfill
Elevation from datum (ft)
(773' msl)
Distance from center of river (ft)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
0.0
100.0
200.0
300.0
400.0

**Figure ..**



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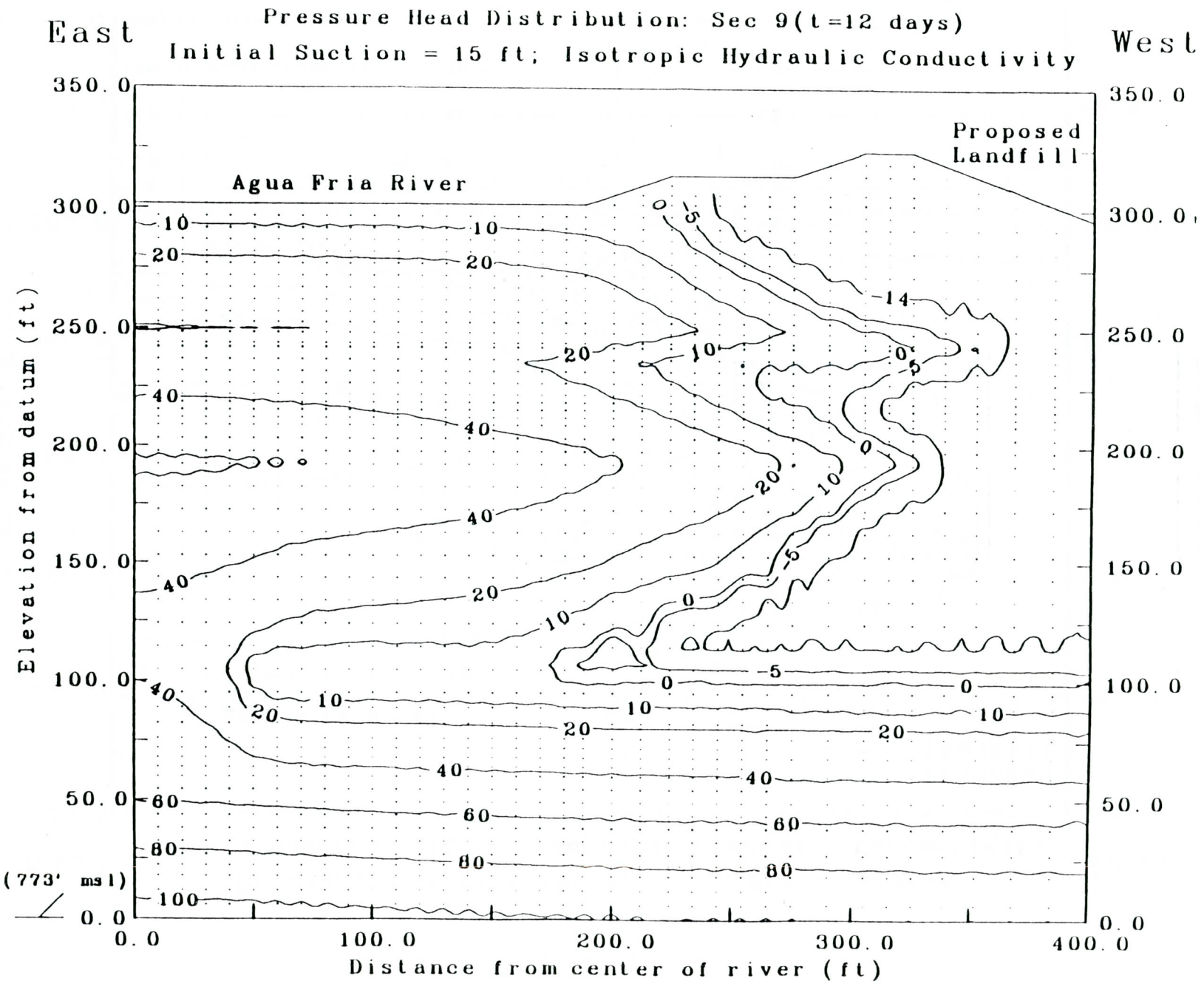
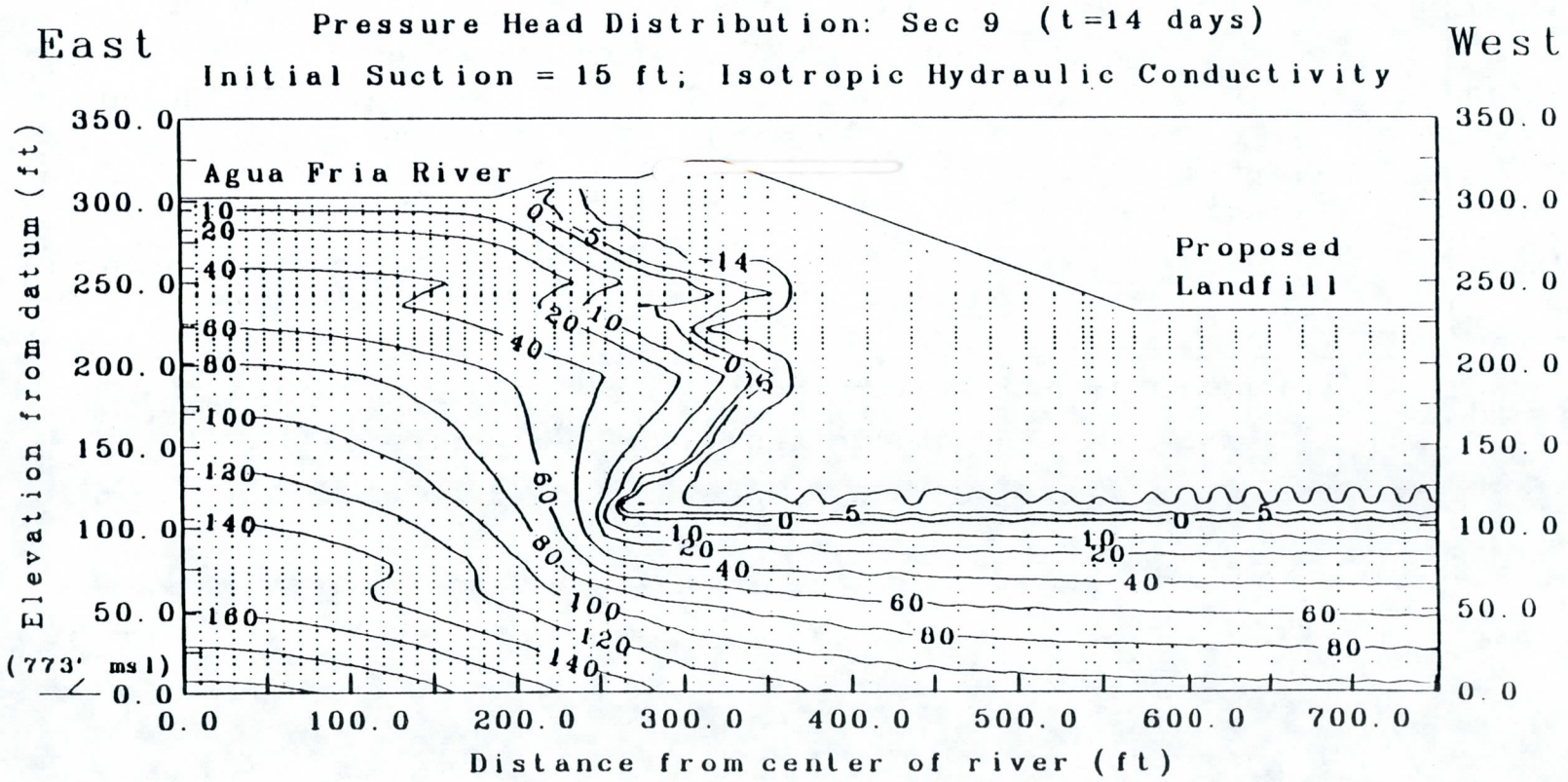

Pressure Head Distribution: Sec 9(t=12 days)
Initial Suction = 15 ft; Isotropic Hydraulic Conductivity
East
West
Agua Fria River
Proposed Landfill
Elevation from datum (ft)
(773' msl)
Distance from center of river (ft)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
100.0
200.0
300.0
400.0

**Figure 20**



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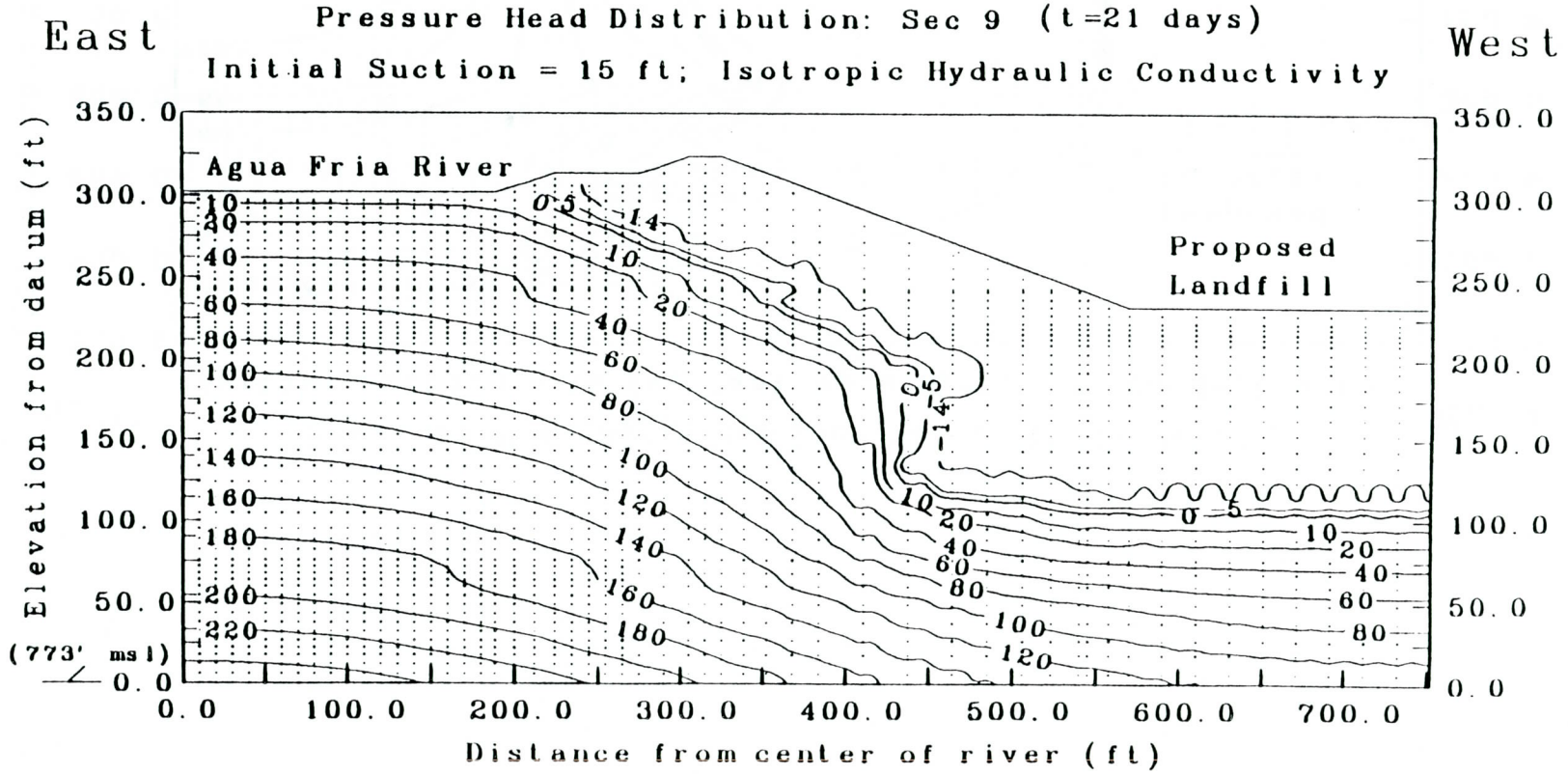
East
Pressure Head Distribution: Sec 9 (t=21 days)
Initial Suction = 15 ft; Isotropic Hydraulic Conductivity
West
Agua Fria River
Proposed Landfill
Elevation from datum (ft)
(773' msl)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
0.0
100.0
200.0
300.0
400.0
500.0
600.0
700.0
Distance from center of river (ft)

Figure 22

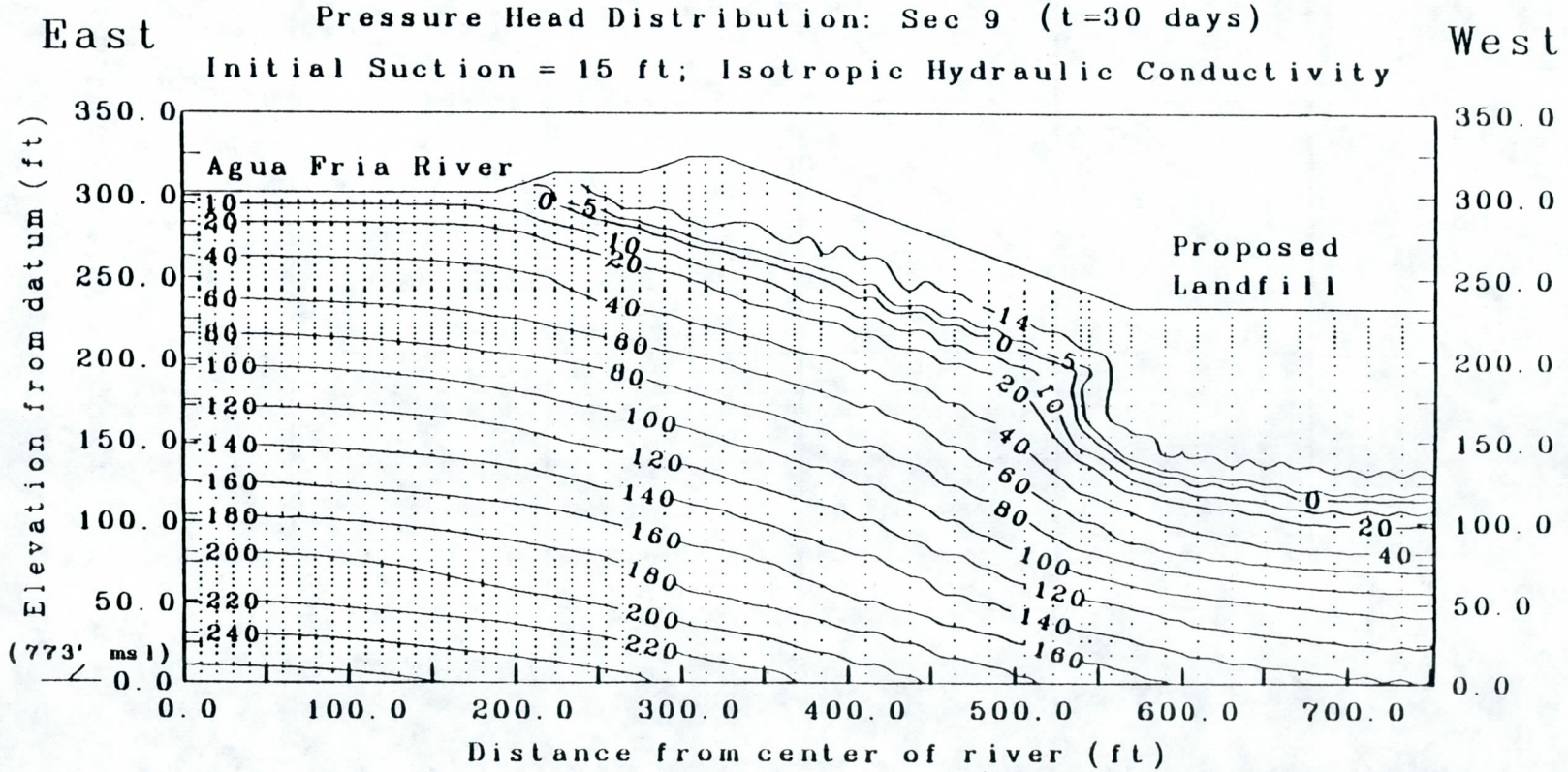




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**Figure 23**

Cumulative Infiltration as a Function of Time

Section 9

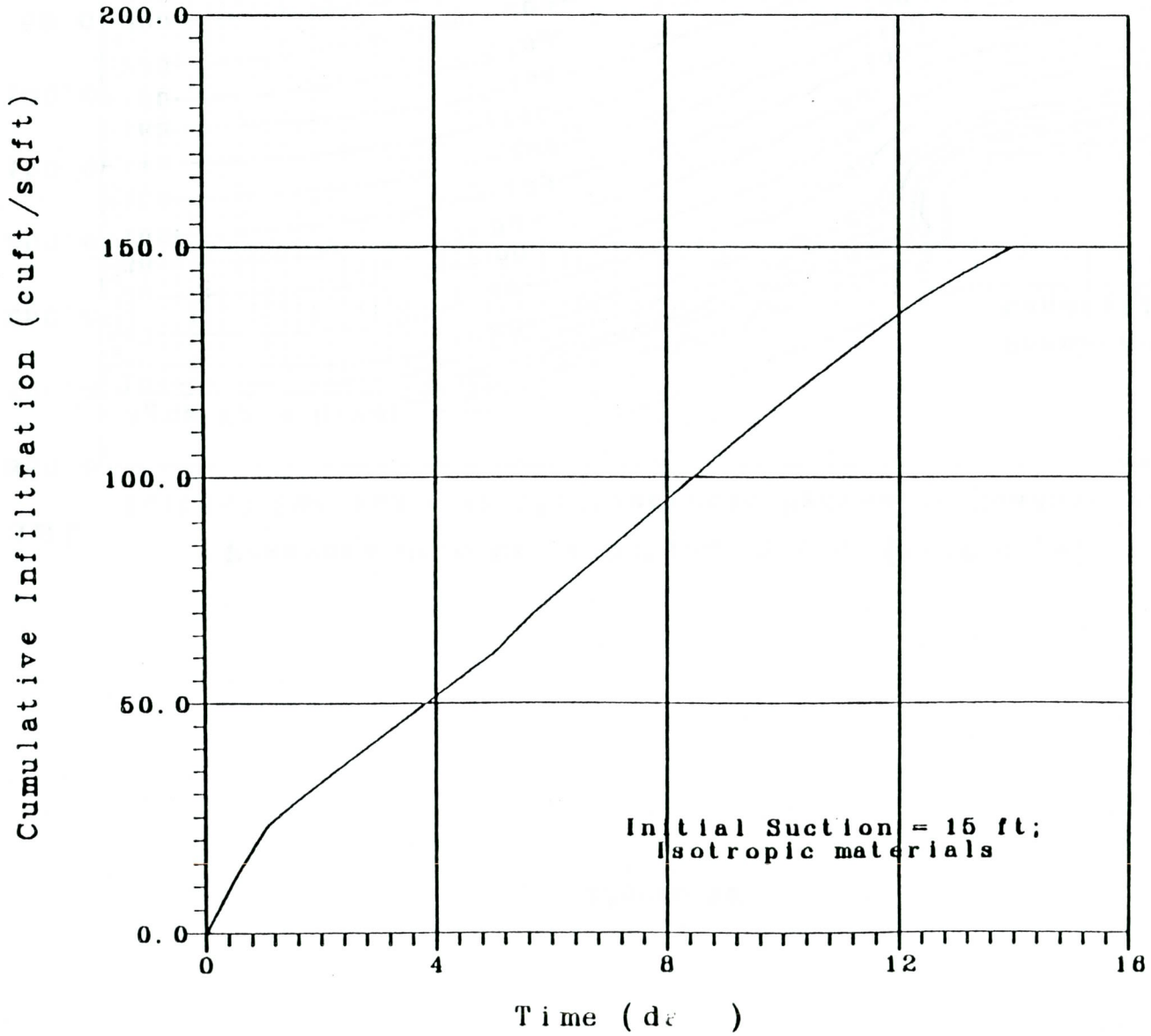

**Figure 24**

Cumulative Infiltration as a Function of Time

Section 9

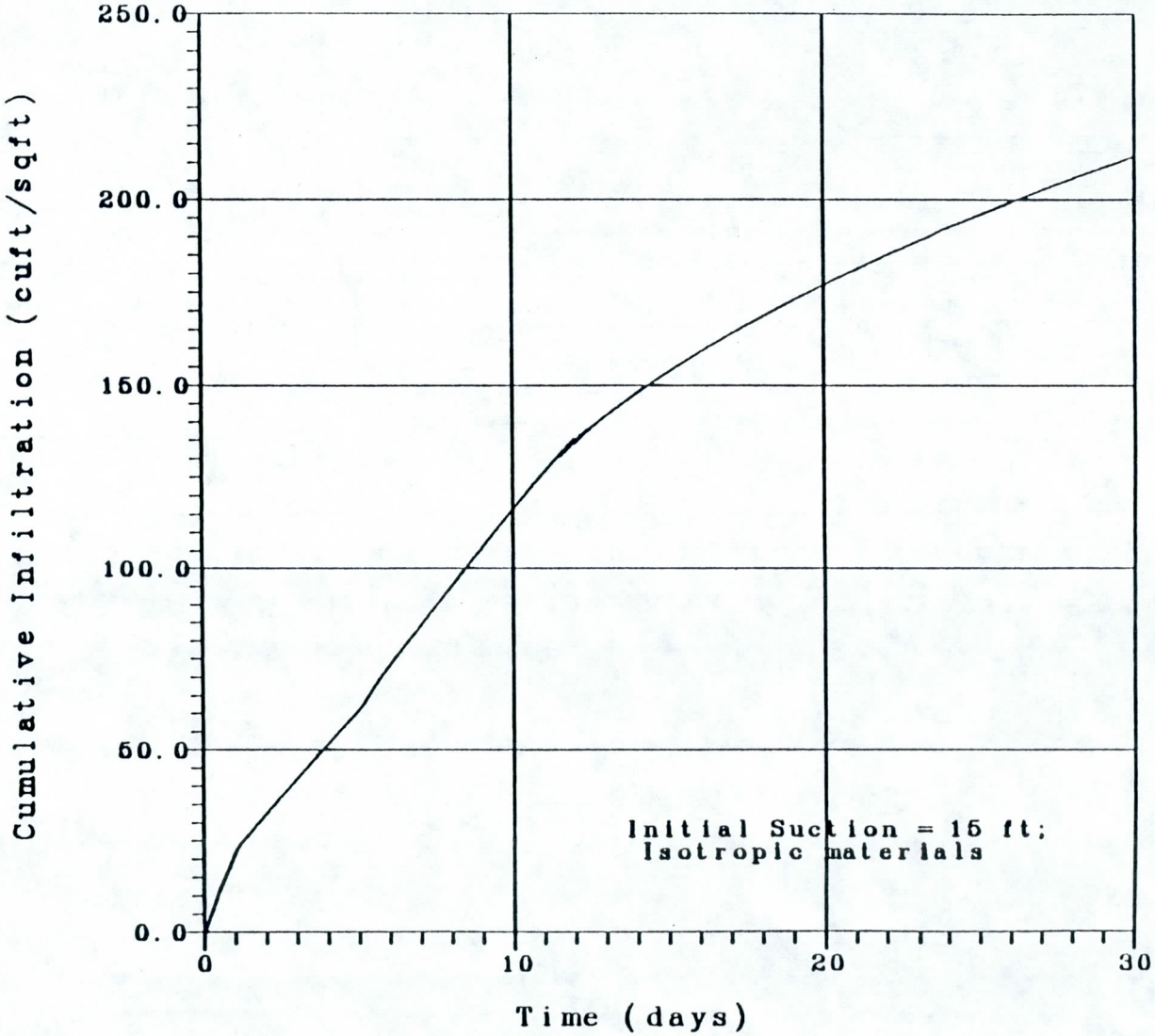




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**Figure 25**

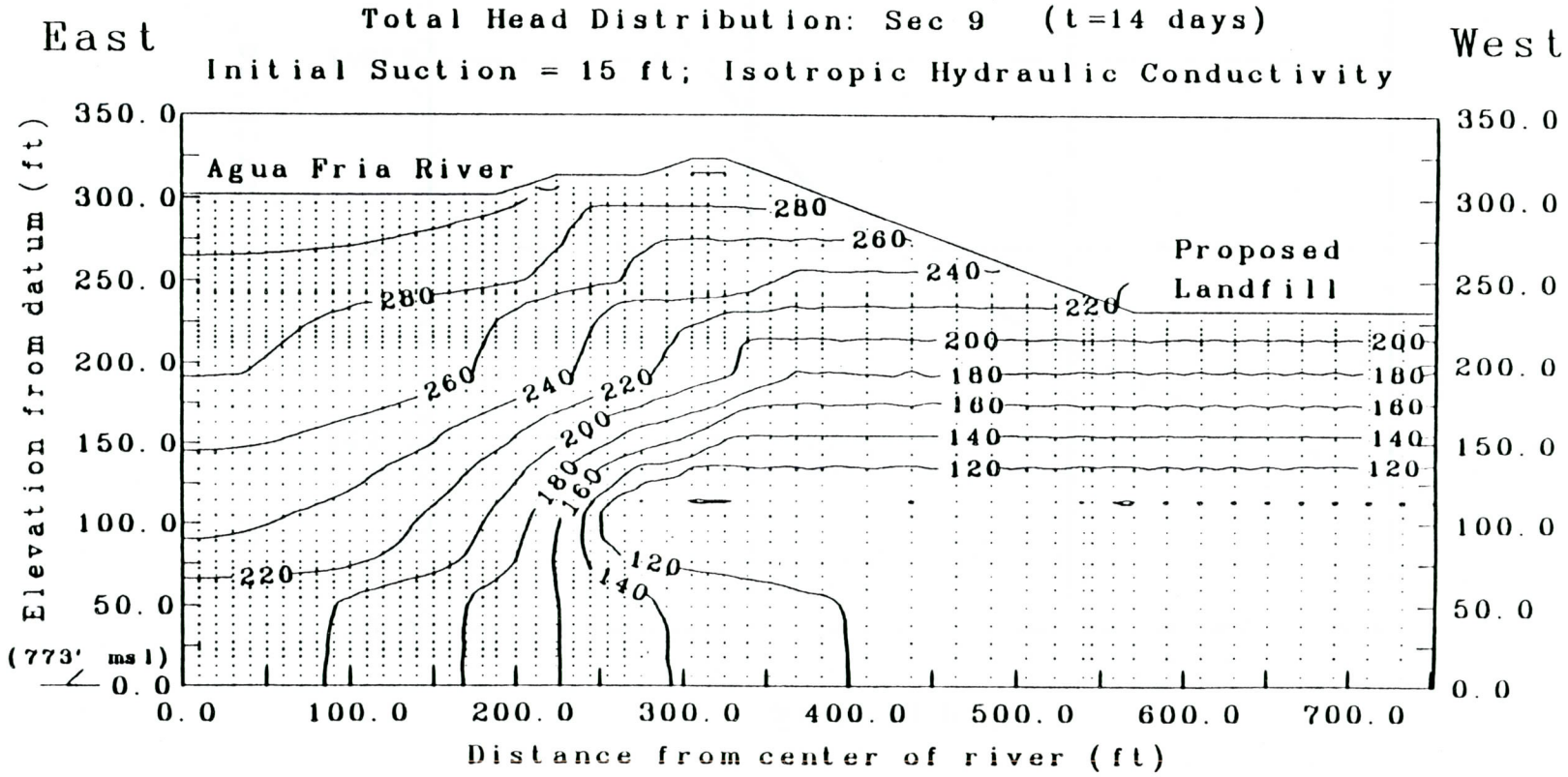
East
Total Head Distribution: Sec 9 (t=14 days)
West
Initial Suction = 15 ft; Isotropic Hydraulic Conductivity
Agua Fria River
Proposed Landfill
Elevation from datum (ft)
(773' msl)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
280
260
240
220
200
180
160
140
120
0.0
100.0
200.0
300.0
400.0
500.0
600.0
700.0
Distance from center of river (ft)

**Figure 26**

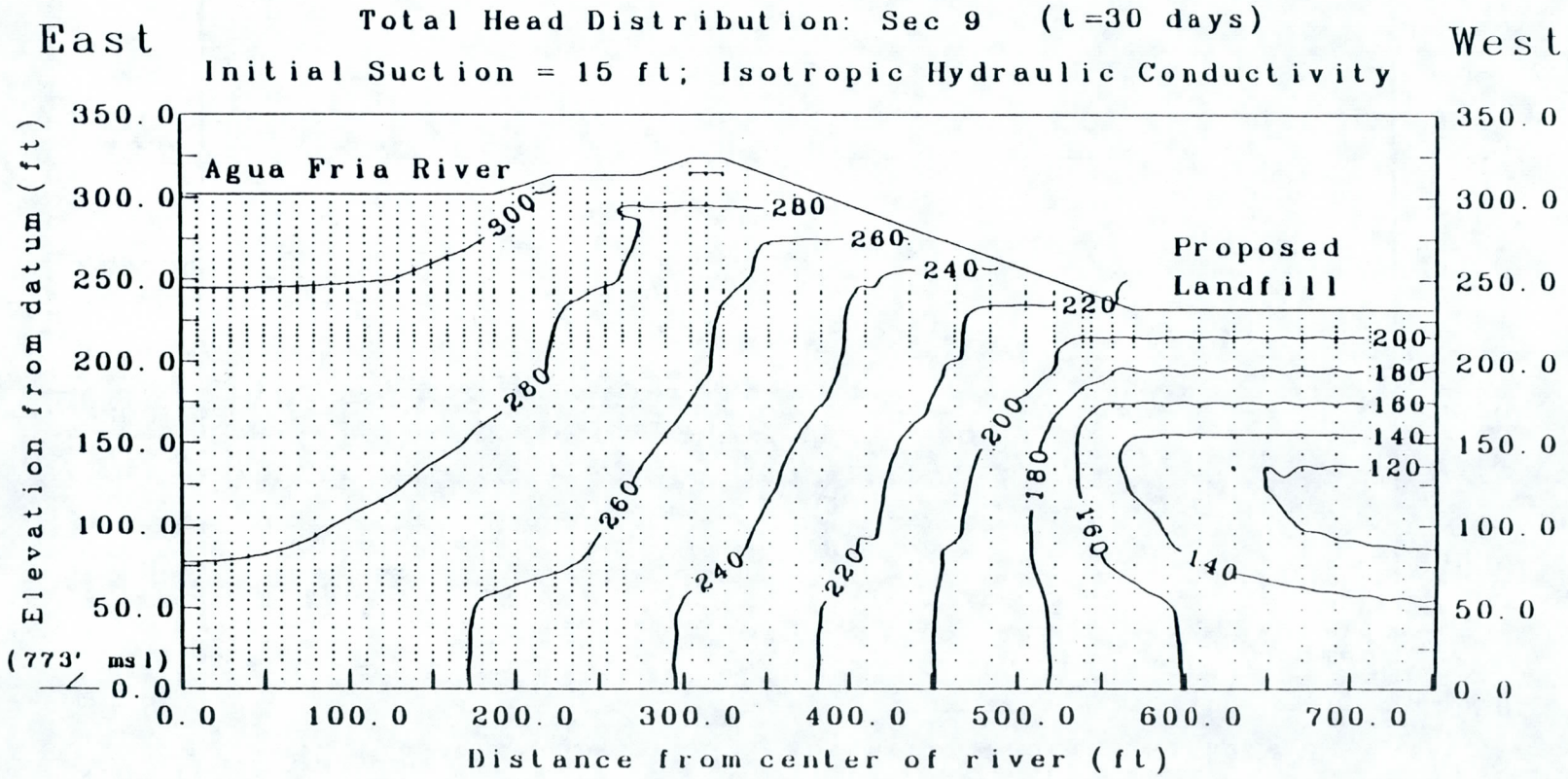




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**Figure 27**

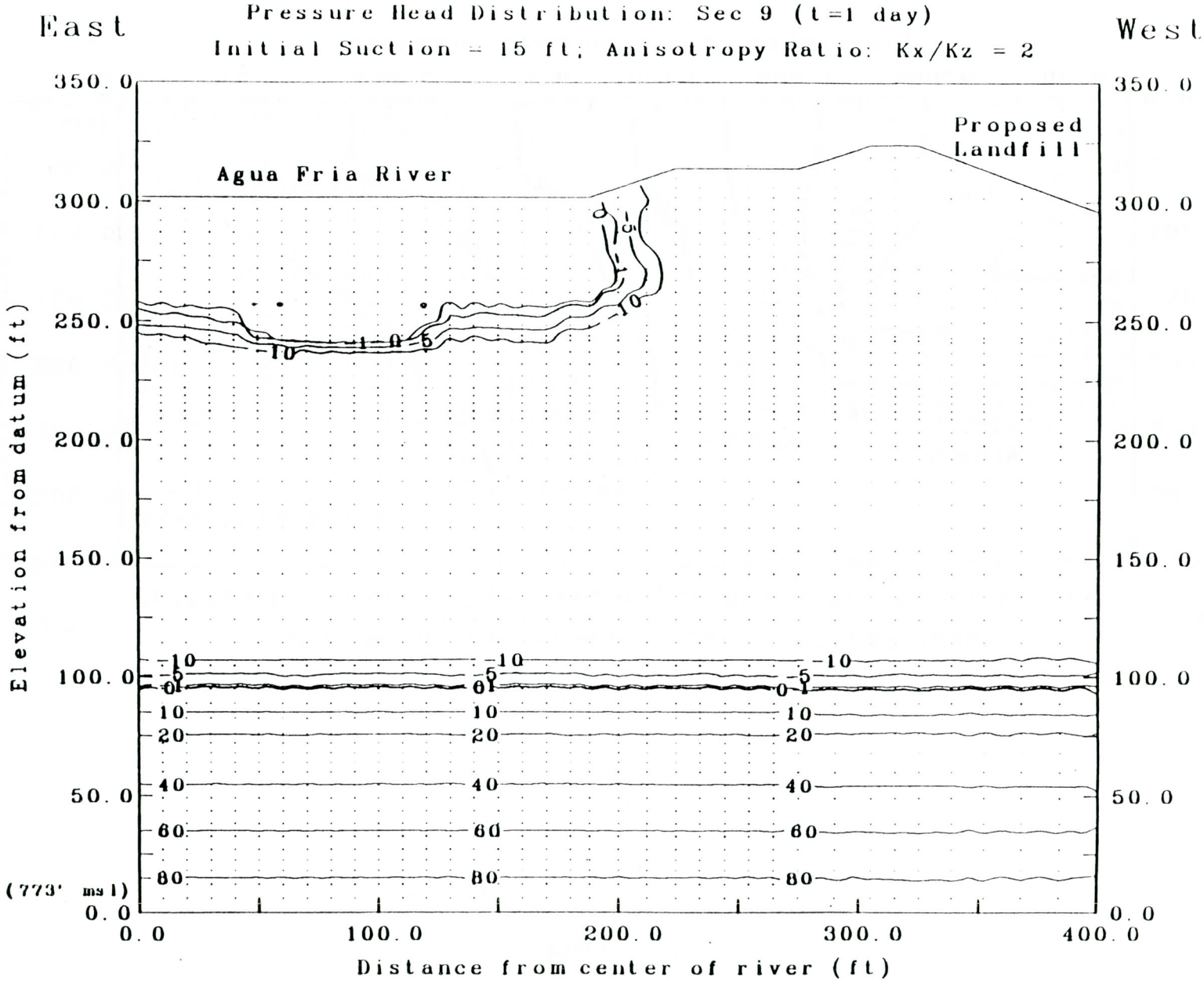
East
West
Pressure Head Distribution: Sec 9 (t=1 day)
Initial Suction = 15 ft; Anisotropy Ratio: Kx/Kz = 2
Agua Fria River
Proposed Landfill
Elevation from datum (ft)
(773' msl)
Distance from center of river (ft)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
100.0
200.0
300.0
400.0
-10
-5
-1
0
5
10
20
40
60
80

**Figure _8**

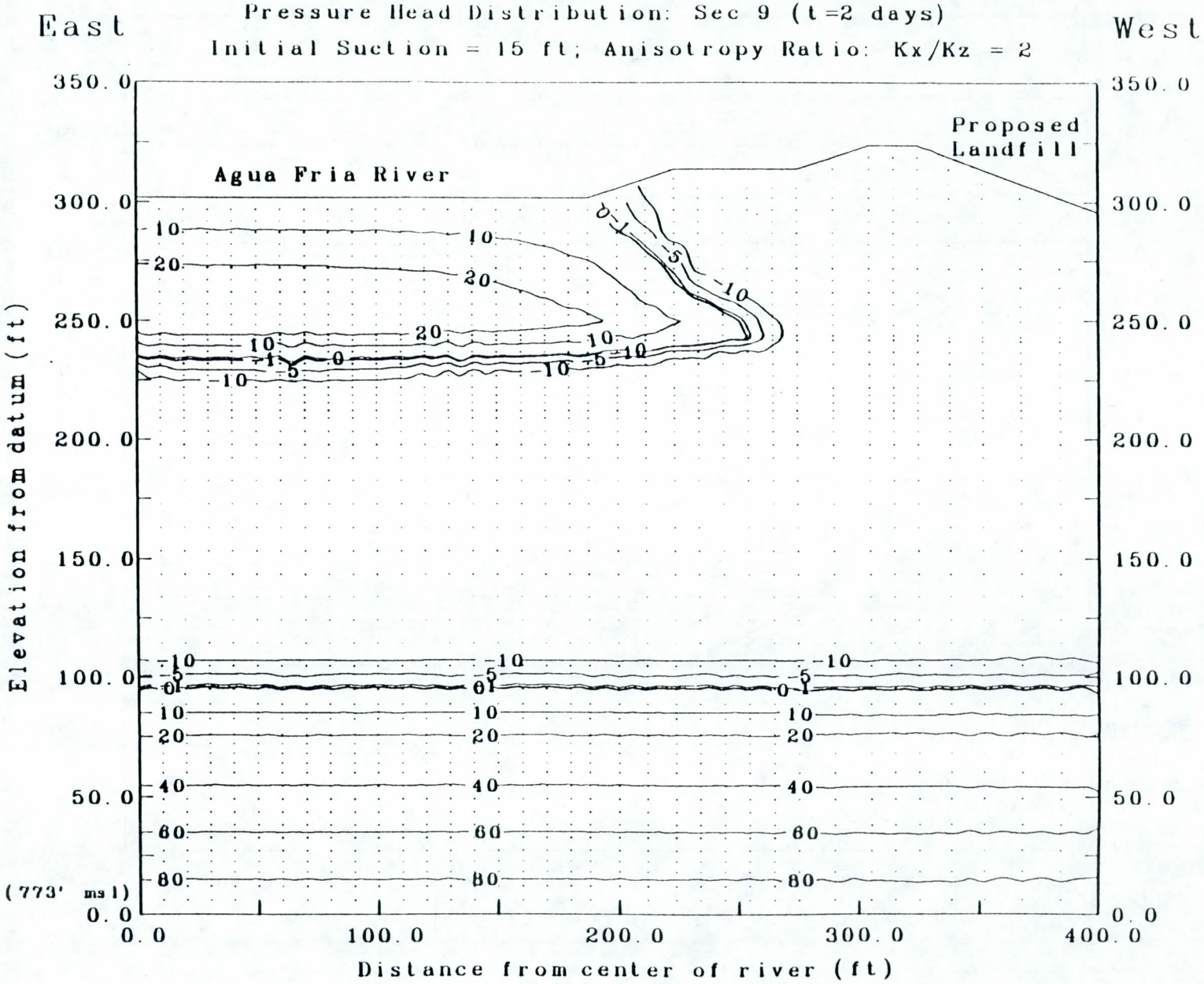




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**Figure 29**

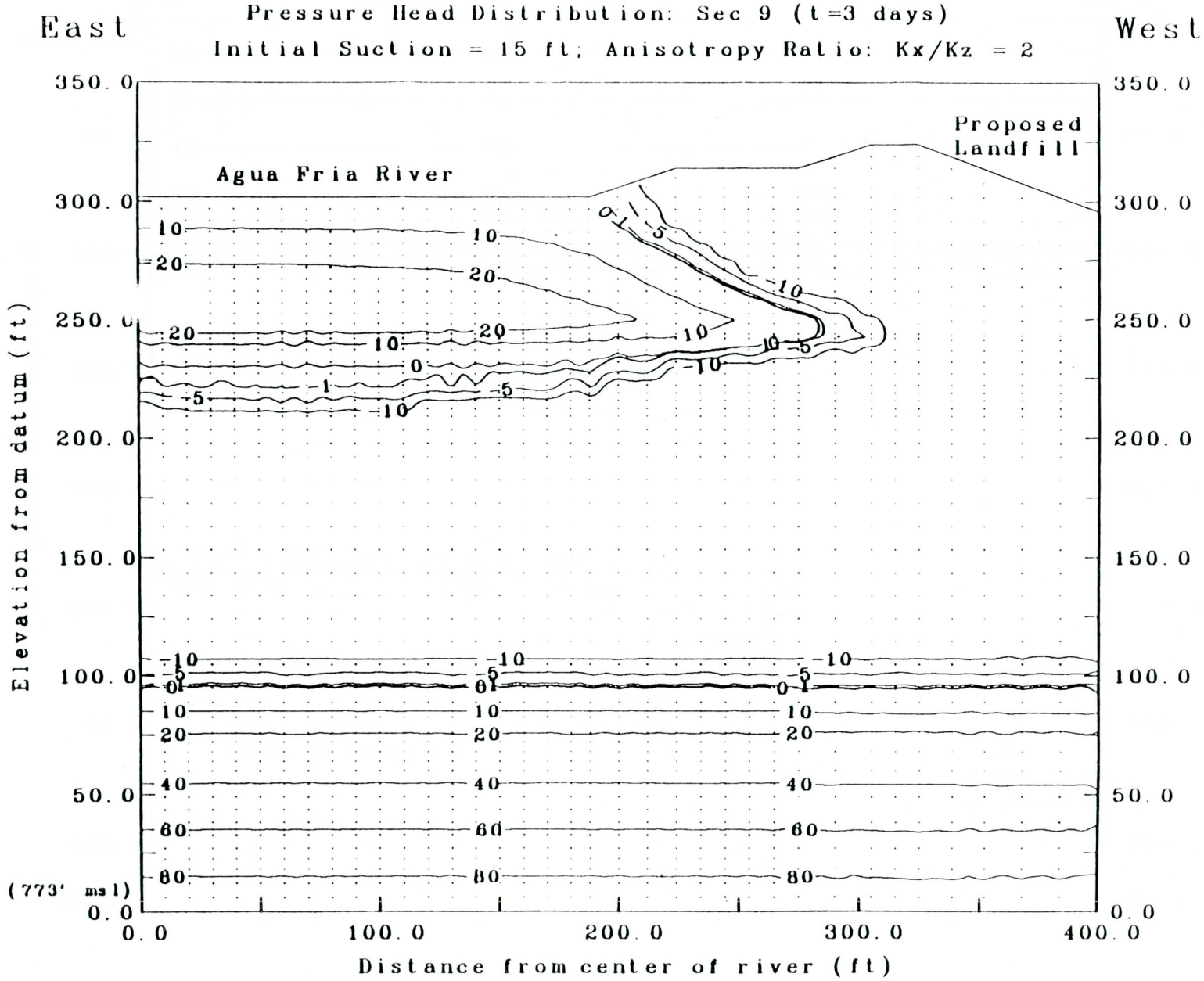
East
West
Pressure Head Distribution: Sec 9 (t=3 days)
Initial Suction = 15 ft; Anisotropy Ratio: Kx/Kz = 2
Agua Fria River
Proposed Landfill
Elevation from datum (ft)
(773' msl)
Distance from center of river (ft)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
0.0
100.0
200.0
300.0
400.0

**Figure 30**

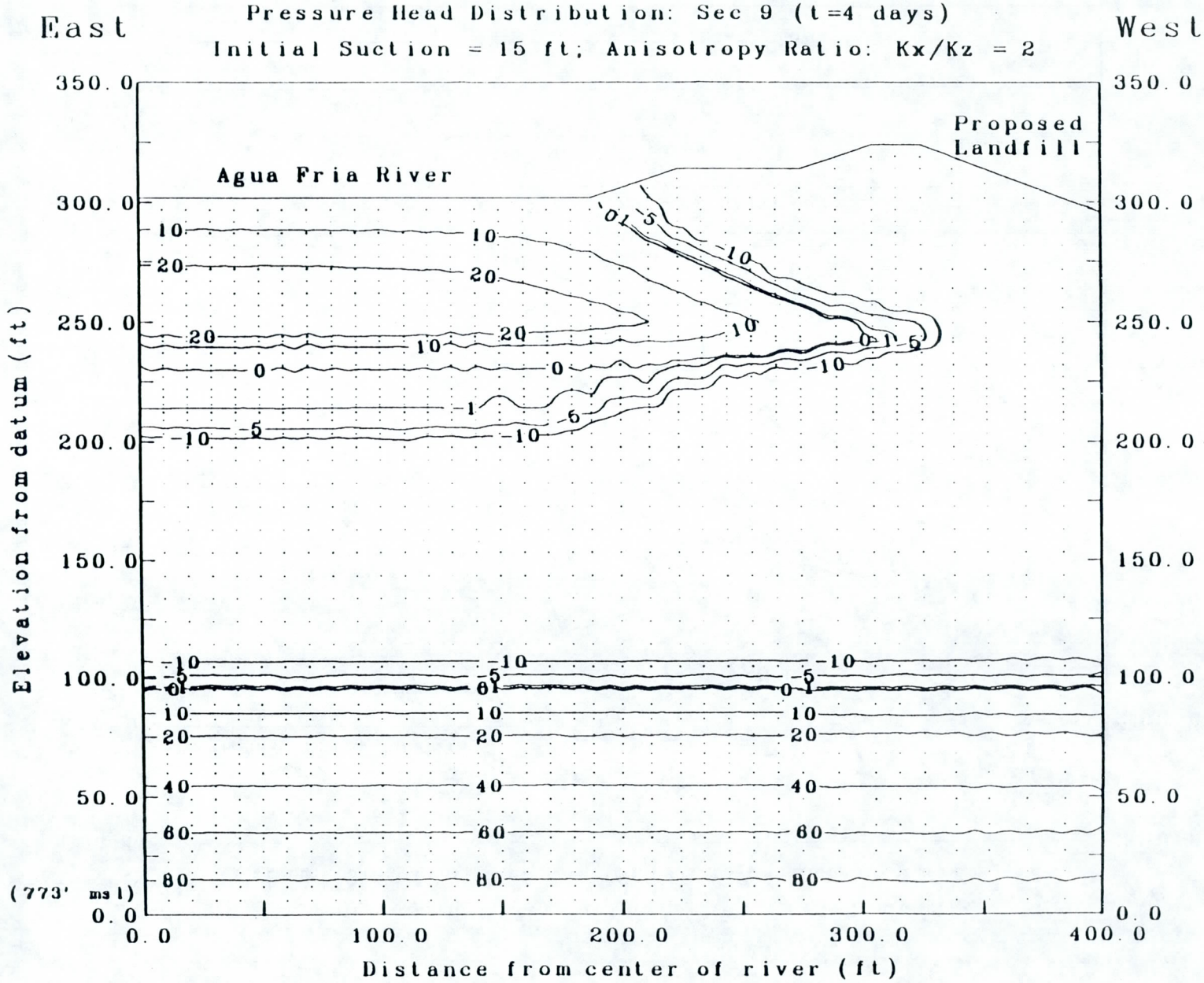




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**Figure 31**

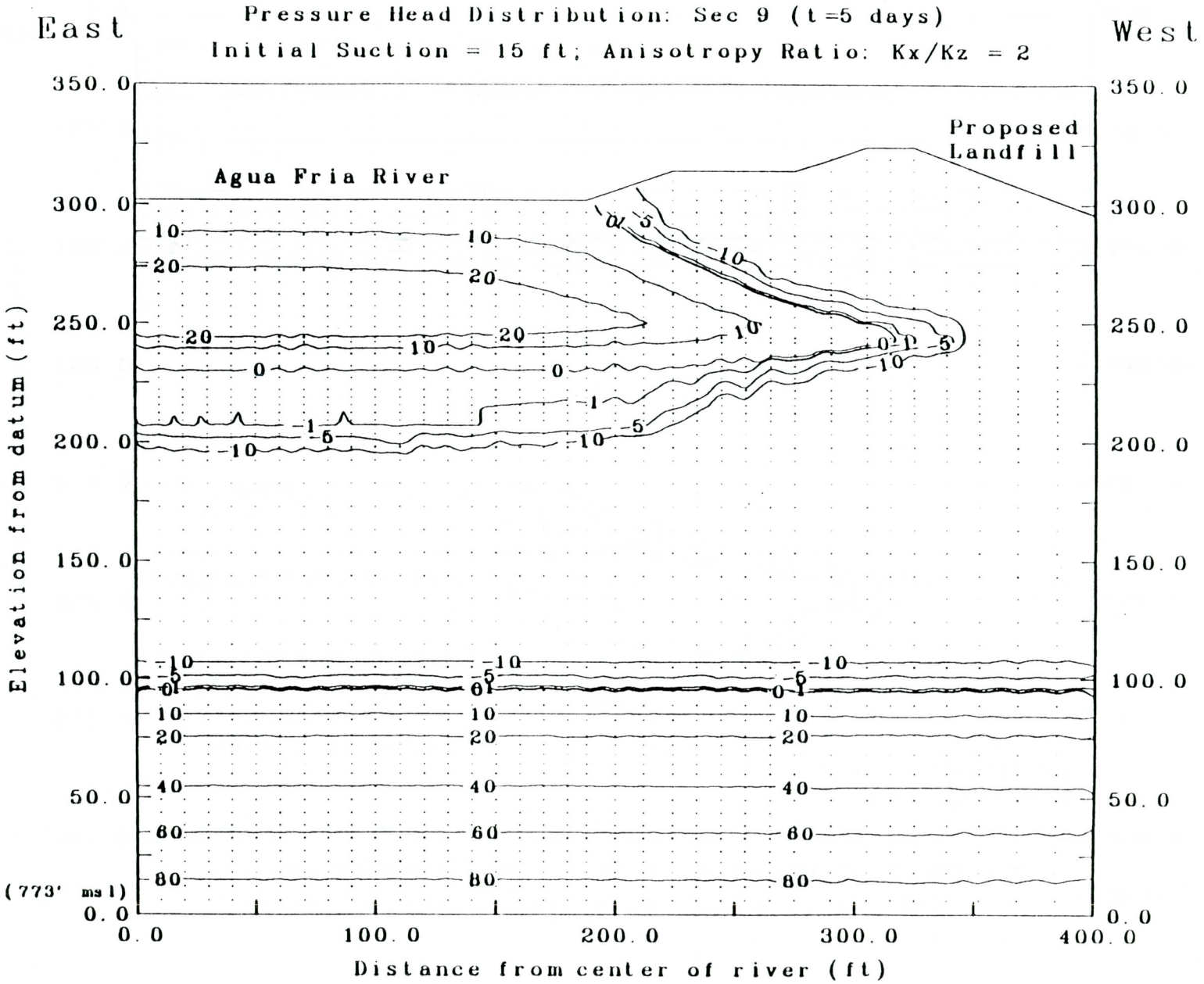
East
West
Pressure Head Distribution: Sec 9 (t=5 days)
Initial Suction = 15 ft; Anisotropy Ratio: Kx/Kz = 2
Agua Fria River
Proposed Landfill
Elevation from datum (ft)
(773' msl)
Distance from center of river (ft)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
0.0
100.0
200.0
300.0
400.0

Figure 12

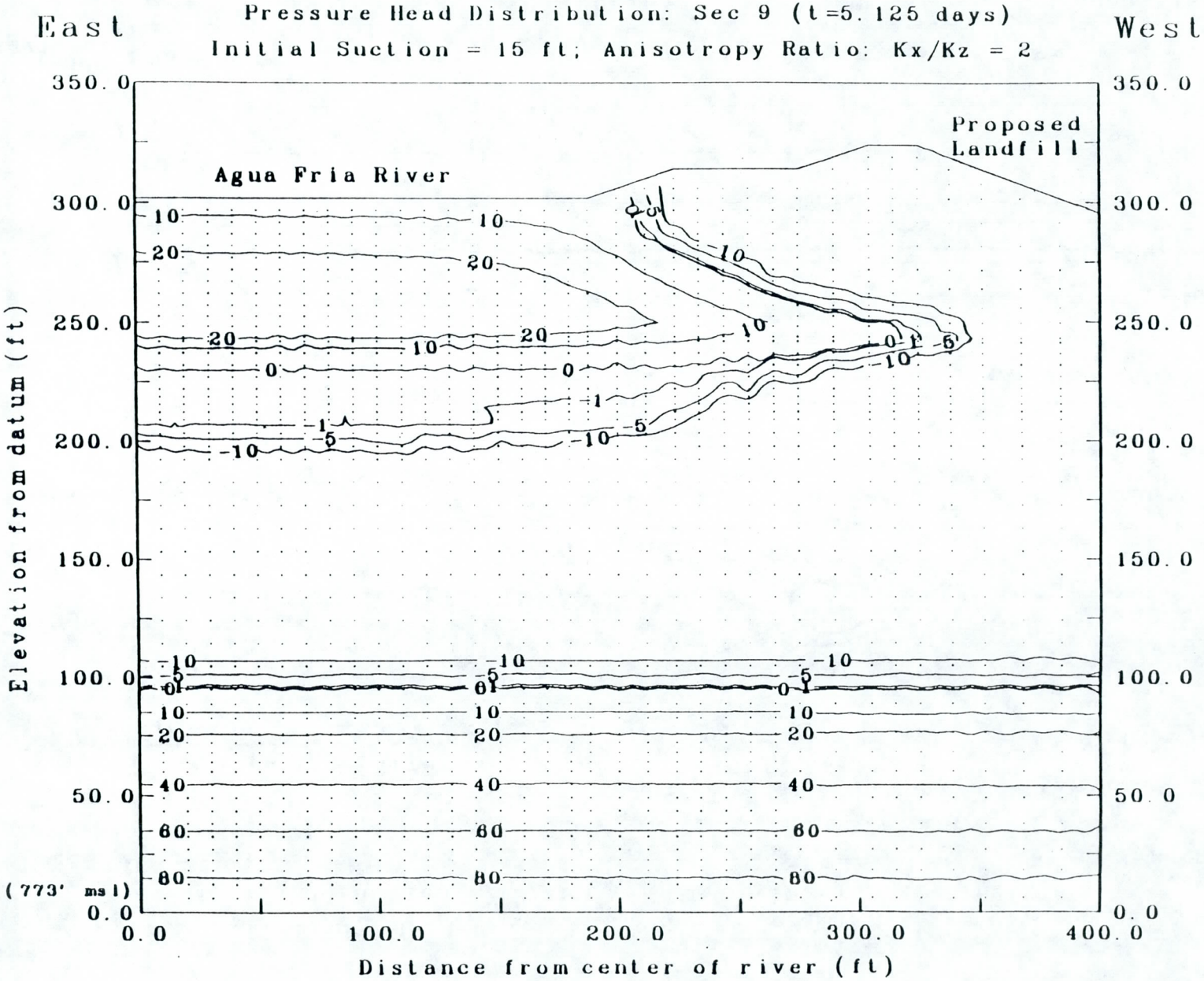




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**Figure 33**

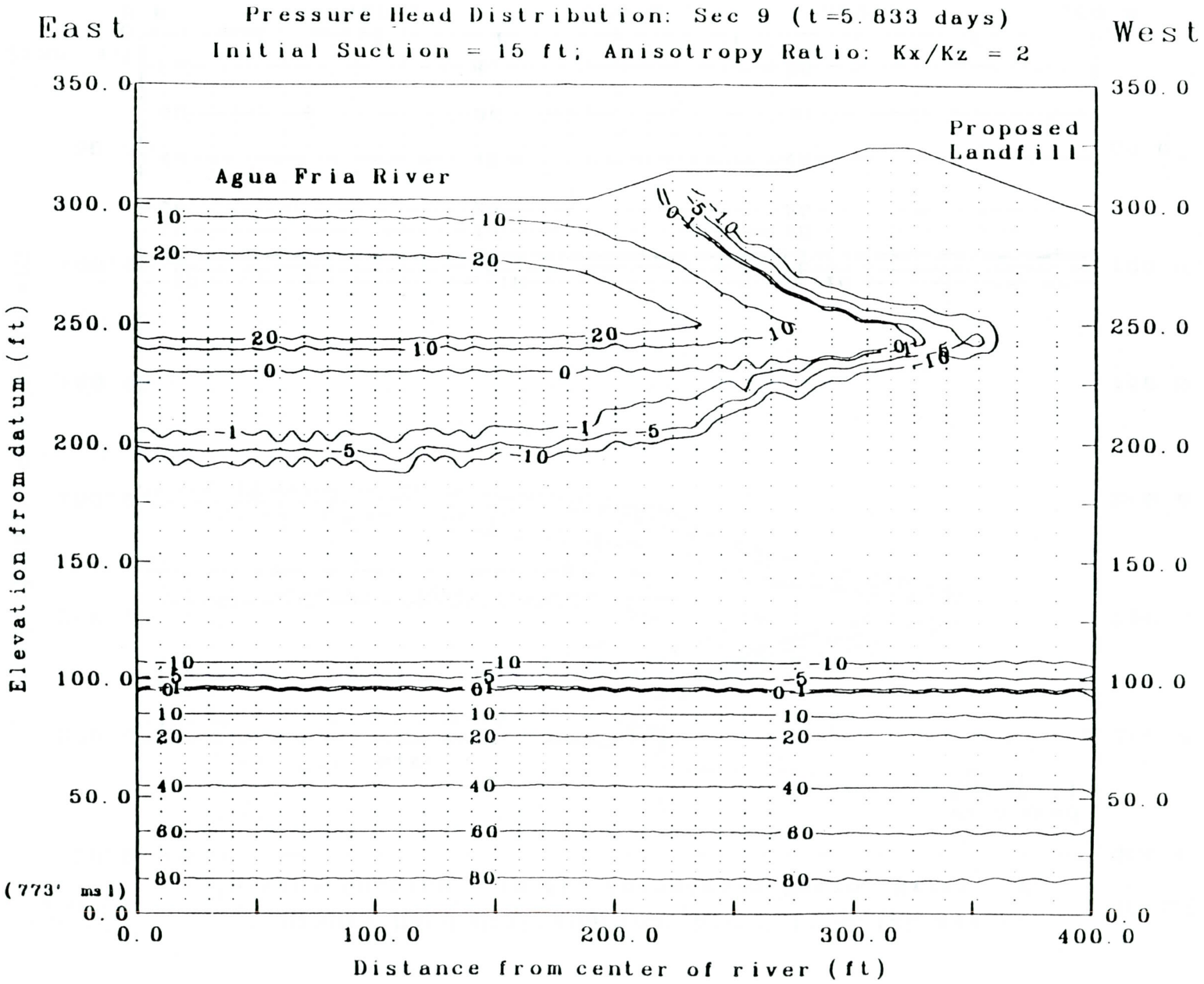
East
West
Pressure Head Distribution: Sec 9 (t=5.833 days)
Initial Suction = 15 ft; Anisotropy Ratio: Kx/Kz = 2
Agua Fria River
Proposed Landfill
Elevation from datum (ft)
(773' msl)
Distance from center of river (ft)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
100.0
200.0
300.0
400.0

**Figure 34**

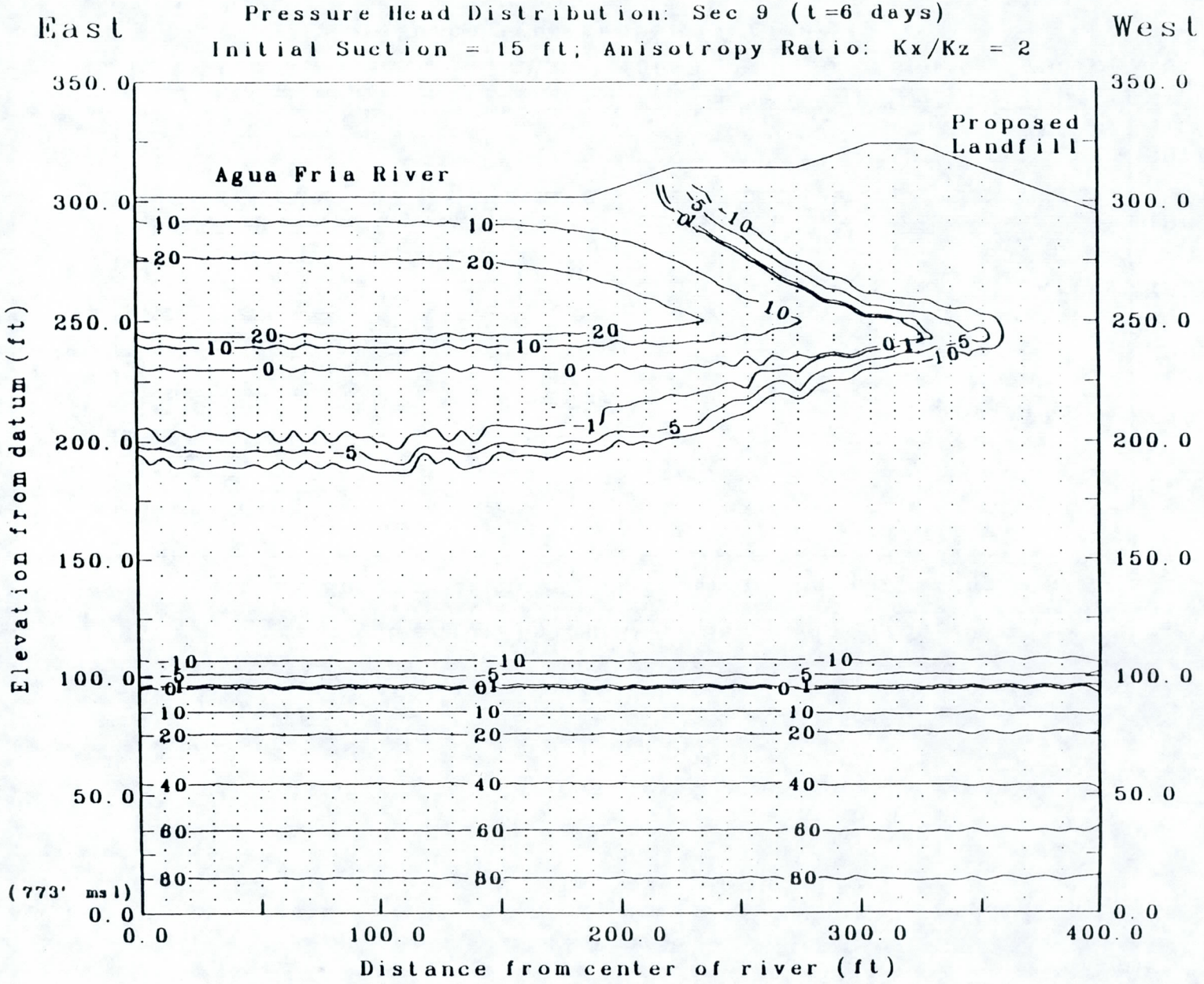




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**Figure 35**

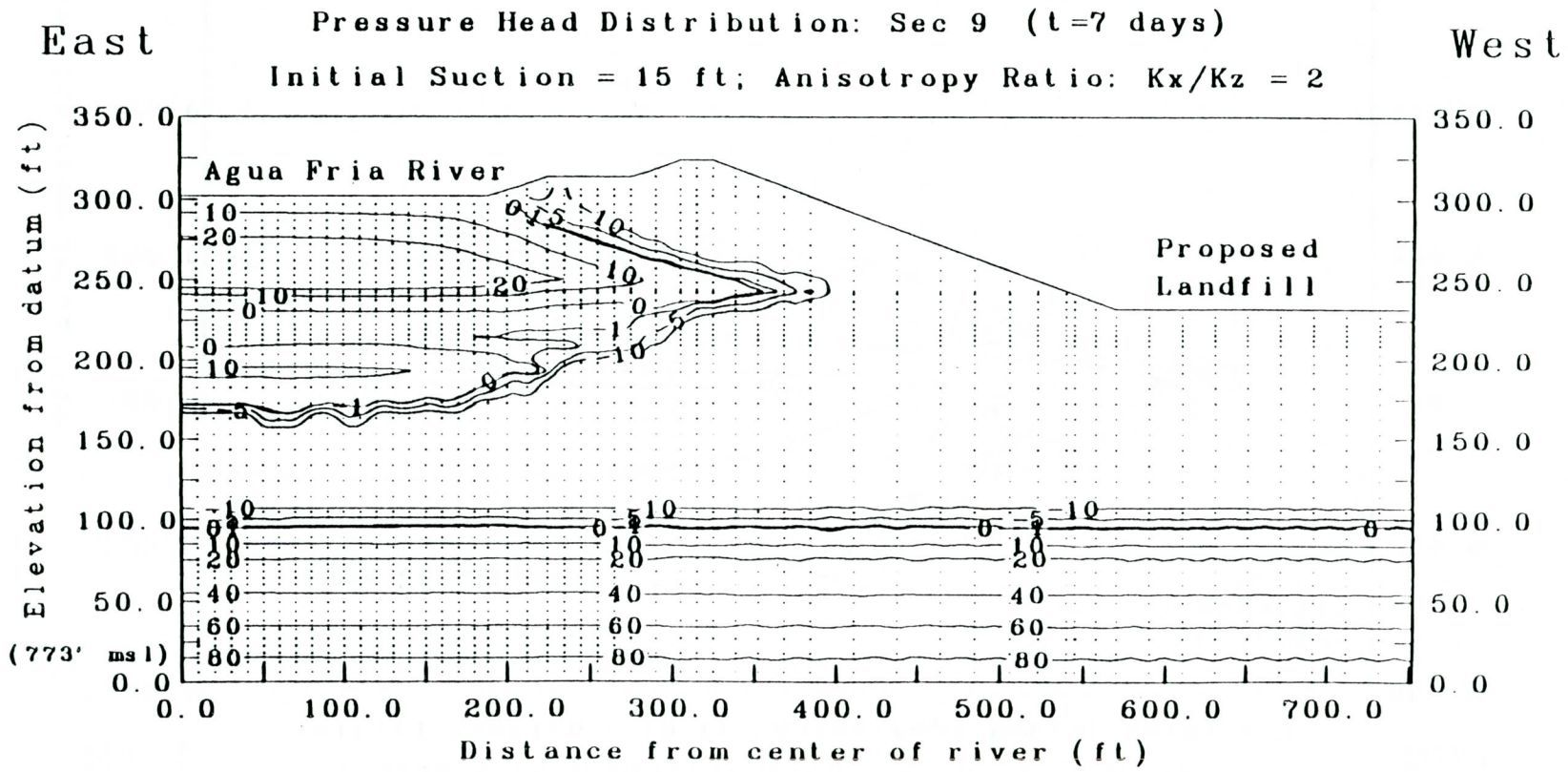
East
Pressure Head Distribution: Sec 9 (t=7 days)
West
Initial Suction = 15 ft; Anisotropy Ratio: Kx/Kz = 2
Agua Fria River
Proposed Landfill
Elevation from datum (ft)
(773' msl)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
0.0
100.0
200.0
300.0
400.0
500.0
600.0
700.0
Distance from center of river (ft)

**Figure 36**

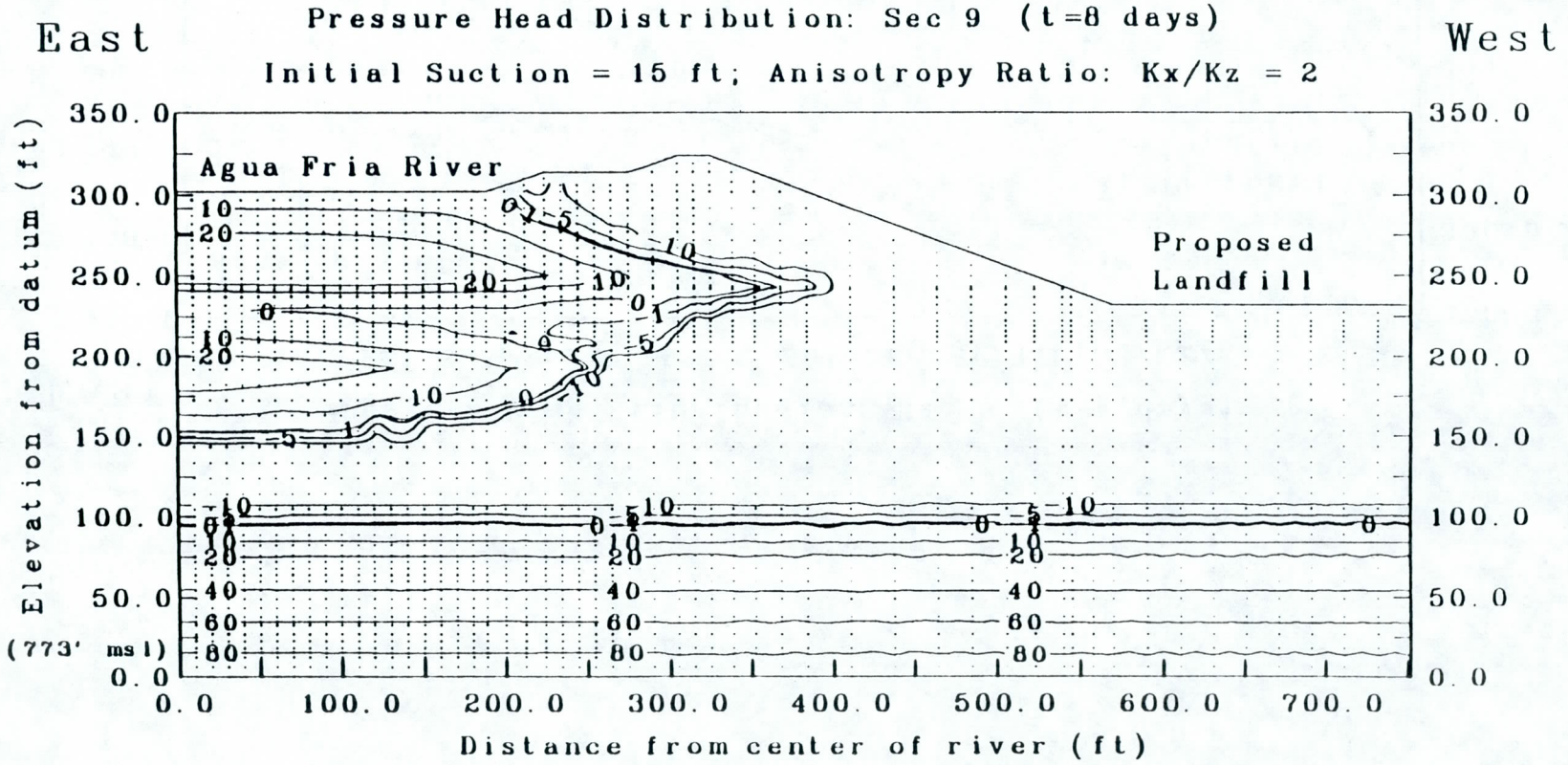




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**Figure 37**

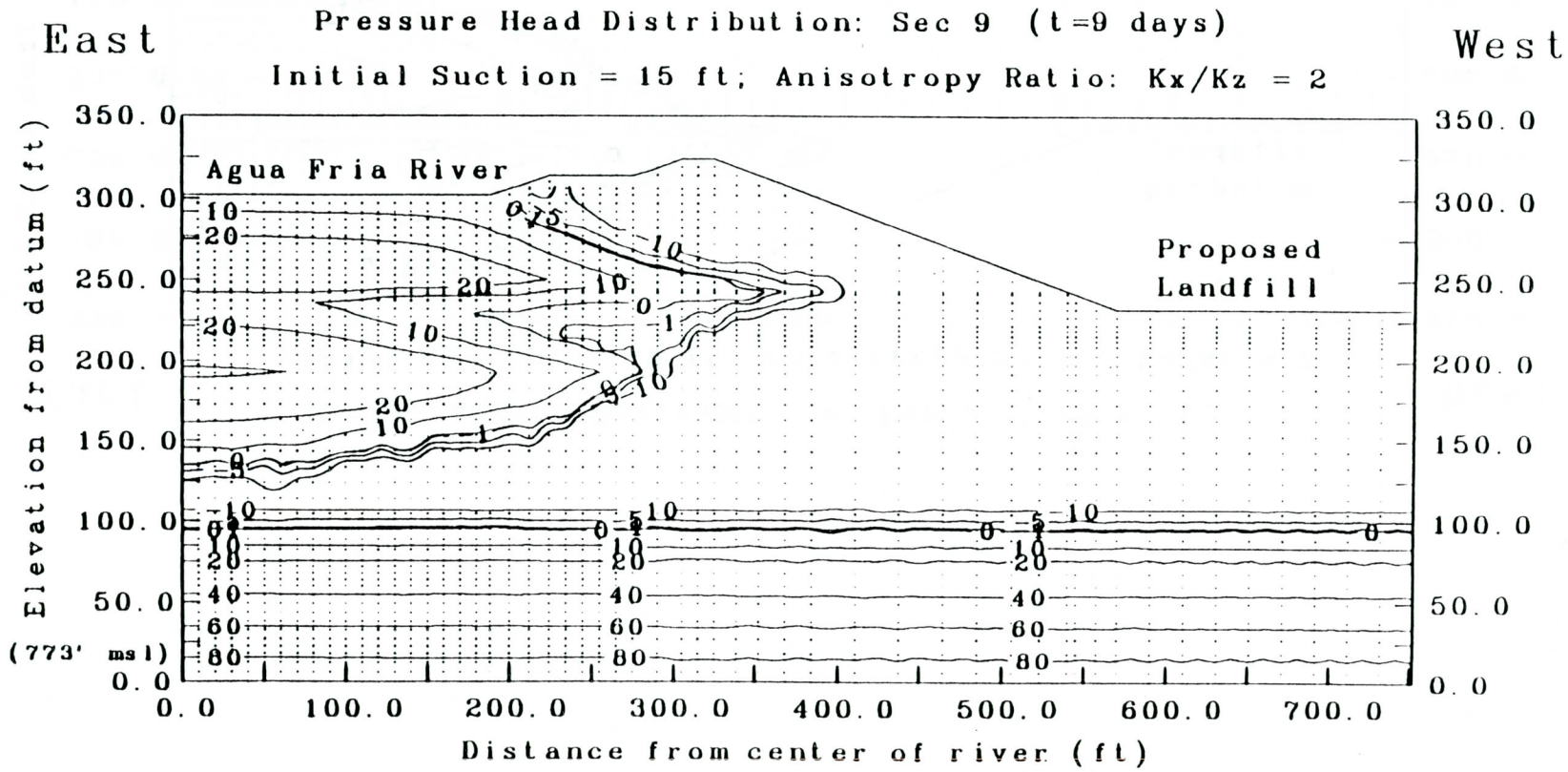
East
Pressure Head Distribution: Sec 9 (t=9 days)
West
Initial Suction = 15 ft; Anisotropy Ratio: Kx/Kz = 2
Agua Fria River
Proposed
Landfill
Elevation from datum (ft)
(773' msl)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
0.0
100.0
200.0
300.0
400.0
500.0
600.0
700.0
Distance from center of river (ft)

**Figure 38**

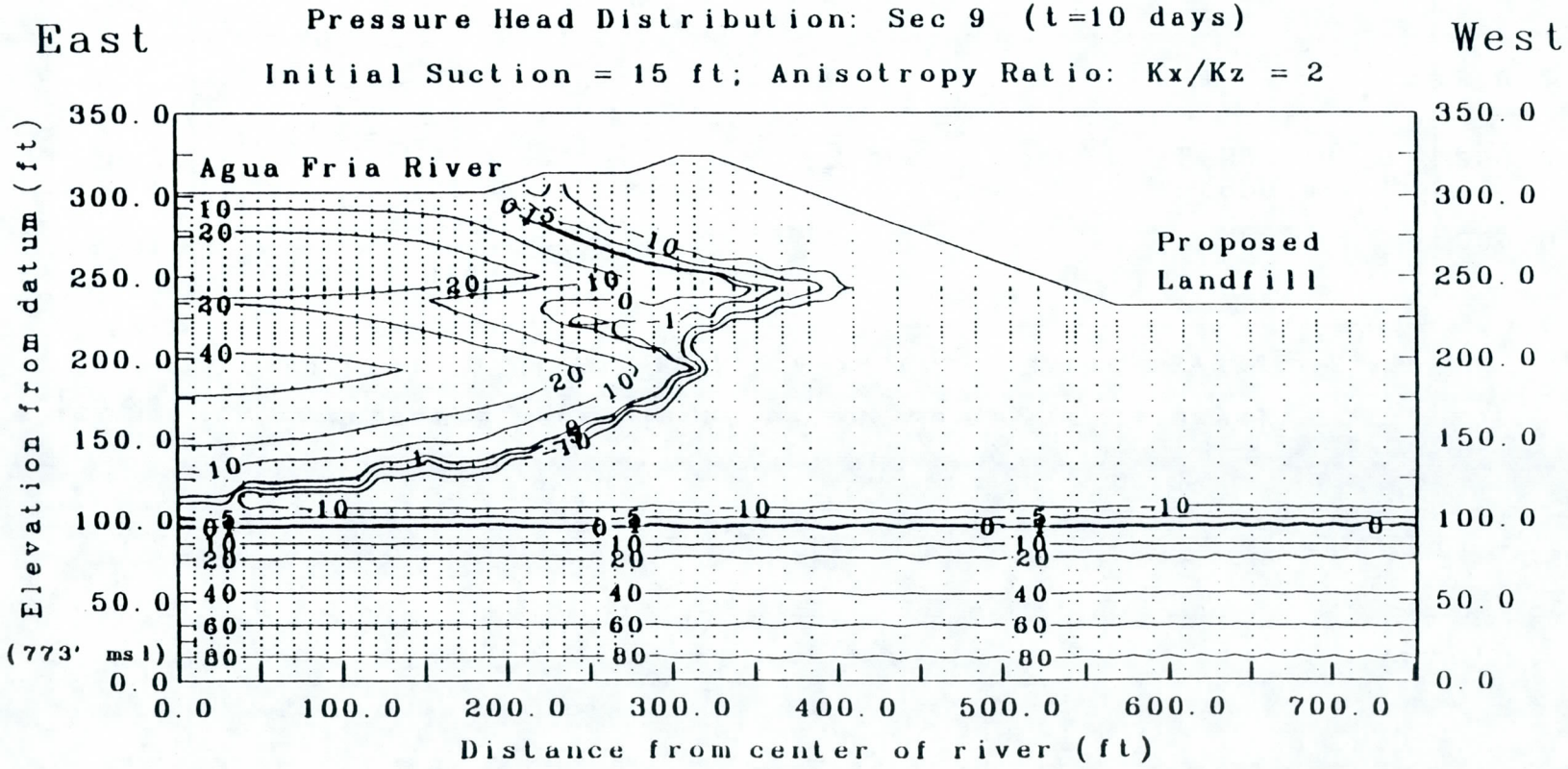




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**Figure 39**

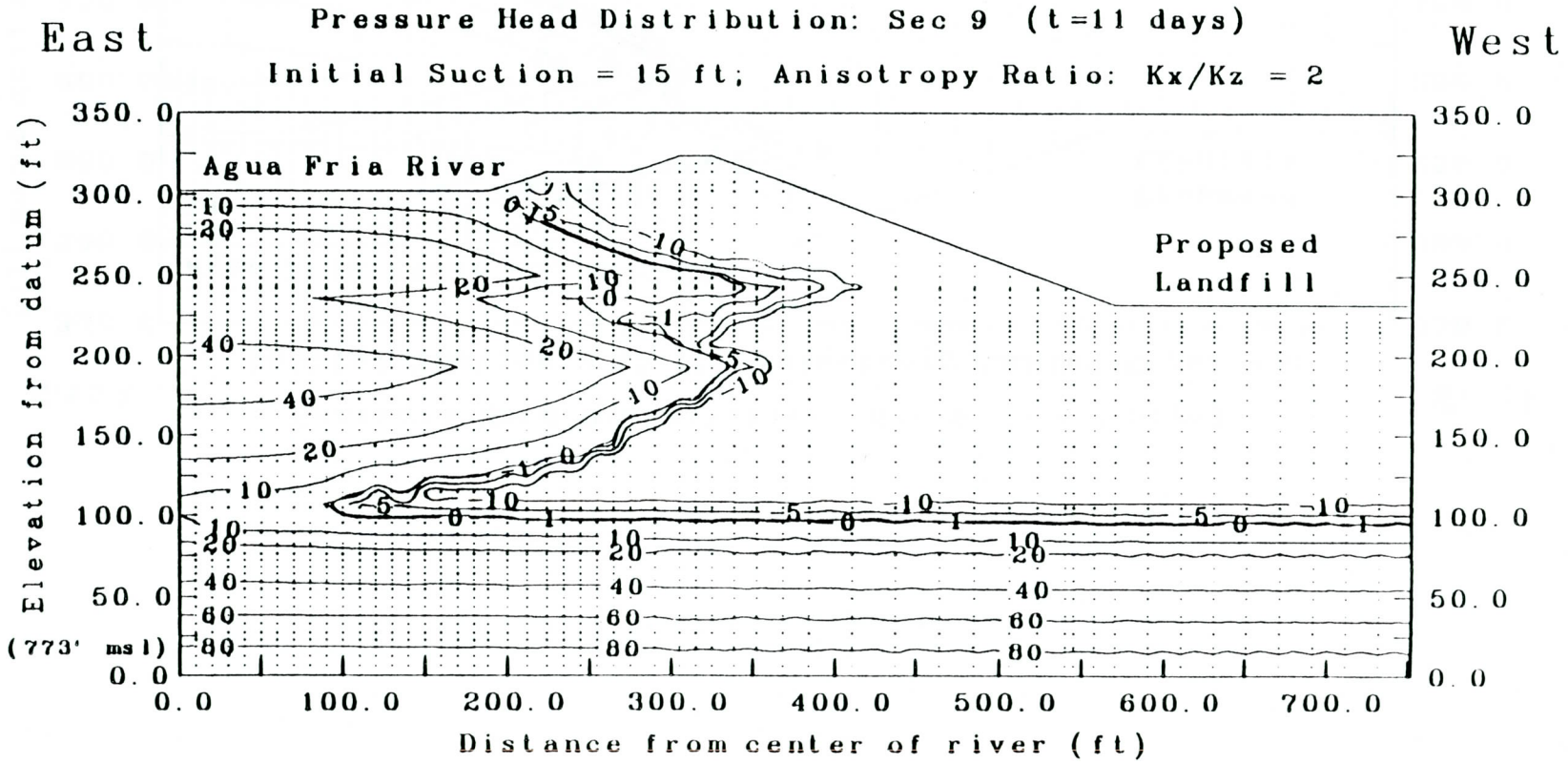
East
Pressure Head Distribution: Sec 9 (t=11 days)
West
Initial Suction = 15 ft; Anisotropy Ratio: Kx/Kz = 2
Agua Fria River
Proposed
Landfill
Elevation from datum (ft)
(773' msl)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
0.0
100.0
200.0
300.0
400.0
500.0
600.0
700.0
Distance from center of river (ft)

Figure 40

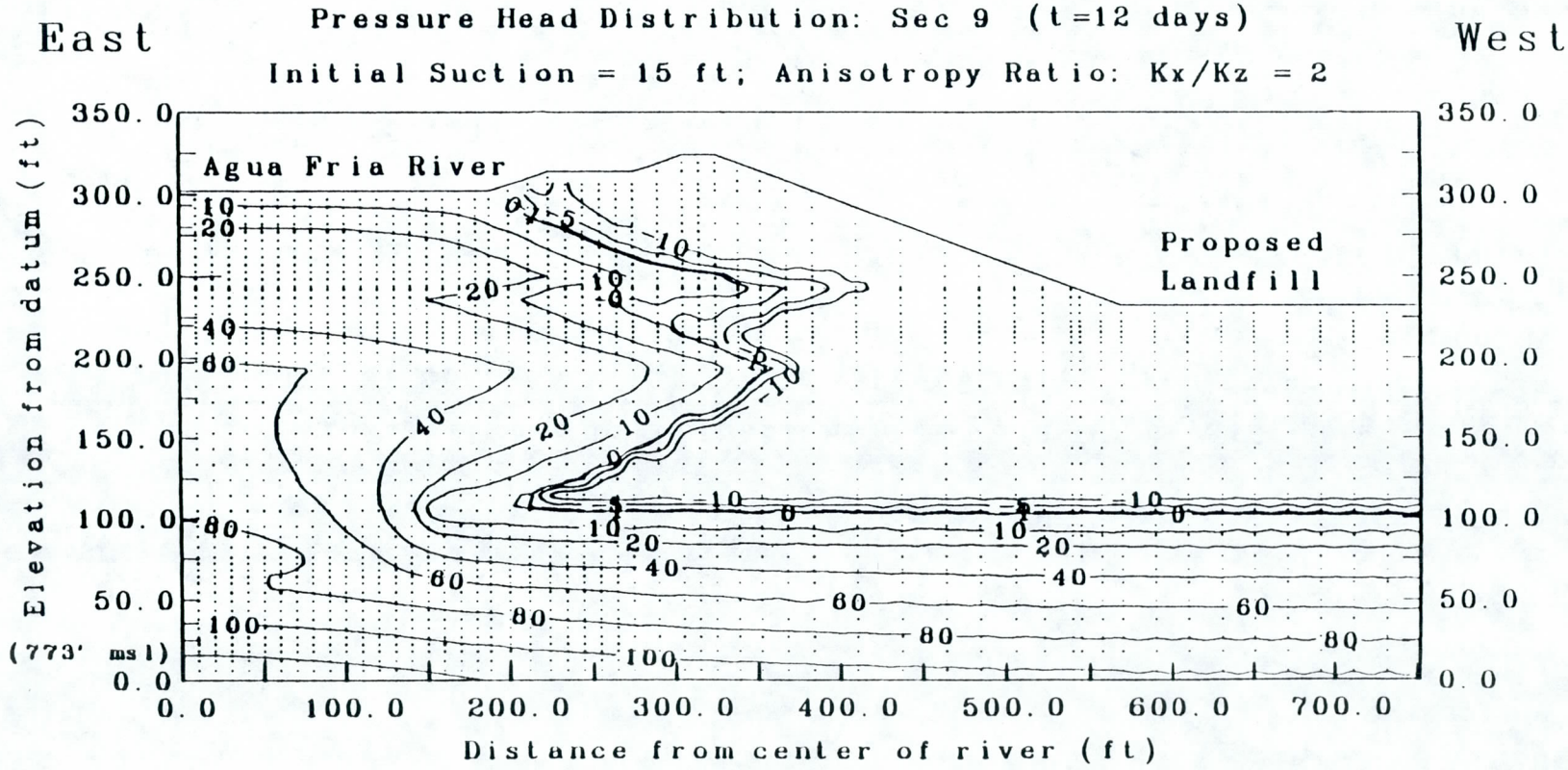




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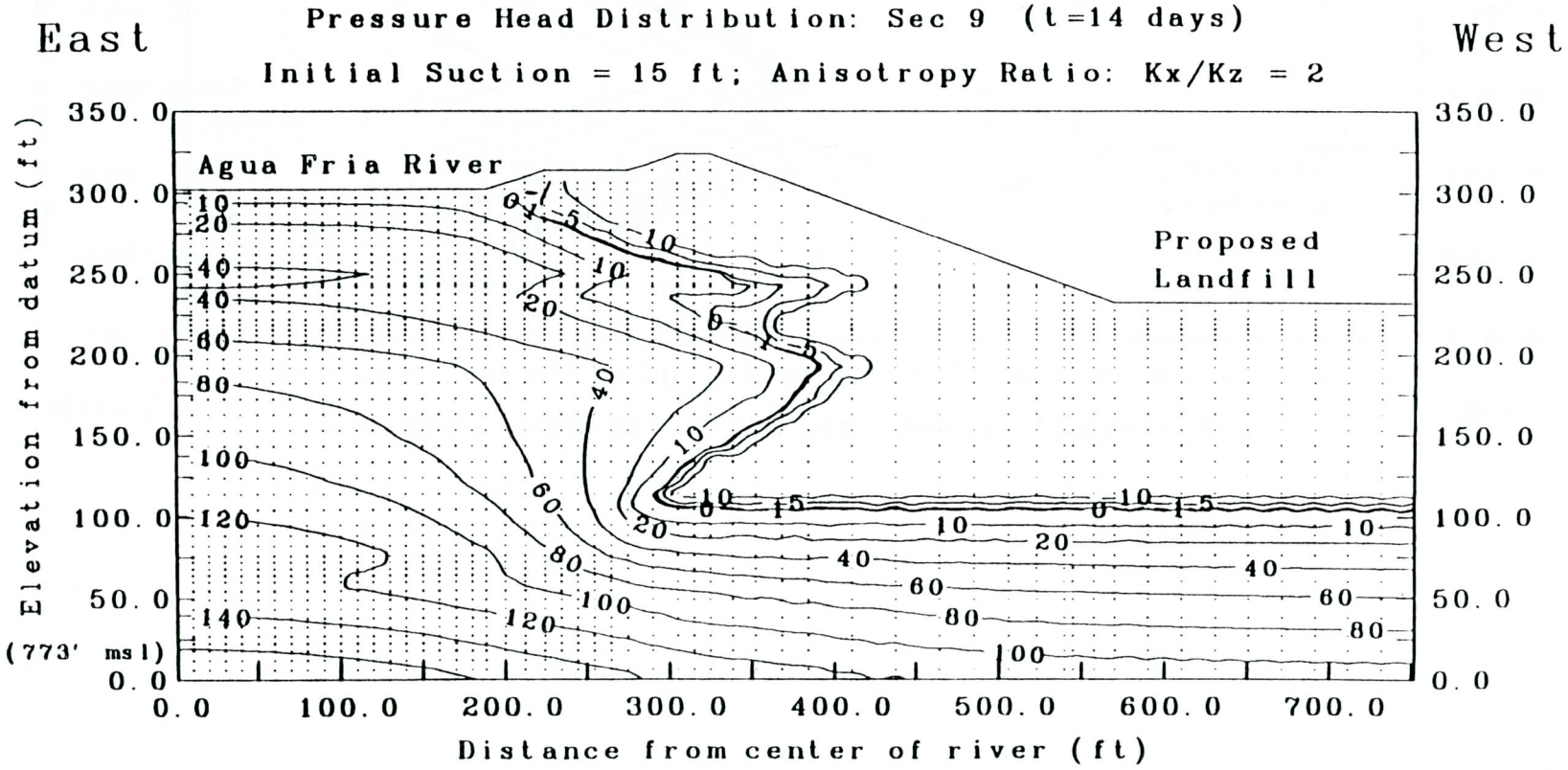
**Figure 41**


East
Pressure Head Distribution: Sec 9 (t=14 days)
West
Initial Suction = 15 ft; Anisotropy Ratio: Kx/Kz = 2
Agua Fria River
Proposed
Landfill
Elevation from datum (ft)
(773' msl)
Distance from center of river (ft)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
100.0
200.0
300.0
400.0
500.0
600.0
700.0

Figur 2

Cumulative Infiltration as a Function of Time

Section 9

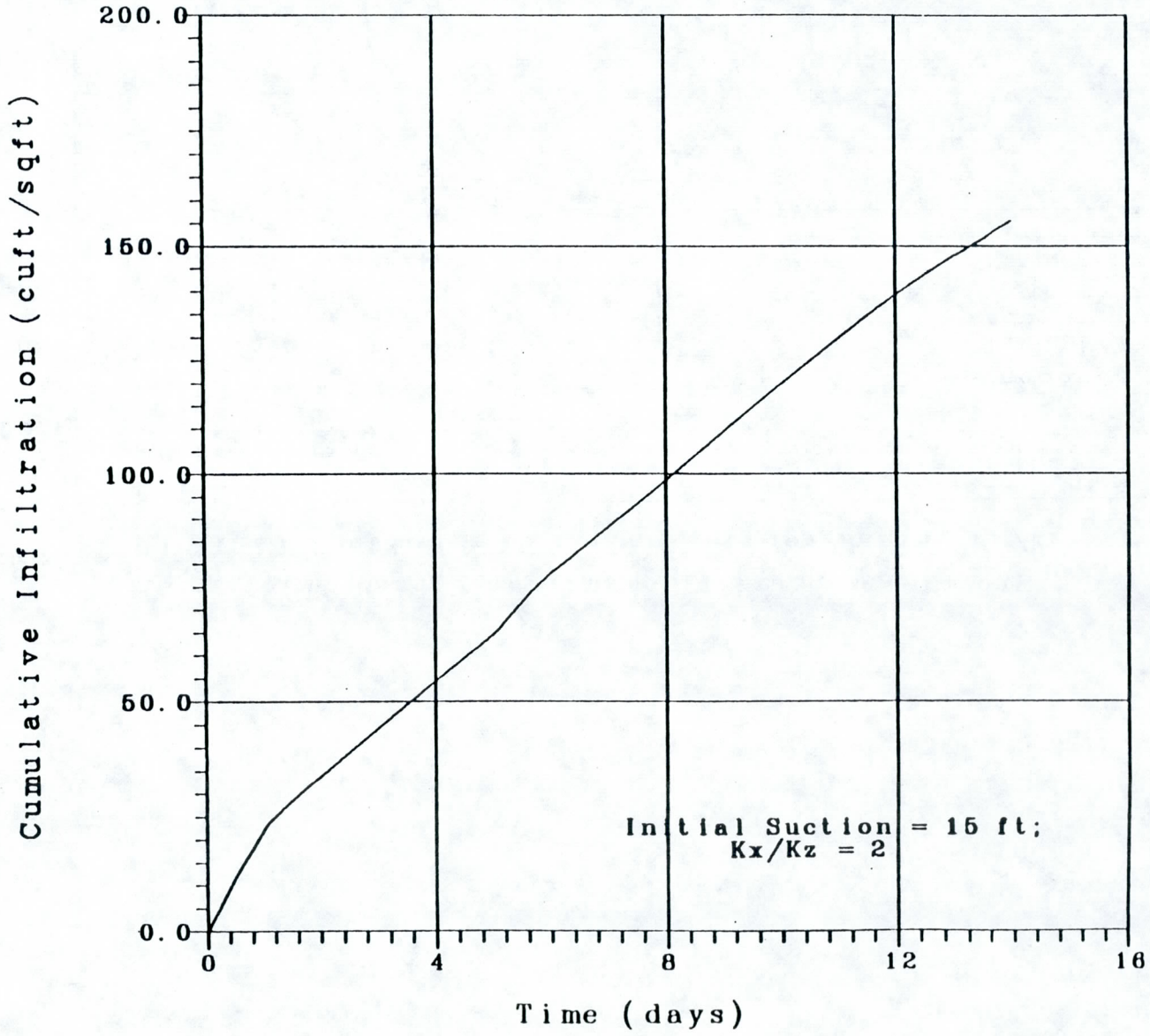





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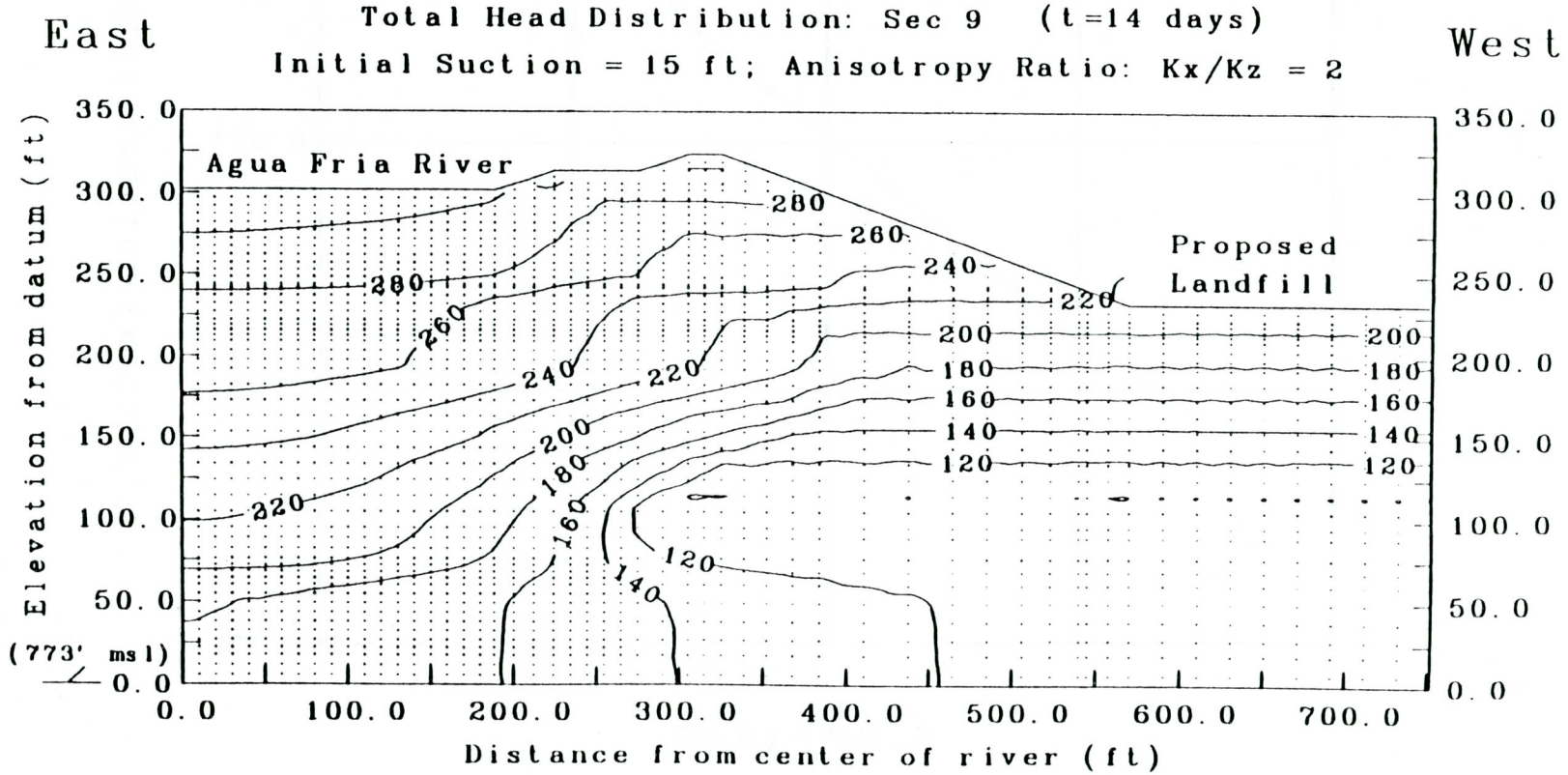
**Figure 43**


East
Total Head Distribution: Sec 9 (t=14 days)
Initial Suction = 15 ft; Anisotropy Ratio: Kx/Kz = 2
West
Agua Fria River
Proposed Landfill
Elevation from datum (ft)
(773' msl)
Distance from center of river (ft)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
100.0
200.0
300.0
400.0
500.0
600.0
700.0
280
260
240
220
200
180
160
140
120

Figur 4

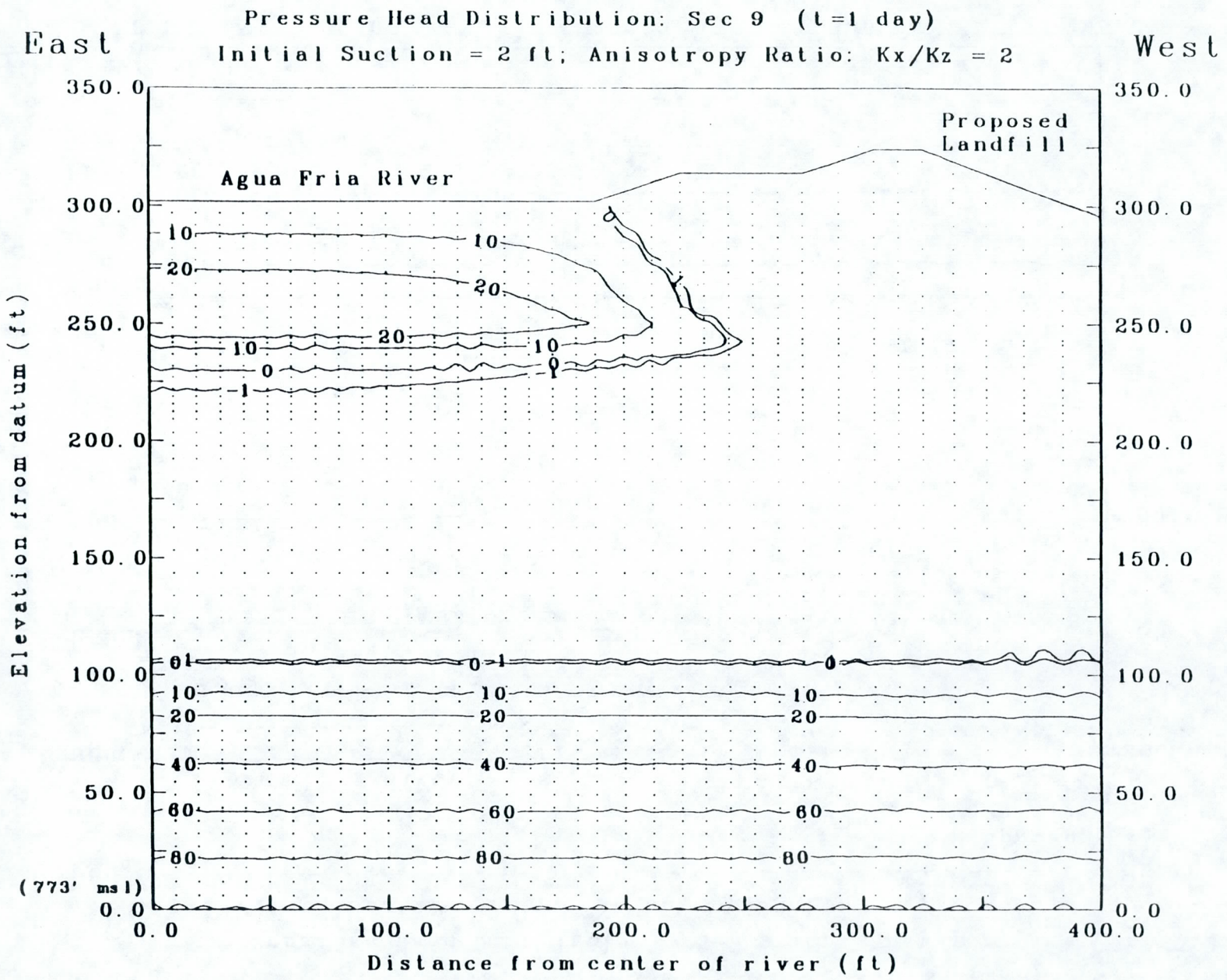





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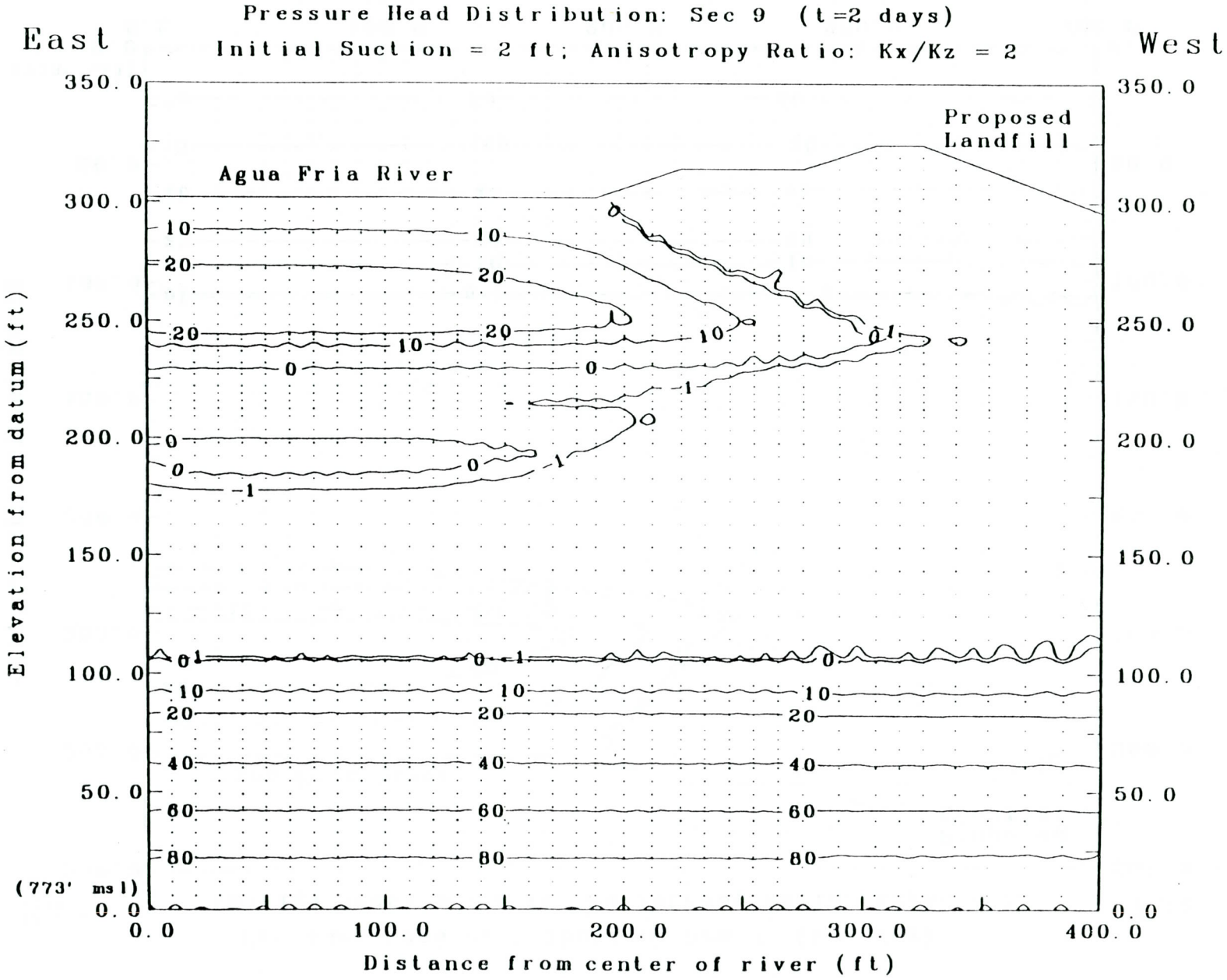
**Figure 45**


Pressure Head Distribution: Sec 9 (t=2 days)
Initial Suction = 2 ft; Anisotropy Ratio: Kx/Kz = 2
East
West
Proposed Landfill
Agua Fria River
Elevation from datum (ft)
(773' msl)
Distance from center of river (ft)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
100.0
200.0
300.0
400.0

Figur 5

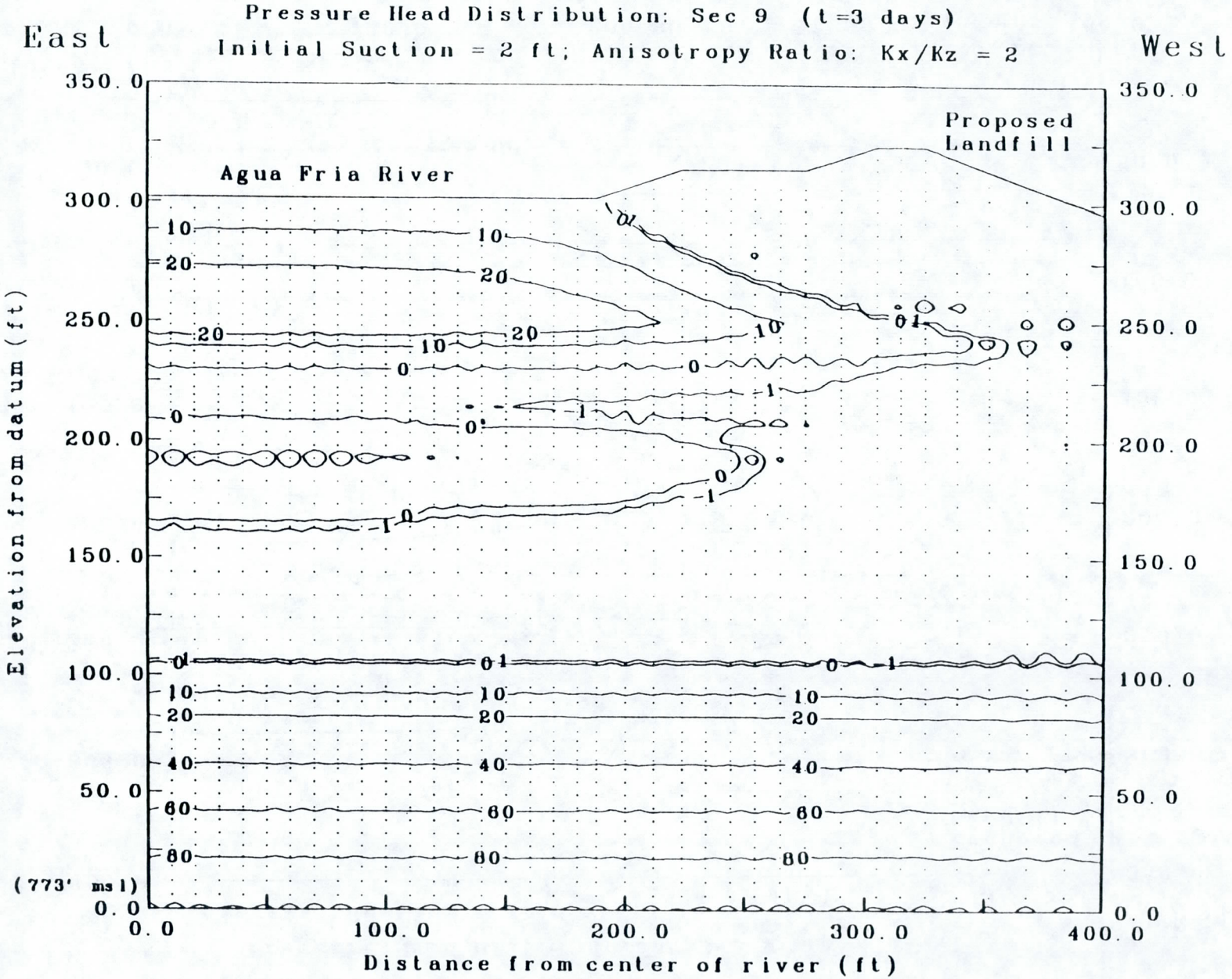





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**Figure 47**

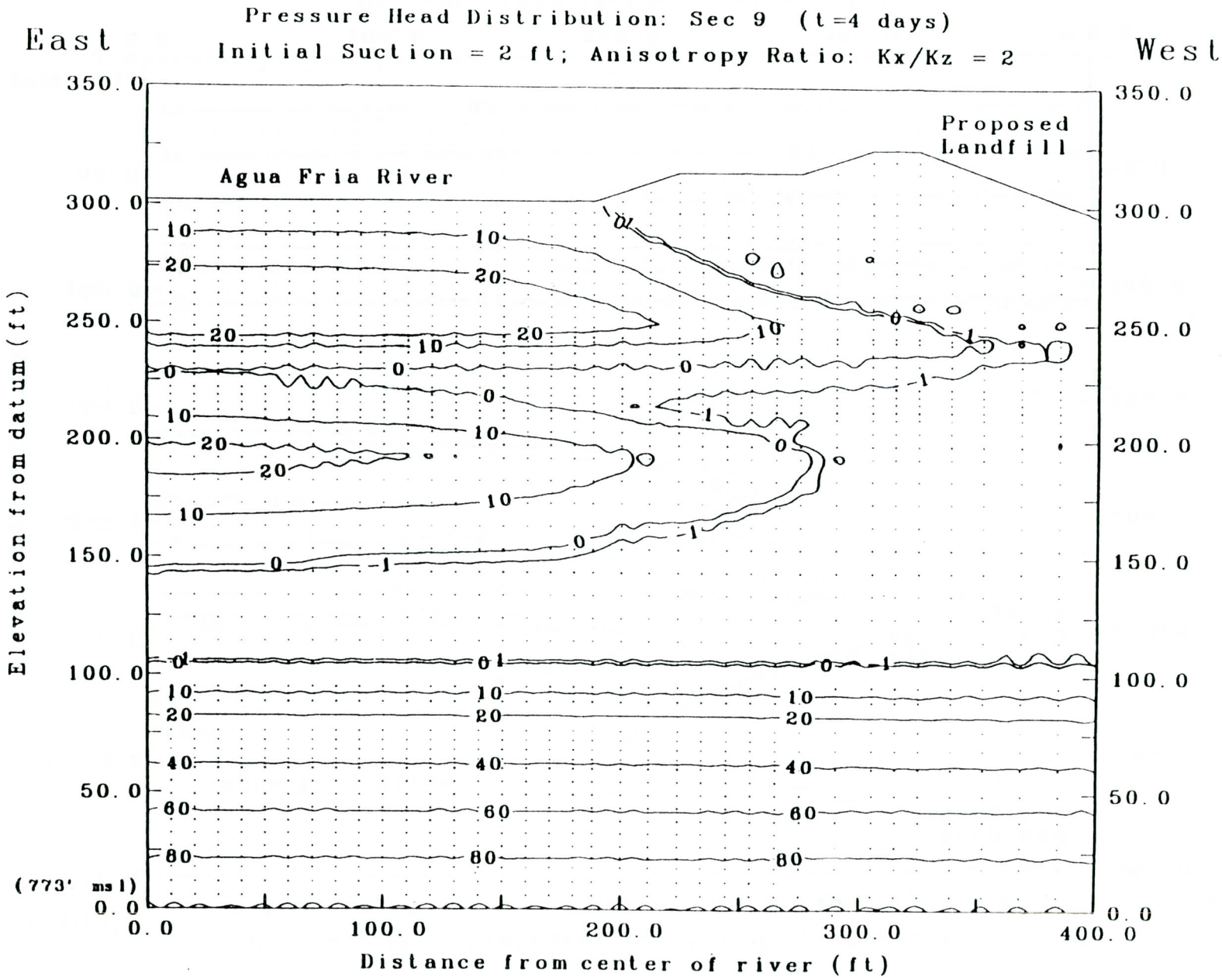
Pressure Head Distribution: Sec 9 (t=4 days)
Initial Suction = 2 ft; Anisotropy Ratio: Kx/Kz = 2
East
West
Proposed Landfill
Agua Fria River
Elevation from datum (ft)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
(773' msl)
0.0
100.0
200.0
300.0
400.0
Distance from center of river (ft)

Figure 3

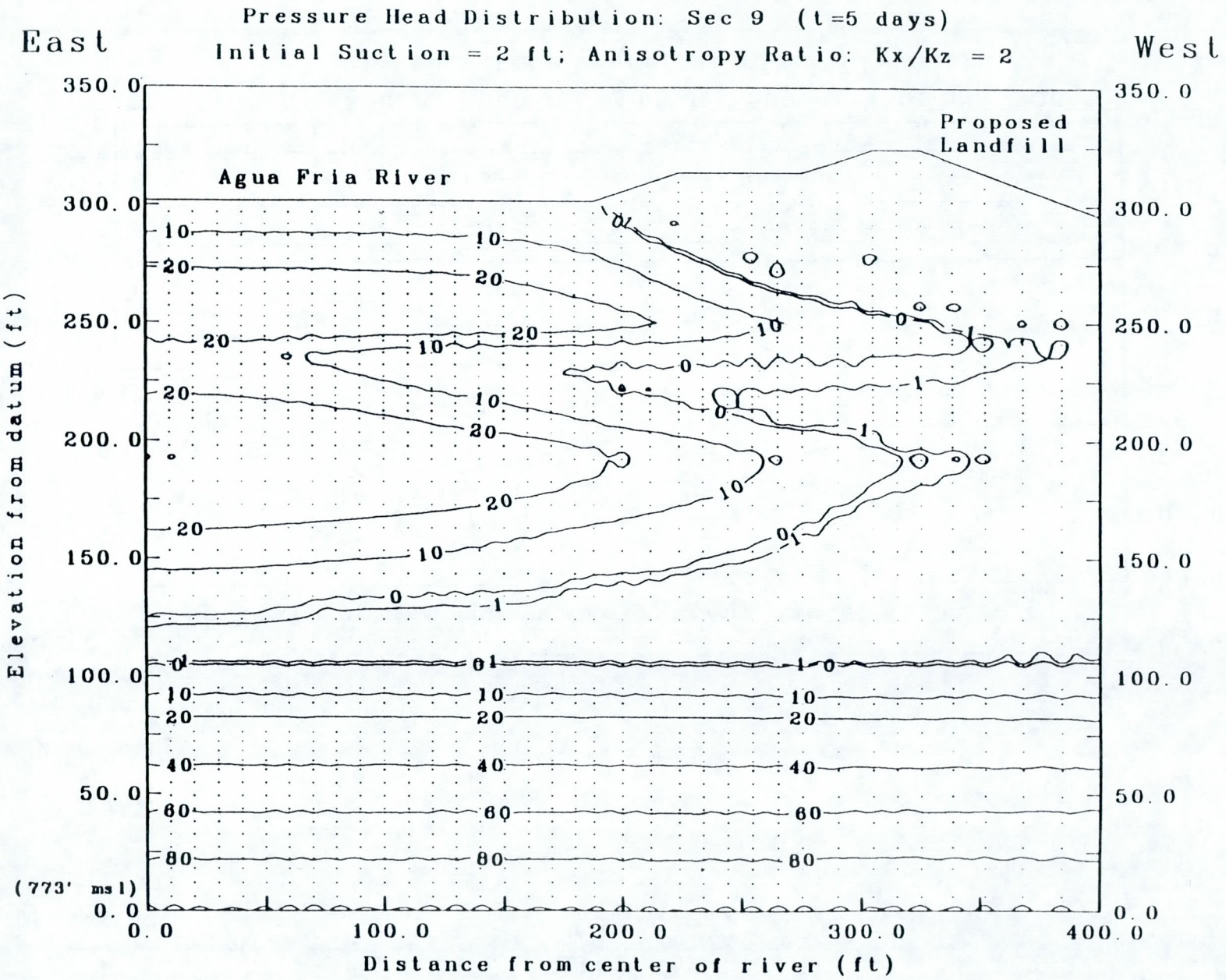




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**Figure 49**

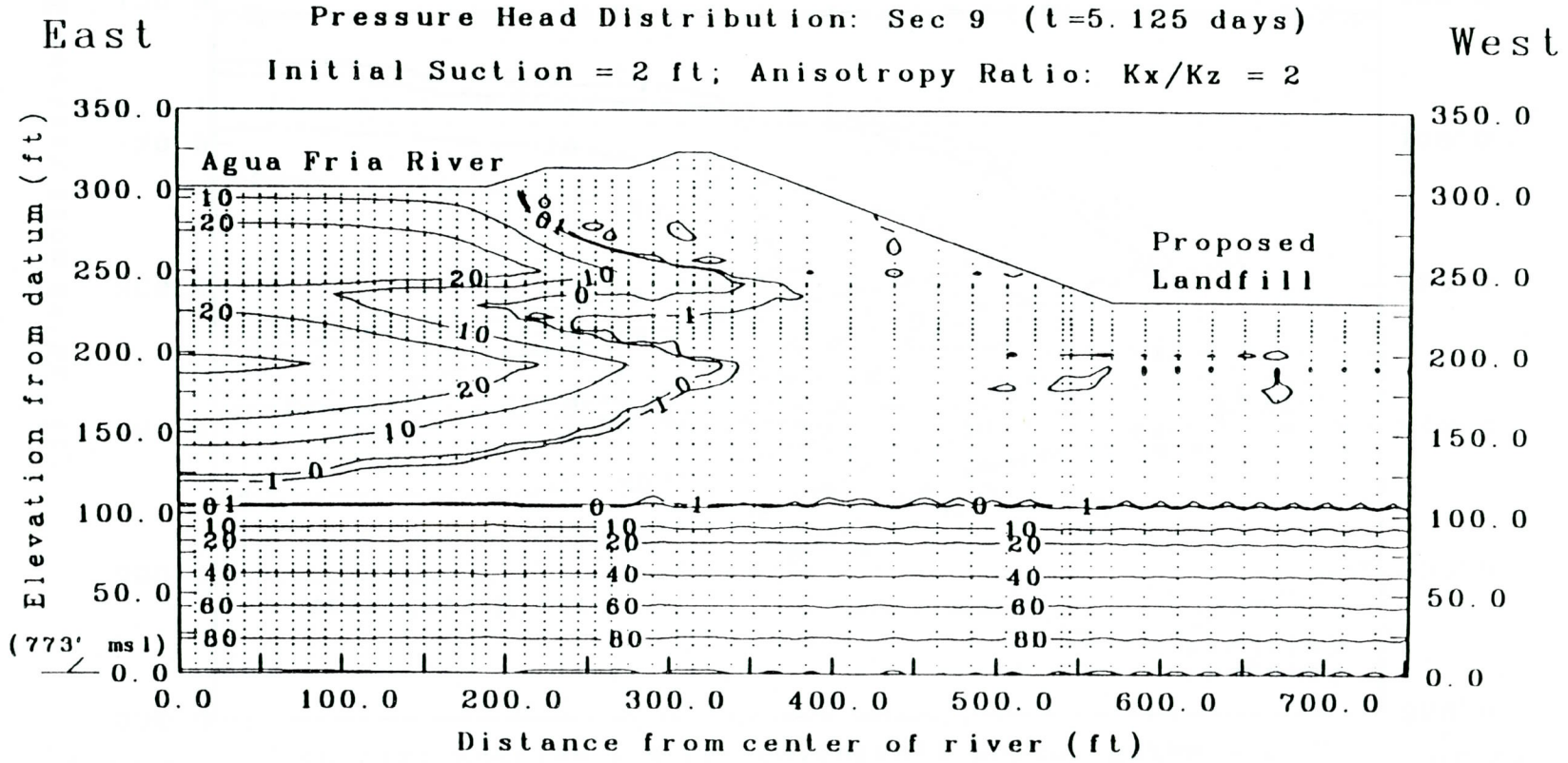
East
Pressure Head Distribution: Sec 9 (t=5.125 days)
Initial Suction = 2 ft; Anisotropy Ratio: Kx/Kz = 2
West
Agua Fria River
Proposed Landfill
Elevation from datum (ft)
(773' msl)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
0.0
100.0
200.0
300.0
400.0
500.0
600.0
700.0
Distance from center of river (ft)

Figure 50

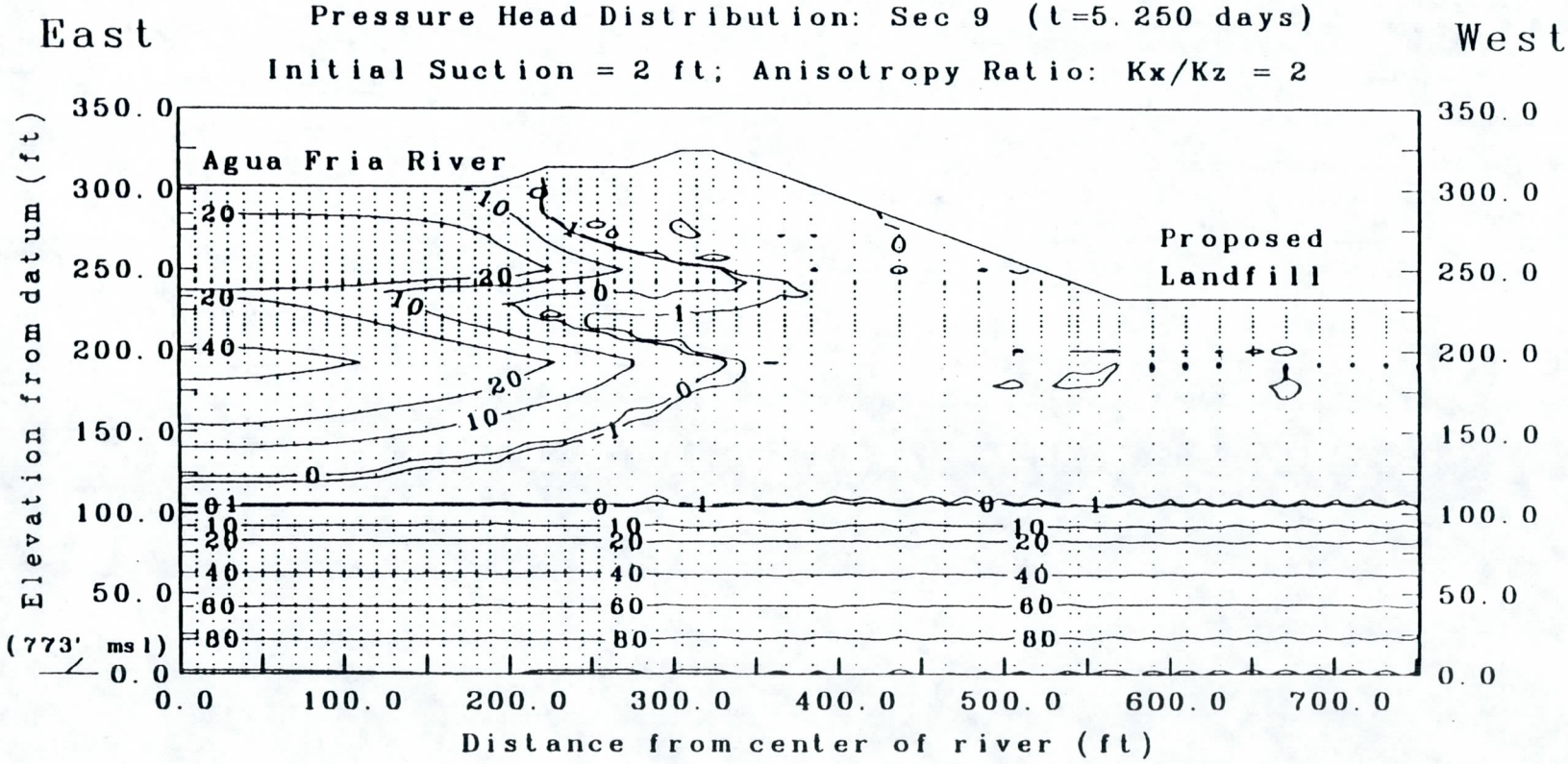




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**Figure 51**

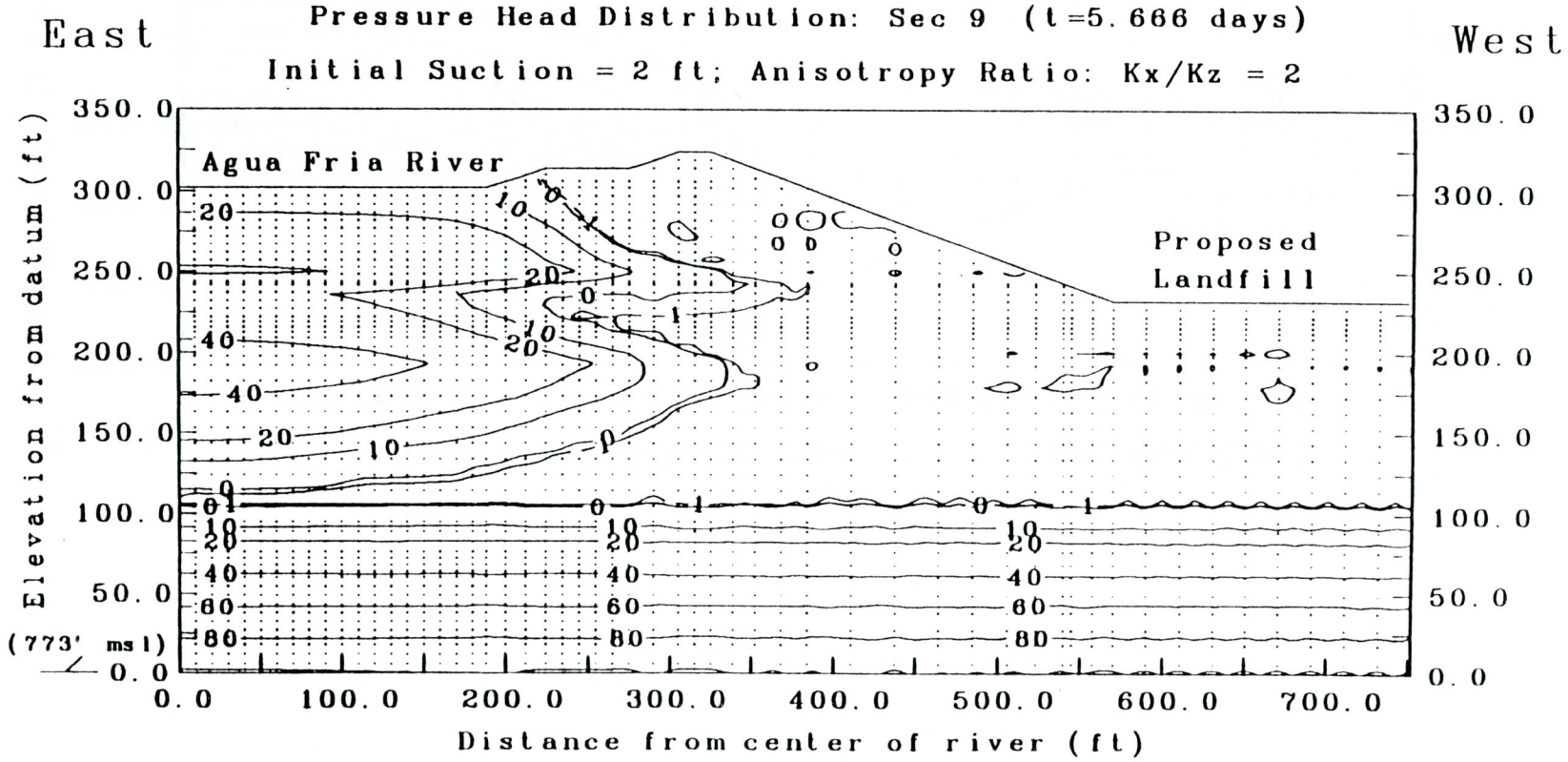
East
West
Pressure Head Distribution: Sec 9 (t=5.666 days)
Initial Suction = 2 ft; Anisotropy Ratio: Kx/Kz = 2
Agua Fria River
Proposed Landfill
Elevation from datum (ft)
(773' msl)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
0.0
100.0
200.0
300.0
400.0
500.0
600.0
700.0
Distance from center of river (ft)

Figure 52

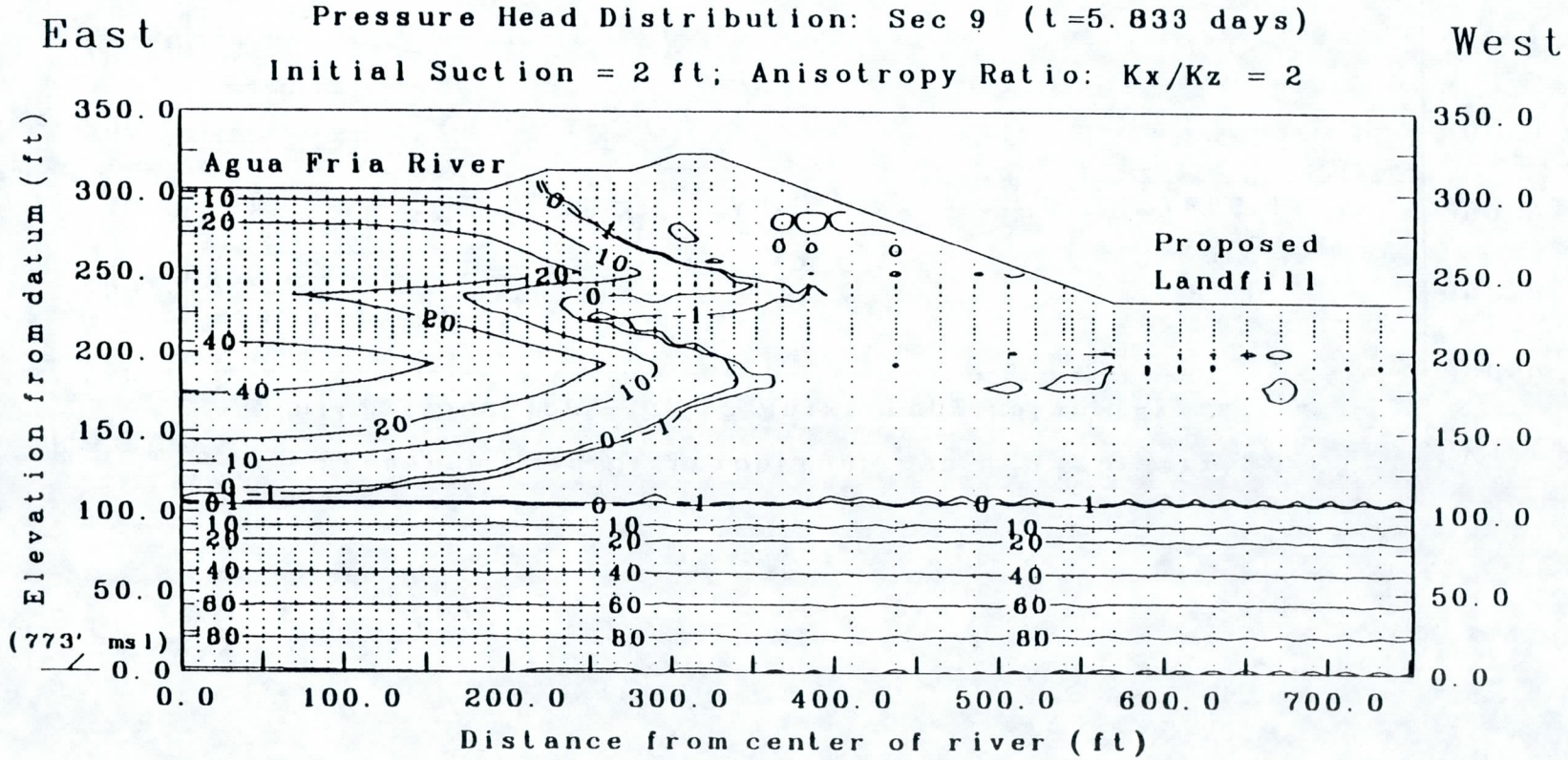




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**Figure 53**

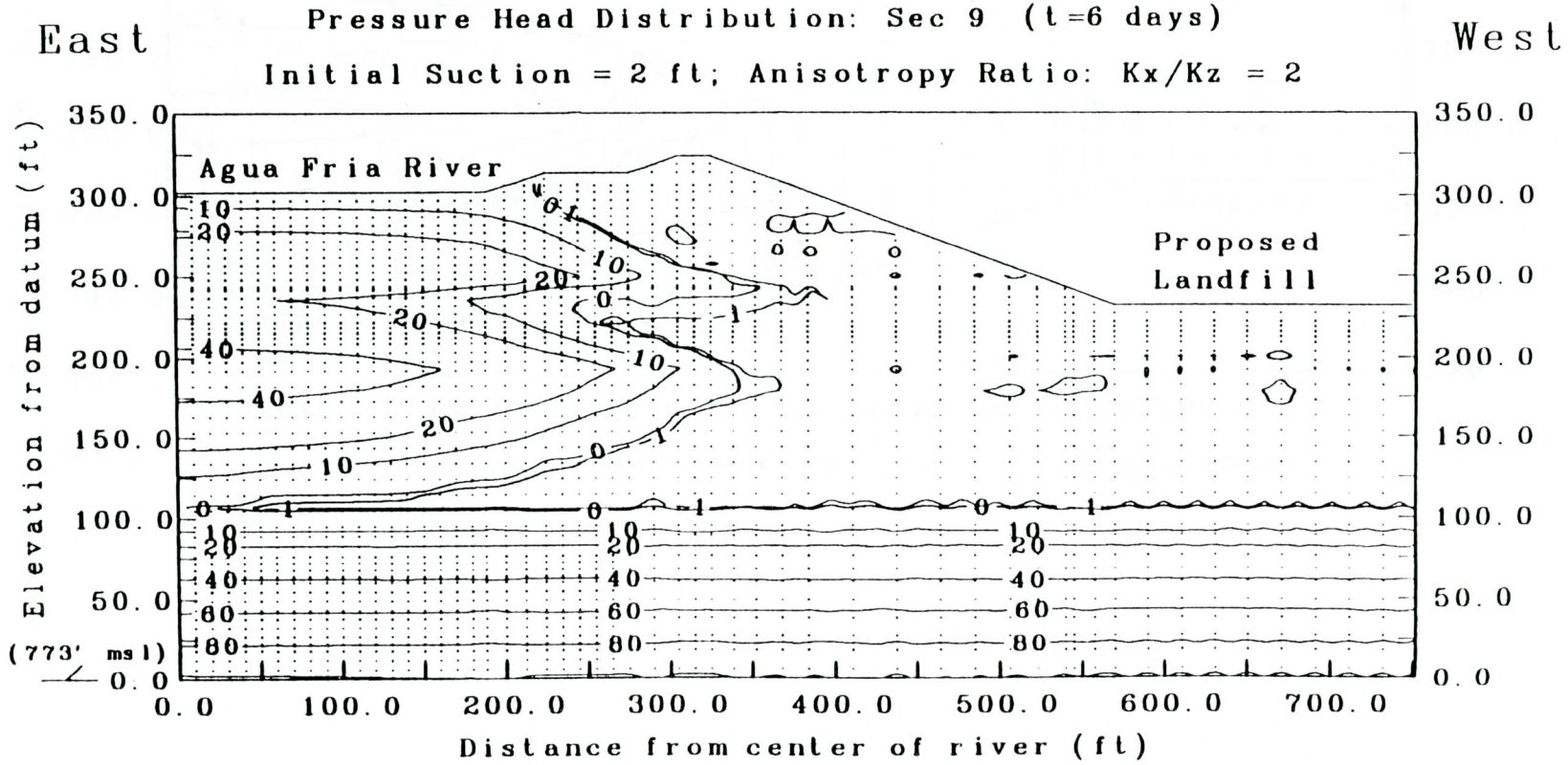
East
Pressure Head Distribution: Sec 9 (t=6 days)
West
Initial Suction = 2 ft; Anisotropy Ratio: Kx/Kz = 2
Agua Fria River
Proposed
Landfill
Elevation from datum (ft)
(773' msl)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
0.0
100.0
200.0
300.0
400.0
500.0
600.0
700.0
Distance from center of river (ft)

Figure 54

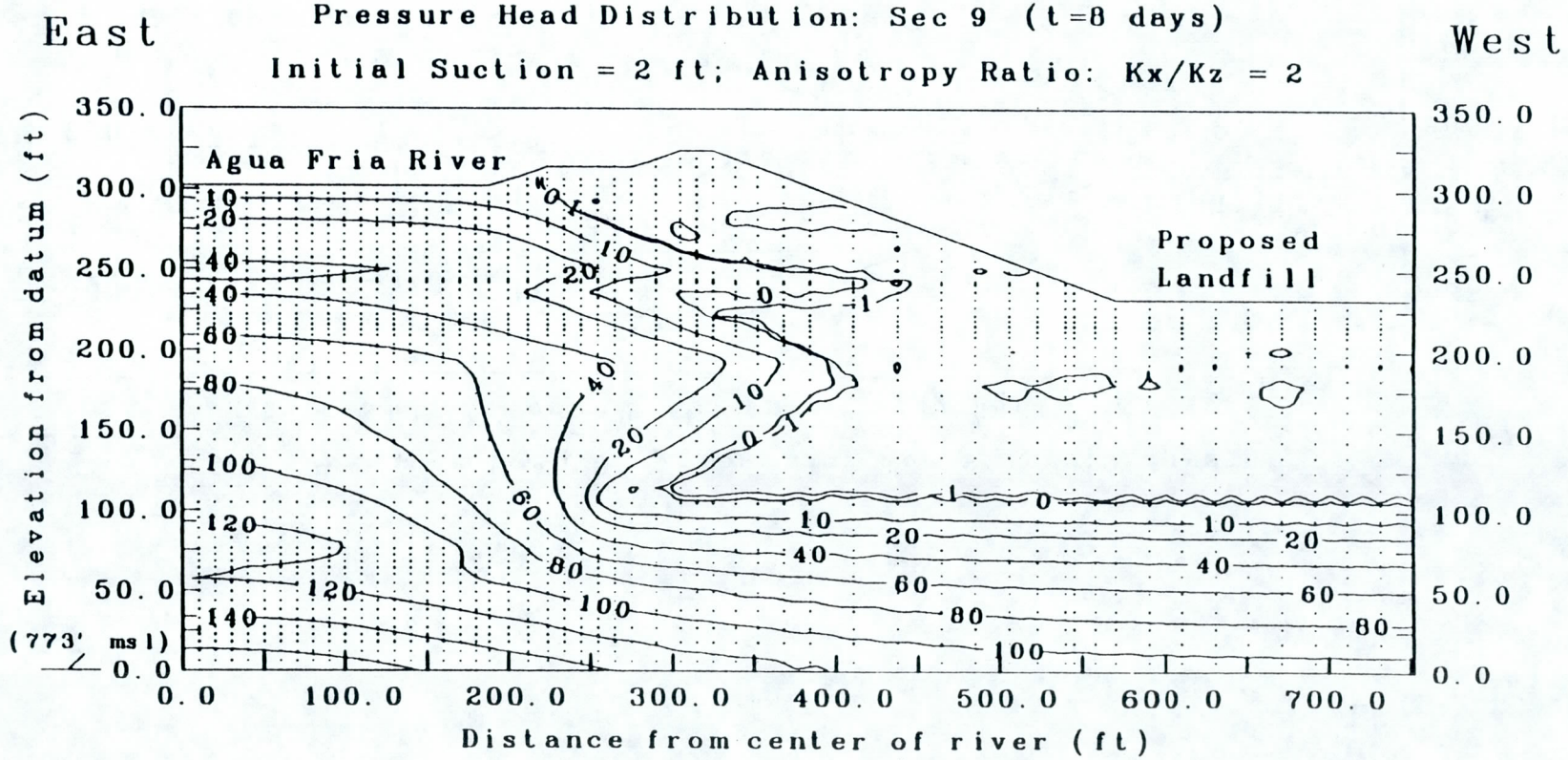




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**Figure 55**

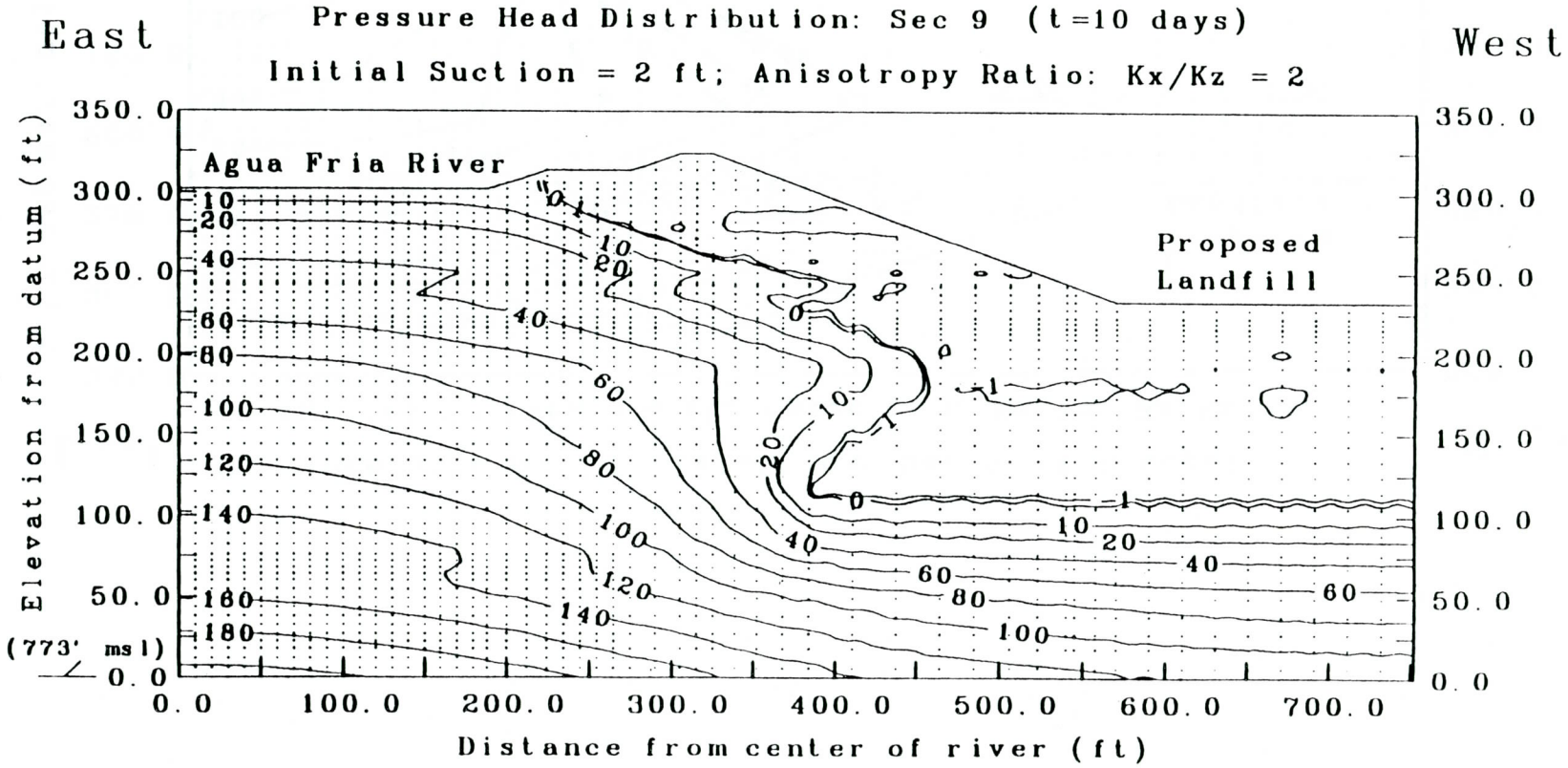
East
West
Pressure Head Distribution: Sec 9 (t=10 days)
Initial Suction = 2 ft; Anisotropy Ratio: Kx/Kz = 2
Agua Fria River
Proposed Landfill
Elevation from datum (ft)
(773' msl)
Distance from center of river (ft)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
100.0
200.0
300.0
400.0
500.0
600.0
700.0

**Figure 56**

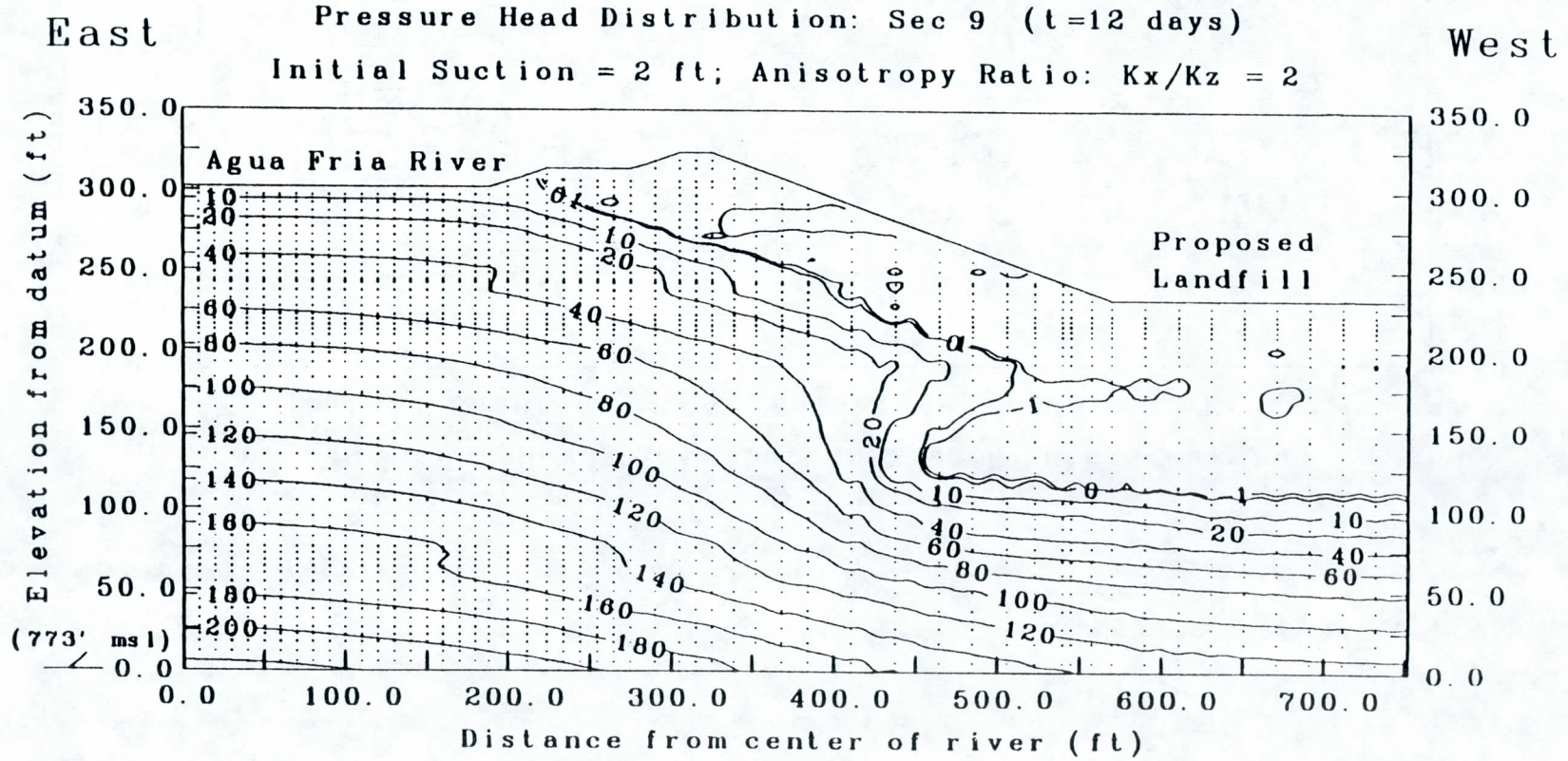




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Figure 57

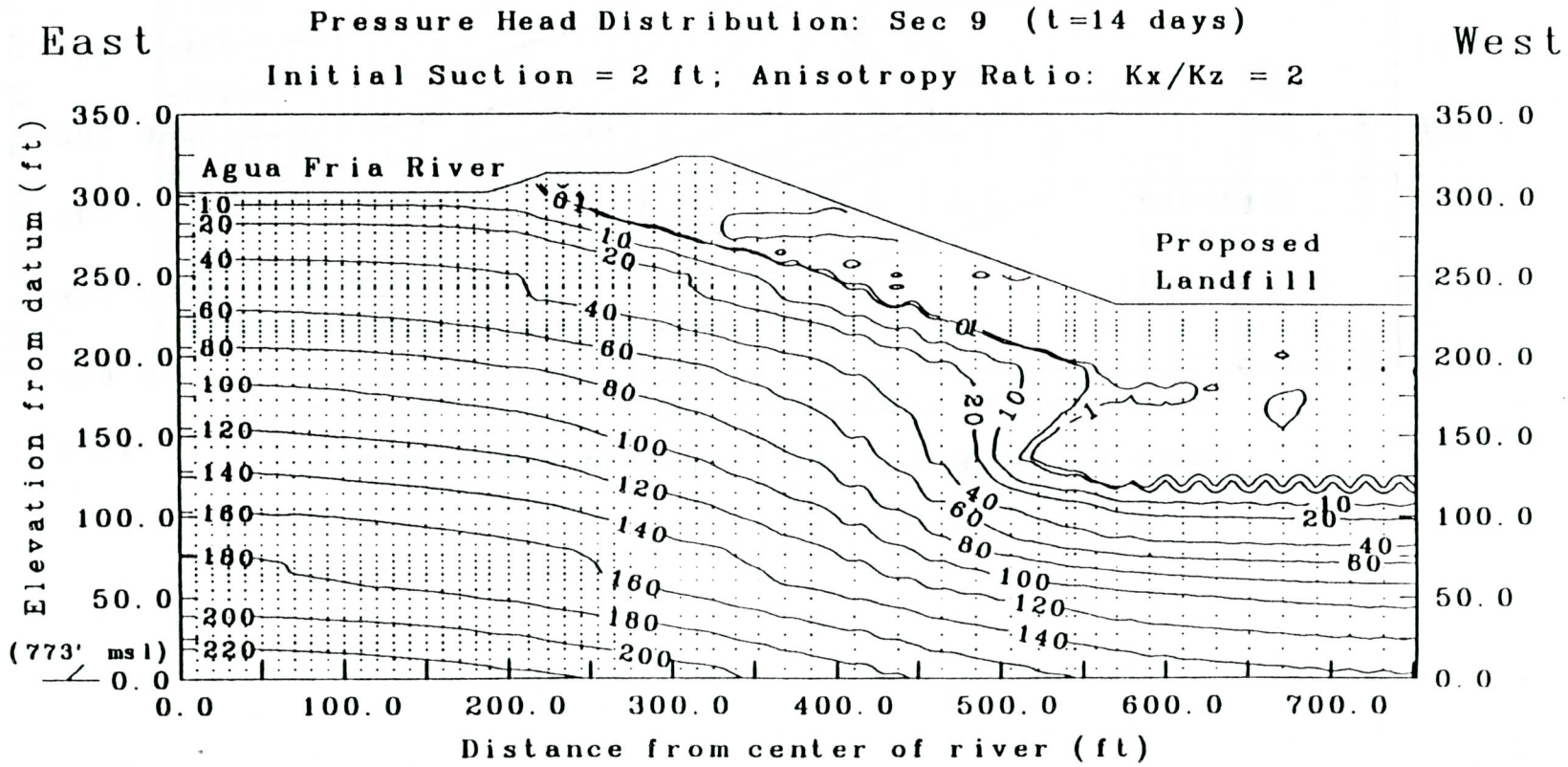
East
West
Pressure Head Distribution: Sec 9 (t=14 days)
Initial Suction = 2 ft; Anisotropy Ratio: Kx/Kz = 2
Agua Fria River
Proposed Landfill
Elevation from datum (ft)
(773' msl)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
0.0
100.0
200.0
300.0
400.0
500.0
600.0
700.0
Distance from center of river (ft)

Figure 3

Cumulative Infiltration as a Function of Time

Section 9

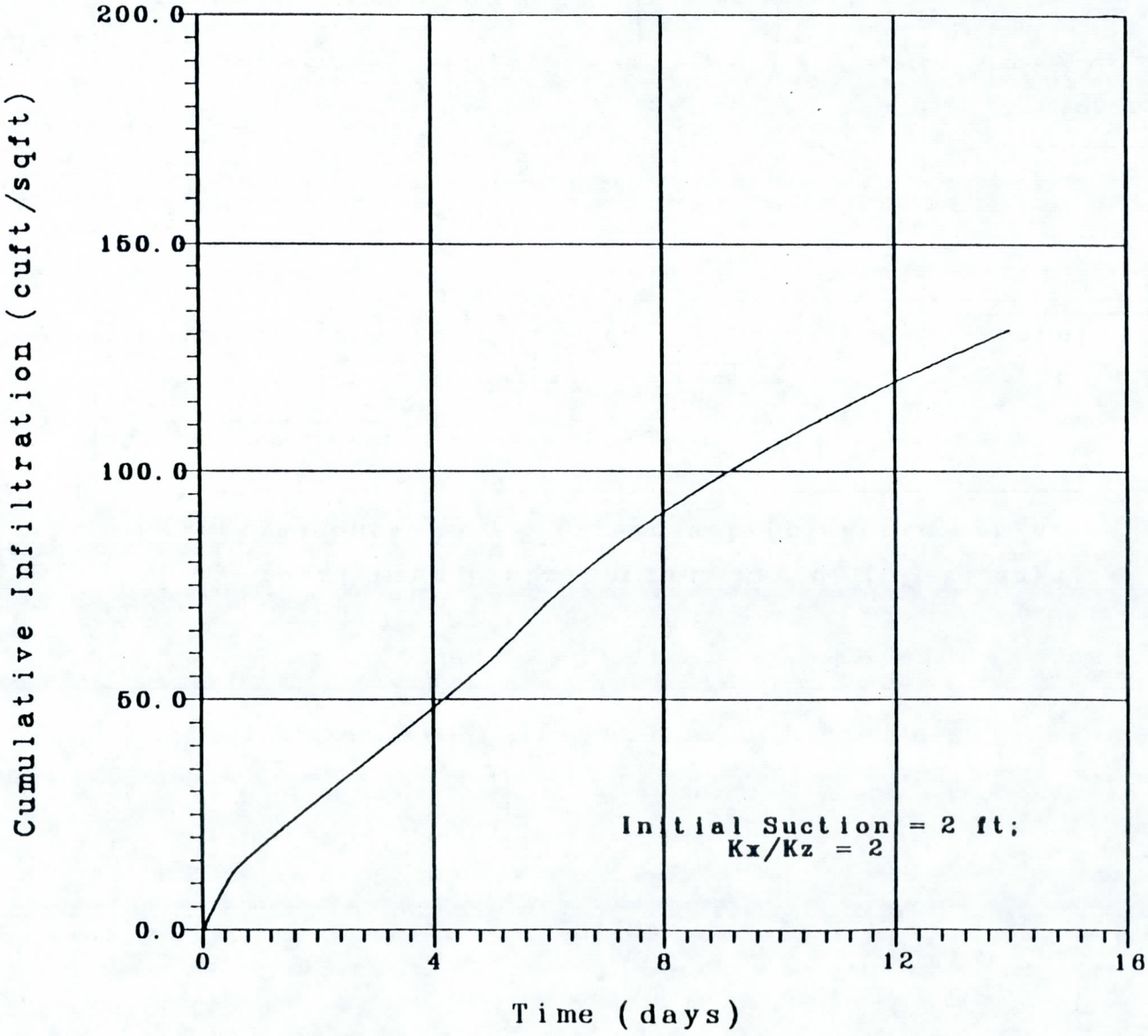




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**Figure 59**

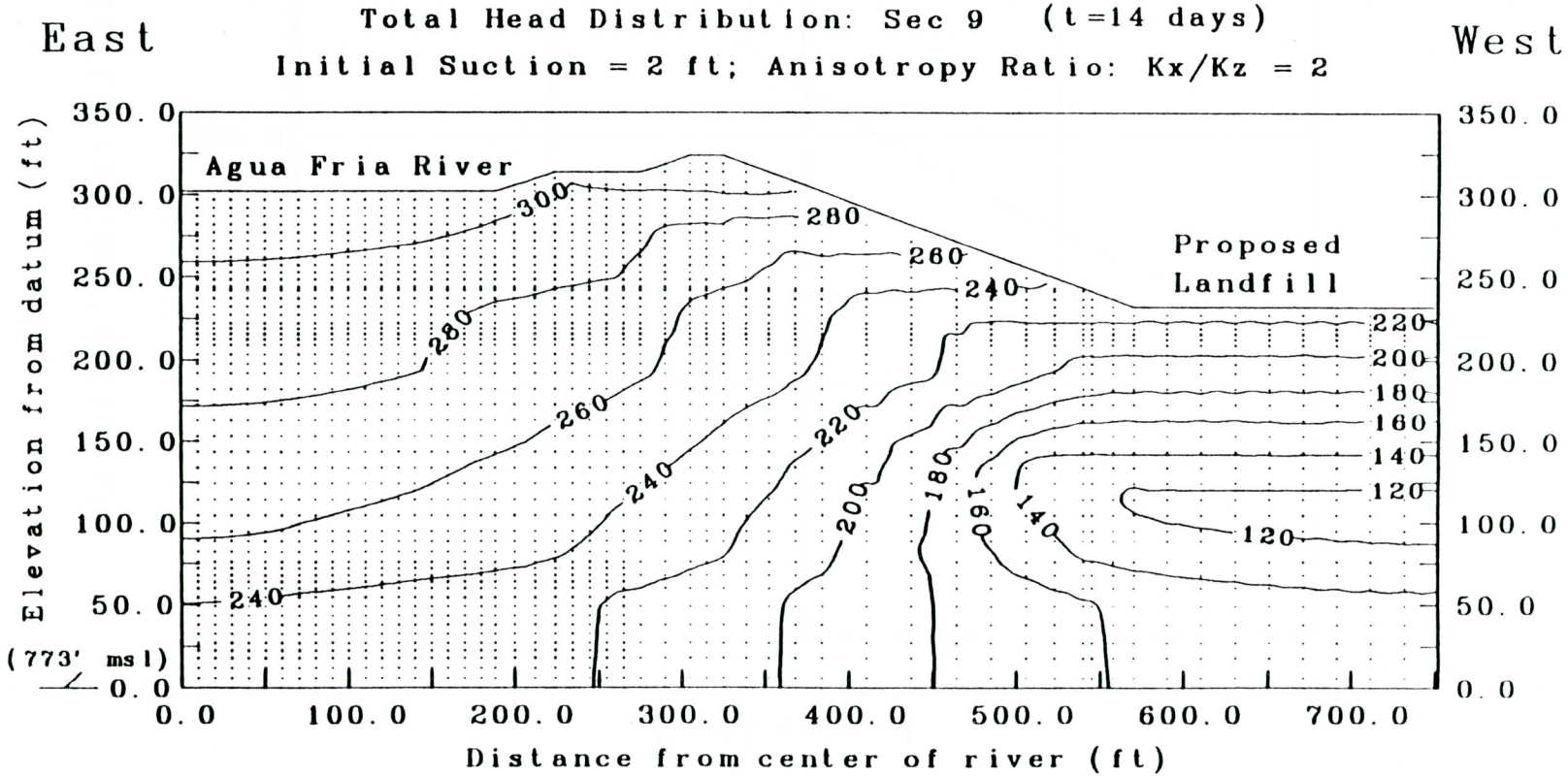
East
Total Head Distribution: Sec 9 (t=14 days)
Initial Suction = 2 ft; Anisotropy Ratio: Kx/Kz = 2
West
Agua Fria River
Proposed Landfill
Elevation from datum (ft)
(773' msl)
350.0
300.0
250.0
200.0
150.0
100.0
50.0
0.0
300
280
260
240
220
200
180
160
140
120
0.0
100.0
200.0
300.0
400.0
500.0
600.0
700.0
Distance from center of river (ft)

TABLE 1: HYDRAULIC PROPERTIES - SECTION 9

| Material No. | USCS | $K_s$ (ft/day) | $\phi$ | $\theta r$ | $\alpha$ ($ft^{-1}$) | n |
|---|---|---|---|---|---|---|
| 1 | SP | 2.84 | 0.25 | 0.05 | 2.13 | 3 |
| 2 | GP | 28.3 | 0.25 | 0.02 | 2.13 | 5 |
| 3 | GW-GM | 23.8 | 0.38 | 0.10 | 0.914 | 4 |
| 4 | SP-SW | 14.2 | 0.28 | 0.05 | 1.52 | 3 |
| 5 | CL | 0.493 | 0.42 | 0.15 | 0.152 | 1.5 |

SOURCES

| Material No. | USCS | BORING | INTERVAL (ft.) | $K_s$ TEST | Van Genuchten Parameters |
|---|---|---|---|---|---|
| 1 | SP | SHC 3 | 43.8-48 | CH | Modified Mualem Cat.No. 4124 Stephens et al (1987) |
| 2 | GP | -- | -- | Esti-mated | Mualem Cat. No. 4124 Stephens et al (1987) |
| 3 | SM-GM | SHC 3 | 19-20.4 | FH | Mualem Cat. No. 4147 Stephens et al (1987) |
| 4 | SP-SW | -- | -- | Esti-mated | Modified Mualem Cat.No. 4124 Stephens et al (1987) |
| 5 | CL w/cal | SHC 3 | 61.1-64.1 | CH | Modified "Clay" Parker et al (1985) |

Note:

$K_s$ = Saturated Hydraulic Conductivity
$\phi$ = Porosity
$\theta r$ = Residual Water Content
$\alpha$ = Model Parameter
n = Model Parameter
CH = Constant Head
FH = Falling Head

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Attachment 10

# QUALITY ASSURANCE PROCEDURES FOR EARTHWORK CONSTRUCTION AT CHOLLA SANITARY LANDFILL

# Quality Assurance Procedures for Earthwork Construction at Cholla Sanitary Landfill

## Browning-Ferris Industries of Arizona
## El Mirage, Arizona

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# I. INTRODUCTION

## A. Purpose

The purpose of this document is to outline the minimum inspection requirements needed to verify that earthwork construction specifications are met or exceeded during construction of Cholla Sanitary Landfill. Quality control work shall be performed by an independent third-party. The independent third-party firm is responsible for incorporating its own Quality Assurance procedures as needed.

## B. Scope

This document addresses the minimum test methods, test frequencies, and documentation necessary to verify adherence to the earthwork construction specifications. Quality Assurance (QA) for synthetic components of the landfill are described in a separate manual. Protocol for reporting test results, certifying compliance with construction specifications, correcting construction deficiencies, and documenting such corrections is also provided.

## C. Definitions

1. The Company: Browning-Ferris Industries of Arizona, Inc. The party for whom the work is to be performed.

2. Quality Control (QC): The formal procedures utilized by project contractors to ensure conformance of their products to plans, specifications, or other criteria established for the project. Contractor Quality Control programs are not a part of this plan, although they will be reviewed to ensure general conformance with the objectives stated herein.

3. Quality Assurance (QA): The formal organization and procedures utilized by the Company for verifying that work performed meets project requirements. The Quality Assurance effort will generally include a comprehensive and continuous review of contractor QC programs and results, as well as construction materials. Independent verification of test results shall be accomplished through inspection and materials testing.

4. Quality Assurance (QA) Engineer: Third party, independent consulting engineering firm providing monitoring of all earthwork aspects of landfill construction, technically accountable to the Company. The QA Engineer is responsible for certification of landfill construction according to the earthwork specifications outlined herein.

5. Engineering Services: A corporate department of Browning-Ferris Industries with civil/geotechnical engineering and engineering geology/hydrology capabilities.

6. Atterberg Limits: The liquid limit, plastic limit, and shrinkage limit for soils (ASTM D4318-84 and D427-83, respectively). The water content



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when the soil behavior changes from the liquid to the plastic state is the liquid limit, from the plastic to the semi-solid state is the plastic limit, and from the semi-solid to the solid state is the shrinkage limit.

7. Compaction: The process of increasing the density or unit weight of soil by rolling, tamping, vibrating, or other mechanical means.

8. Density: Mass density of soil is its weight per unit volume, usually reported in pounds per cubic foot.

9. Grain Size Distribution: Distribution of particle sizes within a soil (ASTM D422-63).

10. In situ: "as is," or as it exists in place naturally.

11. Moisture Content: Ratio of quantity of water in the soil (by weight) to the weight of the soil solids (dry soil), expressed in percentage; also referred to as water content.

12. Optimum Moisture Content (OMC): Moisture content corresponding to maximum dry density as determined by ASTM D1557.

13. Permeability: Ability of pore fluid to travel through a soil mass via interconnected voids. "High" permeability indicates a relatively rapid rate of flow, and low permeability, a relatively slow rate. Rates of permeability are generally reported in centimeters per second (cm/sec).

14. Plasticity: Ability of soil mass to flow or be remolded without raveling or breaking apart. Generally that range of soil water content between the liquid and plastic limit.

15. Sieve (200 Mesh): Refers to soil particle size passing (smaller than or equal to) the U.S. Sieve No. 200.

D. Quality Control Organization

1. Responsibilities: All contractors/vendors shall be responsible for implementing a quality control program.

2. Submittals: All contractors shall be responsible for submitting the procedures and results of the quality control program to the QA Engineer for review, approval, and documentation. Contractors supplying borrow source materials shall submit the procedures and results of borrow source evaluation testing to the QA Engineer prior to shipment of the material to the site. The QA Engineer shall review the procedures and results of the borrow source evaluation and shall notify the contractor that the material is approved for shipment.

E. Quality Assurance Organization

1. Responsibilities: The QA Engineer is responsible for certifying that the final construction is in conformance with the project plans and

specifications. This QA plan outlines the minimum inspection requirements to be used by the Engineer to ensure that the project plans and specifications are met.

2. Administration: The QA Engineer shall be responsible for the overall administration of quality assurance procedures and for the control of quality assurance documents. The Engineer shall be responsible for overseeing and coordinating the work of the quality assurance technicians and testing laboratories.

3. Document Control: The QA Engineer shall initiate a project filing system which shall include at a minimum:

   - File copy of the QA procedures, updated as necessary.
   - Photographic documentation of the construction.
   - Survey measurements.
   - Field and laboratory test results.
   - Daily and weekly field results and reports.
   - Field certification reports including as-built drawings.
   - Non-conformance and corrective action reports.
   - Minutes of construction meetings.

4. Daily Field Reports: The QA Engineer shall prepare daily field reports which shall include, at a minimum, the following:

   a. Date, project name, location, and other identification.

   b. Data on weather conditions.

   c. Reports of meetings held and pertinent results.

   d. Descriptions and locations of construction underway during the time frame of the daily report.

   e. Equipment being used on site.

   f. Descriptions of areas of work being tested.

   g. Description of off-site materials received, including any quality verification documentation.

   h. Decisions made regarding approval of units of material or of work, and/or corrective actions to be taken in instances of substandard quality.

   i. Signature of the QA Engineer.



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5. Test Results: Test results shall be reported on a standard sheet and shall include, at a minimum, the following:

   a. Description of testing activity.

   b. Location of the field testing activity or the location from which the sample was obtained.

   c. Type of testing activity and procedure used (reference standard method when appropriate).

   d. Recorded observation or test data, with all necessary calculations.

   e. Results of testing activity and comparison with specification requirements.

   f. Signature of test performer and QA Engineer.

6. Final Documentation: At project completion, a final certification report shall be issued by the QA Engineer and transmitted to the Company. This document shall include, at a minimum, the following information:

   a. Scope of work.

   b. All daily field reports.

   c. All laboratory and field test results.

   d. Test methods.

   e. Evaluation of all test results with respect to project specifications.

   f. Any non-conformance reports and corrective action reports.

   g. Personnel involved with the project and their respective qualifications.

   h. Record drawings and survey notes.

   i. Certification that construction was completed in accordance with construction drawings and specifications. This certification should be signed and stamped by a professional engineer registered in Arizona.

## II. MATERIALS AND PRECONSTRUCTION TESTING

### A. Low Permeability Soils:

1. All soil to be used for construction of soil liner shall be stockpiled and inspected by the QA Engineer prior to actual use. If a natural deposit is to be used, the QA Engineer shall inspect borrow material as it is hauled to the site.

2. Rock fragments, boulders, and cobbles contained in the soil shall not exceed 3/4 inch in any dimension. Material shall be inspected and limbs, roots, and other deleterious materials shall be removed to the extent practical.

3. The following QA tests shall be performed on stockpiled material to be used for subsequent construction at the specified frequencies and whenever a change in material occurs.

| | Property | Test Method | Frequency |
|---|---|---|---|
| 1. | Gradation | ASTM D1422 | 10,000 $yd^3$ |
| 2. | Atterberg Limits | ASTM D4318 | 10,000 $yd^3$ |
| 3. | Moisture Content | ASTM D2216 | 1,000 $yd^3$ |
| 4. | Moisture-Density Curve | ASTM D1557 | 5,000 $yd^3$ |
| 5. | Laboratory Permeability | ASTM Draft* Method | 10,000 $yd^3$ |
| 6. | Undrained Shear Strength | ASTM D2850 | 5,000 $yd^3$ |

### B. Drainage Layer:

1. Clean subangular, subrounded, rounded, or well rounded sand shall be used.

2. The sand shall have less than 5 percent passing the No. 200 sieve.

3. The sand shall have a demonstrated permeability greater than or equal to 1 cm/sec.

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* See Appendix



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4. The following QA tests shall be performed on stockpiled material at the specified frequencies and whenever a change in material occurs.

| Property | Test Method | Minimum Frequency |
|---|---|---|
| Grain Size | ASTM D422 | 2,500 $yd^3$ |
| Permeability | ASTM D2434 | 10,000 $yd^3$ |

C. Gravel:

1. Clean subangular, subrounded, rounded, or well rounded gravel may be used.

2. The gravel shall have a demonstrated permeability greater than or equal to 10 cm/sec.

3. The gravel shall have less than 3 percent passing the No. 200 sieve.

4. The following QA index test shall be performed on stockpiled material at the specified frequencies and whenever a change in material occurs.

| Property | Test Method | Frequency |
|---|---|---|
| Grain Size | ASTM D422 | 500 $yd^3$ |
| Permeability | ASTM D2434 | 500 $yd^3$ |

D. Protective Soil Cover

1. Soil obtained from on-site excavations or off-site sources.

2. The maximum particle size shall be 3/4 inch. If the protective soil cover is to be placed directly on the geomembrane it shall contain no angular particles greater than 3/8 inch.

3. Shall meet gradation requirements for filter criteria outlined in Geotextile Specification.

4. The following QA index test shall be performed on stockpiled material at the specified frequencies and whenever a change in material occurs.

| Property | Test Method | Frequency |
|---|---|---|
| Grain Size | ASTM D422 | 5,000 $yd^3$ |

E. Earthfill

1. Soil obtained from on-site excavations or off-site sources shall be free of organic and other deleterious material.

2. Maximum particle size of 6 inches.

3. The following QA index tests shall be performed on stockpiled material at the specified frequencies and whenever a change in material occurs.

| Property | Test Method | Frequency |
|---|---|---|
| Compaction | ASTM D1557 | 10,000 $yd^3$ |
| Atterberg Limit | ASTM D4318 | 10,000 $yd^3$ |

## III. QUALITY ASSURANCE FIELD TESTING OF SOILS

A. Excavation

1. After cell excavation is complete, the entire base shall be proof-rolled with a smooth drum roller.

2. Areas that exhibit excessive rutting, heaving, or softening shall be excavated and replaced with compacted suitable material, according to the specifications.

B. Low Permeability Soils

1. During bentonite admixing, the percent bentonite added to the soil shall be determined. The procedure used will depend on the method used by the contractor to add the bentonite.

2. Low permeability soils shall be compacted to a minimum dry density of 90 percent of the maximum dry density determined from the Modified Proctor test (ASTM D-1557). Densities less than 90 percent of the maximum dry density shall be recompacted and/or removed and/or reworked to meet density objectives.

3. The soil shall be compacted within the specified water content.

4. Loose thickness and initial (before compaction) water content shall be tested at least once for every 20,000 square feet of liner area and at least once per lift.

5. Each lift shall be tested to determine compacted density and water content using a nuclear gauge (ASTM D2922) before subsequent lifts are placed.

6. The QA Engineer shall determine the locations of the compacted density and water content tests using a grid pattern with a diagonal spacing of about 65 feet. Test locations for each lift shall be above an



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untested location in the previous lift. The QA Engineer may require testing, in addition to the grid pattern, of areas he believes may not meet specifications.

7. In addition, the QA Engineer shall specify test locations in the sump area. The QA Engineer may select and test locations that he believes may not meet specifications.

8. One sand-cone density (ASTM D1556) shall be taken each day to correlate dry density and water content measurements with those of the nuclear gauge. The results of these tests and correlation to nuclear gauge measurements shall be documented and reported.

9. A thin-walled tube sample of in-place clay liner shall be obtained every 5,000 yards or 1 per lift (whichever results in the greater number of tests) for laboratory permeability (Draft ASTM Method).

10. Whenever the compacted soil liner is disturbed for testing or sampling, any penetrations shall be backfilled by hand-tamped bentonite pellets, bentonite powder, or bentonite/clay mixture.

11. To verify that correct Proctor curve is used, one clay sample shall be obtained for every 5,000 cubic yards placed or every three working days, whichever is less. This sample shall weigh approximately 10 pounds, and shall be compacted in a Modified Proctor mold using Modified Proctor compactive effort at the existing clay water content (1 point Proctor). Atterberg Limits and percent finer than the No. 200 sieve shall be determined every 2,500 $yd^3$.

12. The frequency of confirmatory compaction testing may be relaxed once the Engineer has established a family of moisture-density curves relating to Atterberg limits and percent fines. Once the above-described relationships have been established (on a frequency not less than every five percentage points change in liquid limit), the confirmatory compaction testing can be reduced to confirmatory classification testing (Atterberg limits and percent finer than No. 200 sieve). At least one compaction test should still be run for every week of liner construction.

13. The QA Engineer shall confirm that before placing the next lift, the surface of the previous lift has been scarified to a depth of one to two inches, if required, and the surface has not been allowed to become excessively dry.

14. The placement of fill during times of marginal weather shall be at the discretion of the QA Engineer. No select fill shall be placed or compacted during sustained periods of temperatures below 30°F. Select fill may be placed and compacted during periods of early morning freezing temperatures with warming trends during the day.

15. All sample and test locations shall be documented. The lift in which the testing occurred shall also be documented.

D. Protective Soil Cover

1. All synthetic components shall be approved by the QA Engineer prior to placement of the protective soil cover.

2. Samples shall be collected at a frequency of 1 sample per 2,000 cubic yards. Sample size shall not be less than 10 pounds. Samples shall be tested for grain size distribution.

3. All sample and test locations shall be documented. Test and sample depth shall also be documented.

E. Random Fill

1. Random fill shall be compacted to a minimum dry density of 90 percent of the maximum dry density as determined by ASTM D-1557. Densities less than 90 percent of the maximum dry density shall be recompacted and/or removed and reworked to meet density objectives.

2. The soil shall be compacted within the specified water content range.

3. Loose thickness and initial (before compaction) water content shall be tested at least once per 500 cubic yards, but at least once per lift (if the lift is less than 500 cubic yards).

4. Each lift shall be tested to determine compacted density and water content using a nuclear gauge (ASTM D2922) before subsequent lifts are placed.

5. One sand-cone density (ASTM D1556) shall be taken each day to correlate dry density and water content measurements with those of the nuclear gauge. The results of these tests and correlation to nuclear gauge measurements shall be documented and reported.

6. To verify that the proper compaction curve is being used, one ASTM D1557 test shall be performed per 5,000 cubic yards or 1 per material type. As appropriate 1 point Proctor curves may be used.

## IV. TEST EQUIPMENT CALIBRATION

A. All field test equipment will be kept under control of the QA Engineer. The QA Engineer will be fully trained in the use of equipment, test procedures, and interpretation of results for each piece of test equipment. A copy of the calibration certificate will be kept by the QA Engineer. This equipment shall be calibrated in accordance with the QA Engineers Quality Assurance procedures.

B. Calibration of nuclear density gauges shall conform to the frequencies and methods outlined in ASTM D2922-78 and D3017-78. Unstable or erratic gauges shall not be used for density testing and shall be immediately removed from the site.



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## V. NONCONFORMING TEST RESULTS

### A. Density and Moisture Content

Density and moisture content test locations which fail to meet or exceed construction criteria require reworking. The boundaries of the area to be reworked will be the closest test locations which meet density and moisture content specifications. The non-conforming area shall be reworked, dried, or wetted as necessary and retested. A non-conformance report shall be written for areas which do not meet construction specifications after reworking and retesting.

### B. Laboratory Permeability Tests

Laboratory permeability test results which demonstrate a permeability above $1 \times 10^{-6}$ cm/sec for the low permeability soils or below 1 cm/sec for the drainage layer shall be immediately brought to the attention of the Company by the QA Engineer. Non-conforming permeability test results on the low permeability soils shall trigger an immediate re-evaluation of compaction criteria and a review of all permeability results from low permeability soil samples. Nonconforming drain material shall be removed.

### C. Non-Conformance Reports

All non-conformance reports shall be brought to the attention of the Company by the QA Engineer and shall be documented in the QA files.

## VI. SURVEY CONTROL

### A. Cell Excavation

1. The completed excavation surface shall be surveyed to ensure that actual depths and grades are in accordance with the plans and specifications.

2. A minimum of one cross-section for every 100 lineal feet of cell, measured along the cell length, shall be surveyed. At a minimum, survey points shall be established at the top, mid-point, and bottom of each slope and at three points on the bottom of the cell.

3. In addition, a minimum of one slope profile, for every 100 lineal feet of cell, measure along the cell width, shall be surveyed. At a minimum, survey points shall be established at the top, mid-point, and bottom of each slope and at least one of the survey points on the bottom of the cell established by the longitudinal survey.

4. Acceptable tolerances on survey coordinates shall be +/- 0.2 foot on elevations and +/- 1.0 foot on coordinates.

5. The Quality Assurance technician certifying the survey results shall be a Land Surveyor or Professional Engineer registered in Arizona.

6. The QA Engineer shall certify that the cell excavation meets the requirements in the plans and specifications and submit documentation of such to the Company.

B. Low Permeability Soils

1. The completed low permeability soil liner surface shall be surveyed, before the placement of a geosynthetic, to verify that actual thickness and grades are in accordance with the plans and specifications.

2. A minimum of one cross-section for every 100 lineal feet of cell, measured along the cell length, shall be surveyed. At a minimum, survey points shall be established at the top, mid-point, and bottom of each slope and at three points on the bottom of the cell. These survey points shall be coincident with those of the previous cross-section lines.

3. In addition, a minimum of one slope profile, for every 100 lineal feet of cell, measured along the cell width, shall be surveyed. Slope profiles shall be coincident with the previous slope profiles. At a minimum, survey points shall be established at the top, mid-point, and bottom of each slope and at least one of the survey points on the bottom of the cell established by the longitudinal survey.

4. Acceptable tolerances on survey coordinates shall be +/- 0.2 foot on elevations and +/- 1.0 foot on coordinates.

5. The Quality Assurance technician certifying the survey results shall be a land surveyor or Professional Engineer registered in Arizona.

6. The QA Engineer will certify that the soil liner meets the requirements in the plans and specifications and submit documentation of such to the Company.

C. Leachate Collection and Removal System

1. The completed leachate collection and removal system surfaces shall be surveyed to verify that actual thickness and grades are in accordance with the plans and specifications. Surveys should be completed to determine the elevations at the top of the drainage layer and protective soil cover.

2. A minimum of one cross-section for every 100 lineal feet of cell, measured along the cell length, shall be surveyed. At a minimum, survey points shall be established at the top, mid-point, and bottom of each slope and at three points on the bottom of the cell. These survey points shall be coincident with those of the excavation cross-section lines.

3. In addition, a minimum of one slope profile, for every 100 lineal feet of cell, measured along the cell width, shall be surveyed. Slope profiles shall be coincident with the excavation slope profiles. At a minimum for granular side slope leachate collection and removal systems,



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survey points shall be established at the top, mid-point, and bottom of each slope and at least one of the survey points on the bottom of the cell established by the longitudinal survey.

4. Acceptable tolerances on survey coordinates shall be 0 to +0.2 foot on elevations and +/- 1.0 foot on coordinates.

5. The Quality Assurance technician certifying the survey results shall be a registered land surveyor or Professional Engineer.

6. The QA Engineer will certify that the leachate collection and removal system meets the requirements in the plans and specifications and submit documentation of such to the Company.

Test Method for

Measurement of Hydraulic Conductivity

of Saturated Porous Materials Using a

Flexible Wall Permeameter

Subcommittee: D18.04

Reviewer: David Daniel

Draft No.: 5

Date: March 9, 1989

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## 1. Scope

1.1 This test method covers laboratory measurement of the hydraulic conductivity of water-saturated porous materials with a flexible wall permeameter. (See note 1.)

Note 1 -- The term "hydraulic conductivity" is also referred to as "coefficient of permeability." A more complete discussion of terminology is presented in Section 3.

1.2 This test method may be utilized with undisturbed or compacted specimens that have a hydraulic conducivity less than or equal to $1 \times 10^{-3}$ cm/s.

1.3 The hydraulic conductivity of materials with hydraulic conductivities greater than $1 \times 10^{-3}$ cm/s may be determined by Test Method D 2434.

1.4 The values stated in SI units are to be regarded as the standard. Unless other units are specifically given.

1.5 *This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

## 2. Referenced Documents

2.1 ASTM Standards:

D 653 Terminology Relating to Soil, Rock, and Contained Fluids

D 698 Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 5.5-lb (2.49 kg) Rammer and 12-in (305 mm) Drop

D 854 Test Method for Specific Gravity of Soils

D 1557 Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-lb (4.54 kg) Rammer and 18-in (457 mm) Drop

D 1587 Practice of Thin-Walled Tube Sampling of Soils

D 2113 Practice for Diamond Core Drilling for Site Investigation

D 2216 Method for Laboratory Determination of Water (Moisture) Content in Soil, Rock, and Soil-Aggregate Mixtures

D 2434 Test Method for Permeability of Granular Soils (Constant Head)

D 4220 Practices for Preserving and Transporting Soil Samples

D 4753 Specification for Evaluating, Selecting and Specifying Balances and Scales for Use in Soil and Rock Testing

E 145 Specification for Gravity-Convection and Forced-Ventilation Ovens

D 4767 Test Method for Consolidated-Undrained Triaxial Compression

## 3. Definitions

3.1 *hydraulic conductivity, k* -- measured in units of centimeters per second, is defined from Darcy's law as follows:

$$q = kiA$$

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where:

$q$ = rate of flow, $cm^3/s$,

$i$ = hydraulic gradient (dimensionless)

= $h/L$

$h$ = head loss across test specimen, cm,

$L$ = length of test specimen along the path of water flow, cm,

$A$ = cross-sectional area of test specimen, $cm^2$

Note 2 -- The term "coefficient of permeability" is often used to describe the coefficient $k$ in Darcy's law, but "hydraulic conductivity" is used exclusively in this standard. A more complete discussion of the terminology associated with Darcy's law is given by Olson and Daniel (1981).

3.2 For definitions of other terms used in this test method see Terminology D 653.

## 4. Significance and Use

4.1 This test method provides a means for measuring the hydraulic conductivity of water-saturated porous materials in the laboratory. The test applies to one-dimensional, laminar flow of water through the porous material.

4.2 The hydraulic conductivity of porous materials generally decreases with an increasing amount of air in the pores of the material. This standard applies to water-saturated porous materials containing virtually no air.

4.3 This standard applies to permeation of porous materials with water. Permeation with other liquids, such as chemical wastes, can be accomplished using procedures similar to those described in this standard. However, this standard is only intended to be used when water is the permeant liquid.

4.4 It is assumed that Darcy's law is valid and that the hydraulic conductivity is essentially unaffected by hydraulic gradient.

Note 3 -- The validity of Darcy's law may be evaluated by measuring the hydraulic conductivity of the soil at three hydraulic gradients; if all measured values are similar (within about 25 percent), then Darcy's law may be taken as valid. However, when the hydraulic gradient acting on a test specimen is changed, the state of stress will also change, and, if the specimen is compressible, the volume of the specimen will change. Thus, some change in hydraulic conductivity may occur when the hydraulic gradient is altered even in cases where Darcy's law is valid.

4.5 The correlation between results obtained with this test method and the hydraulic conductivities of in-place field materials has not been fully investigated. Therefore, the results should be applied to field situations with caution and by qualified personnel.

## 5. Apparatus

Note 4: Warning -- Hydraulic conductivity testing using this test method generally requires the use of pressurized apparatus or compressed gases, or both. Appropriate standards of safety should be utilized in completion of all tests using this test method.

5.1 *Hydraulic System* -- Constant head, falling head, or constant rate of flow systems may be utilized provided they meet the criteria outlined as follows:

5.1.1 *Constant Head* -- The system must be capable of maintaining constant hydraulic pressures to within ±5% and shall include means to measure the hydraulic pressures to within the prescribed tolerance. In addition, the head loss across the test specimen must be held constant to within ±5% and shall be measured with the same accuracy or better. Pressures shall be measured by a pressure gage, electronic pressure transducer, or any other device of suitable accuracy.

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5.1.2 *Falling Head* – The system shall allow for measurement of the applied head loss, thus hydraulic gradient, to within 5% or better at any time. The head loss shall be measured with a pressure gage, electronic pressure transducer, engineer's scale, graduated pipette, or any other device of suitable accuracy. Falling head tests may be performed with either a constant tailwater elevation or a rising tailwater elevation.

5.1.3 *Constant Rate of Flow* -- The system must be capable of maintaining a constant rate of flow through the specimen to within 5% or better. Flow measurement shall be by calibrated syringe, graduated pipette, or other device of suitable accuracy. The head loss across the specimen shall be measured to an accuracy of 5% or better using an electronic pressure transducer or other device of suitable accuracy. More information on testing with a constant rate of flow is given by Olsen, Morin, and Nichols (1988).

5.1.4 *System De-airing* -- The hydraulic system shall be designed to facilitate rapid and complete removal of free air bubbles from flow lines.

5.1.5 *Back Pressure System* -- The hydraulic system shall have the capability to apply back pressure to the specimen to facilitate saturation. The system shall be capable of maintaining the applied back pressure throughout the duration of hydraulic conductivity measurements. The back pressure system shall be capable of applying, controlling, and measuring the back pressure to 5% or better of the applied pressure. The back pressure may be provided by a compressed gas supply, a deadweight acting on a piston, or any other method capable of applying and controlling the back pressure to the tolerance prescribed in this paragraph.

Note 5 -- Application of gas pressure directly to the back pressure fluid supply will dissolve gas in the back pressure fluid. A variety of techniques are available to minimize dissolution of gas in the back pressure fluid, including separation of gas and liquid phases with a bladder and frequent replacement of the liquid with de-aired water.

5.2 *Flow Measurement System* -- Both inflow and outflow volumes shall be measured unless the lack of leakage, continuity of flow, and cessation of consolidation or swelling can be verified by other means. Flow volumes shall be measured by a graduated accumulator, graduated pipette, vertical standpipe in conjunction with an electronic pressure transducer, or other volume-measuring device of suitable accuracy.

5.2.1 *Flow Accuracy* -- Requird accuracy for the quantity of flow measured over an interval of time is 5% or better.

5.2.2 *System De-airing and Compliance* -- The flow-measurement system shall contain a minimum of dead space and be capable of complete and rapid system de-airing. System compliance in response to system pressure changes shall be minimized by using a stiff flow measurement system. Rigid tubing, such as metallic or rigid thermoplastic tubing, shall be used.

5.2.3 *Head Losses* -- Head losses in the tubes, valves, porous end pieces, and filter paper may lead to error. To guard against such errors, the permeameter shall be assembled with no specimen inside and then the hydraulic system filled. If a constant or falling head test is to be used, the hydraulic pressures or heads that will be used in testing a specimen shall be applied, and the rate of flow measured with an accuracy of 5% or better. This rate of flow shall be at least ten times greater than the rate of flow that is measured when a specimen is placed inside the permeameter and the same hydraulic pressures or heads are applied. If a constant rate of flow test is to be used, the rate of flow to be used in testing a specimen shall be supplied to the permeameter and the head loss measured. The head loss without a specimen shall be less the 0.1 times the head loss when a specimen is present.

5.3 *Permeameter Cell Pressure System* -- The system for pressurizing the permeameter cell shall be capable of applying and controlling the cell pressure to within 5% of the applied pressure. However, the effective stress on the test specimen (which is the difference between the cell pressure and the pore water pressure) shall be maintained to the desired value with an accuracy of 10% or better. The device for pressurizing the cell may consist of a reservoir

connected to the permeameter cell and partially filled with de-aired water, with the upper part of the reservoir connected to a compressed gas supply or other source of pressure (see Note 6). The gas pressure shall be controlled by a pressure regulator and measured by a pressure gage, electronic pressure transducer, or any other device capable of measuring to the prescribed tolerance. A hydraulic system pressurized by deadweight acting on a piston or any other pressure device capable of applying and controlling the permeameter cell pressure to the tolerance prescribed in this paragraph may be used.

Note 6 -- De-aired water is commonly used for the cell fluid to minimize potential for diffusion of air through the rubber membrane into the specimen. Other fluids, such as oils, which have low gas solubilities are also acceptable, provided they do not react with components of the permeameter.

5.4 *Permeameter Cell* -- An apparatus shall be provided in which the specimen and porous end pieces, enclosed by a membrane sealed to the cap and base, are subjected to controlled fluid pressures.

Note 7 -- The permeameter cell may allow for observation of changes in height of the specimen, either by observation through the cell wall using a cathetometer or other instrument, or by monitoring of either a loading piston or an extensometer extending through the top plate of the cell bearing on the top cap and attached to a dial indicator or other measuring device. The piston or extensometer should pass through a bushing and seal incorporated into the top plate and shall be loaded with sufficient force to compensate for cell pressure acting on the piston tip. If deformations are measured, the deformation indicator shall be a dial indicator or cathetometer graduated to 0.3 mm (0.01 in) or better and having an adequate travel range. Any other measuring device meeting these requirements is acceptable.

Note 8 -- In order to facilitate gas removal, and thus saturation of the hydraulic system, four drainage lines leading to the specimen, two each to the base and top cap, are recommended. The drainage lines shall be controlled by no-volume-change valves, such as ball valves, and shall be designed to minimize dead space in the lines.

5.5 *Specimen Top Cap and Base* -- An impermeable, rigid top cap and base shall be used to support the specimen and provide for transmission of permeant liquid to and from the specimen. The diameter or width of the top cap and base shall be equal to the diameter or width of the specimen ± 5%. The base shall prevent leakage, lateral motion, or tilting, and the top cap shall be designed to receive the piston or extensometer, if used, such that the piston-to-top cap contact area is concentric with the cap. The surface of the base and top cap that contacts the membrane to form a seal shall be smooth and free of scratches.

5.6 *Flexible Membranes* – The flexible membrane used to encase the specimen shall provide reliable protection against leakage. The membrane shall be carefully inspected prior to use and if any flaws or pinholes are evident, the membrane shall be discarded. To minimize restraint to the specimen, the diameter or width of the unstretched membrane shall be between 90 and 95% of that of the specimen. The membrane shall be sealed to the specimen base and cap with rubber 0-rings for which the unstressed, inside diameter or width is less than 90% of the diameter or width of the base and cap, or by any other method that will produce an adequate seal.

Note 9 -- Membranes may be tested for flaws by placing them around a form sealed at both ends with rubber 0-rings, subjecting them to a small air pressure on the inside, and then dipping them into water. If air bubbles come up from any point on the membrane, or if any visible flaws are observed, the membrane shall be discarded.

5.7 *Porous End Pieces* -- The porous end pieces shall be of silicon carbide, aluminum oxide, or other material that is not attacked by the specimen or permeant liquid. The end pieces shall have plane and smooth surfaces and be free of cracks, chips, and nonuniformities. They shall be checked regularly to ensure that they are not clogged.

5.7.1 The porous end pieces shall be the same diameter or width (±5%) as the specimen, and the thickness shall be sufficient to prevent breaking.

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5.7.2 The hydraulic conductivity of the porous end pieces shall be significantly greater than that of the specimen to be tested. The requirements outlined in 5.2.3 ensure this.

5.8 *Filter Paper* -- If necessary to prevent intrusion of material into the pores of the porous end pieces, one or more sheets of filter paper shall be placed between the top and bottom porous end pieces and the specimen. The paper shall have a hydraulic conductivity significantly higher than that of the specimen to be tested. The requirements outlined in 5.2.3 ensure this is the case.

5.9 *Equipment for Compacting a Specimen* -- Equipment (including compactor and mold) suitable for the method of compaction specified by the requester shall be used.

5.10 *Sample Extruder* -- When the material being tested is a soil core, soil core shall be usually removed from the sampler with an extruder. The sample extruder shall be capable of extruding the soil core from the sampling tube in the same direction of travel in which sample entered the tube and with minimum disturbance of the sample. If the soil core is not extruded vertically, care should be taken to avoid bending stresses on the core due to gravity. Conditions at the time of sample extrusion may dictate the direction of removal, but the principal concern is to keep the degree of disturbance minimal.

5.11 *Trimming Equipment* -- Specific equipment for trimming the specimen to the desired dimensions will vary depending on quality and characteristics of the sample; however, the following items listed may be used: lathe, wire saw with a wire about 0.3 mm (0.01 in) in diameter, spatulas, knives, steel rasp for very hard clay specimens, cradle or split mold for trimming specimen ends, and steel straight edge for final trimming of specimen ends.

5.12 *Devices for Measuring the Dimensions of the Specimen* -- Devices used to measure the dimensions of the specimen shall be capable of measuring to the nearest 0.3 mm (0.01 in) and shall be constructed such that their use will not disturb the specimen.

5.13 *Balances* -- The balance shall be suitable for determining the mass of the specimen and shall be selected as discussed in D4753. The mass of specimens less than 100 g shall be determined to the nearest 0.01 g. The mass of specimens 100 g or larger shall be determined to the nearest 0.1 g. The mass of specimens >1000 g shall be determined to the nearest 1.0 g.

5.14 *Equipment for Mounting the Specimen* -- Equipment for mounting the specimen in the permeameter cell shall include a membrane stretcher or cylinder, and ring for expanding and placing O-rings on the base and top cap to seal the membrane.

5.15 *Vacuum Pump* -- To assist with de-airing of permeameter system and saturation of specimens.

5.16 *Temperature Maintaining Device* -- The temperature of the permeameter, test specimen, and reservoir of permeant liquid shall not vary more than ±3°C (± 5.7°F). Normally, this is accomplished by performing the test in a room with a relatively constant temperature. If such a room is not available, the apparatus shall be placed in a water bath, insulated chamber, or other device that maintains a temperature within the tolerance specified above. The temperature shall be periodically measured and recorded.

5.17 *Water Content Containers* -- The containers shall be in accordance with Method D 2216.

5.18 *Drying Oven* -- The oven shall be in accordance with Specification E145.

## 6. Reagents

6.1 *Permeant Water:*

6.1.1 The permeant water is the liquid used to permeate the test specimen and is also the liquid used in backpressuring the specimen.

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6.1.2 *Calcium Sulfate Solution* -- The type of permeant water should be specified by the requestor. If no specification is made, an aqueous solution of 0.005 N calcium sulfate in deaired, distilled water shall be used for the permeant liquid. This solution may be obtained, for example, by dissolving 0.8 g of non-hydrated, reagent-grade calcium sulfate in 10 L of de-aired, distilled water. The type of water utilized shall be indicated in the report.

6.1.3 *Deaired Water* -- To aid in removing as much air from the test specimen as possible, deaired water shall be used. The water is usually deaired by boiling or by storing the water in a chamber subjected to a vacuum. The 0.005 N calcium sulfate solution shall not be deaired by boiling this solution because boiling will change the concentration of the solution. If boiling is used, the distilled water shall be boiled prior to adding calcium sulfate. The exposure of deaired water to a source of air shall be minimized to prevent dissolution of air back into the water.

Note 10 -- Chemical interactions between a permeant liquid and the porous material may lead to variations in hydraulic conductivity. Permeation of the test specimens with other liquids may result in different hydraulic conductivities.

## 7. Test Specimen

7.1 *Size* -- Specimens shall have a minimum diameter of 25 mm (1.0 in) and a minimum height of 13 mm (0.5 in). The height and diameter of the specimen shall be measured to the nearest 0.3 mm (0.01 in) or better. The diameter and height of the specimen shall each be at least 6 times greater than the largest particle size within the specimen.

Note 11 -- Use of small test specimens can lead to erroneous results if the specimen is not large enough to contain a representative distribution of the features that control hydraulic conductivity, e.g., fissures, joints, and slickensides. The minimum diameter and height given in Section 7.1 are based on physical limitations of the measurement system. In general, a test specimen should be sufficiently large to be representative (in terms of hydraulic conductivity) of the material in the field.

7.2 *Undisturbed Specimens* :

7.2.1 Undisturbed test specimens shall be prepared from undisturbed samples recovered from subsurface borings or undisturbed block samples recovered from shallow test pits, as described in Practice D 1587, Practice D 2113, and Practice D 4220.

7.2.2 *Storage* -- Samples shall be stored in sealed sampling tubes so that no moisture is lost during storage. The time of storage shall be minimized, particularly when the soil or soil moisture is expected to react with the sample tubes.

7.3 *Laboratory-Compacted Specimens:*

7.3.1 *Material Clod Sizes* -- Large clods of material should not be broken down prior to compaction unless it is known that they will be broken in field construction, as well. However, hard clods of the material or large particles in the material should not exceed 1/6 of either the height or diameter of the specimen.

7.3.2 *Mixing and Hydration of Soil* -- Measured amounts of the material to be tested and water shall be carefully and thoroughly mixed and then hydrated overnight or longer.

7.3.3 *Compaction Method* -- The material to be tested shall be prepared and compacted inside the mold in a manner specified by the requestor. If the specimen is placed and compacted in layers, the surface of each previously-compacted layer shall be lightly scarified (roughened) with a fork, ice pick, or other suitable object, unless the requester specifically states that scarification is not to be performed. Test Methods D 698 and D 1557 describe two methods of compaction, but any other method specified by the requestor may be used as long as the method is described in the report.

7.3.3.1 The water content of the material to be tested shall be determined in accordance with Method D 2216.

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7.4 *Trimming:*

7.4.1 *Preparation Environment* -- Specimens shall be trimmed, whenever possible, in an environment where changes in moisture content are minimized. A controlled high-humidity room is usually used for this purpose.

7.4.2 Laboratory-compacted or undisturbed samples may be tested without trimming, except for squaring of the ends, provided they have a uniform cross section with ends perpendicular to the axis of the specimen. Devices such as a lathe may be used as an aid in trimming the specimen to a reduced diameter or width. A cradle or split mold is suggested for trimming of the ends of the specimen.

7.4.3 When trimming the ends of the specimen, care shall be exercised to cut the material rather than troweling or smearing it. Where coarse materials exist or crumbling results in excessive irregularity at the ends or deviation from the cross section, the specimen shall be discarded.

7.4.4 The trimmed specimen shall have a length and diameter (or width) that vary by no more than ±5%.

7.4.5 The trimmed surface may be uneven, but indentations in the trimmed surface on the specimen ends must not be deeper than 5% of the length of the specimen for trimming to be considered acequate. Trimmed material shall be used to determine and record the moisture content of the specimen by the procedure specified in Method D 2216.

7.4.6 The mass of the specimen shall be determined (see 5.13) and the dry density and degree of saturation calculated and recorded. The specimen shall then be immediately placed in the permeameter cell and enclosed in the flexible membrane (See 8.1).

## 8. Procedure

8.1 *Specimen Set-Up:*

8.1.1 Cut two filter paper sheets to approximately the same shape as the cross section of the test specimen. Soak the two porous end pieces and filter paper sheets, if used, in a container of de-aired permeant water.

8.1.2 Place the membrane on the membrane expander. Apply a thin coat of silicon high-vacuum grease to the sides of the end caps. Place one porous end piece on the base and place one filter paper sheet, if used, on the porous end piece, followed by the test specimen. Place the second filter paper sheet, if used, on top of the specimen followed by the second porous end piece and the top cap. Place the membrane around the specimen, and using the membrane expander or other suitable O-ring expander, place one or more O-rings to seal the membrane to the base and one or more additional O-rings to seal the membrane to the top cap.

8.1.3 Attach flow tubing to the top cap, if not already attached, assemble the permeameter cell, and fill it with de-aired water or other cell fluid. Attach the cell pressure reservoir to the permeameter cell line and the hydraulic system to the influent and effluent lines. Fill the cell pressure reservoir with deaired water, or other suitable liquid, and the hydraulic system with deaired permeant water. Apply a small confining pressure of 7 to 35 kPa (1 to 3 psi) to the cell and apply a pressure less than the confining pressure to both the influent and effluent systems, and flush permeant water through the flow system. After all visible air has been removed from the flow lines, close the control valves. At no time during saturation of the system and specimen or hydraulic conductivity measurements shall the maximum applied effective stress be allowed to exceed that to which the specimen is to be consolidated.

8.2 *Specimen Soaking (Optional)* -- To aid in saturation, specimens may be soaked under partial vacuum applied to the top of the specimen. Atmospheric presure shall be applied to the specimen base through the influent lines, and the magnitude of the vacuum set to generate a hydraulic gradient across the sample less than that which will be used during hydraulic conductivity measurements.

Note 12 -- Soaking under vacuum is applicable when there are continuous air voids in the specimen. Soaking under vacuum is only recommended for test specimens with initial degrees of saturation below 70%.

8.3 *Backpressure Saturation* -- To saturate the specimen, backpressuring is usually necessary. Figure 1 provides guidance on back pressure required to attain saturation.

Note 13 -- Figure 1 assumes that the water used for back pressure is deaired and that the only source for air to dissolve into the water is air from the test specimen. If air pressure is used to control the back pressure, pressurized air will dissolve into the water, thus reducing the capacity of the water used for back pressure to dissolve air located in the pores of the test specimen. The problem is minimized by using a long (> 5 m) tube that is impermeable to air between the air-water interface and test specimen, by separating the back-pressure water from the air by a material or fluid that is relatively impermeable to air, by periodically replacing the back-pressure water with deaired water, or by other means.

8.3.1 Open the flow line valves and flush out of the system any free air bubbles using the procedure outlined in 8.1.3. If an electronic pressure transducer or other measuring device is to be used during the test to measure pore pressures or applied hydraulic gradient, it should be bled of any trapped air. Take and record an initial reading of specimen height, if being monitored.

8.3.2 Adjust the applied confining pressure to the value to be used during saturation of the sample. Apply backpressure by simultaneously increasing the cell pressure and the influent and effluent pressures in increments. The maximum value of an increment in backpressure shall be sufficiently low so that no point in the specimen is exposed to an effective stress in excess of that to which the specimen will be subsequently consolidated (see Note 14). Maintain each increment of pressure for a period of a few minutes to a few hours, depending upon the characteristics of the specimen. To assist in removal of trapped air, a small hydraulic gradient may be applied across the specimen to induce flow.

Note 14 -- At no time shall a head be applied so that the effective confining stress is less than 7 kPa (1 psi) because of the danger of separation of the membrane from the test specimen.

8.3.3 (Optional) Verify saturation by measuring the B coefficient as described in ASTM standard D4767. The soil shall be considered to be adequately saturated if (1) the B value is 0.95 or greater, or (2) for relatively incompressible materials, e.g., rock, if the B value remains unchanged with application of larger values of back pressure.

Note 15 -- The B coefficient is defined for this type of test as the change in pore water pressure in the porous material divided by the change in confining pressure. Compressible materials that are fully saturated with water will have a B value of 1.0. Relatively incompressible, saturated matenals have B values which are somewhat less than 1.0.

Note 16 -- The saturation of the test specimen shall be confirmed at the completion of the test by calculation of the final degree of saturation. However, measurement of the B coefficient is recommended in addition to calculation of the final degree of saturation because: (1) it is a more accurate method for verifying saturation than computation of the degree of saturation (this computation is very sensitive to small errors in the specific gravity of solids), and (2) saturation can be verified before rather than after the hydraulic conductivity measurements.

Note 17 -- If desired, the B value can be measured after the test specimen has been consolidated. Accurate B-value determination can only be made if no gradient is acting on the specimen and all pore pressure induced by consolidation has dissipated.

8.4 *Consolidation* -- Consolidate the specimen to the effective stress specified by the requestor. Consolidation may be accomplished in stages, if desired.

Note 18 -- The test specimen may be consolidated prior to application of back pressure. Also, the back pressure and consolidation phases may be

completed concurrently if backpressures are applied sufficiently slowly to minimize potential for overconsolidation of the specimen.

8.4.1 Record the specimen height, if being monitored, prior to application of consolidation pressure and periodically during consolidation.

8.4.2 Increase the cell pressure to the level necessary to develop the desired effective stress, and begin consolidation. Drainage may be allowed from the base or top of the specimen, or simultaneously from both ends.

8.4.3 (Optional) Record outflow volumes to confirm that primary consolidation has been completed prior to initiation of the hydraulic conductivity test.

Note 19 – The reason why 8.4.3 is optional is that the requirements of 8.5 ensure that the test specimen is adequately consolidated during permeation because if it is not, inflow and outflow volumes will differ significantly. However, for accurate B-value determination, completion of consolidation should be confirmed (See Note 18).

8.4.4 *(Optional)* To prevent the occurence of additional consolidation during specimen permeation, the applied cell pressure may be reduced and the specimen allowed to rebound. However, at no time shall the specimen be consolidated to an effective stress greater than the maximum value specified by the requestor or existing at the effluent end of the test specimen during permeation.

8.5 *Permeation:*

8.5.1 *Hydraulic Gradient* -- When possible, the hydraulic gradient used for hydraulic conductivity measurements should be similar to that expected to occur in the field. In general, hydraulic gradients from <1 to 5 cover most field conditions. However, the use of small hydraulic gradients can lead to very long testing times for materials having low hydraulic conductivity (less than about $1 \times 10^{-6}$ cm/s). Somewhat larger hydraulic gradients are usually used in the laboratory to accelerate testing, but excessive gradients

must be avoided because high seepage pressures may consolidate the material, material may be washed from the specimen, or fine particles may be washed downstream and plug the effluent end of the test specimen. If no gradient is specified by the requestor, the following guidelines may be followed:

| Hydraulic Conductivity, cm/s | Recommended Maximum Hydraulic Gradient |
|---|---|
| $1 \times 10^{-3}$ to $1 \times 10^{-4}$ | 2 |
| $1 \times 10^{-4}$ to $1 \times 10^{-5}$ | 5 |
| $1 \times 10^{-5}$ to $1 \times 10^{-6}$ | 10 |
| $1 \times 10^{-6}$ to $1 \times 10^{-7}$ | 20 |
| less than $1 \times 10^{-7}$ | 50 |

Note 20 -- Seepage pressures associated with large hydraulic gradients can consolidate soft, compressible specimens and reduce their hydraulic conductivity. It may be necessary to use smaller hydraulic gradients (less than 10) for such specimens.

8.5.2 *Constant Head and Falling Head-Tests* -- Initiate permeation of the specimen by increasing the influent pressure (see Notes 13 and 21). Measure and record the applied head loss across the test specimen. For constant-head tests, the head loss across the specimen shall be kept constant ±5%. For falling-head tests, at no time shall the applied head loss across the specimen be less than 75% of the initial (maximum) head loss during each individual hydraulic conductivity determination. Measure periodically and record the quantity of inflow and outflow, as well as any changes in the height of the specimen, if being monitored. Continue permeation until: (1) the ratio of inflow to outflow rates or flow volumes is between 0.75 and 1.25, and (2) the hydraulic conductivity is steady (see Note 22). A plot of hydraulic conductivity

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versus time or pore volumes of flow is recommended to verify that hydraulic conductivity is steady.

Note 21 -- The effluent pressure should not be decreased because air bubbles that were dissolved by the soil water during backpressuring may come out of solution if the pressure is decreased. The back pressure shall be maintained throughout the permeation phase.

Note 22 -- The hydraulic conductivity shall be considered steady if four or more consecutive hydraulic conductivity determinations fall within ±25% of the mean value for $k \geq 1 \times 10^{-8}$ cm/s or within ± 50% for $k < 1 \times 10^{-8}$ cm/s, and a plot of the hydraulic conductivity versus time shows no significant upward or downward trend.

8.5.3 *Constant Rate of Flow Tests* -- Initiate permeation of the specimen by imposing a constant flow rate (see Note 20). Choose the flow rate so the hydraulic gradient does not exceed the value specified, or if none is specified, the value recommended in 8.5.1 (see Note 19). Measure periodically and record the quantity of outflow, as well as any changes in specimen height, if being monitored. Measure periodically the pressure gradient developed across the specimen. Continue permeation until (1) the ratio of inflow to outflow rates is between 0.75 and 1.25, and (2) hydraulic conductivity is steady (see Note 22).

8.6 *Final Dimensions of the Specimen* -- After completion of permeation, reduce the applied confining, influent, and effluent pressures in a manner that does not generate significant volume change of the test specimen. Then carefully disassemble the permeater cell and remove the specimen. Measure and record the final height, diameter, and total mass of the specimen. Then determine the final water content of the specimen by the procedure of Method D 2216. Dimensions and mass of the test specimen shall be measured to the tolerances specified in 5.13 and 7.1.

Note 23 -- The specimen may swell after removal of back pressure as a result of air coming out of solution. A correction may be made for this effect, provided that changes in the length of the specimen are monitored during the

test. The strain caused by dismantling the cell is computed from the length of the specimen before and after dismantling the cell. The same strain is assumed to have occured in the diameter. The corrected diameter and actual length before the back pressure was removed are used to compute the volume of the test specimen prior to dismantling the cell. The volume prior to dismantling the cell is used to determine the final dry density and degree of saturation.

## 9. Calculations

9.1 *Constant Head and Constant Rate of Flow Tests* -- Calculate the hydraulic conductivity, *k*, as follows:

$$k = QL/Ath$$

where:

*k* = hydraulic conductivity, cm/s,

*Q* = quantity of flow, taken as the average of inflow and outflow, $cm^3$,

*L* = length of specimen along path of flow, cm,

*A* = cross-sectional area of specimen, $cm^2$,

*t* = interval of time, s, over which the flow Q occurs,

*h* = difference in hydraulic head across the specimen, cm of water

9.2 *Falling Head Tests:*

9.2.1 *Falling Head (Constant Tailwater Elevation)* -- Calculate the hydraulic conductivity, *k*, as follows:

$$k = \frac{aL}{At} \ln\left(\frac{h_1}{h_2}\right)$$

where:

*a* = cross-sectional area of the reservoir containing the influent liquid, $cm^2$

*L* = length of the specimen, cm,

*A* = cross-sectional area of the specimen, $cm^2$,

*t* = elapsed time between determination of $h_1$ and $h_2$, s

$h_1$ = head loss across the specimen, cm, at time $t_1$ and

$h_2$ = head loss across the specimen at time $t_2$, cm.

9.2.2 *Falling Head (Rising Tailwater Elevation)* -- Calculate the hydraulic conductivity, $k$, as follows:

$$k = \frac{a_{in}\, a_{out}\, L}{A\, t\, (a_{in} + a_{out})} \ln (h_1/h_2)$$

where:

$a_{in}$ = cross-sectional area of the reservoir containing the influent liquid, $cm^2$

$a_{out}$ = cross-sectional area of the reservoir containing the effluent liquid, $cm^2$,

$L$ = length of the specimen, cm,

$A$ = cross-sectional area of the specimen, $cm^2$,

$t$ = elapsed time between determination of $h_1$ and $h_2$, s,

$h_1$ = head loss across the specimen at time t, cm, and

$h_2$ = head loss across the specimen at time $t_2$, cm,

Note 24 -- For the case in which $a_{out} = a_{in} = a$, the equation for calculating k for a falling head test with a rising tailwater level is:

$$k = \frac{a\, L}{2\, A\, t} \ln \left(\frac{h_1}{h_2}\right)$$

Note that this equation differs from the one given in 9.2.1 for a falling head test with a constant tailwater level by a factor of 2.

9.3 Correct the hydraulic conductivity to that for 20°C (68°F), $k_{20}$, by multiplying $k$ by the ratio of the viscosity of water at test temperature to the viscosity of water at 20°C (68°F), $R_T$, from Table 1, as follows:

$$k_{20} = R_T k$$

## 10. Report

10.1 The report shall include the following information:

10.1.1 Sample identifying information.

10.1.2 Any special selection and preparation process, such as removal of stones or other materials, or indication of their presence, for undisturbed specimen.

10.1.3 Descriptive information on method of compaction.

10.1.4 The initial dimensions of the specimen.

10.1.5 The initial moisture content and dry unit weight of the specimen.

10.1.6 The type of permeant liquid used.

10.1.7 Magnitude of total back pressure.

10.1.8 Maximum and minimum effective consolidation stress.

Note 25 -- The maximum effective stress exists at the effluent end of the test specimen and the minimum stress at the influent end.

10.1.9 Height of specimen after completion of consolidation, if monitored.

10.1.10 The range of hydraulic gradient used.

10.1.11 The final length, diameter, moisture content, dry unit weight, and degree of saturation of the test specimen (see Note 23).

10.1.12 The average hydraulic conductivity for the last four determinations of hydraulic conductivity (obtained while inflow and outflow rates are equal and while hydraulic conductivity is steady, as described in 8.5.2 and 8.5.3), reported with two significant figures, for example, $7.1 \times 10^{-8}$ cm/s, and reported in units of cm/s (plus additional units, if requested).

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10.1.13 (Recommended) A graph or table of hydraulic conductivity versus time or pore volumes of flow.

## 11. Precision and Bias

11.1 Because hydraulic conductivity of porous materials is difficult to determine, especially with materials of low hydraulic conductivity, this test method has been written with the intent of achieving an overall precision in measured hydraulic conductivity of 50% or better.

11.2 No testing has, as yet, been conducted using this test method to determine multilaboratory precision. Intralaboratory and interlaboratory precision and bias have not been established.

## REFERENCES


(1) Lowe, S., and Johnson, T.C., "Use of Back Pressure to Increase Degree of Saturation of Triaxial Test Specimens," ASCE Research Conference on Shear Strength of Cohesive Soils, University of Colorado, Boulder, CO, June, 1960.

2) Bishop, A.W., and Henkel, D.J., The Measurement of Soil Properties in the Triaxial Test, Edward Arnold, London, 1962, 227 p.

3) Head, K.H. Manual of Soil Laboratory Testing, Vol. 3 Effective Stress Tests, John Wiley and Sons, New York, 1986.

(4) Olson, R. E, and D.E. Daniel, "Measurement of the Hydraulic Conductivity of Fine-Grained Soils," ASTM STP 746, 1981, pp. 18-64.

5) Olsen, H.W., Morin, R.H., and R.W. Nichols, "Flow Pump Applications in Triaxial Testing," ASTM STP 977, 1988, pp. 68-81.

TABLE 1 Correction Factor $R_T$ for Viscosity of Water at Various Temperatures

| Temperature (degrees C) | $R_T$ | Temperature (degrees C) | $R_T$ |
|---|---|---|---|
| 0 | 1.783 | 25 | 0.889 |
| 1 | 1.723 | 26 | 0.869 |
| 2 | 1.664 | 27 | 0.850 |
| 3 | 1.611 | 28 | 0.832 |
| 4 | 1.560 | 29 | 0.814 |
| 5 | 1.511 | 30 | 0.797 |
| 6 | 1.465 | 31 | 0.780 |
| 7 | 1.421 | 32 | 0.764 |
| 8 | 1.379 | 33 | 0.749 |
| 9 | 1.339 | 34 | 0.733 |
| 10 | 1.301 | 35 | 0.719 |
| 11 | 1.265 | 36 | 0.705 |
| 12 | 1.230 | 37 | 0.692 |
| 13 | 1.197 | 38 | 0.678 |
| 14 | 1.165 | 39 | 0.665 |
| 15 | 1.135 | 40 | 0.653 |
| 16 | 1.106 | 41 | 0.641 |
| 17 | 1.077 | 42 | 0.629 |
| 18 | 1.051 | 43 | 0.618 |
| 19 | 1.025 | 44 | 0.607 |
| 20 | 1.000 | 45 | 0.598 |
| 21 | 0.976 | 46 | 0.585 |
| 22 | 0.953 | 47 | 0.575 |
| 23 | 0.931 | 48 | 0.565 |
| 24 | 0.910 | 49 | 0.556 |

Note: $R_T = (-0.02452\ T - 1.495)$ where T is the °C.

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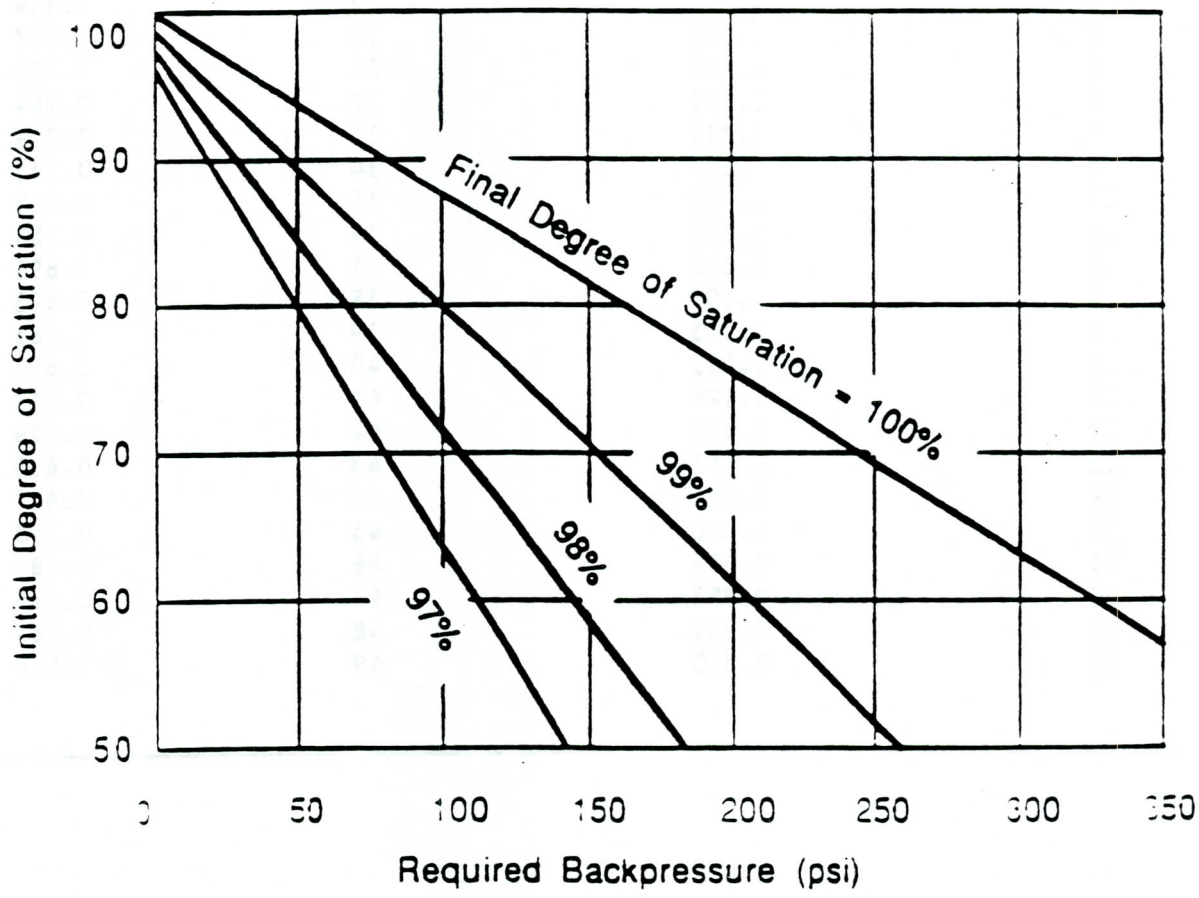


Fig. 1. Back Pressure to Attain Various Degrees of Saturation (from Lowe and Johnson, 1960).

Attachment 11

# QUALITY ASSURANCE PROCEDURES FOR SYNTHETIC CONTAINMENT CONSTRUCTION, CHOLLA SANITARY LANDFILL

# QUALITY ASSURANCE PROCEDURES
# FOR
# SYNTHETIC CONTAINMENT SYSTEM CONSTRUCTION
# CHOLLA SANITARY LANDFILL
# BROWNING-FERRIS INDUSTRIES OF ARIZONA
# EL MIRAGE, ARIZONA

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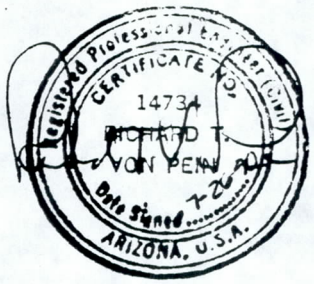


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# I. INTRODUCTION

## A. Purpose

The purpose of this document is to outline the minimum inspection requirements needed to ensure and certify that the design requirements of the Cholla Sanitary Landfill are met or exceeded during construction.

## B. Scope

This document addresses the test methods, test frequencies, and documentation necessary to ensure adherence to the synthetic construction specifications during installation of the landfill synthetic material components. Protocol for reporting test results certifying compliance with construction specifications, correcting construction deficiencies, and documenting such corrections are also provided.

## C. Definitions

1. The Company: Browning-Ferris Industries of Arizona, Inc. The party for whom the work is to be performed.

2. The Engineer: A designated technical on-site representative of the Company.

3. Quality Control (QC): The formal procedures utilized by project contractors and vendors to ensure conformance of their products to plans, specifications, or other criteria established for the project. Note that while vendors or contractors may have multiple levels in their product verification programs, for purposes of this plan, the entire effort shall be considered Quality Control. Vendor and Contractor Quality Control programs are not a part of this plan, although they will be reviewed to ensure general conformance with the objectives stated herein.



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4. Quality Assurance (QA): The formal organization and procedures utilized by the Company for verifying that work performed meets project requirements. The Quality Assurance effort will generally include a comprehensive and continuous review of contractor/vendor QC programs and results, as well as construction materials. Independent verification of test results shall be accomplished through inspection and materials testing.

5. Quality Assurance (QA) Engineer: Third party, independent consulting engineering firm providing monitoring of all geosynthetic aspects of landfill construction, technically accountable to the Company. The QA Engineer is responsible for certification that cell construction is completed according to the specifications.

6. Engineering Services: A corporate department of Browning-Ferris Industries with civil/geotechnical engineering and engineering geology/hydrology capabilities, providing technical assistance to BFI of Arizona.

7. Geomembrane: An impermeable membrane liner or barrier used in civil engineering for geotechnical projects. It can be reinforced with a fabric scrim for added strength or embossed to provide enhanced friction with soil and geotextiles.

8. Geotextile: A relatively porous construction or reinforcement fabric used in civil engineering for geotechnical projects. The fabrics structure may be knit, woven, non-woven (spun-bonded, etc.) mat or net. Geotextiles are also frequently used as a filter or segregation medium, but may also be used as a drainage medium.

9. Permeability: Ability of pore fluid to travel through a material via interconnected voids. "High" permeability indicates relatively rapid flow, and vice versa. Rates of permeability are generally reported in centimeters per second (cm/sec).

10. Planar Transmissivity: Ability of a material to convey a fluid horizontally in its manufactured plane, measured under a load acting normal to its manufactured plane. Values are dependent on hydraulic gradient across test section and normal load.

D. Quality Control Organization

1. Responsibilities: All contractors/vendors shall be responsible for implementing a field/plant quality control program. Contractors/vendors shall be responsible for supplying materials and/or work products that meet the construction specifications.

2. Submittals: All contractors/vendors shall be responsible for submitting the procedures and results of the quality control program to the QA Engineer for review, approval, and documentation.

   a. Synthetic Materials: Contractors/vendors supplying synthetic materials shall submit the procedures and results of the plant quality control program to the QA Engineer prior to shipment of the material to the site. The QA Engineer shall review the procedures and results of the quality control program and shall notify the contractor/vendor that the material is approved for shipment.

E. Quality Assurance Organization

1. Responsibilities: The Cholla Landfill QA organization, administered by the QA Engineer, is responsible for certification of the final construction as being in conformance with the project plans, specifications, and permit conditions. The QA program by this plan outlines the defined minimum inspection requirements needed to ensure that the project plans and specifications are met or exceeded during construction. The QA program is an independent supplement to contractor/vendor quality control programs and shall not be used in place of contractor/vendor quality control.



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2. Administration: The QA Engineer will be supported directly by a technical staff of engineers and technicians, and indirectly by independent materials testing laboratories.

F. Qualifications

1. QA Personnel:

All Quality Assurance personnel shall be properly trained and qualified to test and inspect landfill construction. The Senior Quality Assurance Engineer should have sufficient education and technical and administrative experience to perform his responsibilities. He should have demonstrated knowledge of specific construction practices relating to geosynthetics, regulations and specifications, observation and testing procedures, and documentation procedures.

Other Quality Assurance personnel need to have adequate formal training in observation and testing procedures. The Quality Assurance firm shall submit qualifications of their planned quality control personnel to the Engineer. These qualifications should include education, experience, and special training. These qualifications shall be kept on permanent file as a record of the construction process.

G. Document Control

1. General:

The QA Engineer will be responsible for the overall administration and control of the project QC and QA documents. The QA Engineer will verify that a QA filing system is implemented and will include, at a minimum, the following:

- File copy of the QA Plan, updated as necessary

- Photographic construction documentation

- Certification by material suppliers and contractors relating to their QC Plan.

- Survey measurements

- Plant, field, and laboratory test results of synthetic materials

- Daily and weekly field reports

- Field certification reports including record drawings

- Deficiency reports and corrective actions

- Minutes of weekly construction meetings

2. Final Documentation:

   Record drawings and acceptance reports will be transmitted as part of the final construction documentation reports for the landfill. These construction documentation reports will be transmitted by the QA Engineer to the Company.

3. Document Control: The QA Engineer shall initiate a project filing system which shall include at a minimum:

   a. File copy of the QA procedures, updated as necessary.

   b. Photographic construction documentation.

   c. Survey measurements.

   d. Field and laboratory test results.

   e. Daily and weekly field results and reports.



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f. Field certification reports including as-built drawings.

g. Non-conformance and corrective action reports.

h. Minutes of construction meetings.

4. Daily Field Reports: The QA Engineer shall prepare daily field reports which shall include, as a minimum, the following:

a. Date, project name, location, and other identification.

b. Data on weather conditions.

c. Reports of any meetings held and any pertinent results.

d. Descriptions and locations of construction underway during the time frame of the daily report.

e. Equipment working on site.

f. Descriptions of areas of work being tested.

g. Description of off-site materials received, including any quality verification documentation.

h. Decisions made regarding approval of units of material or of work, and/or corrective actions to be taken in instances of substandard quality.

i. Signature of the QA Engineer.

5. Test Results: Test results shall be reported on standard sheets and shall include, at a minimum, the following:

a. Description or title of testing activity.

b. Location of the field testing activity or the location from which the sample was obtained.

c. Type of testing activity and procedure used (reference standard method when appropriate).

d. Recorded observation or test data, with all necessary calculations.

e. Results of testing activity and comparison with specification requirements.

f. Signature of test performer and QA Engineer.

6. Final Documentation: At the completion of the project, a final certification report shall be issued by the QA Engineer and transmitted to the Company. This document shall include, at a minimum, the following information:

a. Scope of work.

b. All daily field reports.

c. All laboratory and field test results.

d. Test methods.

e. Evaluation of all test results with respect to project specifications.

f. Any non-conformance reports and corrective action reports.

g. Personnel involved with the project and their respective qualifications.

h. As-built drawings and survey notes.



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i. Certification of final construction as meeting or exceeding construction specifications. This certification should be signed and stamped by a registered, professional engineer.

A. Synthetic Materials

1. General:

a. All synthetic material delivered to the site will be visually inspected by the liner vendor QC representative and the QA Engineer or technician for damage in transit, and checked for agreement with the Bill of Lading. A signed copy of the Bill of Lading accepting synthetic materials as shipped will be kept in the QA files.

b. Any non-conforming material will immediately be segregated and tagged "DO NOT USE." The liner vendor QC representative will contact the QA Manager concerning any non-conforming materials.

c. Manufacturing Quality Control Cards will be available to the QA Engineer or technicians during their inspection of incoming synthetics. Holes, tears, or other visible defects shall be clearly marked and documented. All holes, tears, or other visible defects shall be repaired.

d. After material acceptance, Manufacturing Quality Control Cards will be retained by the synthetic vendor QC representative until project completion. Upon project completion, they will be forwarded to the QA Engineer for placement in the project file.

e. Items which cannot be repaired or modified to meet the construction plans and specifications shall be removed and replaced. All non-conforming situations will be reported to the QA Engineer for initiation of non-conformance reports.

f. All non-conforming conditions will be brought immediately to the attention of the QA Manager by the QA Engineer by means of the Non-Conformance Report. A written disposition will be provided for each non-conforming condition, those actions required to bring the item into conformance, and any appropriate documentation required. Copies of dispositions shall be forwarded to the QA Engineer for placement in the project files.

2. Geomembrane:

a. The Geosynthetic contractor/vendor will be required to submit his Quality Control program to the QA Engineer prior to initiating field work. As a minimum, the Geosynthetic contractor/vendor shall perform the following tests at the following frequencies prior to shipping geomembrane material to the site:

| Property | Test Method | Frequency |
|---|---|---|
| HDPE RESIN | | |
| Specific Gravity | ASTM D792 Method A | Every batch of resin |
| Melt Flow Index | ASTM D1238 Condition E | Every batch of resin |
| HDPE SHEET | | |
| Tensile Properties | ASTM D638 | Every 15,000 ft |
| Carbon Black Content | ASTM D1603 | Every 15,000 ft |
| Puncture Resistance | FTMS 101C Method 2031 | Every 15,000 ft |
| Modulus of Elasticity | ASTM D638 | Every 15,000 ft |
| Tear Resistance | ASTM D1004 Die C | Every 15,000 ft |
| Dimensional Stability | ASTM D1204 | Every 15,000 ft |
| Environmental Stress Crack | 212AF, 15 min. ASTM D1693 | Every 60,000 ft |
| Low Temperature Brittleness | ASTM D746 | Every 60,000 ft |

Test results shall be submitted to the Company prior to shipping the HDPE rolls. Environmental Stress Crack (ASTM D1693) test



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results shall be submitted to the Company within 45 days of shipping.

b. The HDPE geomembrane shall meet or exceed the specifications indicated in Table 1.

3. Geotextile:

a. Geotextiles shall be (1) nonwoven, needlepunched, continuous filament polyester materials, or (2) nonwoven, needlepunched continuous filament polypropylene materials.

b. Geotextile will be tested prior to shipment to ensure that the physical properties of the finished product are in accordance with the construction specifications.

c. The results of the contractor/vendor QC testing shall be submitted to the QA Engineer prior to shipment of the material to the site.

d. For every 20,000 square feet of geotextile produced for installation in the landfill, one coupon at least 5 feet by 4 feet shall be retained intact by the manufacturer until construction of the landfill. The coupon shall be transferred to the Company at the completion of landfill construction.

4. HDPE Pipe:

a. HDPE pipe will be tested prior to shipment to ensure that the physical properties are in accordance with the construction specifications.

b. The results of the contractor/vendor QC testing shall be submitted to the QA Engineer prior to shipment of the material to the site.

c. For every 1,500 lineal feet of pipe produced for use in the landfill, one section of pipe at least 5 feet in length shall be retained intact by the manufacturer until construction of the landfill, in which the pipe is used, is complete. Upon completion, the sample shall be transferred to the Company.

5. ADS Pipe:

a. ADS pipe shall be tested prior to shipment to ensure that the physical properties are in accordance with the construction specifications.

b. The results of the contractor/vendor QC testing shall be submitted to the QA Engineer prior to shipment of the material to the site.

## III. QUALITY ASSURANCE FIELD TESTING OF SYNTHETICS

### A. Material Placement

1. The QA Engineer or technician shall visually inspect the placement and overlap of the synthetic landfill components to certify that the placement and overlap of the components is in accordance with the construction specifications.

2. Geomembrane placement shall not proceed at an ambient temperature below 40 degrees F unless otherwise authorized by the QA Engineer. Geomembrane placement shall not be done during any precipitation, in the presence of excessive moisture (e.g., fog, dew), in an area of ponded water, or in the presence of excessive winds.

3. The Geosynthetics Contractor shall ensure the following:

- No equipment used shall damage the geosynthetics by handling, trafficking, leakage of hydrocarbons, or other means.



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- No personnel working on the geosynthetics shall smoke, wear damaging shoes, or engage in other activities which could damage the geosynthetics.

- The method used to unroll the panels shall not cause scratches or crimps in the geomembrane and shall not damage the supporting soil.

- The prepared surface underlying the geomembrane must not be allowed to deteriorate after acceptance, and must remain acceptable up to the time of geomembrane placement.

- All geosynthetic elements must be clean and free of debris prior to placement of overlying layer.

- The method used to place the geomembrane panels shall minimize wrinkles (especially differential wrinkles between adjacent panels).

- Direct contact with the geosynthetics shall be minimized.

4. Any material damaged during material placement shall be repaired or replaced by the installing contractor/vendor at the discretion of the QA Engineer and under the observation of the QA Engineer or technician.

B. HDPE Seams

1. Field seaming may be extrusion or fusion welding or a combination of these methods. Solvent welding is not acceptable.

2. Extrusion welding apparatus shall be equipped with gauges which indicate the temperature in the apparatus and at the nozzle. The Geosynthetic Contractor shall provide documentation to the QA Engineer regarding extrudate and shall certify in writing that the extrudate is compatible with the specifications, and in any event is comprised of the same resins as the geomembrane sheeting.

3. No horizontal seams will be within 10 feet from the toe of the slope.

4. No seaming should be attempted above 40 degrees C. (104 degrees F.) ambient air temperature. Below 5 degrees C (41 degrees F) the Geosynthetic Contractor must demonstrate that conditions are favorable for seaming.

5. The QA Engineer may require that a moveable protective layer of plastic be placed directly below each overlap of geomembrane that is to be seamed in order to prevent moisture build-up between the sheets.

6. The QA Engineer shall require that any water which may pond behind the geomembrane be removed and shall require repair or replacement of the underlying soil if necessary.

7. The Geosynthetic Contractor shall maintain at least one spare operable seaming apparatus on site. Equipment used for seaming shall not damage the geomembrane.

8. If seam overlap grinding is required, the process shall be completed according to the manufacturer's instructions within one hour of the seaming operation and in a way that does not damage the geomembrane.

9. Any material damaged during the grinding process shall be repaired or replaced by the contractor/vendor responsible for the grinding. The repair or replacement of material shall be made at the discretion of the QA Engineer and under the observation of the QA Engineer or technician.

C. Test Seams

1. Test seams will be made each day prior to commencing field seaming. These seams will be made on fragment pieces of geomembrane liner



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to verify that seaming conditions are adequate. Such test seams will be made at the beginning of each seaming period and, at the QA Engineer's discretion, at least once every four hours, for each seaming apparatus used that day. Also, each seamer will make at least one test seam each day.

2. The test seam sample will be at least 3 feet long by 1 foot wide with the seam centered lengthwise. Six adjoining specimens 1 inch wide each will be die cut from the test seam sample. These specimens, which will be tested with a tensiometer in the field for shear (3 specimens) and peel (3 specimens) by the Geosynthetics Contractor and witnessed by the QA Engineer, should not fail in the seam. If a test seam fails, the entire operations will be repeated. If the additional test seam fails, the seaming apparatus or seamer will not be accepted and will not be used for seaming until the deficiencies are corrected and two consecutive successful full test seams are achieved. Test seam failure is defined as failure of any one of the specimen tested in shear or peel.

3. The QA Engineer will observe all test seam procedures. The remainder of the successful test seam sample will be assigned a number and marked accordingly by the QA Engineer, who will also log the date, hour, ambient temperature, number of seaming unit, name of seamer, and pass or fail description. The sample itself should be retained in the Company's archives. In addition, at least one tested specimen from each test as selected by the QA Engineer will be retained by the QA Engineer. The QA Engineer will transmit these specimens to the Company following acceptance of the geomembrane materials and installation by the Company.

4. The criteria for determining a passing machine welded seam in peel shall be: (a) failure is by Film Tear Bond (FTB), National Sanitation Foundation (NSF), Standard 54, definition 2.15, and (b) yield strength for the seam is not less than 60 percent of the minimum tensile strength at yield given in Table 1, and (c) no greater than 10 percent of

the seam width peels (separates) at any point. The criteria for determining a passing machine welded seam in shear (ASTM D4437) shall be: (a) failure is by FTB, and (b) yield strength for the seam is not less than 90 percent of the minimum tensile strength at yield given in Table 1, and (c) yield strain for the seam is at least 10 percent, and (d) break strain for the seam is at least 50 percent.

5. The criteria for determining a passing hand welded seam in peel shall be: (a) failure is by FTB, and (b) yield strength for the seam is not less than 60 percent of the minimum tensile strength at yield given in Table 1, and (c) no greater than 1/8 inches separation occurs from the edge of the sheet at any point. The criteria for determining a passing hand welded seam in shear (ASTM D4437) shall be: (a) failure is by FTB, and (b) yield strength for the seam is not less than 90 percent of the minimum tensile strength at yield given in Table 1, and (c) yield strain for the seam is at least 10 percent, and (d) break strain for the seam is at least 50 percent.

D. Non-Destructive Seam Testing Procedures

1. The Geosynthetic Contractor shall, under the observation of the QA Engineer or technician, nondestructively test all field seams over their full length using a vacuum test unit, air pressure device, or other approved method. Continuity testing shall be carried out as the seaming work progresses, not at the completion of all field seaming.

2. The following procedures shall apply to locations where seams cannot be nondestructively tested:

   a. All such seams shall be cap-stripped with the same geomembrane.

   b. If the seam is accessible to testing equipment prior to final installation, the seam shall be nondestructively tested prior to final installation.



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c. If the seam cannot be tested prior to final installation, the seaming and cap-stripping operations shall be observed by the Geosynthetic Quality Assurance Technician for uniformity and completeness.

3. Vacuum Box Testing

a. The equipment shall be comprised of the following:

- A vacuum box assembly consisting of a rigid housing, a transparent viewing window, a soft neoprene gasket attached to the bottom, port hole or valve assembly, and a vacuum gauge.

- A steel vacuum tank and pump assembly equipped with a pressure controller and pipe connections.

- A rubber pressure/vacuum hose with fittings and connections.

- A bucket and wide paint brush.

- A soapy solution.

b. The following procedures shall be followed:

- Energize the vacuum pump and reduce the tank pressure to approximately 10 inches of mercury, i.e., 5 psi gauge (35 kPa).

- Wet a strip of geomembrane approximately 12 inches by 48 inches (0.3m by 1.2m) with the soapy solution.

- Place the box over the wetted area.

- Close the bleed valve and open the vacuum valve.

- Ensure that a leak tight seal is created.

- For a period of not less than 30 seconds, examine the geomembrane through the viewing window for the presence of soap bubbles.

- If no bubble appears after 30 seconds, close the vacuum valve and open the bleed valve, move the box over the next adjoining area with a minimum 3 inches (75mm) overlap, and repeat the process.

- All areas where soap bubbles appear shall be marked and repaired in accordance with Section III F.

4. Air Pressure Testing

The following procedures are applicable to those processes which produce a double seam with an enclosed space.

a. The equipment shall be comprised of the following:

- An air pump (manual or motor driven) equipped with a pressure gauge capable of generating and sustaining a pressure between 25 and 30 psi (160 and 200 kPa) and mounted on a cushion to protect the geomembrane.

- A rubber hose with fittings and connections.

- A sharp hollow needle, or other approved pressure feed device.

b. The following procedures shall be followed:

- Seal both ends of the seam to be tested.

- Insert needle or other approved pressure feed device into the tunnel created by the fusion weld.



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- Insert a protective cushion between the air pump and the geomembrane.

- Energize the air pump to a pressure between 25 and 30 psi (160 and 200 kPa), close valve, and sustain pressure for approximately 5 minutes.

- At the conclusion of the air test, the opposite end of the seam shall be slit and the subsequent drop in pressure monitored. This will insure that the entire seam was completely tested.

- If loss of pressure exceeds 3 psi or 10mm mercury (15 kPa) or does not stabilize, locate faulty area and repair in accordance with Section III F.

- Remove needle or other approved pressure feed device and seal.

E. Destructive Seam Testing

1. General:

Destructive seam tests shall be performed at selected locations. The purpose of these tests is to evaluate seam strength. Seam strength testing shall be done as the seaming work progresses, not at the completion of all field seaming.

Destructive test samples shall be collected at a minimum average frequency of one test location per 500 feet (150m) of seam length.

Samples, in addition to the minimum average frequency, shall be taken as required by the Geosynthetic Quality Assurance Liner Technician.

Test locations shall be determined during seaming, and may be prompted by suspicion of excess crystallinity, contamination, offset welds, or any other potential cause of imperfect welding. The Geosynthetic Quality Assurance Liner Technician shall be responsible for choosing the locations.

The Geosynthetic Contractor shall not be informed in advance of the locations where the seam samples will be taken.

The Company reserves the right to increase the frequency in accordance with actual performance results of samples taken.

Sample shall be cut by the Geosynthetic Contractor at locations designated by the Geosynthetic Quality Assurance Liner Technician as the seaming progresses in order to obtain laboratory test results before the geomembrane is covered by another material.

Each sample shall be numbered and the sample number and location identified on the panel layout drawing.

All holes in the geomembrane resulting from destructive seam sampling shall be immediately repaired. The continuity of the new seams in the repaired area shall be non-destructively tested.

The samples shall be 12 inches (0.3m) wide by 44 inches (1.1m) long with the seam centered lengthwise. One 1-inch (25mm) wide strip shall be cut from each end of the sample and these shall be tested in the field. The remaining sample shall be cut into three parts and distributed as follows:

- One portion to the Geosynthetic Contractor for laboratory testing, 12 in. by 12 in. (0.3m x 0.3m).

- One portion for Geosynthetic Quality Assurance testing, 12 in. x 18 in. (0.3m x 0.5m).



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- One portion to the Company for archive storage, 12 in. x 12 in. (0.3m x 0.3m).

The Geosynthetic Quality Assurance laboratory testing shall include "Seam Strength" and "Peel Adhesion" (ASTM 4437 with type M-1 specimen 0.5 in. (10mm) wide tested at 2 in. (50mm) per minute). The minimum acceptable values to be obtained in these tests are those indicated in Table 1 of the specifications. At least five specimens shall be tested for each test method. Specimens shall be selected alternately by test from the samples (i.e., peel, shear, peel, shear . . .). To be acceptable, four out of five replicates shall pass seam strength and peel adhesion criteria.

The two 1-inch (25mm) wide strips shall be tested in the field, by hand or tensiometer, for peel and shear, respectively, and shall not fail in the seam. If any field test samples fails to pass, then the procedures outlined below shall be followed.

2. Procedures for a Destructive Test Failure:

The following procedures shall apply whenever a sample fails the destructive test, whether the test is conducted by a Geosynthetic Quality Assurance laboratory, the Geosynthetic Contractor's laboratory, or by field tensiometer. The Geosynthetic Contractor shall have two options:

- The Geosynthetic Contractor can reconstruct the seam between any two passed test locations.

- The Geosynthetic Contractor can trace the welding path to an intermediate location (at 10 feet (3m) minimum from the location of the failed test in each direction) and take a small sample for an additional field test at each location. If these additional samples pass the tests, then full laboratory samples shall be taken. If these laboratory samples pass the tests, then the seam shall be restruc-

tured between these locations. If either sample fails, then the process shall be repeated to establish the zone in which the seam should be reconstructed.

In any case, all acceptable seams must be bounded by two locations from which samples passing laboratory destructive tests have been taken. In cases exceeding 150 feet (50m) of reconstructed seam, a sample taken from within the reconstructed zone must pass destructive testing.

Whenever a sample fails, additional testing may be required for seams that were welded by the same welder and/or welding apparatus or welded during the same time shift.

F. Repairs and Defects

1. All seams and non-seam areas of the geomembrane will be examined for identification of defects, holes, blisters, undispersed raw materials, and any sign of contamination by foreign matter. The surface of the geomembrane shall be clean at the time of examination. The geomembrane surface shall be broomed or washed by the Geosynthetic Contractor if the amount of dust or mud inhibits examination. The Geosynthetic Contractor shall ensure that this examination of the geomembrane precedes any seaming of that section.

2. Each suspect location both in seam and non-seam areas shall be nondestructively tested. Each location which fails the nondestructive testing shall be marked by the Geosynthetic Quality Assurance Liner Technician and repaired by the Geosynthetic Contractor. Work shall not proceed with any materials which will cover locations which have been repaired until laboratory test results with passing values are available.

3. Any portion of the geomembrane exhibiting a flaw, or failing a destructive or nondestructive test, shall be repaired by the Installer. Several



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procedures exist for the repair of these areas. The final decision as to the appropriate repair procedures shall be agreed upon between the Project Manager, and the Geosynthetic Contractor. The procedures available include:

- Patching, used to repair large holes, tears, undispersed raw materials, and contamination by foreign matter.

- Spot welding or seaming, used to repair small tears, pinholes, or other minor, localized flaws.

- Capping, used to repair large lengths of failed seams.

- Removing the bad seam and replacing with a strip of new material welded into place (used with lengths of fusion seams).

4. In addition, the following provisions shall be satisfied:

   - Surfaces of the geomembrane which are to be repaired shall be abraded no more than one hour prior to the repair.

   - All surface must be clean and dry at the time of the repair.

   - All seaming equipment used in repairing procedures must be approved.

   - The repair procedures, materials, and techniques shall be approved in advance of the specific repair by the Project Manager and Geomembrane Contractor.

   - Patches or caps shall extent at least 6 inches (150mm) beyond the edge of the defect, and all corners of patches shall be rounded with a radius of at least 3 inches (75mm).

- The geomembrane below large caps should be appropriately cut to avoid water or gas collection between the two sheets.

5. Each repair shall be numbered and logged. Each repair shall be non-destructively tested. Repairs which pass the nondestructive test shall be taken as an indication of an adequate repair. Large caps may be of sufficient extent to require destructive test sampling, at the discretion of the Geosynthetic Quality Assurance Engineer. Failed tests indicate that the repair shall be redone and retested until a passing test results.

G. Cell Piping

1. The QA Engineer or technician shall observe the construction of the lateral, header, and riser piping.

2. The QA Engineer or technician shall observe that pipe connections are made by:

   - Heat fusion weld

   - material extrusion weld

   - threaded connection, or

   - snap-couplings (mechanical) and/or friction couplings.

   The QA Engineer or technician shall ensure that no solvent welds are used.

3. The QA Engineer or technician shall observe the construction of the gravel envelopes around the cell piping and shall certify construction specifications are met.



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4. The QA Engineer or technician shall observe the construction of the leachate collection standpipe and shall certify the construction specifications are met.

5. The QA Engineer or technician shall observe the construction of the trench and leachate transmission line and shall certify the construction specifications are met or exceeded.

6. The contractor/vendor installing the leachate transmission layer shall hydrotest the line to ensure no leakage occurs. This hydrotest shall be performed under the observation and approval of the QA Engineer. The QA Engineer shall certify the performance and results of the hydrotest.

## IV. TEST EQUIPMENT CALIBRATION

All field test equipment will be kept under control of the QA Engineer. The QA Engineer will be fully trained in the use of equipment, test procedures, and interpretation of results for each piece of test equipment. A copy of the calibration certificate will be kept by the QA Engineer.

## V. NONCONFORMING TEST RESULTS

### A. Synthetic Materials

Synthetic materials which do not meet or exceed construction specifications shall be marked and brought to the attention of the contractor/vendor. Repair or replacement of the synthetic material shall be at the discretion and under the observation of the QA Engineer. A non-conformance report shall be written for synthetic materials which do not meet or exceed construction specifications after repair or replacement.

B. Non-Conformance Reports

All non-conformance reports shall be brought to the attention of the Engineer by the QA Engineer and shall be documented in the QA files.



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## TABLE 1

## REQUIRED PHYSICAL PROPERTIES OF MEMBRANE LINER (HDPE)

| Property | Test Method | Values 60 Mil HDPE |
|---|---|---|
| Thickness | ASTM D-1593 para. 9.1.3 | 60 miles ± 10% with Average ≥ 60 Mils |
| Sheet Density | ASTM D-792 Method A | >0.94 |
| Tensile Properties | ASTM D638 | |
| Strength at Yield | | 126 lb/in (min.) |
| Elongation at Yield | | 12% (min.) |
| Strength at Break | | 228 lb/in. (min.) |
| Elongation at Break | | 560% (min.) |
| Tear Resistance | ASTM D1004, Die C | 39 lb. (min.) |
| Low Temperature Impact | ASTM D-746 | -40°F or Lower |
| Environmental Stress Crack | ASTM D1693 | 1,500 hrs. (min.) |
| Puncture Resistance | FTMS 101C Method 2066 | 72 lbs. (min.) |
| Carbon Black Content | ASTM D1603 | 2.0% - 3.0% |
| Carbon Black Dispersion | ASTM D-3015 | A1, A2, or B1 |
| Dimensional Stability | ASTM D-1204 (212°F, 1 hr.) | ±1.5% Change, max. |



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Attachment 12

FISSURE MONITORING PLAN

Attachment 13

BFI LETTER OF APRIL 4 TO MR. J. C. LERMAN

GEOSERVICES LETTER OF MAY 8 TO MR. J. C. LERMAN

EMCON LETTER OF MAY 9 TO MS. J. HANEY

Waste Systems
BROWNING-FERRIS INDUSTRIES
Phoenix District

HAND DELIVERED

April 4, 1990

Mr. Juan-Carlos Lerman
Water Permits Unit
Office of Water Quality
Arizona Department of Environmental Quality
2005 North Central Avenue
Phoenix, Arizona 85004

RECEIVED
APR 09 1990
EMCON

Re: Cholla Sanitary Landfill

Dear Mr. Lerman:

This letter outlines our understanding of your February 20, 1990 letter requesting additional information to clarify and complement documentation of our application for an Aquifer Protection Permit for the proposed Cholla Sanitary Landfill. This letter also addresses the issues discussed during the meeting held with ADEQ staff in your offices on March 13, 1990.

Your request was divided in two parts: (1) Geologic and Hydrogeologic Hazards and (2) Site-Specific Landfill Design, with specific requests for additional studies, clarification, data, testing, modeling, analyses, redesign, and monitoring plans.

Based on our review of your letter and the subsequent discussions held with you and other ADEQ personnel on March 13, we are planning to provide responses to your requests as follows.

1. Unsaturated Ground-Water Flow Model

   The unsaturated ground-water flow model was questioned as to validity, sensitivity to various input parameters, reasonableness and potential results under long flow periods. These questions will be addressed by the following:

   a. Presenting summaries of the original verification information developed by the model authors and users of the model and, if applicable, by descriptions of case histories.

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b. Conducting a sensitivity analysis of the effect of the model parameters on the results of the model calculations. Laboratory test data provided by Daniel B. Stephens & Associates will replace field-derived and assumed values used in the model. The hydraulic properties of interest are saturated and unsaturated hydraulic conductivity, and moisture retention characteristics.

   The sensitivity analysis will be performed on two representative soil types (layers) on Section 9, which typifies the natural materials extant between the channel of the Agua Fria river and the proposed landfill excavation. Section 9 was chosen because it contains the most unfavorable combination of fine grained and coarse grained soils.

c. Running the model under assumed long flow period conditions to establish the length of time required for the saturation front to impact the liner system. Although sustained flows of more than 8 days of significance are not realistic for flooding episodes in the Agua Fria river, we will assume that after the 99,000 cfs 100-year flood peak has subsided, water will continue flowing in the river with a depth of 1.5 feet (representing a flow of approximately 10,000 cfs) for a total flood event lasting 30 days from beginning to end.

   We want to emphasize that this sustained 30-day flow represents an unrealistic situation for this river, because the peak 100-year flow of about 99,000 cfs under present conditions will be reduced by about two thirds (to 33,000 cfs) when the New Waddel dam is completed in a couple of years, and because the flows used in the analysis are substantially longer in duration than the 100-year storm.

2. Effects of Ground-Water Reaching Liner

   If the unsaturated ground-water flow model run under extended flow conditions shows that the saturation front reaches the liner, the impact of such a condition will be evaluated under the following scenarios: (1) an empty module, (2) a partially filled module, and (3) the completed landfill.

3. Geomembrane Testing

   The validity of the liner testing performed to simulate the effects of an earth fissure occurring under the site was questioned in your letter. From the discussions held on March 13 with the ADEQ staff, it was clear that the staff was concerned about cavities developing under the liner as a result of fissure erosion due to surface water flow.

Instead of conducting additional physical testing of HDPE liner materials under the various conditions you stated in your letter, we propose the following alternative which meets your objectives and provides independent verification of the physical tests already performed.

The ability of a composite soil/geosynthetic liner system to support loads due to the removal of underlying support can be evaluated using recently developed numerical models. The numerical model, SSCOMF, developed at the University of California, Berkeley, uses the finite element technique to simulate the site-specific design features of the landfill. We propose to use this technique to calculate the largest void size that the proposed liner system may bridge, as well as the maximum load the liner system can carry over such a void. This analysis, to be conducted under the direction of Dr. R. Bonaparte of GeoServices, Inc., will supplement and validate the testing already conducted.

Results of the numerical liner modeling will also be used to plan mitigation and remediation measures.

4. Fissure and Vadose Zone Monitoring

We will evaluate the following geophysical methods for applicability to fissure monitoring:

- microgravity
- seismic tomography
- seismic refraction, reflection and shear-wave
- time domain reflectometry (TDR)
- fiber optics

Alternative methods available to monitor for fissures at the site will be evaluated and the most appropriate proposed. Similarly, a vadose zone monitoring plan will be proposed.

5. Subsidence Monitoring

As discussed during the March 13 meeting, the analytical subsidence models used in the mining industry were developed and are used in hard rock mines, not in alluvial environments such as at the proposed Cholla landfill. Furthermore, those models are based on collapse over voids due to withdrawal of ore (coal, salt, metals). No such collapse is possible at the site since no mining is occurring or is known to be planned in the



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future under the site. As described in the submitted application, even if the salt caverns situated three fourths of a mile south of the site collapsed, no effects would be experienced at the site since it would be outside the zone of collapse influence.

Based on our discussions, the requested study to evaluate the potential for subsidence and formation of fissure zones capable of causing damage to engineering structures is not warranted and it was agreed that this analysis is not necessary.

6. BADCT Demonstration

It was stated that the application did not demonstrate BADCT because the design was not site specific. We will provide additional design justification that details why the proposed design of the Cholla Sanitary Landfill has been dictated by site-specific conditions fulfilling BADCT requirements.

7. Other Requests

Other data and/or clarifications will be provided as requested. A discussion of their relevance or applicability to the project will be presented where appropriate.

We feel that the review process could be enhanced if our design consultants had the opportunity to discuss informally the issues raised by ADEQ reviewers. This would provide for clarification of written correspondence, allow explanations based on evidence, and avoid miscommunication. We hope that ADEQ reviewers are available to meet with our consultants and address remaining technical issues to expedite the review of our application.

It is our understanding, based on the March 13 meeting, that ADEQ personnel will review the additional analyses proposed in this letter and advise us whether you agree or disagree with the proposed additional work. We believe it is important to reach a mutual understanding at this point with respect to the additional work to be performed. Accordingly, we await your reply.

EMCON
ASSOCIATES
Consultants in Wastes
Management and
Environmental Control

May 9, 1990
Project 372-15.01

Ms. Jeanmarie Haney
State Programs Hydrology Unit
Arizona Department of Environmental Quality
2005 North Central Avenue
Phoenix, Arizona 85004

Re: Cholla Landfill

Dear Ms. Haney:

During the April 27, 1990 meeting with you, Juan-Carlos Lerman, Barry Abbott, Drs. Tom Maddock and Mike Sully of the University of Arizona, and Mike Green, we discussed the sensitivity analysis of unsaturated ground-water flow parameters on the model results. The sensitivity analysis was proposed in Mr. Brett Frazier's letter of April 4, 1990 to Mr. Lerman.

As agreed at the meeting, Drs. Tom Maddock and Mike Sully will run the UNSAT2 model for Section 9 with the following changes in the hydraulic parameters and the initial condition:

Run #1: Values of the saturated hydraulic conductivity used in the earlier simulations will be used for the values of the saturated hydraulic conductivity for the vertical direction. These values will be increased by a factor of 2 and used for the values of the saturated hydraulic conductivity for the horizontal direction (as presented on Table 1). 14 days will be simulated and further days will be added if the anisotropy produces significantly different results.

Run #2: The initial water content in the layers in Section 9 will be increased by decreasing the background suction from 14 feet to 2 feet. The values of saturated hydraulic conductivity used will be the same as for Run #1 with the ratio of horizontal to vertical saturated hydraulic conduc-

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tivity being 2/1. 14 days will be simulated with further days being added if the increase in initial water content produces significantly different results.

Also as agreed at the meeting, even if the additional runs do not show that the saturation front reaches the liner, we will provide an evaluation of what impacts might be expected on the liner from a saturation front reaching the liner. While we believe this evaluation to be unnecessary, it will be provided to answer questions that may arise on this subject.

If you feel that the understandings set forth in this letter are not correct, please contact us immediately. In the absence of objections, we will proceed as described in this letter.

Very truly yours,

EMCON Associates

Leonardo Alvarez
Vice President

LA:ljt

cc: Skip Hellerud, ADEQ
Roger Kenett, ADEQ
Juan-Carlos Lerman, ADEQ
Brett Frazier, BFI
David Peters, BFI
Lee Schoon, BFI
Doug Jorden, SLF
Mike Green, EMCON

Table 1

HYDRAULIC PROPERTIES
SECTION 9

| Material | USCS[1] | $K_v$[2] (ft/day) | $K_h$[3] (ft/day) |
|---|---|---|---|
| 1 | SP | 2.84 | 5.68 |
| 2 | GP | 28.3 | 56.6 |
| 3 | GW-GM | 23.8 | 47.6 |
| 4 | SP-SW | 14.2 | 28.4 |
| 5 | CL | 0.493 | 0.986 |

1. USCS (Unified Soil Classification System): from boring logs and laboratory test results.
2. Vertical saturated hydraulic conductivity.
3. Horizontal saturated hydraulic conductivity.



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**GEOSERVICES INC.** CONSULTING ENGINEERS

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8 May 1990

Mr. Juan-Carlos Lerman
Water Permits Unit
Office of Water Quality
Arizona Department of Environmental Quality
2005 North Central Avenue
Phoenix, Arizona 85004

Subject: BFI Cholla Sanitary Landfill
Evaluation of the Effects of an Earth Fissure
on the Landfill Liner System
GeoServices Project Number: P1419-02
GeoServices Document Number: N900318

Dear Mr. Lerman,

It was a pleasure meeting with you at our 27 April 1990 meeting in Phoenix, Arizona. As discussed at our meeting, we are proceeding with the analyses of the proposed Cholla Sanitary Landfill liner system as described in Browning-Ferris Industries' 4 April 1990 letter. We are using the soil arching-tensioned membrane model to conduct these analyses. This model is being substituted in the analyses for the SSCOMP finite element model, which we originally intended to use, due to numerical problems that developed in SSCOMP when we attempted to model an earth fissure. The use of the soil arching-tensioned membrane model is conservative in comparison to the use of the SSCOMP model because the soil arching-tensioned membrane model cannot account for soil dilatancy, which tends to decrease the soil pressure exerted on the geomembrane, nor the elongation of the geomembrane in the anchorage zone, which for a given soil pressure, tends to decrease the tension in the geomembrane. As we discussed, we will continue to look for a suitable alternative finite element model to complement the soil arching-tensioned membrane model and to verify the conservatism of our approach.

The purpose of the soil arching-tensioned membrane analyses is to evaluate the influence of a hypothetical earth fissure immediately beneath

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the landfill on the integrity of the landfill's liner system. The analyses will be performed for the specific Cholla Sanitary Landfill liner system, which is depicted in Figure 1. Fissures that are initially 1-in. (25 mm) wide will be modeled; wider fissures that might result from the hypothetical case of fissure erosion due to water flow under the landfill will also be modeled. Eroded fissure widths of up to 3.3 ft (1 m) will be analyzed. The results of the analyses will include the vertical deflection of the liner system, the tensile strain in the geomembrane component of the liner system, and a determination of whether the deflections and strains are within acceptable limits. The tensile strain limit will be selected based on a consideration of the potential for geomembrane rupture and stress cracking at geomembrane seams.

For the analyses, the strength properties of the soils and geosynthetics to be used in the liner system will be documented thoroughly. Where possible, actual test data will be used as input for the analyses. For the geomembranes, stress-strain test data on products from all potential suppliers will be solicited. This test data will be used in the analyses and included in our final report. The anisotropic strength properties of the geomembrane liner will be accounted for in the model when computing deformations.

To determine the sensitivity of the limit equilibrium model to the properties of the soil, waste, and geomembrane component of the liner system, a parametric analysis will be conducted. The critical parameters will be varied over a range of values; the range of values for the soil and waste will be consistent with those in the Engineering Master Plan prepared by Emcon Associates. A range for the geomembrane will be consistent with the range of values for commercially available products.

The results of our analyses will be presented in a report containing a description of the method of analysis, design assumptions, and conclusions. Appendices to the reports will include the detailed calculations, applicable test results and cited references.

We are currently proceeding with the analyses described in this letter. Please contact us immediately if you have any comments or require additional information on the analyses we propose.

Sincerely,

Rudy Bonaparte, Ph.D., P.E.
Principal

Copy to: Mr. Brett Frazier, BFI
Mr. David Peters, BFI
Mr. Gary Johnson, BFI
Mr. Doug Jorden, SL&F
Mr. Leo Alvarez, Emcon
Mr. Roger Kennett, ADEQ
Mr. Skip Hellerud, ADEQ

N900318

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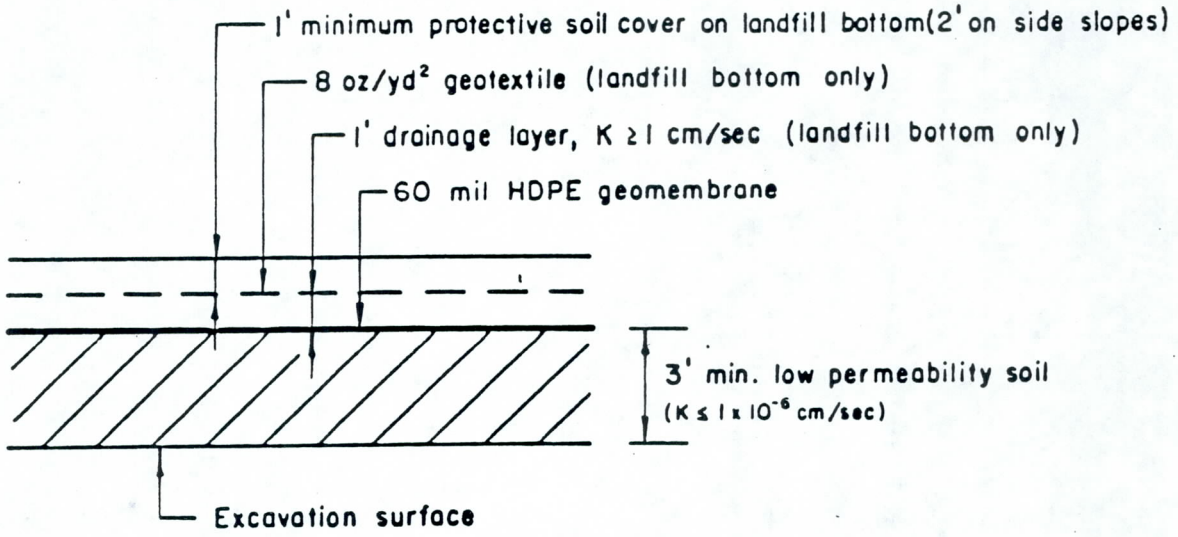


# LINER SYSTEM

SOURCE: EMCON ASSOCIATES ENGINEERING
MASTER PLAN, SEPTEMBER 1989

Attachment 14

# ANALYSIS OF THE EFFECT OF AN EARTH FISSURE ON LINING SYSTEM INTEGRITY, CHOLLA SANITARY LANDFILL

GeoServices Inc.
90.06.06/N900314

# ANALYSIS OF THE EFFECT OF AN EARTH FISSURE ON LINING SYSTEM INTEGRITY

# CHOLLA SANITARY LANDFILL EL MIRAGE, ARIZONA

Prepared for

Browning-Ferris Industries
1580 East Elwood
Phoenix, Arizona 85036

Prepared by

GeoServices Inc. Consulting Engineers
5950 Live Oak Parkway, Suite 330
Norcross, Georgia 30093

GeoServices Project Number: P1419-02
June 1990

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# 1. INTRODUCTION

## 1.1 Purpose

GeoServices Inc. Consulting Engineers (GeoServices) has conducted an analysis of the lining system for the proposed Cholla Sanitary landfill (Cholla landfill) in El Mirage, Arizona. The purpose of the analysis was to assess the integrity of the lining system in the event that an earth fissure developed in the foundation soils beneath the proposed landfill. This evaluation was carried out at the request of Mr. David Peters, P.E., and Mr. Brett Frazier, both of Browning-Ferris Industries (BFI). The report was prepared by Ms. Heidi Rubin, E.I. and Dr. Rudy Bonaparte, and it was reviewed by Mr. Allen E. Blodgett, P.E., all of GeoServices.

This report summarizes the results of the analyses and presents our conclusions as to the ability of the lining system to maintain its integrity in the event of earth fissure development. The report presents the results of analyses for two hypothetical cases:

- a 0.5-in. (13-mm) wide earth fissure develops immediately beneath the landfill lining system; and
- an erosion-widened earth fissure of some maximum acceptable width develops immediately beneath the landfill lining system.

## 1.2 Scope and Organization

The scope of this report is consistent with the scope described in the 8 May 1990 letter from GeoServices to Mr. Juan-Carlos Lerman of the Arizona Department of Environmental Quality (ADEQ).

The organization of the report is as follows:

- background information is presented in Section 2;



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- the material properties used in the analyses are presented in Section 3;

- the method of analysis and results are presented in Section 4; and

- conclusions are presented in Section 5.

Appendix A to this report presents the results of laboratory tension tests on specimens of high density polyethylene (HDPE) geomembrane liner. These test results are used in the report to define a tension-strain constitutive relationship for the geomembrane. Appendix B presents a copy of the paper by Giroud et al. [1990] that forms the basis for the analyses presented herein.

# 2. BACKGROUND INFORMATION

## 2.1 Site Conditions

An Engineering Master Plan (dated September 1989) has been prepared for the proposed Cholla landfill by EMCON Associates, San Jose, California (EMCON [1989]). Based on information in the Engineering Master Plan, the Cholla landfill site is underlain by a variable sequence of alluvial deposits consisting of clay, silt, sand, gravel, and cobbles. The upper 8 to 12 ft (2.4 to 3.6 m) of the alluvial deposits generally consist of silty sands which overlie sandy to clayey gravels interbedded with gravelly and clayey sands to a depth of 60 ft (18 m). From a depth of 60 ft (18 m) to about 250 ft (76 m), the alluvial deposits consist of a heterogeneous mixture of lenticular layers of clay, silt, sand, gravel, and cobbles.

## 2.2 Earth Fissure Development

ADEQ has expressed concern over the potential for the development of subsidence-induced earth fissures in the alluvial deposits beneath the Cholla landfill. ADEQ has hypothesized a scenario wherein: (i) a fissure, initially 0.25 to 0.5 in. (6 to 13 mm) in width and very long in length, develops beneath the landfill; (ii) during a storm event, erosion of the fissure increases its width; and (iii) the erosion-widened fissure undermines support (i.e., undermines the low-permeability soil layer) for the landfill lining system.

## 2.3 Landfill Geometry

The Engineering Master Plan for the Cholla landfill includes a final grading plan, excavation and preparation plan, and details of the lining system. Based on these drawings, the landfill is somewhat rectangular in shape, being larger in the north-south direction compared to the east-west direction. The lined landfill area will eventually encompass 170 acres (70 hectares). The landfill will have slopes of 2 percent on the bottom of the landfill and 3H:1V (horizontal:vertical) on the sides. The slope of the landfill cover system will



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vary from a minimum of 5 percent to a maximum of 4H:1V. The maximum thickness of waste in the landfill after filling will be approximately 150 ft (46 m). Excavation depths at the site will be up to 110 ft (33 m).

## 2.4 Lining System

A cross-section of the lining system for the proposed Cholla landfill, taken from EMCON [1989], is shown in Figure 1. The components of the lining system from top to bottom are as follows:

- 1-ft (0.3-m) thick protective soil cover (2-ft (0.6-m) thick on side slopes);
- geotextile separator;
- 1-ft (0.3-m) thick granular drainage layer (absent on side slopes);
- 60-mil (1.5-mm) thick HDPE geomembrane liner; and
- 3-ft (1-m) thick layer of low-permeability bentonite-treated soil having a hydraulic conductivity of less than $1 \times 10^{-6}$ cm/s.

# 3. MATERIAL PROPERTIES

## 3.1 Introduction

This section presents the properties of the waste, lining system soils, and lining system geomembrane required as input parameters for the analyses in this report. The shear strengths of the waste and soils are reported in terms of total stresses, which is appropriate for the analyses presented herein.

## 3.2 Waste

In Section 4 of this report, a soil arching-tensioned membrane analytical model will be used to evaluate the response of the Cholla landfill lining system to the development of an earth fissure beneath the lining system. For arching to be effective, the soil and waste above the lining system must have an angle of internal friction of at least 20°. As described below, the information available from the technical literature indicates that municipal solid waste has a friction angle in excess of 20°.

The shear strength of municipal solid waste from a landfill in Italy has been estimated by Pagotto and Rimoldi [1987] based on the results of plate load tests. From their results, they estimated a total-stress friction angle for the waste of 22° and a total-stress cohesion of 630 psf (30 kPa).

EMCON [1989] reported the results of a full-scale load test on a typical "refuse" landfill. The load test was conducted by Converse Davis Dixon Associates. A back-analysis of the strength of the refuse at failure resulted in a calculated total-stress friction angle of 27°.

A correlation between standard penetration test (SPT) results and friction angle has been established for granular soils [Peck et al., 1974]. Using this correlation and limited data from SPT tests on waste by Sowers [1968], Frydman and Baker [1986], and GeoServices' project files, values of friction angle for waste ranging from 25° to 30° are obtained. These values are consistent with



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stable slopes that have been observed at existing landfills (i.e., waste slopes at 2H:1V and steeper).

Based on the above discussion, it is concluded that the total stress friction angle of municipal solid waste exceeds 20°.

For the analyses in this report, the average unit weight of waste and daily cover is assumed to be 60 lb/ft$^3$ (9.4 kN/m$^3$) which is the same as that assumed by EMCON [1989].

## 3.3 Lining System Soils

The lining system soils include the compacted low-permeability soil layer beneath the HDPE geomembrane liner and the protective soil cover and granular drainage layer above the HDPE geomembrane liner.

For the soil arching-tensioned membrane analysis, it is assumed that the earth fissure propagates vertically through the low-permeability soil layer to the geomembrane liner, as illustrated in Figure 2. As a consequence, the properties of this layer do not affect the analysis results.

The angles of internal friction of the protective soil cover and granular drainage layer must exceed 20° for the results of the soil arching-tensioned membrane analysis to be valid. According to EMCON [1989], the granular drainage material will consist of "clean rounded to subrounded gravel" while the protective soil cover will consist of "mixtures of gravels, sands, or finer-grained materials". Friction angles for granular soils such as these almost always exceed 30° [e.g., Holtz and Kovacs, 1981]. Therefore, the strengths of these materials are more than adequate to allow them to be modeled using the soil arching-tensioned membrane theory.

The average unit weight of the protective soil cover and granular drainage layer is assumed to be 125 lb/ft$^3$ (19.6 kN/m$^3$).

## 3.4 HDPE Geomembrane Liner

The properties of the HDPE geomembrane that are important for the soil arching-tensioned membrane analysis are the tensile yield stress and strain of the geomembrane and its tensile modulus. The tensile properties of the geomembrane seams should also be considered, since if an earth fissure were to develop, it could do so beneath a seam.

For this project, ten tension tests were performed on specimens of four different 60-mil (1.5-mm) thick HDPE geomembranes. The four different geomembranes are from the four manufacturers whose materials may be considered for use on the Cholla landfill project. The geomembrane materials that were tested are:

- Gundline HD from Gundle Lining Systems, Inc.;
- Enviroseal 60 from National Seal Company;
- Hyperflex from SLT Environmental Inc.; and
- Poly-Flex from Poly-Flex Inc.

For each geomembrane material, five of the ten tests were carried out applying the tension in the specimen machine direction. The other five tests were conducted by applying the tension in the specimen cross-machine direction. All tests were performed in accordance with American Society for Testing and Materials (ASTM) Standard D 882-83 "Test Methods for Tensile Properties of Thin Plastic Sheeting." The tests were conducted on 1-in. (25-mm) wide specimens at a strain rate of 2 in./min (0.83 mm/s).

The results of the laboratory tension tests are summarized in Table 1 and the stress-strain curves from each of the tests is given in Appendix A.



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From Table 1, it can be seen that the yield tension, $f_y$, of the tested geomembranes varies from 1,670 to 2,189 lb/ft (24 to 32 kN/m) in the machine direction and 1,793 to 2,064 lb/ft (26 to 30 kN/m) in the cross-machine direction. The tensile strain at yield varied from 16.2 to 18.3 percent in the machine direction and from 15.0 to 16.3 percent in the cross-machine direction. From the test results given above, it appears that the properties of the tested geomembranes are similar to each other. It also appears that the geomembranes exhibit only a small amount of anisotropy with respect to their tension-strain behavior. Based on these results, it is concluded that a single set of tensile properties can be used to represent all of the tested geomembranes in the soil arching-tensioned membrane analysis presented in the next section of this report. It is also concluded that it is not necessary to model geomembrane anisotropy in the analysis; the geomembrane can be assumed to be isotropic.

A review of the data in Table 1 reveals that a yield tension of 1,670 lb/ft (24 kN/m) is conservative for a 60-mil (1.5-mm) thick HDPE geomembrane tested in accordance with ASTM D 882-83. This value of tension should not be used in design, however, since ASTM D 882-23 is a short-term tension test that does not account for the long-term creep of polyethylene. To mitigate the potential for long-term creep, the tension in the geomembrane should be maintained at a value well below the yield tension obtained from a short-term test. For the analyses in this report, an allowable tension, $f_a$, equal to 30 percent of the yield tension will be used. Thus, $f_a = 0.3f_y = 500$ lb/ft (7.3 kN/m).

The allowable tension given above is suitable for properly fabricated HDPE seams. Construction quality control requirements for HDPE seams usually call for a seam yield strength in excess of 95 percent of the specified minimum geomembrane tensile strength, a film tearing bond, and a minimum yield strain of 10 percent. The allowable tension given above will also minimize the potential for stress cracking of HDPE geomembrane seams. Based on data from Halse et al. [1989], the potential for stress cracking is low in properly fabricated HDPE geomembrane seams if they are subjected to sustained stresses of 30 percent or less of the yield tension of the geomembrane itself. An allowable geomembrane

tension, $f_a$, of 500 lb/ft (7.3 kN/m) is consistent with the test results from Halse et al.

The value of $f_a$ given above was derived from the results of tests conducted at a temperature of 70°F (21°C). This temperature should be in the same range as the in-service temperature of the geomembrane. Therefore, the use of a value of $f_a$ from tests at 70°F (21°C) is appropriate. During construction, the temperature of the geomembrane will reach levels well above 70°F (21°C) due to direct exposure to sunlight. Schlegel [1986] has reported the results of tests on their HDPE geomembrane material showing that 2,000 hours exposure to a temperature of 195°F (90°C) had no effect on the tensile yield strength of the geomembrane and only a slight effect on the tensile yield strain. In addition, any geomembrane used on the Cholla landfill project will contain at least 2 percent by weight carbon black and an anti-oxidant package. These additives will prevent exposure-induced ultra-violet light or thermal oxidation damage to the geomembrane. In conclusion, exposure to sunlight and elevated temperatures during installation should not adversely affect the in-service performance of the HDPE geomembrane used on the Cholla landfill project.

As evidenced by the test results reported in Table 1, the tensile stiffnesses of HDPE geomembranes decrease with increasing tensile strain. Due to the viscoelastic properties of HDPE, the tensile stiffnesses of HDPE geomembranes also decrease with increasing duration of load. The values of tensile stiffness given in Table 1 are based on short-term tests. These values should be decreased to account for the effect of long-term creep. Limited available data from Schlegel [1986] suggests that the tensile stiffness should be reduced by a factor of 2 to 3 to account for long-term creep. For this reason, a secant tensile stiffness of 10,000 lb/ft (146 kN/m) will be used in the soil arching-tensioned membrane analysis presented in the next section. This value of tensile stiffness is consistent with the data in Table 1 and results in an allowable geomembrane strain, $\epsilon a$, of 5 percent at the allowable geomembrane stress, $f_a$, of 500 lb/ft (7.3 kN/m).



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# 4. METHOD OF ANALYSIS

## 4.1 Soil Arching-Tensioned Membrane Model

The analytical model used in this report to evaluate the effect of an earth fissure on the Cholla landfill lining system is the soil arching-tensioned membrane model developed by Bonaparte and Berg [1987] and Giroud et al. [1988, 1990]. In the model, the soil layers and geosynthetics are initially resting on a firm foundation (Figure 2a). At some point in time, an earth fissure is assumed to open in the foundation soil beneath the landfill and then to propagate vertically upward to the level of the geomembrane liner (Figure 2b). Under the weight of the overlying soil and waste, the geomembrane deflects into the fissure. The deflection has two effects (Figure 3): bending of the soil and waste (which transfers a portion of the load due to the weight of these materials away from the fissure to the firm foundation soil on either side of the fissure); and stretching of the geomembrane (which causes this material to act as a tensioned membrane that transfers the remaining portion of the weight to the firm foundation soil adjacent to the fissure).

## 4.2 Soil Arching Model

Using the soil arching theory, Giroud et al. [1990] established the following equation relating the pressure acting on the geomembrane liner, the width of the earth fissure, the height of soil and waste above the fissure, the average unit weight of the soil and waste overlying the fissure, and any normal surcharge loads applied on top of the waste:

$$p = 2\gamma b(1 - e^{-0.5H/b}) + qe^{-0.5H/b} \qquad \text{(Equation 1)}$$



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where: $\gamma$ = average unit weight of the material (i.e., soil and waste)within a height of 2b above the geomembrane ($lb/ft^3$ or $kN/m^3$);
b = width of earth fissure (ft or m);
H = height of soil and waste above the geomembrane (ft or m);
q = uniform normal surcharge stress ($lb/ft^2$ or kPa); and
p = pressure acting on the geomembrane, as shown in Figure 3 ($lb/ft^2$ or kPa).

A study of Equation 1 shows that the term $e^{-0.5H/b}$ tends towards zero as H/b becomes large. At the Cholla landfill, the term H/b will be large since the height of the landfill far exceeds the possible width of an earth fissure. Therefore, as a very good approximation:

$$p = 2\gamma b \qquad \text{(Equation 2)}$$

Table 2 presents values of p for a range of values of fissure width, b. It can be seen that as the width of an earth fissure increases, the pressure acting on the overlying geomembrane increases proportionally.

## 4.3 Tensioned Membrane Model

When a geomembrane overlying a fissure is subjected to a normal stress, it deforms into a curved shape and tensile stresses are generated. This is known as the "tensioned membrane effect" (Figure 3). If an earth fissure were to develop beneath the Cholla landfill, it would be an approximately linear feature. As a consequence, the geomembrane deflection into the fissure would occur under plane-deformation conditions. Under these conditions, the deformed shape of a geomembrane when subjected to a uniform normal stress will be a circle. The strain in the geomembrane for this set of conditions is given by:

$$\epsilon = 2\Omega \sin^{-1}[1/(2\Omega)]-1 \qquad \text{(Equation 3)}$$

where:

$$\Omega = 1/4[(2y/b) + (b/2y)] \qquad \text{(Equation 4)}$$

and:

$\epsilon$ = tensile strain in the geomembrane (dimensionless);
$\Omega$ = deflection factor (dimensionless); and
y = maximum geomembrane deflection (ft or m).

The reference by Giroud et al. [1990] in Appendix B contains a tabulated set of values for the interdependent parameters $\epsilon$, $\Omega$, and y/b.

## 4.4 Combined Model

The soil arching and tensioned membrane models can be combined by considering static force equilibrium and the tension-strain behavior of the geomembrane. From static equilibrium:

$$\alpha = pb\ \Omega \qquad \textit{(Equation 5)}$$

where: $\alpha$ = tension in the geomembrane (lb/ft or kN/m).

From the tension-strain behavior of the geomembrane:

$$\alpha = K\epsilon$$

where: K = tensile stiffness of the geomembrane (lb/ft or kN/m).

Combining equations 2, 5, and 6, and setting the strain in the geomembrane equal to the maximum allowable value results in the following:

$$b_{max} = (k\epsilon_{\alpha}/2\gamma\Omega_{\alpha})^{0.5} \qquad \text{(Equation 7)}$$



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where: $b_{max}$ = maximum acceptable fissure width (ft or m);
$\epsilon_\alpha$ = allowable geomembrane strain (dimensionless); and
$\Omega_\alpha$ = value of $\Omega$ at $\epsilon_\alpha$ (dimensionless).

The value of $b_{max}$ given by Equation 7 is the maximum fissure width that the HDPE geomembrane can span without developing a tension greater than $f_a$.

## 4.5 Analysis of Initial Earth Fissure

It is hypothesized that if an earth fissure were to develop beneath the Cholla landfill, it would initially have a width of 0.5 in. (13 mm). For this case, b = 0.5 in. (13 mm), p = 10.4 lb/ft$^2$ (0.5 kPa), and K = 10,000 lb/ft (146 kN/m). Combining Equations 3, 5, and 6, results in:

$$2\sin^{-1}[1/(2\Omega)] - (1/\Omega) = pb/K \qquad \text{(Equation 8)}$$

Solving Equation 3 results in pb/K = 4 x $10^{-5}$ and $\Omega$ = 10. Then, from Equation 3, $\epsilon$ = 0.04 percent. Finally, from Equation 5, $\alpha$ = 4 lb/ft (0.06 kN/m).

From the preceding calculations, it can be seen that the geomembrane tension calculated using the soil arching-tensioned membrane analysis is negligibly small for an earth fissure width of 0.5 in. (12 mm). On this basis, it is concluded that if an earth fissure were to develop beneath the Cholla landfill, and it were to maintain its initial width and not undergo erosion-induced widening, the presence of the fissure would not adversely affect the performance of the landfill lining system.

## 4.6 Maximum Acceptable Earth Fissure Width

From Section 3.4 of this report, $\epsilon_\alpha$ = 5 percent. From Table 2 of the paper by Giroud et al. [1990] in Appendix B, $\Omega_\alpha$ = 0.97 for $\epsilon_\alpha$ = 5 percent. For K = 10,000 lb/ft (146 kN/m) and $\gamma$ = 125 lb/ft$^3$ (19.6 kN/m$^3$), $b_{max}$ = 1.44 ft (0.44 m). Thus, based on the soil arching-tensioned membrane analysis and the material

properties assumed in this report, the maximum acceptable erosion-widened earth fissure width for the Cholla landfill (as presently designed) is 1.44 ft (0.44 m).

The value of $b_{max}$ given above represents the maximum fissure width that the landfill lining system can span without the geomembrane tension exceeding $f_a$. In reality, the lining system could probably span slightly larger fissures since the soil arching-tensioned membrane model and the values of the input parameters used with the model are conservative and the load-carrying capacity of the geotextile filter layer has been neglected.

## 4.7 Limited Parametric Analysis

The three main variables affecting the results of the soil arching-tensioned membrane analysis are the width of the earth fissure, b, the allowable geomembrane tension, $f_a$, and the geomembrane tensile stiffness, K. The affect of each of these variables is described below.

*Width of Earth Fissure:* The influence of earth fissure width on geomembrane tensile strain is shown in Figure 4. It can be seen that the geomembrane tensile strain increases rapidly with increasing width of the earth fissure.

*Allowable Geomembrane Tension:* If the allowable geomembrane tension were changed, the allowable geomembrane strain would change proportionally (based on Equation 6). Figure 4 can be used to evaluate the maximum acceptable earth fissure width, $b_{max}$, in the event that a smaller or larger allowable geomembrane strain is used. As previously noted, the value of allowable geomembrane strain (and tension) used in this report is believed to be slightly conservative.

*Geomembrane Tensile Stiffness:* From Equation 7, the affect of geomembrane tensile stiffness on the maximum acceptable fissure width, $b_{max}$, can be deduced. For a given value of allowable geomembrane strain, the maximum acceptable fissure width increases in proportion to the square root of the increase in geomembrane



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tensile stiffness. As a result, if the tensile stiffness of the geomembrane was increased from the value assumed in this report (by using a thicker material for example), the maximum acceptable fissure width would increase by the square root of the ratio of the tensile stiffnesses. Since geomembrane tensile stiffness is directly proportional to geomembrane thickness, the use of a 100-mil (2.5-mm) thick geomembrane rather than a 60-mil (1.5-mm) thick material would increase $b_{max}$ by 30 percent (i.e., the lining system could span a 1.86 ft (0.57 mm) fissure rather than 1.44 ft (0.44 m)).

## 5. CONCLUSIONS

An analysis has been conducted on the affect of earth fissures on the Cholla landfill lining system. The analysis was carried out using a soil arching-tensioned membrane model. Two hypothetical cases were analyzed. In the first case, it was assumed that a 0.5-in. (12-mm) wide earth fissure developed immediately beneath the landfill lining system. For this case, a geomembrane tension of 4 lb/ft (0.06 kN/m) was calculated. This tension is well below the allowable geomembrane tension of 500 lb/ft (7.3 kN/m) applicable to the 60-mil (1.5-mm) thick HDPE geomembrane that will be used on the Cholla landfill project. Based on this result, if an earth fissure were to develop beneath the Cholla landfill, and if it were to maintain its initial width and not undergo erosion-induced widening, the presence of the fissure would not adversely affect the performance of the landfill lining system.

In the second case analyzed in the report, it was assumed that the earth fissure immediately beneath the lining system was eroded, increasing its width to some greater value. For this case, an analysis was conducted to determine the maximum fissure width the lining system could span without exceeding the allowable geomembrane tension of 500 lb/ft (7.3 kN/m). The analysis in this report resulted in a maximum acceptable fissure width of 1.44 ft (0.44 m). As pointed out in the report, the Cholla landfill lining system can probably span a slightly larger fissure than that reported above since the soil arching-tensioned membrane model used in this report and the input parameters used with the model are conservative and the load carrying capacity of the geotextile filter layer has been neglected.



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## REFERENCES

Bonaparte, R. and Berg, R.R., The Use of Geosynthetics to Support Roadways Over Sinkhole Prone Areas. In *Proceedings of the Second Multidisciplinary Conference on Sinkholes and the Environmental Impacts of Karst*, Orlando, FL, Feb 1987, Rotterdam, pp. 437-45.

EMCON Associates, "Application for Aquifer Protection Permit and Approval of Solid Waste Disposal Operations Plans, Cholla Sanitary Landfill, El Mirage, Arizona, Part I: Engineering Master Plan", Prepared for Browning-Ferris Industries of Arizona, Sep 1989.

Frydman, S. and Baker, R., "Construction of a Bus Parking Station on a Waste Deposit Site", *Proceedings of a Conference on Building on Marginal and Derelict Lands*, Institution of Civil Engineers, Glasgow, England, May 1986, pp. 255-266.

Giroud, J.P., Bonaparte, R., Beech, J.F. and Gross, "B.A., Load Carrying Capacity of a Soil Layer Supported by a Geosynthetic Located on a Void", *Proceedings of the International Geotechnical Symposium on Theory and Practice of Earth Reinforcement*, Fukuoka/Kyushu, JAPAN, Oct 1988, Balkema, Rotterdam, pp. 185-90.

Giroud, J.P., Bonaparte, R., Beech, J.F., and Gross, B.A., "Design of Soils Layer-Geosynthetic Systems Overlying Voids", *Geotextiles and Geomembranes*, Vol. 9, No. 1, 1990, pp. 11-50.

Holtz, R.D. and Kovacs, W.D., "An Introduction to Geotechnical Engineering", Prentice-Hall, Inc., Englewood, NJ, 1981, 733 p.

Halse, Y.H., Koerner, R.M., and Lord, A.E., "Laboratory Evalaution of Stress Cracking in HDPE Geomembrane Seams", *Proceedings of a Seminar on Durability and Aging of Geosynthetics*, Geosynthetic Research Institute, Philadelphia, PA, Dec 1988.



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Pagotto, A. and Rimoldi, P., "Design and Construction of a Geogrid Reinforced Embankment Over Waste Material", *Proceedings, Geosynthetics '87*, Vol. 1, New Orleans, LA, Feb 1987, pp. 320-331.

Peck, R.B., Hanson, W.E., and Thornburn, T.H., *"Foundation Engineering"*, John Wiley and Sons, Inc., New York, 1974, p. 310.

Schlegel Lining Technology, Inc., Technical Data Sheets, Houston, TX, 1986, p. 4.2.5.

Sowers, G.F., "Foundation Problems in Sanitary Landfills", *Journal of the Sanitary Engineering Division*, ASCE, 94 SA 1, Feb 1968, pp. 103-116.

Table 1

Tensile Properties of HDPE Geomembrane

| | Yield Tension (lb/ft) | Yield Strain (lb/ft) | Initial Stiffness (lb/ft) | 2% Stiffness (lb/ft) | 5% Stiffness (lb/ft) |
|---|---|---|---|---|---|
| Gundline HD (MD) | 1,777 | 16.2 | 60,800 | 27,600 | 24,000 |
| Gundline HD (XD) | 1,895 | 15.0 | 65,900 | 29,000 | 26,000 |
| Enviroseal 60 (MD) | 1,670 | 18.3 | 56,100 | 21,500 | 20,800 |
| Enviroseal 60 (XD) | 1,793 | 15.0 | 56,700 | 26,700 | 24,600 |
| Hypreflex (MD) | 2,189 | 16.4 | 64,900 | 30,500 | 29,100 |
| Hyperflex (XD) | 2,064 | 16.3 | 63,700 | 32,000 | 28,200 |
| Poly-Flex (MD) | 1,811 | 16.7 | 64,640 | 27,800 | 24,200 |
| Poly-Flex (XD) | 1,944 | 15.3 | 72,100 | 30,100 | 26,500 |

Notes: (1) All geomembranes were 60-mil (1.5-mm) thick. All testing was performed in accordance with ASTM Standard D 882-83 "Test Methods for Tensile Properties of Thin Plastic Sheeting." The tests were conducted on 1-in. (25-mm) wide specimens at a strain rate of 2 in./min (0.83 mm/s). Each reported result is the mean value of five individual specimen results.

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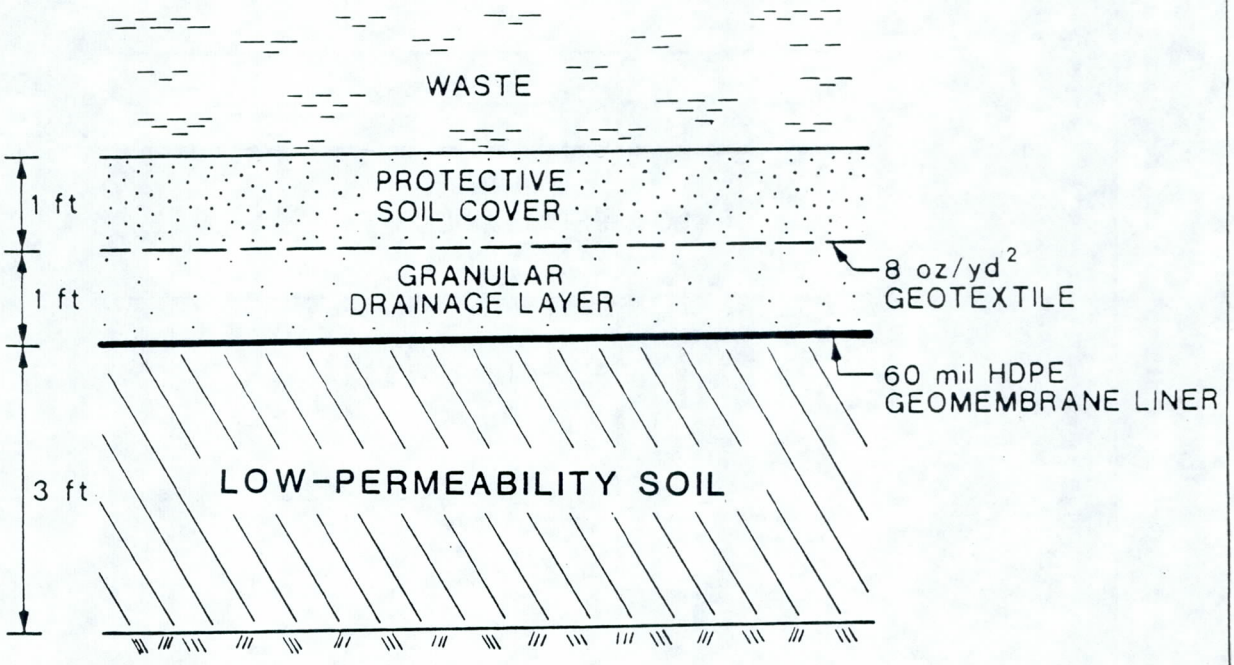
Table 2

Pressure Acting on the Geomembrane
Versus the Width of the Earth Fissure

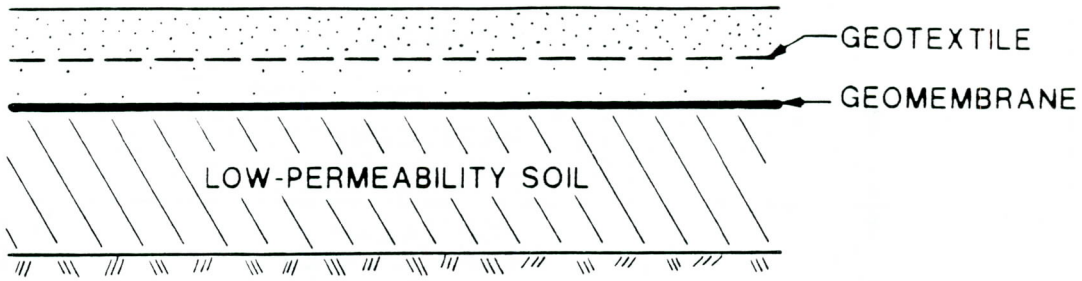
| Fissure Width (ft) | Pressure on Geomembrane ($lb/ft^2$) |
|---|---|
| 0.1 | 25 |
| 0.2 | 50 |
| 0.4 | 100 |
| 0.6 | 150 |
| 0.8 | 200 |
| 1.0 | 250 |
| 1.2 | 300 |
| 1.4 | 350 |
| 1.6 | 400 |
| 1.8 | 450 |
| 2.0 | 500 |



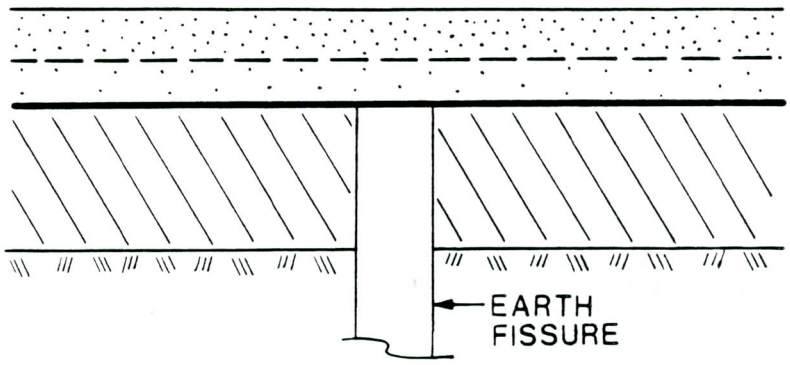

NOTE: THE GRANULAR DRAINAGE LAYER IS ABSENT ON THE LANDFILL SLOPES. THE PROTECTIVE SOIL COVER IS 2 ft THICK ON THE SLOPES.

LINING SYSTEM
CHOLLA SANITARY LANDFILL

a)

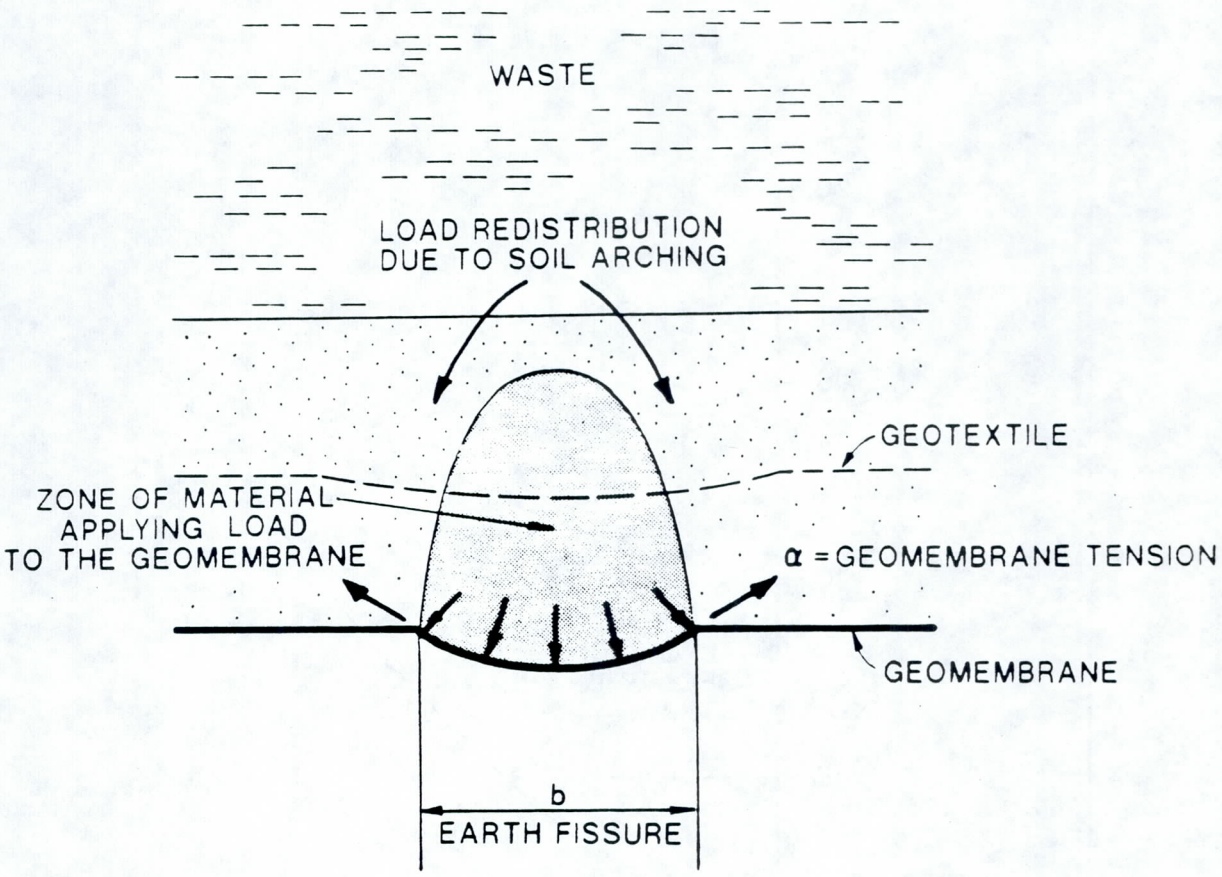



b)

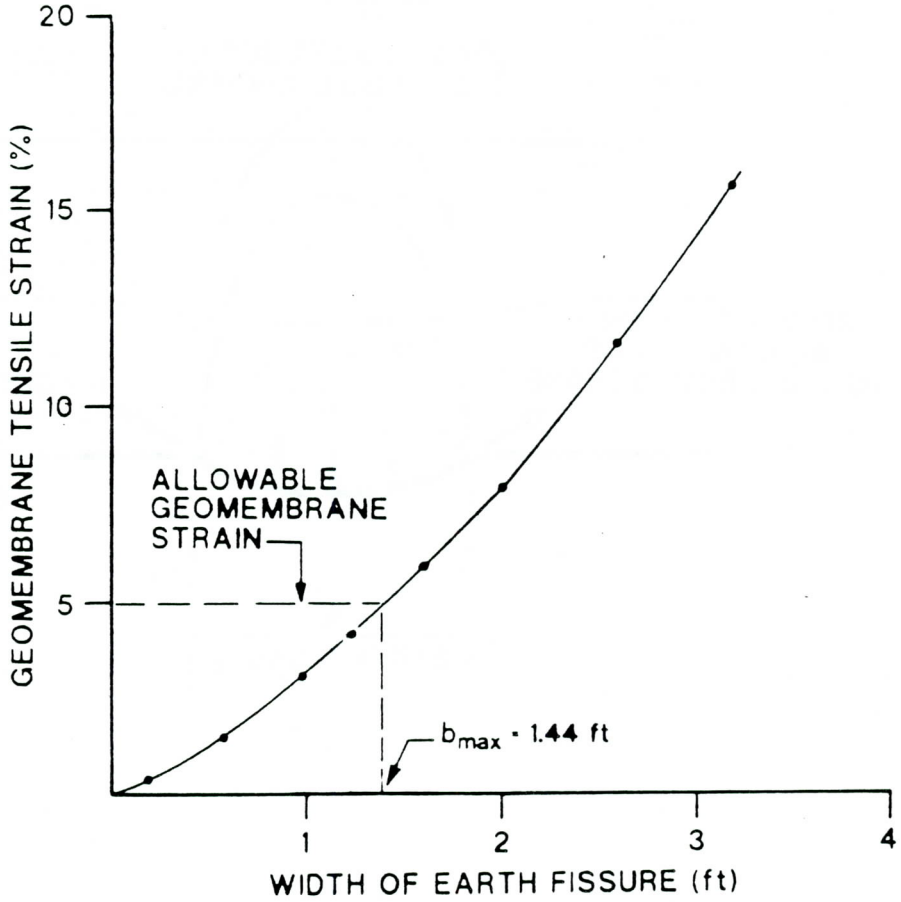



DEVELOPMENT OF EARTH FISSURE
CHOLLA SANITARY LANDFILL

| | |
|---|---|
| GeoServices Inc. | FIGURE NO. 2 |
| CONSULTING ENGINEERS | PROJECT NO. P1419-02 |
| | DOCUMENT NO. N900314 |
| | PAGE NO. |




SOIL ARCHING TENSIONED MEMBRANE MODEL
CHOLLA SANITARY LANDFILL




RELATIONSHIP BETWEEN GEOMEMBRANE
TENSILE STRAIN AND WIDTH OF EARTH FISSURE
CHOLLA SANITARY LANDFILL

| | |
|---|---|
| **GeoServices Inc.** | FIGURE NO. 4 |
| CONSULTING ENGINEERS | PROJECT NO. P1419-02 |
| | DOCUMENT NO. N900314 |
| | PAGE NO. |

# APPENDIX A

# LABORATORY RESULTS
# GEOMEMBRANE TENSION TESTS

BEDSYNTEC,INC.

TENSILE TEST 2

Test type: Tensile

Operator name: SLP

Sample Identification: GA574TM
Interface Type: 1011 Series
Machine Parameters of test:
Sample Rate (pts/sec): 4.00
Crosshead Speed (in/min): 2.000

Instron Corporation
Series IX Automated Materials Testing System v4.05a
Test Date: May 24,1990

Sample Type: 00IV

Humidity ( % ): 50
Temperature (deg. C): 23

SAMPLE ID#:(16 CHAR) G119-015GM00001
LOWER CASE md OR ma: md
CLIENT: GSV/ATLANTA
JOB#: G118-015
CUSTOMER SAMPLE: POLYAMERICA-60 mil

Dimensions:

| | Spec. 1 | Spec. 2 | Spec. 3 | Spec. 4 | Spec. 5 | Spec. 6 |
|---|---|---|---|---|---|---|
| Thickness (in) | .056600 | .059300 | .059600 | .059400 | .059600 | .059500 |
| Width (in) | .25000 | .25000 | .25000 | .25000 | .25000 | .25000 |
| Gauge length (in) | 1.3000 | 1.3000 | 1.3000 | 1.3000 | 1.3000 | 1.3000 |
| Specimen G.L. (in) | 2.5000 | 2.5000 | 2.5000 | 2.5000 | 2.5000 | 2.5000 |

Out of 6 specimens, 1 excluded.

| Specimen Number | LOAD AT YIELD (lb) | STRESS AT YIELD (psi) | STRAIN AT YIELD (%) | TENSION AT YIELD (lbs/in) | LOAD AT BREAK (lb) | STRESS AT BREAK (psi) | STRAIN AT BREAK (%) | TENSION AT BREAK (lbs/in) |
|---|---|---|---|---|---|---|---|---|
| 1 | 36.75 | 2597. | 16.75 | 147.0 | 67.35 | 4760. | 1447. | 269.4 |
| 2 | 36.60 | 2469. | 16.62 | 146.4 | 67.15 | 4530. | 1356. | 268.6 |
| 3 | 38.10 | 2557. | 16.64 | 152.4 | 66.20 | 4443. | 1375. | 264.8 |
| 4 | 38.10 | 2566. | 16.75 | 152.4 | 64.85 | 4367. | 1335. | 259.4 |
| 5 | 39.05 | 2621. | 16.61 | 156.2 | 63.10 | 4235. | 1300. | 252.4 |
| *Excluded* | 39.70 | 2669. | 16.74 | 158.8 | 61.90 | 4161. | 1316. | 247.6 |
| Mean: | 37.72 | 2562. | 16.68 | 150.9 | 65.73 | 4467. | 1363. | 262.9 |
| Standard Deviation: | 1.03 | 58. | .07 | 4.1 | 1.77 | 196. | 55. | 7.1 |

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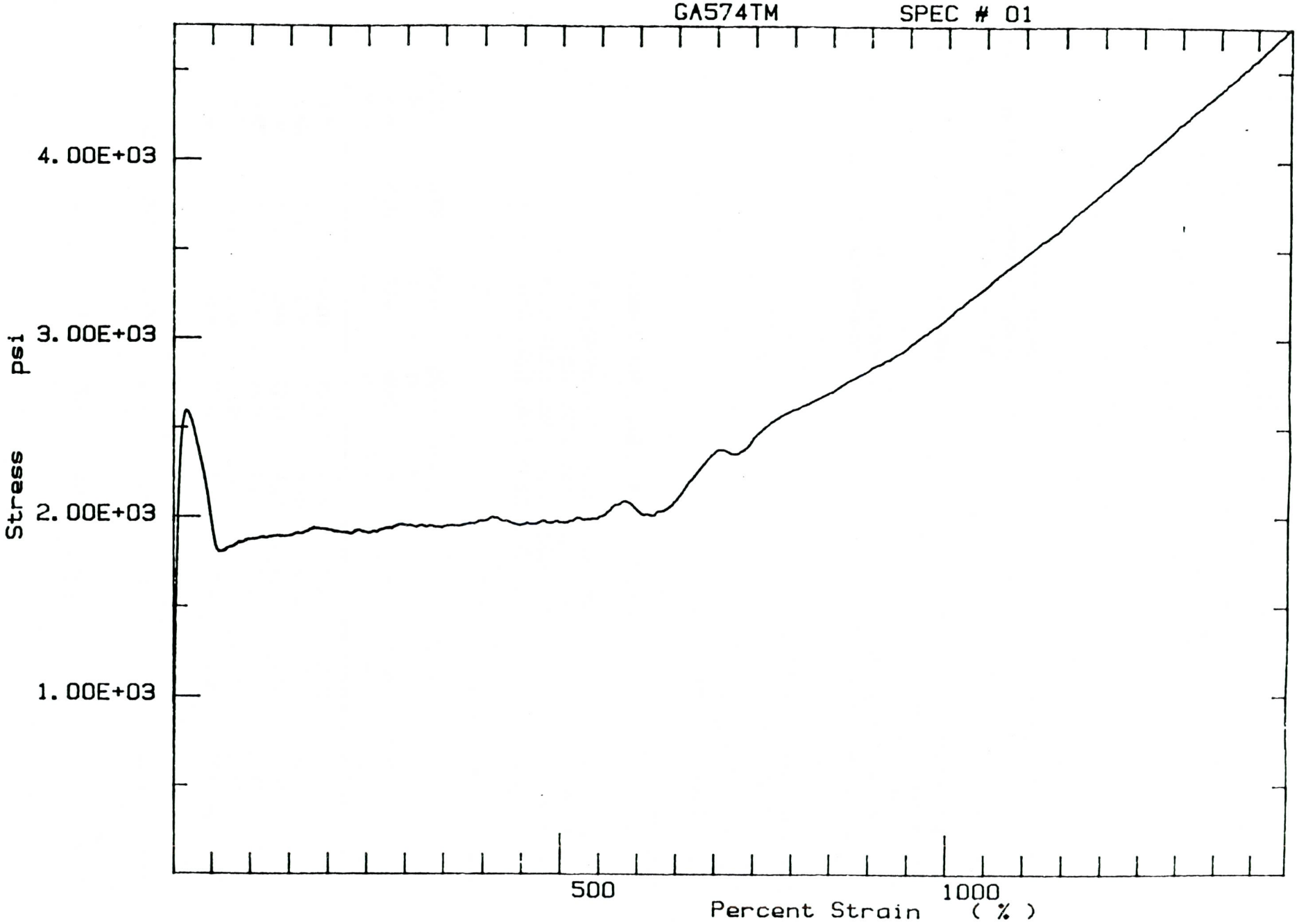
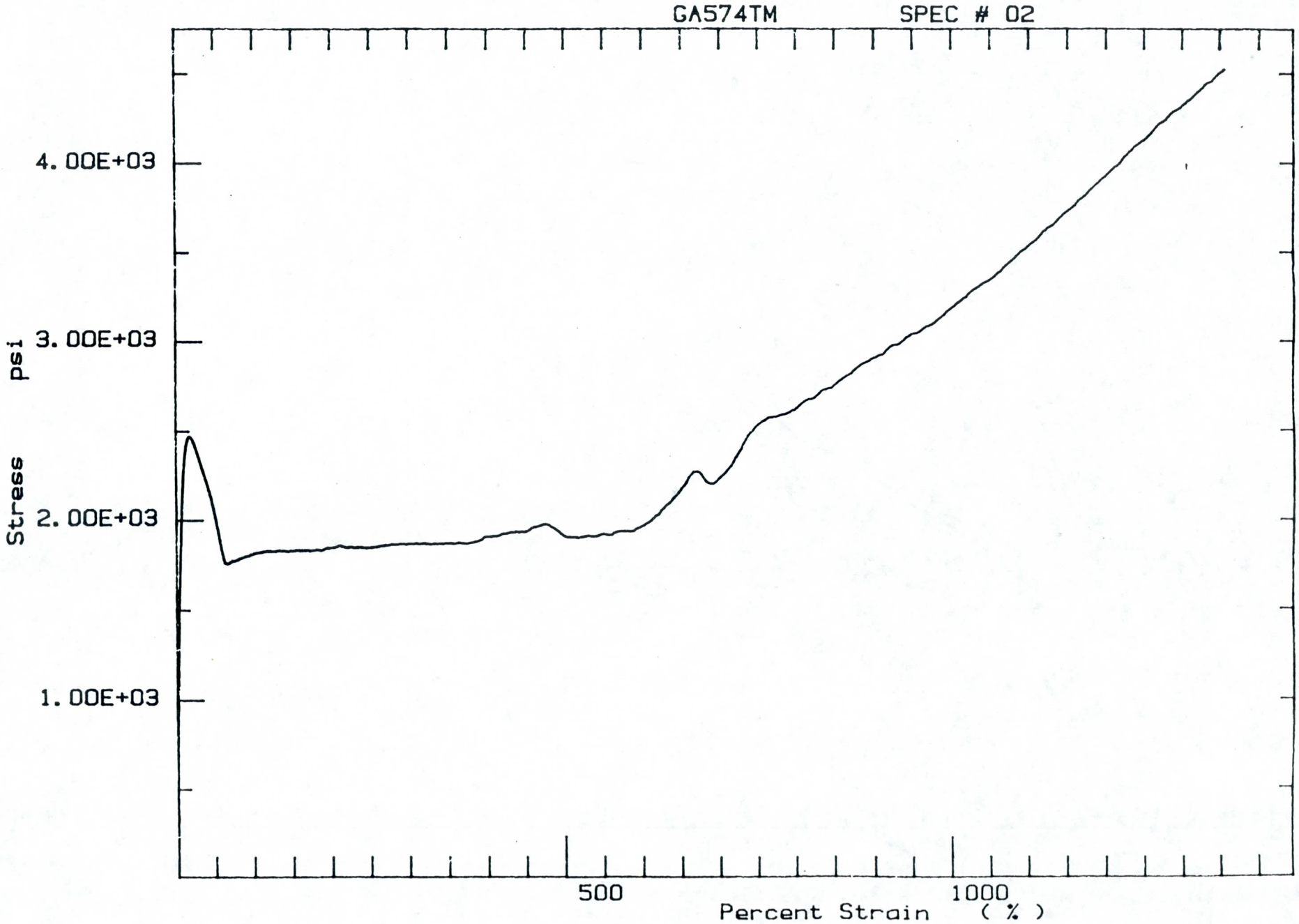

GA574TM
SPEC # 01
4.00E+03
3.00E+03
2.00E+03
1.00E+03
Stress psi
500
1000
Percent Strain ( % )

DRAFT

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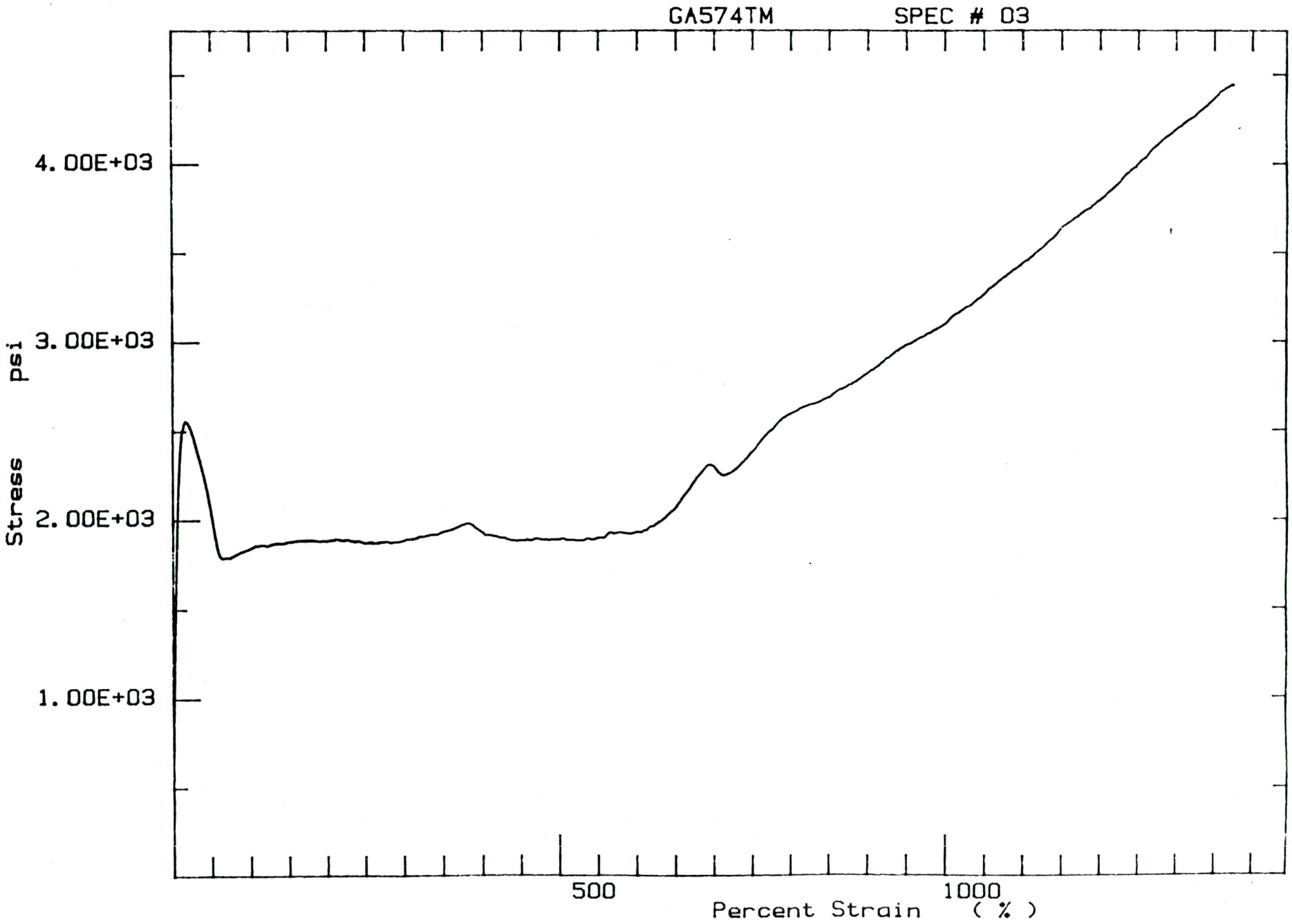
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Stress psi
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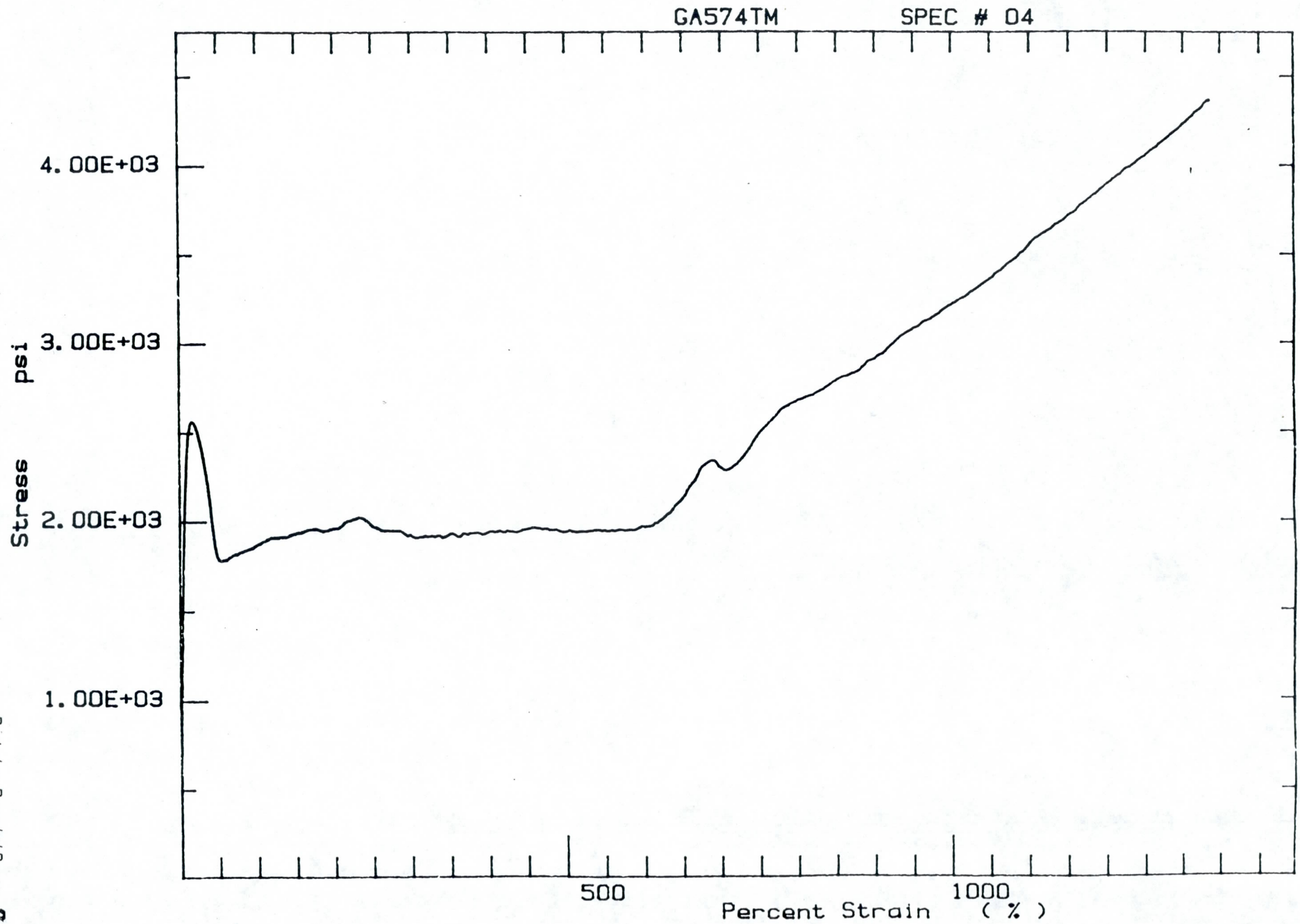
1000
Percent Strain ( % )

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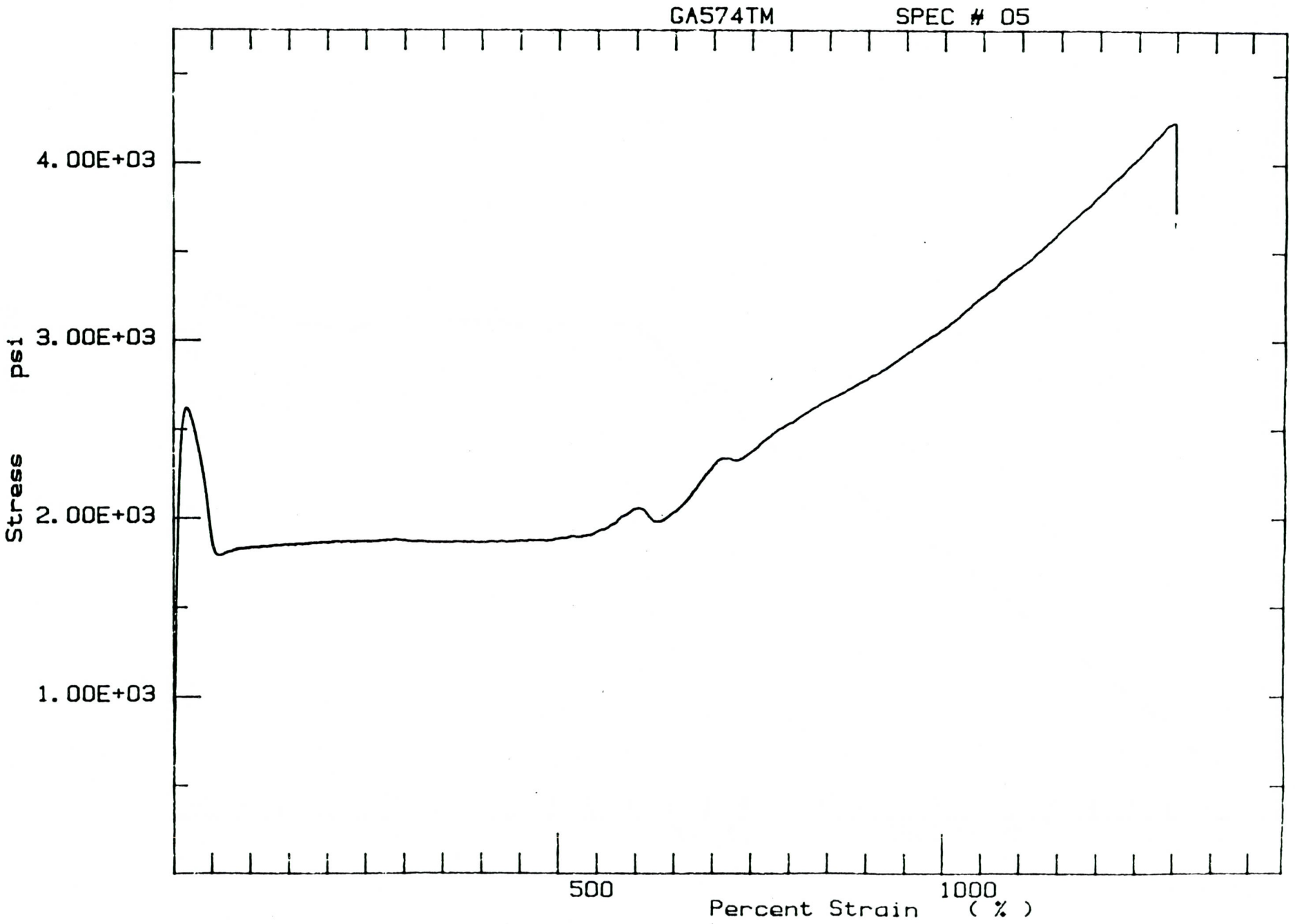
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Stress psi
500
1000
Percent Strain ( % )

GEOSYNTEC, INC.

TENSILE TEST 2

Test type: Tensile

Operator name: SLP

Sample Identification: GA574TX
Interface Type: 1011 Series
Machine Parameters of test:
Sample Rate (pts/sec): 4.00
Crosshead Speed (in/min): 2.000

Instron Corporation
Series IX Automated Materials Testing System v4.05a
Test Date: May 24,1990

Sample Type: 00IV

Humidity ( % ): 50
Temperature (deg. C): 23

SAMPLE ID#:(1a CHAR) G118-015GM00001
LOWER CASE md OR xd: xd
CLIENT: GSV/ATLANTA
JOB#: G118-015
CUSTOMER SAMPLE: POLYAMERICA-60 mil

Dimensions:

| | Spec. 1 | Spec. 2 | Spec. 3 | Spec. 4 | Spec. 5 | Spec. 6 |
|---|---|---|---|---|---|---|
| Thickness (in) | .059700 | .059600 | .057600 | .060000 | .060400 | .059500 |
| Width (in) | .25000 | .25000 | .25000 | .25000 | .25000 | .25000 |
| Gauge length (in) | 1.3000 | 1.3000 | 1.3000 | 1.3000 | 1.3000 | 1.3000 |
| Specimen G.L. (in) | 2.5000 | 2.5000 | 2.5000 | 2.5000 | 2.5000 | 2.5000 |

Out of 6 specimens, 1 excluded.

| Specimen Number | LOAD AT YIELD (lb) | STRESS AT YIELD (psi) | STRAIN AT YIELD (%) | TENSION AT YIELD (lbs/in) | LOAD AT BREAK (lb) | STRESS AT BREAK (psi) | STRAIN AT BREAK (%) | TENSION AT BREAK (lbs/in) |
|---|---|---|---|---|---|---|---|---|
| 1 | 41.20 | 2760. | 15.35 | 164.8 | 69.55 | 4660. | 1498. | 278.2 |
| 2 | 40.35 | 2708. | 14.68 | 161.4 | 70.30 | 4718. | 1472. | 281.2 |
| 3 | 39.55 | 2747. | 15.35 | 158.2 | 60.35 | 4191. | 1368. | 241.4 |
| *Excluded* | 40.05 | 2670. | 15.99 | 160.2 | 58.75 | 3917. | 1261. | 235.0 |
| 5 | 41.30 | 2735. | 15.98 | 165.2 | 69.90 | 4629. | 1509. | 279.6 |
| 6 | 40.05 | 2692. | 15.35 | 160.2 | 72.00 | 4840. | 1500. | 288.0 |
| Mean: | 40.49 | 2729. | 15.34 | 162.0 | 68.42 | 4608. | 1470. | 273.7 |
| Standard Deviation: | .75 | 28. | .46 | 3.0 | 4.61 | 247. | 58. | 18.4 |

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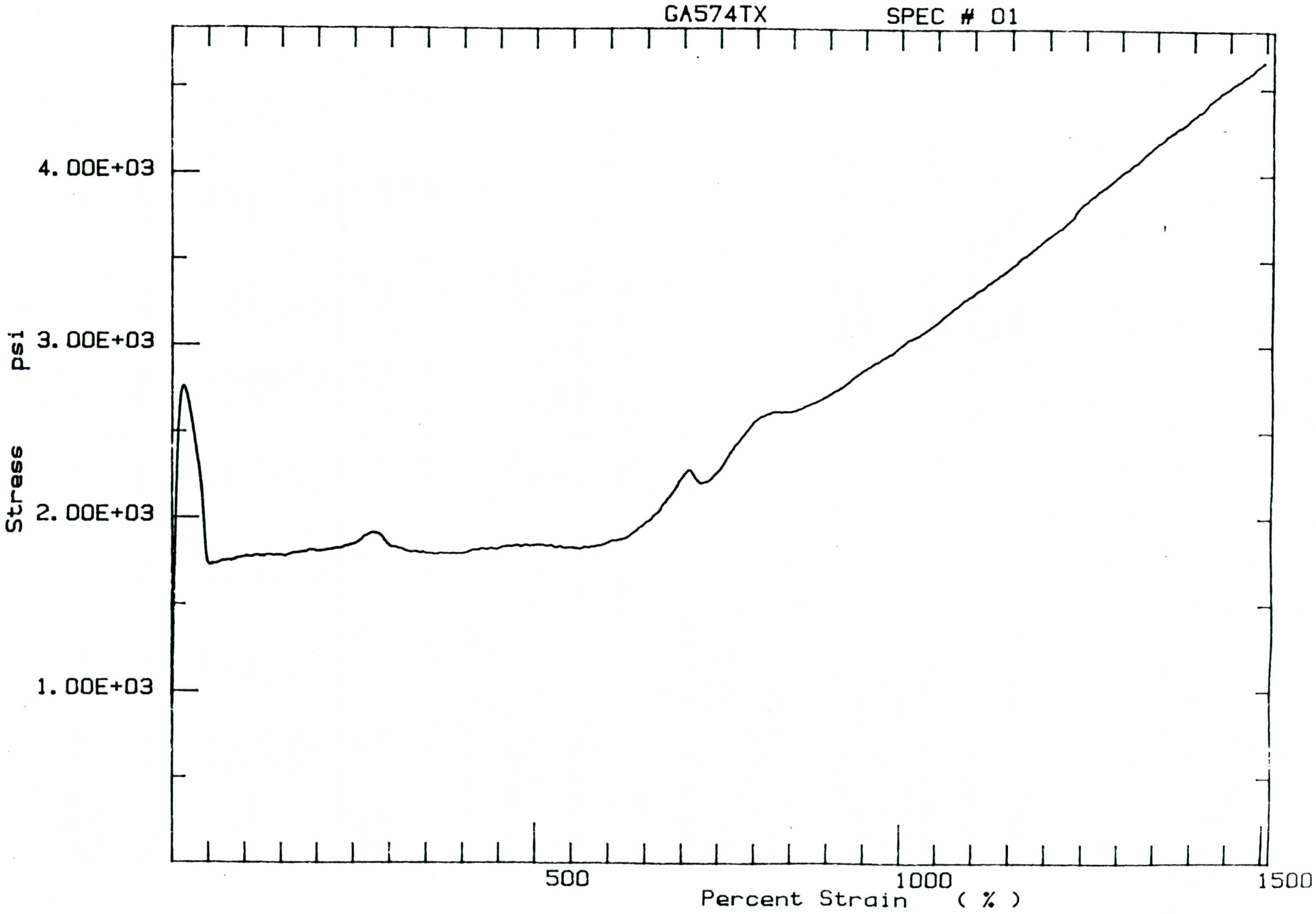
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1000
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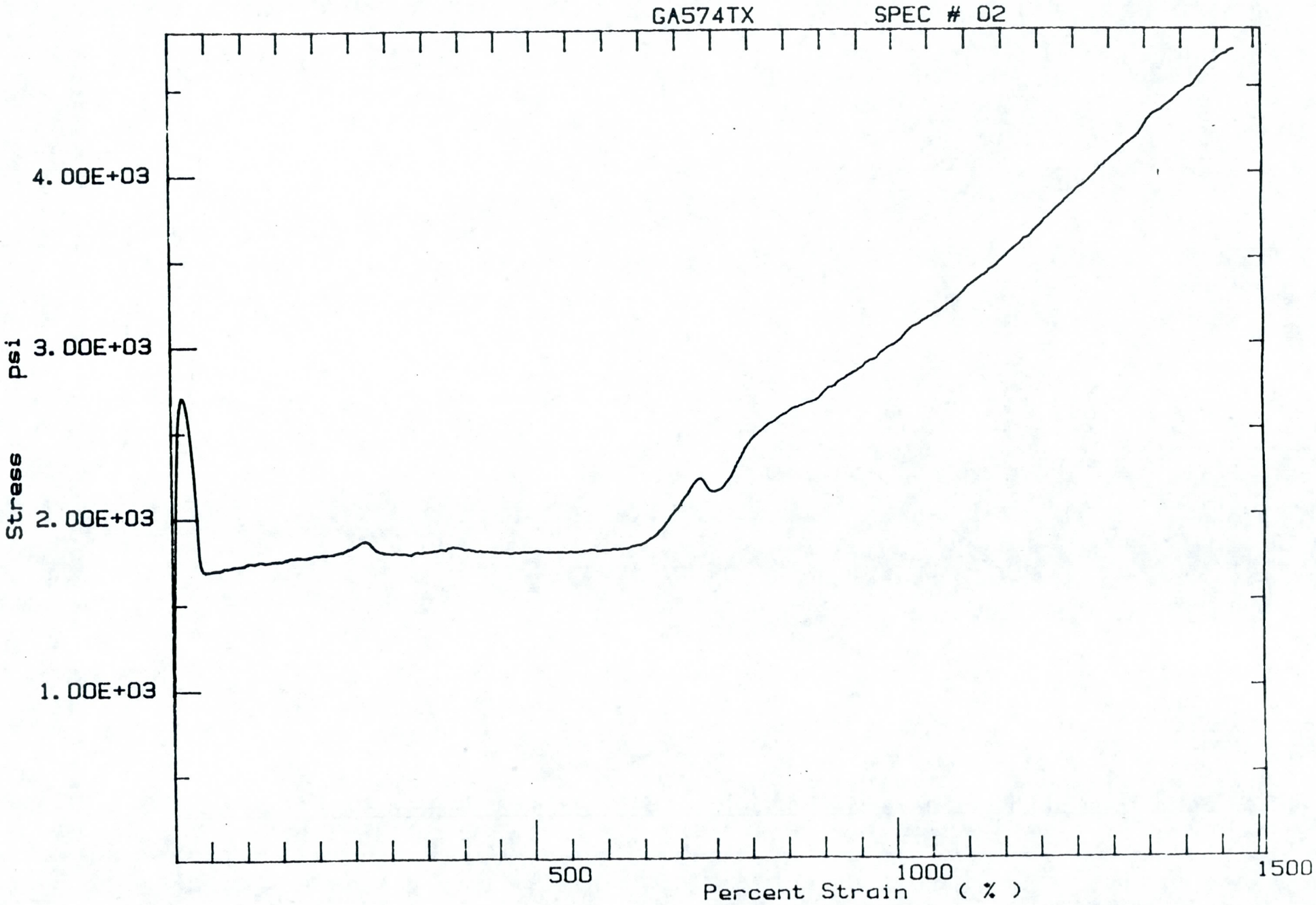
Percent Strain ( % )

DRAFT

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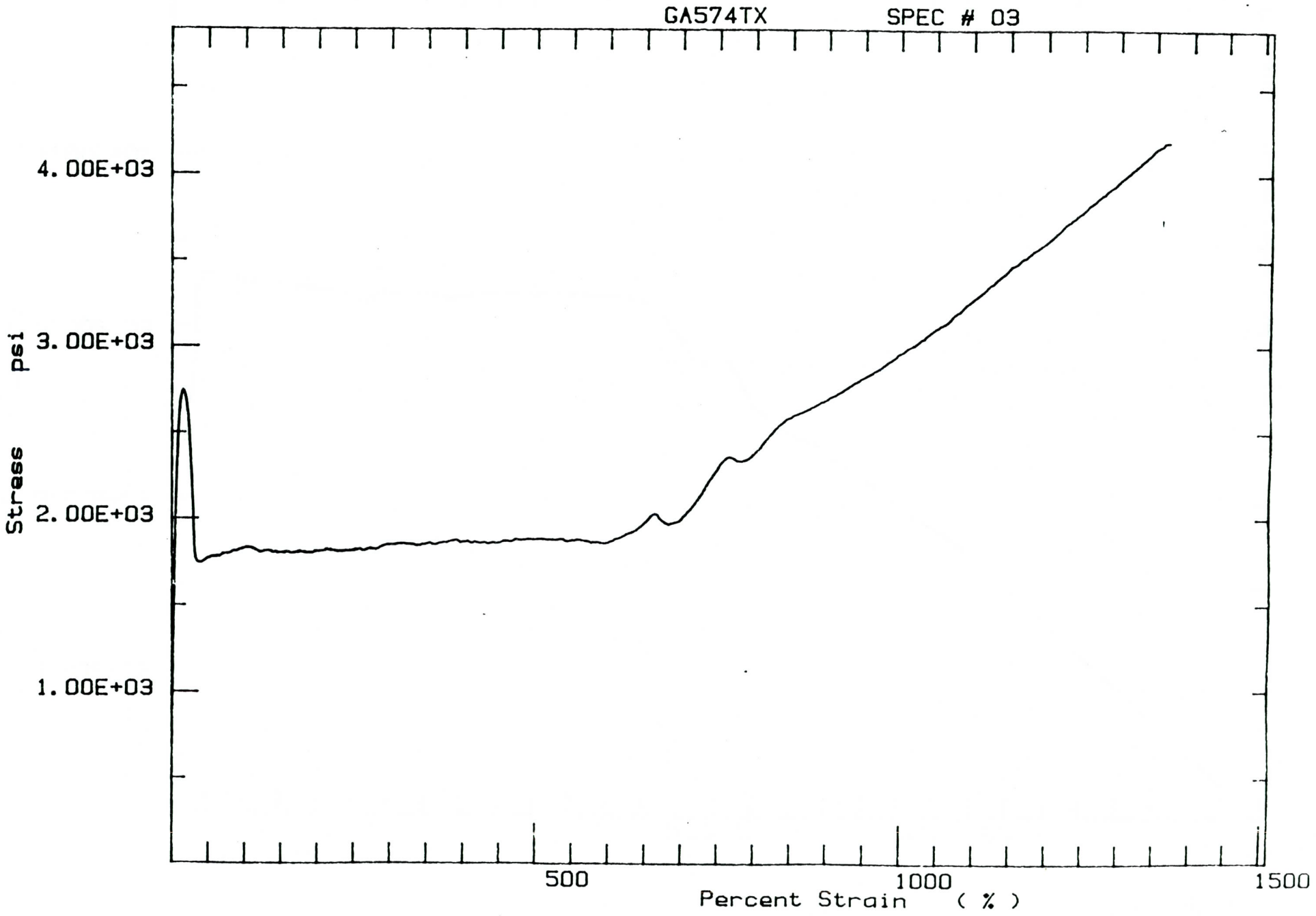
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1000
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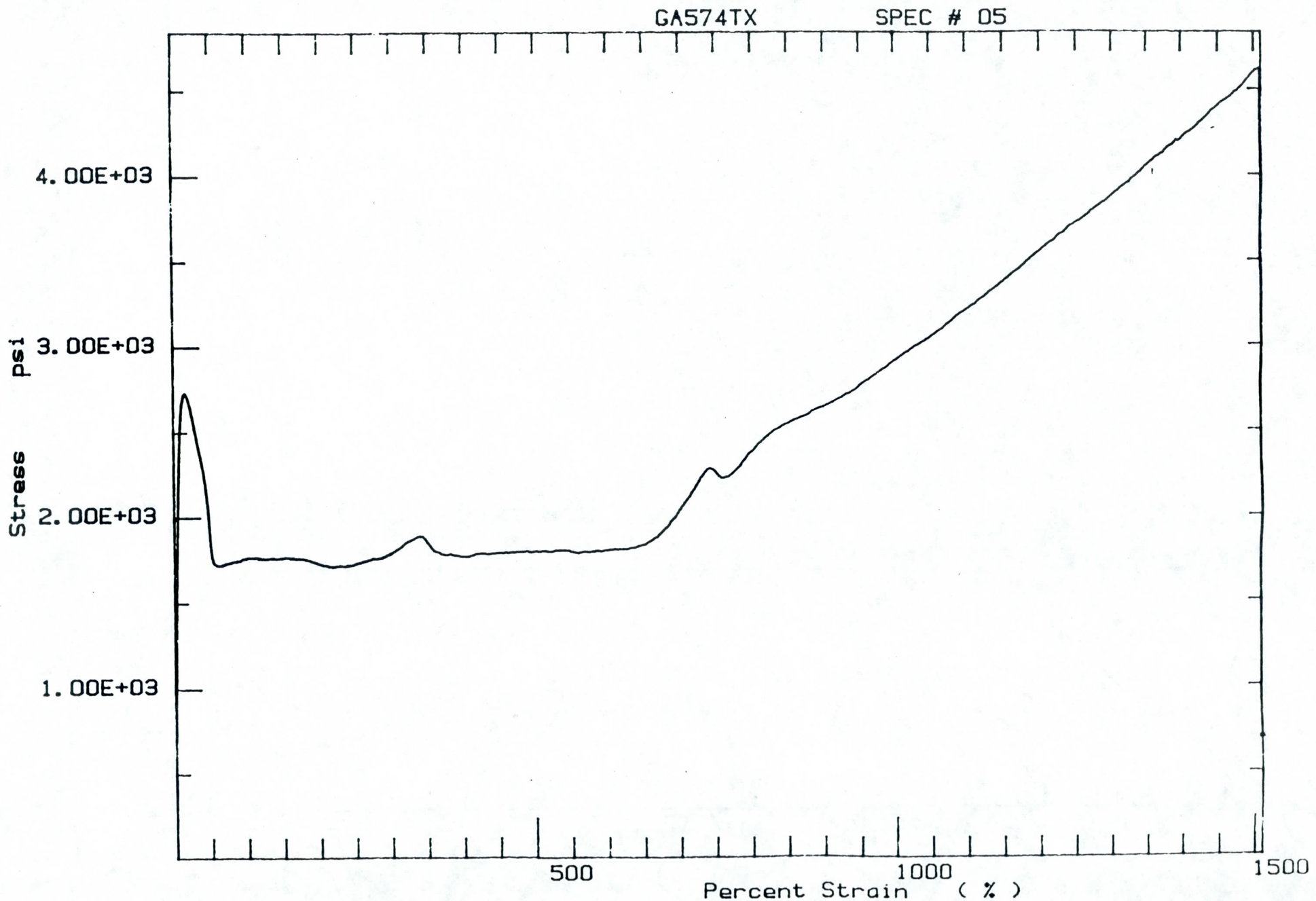
Percent Strain ( % )

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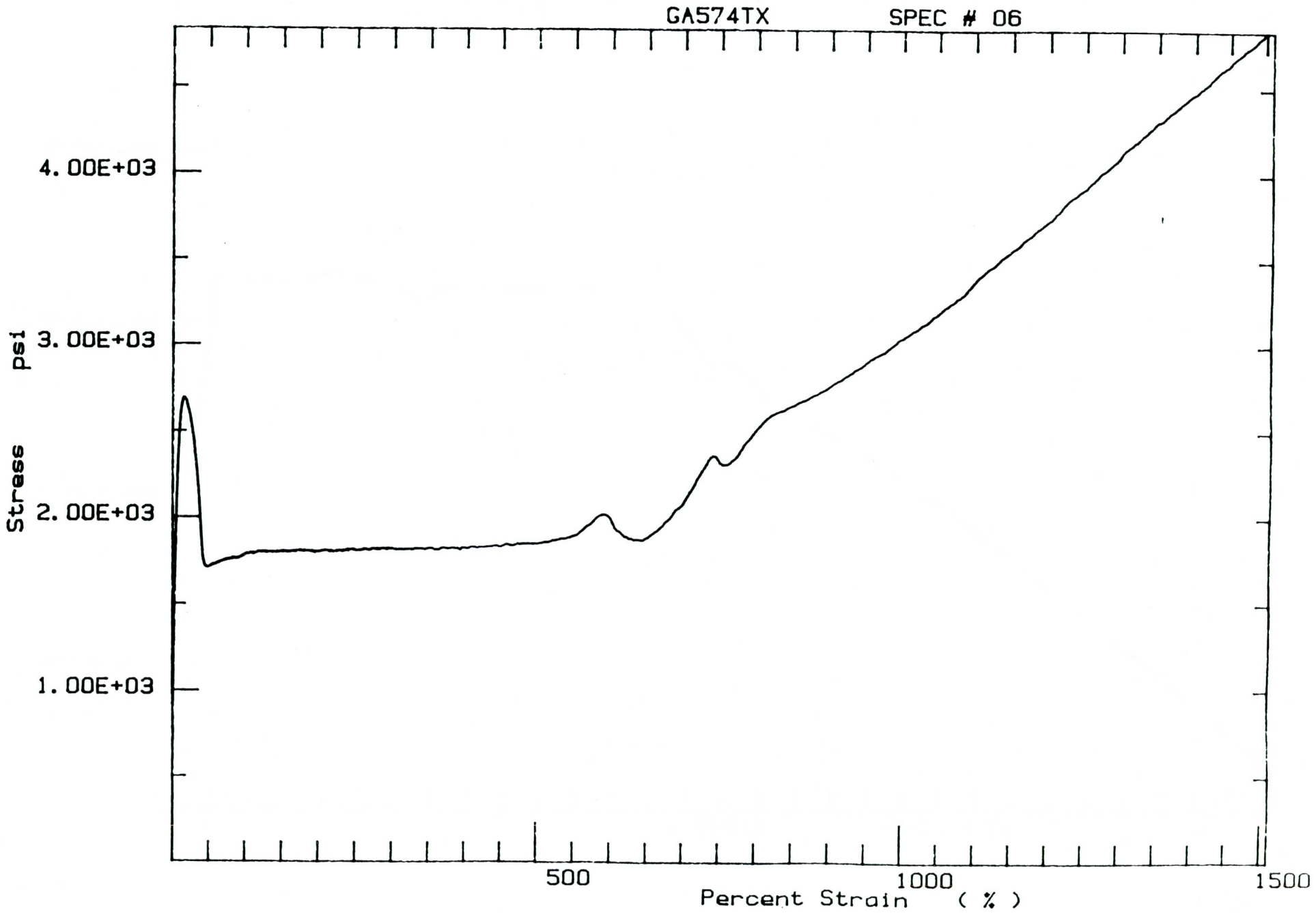
GA574TX
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3.00E+03
2.00E+03
1.00E+03
Stress psi
500
1000
1500
Percent Strain ( % )

[illegible] INC.

[illegible] Instron Corporation
Series IX Automated Materials Testing System v4.05a
Test Date: May 24,1990

[illegible]

Sample [illegible]: G4575TM
[illegible] Series
[illegible] test:
[illegible] 4.[illegible]
[illegible] (in/min): 2.000

Sample Type: 001V

Humidity ( % ): 50
Temperature (deg. C): 23

SAMPLE [illegible] 6118-015GM00002
[illegible] md
[illegible] GSV/ATLANTA
JOB#: 6118-015
[illegible] HSC-60 mil

[illegible]

| | Spec. 1 | Spec. 2 | Spec. 3 | Spec. 4 | Spec. 5 | Spec. 6 |
|---|---|---|---|---|---|---|
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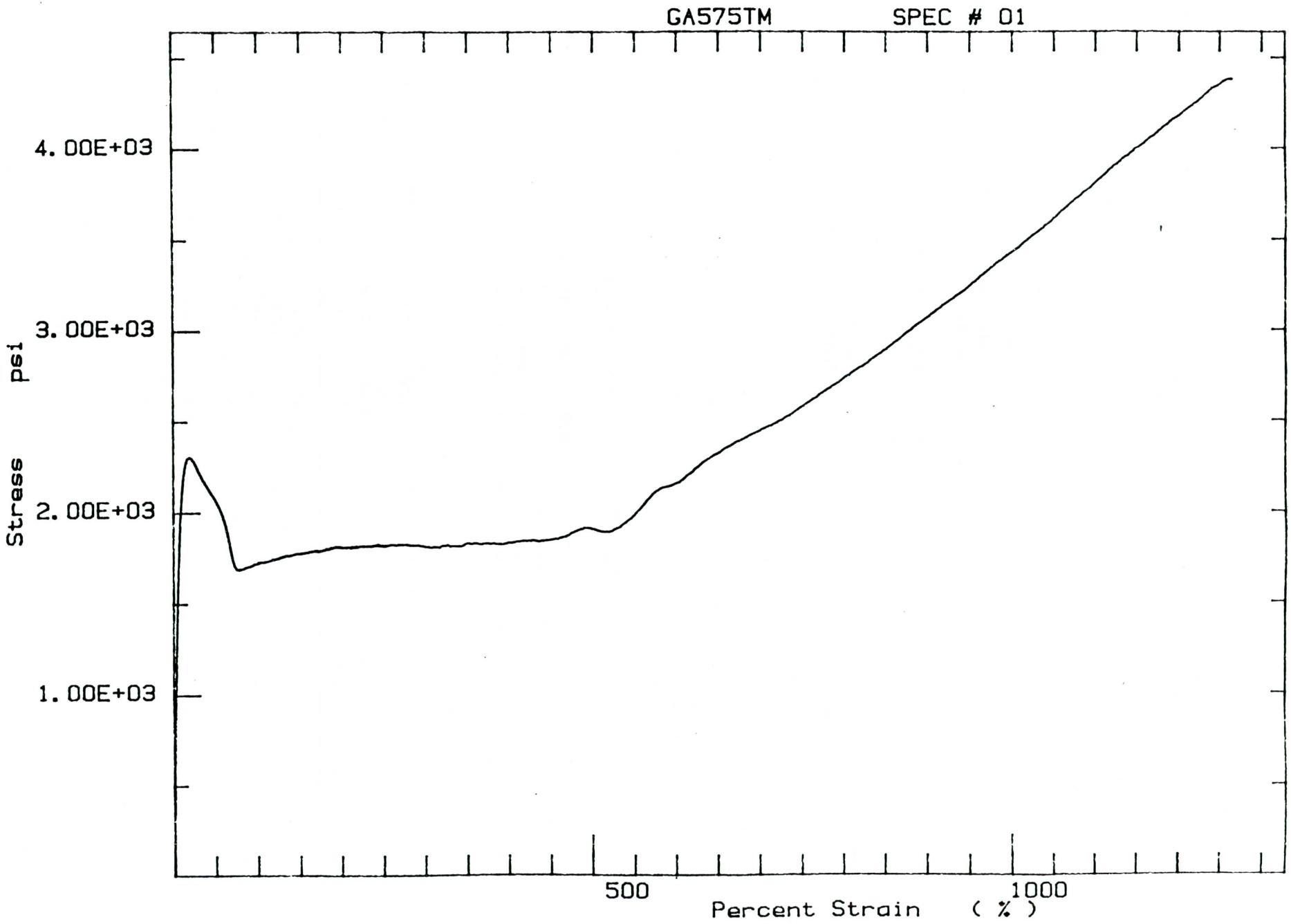
| [illegible] | .25000 | .25000 | .25000 | .25000 | .25000 | .25000 |
| [illegible] | 1.3000 | 1.3000 | 1.3000 | 1.3000 | 1.3000 | 1.3000 |
| [illegible] | 2.5000 | 2.5000 | 2.5000 | 2.5000 | 2.5000 | 2.5000 |

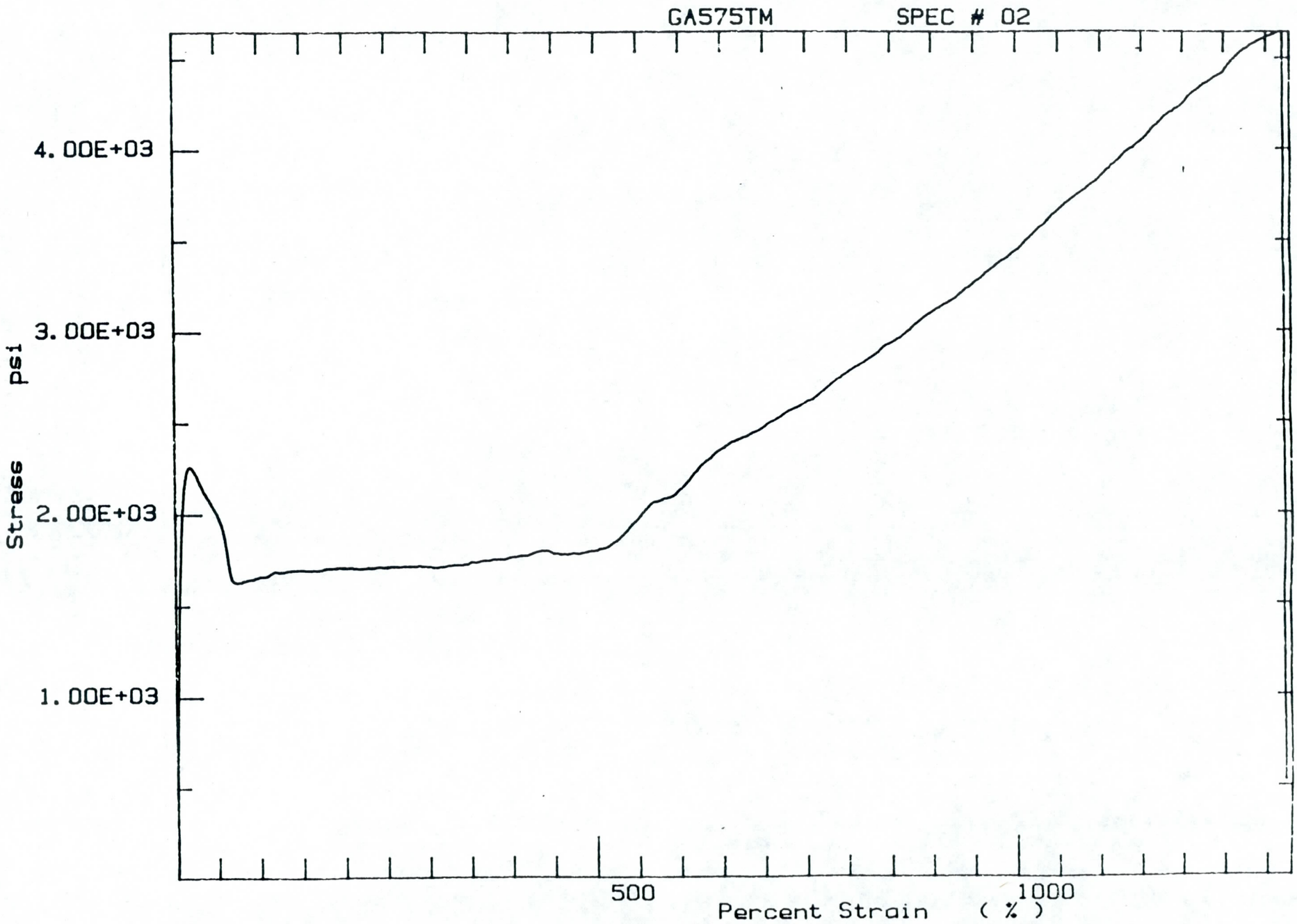
[illegible] 1 excluded.

| Specimen Number | [illegible] | STRESS AT YIELD (psi) | STRAIN AT YIELD (%) | TENSION AT YIELD (lbs/in) | LOAD AT BREAK (lb) | STRESS AT BREAK (psi) | STRAIN AT BREAK (%) | TENSION AT BREAK (lbs/in) |
|---|---|---|---|---|---|---|---|---|
| | [illegible] | 2307. | 19.70 | 138.2 | 65.60 | 4381. | 1265. | 262.4 |
| | [illegible] | 2264. | 18.55 | 137.0 | 70.15 | 4638. | 1321. | 280.6 |
| | [illegible] | 2324. | 17.90 | 139.2 | 65.65 | 4384. | 1285. | 262.6 |
| [illegible] | [illegible] | 2295. | 17.28 | 138.6 | 65.60 | 4344. | 1225. | 262.4 |
| | [illegible] | 2320. | 17.33 | 140.6 | 69.25 | 4571. | 1282. | 277.0 |
| | [illegible] | 2349. | 18.04 | 141.2 | 68.40 | 4552. | 1328. | 273.6 |
| Mean | [illegible] | 2313. | 18.31 | 139.2 | 67.81 | 4505. | 1296. | 271.2 |
| Standard Deviation | [illegible] | 31. | .89 | 1.7 | 2.09 | 117. | 27. | 8.4 |

Printed on Recycled Paper


GA575TM
SPEC # 01
4.00E+03
3.00E+03
2.00E+03
1.00E+03
Stress psi
500
1000
Percent Strain ( % )

DRAFT

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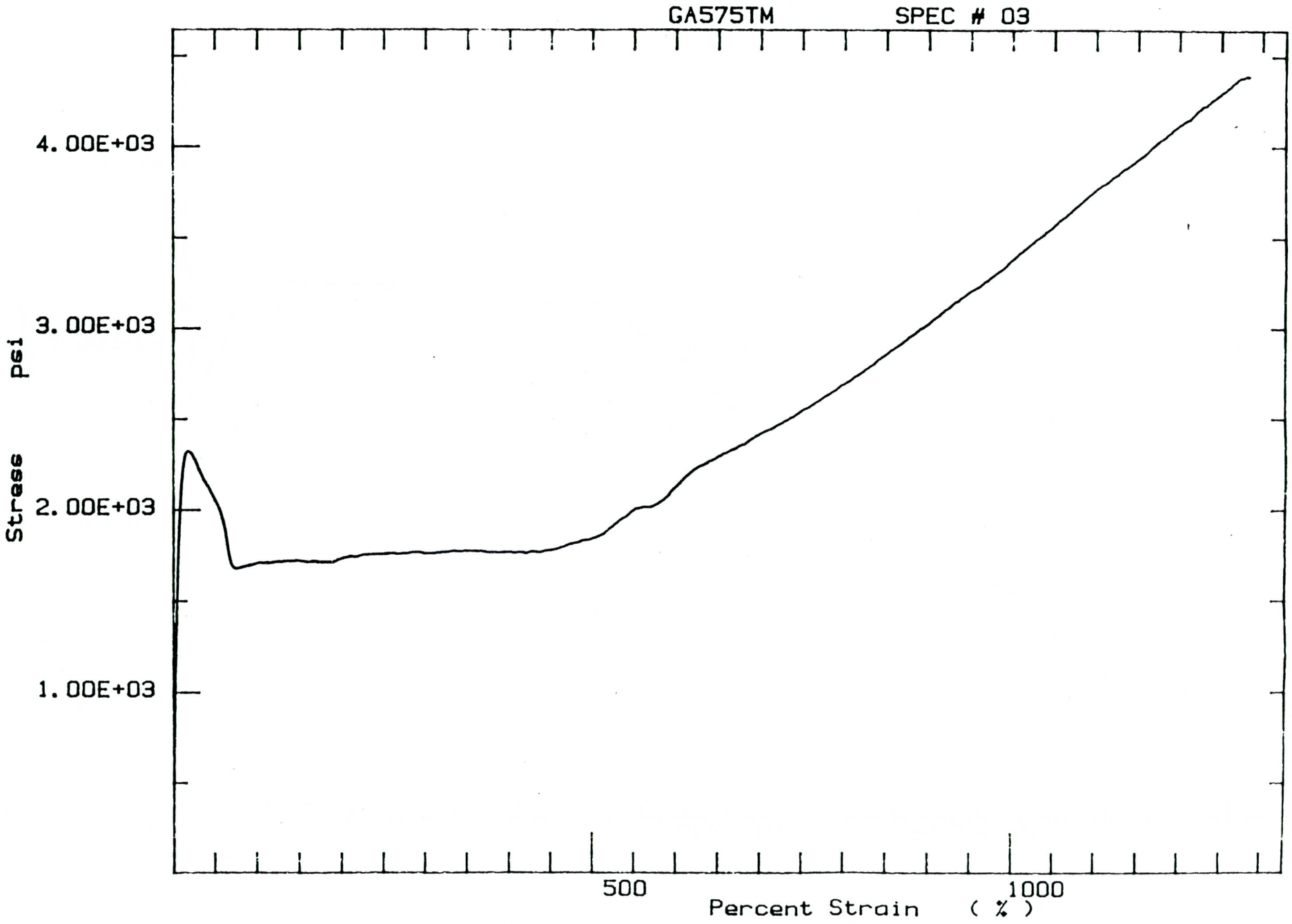
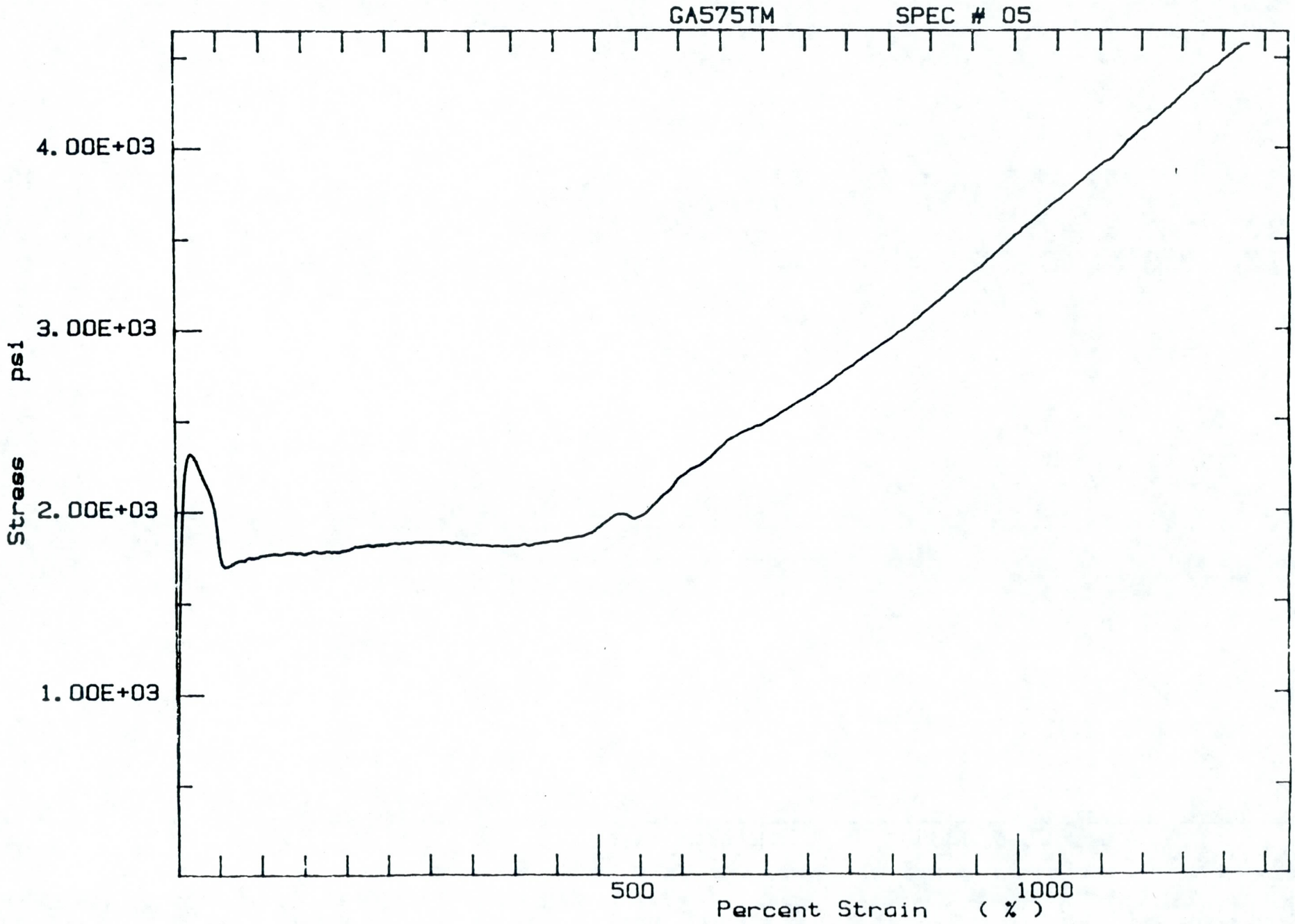

GA575TM
SPEC # 03
4.00E+03
3.00E+03
2.00E+03
1.00E+03
Stress psi
500
1000
Percent Strain ( % )

DRAFT

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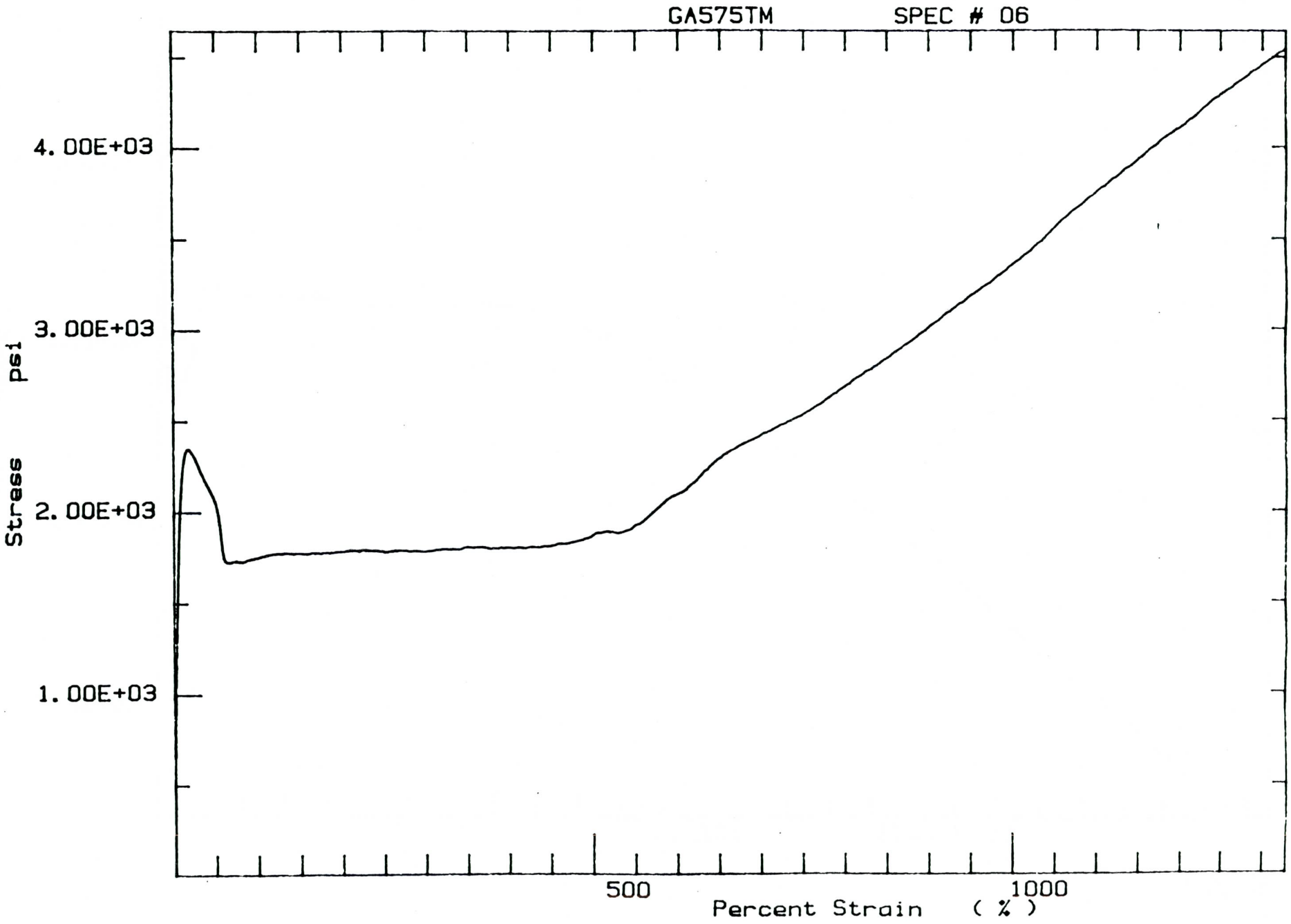
GA575TM
SPEC # 06
4.00E+03
3.00E+03
2.00E+03
1.00E+03
Stress psi
500
1000
Percent Strain ( % )

GEOSYNTEC, INC.

TENSILE TEST 1

Test type: Tensile

Operator name: SLP

Sample Identification: GA575TX
Interface Type: 1011 Series
Machine Parameters of test:
Sample Rate (pts/sec): 4.00
Crosshead Speed (in/min): 2.000

Instron Corporation
Series IX Automated Materials Testing System v4.05a
Test Date: May 24,1990

Sample Type: 00IV

Humidity ( % ): 50
Temperature (deg. C): 23

SAMPLE ID#: (15 CHAR) G118-015GM00002
LOWER CASE ad OR xd: xd
CLIENT: GSV/ATLANTA
JOB#: G118-015
CUSTOMER SAMPLE: NSC-60 #1

Dimensions:

| | Spec. 1 | Spec. 2 | Spec. 3 | Spec. 4 | Spec. 5 | Spec. 6 |
|---|---|---|---|---|---|---|
| Thickness (in) | .060500 | .060300 | .060200 | .060000 | .060400 | .060500 |
| Width (in) | .25000 | .25000 | .25000 | .25000 | .25000 | .25000 |
| Gauge length (in) | 1.3000 | 1.3000 | 1.3000 | 1.3000 | 1.3000 | 1.3000 |
| Specimen G.L. (in) | 2.5000 | 2.5000 | 2.5000 | 2.5000 | 2.5000 | 2.5000 |

Out of 6 specimens, 1 excluded.

| Specimen Number | LOAD AT YIELD (lb) | STRESS AT YIELD (psi) | STRAIN AT YIELD (%) | TENSION AT YIELD (lbs/in) | LOAD AT BREAK (lb) | STRESS AT BREAK (psi) | STRAIN AT BREAK (%) | TENSION AT BREAK (lbs/in) |
|---|---|---|---|---|---|---|---|---|
| 1 | 37.25 | 2463. | 15.34 | 149.0 | 78.85 | 5213. | 1530. | 315.4 |
| 2 | 37.70 | 2501. | 14.82 | 150.8 | 76.15 | 5051. | 1524. | 304.6 |
| 3 | 37.90 | 2518. | 14.19 | 151.6 | 71.70 | 4764. | 1417. | 286.8 |
| *Excluded* | 37.65 | 2510. | 14.70 | 150.6 | 61.85 | 4123. | 1230. | 247.4 |
| 5 | 37.35 | 2487. | 15.99 | 150.2 | 73.25 | 4851. | 1480. | 293.0 |
| 6 | 36.35 | 2403. | 14.71 | 145.4 | 75.95 | 5021. | 1473. | 303.8 |
| Mean: | 37.35 | 2474. | 15.01 | 149.4 | 75.18 | 4980. | 1485. | 300.7 |
| Standard Deviation: | .61 | 45. | .68 | 2.4 | 2.78 | 176. | 46. | 11.1 |

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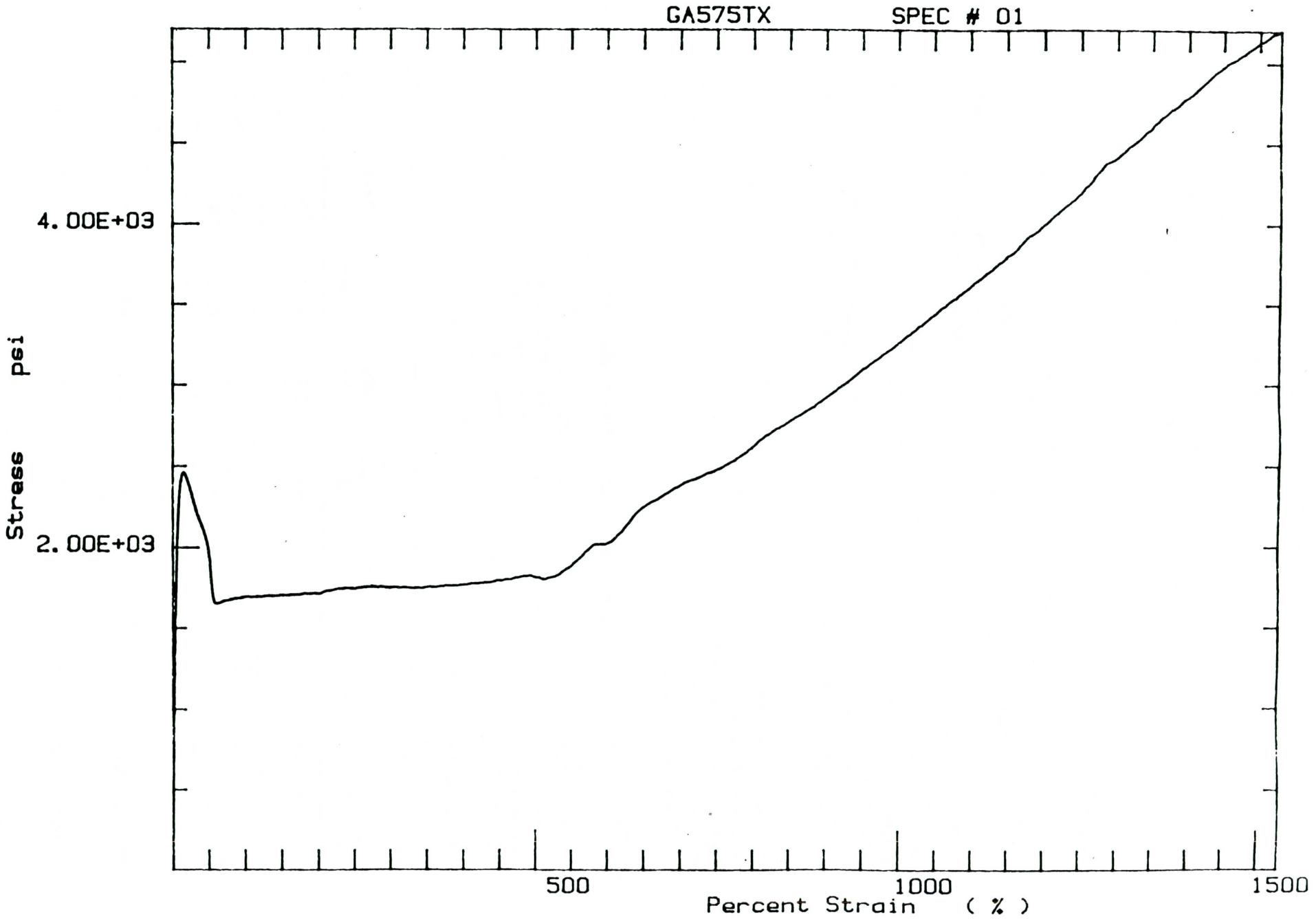
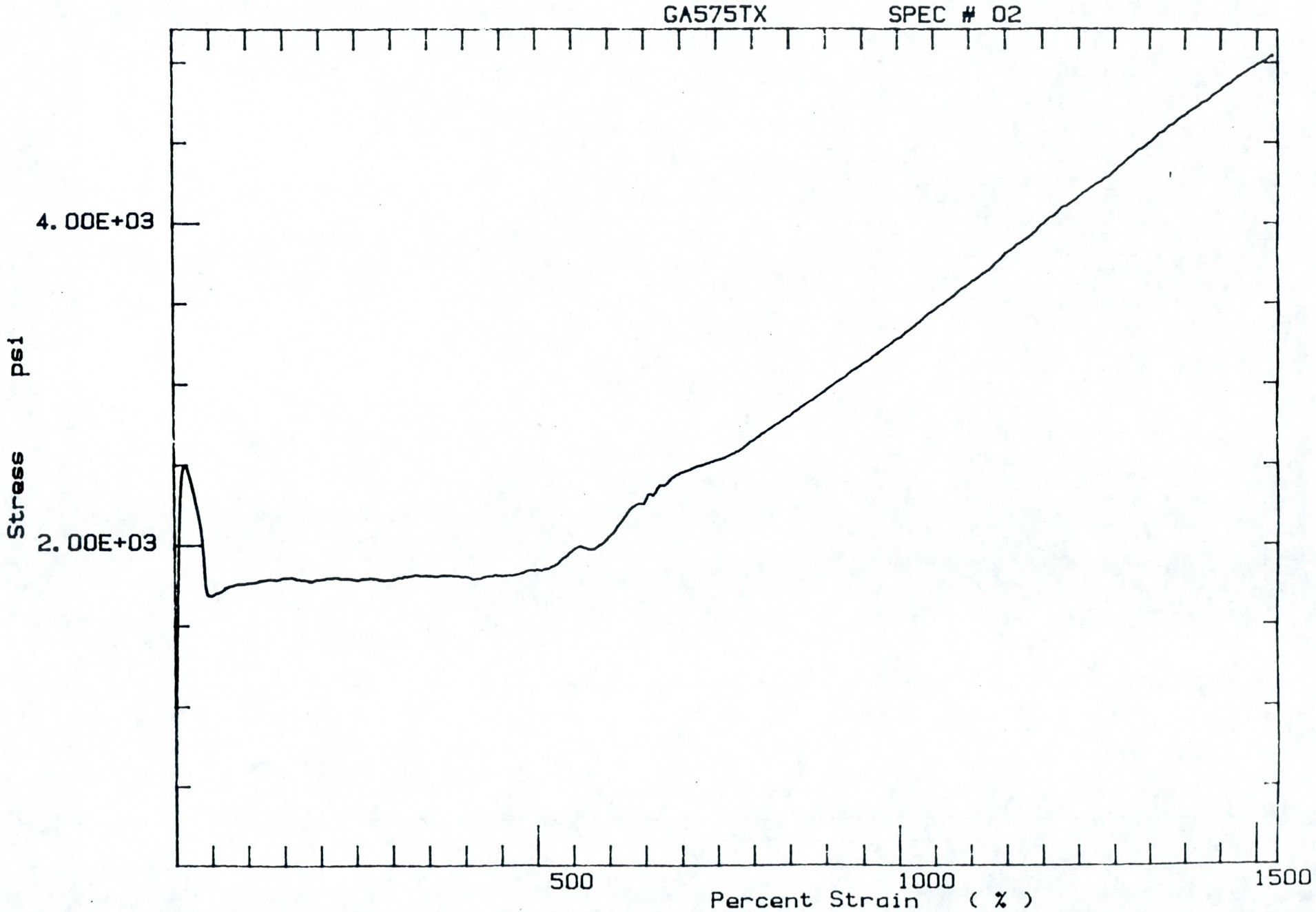

GA575TX
SPEC # 01
4.00E+03
2.00E+03
Stress psi
500
1000
1500
Percent Strain ( % )

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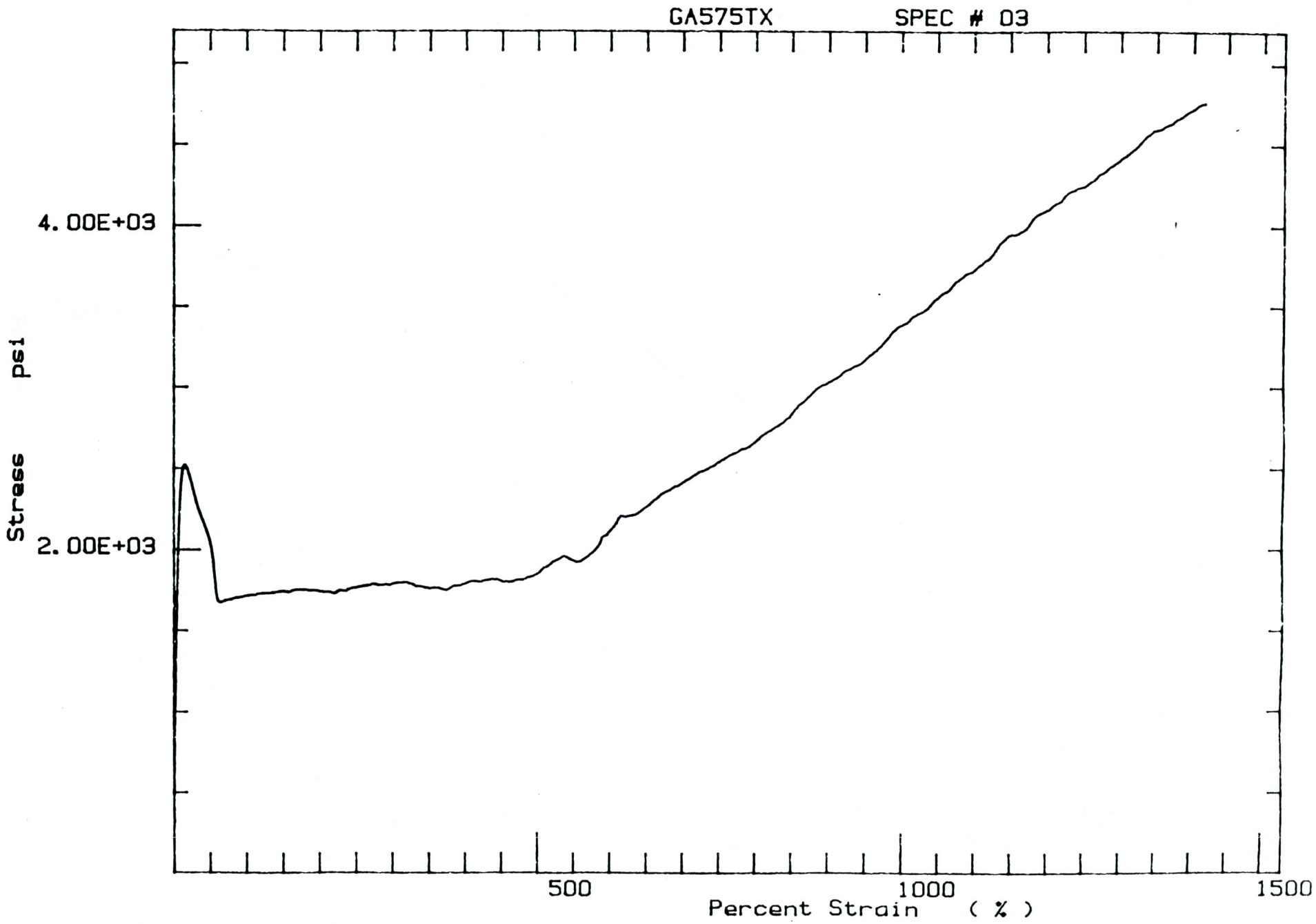
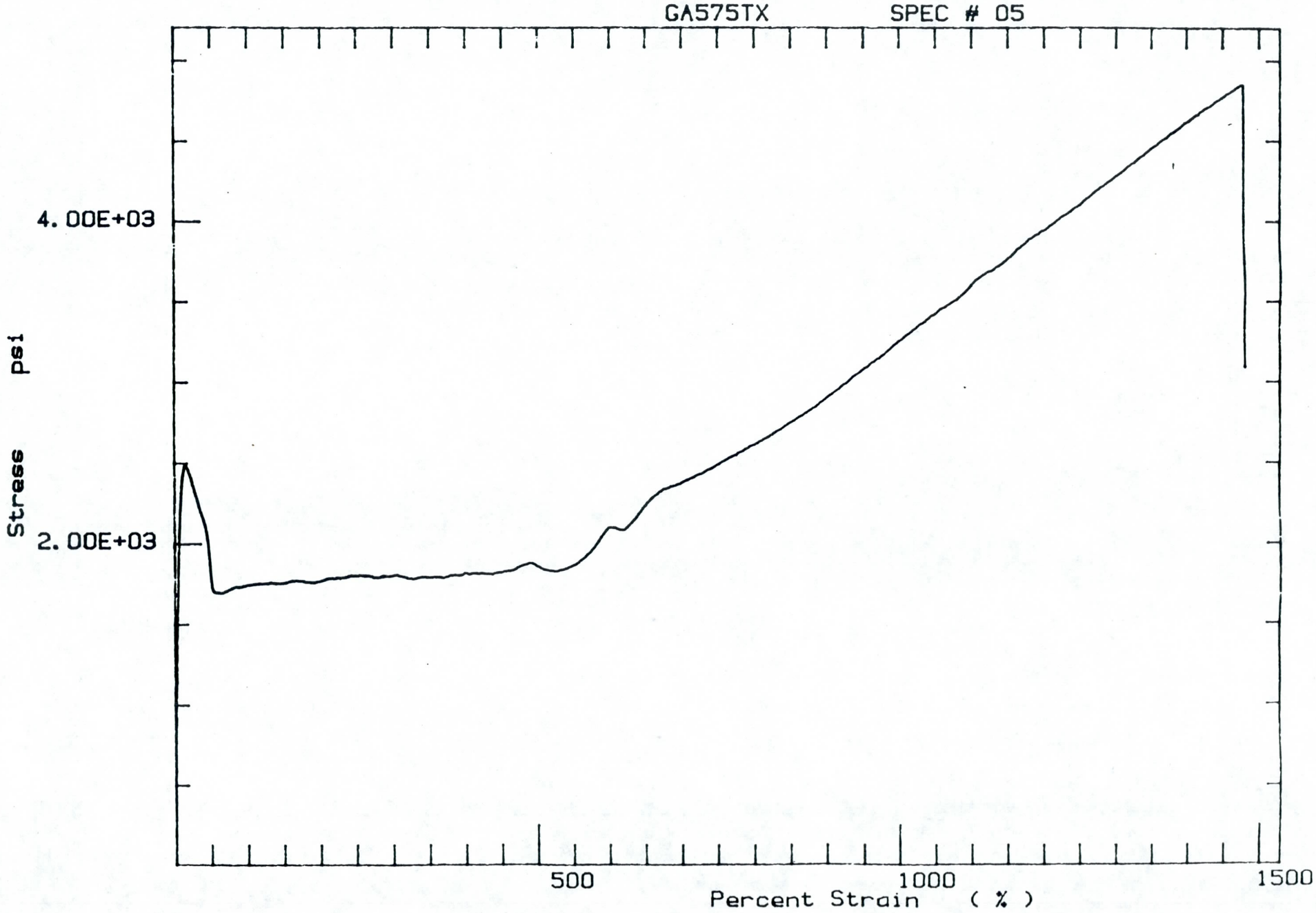

GA575TX
SPEC # 03
4.00E+03
2.00E+03
Stress psi
500
1000
1500
Percent Strain ( % )

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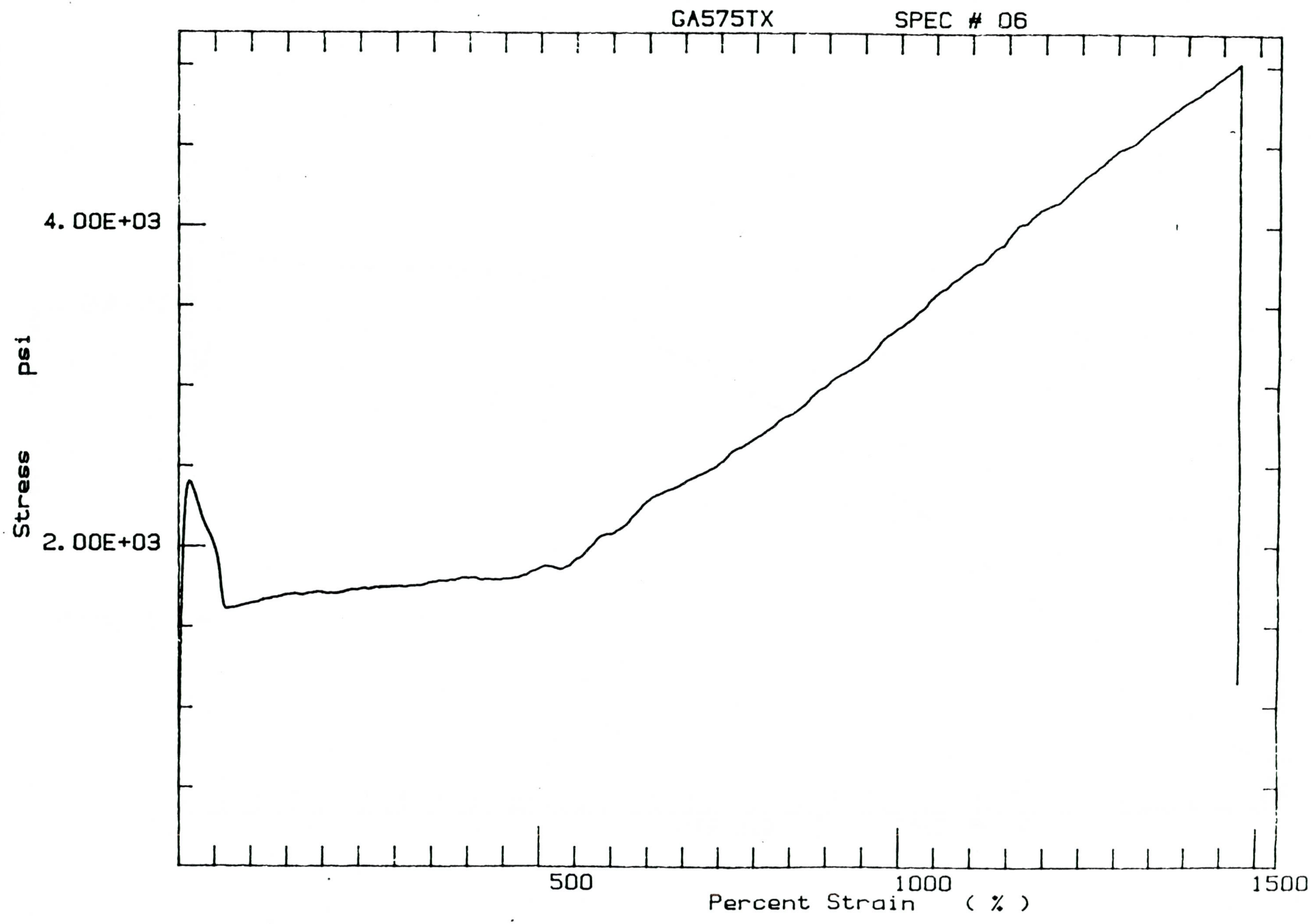
GA575TX
SPEC # 06
4.00E+03
2.00E+03
Stress psi
500
1000
1500
Percent Strain ( % )

GEOSYNTEC,INC.

TENSILE TEST I

Test type: Tensile

Operator name: SLP

Sample Identification: GA57sTM
Interface Type: 1011 Series
Machine Parameters of test:
Sample Rate (pts/sec): 4.00
Crosshead Speed (in/min): 2.000

Instron Corporation
Series IX Automated Materials Testing System v4.05a
Test Date: May 24,1990

Sample Type: 00IV

Humidity ( % ): 50
Temperature (deg. C): 23

SAMPLE ID#:(16 CHAR) G118-015GM00003
LOWER CASE mm OR yd: mm
CLIENT: GSV/ATLANTA
JOB#: G118-015
CUSTOMER SAMPLE: SLT-60 mil

Dimensions:

| | Spec. 1 | Spec. 2 | Spec. 3 | Spec. 4 | Spec. 5 | Spec. 6 |
|---|---|---|---|---|---|---|
| Thickness (in) | .065400 | .072800 | .075500 | .070000 | .073000 | .070000 |
| Width (in) | .25000 | .25000 | .25000 | .25000 | .25000 | .25000 |
| Gauge length (in) | 1.3000 | 1.3000 | 1.3000 | 1.3000 | 1.3000 | 1.3000 |
| Specimen G.L. (in) | 2.5000 | 2.5000 | 2.5000 | 2.5000 | 2.5000 | 2.5000 |

Out of 6 specimens, 1 excluded.

| Specimen Number | LOAD AT YIELD (lb) | STRESS AT YIELD (psi) | STRAIN AT YIELD (%) | TENSION AT YIELD (lbs/in) | LOAD AT BREAK (lb) | STRESS AT BREAK (psi) | STRAIN AT BREAK (%) | TENSION AT BREAK (lbs/in) |
|---|---|---|---|---|---|---|---|---|
| 1 | 41.30 | 2526. | 15.96 | 165.2 | 88.20 | 5394. | 1545. | 352.8 |
| 2 | 49.00 | 2692. | 16.63 | 196.0 | 95.85 | 5266. | 1513. | 383.4 |
| 3 | 48.55 | 2572. | 16.00 | 194.2 | 97.45 | 5163. | 1492. | 389.8 |
| 4 | 44.95 | 2569. | 16.63 | 179.8 | 92.70 | 5297. | 1499. | 370.8 |
| *Excluded* | 48.40 | 2652. | 16.67 | 193.6 | 90.80 | 4975. | 1420. | 363.2 |
| 6 | 44.25 | 2529. | 16.63 | 177.0 | 88.40 | 5051. | 1489. | 353.6 |
| Mean: | 45.61 | 2578. | 16.37 | 182.4 | 92.52 | 5234. | 1508. | 370.1 |
| Standard Deviation: | 3.20 | 68. | .36 | 12.8 | 4.22 | 132. | 23. | 16.9 |

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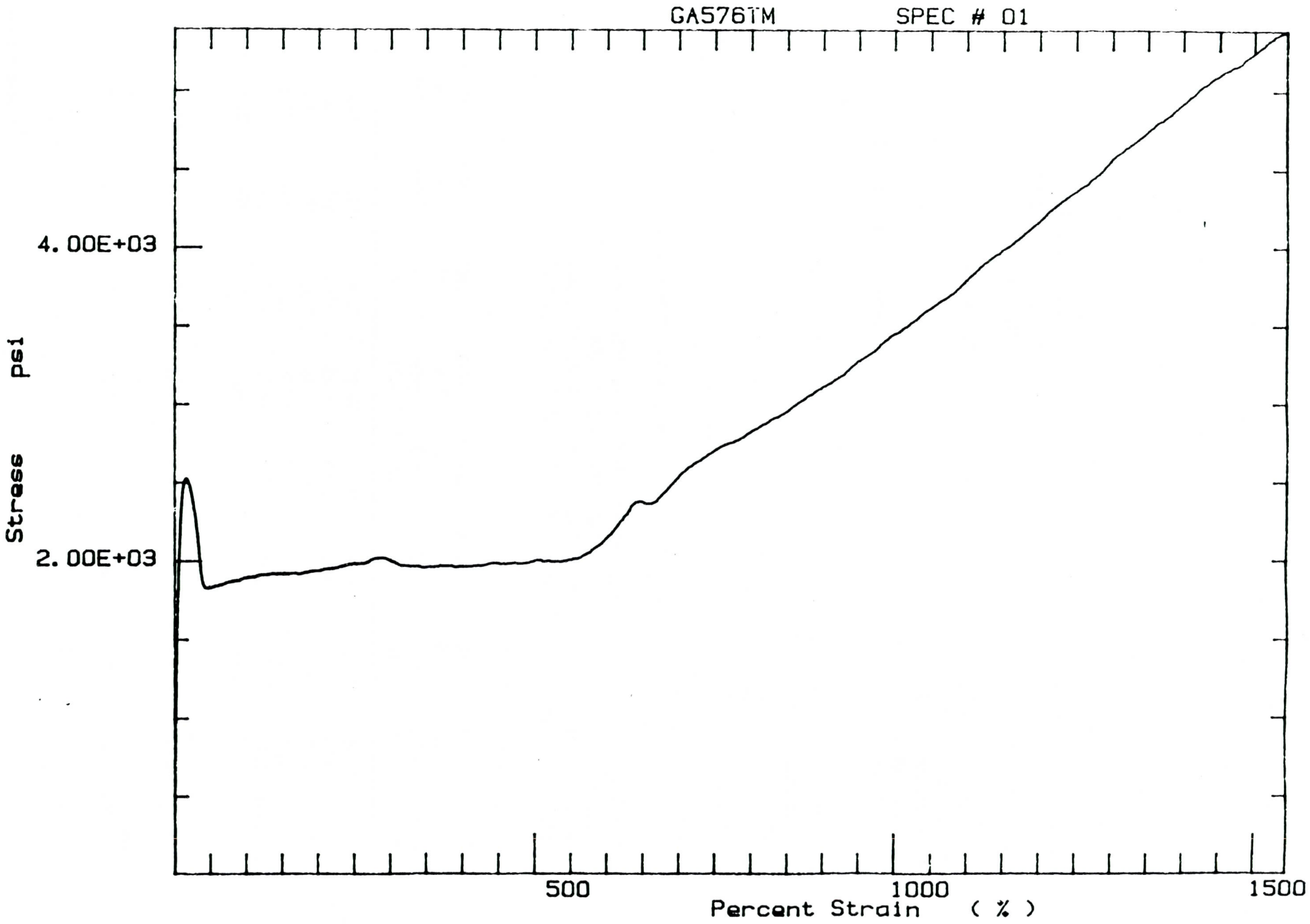
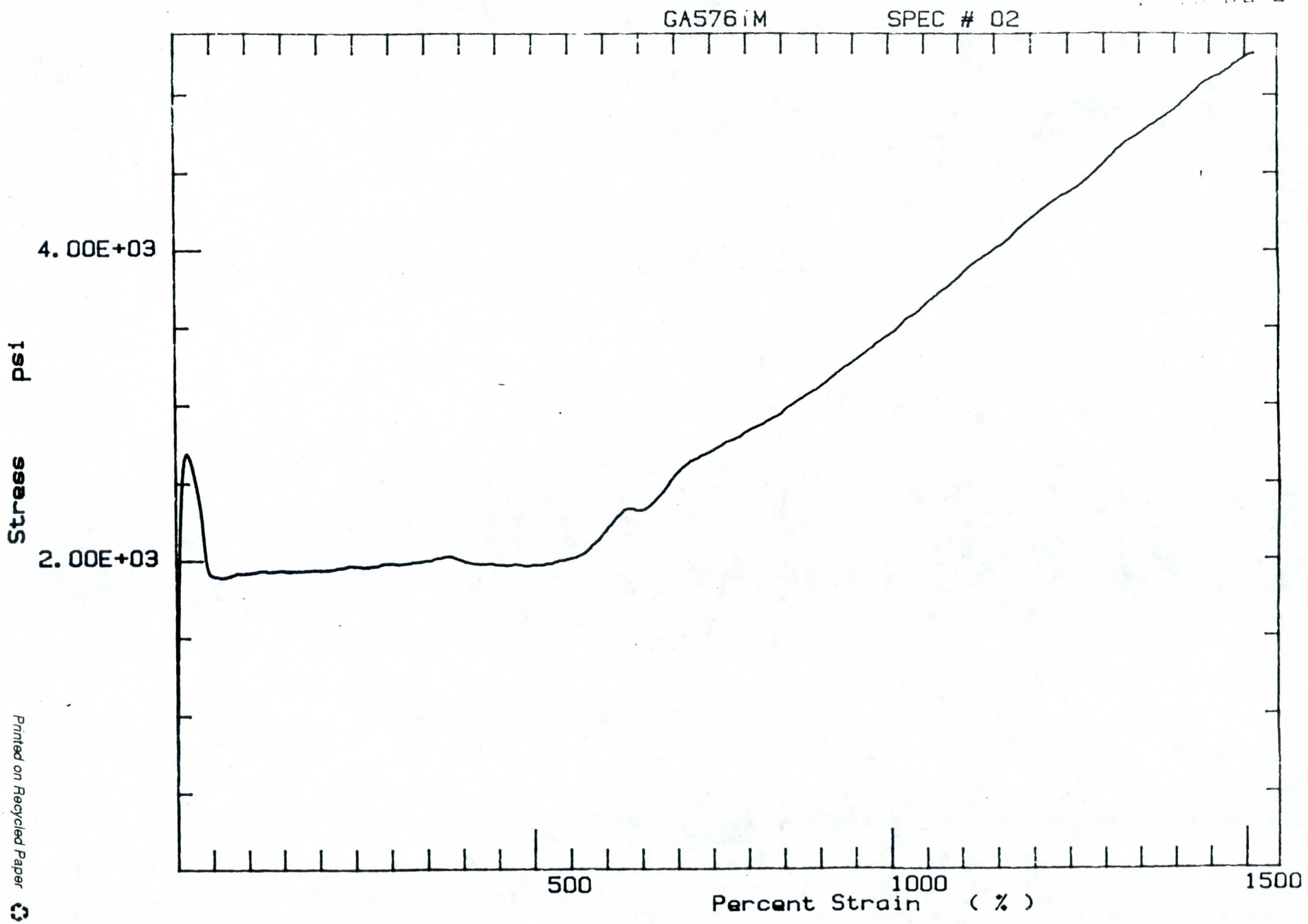

GA576TM
SPEC # 01
4.00E+03
2.00E+03
Stress psi
500
1000
1500
Percent Strain ( % )

DRAFT

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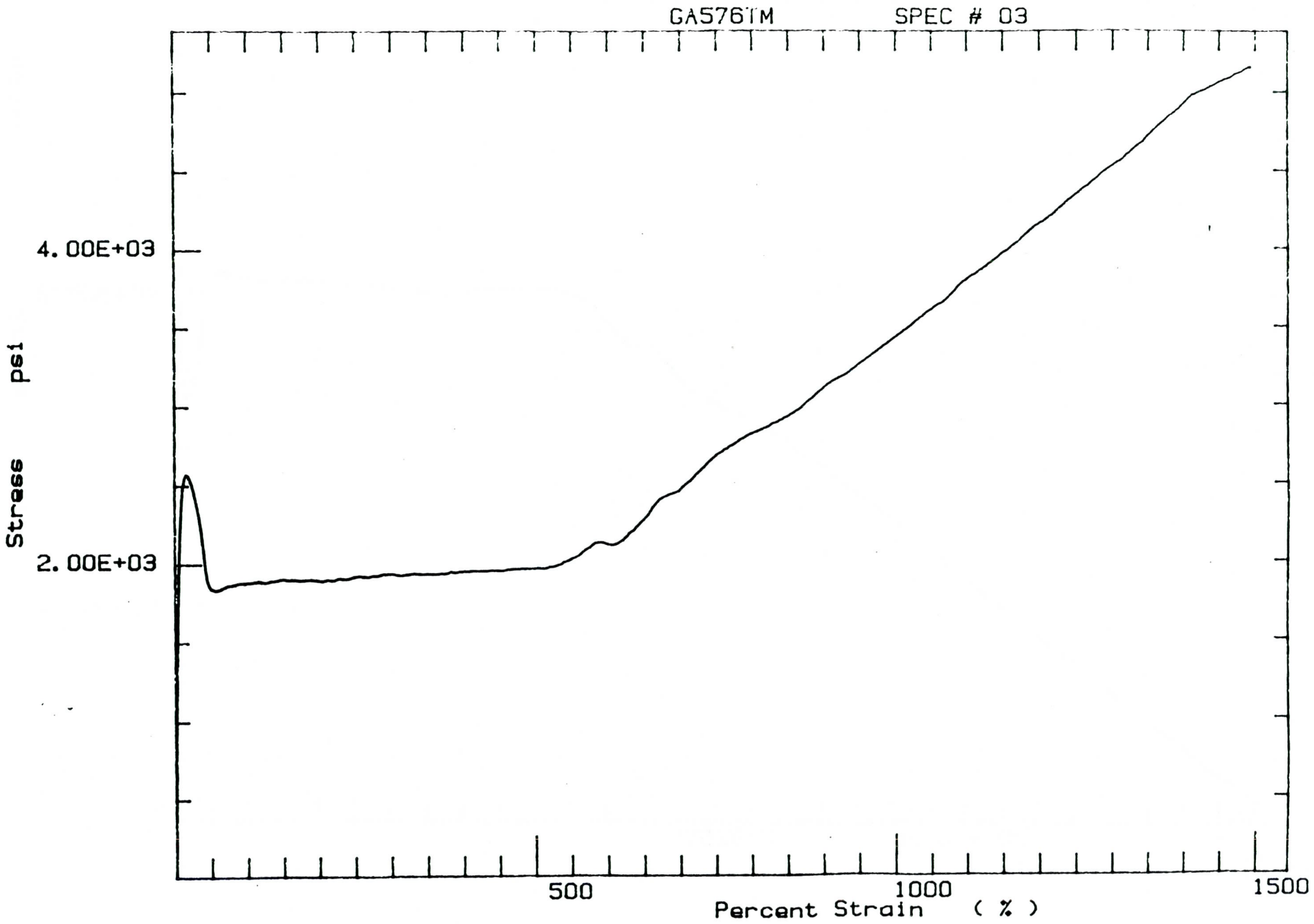
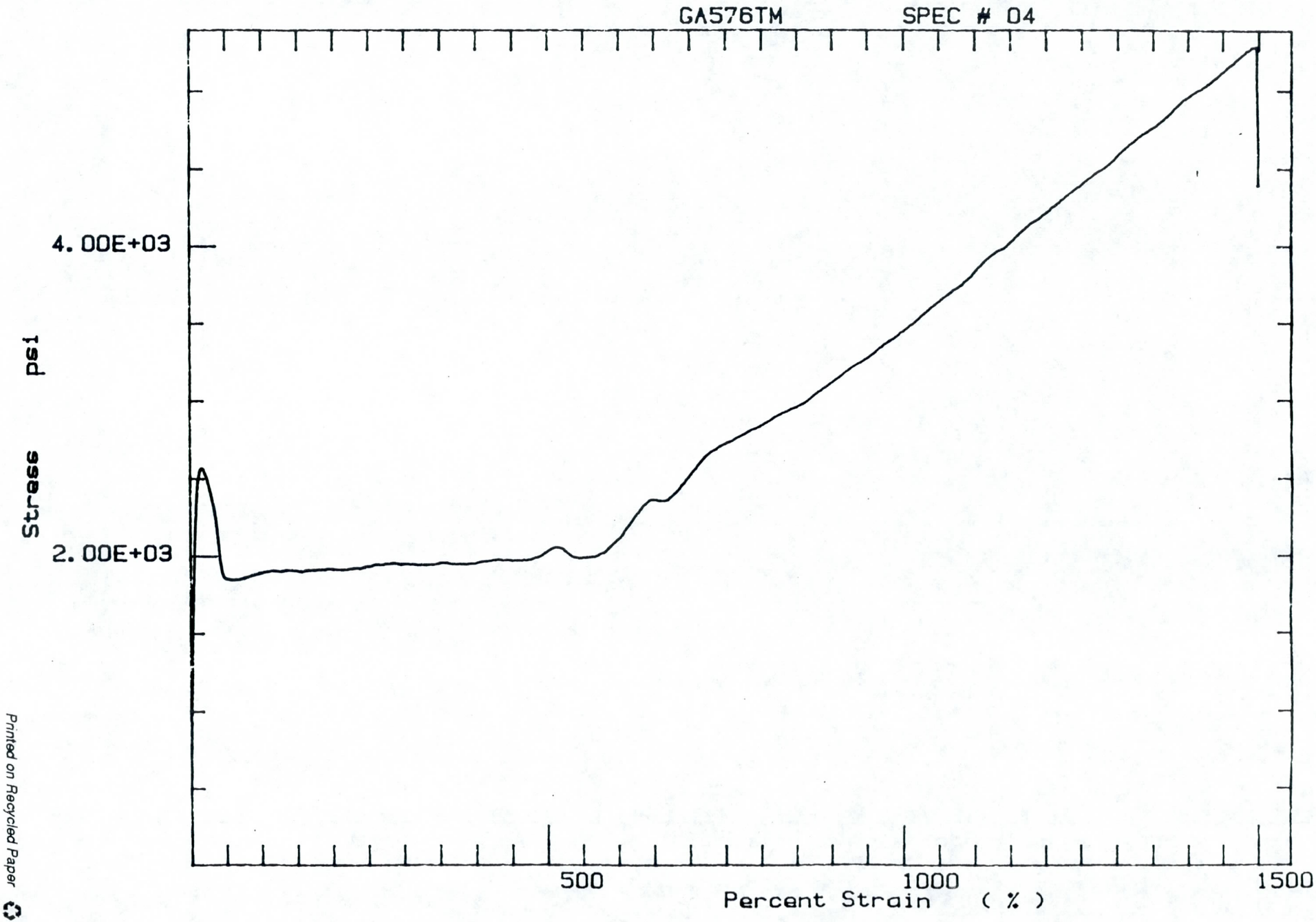

GA576TM
SPEC # 03
4.00E+03
2.00E+03
Stress psi
500
1000
1500
Percent Strain ( % )

DRAFT

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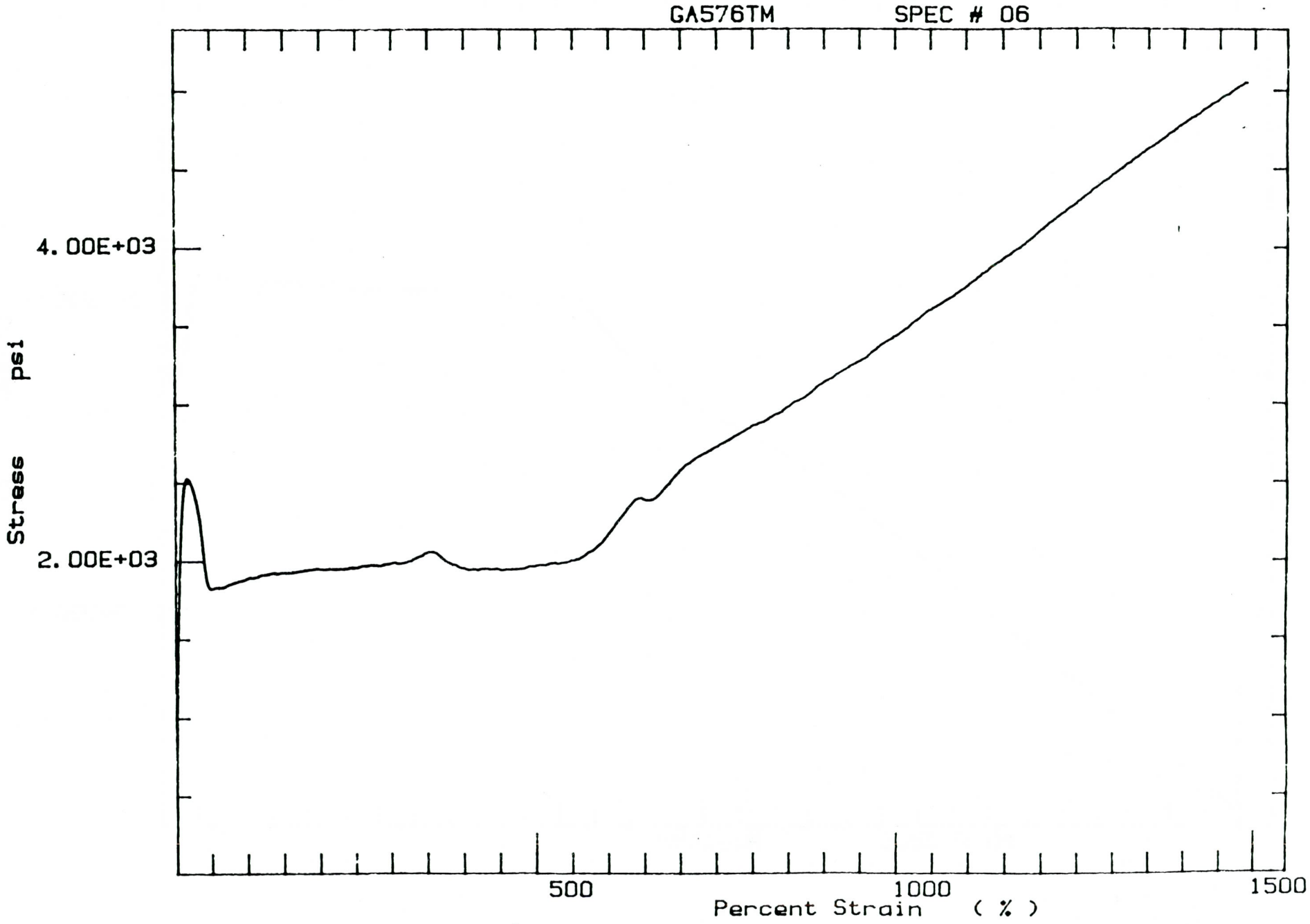
GA576TM
SPEC # 06
4.00E+03
2.00E+03
Stress psi
500
1000
1500
Percent Strain ( % )

GEOSYNTEC,INC.

TENSILE TEST 1

Test type: Tensile

Operator name: BLP

Sample Identification: 3A576TX
Interface Type: 1011 Series
Machine Parameters at test:
Sample Rate (pts/sec): 4.00
Crosshead Speed (in/min): 2.000

Instron Corporation
Series IX Automated Materials Testing System v4.05a
Test Date: May 24,1990

Sample Type: 00IV

Humidity ( % ): 50
Temperature (deg. C): 23

SAMPLE ID#:(16 CHAR) G118-015GM00003
LOWER CASE or OR no: nd
CLIENT: GSV/ATLANTA
JOB#: G118-015
CUSTOMER SAMPLE: GLT-60 mil

Dimensions:

| | Spec. 1 | Spec. 2 | Spec. 3 | Spec. 4 | Spec. 5 |
|---|---|---|---|---|---|
| Thickness (in) | .063600 | .064100 | .064500 | .065200 | .063000 |
| Width (in) | .25000 | .25000 | .25000 | .25000 | .25000 |
| Gauge length (in) | 1.3000 | 1.3000 | 1.3000 | 1.3000 | 1.3000 |
| Specimen G.L. (in) | 2.5000 | 2.5000 | 2.5000 | 2.5000 | 2.5000 |

Out of 5 specimens, 0 excluded.

| Specimen Number | LOAD AT YIELD (lb) | STRESS AT YIELD (psi) | STRAIN AT YIELD (%) | TENSION AT YIELD (lbs/in) | LOAD AT BREAK (lb) | STRESS AT BREAK (psi) | STRAIN AT BREAK (%) | TENSION AT BREAK (lbs/in) |
|---|---|---|---|---|---|---|---|---|
| 1 | 41.50 | 2610. | 16.00 | 166.0 | 93.45 | 5248. | 1482. | 333.8 |
| 2 | 43.75 | 2730. | 16.12 | 175.0 | 88.75 | 5538. | 1553. | 355.0 |
| 3 | 43.30 | 2685. | 16.11 | 173.2 | 77.40 | 4800. | 1409. | 309.6 |
| 4 | 43.75 | 2684. | 17.16 | 175.0 | 82.00 | 5031. | 1435. | 328.0 |
| 5 | 42.75 | 2714. | 16.02 | 171.0 | 81.00 | 5143. | 1422. | 324.0 |
| Mean: | 43.01 | 2685. | 16.28 | 172.0 | 82.52 | 5152. | 1460. | 330.1 |
| Standard Deviation: | .94 | 46. | .49 | 3.8 | 4.14 | 273. | 59. | 16.5 |

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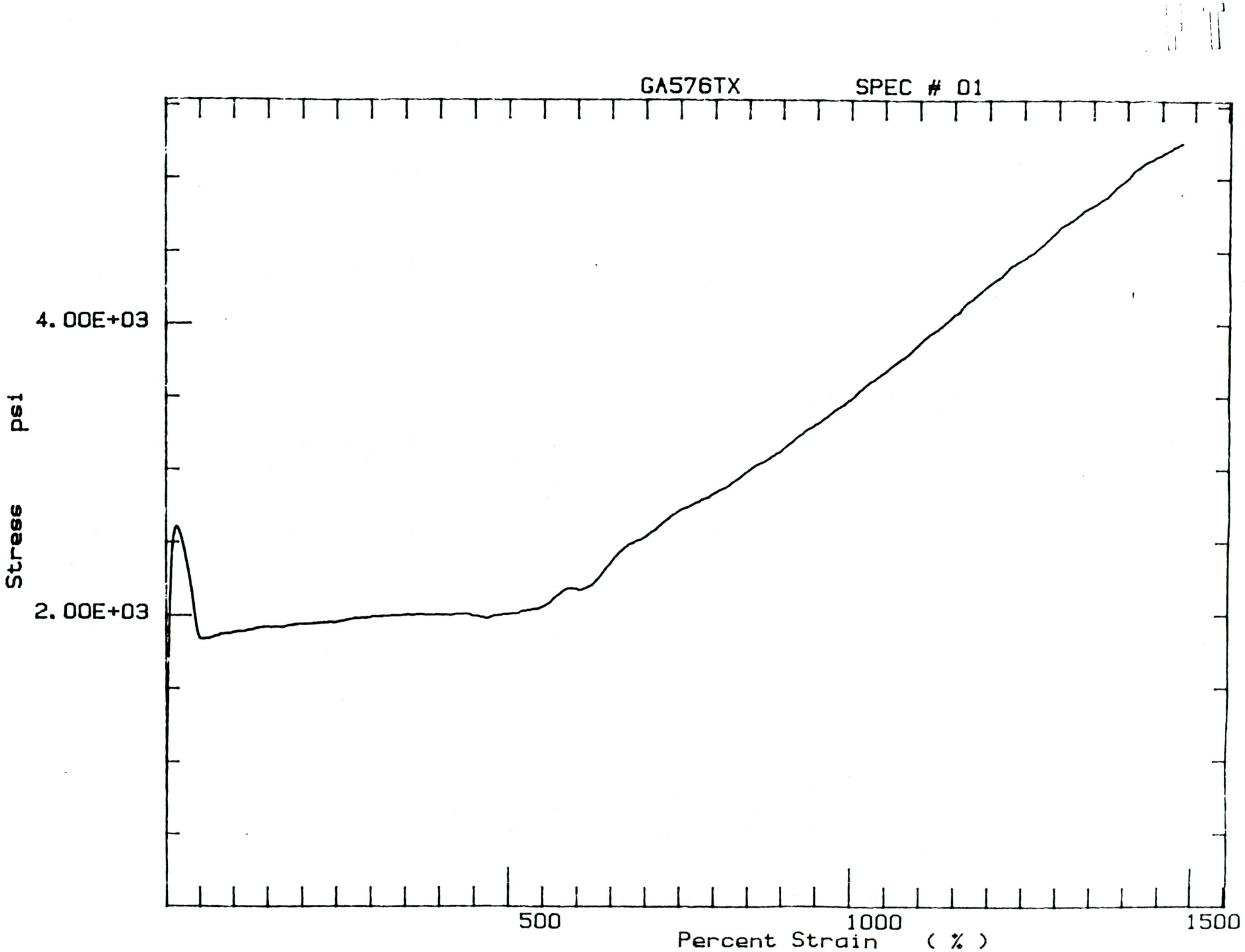
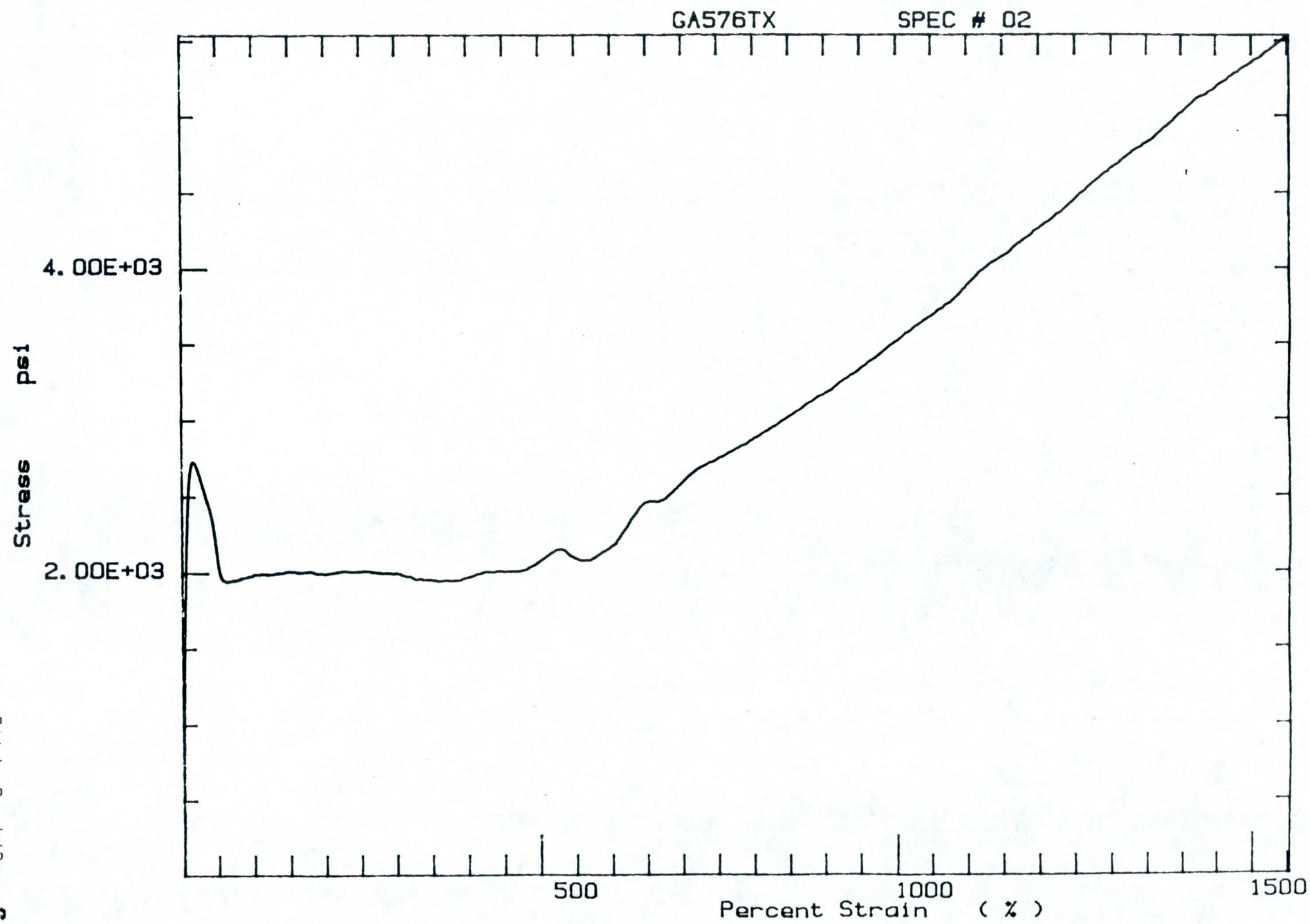

GA576TX
SPEC # 01
4.00E+03
2.00E+03
Stress psi
500
1000
1500
Percent Strain ( % )

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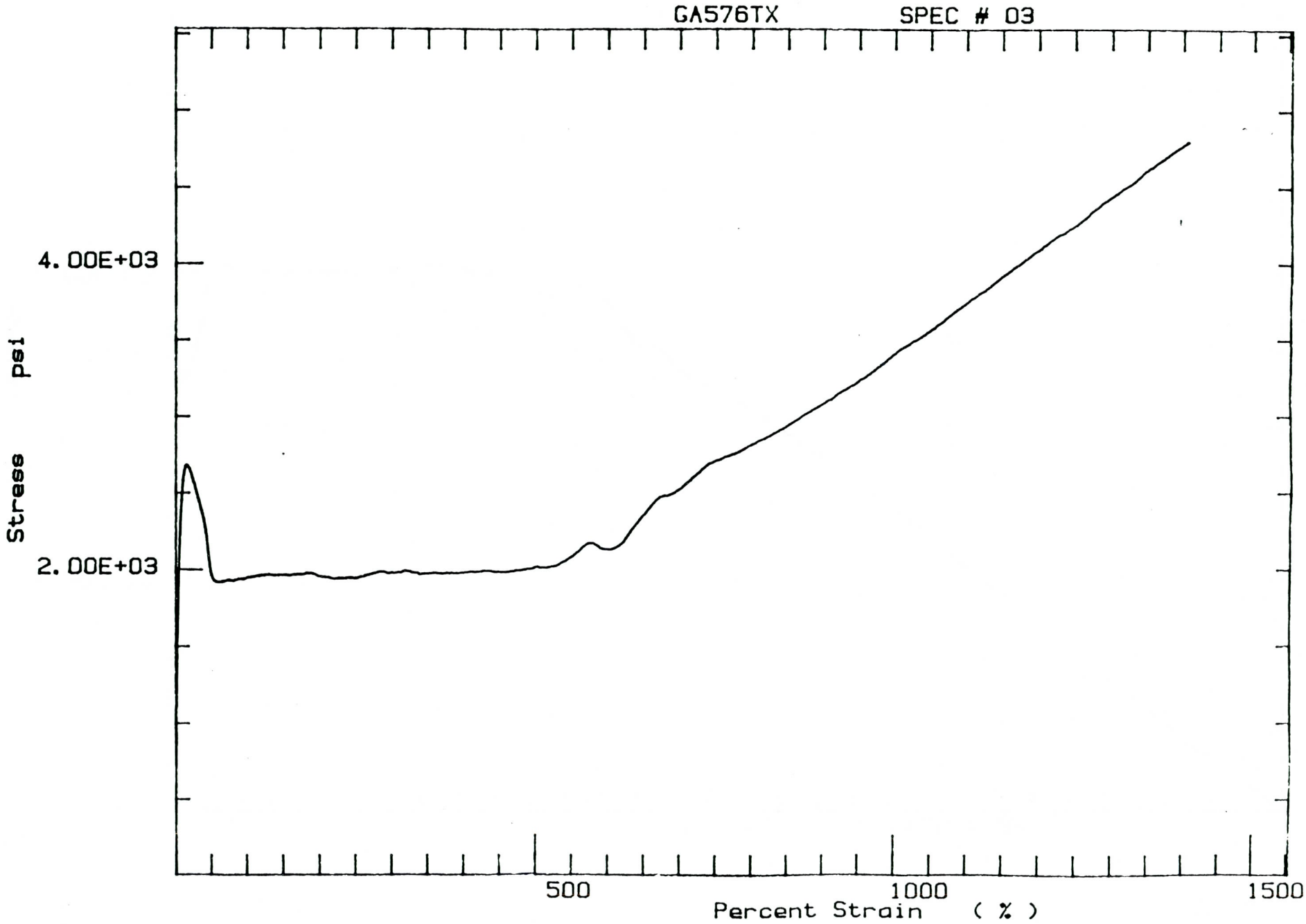
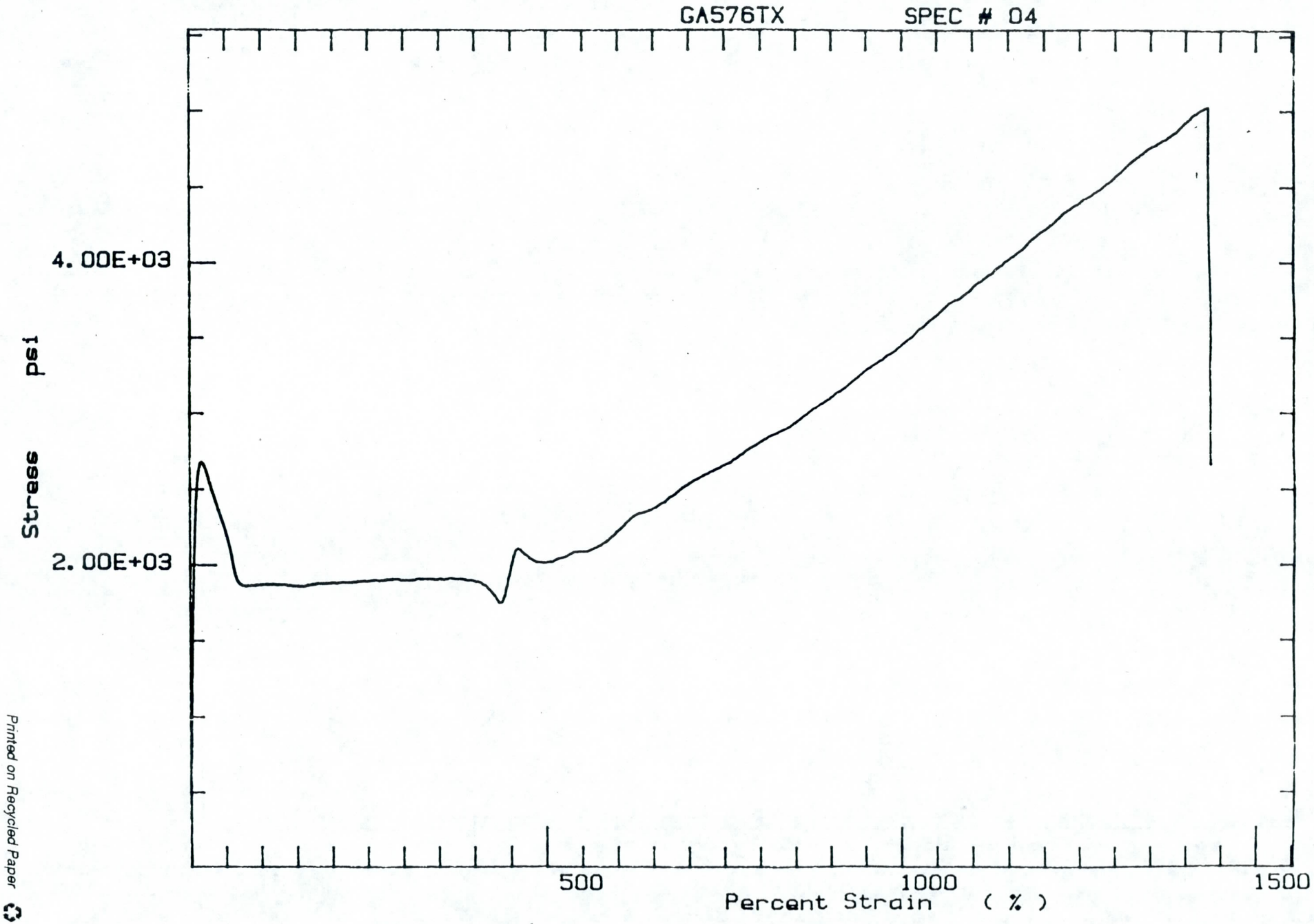

GA576TX
SPEC # 03
4.00E+03
2.00E+03
Stress psi
500
1000
1500
Percent Strain ( % )

DRAFT

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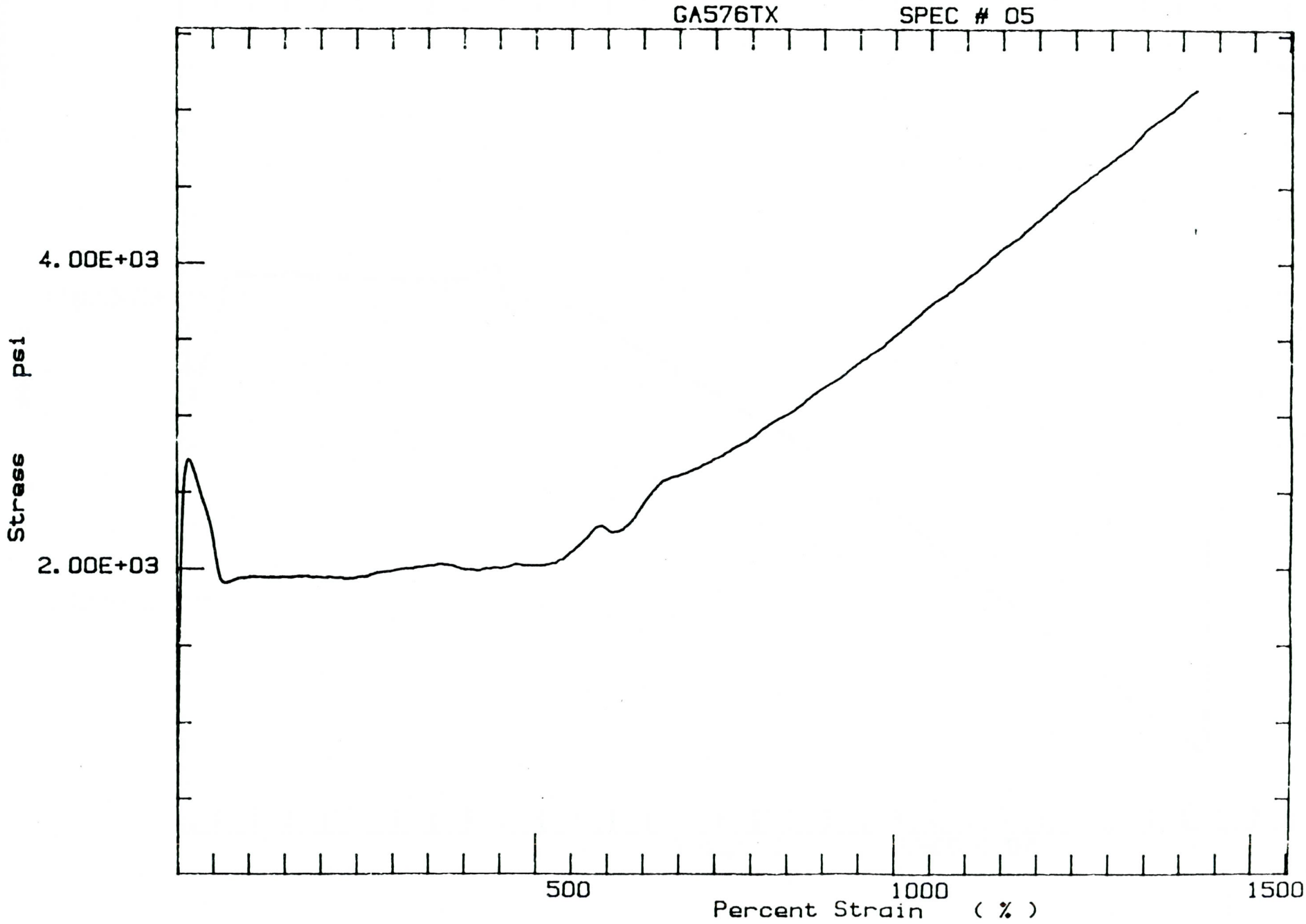
GA576TX
SPEC # 05
4.00E+03
2.00E+03
Stress psi
500
1000
1500
Percent Strain ( % )

GEOSYNTEC, INC.

TENSILE TEST 2

Test type: Tensile

Instron Corporation
Series IX Automated Materials Testing System v4.05a
Test Date: May 25,1990

Operator name: BLP

Sample Identification: GH577TM
Sample Type: WOIV
Interface Type: 1011 Series
Machine Parameters of test:
Sample Rate (pts/sec): 4.00
Crosshead Speed (in/min): 2.000

Humidity ( % ): 50
Temperature (deg. C): 23

SAMPLE ID#:(16 CHAR) G118-015GM00004
LOWER CASE md OR xd: md
CLIENT: GSV/ATLANTA
JOB#: G118-015
CUSTOMER SAMPLE: GUNDLE-60 mil

Dimensions:

| | Spec. 1 | Spec. 2 | Spec. 3 | Spec. 4 | Spec. 5 | Spec. 6 |
|---|---|---|---|---|---|---|
| Thickness (in) | .059100 | .056800 | .058000 | .059000 | .059100 | .058300 |
| Width (in) | .25000 | .25000 | .25000 | .25000 | .25000 | .25000 |
| Gauge length (in) | 1.3000 | 1.3000 | 1.3000 | 1.3000 | 1.3000 | 1.3000 |
| Specimen G.L. (in) | 2.5000 | 2.5000 | 2.5000 | 2.5000 | 2.5000 | 2.5000 |

Out of 5 specimens, 1 excluded.

| Specimen Number | LOAD AT YIELD (lb) | STRESS AT YIELD (psi) | STRAIN AT YIELD (%) | TENSION AT YIELD (lbs/in) | LOAD AT BREAK (lb) | STRESS AT BREAK (psi) | STRAIN AT BREAK (%) | TENSION AT BREAK (lbs/in) |
|---|---|---|---|---|---|---|---|---|
| 1 | 38.60 | 2613. | 16.03 | 154.4 | 66.50 | 4501. | 1241. | 266.0 |
| 2 | 35.30 | 2486. | 16.50 | 141.2 | 71.30 | 5021. | 1333. | 285.2 |
| *Excluded* | 37.15 | 2562. | 17.95 | 148.6 | 65.05 | 4486. | 1242. | 260.2 |
| 4 | 36.95 | 2505. | 15.36 | 147.8 | 75.40 | 5112. | 1368. | 301.6 |
| 5 | 37.65 | 2548. | 16.49 | 150.6 | 68.05 | 4606. | 1275. | 272.2 |
| 6 | 36.60 | 2511. | 16.63 | 146.4 | 70.35 | 4827. | 1291. | 281.4 |
| Mean: | 37.02 | 2533. | 16.20 | 148.1 | 70.32 | 4813. | 1302. | 281.3 |
| Standard Deviation: | 1.23 | 50. | .52 | 4.9 | 3.41 | 261. | 50. | 13.6 |

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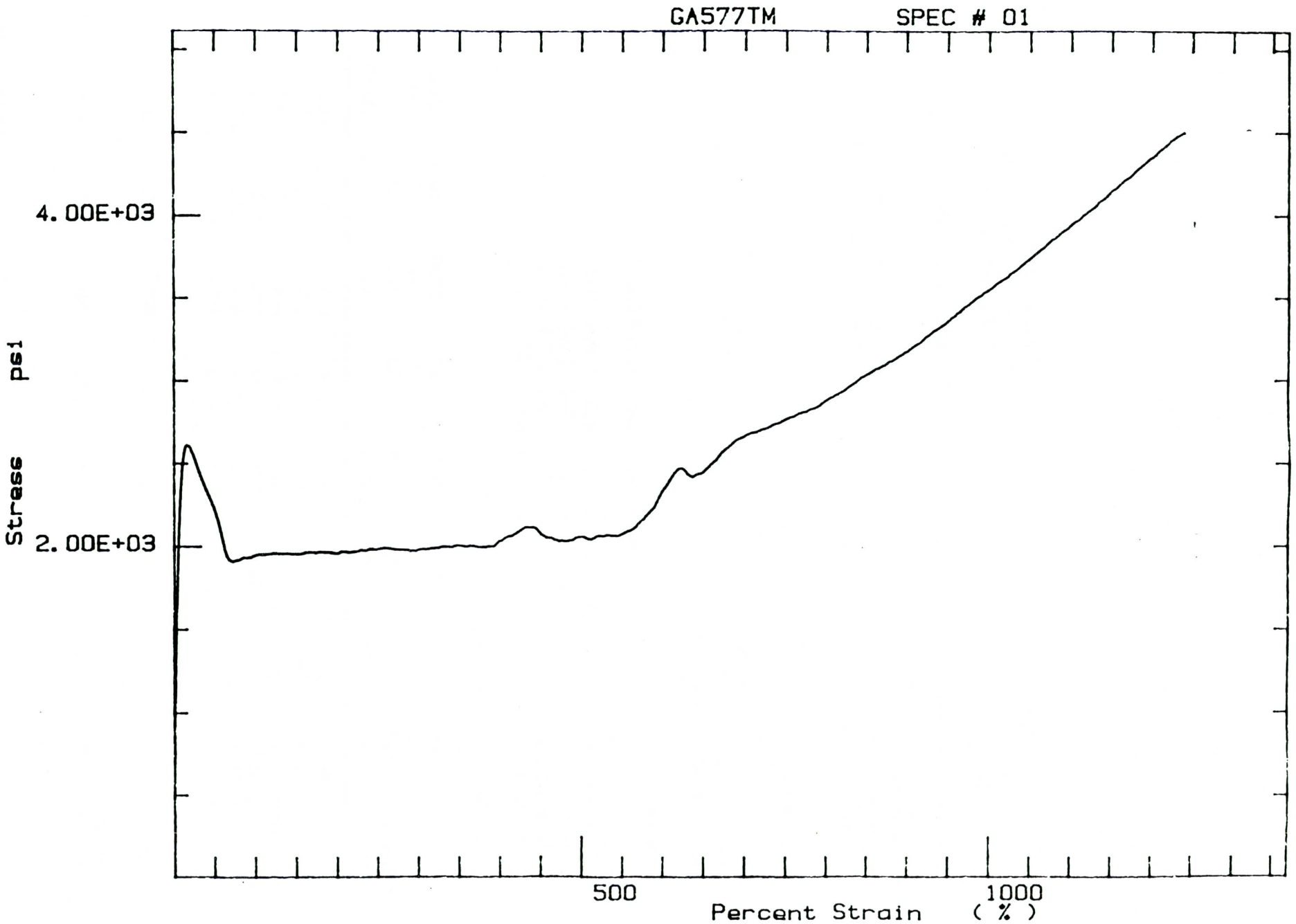
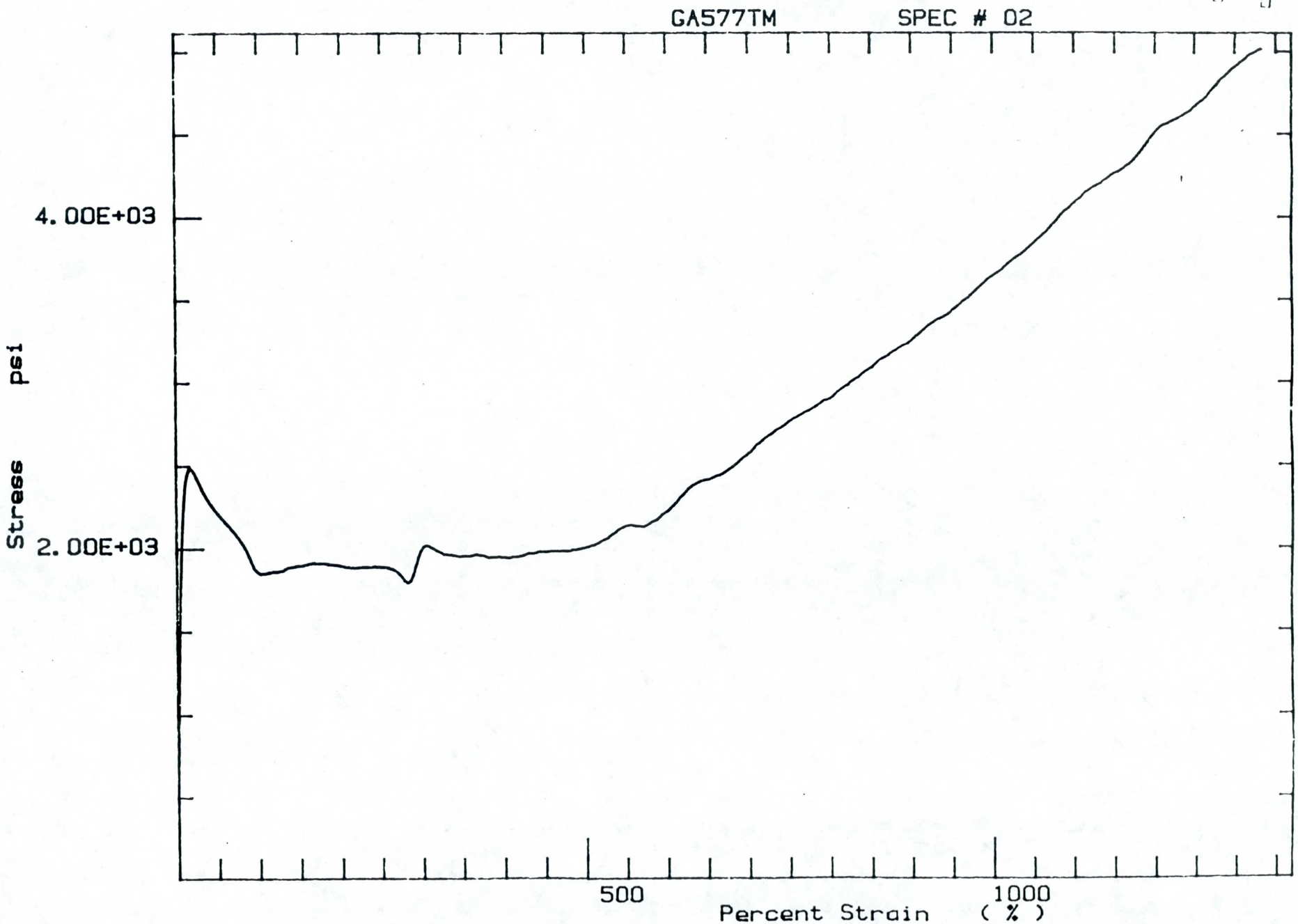

GA577TM
SPEC # 01
4.00E+03
2.00E+03
Stress psi
500
1000
Percent Strain ( % )

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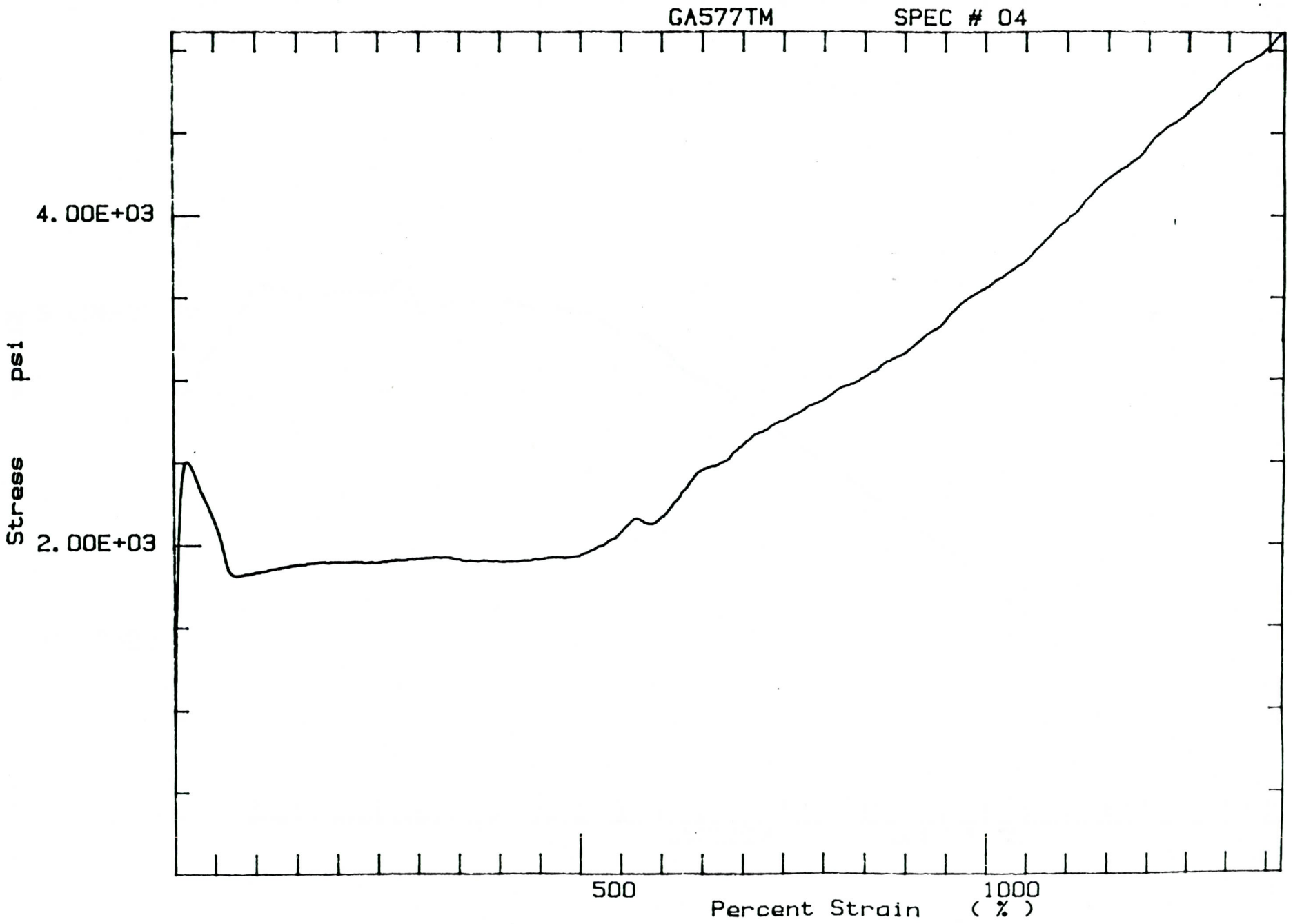
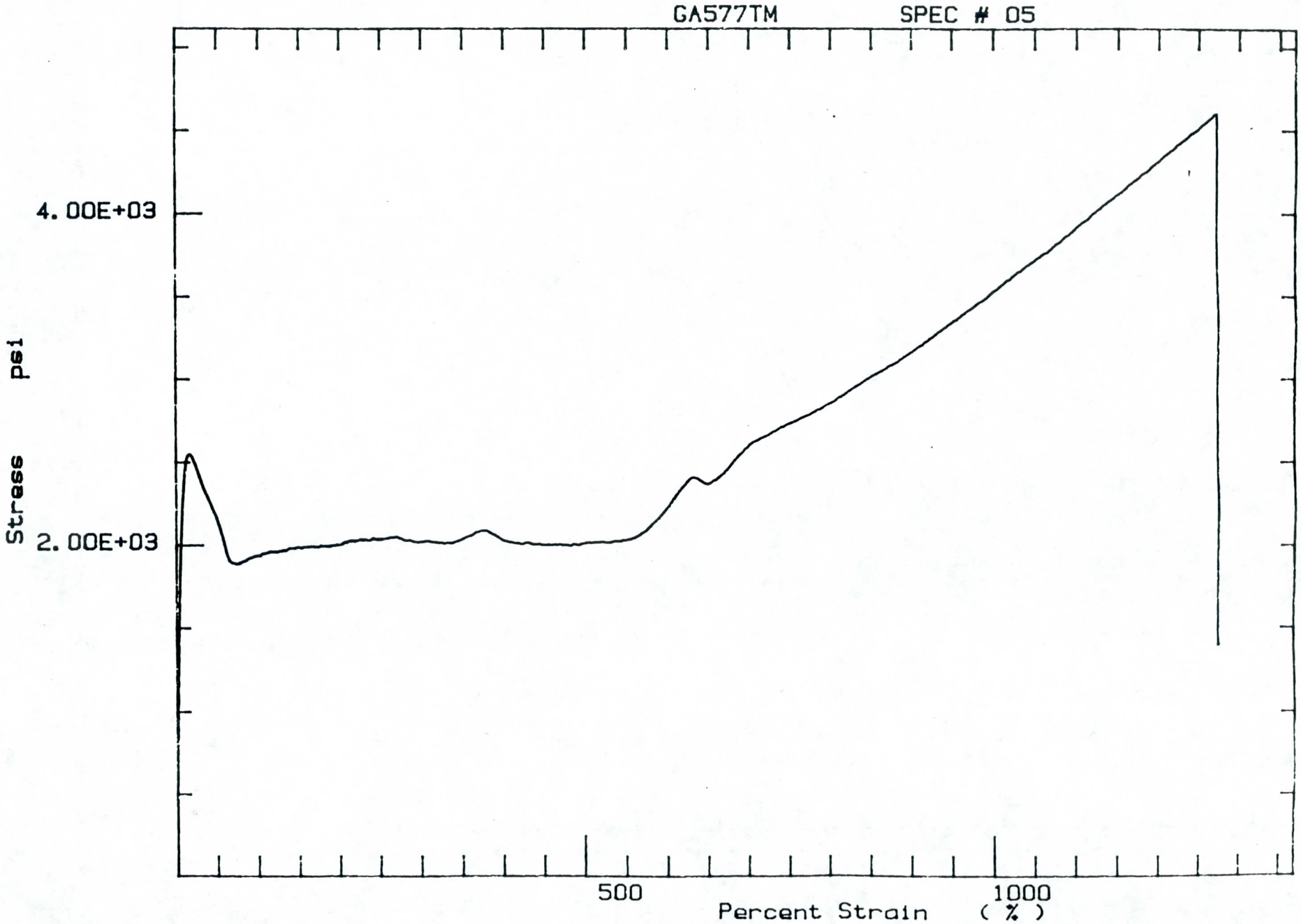

GA577TM
SPEC # 04
4.00E+03
2.00E+03
Stress psi
500
1000
Percent Strain ( % )

DRAFT

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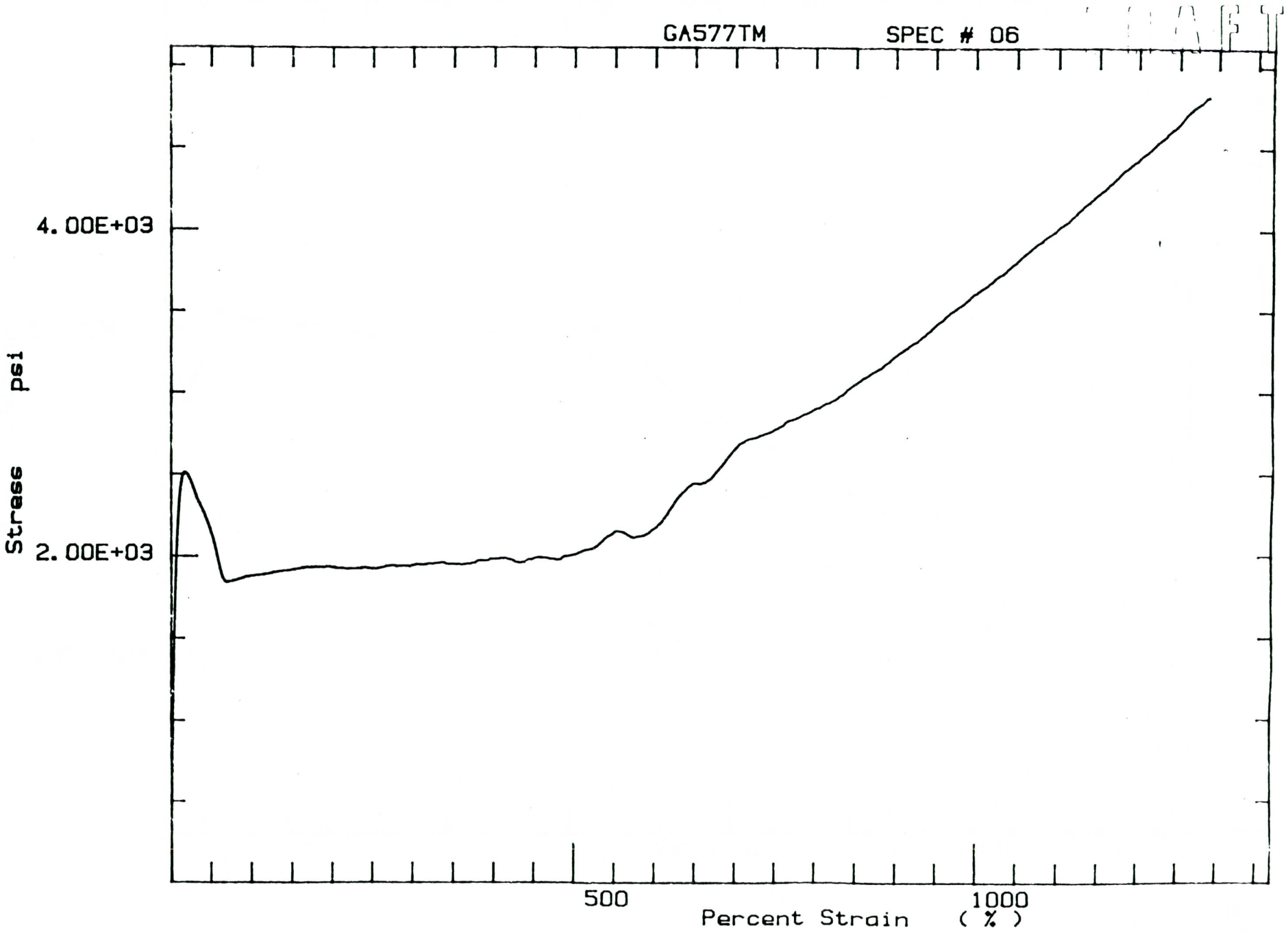
GA577TM
SPEC # 06
4.00E+03
2.00E+03
Stress psi
500
1000
Percent Strain ( % )

GEOSYNTEC,inc.

TENSILE TEST 2

Test type: Tensile

Operator name: SLP

Sample Identification: GA577TX
Interface Type: 1011 Series
Machine Parameters of test:
Sample Rate (pts/sec): 4.00
Crosshead Speed (in/min ): 2.000

Instron Corporation
Series IX Automated Materials Testing System v4.05a
Test Date: May 25,1990

Sample Type: 00IV

Humidity ( % ): 50
Temperature (deg. C): 23

SAMPLE ID#:(1s CHAR) G118-015GM00004
LOWER CASE ad OR xd: xd
CLIENT: GSV/ATLANTA
JOB#: G118-015
CUSTOMER SAMPLE: BUNDLE-60 mil

Dimensions:

| | Spec. 1 | Spec. 2 | Spec. 3 | Spec. 4 | Spec. 5 | Spec. 6 |
|---|---|---|---|---|---|---|
| Thickness (in) | .057100 | .057400 | .057000 | .056900 | .057200 | .057600 |
| Width (in) | .25000 | .25000 | .25000 | .25000 | .25000 | .25000 |
| Gauge length (in) | 1.3000 | 1.3000 | 1.3000 | 1.3000 | 1.3000 | 1.3000 |
| Specimen G.L. (in) | 2.5000 | 2.5000 | 2.5000 | 2.5000 | 2.5000 | 2.5000 |

Out of 6 specimens, 1 excluded.

| Specimen Number | LOAD AT YIELD (lb) | STRESS AT YIELD (psi) | STRAIN AT YIELD (%) | TENSION AT YIELD (lbs/in) | LOAD AT BREAK (lb) | STRESS AT BREAK (psi) | STRAIN AT BREAK (%) | TENSION AT BREAK (lbs/in) |
|---|---|---|---|---|---|---|---|---|
| 1 | 39.35 | 2757. | 14.71 | 157.4 | 70.45 | 4935. | 1333. | 281.8 |
| 2 | 39.05 | 2721. | 14.19 | 156.2 | 66.85 | 4659. | 1264. | 267.4 |
| 3 | 39.70 | 2786. | 15.49 | 158.8 | 74.30 | 5214. | 1399. | 297.2 |
| 4 | 39.75 | 2794. | 15.35 | 159.0 | 75.45 | 5304. | 1422. | 301.8 |
| 5 | 39.50 | 2762. | 15.48 | 158.0 | 71.95 | 5031. | 1396. | 287.8 |
| *Excluded* | 37.95 | 2635. | 15.87 | 151.8 | 64.15 | 4455. | 1228. | 256.6 |
| Mean: | 39.47 | 2764. | 15.04 | 157.9 | 71.80 | 5029. | 1363. | 287.2 |
| Standard Deviation: | .28 | 29. | .57 | 1.1 | 3.39 | 253. | 64. | 13.5 |

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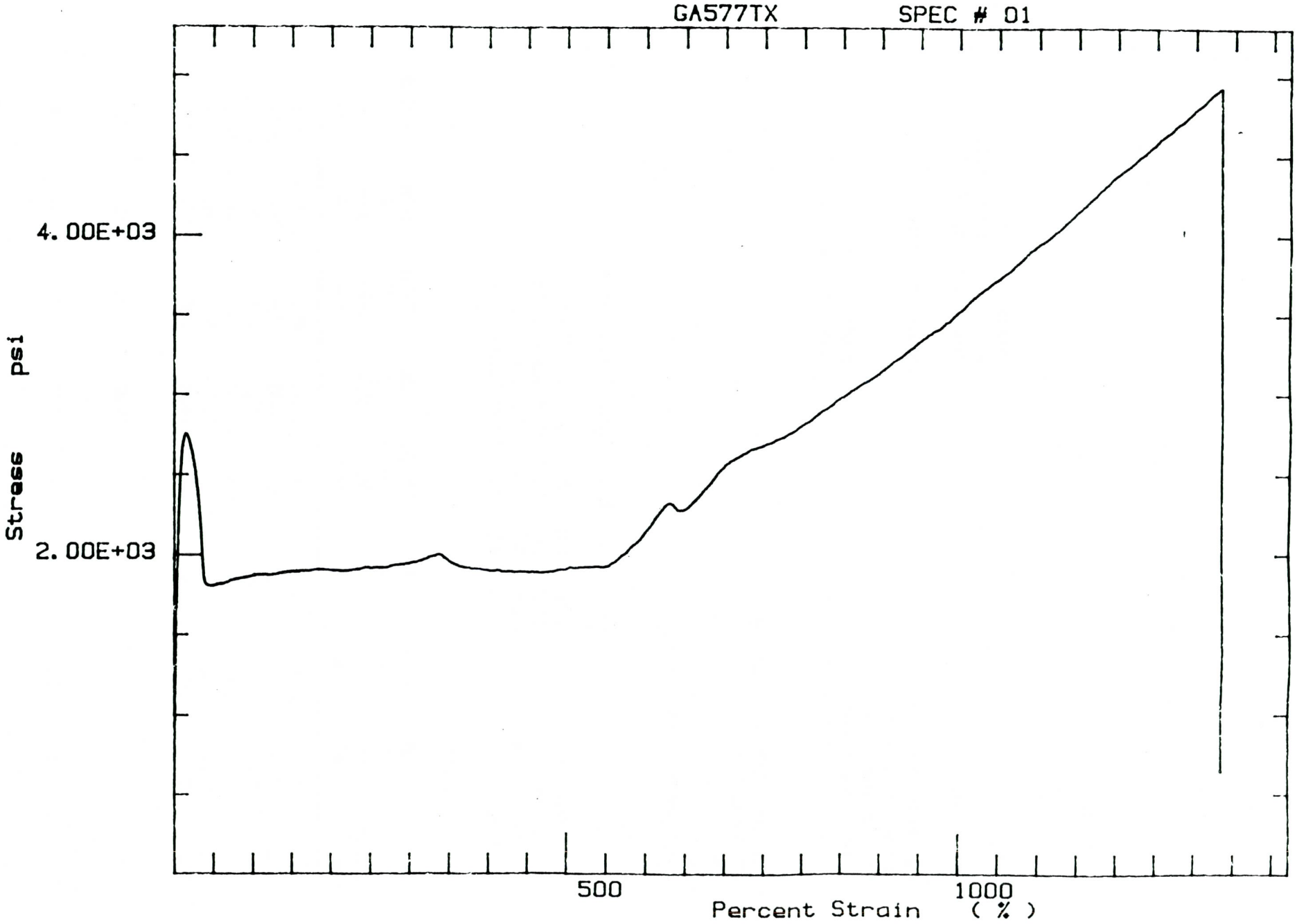
GA577TX
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2.00E+03
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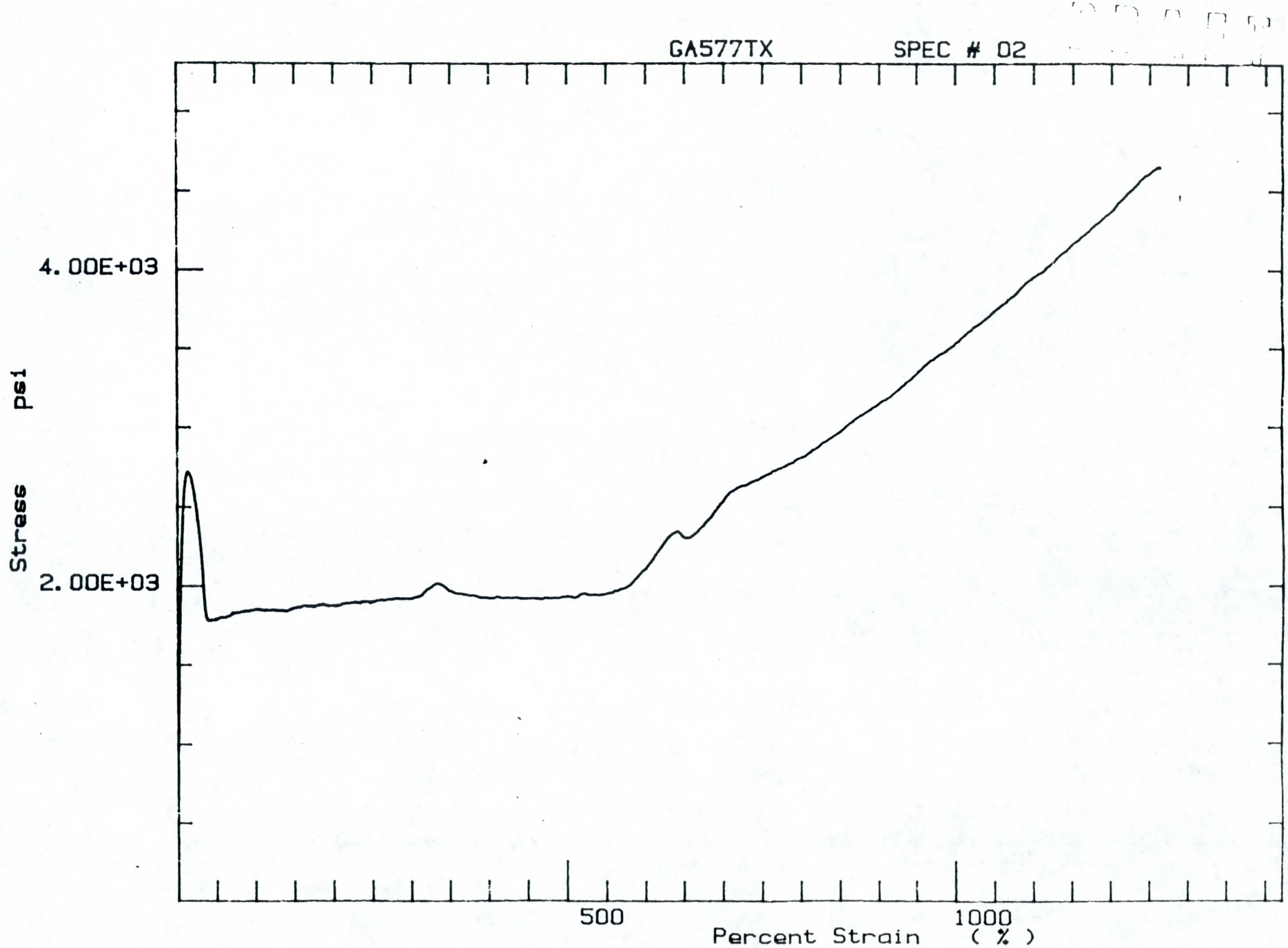
1000
Percent Strain ( % )

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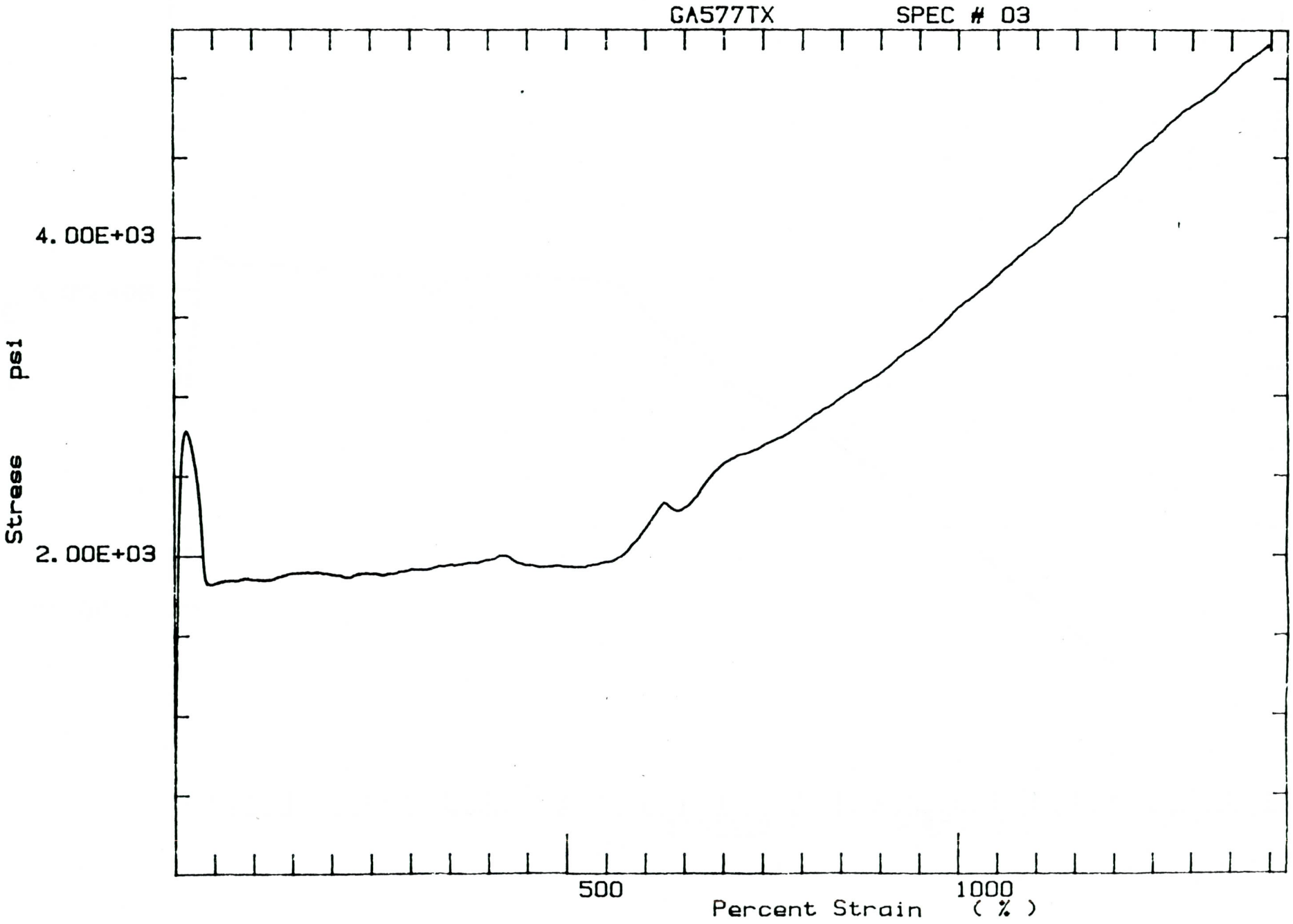
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Percent Strain ( % )

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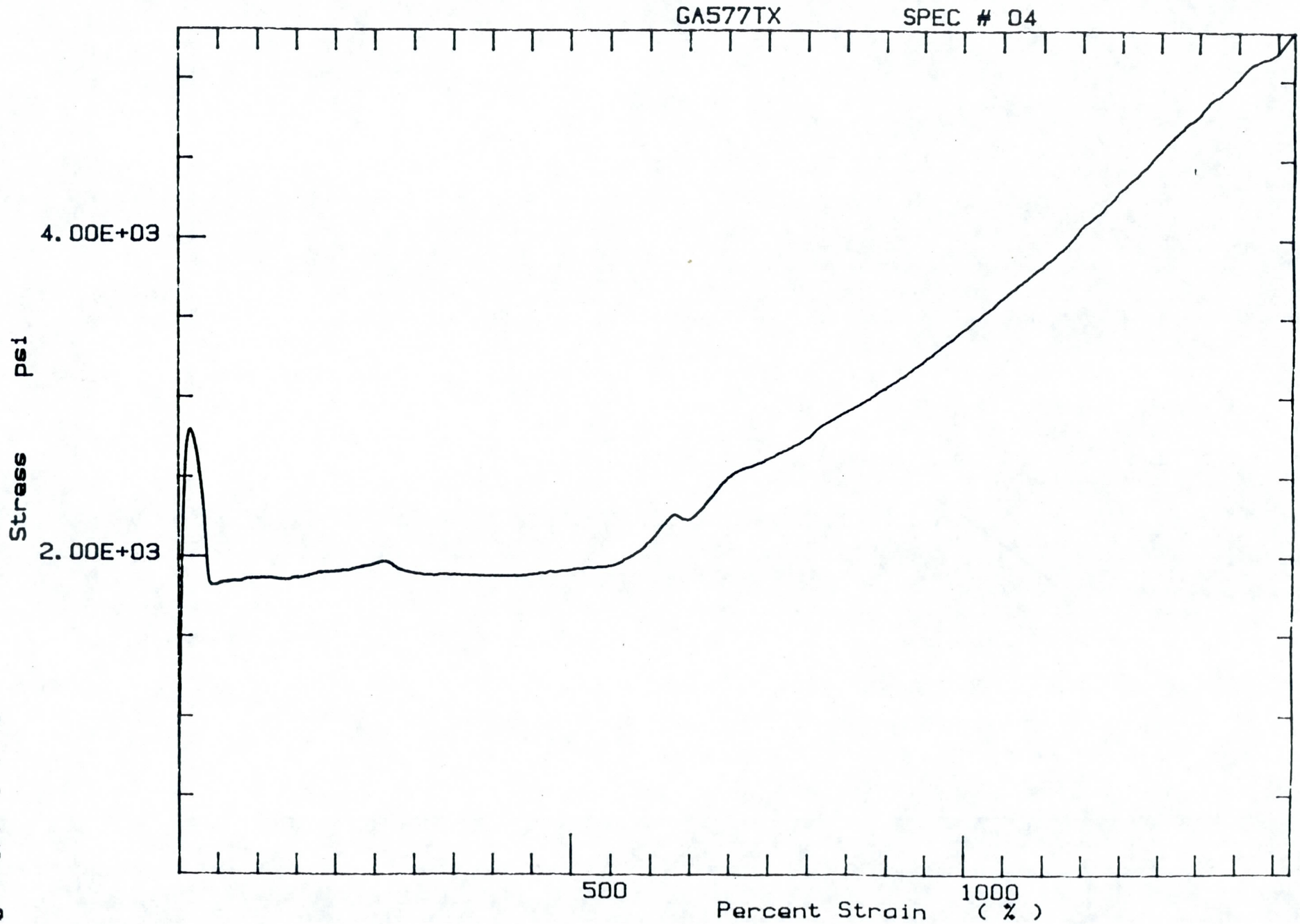


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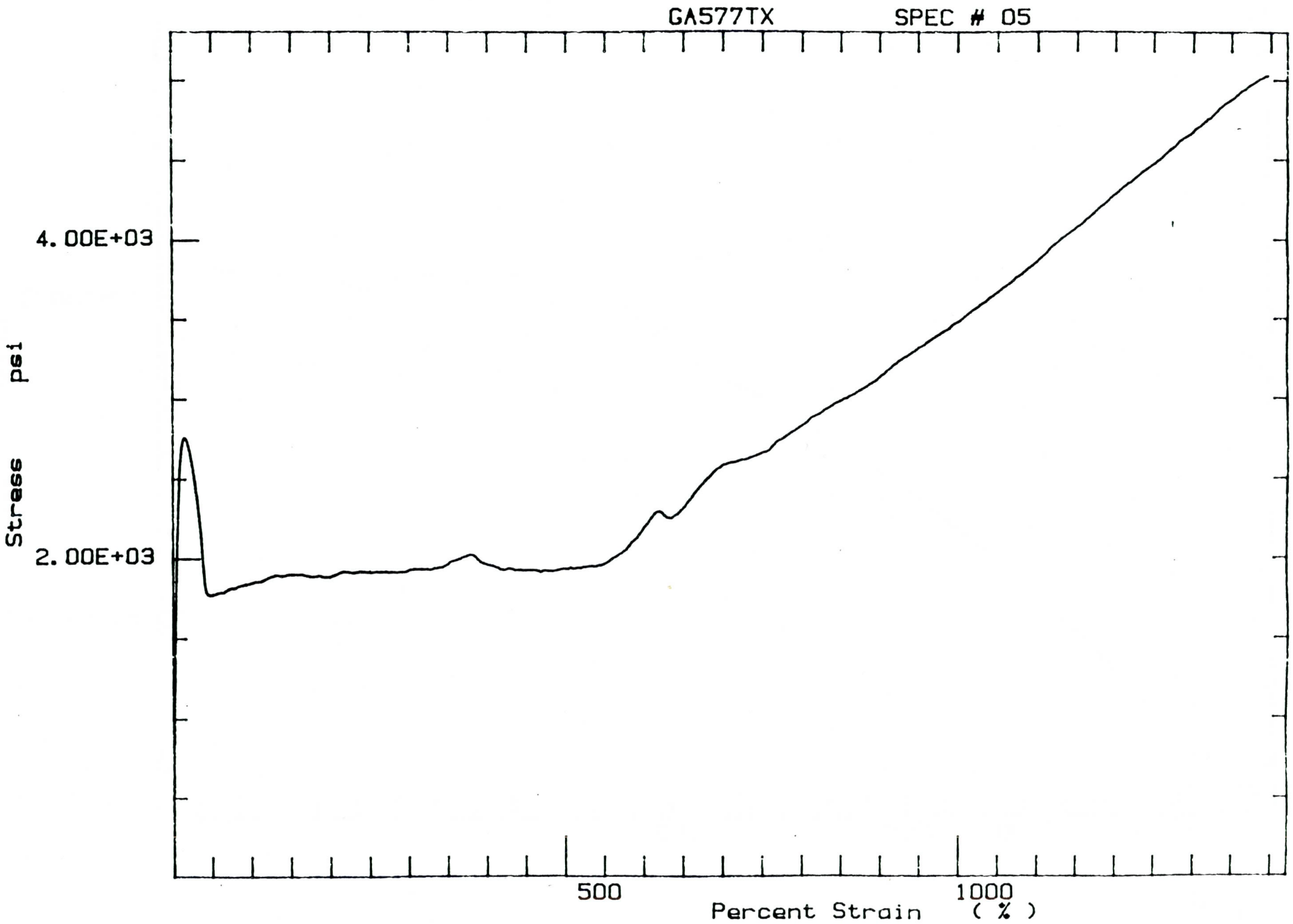
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2.00E+03
Stress psi
500
1000
Percent Strain ( % )

# APPENDIX B

# REFERENCE ON SOIL ARCHING-TENSION MEMBRANE ANALYSIS

*Geotextiles and Geomembranes* 9 (1990) 11–50

# Design of Soil Layer-Geosynthetic Systems Overlying Voids

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*ABSTRACT*

*This paper presents equations, tables, and charts to design soil layer-geosynthetic systems to span voids such as tension cracks, sinkholes, dissolution cavities, and depressions in foundation soils due to differential settlements or localized subsidence. These equations, tables, and charts were developed by combining tensioned membrane theory (for the geosynthetic) with arching theory (for the soil layer), thereby providing a more complete design approach than one that considers tensioned membrane theory only.*

*Design examples are presented to illustrate the solution of typical problems such as: selection of the required geosynthetic properties, determination of the maximum void size that can be bridged by a given system, and evaluation of the load-bearing capacity of a given system.*

## NOTATION

| | |
|---|---|
| $b$ | Width of the infinitely long void (m) |
| $c$ | Cohesion of the soil (N/m$^2$) |
| $D$ | Depth of the void (m) |
| $H$ | Thickness of the soil layer (m) |
| $K$ | Coefficient of lateral earth pressure (dimensionless) |
| $K_a$ | Coefficient of active earth pressure (dimensionless) |
| $p$ | Pressure on the geosynthetic (i.e. vertical stress at the bottom of the soil layer) over the void area (N/m$^2$) |



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$p_{lim}$ Limit value for the pressure on the geosynthetic, over the void area ($N/m^2$)
$p_b$ Pressure transmitted to the bottom of the void ($N/m^2$)
$p_0$ Pressure on the geosynthetic over the void area neglecting soil arching ($N/m^2$)
$q$ Uniformly distributed normal stress applied on top of the soil layer ($N/m^2$)
$r$ Radius of the circular void (m)
$r_{max}$ Maximum radius of a circular void which can be bridged by a given geosynthetic (m)
$s$ Soil shear strength ($N/m^2$)
$y$ Geosynthetic deflection (m)
$z$ Depth measured from the top of the soil layer (m)

$\alpha$ Geosynthetic tension (force per unit width) corresponding to the geosynthetic strain $\varepsilon$ (N/m)
$\alpha_{lim}$ Limit value for the required geosynthetic tension (N/m)
$\varepsilon$ Geosynthetic strain (dimensionless)
$\gamma$ Unit weight of soil ($N/m^3$)
$\Omega$ Factor related to $y$ and $\varepsilon$ (dimensionless)
$\phi$ Friction angle of the soil (degrees and dimensionless)
$\sigma_H$ Horizontal stress at depth $z$ ($N/m^2$)
$\sigma_V$ Vertical stress at depth $z$ ($N/m^2$)

# INTRODUCTION

## Description of the Problem

In many practical situations, a load is applied on a soil layer-geosynthetic system that will eventually overlie a void. (In this paper, the term 'void' is used generically for cracks, cavities, depressions, etc.) Two typical examples are a road embankment or a lining system for a reservoir constructed on a foundation where localized subsidence may develop.

The design engineer has to verify that, should subsidence develop, the geosynthetic layer can support the loads applied by the overlying soil and any other source (such as traffic on the road or the liquid in the reservoir) without failing or undergoing excessive deflection. The soil-geosynthetic system deflects over the void, and, from a design standpoint, three possibilities must be considered:

- The geosynthetic fails (Fig. 1(a)).

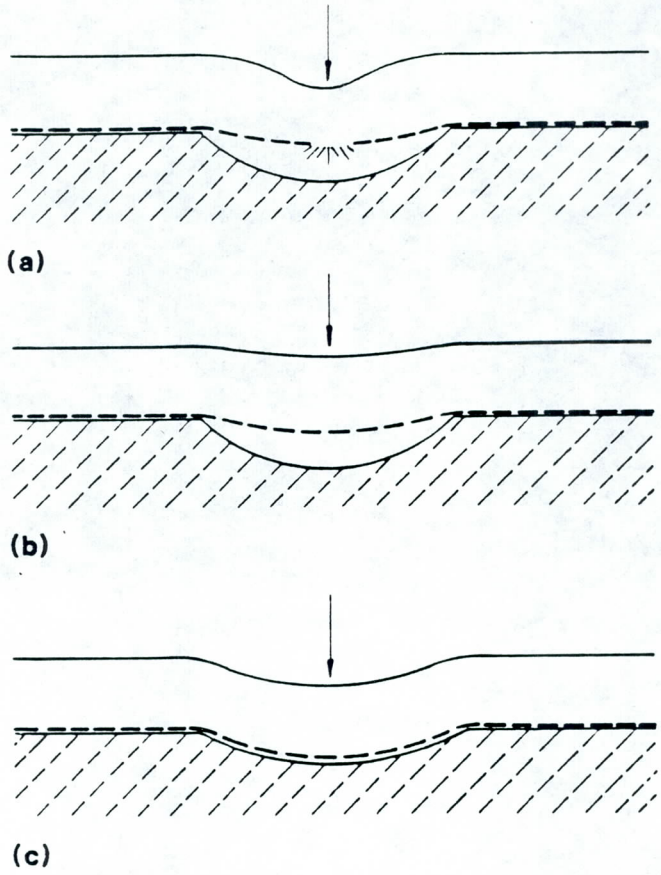


Fig. 1. Three design situations: (a) the geosynthetic fails; (b) the geosynthetic undergoes limited deflection and bridges the void; and (c) the geosynthetic deflects until it comes in contact with the bottom of the void.

- The geosynthetic undergoes limited deflection and bridges the void (Fig. 1(b)).
- The geosynthetic deflects until it comes in contact with the bottom of the void (Fig. 1(c)).

## The Nature of Voids

Examples of voids that can develop under a geosynthetic are discussed below:

*Tension Cracks*

Such cracks can occur in non-saturated cohesive soils subjected to tensile stresses and/or differential movements caused by settlement or other

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**Fig. 2.** Large tension crack formed under a geomembrane liner.

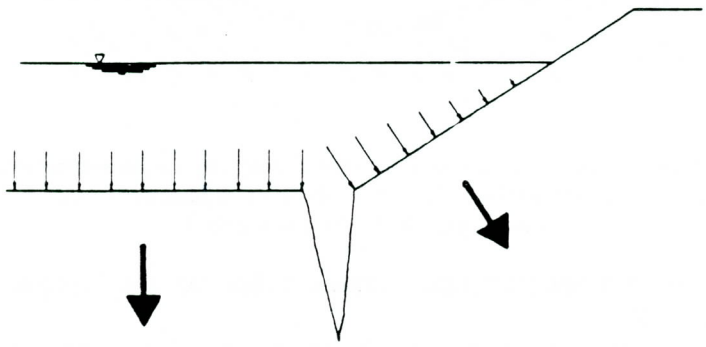

**Fig. 3.** Mechanism of tension crack formation at the toe of the side slope of a reservoir (not to scale). (After Loudière and Perrin.[1])

mechanisms. A case has been reported[1] where very large cracks (0·1–0·3 m wide) developed in the cohesive soil located under the geomembrane liner of a reservoir (Fig. 2). The cracks occurred near the toe of the side slopes of the reservoir. In this area, tensile stresses and differential movements resulted from the different water pressure orientations on the bottom and on the slopes, as shown in Fig. 3.

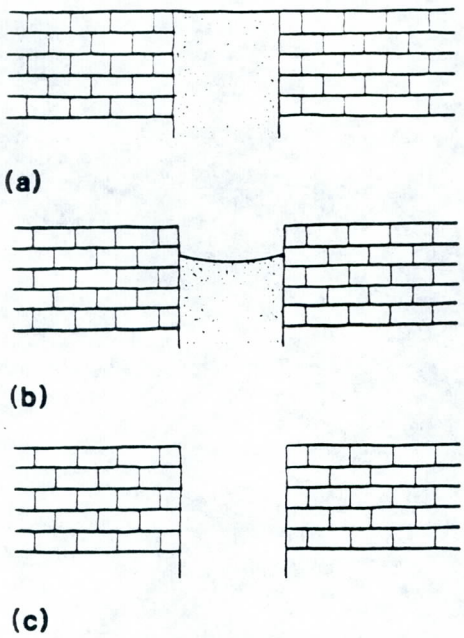


**Fig. 4.** Sinkhole in a karstic limestone mass: (a) before collapse; (b) after partial collapse; and (c) after complete collapse.

*Fissures and Cracks in Bedrock*

Soil layers or masses are sometimes constructed on a bedrock with fissures or cracks. A rare but important case is the construction of the clay core of a dam on a bedrock where cracks may develop. Some dam failures have resulted from this situation.

*Sinkholes due to Karstic Collapse*

Karstic limestone masses contain pockets or chimneys filled with soil. Water or other liquids seeping through a karstic limestone mass may remove soil from these pockets or chimneys, thereby creating a void which can be on the order of one to several meters in diameter (Fig. 4). These voids are usually referred to as sinkholes. The bursting of a geomembrane liner installed on a mass of karstic limestone which subsequently collapsed has been described by Giroud and Goldstein[2] and Giroud.[3] Karstic collapses can occur under other types of structures, such as road embankments, as discussed by Bonaparte and Berg.[4]

*Soil Dissolution*

Dissolution cavities can be caused by water in soils containing gypsum or by acid in soils containing calcium carbonate. The senior author has

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**Fig. 5.** Dissolution Cavity. This cavity in high gypsum content soil was caused by water leaking through a concrete canal liner.

observed cavities about one meter deep and one meter wide caused by: (i) water leaking through the concrete liner of canals constructed in soils with a high gypsum content (Fig. 5); and (ii) phosphoric acid leaking through a faulty seam of the geomembrane liner of a reservoir constructed on a high calcium-carbonate content soil (Fig. 6).[5]

*Differential Settlement*

Depressions in the ground surface may be formed when a localized area settles more than the rest ('differential settlement'). There are many situations where depressions result from differential settlement. These include depressions resulting from: (i) differential settlement of municipal solid waste (resulting from the heterogeneity of the waste) affecting a geosynthetic-soil cover system placed on the waste; (ii) settlement of a

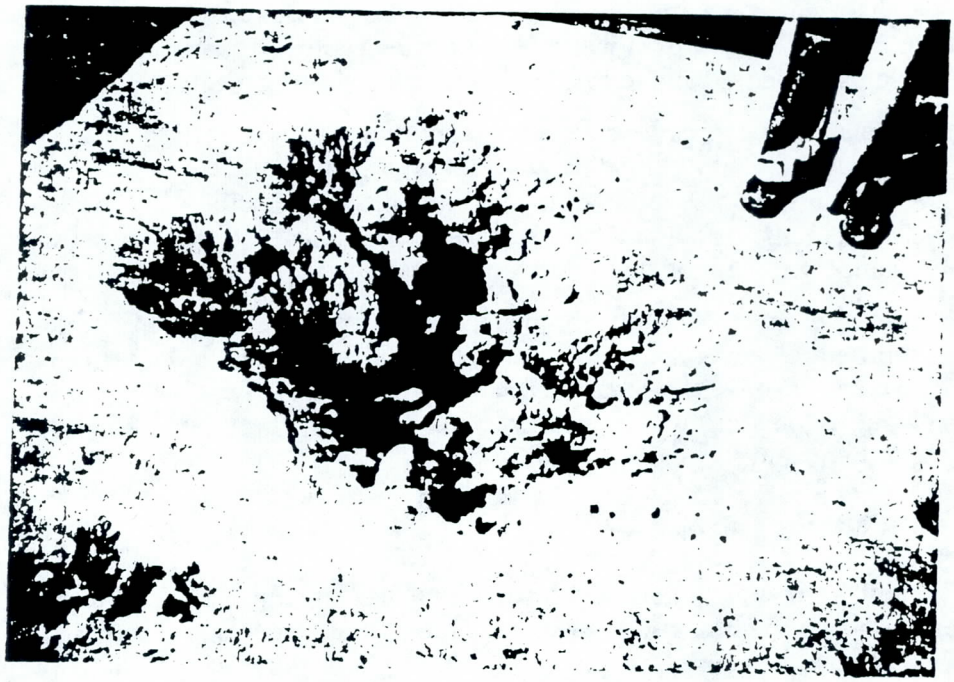

**Fig. 6.** Dissolution Cavity. This cavity in high calcium-carbonate content soil was caused by phosphoric acid leaking through a geomembrane liner.

localized lens of compressible soil; (iii) thawing of subsurface ice lenses; and (iv) settlement of a poorly compacted trench backfill. Tisserand[6] has reported a case of geomembrane failure over the depression resulting from trench backfill settlement. Differential settlement due to lenses of compressible soils frequently occur under road embankments.

*Localized Subsidence*

The surface of the ground may be locally depressed as a result of the collapse of underground cavities such as: natural caves, tunnels, mine workings, pipes, and tanks. Localized subsidence may also occur at the surface of municipal solid waste as a result of the collapse of deteriorating structures such as refrigerators.

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## Classification of Voids

Two shapes of voids are considered in the study presented in this paper: infinitely long voids with a width $b$ and circular voids with a diameter $2r$. The voids presented above can therefore be put into two categories:

- Cracks and depressions resulting from trench backfill settlement may be modeled approximately as an infinitely long void.
- Karstic sinkholes, dissolution cavities, municipal solid waste settlement, lens settlement, soil surface depressions and ground subsidence may be modeled approximately as a circular void.

In the case of cracks and complete karstic collapse (Fig. 4(c)), the geosynthetic deflects without reaching the bottom of the void. With the other types of voids, the geosynthetic may or may not reach the bottom of the void, depending on the geometry of the void, the modulus of the geosynthetic and the applied loads.

## Load-Carrying Mechanism

The soil layer and underlying geosynthetic are assumed initially to be resting on a firm foundation. At some point in time, a void of a certain size opens below the geosynthetic. Under the weight of the soil layer and any applied loads, the geosynthetic deflects. The deflection has two effects; *bending* of the soil layer and *stretching* of the geosynthetic.

The *bending* of the soil layer generates arching inside the soil, which transfers part of the applied load away from the void area, as shown in Fig. 7. As a result, the vertical stress, $\sigma_V$, over the void area is smaller than the average vertical stress, $\gamma H + q$, due to the weight of a soil layer of thickness $H$ and an applied uniform normal stress of magnitude $q$.

The *stretching* of the geosynthetic mobilizes a portion of the geosynthetic's strength. Consequently, the geosynthetic acts as a 'tensioned membrane' and can carry a load applied normally to its surface. As a result of geosynthetic stretching, two cases can be considered:

- In the first case, the stretched geosynthetic comes in contact with the bottom of the void. The mobilized portion of the geosynthetic strength carries a portion of the load applied normal to the surface of the geosynthetic. The rest of the load is transmitted to the bottom of the void.
- In the second case, the geosynthetic does not deflect enough to come in contact with the bottom of the void. In this case, either the geosynthetic is strong enough to support the entire load applied normal to its surface or it fails.

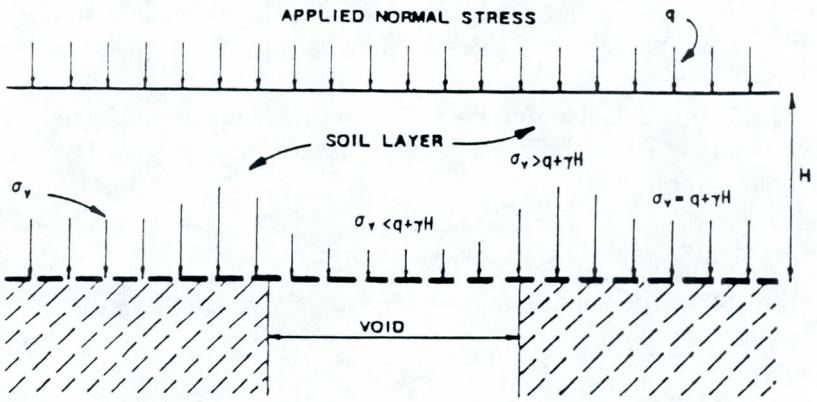


**Fig. 7.** Effect of soil arching on load distribution.

In summary, the soil-geosynthetic system deflects and the geosynthetic stretches until it fails (Fig. 1(a)) or until an equilibrium condition is reached (Fig. 1(b) or 1(c)).

## Scope of this Paper

This paper presents the development and use of equations, tables, and charts for the case of a soil layer subjected to a uniformly distributed normal load and resting on a geosynthetic overlying a rigid foundation containing a single infinitely long void (plane-strain problem) or circular void (axisymmetric problem). The parameters considered in this paper are:

- Geometric Parameters: These include the thickness of the soil layer and the geometry of the void (width of an infinitely long void or diameter of a circular void, and depth of void) (Fig. 8).
- Mechanical Parameters: These include the soil mechanical properties and the geosynthetic tensile behavior (expressed by its tension–strain curve).
- Loading Conditions: These include the unit weight of the soil layer and the load exerted on the top of the soil layer, which is assumed to be normal and uniformly distributed.

The equations, tables, and charts make it possible to solve design problems such as:

- select the required geosynthetic mechanical properties when the geometric parameters and the loading conditions are known;

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- determine the required thickness of the soil layer associated with a given geosynthetic over a given void and subjected to given loading conditions;
- determine the void size that a given geosynthetic may bridge when it is associated with a given soil layer subjected to given loading conditions; and
- determine the maximum load which can be carried by a given soil-geosynthetic system over a given void.

The solution of any of the above design problems depends on the allowable geosynthetic strain.

### Originality of this Paper

The use of tensioned membrane theory to evaluate the load-carrying capacity of a geosynthetic bridging a void was presented by Giroud.[7] Subsequently, Giroud[8] developed a design chart based on tensioned membrane theory. This chart has often been used to evaluate the load-carrying capacity of a soil layer associated with a geosynthetic. By doing so, the internal shear strength of the soil layer is neglected, and this can be very conservative. Therefore, Bonaparte and Berg[4] combined arching theory (for the soil layer) with tensioned membrane theory (for the geosynthetic) to formulate a more complete design approach.

This paper significantly extends the earlier work of Giroud[7,8] and Bonaparte and Berg[4] and provides an extensive analysis of soil-geosynthetic system bridging a void.

## ANALYSIS

### Assumptions

The void can be either circular (diameter $2r$) or infinitely long (width $b$). Regarding the bottom of the void, two cases can be considered: (i) a bottomless void (Fig. 8(a)); and (ii) a bottom with a maximum depth $D$ and a spherical shape (for the circular void) or a cylindrical shape with a circular cross section (for the infinite void) (Fig. 8(b)). From a design standpoint, both cases are identical if the deflection $y$ of the geosynthetic is less than the depth of $D$ of the void.

The soil layer is assumed to be horizontal and to have a uniform thickness $H$. The stress $q$ applied on the soil layer is assumed to be normal and uniformly distributed.

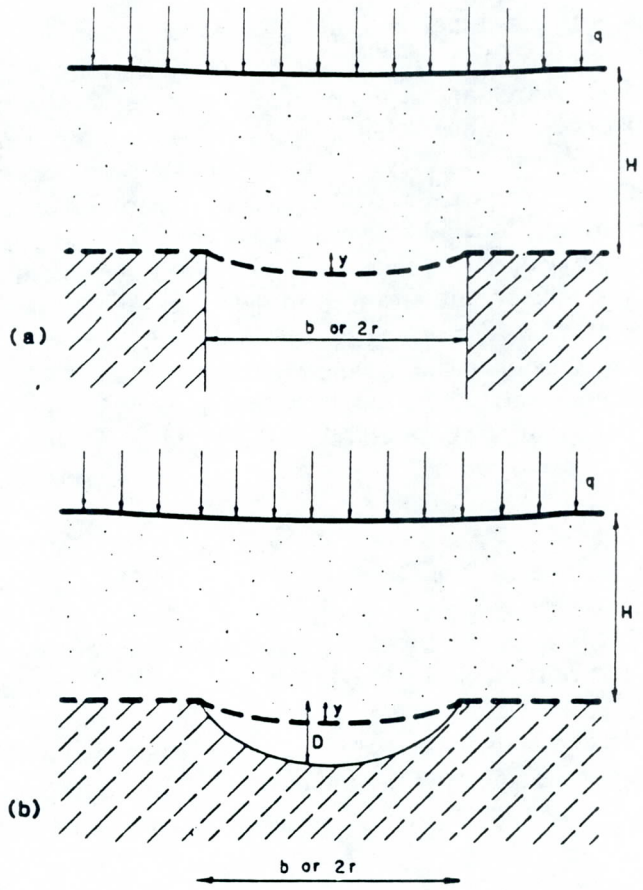


**Fig. 8.** Schematic cross section for theoretical analysis. Two cases can be considered: (a) the void is bottomless; and (b) the bottom of the void is assumed to have a circular cross section and the depth of the void is $D$. The void located under the geosynthetic is either infinitely long (with a width $b$), or circular (with a diameter $2r$); $y$ is the geosynthetic deflection.

Relevant geosynthetic properties are the tension–strain curve or, at least, the tension $\alpha$ corresponding to the design strain $\varepsilon$.

Relevant soil properties are the friction angle $\phi$ and the cohesion $c$. For the analysis presented in this paper, the cohesion is neglected. In other words, the charts are established for $c = 0$ and can be conservatively used for $c > 0$. Also, it will be shown that the friction angle $\phi$ does not have a significant influence on the analysis results if it is equal to or greater than 20°.

## Approach

The problem under consideration involves a complex soil-geosynthetic interaction. The problem can be greatly simplified, however, if the soil response (arching) is uncoupled from the geosynthetic response (tensioned membrane). Therefore, a two-step approach is used. First, the behavior of the soil layer is analyzed using classical *arching theory*. This step gives the pressure at the base of the soil layer on the portion of the geosynthetic located above the void. Second, *tensioned membrane theory* is used to establish a relationship between the pressure on the geosynthetic, the tension and strain in the geosynthetic, and the deflection of the geosynthetic. Accordingly, the following sections deal with arching theory, tensioned membrane theory, and the combination of both.

An inherent assumption in this uncoupled two-step approach is that the soil deformation required to generate the soil arch is compatible with the tensile strain required to mobilize the geosynthetic tension. This assumption has not been verified.

## Arching Theory (see Fig. 9)

When the geosynthetic deflects, arching develops in the soil layer. As a result, a portion of the applied stress is transmitted laterally and, consequently, the normal stress transmitted to the portion of the geosynthetic located above the void is smaller than the average vertical stress due to the weight of the soil layer and the uniformly distributed normal stress applied on top of the soil layer (Fig. 7). The procedures for calculating the reduced

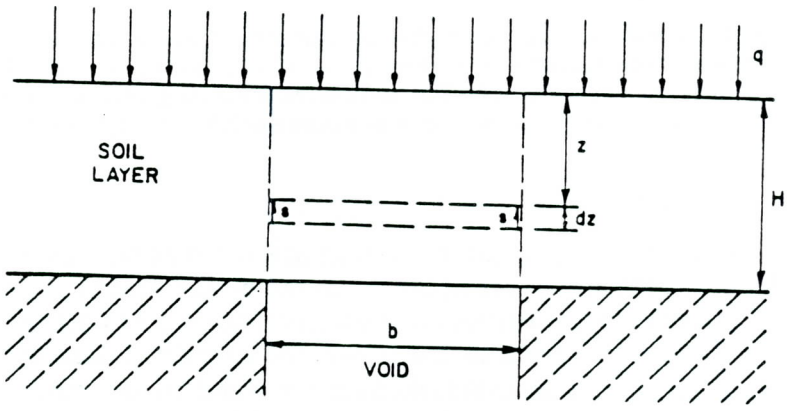


**Fig. 9.** Derivation of arching equation.

stress transmitted to the portion of the geosynthetic located above the void are presented below for an infinitely long void and a circular void.

*Infinitely Long Void*

Terzaghi[9] has established equations for soil arching over an infinitely long void assuming that the lateral load transfer is achieved through shear stresses along vertical planes located at the edges of the void (Fig. 9). As a result of this assumption, the incremental change in vertical stress, $d\sigma_V$, due to an incremental change in depth, $dz$, is given by

$$d\sigma_V = [\gamma - 2(s/b)]dz \tag{1}$$

where: $b$ = width of the infinitely long void; $\sigma_V$ = vertical stress at depth $z$; $\gamma$ = unit weight of soil; $z$ = depth measured from the top of the soil layer; and $s$ = soil shear strength. Basic SI units are: $b$(m), $\sigma_V$ (N/m$^2$), $\gamma$ (N/m$^3$), $z$ (m), and $s$ (N/m$^2$).

The soil shear strength along a vertical plane is expressed by

$$s = c + \sigma_H \tan\phi \tag{2}$$

where: $c$ = cohesion of the soil; $\sigma_H$ = horizontal stress at depth $z$; and $\phi$ = friction angle of the soil. Basic SI units are: $s$ (N/m$^2$), $c$ (N/m$^2$), $\sigma_H$ (N/m$^2$), and $\phi$ (degrees); $\phi$ is dimensionless.

The relationship between the horizontal stress and the vertical stress is given by the following classical relationship

$$\sigma_H = K\sigma_V \tag{3}$$

where: $K$ = coefficient of lateral earth pressure (dimensionless).

It should be noted that many of the relationships presented in this paper are valid for both effective and total stress conditions; however, eqn (3) is valid only for effective stress conditions.

Combining eqns (1), (2) and (3) and solving the differential equation for the boundary condition $\sigma_V = q$ for $z = 0$ gives

$$\sigma_V = \frac{b(\gamma - 2c/b)}{2K\tan\phi}[1 - e^{-K\tan\phi(2z/b)}] + q\,e^{-K\tan\phi(2z/b)} \tag{4}$$

where: $q$ = uniformly distributed normal stress applied on the top of the soil layer (basic SI unit: N/m$^2$); all other notations as defined above and in the Notations section.

The pressure on top of the geosynthetic, over the void area, $p$, is the

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value of $\sigma_V$ for $z = H$ in eqn (4). If the soil cohesion, $c$, is assumed to equal zero, the value of $p$ is

$$p = \frac{\gamma b}{2K \tan \phi}[1 - e^{-2K \tan \phi H/b}] + q\, e^{-2K \tan \phi H/b} \quad (5)$$

where: $p$ = pressure on top of the geosynthetic (i.e. vertical stress at the bottom of the soil layer), over the void area (basic SI unit: $N/m^2$); and other notations as defined above and in the Notations section.

*Circular Void*

Using the same approach, Kezdi[10] has established that eqn (5) can be used for a circular void if $b$ is replaced by $r$ (and not by $2r$), which shows that arching is twice as significant for a circular void compared to an infinitely long void.

*Practical Approximate Equations*

Selection of the value of the coefficient of lateral earth pressure is not easy since the state of stress of the soil in the zone where arching develops is not fully understood. Handy[11] has made a thorough analysis of soil arching and proposed the following value

$$K = 1{\cdot}06(\cos^2 \theta + K_a \sin^2 \theta) \quad (6)$$

with

$$\theta = 45° + \phi/2 \quad (7)$$

and

$$K_a = \tan^2(45° - \phi/2) \quad (8)$$

where: $K_a$ = coefficient of active earth pressure (dimensionless); and other notations as defined above and in the Notations section.

Another approach would consist of using the coefficient of earth pressure at rest, expressed as follows, according to Jaky[12]

$$K = 1 - \sin \phi \quad (9)$$

In eqn (5), $K$ is multiplied by $\tan\phi$. Values of $K\tan\phi$, calculated using eqns (6) and (9), are given in Table 1. It appears that $K\tan\phi$ does not vary significantly with $\phi$, if $\phi$ is equal to or greater than 20°, which is the case for virtually all granular soils and for many fine-grained soils under drained conditions. Therefore, a constant value of 0·25 can be used for $K\tan\phi$ when $\phi$ is equal to or greater than 20°. As a result, eqn (5) becomes

$$p = 2\gamma b(1 - e^{-0{\cdot}5H/b}) + q\, e^{-0{\cdot}5H/b} \quad (10)$$

**TABLE 1**
Values of $K \tan \phi$

| *Soil friction angle ($\phi$, degrees)* | *Values of $K \tan \phi$* | |
|---|---|---|
| | *Using K from Handy (eqn (6))* | *Using K from Jaky (eqn (9))* |
| 0 | 0 | 0 |
| 5 | 0·08 | 0·08 |
| 10 | 0·15 | 0·15 |
| 15 | 0·21 | 0·20 |
| 20 | 0·25 | 0·24 |
| 25 | 0·29 | 0·27 |
| 30 | 0·31 | 0·29 |
| 35 | 0·32 | 0·30 |
| 40 | 0·32 | 0·30 |
| 45 | 0·31 | 0·29 |
| 50 | 0·30 | 0·28 |
| 55 | 0·27 | 0·26 |

Two values of $K$, the coefficient of lateral earth pressure, are considered: the value proposed by Handy[11] for arching and the value proposed by Jaky[12] for the 'at rest' state of stress.

Like eqn (5), eqn (10) is also valid for the circular void if $b$ is replaced by $r$.

Equation 10 was used to establish Tables 3 and 4, and the charts given in Figs 11 and 14.

*Comment on the Validity of Arching Theory*

The analysis presented above is the classical analysis by Terzaghi.[9] This analysis does not consider soil dilatancy, which can increase the horizontal stress in the soil, thereby increasing the ability of the soil to arch. Therefore, the analysis presented in this paper can be considered conservative from this viewpoint. On the other hand, the analysis may not be conservative for loose soils that tend to contract when sheared.

## Tensioned Membrane Theory

The tensioned membrane theory has been used by Giroud[3,7] to deal with the case of a geosynthetic overlying a void and subjected to a uniformly distributed stress normal to its surface.

The equations given below have been established with the following assumptions: (i) the strain in the portion of the geosynthetic overlying the

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void (i.e. the deflected portion of the geosynthetic) is uniformly distributed; and (ii) the strain in the portion of the geosynthetic outside the void area is zero and, therefore, that portion of the geosynthetic does not move (i.e. the geosynthetic does not slide toward the void). These two assumptions greatly simplify the analysis, but no attempt has been made to evaluate their range of validity.

*Infinitely Long Void*

In the case of an infinitely long void, the deflected shape of the geosynthetic across the width of the void is cylindrical with a circular cross section, the strain is uniform, and the following relationships exist

$$1 + \varepsilon = 2\Omega \sin^{-1}[1/(2\Omega)] \qquad \text{(valid if } y/b \leqq 0{\cdot}5) \tag{11}$$

$$1 + \varepsilon = 2\Omega\{\pi - \sin^{-1}[1/(2\Omega)]\} \qquad \text{(valid if } y/b \geqq 0{\cdot}5) \tag{12}$$

where: $\varepsilon$ = geosynthetic strain; $y$ = geosynthetic deflection; $b$ = width of the infinitely long void; and $\Omega$= dimensionless factor. Basic SI units are: $y$(m) and $b$(m); $\varepsilon$ and $\Omega$ are dimensionless.

The dimensionless factor $\Omega$ is defined by

$$\Omega = (1/4)[2y/b + b/(2y)] \tag{13}$$

As a result of eqns (11), (12) and (13), there is a unique relationship between $y/b$, $\varepsilon$ and $\Omega$, which is given in Table 2 and shown in Fig. 10.

It is interesting to note that as $\varepsilon$ tends towards zero eqn (11) tends toward

$$\Omega = 1/\sqrt{24\varepsilon} \tag{14}$$

This equation gives a good approximation of $\Omega$ when $\varepsilon$ is less than 1% (see Fig. 10).

Giroud[3,7] has also shown that the tension in the geosynthetic, in the case of an infinitely long void, is given by

$$\alpha = pb\Omega \tag{15}$$

where: $\alpha$ = geosynthetic tension; $p$ = pressure on the geosynthetic over the void area (i.e. vertical stress at the bottom of the soil layer over the void area); $b$ = width of the infintely long void; $\Omega$ = dimensionless factor

**TABLE 2**
Values of $\Omega$ as a Function of Deflection or Strain

| $y/b$ or $y/(2r)$ | $\varepsilon(\%)$ | $\Omega$ | $y/b$ or $y/(2r)$ | $\varepsilon(\%)$ | $\Omega$ |
|---|---|---|---|---|---|
| 0·000 | 0·000 | ∞ | 0·242 | 15·00 | 0·64 |
| 0·010 | 0·027 | 12·51 | 0·250 | 15·91 | 0·62 |
| 0·020 | 0·107 | 6·26 | 0·260 | 17·15 | 0·61 |
| 0·030 | 0·240 | 4·18 | 0·270 | 18·43 | 0·60 |
| 0·040 | 0·425 | 3·15 | 0·280 | 19·75 | 0·59 |
| 0·050 | 0·663 | 2·53 | 0·282 | 20·00 | 0·58 |
| 0·060 | 0·960 | 2·11 | 0·290 | 21·10 | 0·58 |
| 0·061 | 1·000 | 2·07 | 0·300 | 22·50 | 0·57 |
| 0·070 | 1·30 | 1·82 | 0·310 | 23·93 | 0·56 |
| 0·080 | 1·70 | 1·60 | 0·317 | 25·00 | 0·55 |
| 0·087 | 2·00 | 1·47 | 0·320 | 25·39 | 0·55 |
| 0·090 | 2·15 | 1·43 | 0·330 | 26·89 | 0·54 |
| 0·100 | 2·65 | 1·30 | 0·340 | 28·43 | 0·54 |
| 0·107 | 3·00 | 1·23 | 0·350 | 30·00 | 0·53 |
| 0·110 | 3·20 | 1·19 | 0·360 | 31·60 | 0·53 |
| 0·120 | 3·80 | 1·10 | 0·370 | 33·23 | 0·52 |
| 0·123 | 4·00 | 1·08 | 0·380 | 34·90 | 0·52 |
| 0·130 | 4·45 | 1·03 | 0·381 | 35·00 | 0·52 |
| 0·138 | 5·00 | 0·97 | 0·390 | 36·60 | 0·52 |
| 0·140 | 5·15 | 0·96 | 0·400 | 38·32 | 0·51 |
| 0·150 | 5·90 | 0·91 | 0·410 | 40·00 | 0·52 |
| 0·151 | 6·00 | 0·90 | 0·420 | 41·86 | 0·51 |
| 0·160 | 6·69 | 0·86 | 0·430 | 43·67 | 0·51 |
| 0·164 | 7·00 | 0·84 | 0·437 | 45·00 | 0·50 |
| 0·170 | 7·54 | 0·82 | 0·440 | 45·51 | 0·50 |
| 0·175 | 8·00 | 0·80 | 0·450 | 47·38 | 0·50 |
| 0·180 | 8·43 | 0·78 | 0·460 | 49·27 | 0·50 |
| 0·186 | 9·00 | 0·76 | 0·464 | 50·00 | 0·50 |
| 0·190 | 9·36 | 0·75 | 0·470 | 51·18 | 0·50 |
| 0·197 | 10·00 | 0·73 | 0·480 | 53·13 | 0·50 |
| 0·200 | 10·35 | 0·72 | 0·490 | 55·00 | 0·50 |
| 0·210 | 11·37 | 0·70 | 0·500 | 57·08 | 0·50 |
| 0·216 | 12·00 | 0·69 | 0·562 | 70·00 | 0·50 |
| 0·220 | 12·44 | 0·68 | 0·631 | 85·00 | 0·51 |
| 0·230 | 13·56 | 0·66 | 0·696 | 100·00 | 0·53 |
| 0·240 | 14·71 | 0·64 | 0·819 | 130·00 | 0·56 |

This table also gives values of the strain as a function of the deflection, and vice versa. (See also Fig. 10.) Notations: $\Omega$ = dimensionless factor used for the calculation of the tension in the geosynthetic; $y$ = geosynthetic deflection; $b$ = width of the infinitely long void; $2r$ = diameter of the circular void; and $\varepsilon$ = geosynthetic strain. (Note: in the case of a circular void, the values of $\varepsilon$ and $\Omega$ given in this table are approximate.)

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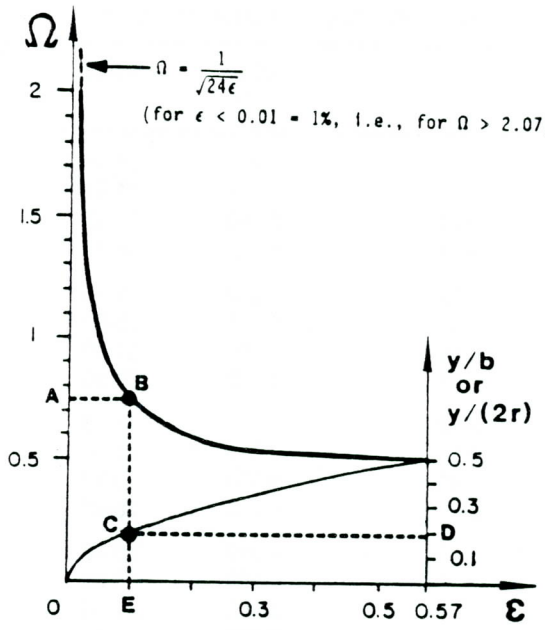


**Fig. 10.** Dimensionless factor Ω. (See also Table 2.) Notations: $b$ = width of the infinitely long void; $2r$ = diameter of the circular void; $y$ = geosynthetic deflection; $\varepsilon$ = geosynthetic strain; and Ω = dimensionless factor. (Note that, in the case of a circular void, $y$ is divided by $2r$, not by $r$.) This chart can be used as follows: (i) entering a known value of the geosynthetic strain, $\varepsilon$, in E and following EBA gives the value of Ω in A; (ii) entering a known value of the relative deflection, $y/b$ *or* $y/(2r)$, in D and following DCBA gives the value of Ω in A; (iii) entering a known value of the relative deflection, $y/b$ or $y/(2r)$, in D and following DCE gives the value of $\varepsilon$ in E; and (iv) vice versa. (For example, $\varepsilon$ = 0·1 (10%), Ω = 0·73, and $y/b$ = 0·197 are related.)

given in Table 2 and Fig. 10 as a function of $\varepsilon$ or $y/b$; $\varepsilon$ = geosynthetic strain; and $y$ = geosynthetic deflection. Basic SI units are: $\alpha$ (N/m), $p$ (N/m$^2$), $b$ (m), and $y$ (m); Ω and $\varepsilon$ are dimensionless.

*Circular Void*

As described by Giroud,[7] the deflected shape of the geosynthetic is not a sphere in the case of a circular void. As a consequence, incorporating $2r$ (diameter) instead of $b$ (width) into eqns (11), (12) and (13), gives only an *approximate* value of the *average* geosynthetic strain, $\varepsilon$.

Since the strain is not uniform, the tension, $\alpha$, in the case of a circular void is not uniformly distributed in the geosynthetic and its *average* value is given *approximately* by eqn (15) with $r$ substituted for $b$.[3,7] It should be

noted that, for a circular void, $r$ is substituted for $b$ in eqn (15) whereas $2r$ is used to determine $\Omega$, as indicated in Table 2 and Fig. 10.

Equation (15) can be used for a circular void only if the geosynthetic has isotropic tensile characteristics, i.e. the same tensile characteristics in all directions. If this is not the case, recommendations given in the section 'Discussion of Special Problems' should be followed.

*Applications of Tensioned Membrane Theory*

Tensioned membrane theory can be used alone (i.e. not combined with arching theory) to solve design problems relating to the case of a geosynthetic acting alone and subjected to a uniformly distributed pressure. This typically occurs in the case of geomembranes directly overlying a void and subjected to pressure from a liquid. Typical design problems are as follows:

- Determine the maximum pressure that a geomembrane can withstand over a void of a given size.
- Select the required geomembrane properties for a geomembrane to bridge a given void when it is subjected to a given pressure.
- Determine the void size that a given geomembrane may bridge when it is subjected to a given pressure.
- Determine the deflection of a geomembrane subjected to a given pressure on a given void, and determine if the deflected geomembrane will come in contact with the bottom of the void.

A chart has been published[2] to help solve these problems. It is also possible to use Table 3 with $H = 0$.

## Combination of Arching and Tensioned Membrane Theories

The problem of a bottomless void is entirely solved by using eqns (10) and (15). The case when the geosynthetic comes in contact with the bottom of the void is more complex and will be discussed later in this paper, in the section 'Discussion of Special Problems'.

Equation (10) gives a relationship between the applied stress, the soil layer thickness, the void size, and the pressure on the geosynthetic. This equation was established using *arching theory*.

Equation (15) gives a relationship between the pressure on the geosynthetic, the void size, and the geosynthetic tensile characteristics (tension and strain). This equation was established using *tensioned membrane theory*.

The solution of typical design problems using the equations mentioned above is discussed in the next section.

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## SOLUTION OF TYPICAL DESIGN PROBLEMS

### Overview of the Methods Used

In the presentation of the scope of this paper, a list of typical design problems was given. Solutions to these problems are presented below for the case when the geosynthetic does not come in contact with the bottom of the void. Solutions for the case where the geosynthetic comes in contact with the bottom of the void are presented in the section 'Discussion of Speecial Problems'.

*Allowable Strain and Deflection*

In all of the design cases considered below, the solution depends on the value of $\Omega$, which depends either on the allowable geosynthetic strain, $\varepsilon$, or the allowable geosynthetic deflection, $y$. The *allowable geosynthetic strain* is the lesser of the maximum design strain for the considered geosynthetic and the strain beyond which the soil layer would be unacceptably deformed or cracked. The *allowable geosynthetic deflection* is considered when excessive deflection of the soil surface impairs the serviceability of the system. No method is proposed in this paper to evaluate the deflection of the soil surface; however, in the case of relatively thin soil layers, the soil surface deflection can be assumed to be on the same order as the geosynthetic deflection. In some instances, both the allowable geosynthetic strain and the allowable geosynthetic deflection may need to be considered.

*Equations and Notations*

All equations presented below were obtained by combining eqns (10) and (15). Notations for all subsequent equations are: $b$ = width of the infinitely long void; $r$ = radius of the circular void; $\Omega$ = dimensionless factor given in Table 2 as a function of $\varepsilon$ or $y$; $H$ = soil layer thickness; $p$ = normal stress applied on the portion of the geosynthetic located over the void ('pressure on the geosynthetic'); $q$ = uniformly distributed normal stress applied on the top of the soil layer; $y$ = geosynthetic deflection; $\alpha$ = geosynthetic tension; $\gamma$ = unit weight of soil; and $\varepsilon$ = geosynthetic strain. Basis SI units are: $b$ (m), $r$ (m), $H$ (m), $p$ (N/m$^2$), $q$ (N/m$^2$), $y$ (m), $\alpha$ (N/m), and $\gamma$ (N/m$^3$); $\Omega$ and $\varepsilon$ are dimensionless.

*Factor of Safety*

In the following sections, each design problem is illustrated by an example. For the sake of simplicity, no factor of safety is used in the design examples. Engineers using the equations, tables, and charts presented in

this paper should use appropriate factors of safety. The factor of safety can be applied to the geosynthetic tension or the applied loads, with application to the geosynthetic tension being more common. The factor of safety should not be applied to the soil shear strength (as is commonly the case in geotechnical problems) due to the insensitivity of the arching theory results (eqn (5)) to the soil shear strength.

**Determination of Required Geosynthetic Properties**

The relevant equation for an infinitely long void is

$$\alpha/\Omega = pb = 2\gamma b^2(1 - e^{-0.5H/b}) + qb\,e^{-0.5H/b} \tag{16}$$

Equation (16) can be rewritten in a dimensionless form as follows:

$$\frac{\alpha}{\gamma b^2 \Omega} = \frac{p}{\gamma b} = 2(1 - e^{-0.5H/b}) + \frac{q}{\gamma b} e^{-0.5H/b} \tag{17}$$

Equations (16) and (17) can be used for a circular void if $b$ is replaced by $r$. Equation (17) was used to establish the chart in Fig. 11.

The above equations can be used to solve problems that consist of determining the required geosynthetic tension, $\alpha$, for a given strain, $\varepsilon$, when all other parameters are given ($b$ or $r$, $q$, $H$, and $\gamma$). Alternatively, the chart given in Fig. 11 and the corresponding Table 3 can be used.

*Example 1.* The bedding soil supporting a geomembrane liner is placed on a geosynthetic reinforcement resting on a soil where karstic sinkholes may develop (Fig. 12). The function of the geosynthetic reinforcement is to support the bedding soil and the geomembrane liner should a sinkhole develop. The thickness of the bedding soil layer is 0·45 m and the depth of water on the geomembrane when the reservoir is full is 9 m. The unit weight of the bedding soil is 19,600 N/m$^3$. A deep sinkhole with a radius of 0·75 m is assumed for design purposes. Since the function of the geosynthetic reinforcement is only to act as a 'safety net', a rather large geosynthetic reinforcement strain is acceptable: $\varepsilon = 10\%$. What is the required geosynthetic reinforcement tensile strength?
First, the applied stress, $q$, is calculated

$$q = 1000 \times 9{\cdot}81 \times 9 = 88\,290 \text{ N/m}^2$$

Then, eqn (16) is used as follows, with $H/r = 0{\cdot}45/0{\cdot}75 = 0{\cdot}6$

$$\alpha/\Omega = 2 \times 19\,600 \times (0{\cdot}75)^2(1 - e^{-0.3}) + 88\,290 \times 0{\cdot}75\,e^{-0.3}$$

$$\alpha/\Omega = 54\,395 \text{ N/m}$$

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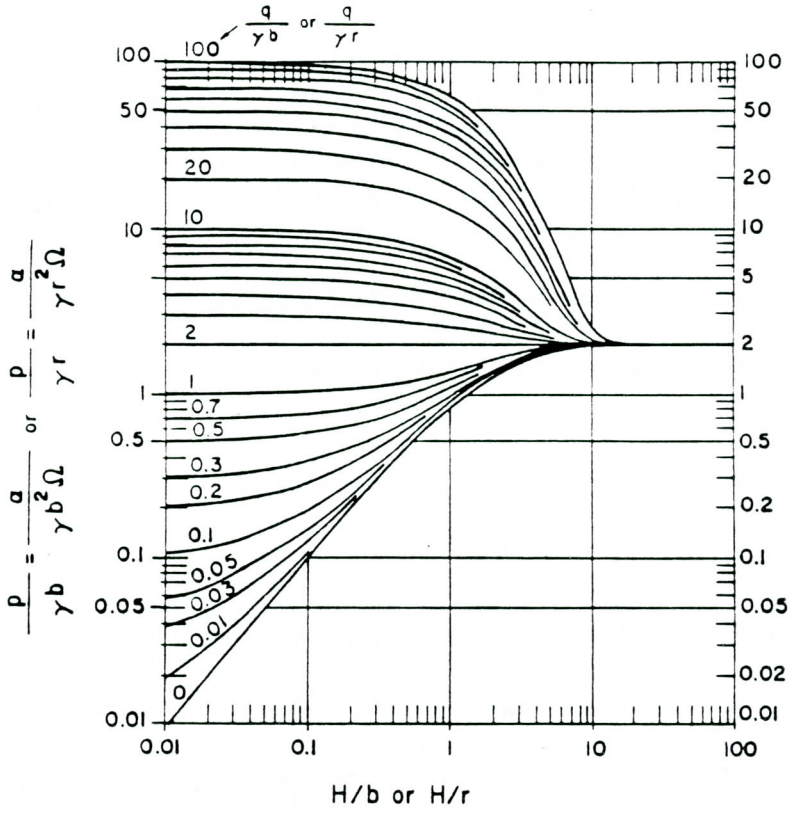


**Fig. 11.** Pressure on and tension in the geosynthetic. An example of use of this chart is given in Fig. 13. Notations: $p$ = pressure on the geosynthetic over the void area; $q$ = uniformly distributed normal stress applied on the top of the soil layer; $H$ = thickness of the soil layer; $\gamma$ = unit weight of soil; $b$ = width of the infinitely long void; $r$ = radius of the circular void; $\alpha$ = geosynthetic tension; and $\Omega$ = dimensionless factor given in Table 2 and Fig. 10. (Values of $p/(\gamma b)$ or $p/(\gamma r)$ used to draw the curves in this figure can be found in Table 3.)

Finally, according to Table 2 or Fig. 10, $\Omega = 0{\cdot}73$ for $\varepsilon = 10\%$. Therefore, the required value of the geosynthetic tension at a 10% strain is:

$$\alpha = 0{\cdot}73 \times 54\,395 = 39\,708 \text{ N/m} = 40 \text{ kN/m}$$

The same problem can be solved using the tables and charts with

$$H/r = 0{\cdot}45/0{\cdot}75 = 0{\cdot}6 \text{ and}$$

$$q/(\gamma r) = 88\,290/(19\,600 \times 0{\cdot}75) = 6{\cdot}0$$

Table 3 or the chart given in Fig. 11 (see also Fig. 13) gives:

$\alpha/(\gamma r^2 \Omega) = 4{\cdot}963$ hence

$\alpha = (4{\cdot}963) \times 19\,600 \times (0{\cdot}75)^2 \times 0{\cdot}73 = 39\,943$ N/m $= 40$ kN/m

It is interesting to compare the required geosynthetic reinforcement tension calculated above to that required if the bedding soil is a layer of compacted clay associated with the geomembrane to form a composite liner. In this case, it is important that the integrity of the clay layer be maintained. Therefore, the geosynthetic reinforcement strain must be small enough to prevent the development of tension cracks in the clay layer. Calculations similar to the above, with $\varepsilon = 1\%$ instead of 10%, give a required geosynthetic reinforcement tension of 113 kN/m, which is about three times greater than 40 kN/m. Therefore, the geosynthetic reinforcement required in the case of a 1% allowable strain has a tension about three times greater, and consequently a modulus about 30 times greater, than in the case of a 10% allowable strain. (Several layers of a very high-modulus geotextile would probably be needed.)

## Determination of Required Soil Layer Thickness

The relevant equation for an infinitely long void is

$$H = 2b \ln \frac{[q/(\gamma b)] - 2}{[\alpha/(\gamma b^2 \Omega)] - 2} \tag{18}$$

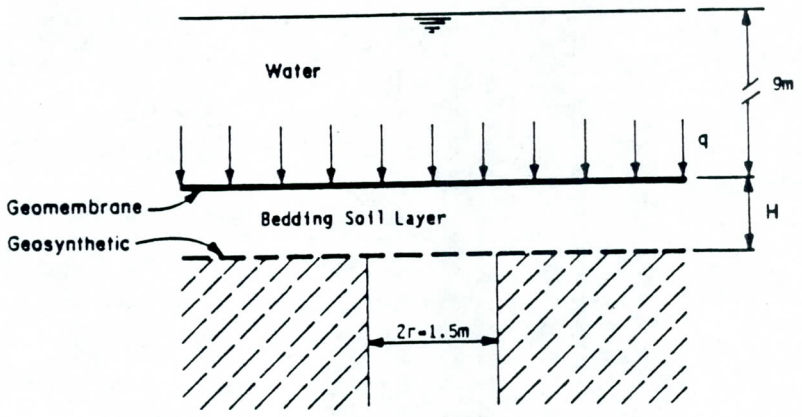


**Fig. 12.** Cross section for design examples.

**TABLE 3**
Pressure on the Geosynthetic

| $q/(\gamma b)$ or $q/(\gamma r)$ | $H/b$ or $H/r$ | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0 | 0·01 | 0·03 | 0·1 | 0·3 | 0·5 | 0·6 | 1·0 | 3·0 | 5·0 | 7·0 | 10·0 | 20·0 | ∞ |
| | (Values of $p/(\gamma/b) = \alpha/(\gamma b^2 \Omega)$ or $p/(\gamma r) = \alpha/(\gamma r^2 \Omega)$) | | | | | | | | | | | | | |
| 0·0 | 0 | 0·010 | 0·030 | 0·098 | 0·279 | 0·442 | 0·518 | 0·787 | 1·554 | 1·836 | 1·940 | 1·987 | 2·000 | 2·000 |
| 0·01 | 0·010 | 0·020 | 0·040 | 0·107 | 0·287 | 0·450 | 0·526 | 0·793 | 1·556 | 1·837 | 1·940 | 1·987 | 2·000 | 2·000 |
| 0·03 | 0·030 | 0·040 | 0·059 | 0·126 | 0·304 | 0·466 | 0·541 | 0·805 | 1·560 | 1·838 | 1·941 | 1·987 | 2·000 | 2·000 |
| 0·05 | 0·050 | 0·060 | 0·079 | 0·145 | 0·322 | 0·481 | 0·555 | 0·817 | 1·565 | 1·840 | 1·941 | 1·987 | 2·000 | 2·000 |
| 0·1 | 0·100 | 0·109 | 0·128 | 0·193 | 0·365 | 0·520 | 0·592 | 0·848 | 1·576 | 1·844 | 1·943 | 1·987 | 2·000 | 2·000 |
| 0·2 | 0·200 | 0·209 | 0·227 | 0·288 | 0·451 | 0·598 | 0·667 | 0·908 | 1·598 | 1·852 | 1·946 | 1·988 | 2·000 | 2·000 |
| 0·3 | 0·300 | 0·308 | 0·325 | 0·383 | 0·537 | 0·676 | 0·741 | 0·969 | 1·621 | 1·860 | 1·949 | 1·989 | 2·000 | 2·000 |
| 0·5 | 0·500 | 0·507 | 0·522 | 0·573 | 0·709 | 0·832 | 0·889 | 1·090 | 1·665 | 1·877 | 1·955 | 1·990 | 2·000 | 2·000 |
| 0·7 | 0·700 | 0·706 | 0·719 | 0·763 | 0·881 | 0·988 | 1·037 | 1·212 | 1·710 | 1·893 | 1·961 | 1·991 | 2·000 | 2·000 |
| 1·0 | 1·000 | 1·005 | 1·015 | 1·049 | 1·139 | 1·221 | 1·259 | 1·393 | 1·777 | 1·918 | 1·970 | 1·993 | 2·000 | 2·000 |
| 1·5 | 1·500 | 1·502 | 1·507 | 1·524 | 1·570 | 1·611 | 1·630 | 1·697 | 1·888 | 1·959 | 1·985 | 1·997 | 2·000 | 2·000 |
| 2·0 | 2·000 | 2·000 | 2·000 | 2·000 | 2·000 | 2·000 | 2·000 | 2·000 | 2·000 | 2·000 | 2·000 | 2·000 | 2·000 | 2·000 |
| 2·5 | 2·500 | 2·498 | 2·493 | 2·476 | 2·430 | 2·389 | 2·370 | 2·303 | 2·112 | 2·041 | 2·015 | 2·003 | 2·000 | 2·000 |
| 3·0 | 3·000 | 2·995 | 2·985 | 2·951 | 2·861 | 2·779 | 2·741 | 2·607 | 2·223 | 2·082 | 2·030 | 2·007 | 2·000 | 2·000 |
| 4·0 | 4·000 | 3·990 | 3·970 | 3·902 | 3·721 | 3·558 | 3·482 | 3·213 | 2·446 | 2·164 | 2·060 | 2·013 | 2·000 | 2·000 |
| 5·0 | 5·000 | 4·985 | 4·955 | 4·854 | 4·582 | 4·336 | 4·222 | 3·820 | 2·669 | 2·246 | 2·091 | 2·020 | 2·000 | 2·000 |

| | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 6·0 | 6·000 | 5·980 | 5·940 | 5·805 | 5·443 | 5·115 | 4·963 | 4·426 | 2·893 | 2·328 | 2·121 | 2·027 | 2·000 | 2·000 |
| 7·0 | 7·000 | 6·975 | 6·926 | 6·756 | 6·304 | 5·894 | 5·704 | 5·033 | 3·116 | 2·410 | 2·151 | 2·034 | 2·000 | 2·000 |
| 8·0 | 8·000 | 7·970 | 7·911 | 7·707 | 7·164 | 6·673 | 6·445 | 5·639 | 3·339 | 2·493 | 2·181 | 2·040 | 2·000 | 2·000 |
| 9·0 | 9·000 | 8·965 | 8·896 | 8·659 | 8·025 | 7·452 | 7·186 | 6·246 | 3·562 | 2·575 | 2·211 | 2·047 | 2·000 | 2·000 |
| 10 | 10·000 | 9·960 | 9·881 | 9·610 | 8·886 | 8·230 | 7·927 | 6·852 | 3·785 | 2·657 | 2·242 | 2·054 | 2·000 | 2·000 |
| 15 | 15·000 | 14·935 | 14·806 | 14·366 | 13·189 | 12·124 | 11·631 | 9·885 | 4·901 | 3·067 | 2·393 | 2·088 | 2·000 | 2·000 |
| 20 | 20·000 | 19·910 | 19·732 | 19·122 | 17·493 | 16·018 | 15·335 | 12·918 | 6·016 | 3·478 | 2·544 | 2·121 | 2·000 | 2·000 |
| 25 | 25·000 | 24·885 | 24·658 | 23·878 | 21·796 | 19·912 | 19·039 | 15·950 | 7·132 | 3·888 | 2·695 | 2·155 | 2·000 | 2·000 |
| 30 | 30·000 | 29·860 | 29·583 | 28·634 | 26·100 | 23·806 | 22·743 | 18·983 | 8·248 | 4·298 | 2·846 | 2·189 | 2·000 | 2·000 |
| 40 | 40·000 | 39·810 | 38·434 | 38·147 | 34·707 | 31·594 | 30·151 | 25·048 | 10·479 | 5·119 | 3·148 | 2·256 | 2·000 | 2·000 |
| 50 | 50·000 | 49·761 | 49·285 | 47·659 | 43·314 | 39·382 | 37·559 | 31·113 | 12·710 | 5·940 | 3·449 | 2·323 | 2·000 | 2·000 |
| 60 | 60·000 | 59·711 | 59·136 | 57·171 | 51·921 | 47·170 | 44·967 | 37·179 | 14·942 | 6·761 | 3·751 | 2·391 | 2·000 | 2·000 |
| 70 | 70·000 | 69·661 | 68·988 | 66·684 | 60·528 | 54·958 | 52·376 | 43·244 | 17·173 | 7·582 | 4·053 | 2·458 | 2·000 | 2·000 |
| 80 | 80·000 | 79·611 | 78·839 | 76·196 | 69·135 | 62·746 | 59·784 | 49·309 | 19·404 | 8·403 | 4·355 | 2·526 | 2·000 | 2·000 |
| 90 | 90·000 | 89·561 | 88·690 | 85·708 | 77·742 | 70·534 | 67·192 | 55·375 | 21·635 | 9·223 | 4·657 | 2·593 | 2·000 | 2·000 |
| 100 | 100·000 | 99·511 | 98·541 | 95·220 | 86·349 | 78·322 | 74·600 | 61·440 | 23·867 | 10·044 | 4·959 | 2·660 | 2·000 | 2·000 |

This table gives $p/(\gamma b)$ or $p/(\gamma r)$ and the geosynthetic tension as a function of the other parameters involved. Notation: $p$ = pressure on the geosynthetic over the void area; $q$ = uniformly distributed normal stress applied on the top of the soil layer; $H$ = thickness of the soil layer; $\gamma$ = unit weight of the soil in the soil layer; $b$ = width of the infinitely long void; $r$ = radius of the circular void; $\alpha$ = geosynthetic tension; and $\Omega$ = dimensionless factor given in Table 2 as a function of the geosynthetic strain, $\varepsilon$. Note that: values of $p/(\gamma b)$ or $p/(\gamma r)$ for $H/b = 0$ are identical to values of $q/(\gamma b)$ or $q/(\gamma r)$; and $p = 2\gamma b$ if $H$ is greater than approximately $20b$ and $p = 2\gamma r$ if $H$ is greater than approximately $20r$. (See the chart given in Fig. 11.)



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The same equation can be used for a circular void by substituting $r$ for $b$.

The above equations can be used to solve problems that consist of determining the required soil layer thickness, $H$, when all other parameters are given ($b$ or $r$, $q$, $\gamma$, $\alpha$, and $\varepsilon$). Alternatively, the charts given in Fig. 11, and the corresponding Table 3, can be used.

*Example 2.* This example is identical to Example 1, except that the soil layer thickness, $H$, is unknown, and the geosynthetic tension at a strain $\varepsilon = 10\%$ is known and is equal to 40 kN/m. What is the required soil layer thickness?

From Example 1, the relevant parameters are: $q = 88\,290$ N/m$^2$; $\gamma = 19\,600$ N/m$^3$; and $r = 0{\cdot}75$ m.

In order to use eqn (18), the following values must be calculated

$q/(\gamma r) = 6{\cdot}0$ (from Example 1)
$\alpha/(\gamma r^2 \Omega) = 40\,000/(19\,600 \times (0{\cdot}75)^2 \times 0{\cdot}73) = 4{\cdot}97$

Hence, using eqn 18

$$H = 2 \times 0{\cdot}75 \times \ln \frac{6{\cdot}0 - 2}{4{\cdot}97 - 2} = 0{\cdot}44 \text{ m}$$

It is also possible to solve this problem using Table 3 or Fig. 11 which gives $H/r = 0{\cdot}6$ for $q/(\gamma r) = 6{\cdot}0$ and $\alpha/(\gamma r^2 \Omega) = 4{\cdot}97$ (see Fig. 13). Hence, $H = 0{\cdot}6 \times 0{\cdot}75 = 0{\cdot}45$ m.

**Determination of Maximum Void Size**

There is no simple equation giving the void size ($b$ or $r$) as a function of the other parameters. In order to determine the maximum void size that a given soil layer-geosynthetic system can bridge, it is necessary to solve eqn (16) by trial and error. To facilitate the process, a chart has been established (Fig. 14) by rewriting the two parts of eqn (17) in a dimensionless form as follows:

$$\frac{p}{\gamma H} = \frac{2(1 - e^{-0{\cdot}5H/b})}{H/b} + \frac{q}{\gamma H} e^{-0{\cdot}5H/b} \tag{19}$$

$$\frac{p}{\gamma H} = \frac{\alpha}{\gamma H^2 \Omega} \frac{H}{b} \tag{20}$$

In Fig. 14, eqn (19) is represented by a family of curves and eqn (20) is represented by a family of straight lines at 45°. For a given set of parameters, the abscissa of the intersection between the relevant curve

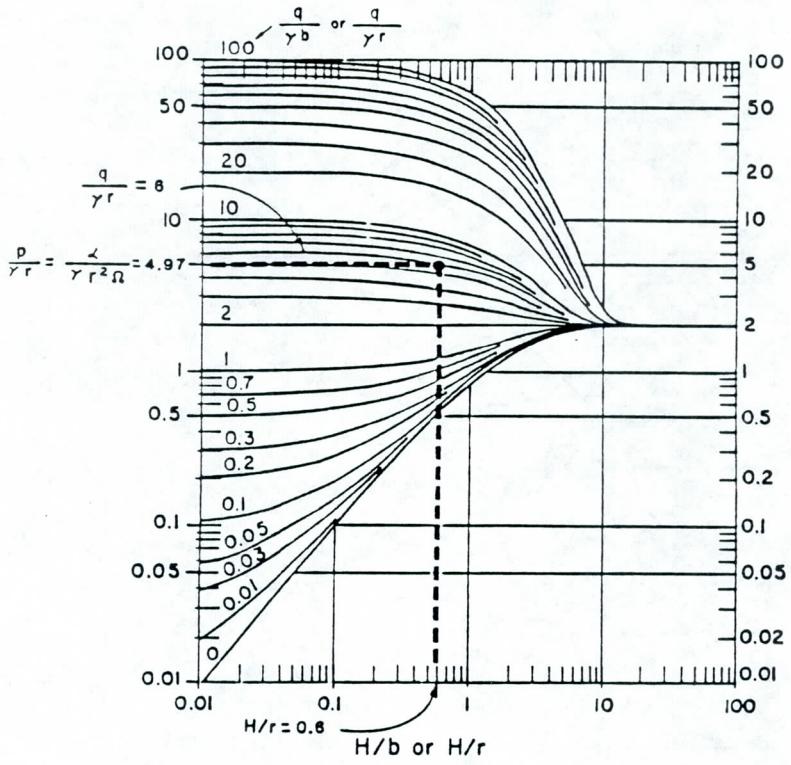


**Fig. 13.** Example of use of the chart given in Fig. 11.

and the relevant straight line gives the maximum value of the width, $b$, of an infinitely long void or the radius, $r$, of a circular void.

*Example 3.* This example is identical to Example 1, except that the radius of the void, $r$, is unknown, and the geosynthetic tension at a strain $\varepsilon = 10\%$ is known and is equal to 40 kN/m. What maximum void radius can be bridged by the considered soil-geosynthetic system?
From Example 1, the relevant parameters are: $q = 88\,290\,\text{N/m}^2$; $\gamma = 19\,600\,\text{N/m}^3$; and $H = 0{\cdot}45\,\text{m}$.

In order to use the chart given in Fig. 14, the following must be calculated

$$q/(\gamma H) = 88\,290/(19\,600 \times 0{\cdot}45) = 10{\cdot}0$$

$$\alpha/(\gamma H^2 \Omega) = 40\,000/(19\,600 \times (0{\cdot}45)^2 \times 0{\cdot}73) = 13{\cdot}8$$

(Note: $\Omega = 0{\cdot}73$ is obtained from Table 2 with $\varepsilon = 10\%$)

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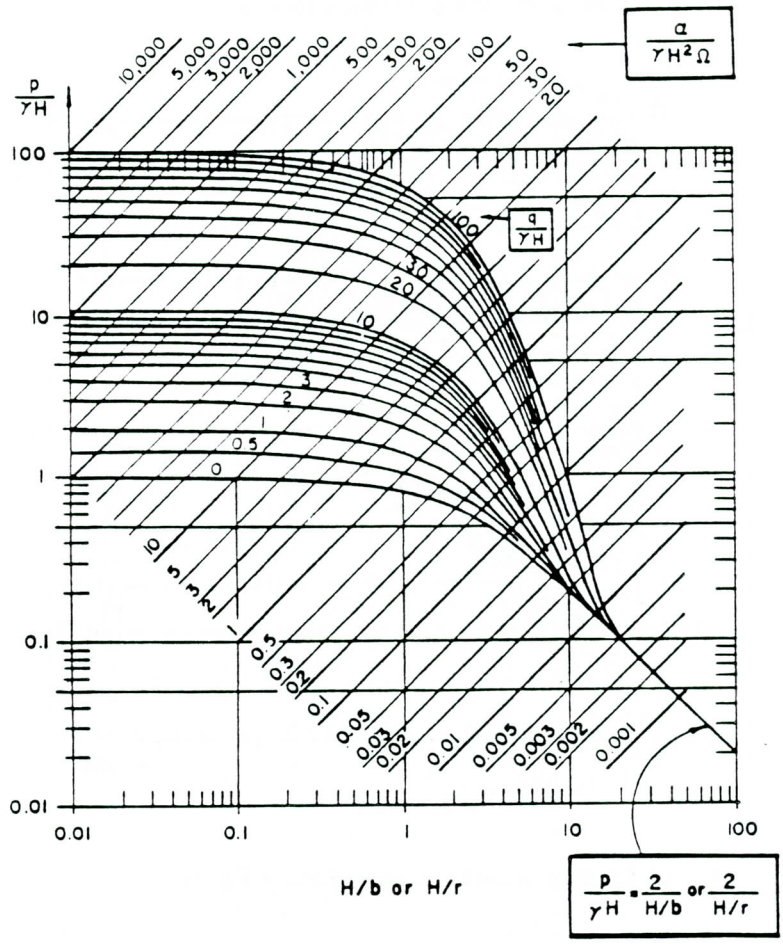


**Fig. 14.** Pressure on and tension in the geosynthetic. An example of use of this chart is given in Fig. 15. Notations: $p$ = pressure on the geosynthetic over the void area; $q$ = uniformly distributed normal stress applied on the top of the soil layer; $H$ = thickness of the soil layer; $\gamma$ = unit weight of soil; $b$ = width of the infinitely long void; $r$ = radius of the circular void; $\alpha$ = geosynthetic tension; and $\Omega$ = dimensionless factor given in Table 2 and Fig. 10. (Values of $p/(\gamma H)$ which were used to draw the curves in this figure can be found in Table 4.)

In Fig. 14, the curve related to $q/(\gamma H) = 10$ and the straight line at 45° related to $\alpha/(\gamma H^2\Omega) = 13{\cdot}8$ intersect at a point the abscissa of which is $H/r = 0{\cdot}6$ (see Fig. 15). Hence

$$r_{max} = 0{\cdot}45/0{\cdot}6 = 0{\cdot}75\ \text{m}$$

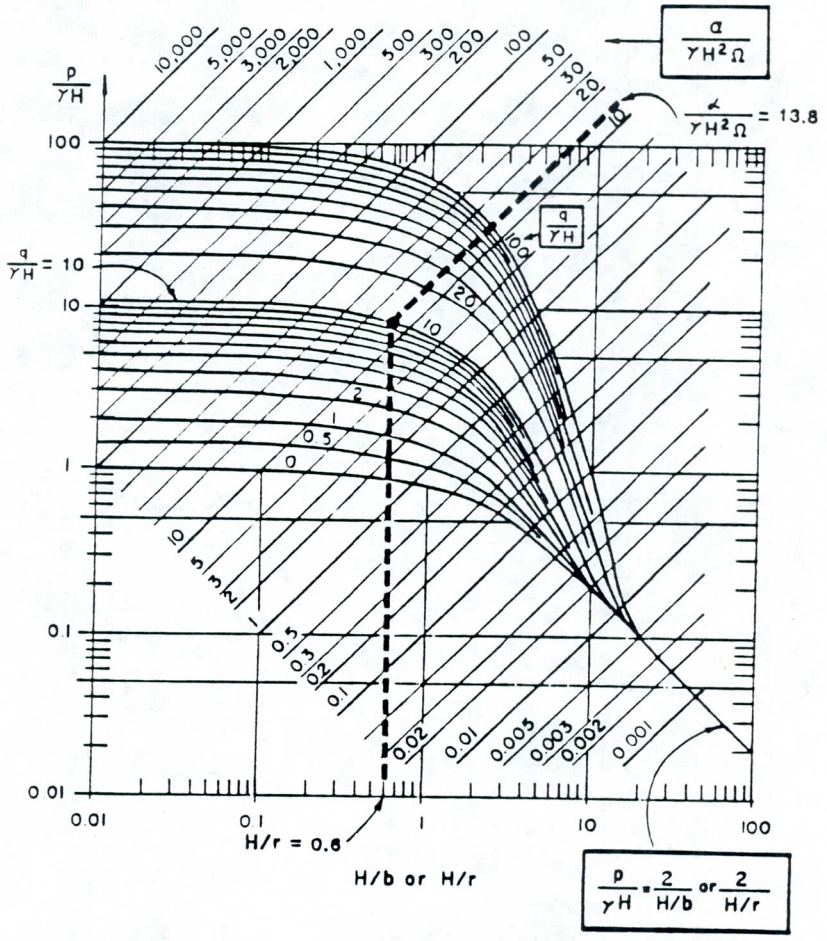


**Fig. 15.** Example of use of the chart given in Fig. 14.

## Determination of the Maximum Load

The relevant equation for an infinitely long void is

$$q = 2\gamma b + \left\{ \frac{[\alpha/(\gamma b^2 \Omega)] - 2}{e^{-0.5H/b}} \right\} \gamma b \tag{21}$$

The same equation can be used for a circular void by substituting $r$ for $b$.

The above equation can be used to solve problems that consist of determining the maximum uniform normal stress, $q$, which can be applied on the top of the soil layer, when all other parameters are given ($b$ or $r$, $\gamma$, $H$, $\alpha$, and $\varepsilon$). Alternatively, the charts given in Fig. 11 or 14 can be used, as well as Table 3 or 4.

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**TABLE 4**
Pressure on the Geosynthetic

| $q/(\gamma H)$ | $H/b$ or $H/r$ | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0 | 0·01 | 0·1 | 0·3 | 0·5 | 0·7 | 1·0 | 3·0 | 5·0 | 7·0 | 10·0 | 20·0 | ∞ |
| | (Values of $p/(\gamma H)$) | | | | | | | | | | | | |
| 0·0 | ∞ | 0·998 | 0·975 | 0·929 | 0·885 | 0·844 | 0·787 | 0·518 | 0·367 | 0·277 | 0·199 | 0·100 | 0 |
| 0·5 | ∞ | 1·495 | 1·451 | 1·359 | 1·274 | 1·196 | 1·090 | 0·629 | 0·408 | 0·292 | 0·202 | 0·100 | 0 |
| 1·0 | ∞ | 1·993 | 1·927 | 1·789 | 1·664 | 1·548 | 1·393 | 0·741 | 0·449 | 0·307 | 0·205 | 0·100 | 0 |
| 2·0 | ∞ | 2·988 | 2·878 | 2·650 | 2·442 | 2·253 | 2·000 | 0·964 | 0·531 | 0·337 | 0·212 | 0·100 | 0 |
| 3·0 | ∞ | 3·983 | 3·829 | 3·511 | 3·221 | 2·958 | 2·607 | 1·187 | 0·613 | 0·368 | 0·219 | 0·100 | 0 |
| 4·0 | ∞ | 4·978 | 4·780 | 4·371 | 4·000 | 3·663 | 3·213 | 1·410 | 0·696 | 0·398 | 0·226 | 0·100 | 0 |
| 5·0 | ∞ | 5·973 | 5·732 | 5·232 | 4·779 | 4·367 | 3·820 | 1·634 | 0·778 | 0·428 | 0·232 | 0·100 | 0 |
| 6·0 | ∞ | 6·968 | 6·683 | 6·093 | 5·558 | 5·072 | 4·426 | 1·857 | 0·860 | 0·458 | 0·239 | 0·100 | 0 |
| 7·0 | ∞ | 7·963 | 7·634 | 6·954 | 6·336 | 5·777 | 5·033 | 2·080 | 0·942 | 0·488 | 0·246 | 0·100 | 0 |
| 8·0 | ∞ | 8·958 | 8·585 | 7·814 | 7·115 | 6·481 | 5·639 | 2·303 | 1·024 | 0·519 | 0·253 | 0·100 | 0 |
| 9·0 | ∞ | 9·953 | 9·536 | 8·675 | 7·894 | 7·186 | 6·246 | 2·526 | 1·106 | 0·549 | 0·259 | 0·100 | 0 |
| 10 | ∞ | 10·948 | 10·488 | 9·536 | 8·673 | 7·891 | 6·852 | 2·749 | 1·188 | 0·579 | 0·266 | 0·100 | 0 |
| 15 | ∞ | 15·923 | 15·244 | 13·839 | 12·567 | 11·414 | 9·885 | 3·865 | 1·598 | 0·730 | 0·300 | 0·101 | 0 |
| 20 | ∞ | 20·898 | 20·000 | 18·143 | 16·461 | 14·938 | 12·918 | 4·981 | 2·009 | 0·881 | 0·333 | 0·101 | 0 |
| 25 | ∞ | 25·873 | 24·756 | 22·446 | 20·355 | 18·461 | 15·950 | 6·096 | 2·419 | 1·032 | 0·367 | 0·101 | 0 |
| 30 | ∞ | 30·848 | 29·512 | 26·750 | 24·249 | 21·984 | 18·983 | 7·212 | 2·830 | 1·183 | 0·401 | 0·101 | 0 |
| 40 | ∞ | 40·798 | 39·025 | 35·357 | 32·037 | 29·031 | 15·048 | 9·443 | 3·651 | 1·485 | 0·468 | 0·102 | 0 |
| 50 | ∞ | 50·748 | 48·537 | 43·964 | 39·825 | 36·078 | 31·113 | 11·674 | 4·471 | 1·787 | 0·536 | 0·102 | 0 |
| 60 | ∞ | 60·698 | 58·049 | 52·571 | 47·613 | 43·125 | 37·179 | 13·906 | 5·292 | 2·089 | 0·603 | 0·103 | 0 |
| 70 | ∞ | 70·648 | 67·561 | 61·178 | 55·401 | 50·172 | 43·244 | 16·137 | 6·113 | 2·391 | 0·670 | 0·103 | 0 |
| 80 | ∞ | 80·599 | 77·074 | 69·786 | 63·189 | 57·219 | 49·309 | 18·368 | 6·934 | 2·693 | 0·738 | 0·104 | 0 |
| 90 | ∞ | 90·549 | 86·586 | 78·392 | 70·977 | 64·266 | 55·375 | 20·600 | 7·755 | 2·995 | 0·805 | 0·104 | 0 |
| 100 | ∞ | 100·499 | 96·098 | 86·999 | 78·765 | 71·313 | 61·440 | 22·831 | 8·576 | 3·297 | 0·872 | 0·105 | 0 |

This table gives $p/(\gamma H)$. Notation: $p$ = pressure on the geosynthetic over the void area; $q$ = uniformly distributed normal stress applied on the top of the soil layer, $H$ = thickness of the soil layer, $\gamma$ = unit weight of the soil in the soil layer, $b$ = width of the infinitely long void, and $r$ = radius of the circular void. Note that values of $p/(\gamma H)$ are equal to: $1 + q/(\gamma H)$ if $H/b = 0$ or $H/r = 0$, and $2b/H$ or $2r/H$ if $H/b > 20$ or $H/r > 20$. (See the chart given in Fig. 14.)

*Example 4.* This example is identical to Example 1, except that the stress on top of the soil layer, $q$, is unknown, and the geosynthetic tension at strain $\varepsilon = 10\%$ is known and is equal to 40 kN/m. What maximum stress on top of the soil layer can be supported by the soil-geosynthetic system?
From Example 1, the relevant parameters are: $H = 0{\cdot}45$ m; $r = 0{\cdot}75$ m; and $\gamma = 19\,600$ N/m$^3$.
In order to use eqn (21), the value of $\Omega$ must be obtained first from Table 2

$$\Omega = 0{\cdot}73 \quad \text{for } \varepsilon = 10\%.$$

Then, eqn (21) is used as follows

$$q = 2 \times 19\,600 \times 0{\cdot}75 + \left\{ \frac{[40\,000/(19\,600 \times (0{\cdot}75)^2 \times 0{\cdot}73)] - 2}{e^{-0{\cdot}5 \times 0{\cdot}45/0{\cdot}75}} \right\} 19\,600 \times 0{\cdot}75$$

$$= 88\,334 \text{ N/m}^2$$

The problem can also be solved using charts and tables. To use Table 3 or the chart given in Fig. 11, the following must be calculated:

$$H/r = 0{\cdot}45/0{\cdot}75 = 0{\cdot}6$$

$$\alpha/(\gamma r^2 \Omega) = 40\,000/(19\,600 \times (0{\cdot}75)^2 \times 0{\cdot}73) = 4{\cdot}97$$

With $H/r = 0{\cdot}6$ and $\alpha/(\gamma r^2 \Omega) = 4{\cdot}97$, Table 3 or the chart given in Fig. 11 show that $q/(\gamma r) = 6$ (see Fig. 13). Therefore

$$q = 6 \times 19\,600 \times 0{\cdot}75 = 88\,200 \text{ N/m}^2 = 88 \text{ kN/m}^2$$

To use the chart given in Fig. 14, the following must be calculated

$$\alpha/(\gamma H^2 \Omega) = 40\,000/(19\,600 \times (0{\cdot}45)^2 \times 0{\cdot}73) = 13{\cdot}8$$

With $H/r = 0{\cdot}6$ and $\alpha/(\gamma H^2 \Omega) = 13{\cdot}8$, the chart given in Fig. 14 shows that $q/(\gamma H) = 10$ (see Fig. 15). Therefore

$$q = 10 \times 19\,600 \times 0{\cdot}45 = 88\,200 \text{ N/m}^2 = 88 \text{ kN/m}^2$$

## DISCUSSION OF SPECIAL PROBLEMS

### Anisotropic Geosynthetic

A geosynthetic is isotropic regarding a given characteristic when this characteristic has the same value in all directions. In this paper, a geosynthetic will be considered isotropic when it has the same tension–strain

$H/r > 20$ (See the chart given in Fig. 14)

curve in all directions. This requirement is fulfilled by some nonwoven geotextiles. Woven geotextiles and biaxial geogrids are stronger in two directions ('principal directions') than in the others and, therefore, they are anisotropic. However, we assume that the design method presented in this paper can be used with woven geotextiles and biaxial geogrids that have the same tensile characteristics in the two principal directions (i.e. in the design, these materials are considered isotropic).

Special precautions must be taken when using the design method presented in this paper for geosynthetics that cannot be considered isotropic, as discussed below.

*Infinitely Long Void*

In the case of an infinitely long void, no geosynthetic tension is required in the direction of the length of the void (according to the plane-strain model which corresponds to an infinitely long void). Therefore, the value of $\alpha$ to be used in the equations, tables, and charts related to the infinitely long void is the geosynthetic tension in the direction of the width of the void for the considered design strain. However, some strength is required lengthwise in places where the actual situation departs from a pure plane-strain situation (for instance near the end of the void).

*Circular Void*

In the case of a circular void, the tensioned membrane equation (eqn (15)) is valid only if the geosynthetic has isotropic tensile characteristics. For practical purposes, eqn (15), and other equations as well as tables and charts related to circular voids, can be used for woven geotextiles and biaxial geogrids that have the same tension–strain curve *in the two principal directions* (instead of *in all directions* for a truly isotropic material). For woven geotextiles and biaxial geogrids that have different tensile characteristics in the two principal directions, two cases can be considered, depending on the ratio between the geosynthetic tensions at the design strain in the weak and the strong directions: (i) if the ratio is more than 0·5, $\alpha$ should be taken equal to the tension in the weak direction; and (ii) if the ratio is less than 0·5, $\alpha$ should be taken equal to half the tension in the strong direction.

The rationale for the above recommendation is as follows. There are two conservative approaches and the less conservative, which is closer to reality, should be selected.

The first conservative approach consists of designing with an isotropic geosynthetic weaker than the considered anisotropic geosynthetic. This is achieved by taking the geosynthetic strength in all directions equal to the

strength in the weak direction, $\alpha_{weak}$. Equation (15) thus gives for the pressure which can be carried by the geosynthetic

$$p_1 = \frac{\alpha_{weak}}{r\Omega} \tag{22}$$

The second conservative approach consists of designing with: (i) a void larger than the circular void by replacing the circular void by an infinitely long void with a width, $b$, equal to the diameter, $2r$, of the circular void; and (ii) a geosynthetic weaker than the considered anisotropic geosynthetic by neglecting the tensile strength in the weak direction ($\alpha_{weak} = 0$). Equation (15) thus gives for the pressure which can be carried by the geosynthetic

$$p_2 = \frac{\alpha_{strong}}{b\Omega} = \frac{\alpha_{strong}}{2r\Omega} \tag{23}$$

To compare $p_1$ and $p_2$, it is important to note that the values of $\Omega$ in eqns (22) and (23) are identical because they are both determined for $y/(2r)$, according to Table 2. Therefore, the comparison between $p_1$ and $p_2$ boils down to a comparison between $\alpha_{weak}$ and $0{\cdot}5\ \alpha_{strong}$.

It appears that

$$p_1 > p_2 \quad \text{if} \quad \alpha_{weak} > 0{\cdot}5\alpha_{strong}$$

$$p_1 < p_2 \quad \text{if} \quad \alpha_{weak} < 0{\cdot}5\alpha_{strong}$$

hence the above recommendation.

There is another consideration when an anisotropic geosynthetic is used over a circular void. The complex pattern of strains in the geosynthetic resulting from different tensions in different directions may have a detrimental effect on the behavior of the geosynthetic. Therefore, it is recommended that for holes which can be modeled as circular, one of the following solutions be adopted: (i) an isotropic geosynthetic (only some nonwoven geotextiles are isotropic but usually they do not have adequate tensile characteristics for this application); or (ii) a 'practically isotropic' geosynthetic (such as a woven geotextile or a biaxial geogrid having similar tension–strain curves in the two principal directions); or (iii) two perpendicularly orientated layers of the same anisotropic geosynthetic.

### Geosynthetic in Contact with Void Bottom

In some cases, the geosynthetic elongates to the point that it comes in contact with the bottom of the void (Fig. 1(c)); the geosynthetic deflection

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is then equal to the void depth ($y = D$). In design, these cases correspond to a calculated geosynthetic deflection greater than or equal to the void depth ($y \geqq D$). Usually, the design is complete when it is found that $y \geqq D$. However, it may be of interest to *determine the pressure actually transmitted to the bottom of the void.* This pressure is obtained by subtracting the pressure inducing geosynthetic tension (which results from the tensioned membrane effect) from the pressure exerted by the soil layer on the geosynthetic.

In the case of an infinitely long void, the following equation can be obtained by subtracting the pressure given by eqn (15) from the pressure given by eqn (10)

$$p_b = 2\gamma b(1 - e^{-0.5H/b}) + q\,e^{-0.5H/b} - \frac{\alpha}{b\Omega} \tag{24}$$

where: $p_b$ = pressure transmitted to the bottom of the void; $\gamma$ = unit weight of the soil (in the soil layer above the geosynthetic); $b$ = width of the infinitely long void; $H$ = soil layer thickness; $q$ = uniformly distributed normal stress applied on the top of the soil layer; $\alpha$ = geosynthetic tension corresponding to the geosynthetic strain, $\varepsilon$, when the geosynthetic is in contact with the bottom of the void (i.e., $\varepsilon$ corresponding to a deflection $y = D$ in Table 2); $\Omega$ = dimensionless factor given in Table 2 as a function of $\varepsilon$ or $y$; and $y$ = geosynthetic deflection, which, in this case, is equal to $D$; and $D$ = depth of the void. Basic SI units are: $p_b$ (N/m$^2$), $\gamma$ (N/m$^3$), $b$ (m), $H$ (m), $q$ (N/m$^2$), $\alpha$ (N/m), $y$ (m), and $D$ (m); $\Omega$ is dimensionless. Note that eqn (24) assumes that the shape of the bottom of the void is approximately cylindrical with a circular cross section, so the geosynthetic will come in contact with all points on the surface of the void at the same time. If this were not the case, portions of the geosynthetic which come in contact with the bottom of the void last would elongate more than the others.

The same equation can be used for a circular void by substituting $r$ for $b$, with $r$ = radius of the circular void.

If a negative value were obtained for $p_b$ when using the above equation, it would mean that the load on the geosynthetic is not large enough to force the geosynthetic to come in contact with the bottom of the void.

*Example 5.* This example is identical to Example 1 except that: (i) the void is not bottomless but has a depth $D = 0{\cdot}2$ m; and (ii) the geosynthetic tension–strain curve is assumed to be a straight line between the origin and a tension $\alpha = 40$ kN/m for a strain $\varepsilon = 10\%$. What is the stress transmitted to the bottom of the hole?

From Example 1, the relevant parameters are: $r = 0{\cdot}75$ m; $H = 0{\cdot}45$ m; $\gamma = 19\,600$ N/m$^3$; and $q = 88\,290$ N/m$^2$.

First, the approximate value of the average strain of the geosynthetic when it is in contact with the bottom of the void (assumed spherical) must be determined using Table 2 with $y$ (geosynthetic deflection) = $D$ (void depth)

$$y/2r = D/2r = 0{\cdot}2/(2 \times 0{\cdot}75) = 0{\cdot}133$$

Hence, interpolating in Table 2, $\varepsilon = 4{\cdot}65\%$ and $\Omega = 1{\cdot}01$.

Then, the geosynthetic tension corresponding to a 4·65% geosynthetic strain can be calculated as follows:

$$\alpha = 40\,000 \times 4{\cdot}65/10 = 18\,600 \text{ N/m}$$

Finally, eqn (24) can be used with the values $H/r = 0{\cdot}6$ and $q = 88\,290$ N/m$^2$ determined in Example 1. This equation gives the stress transmitted to the bottom of the void as follows

$$p_b = 2 \times 19\,600 \times 0{\cdot}75(1 - e^{-0{\cdot}3}) + 88\,290\, e^{-0{\cdot}3} - \frac{18\,600}{0{\cdot}75 \times 1{\cdot}01}$$

$$= 73\,029 - 24\,554 = 48\,475 \text{ N/m}^2 = 48{\cdot}5 \text{ kN/m}^2$$

Therefore, this design example can be summarized as follows:

- A stress of 88·3 kN/m$^2$ is applied on top of the soil layer.
- As a result of soil arching, the soil layer transmits only a stress of 73 kN/m$^2$ to the top of the geosynthetic.
- As a result of the tensioned membrane effect, the geosynthetic supports 24·5 kN/m$^2$.
- The remainder, 48·5 kN/m$^2$, is transmitted to the bottom of the void.

It should be noted that, if the depth of the void had been $D = 0{\cdot}3$ m, the strain of the geosynthetic would have been 10% and the last term of the above equation would have been

$$\frac{\alpha}{r\Omega} = \frac{40\,000}{0{\cdot}75 \times 0{\cdot}73} = 73\,059 \text{ N/m}^2$$

Hence, $p_b = 0$. In this case, Example 5 becomes identical to Example 1.

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**Influence of Soil Layer Thickness**

The influence of the thickness of the soil layer is illustrated in Fig. 11. Three cases can be considered:

(1) *Large Applied Stress.* If the applied stress, $q$, is large (i.e. $q > 2\gamma b$ or $2\gamma r$), the pressure, $p$, on the geosynthetic and consequently the required geosynthetic tension, $\alpha$, decrease towards a limit when the soil layer thickness increases. In this case, *it is beneficial to increase the thickness of the soil layer.* For each particular situation, the amount by which the thickness should be increased can be determined using the chart given in Fig. 11 or Table 3. The chart and table show that it would be useless to increase the soil layer thickness beyond a limiting value of $H = 20b$ or $20r$.

(2) *Small Applied Stress.* If the applied stress, $q$, is small (i.e. $q < 2\gamma b$ or $2\gamma r$), the pressure, $p$, on the geosynthetic and consequently the required geosynthetic tension, $\alpha$, increase toward a limit when the soil thickness increases. In this case, from the perspective of the design of the geosynthetic, *it is detrimental to increase the thickness of the soil layer.* (This is because the added load due to soil weight is not fully compensated by the effect of soil arching.)

(3) *Limit Applied Stress.* If the applied stress, $q$, equals the limit (i.e. $q = 2\gamma b$ or $2\gamma r$), the pressure, $p$, on the geosynthetic remains constant and equal to $q$, regardless of the soil layer thickness.

The limit values for $p$ and $\alpha$ are independent of the applied stress, $q$. The limit value for the pressure on the geosynthetic is

$$p_{lim} = 2\gamma b \quad \text{for an infinitely long void} \tag{25}$$

The limit value for the required geosynthetic tension is

$$\alpha_{lim} = 2\gamma b^2 \Omega \quad \text{for an infinitely long void} \tag{26}$$

Equations (25) and (26) can be used for a circular void by substituting $r$ for $b$.

**Comparison with Tensioned Membrane Theory**

In the past, the tensioned membrane theory has been used alone to evaluate the required tensile characteristics of a geosynthetic located beneath a soil layer and bridging a void. This method neglects arching in

the soil layer and is, therefore, conservative. This conservativeness can be evaluated by comparing the pressure on the geosynthetic over the void area, $p$, calculated taking soil arching into account to the following value obtained by neglecting soil arching

$$p_0 = \gamma H + q \tag{27}$$

where: $p_0$ = pressure on the geosynthetic over the void area neglecting soil arching; $\gamma$ = unit weight of the soil in the soil layer; $H$ = thickness of the soil layer; and $q$ = uniformly distributed normal stress applied on the top of the soil layer. Basic SI units are: $p_0$ (N/m$^2$), $\gamma$ (N/m$^3$), $H$ (m), and $q$ (N/m$^2$).

The pressure, $p$, obtained taking soil arching into account is given by eqn (10).

Values of $p/p_0$ are given in Table 5 and Fig. 16. It appears that neglecting soil arching is conservative. However, when the soil thickness, $H$, is large

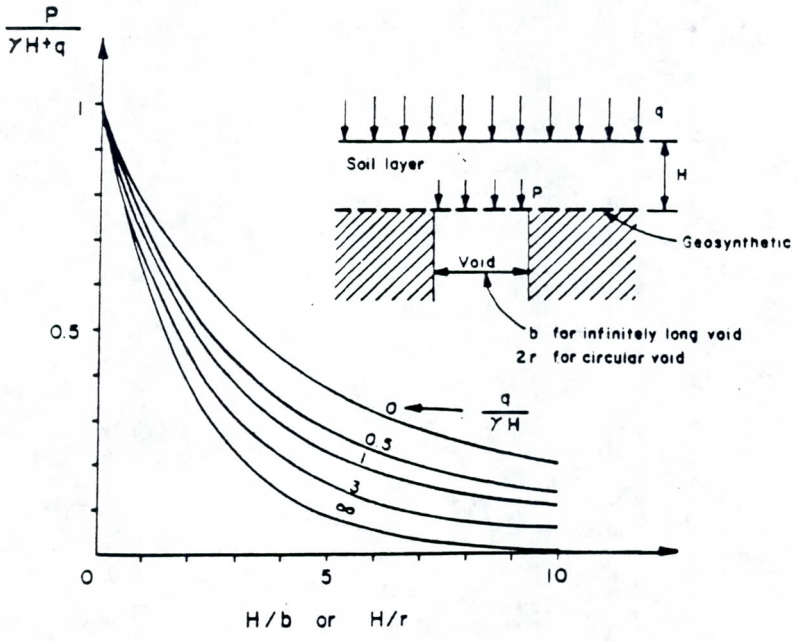


**Fig. 16.** Effectiveness of soil arching. The curves give the ratio between the pressure, $p$, on the geosynthetic over the void area, calculated taking soil arching into account, and the pressure $p_0 = \gamma H + q$ obtained by neglecting soil arching. The values of $p/p_0$ used to plot the curve can be found in Table 5.

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**TABLE 5**
Effectiveness of Soil Arching

| | *H/b or H/r* | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $q/(\gamma H)$ | 0 | 0·3 | 0·6 | 1 | 1·5 | 2 | 2·3 | 3 | 4 | 5 | 10 | 20 | ∞ |
| | *(Values of $p/p_0$)* | | | | | | | | | | | | |
| 0 | 1 | 0·929 | 0·864 | 0·787 | 0·704 | 0·632 | 0·571 | 0·518 | 0·432 | 0·367 | 0·199 | 0·100 | 0 |
| 0·5 | 1 | 0·906 | 0·823 | 0·727 | 0·626 | 0·544 | 0·476 | 0·420 | 0·333 | 0·272 | 0·135 | 0·067 | 0 |
| 1 | 1 | 0·895 | 0·802 | 0·697 | 0·588 | 0·500 | 0·429 | 0·371 | 0·284 | 0·225 | 0·103 | 0·050 | 0 |
| 2 | 1 | 0·883 | 0·782 | 0·667 | 0·549 | 0·456 | 0·381 | 0·321 | 0·234 | 0·177 | 0·071 | 0·033 | 0 |
| 3 | 1 | 0·878 | 0·772 | 0·652 | 0·530 | 0·434 | 0·358 | 0·297 | 0·210 | 0·153 | 0·055 | 0·025 | 0 |
| 5 | 1 | 0·872 | 0·761 | 0·637 | 0·511 | 0·412 | 0·334 | 0·272 | 0·185 | 0·130 | 0·039 | 0·017 | 0 |
| 10 | 1 | 0·867 | 0·752 | 0·623 | 0·493 | 0·392 | 0·312 | 0·250 | 0·162 | 0·108 | 0·024 | 0·009 | 0 |
| 20 | 1 | 0·864 | 0·747 | 0·615 | 0·483 | 0·380 | 0·300 | 0·237 | 0·149 | 0·096 | 0·016 | 0·005 | 0 |
| ∞ | 1 | 0·861 | 0·741 | 0·607 | 0·472 | 0·368 | 0·287 | 0·223 | 0·135 | 0·082 | 0·007 | 0·000 | 0 |

This table gives the ratio of the pressure on the geosynthetic over the void area calculated taking arching into account ($p$) or neglecting arching ($p_0$). The value of $p$ is given by eqn (10). The value of $p_0$ is given by eqn (27). Notation: $q$ = uniformly distributed stress applied on the top of the soil layer; $\gamma$ = unit weight of the soil in the soil layer; $H$ = soil layer thickness; $b$ = width of an infinitely long void; and $r$ = radius of circular void. (See also Fig. 16.)

## REFERENCES


1. Loudière, D. & Perrin, J., Désordres dans les revêtements minces de bassin. In *Colloque sur l'étanchéité superficielle des bassins, barrages, et canaux*, Vol. 1, Paris, Feb. 1983. CEMAGREF, Antony, France, pp. 147–52.
2. Giroud, J. P., & Goldstein, J. S., Geomembrane liner design. *Waste Age* (Sept. 1982) 27–30.
3. Giroud, J. P., *Geotextiles and Geomembranes, Definitions, Properties and Design*. I.F.A.I. Publisher, St Paul, MN, 1984, 325 pp.
4. Bonaparte, R. & Berg, R. R., The use of geosynthetics to support roadways over sinkhole prone areas. In *Proceedings of the Second Multidisciplinary Conference on Sinkholes and the Environmental Impacts of Karst*, Orlando, FL, Feb. 1987. Balkema, Rotterdam, pp. 437–45.
5. Giroud, J. P. & Fluet, J. E., Jr, Quality assurance of geosynthetic lining systems. *Geotextiles and Geomembranes*, **3** (1986) 249–87.
6. Tisserand, C., Le réservoir de Guazza: un grand bassin à membrane non protégée. *Colloque sur l'étanchéité superficielle des bassins, barrages et canaux*, Vol. 1, Paris, Feb. 1983. CEMAGREF, Antony, France, pp. 87–9.
7. Giroud, J. P., Designing with geotextiles. *Matériaux et Constructions*, **14** (1981), 257–72.
8. Giroud, J. P., Design of geotextiles associated with geomembranes. In *Proceedings of the Second International Conference on Geotextiles*, Vol. 1, Las Vegas, USA, Aug. 1982, pp. 37–42.
9. Terzaghi, K., *Theoretical Soil Mechanics*. John Wiley and Sons, Inc., New York, 1943, 510 pp.
10. Kezdi, A., Lateral earth pressure. In *Foundation Engineering Handbook*, ed. H. F. Winterkorn & H. Y. Fang. Van Nostrand Reinhold, New York, 1975, pp. 197–220.
11. Handy, R. L., The arch in soil arching. *Journal of Geotechnical Engineering*, ASCE, **111** (1985) 302–18.
12. Jaky, J., The coefficient of earth pressure at rest. *Journal for Society of Hungarian Architects and Engineers*, Budapest, Oct. 1944, pp. 355–8 (in Hungarian).
13. Giroud, J. P., Bonaparte, R., Beech, J. F. & Gross, B. A., Load carrying capacity of a soil layer supported by a geosynthetic located on a void. In *Proceedings of the International Geotechnical Symposium on Theory and Practice of Earth Reinforcement*, Fukuoka/Kyushu, Japan, Oct. 1988. Balkema, Rotterdam, pp. 185–90.


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compared to the width or radius of the void, neglecting soil arching is over-conservative.

## CONCLUSION

This paper has presented an approach to the design of soil layer-geosynthetic systems overlying voids. The design approach superimposes arching theory for the soil layer with tensioned membrane theory for the geosynthetic. The analysis presented in this paper shows that neglecting soil arching would be over-conservative in many instances. The paper presents equations, tables, and charts that make it easy to perform design analyses for a range of possible field situations.

The analysis shows that the thickness of the soil layer associated with the geosynthetic plays a significant role. In contrast, the soil mechanical properties do not. It should not be inferred, however, that any soil will provide the same degree of arching. The equations used to prepare the tables and charts assume that the friction angle of the soil is at least 20°. Granular soils virtually always meet this condition. However, they should be well compacted to ensure arching because loose granular soils tend to contract when they are sheared or vibrated, which may destroy the arch.

Further refinements of the method presented herein can be considered. For instance, it is possible that the degree of soil arching (i.e. the amount of soil shear strength mobilized) depends on the geosynthetic strain, whereas the method presented in this paper does not consider the concept of degree of soil arching. Also, the method could be expanded to include cohesive soils, and could be refined to take into account elongation of the geosynthetic in the anchorage zone. Lastly, the method could be expanded to consider a system of regularly spaced voids.

In spite of its limitations, the method presented in this paper is believed to be a useful tool for engineers designing soil-geosynthetic systems resting on subgrades where voids may develop.

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