START

0013910

Department of Energy

Richland Operations Office
P.O. Box 550
Richland, Washington 99352

9102148

APR 23 1991

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CORRESPONDENCE CONTROL

91-WOB-132

Mr. Timothy L. Nord
Hanford Project Manager
Washington State Department of Ecology
Mail Stop PV-11
Olympia, Washington 98504-8711

BEST AVAILABLE COPY

Dear Mr. Nord:

RESPONSE TO APRIL 3, 1991 REQUEST, REGARDING THE LIQUID EFFLUENT RETENTION FACILITY (LERF)

References: 1) Letter, S. H. Wisness, U.S. Department of Energy-Richland Operations Office to T. L. Nord, Ecology, "Engineering Change Notice W-105-72 on the Liquid Effluent Retention Facility," dated April 11, 1991. 13573

2) Letter, S. H. Wisness, U.S. Department of Energy-Richland Operations Office to T. L. Nord, Ecology, "Liquid Effluent Retention Facility Construction Quality Assurance Plan," dated April 8, 1991. no

3) Letter, S. H. Wisness, U.S. Department of Energy-Richland Operations Office to T. L. Nord, Ecology, "Transmittal of Soil/Bentonite Permeability Final Report and Additional Information Requested," dated April 5, 1991. no

4) Letter, T. L. Nord, Ecology, to S. H. Wisness, U.S. Department of Energy-Richland Operations Office, "Liquid Effluent Retention Facility (LERF) Basin Construction," dated April 3, 1991. 13488

Reference 4 requested specific information prior to start of the Liquid Effluent Retention Facility (LERF) basin construction. The majority of this information has already been provided to Ecology as indicated below. The information which has not yet been provided is attached.

1. <u>Construction Quality Control (CQA) Documentation</u>

A copy of the CQA document was provided to Gary Anderson on April 5, 1991, and transmitted by U.S. Department of Energy, Richland Operations Office (DOE-RL) to you on April 8, 1991, via Reference 2.0.

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2. <u>Revised C4 Specification</u>

   The revised specification was telefaxed to Gary Anderson of your staff on April 5, 1991, and transmitted to Ecology on April 11, 1991, via Reference 2.0.

3. <u>Revised C2 Specification</u>

   The revised C2 Specification was provided to your staff at the Unit Manager's on February 1, 1991, as documented in the respective meeting minutes.

4. <u>Contractor's Implementation Plan</u>

   This document is identified as the contractor's installation document, Submittal 12D, as the "Contractor's Implementation Plan". This document was given to Gary Anderson on April 5, 1991 and was transmitted on April 5, 1991, via Reference 3.

5. <u>Final Chen Northern Report</u>

   A copy was given to Gary Anderson on April 5, 1991, and transmitted by DOE-RL to Ecology on April 5, 1991, via Reference 3.

6. <u>Independent Assessment for Surface Impoundment</u>

   An assessment certified by a professional engineer that the surface impoundments' dikes have sufficient structural integrity to withstand the stress of the pressure exerted by the types and amounts of wastes placed in them and will not fail due to scouring and piping is attached (Attachment 1) per your request.

7. <u>Assure 2 Foot Thickness Under Sump</u>

   The drawings which have been provided to you do not specifically call out (requirements) for thickness under the sump. However, the information can be derived by a "circuitous" route by examining other drawings and an Non Conformance Report with survey data. For clarification, an Engineering Change Notice is attached that adds a note to the drawing to maintain three (3) feet under the sump (Attachment 2).

8. M-26-04 Milestone at Risk

Your letter indicated that M-26-04 had been identified at risk during a Unit Manager's Meeting which took place on March 26, 1991. The schedule identified to meet this milestone is a success orientated schedule and contains many areas of risk. However, to this date, WHC has not formally identified this particular milestone as being at risk. Attachment 3 is a hard copy excerpt of the presentation.

I believe this satisfies all requests you have identified to DOE-RL to date regarding LERF. If you have any questions please contact Mr. Cliff Clark of this office on 376-9333.

Sincerely,

Steven H. Wisness,
Hanford Project Manager
Tri-Party Agreement

Attachments

cc w/att:
P. Stasch, Ecology
G. Anderson, Ecology
T. Michelena, Ecology
P. Day, EPA
~~T. B. Veneziano~~, WHC
D. E. Kelley, WHC

Attachment L-0

KAISER ENGINEERS HANFORD

INTEROFFICE MEMORANDUM

TO S. L. Peterson

DATE March 29, 1991

FROM E. A. Goakey

COPIES TO

JOB NO. N/A

SUBJECT RESPONSE TO LETTER OF INSTRUCTION #55, ISSUE 11 AND 16

Please accept this letter as certification that the dike portion of the basins has been designed for structural integrity to prevent failure without dependence on any liner system included in the surface impoundment construction. The dike will withstand the stress of the pressure exerted by the types and amounts of wastes in the impoundment. The dike has a safety factor greater than 3 against failure by sliding and the top of sides are stabilized with a 3 inch layer of crushed gravel to prevent water and wind erosion of the surfaces.

Calculations are attached.

EAG:sme
Attachments

EDGAR A. GOAKEY
STATE OF WASHINGTON
9813
REGISTERED PROFESSIONAL ENGINEER

Edgar A. Goakey 3/29/91

29

9112170720

KAISER ENGINEERS
HANFORD

Transmittal No. TR-W-105-PM-116

DOCUMENT TRANSMITTAL

Date: April 19, 1991

To: A. G. Lassila

From: S. L. Petersen

Project/Work Order Number: W-105

Project/Work Order Title: 242-A Evaporator Interim Retention Basins

Subject: Additional Information on Part B Permit Application

No. of Copies	Company and Distribution	Mailing Address
DOE		
1	A. G. Lassila	A5-18
WHC		
0	L. R. Tollbom	R3-30
KEH		
1	S. L. Petersen	E6-50
1	Engineering Document Control	E6-52
0	Transmittal Clerk	E6-52

Attached Are	Purpose	Comments	Please
[] Prints	[X] Information	[] Preliminary	[] Comment
[] Specifications	[] Action	[] Unchecked	[] Approve
[] Travelers	[] Signature	[] Checked	[] Destroy Previous Issue
[X] Appr. Data	[] Update	[] Final	[] Return Previous Issue
[] Forms	[] Review	[] Approved	[] Note Revision
[] Library Material	[]	[] Working Copies	[] Note Holds
[] Procedures	[]	[] Controlled Copies	[] File
[] Other:		[] Other:	

Document Numbers, Titles, and/or Comments

Attached is further information requested by WDOE related to WAC-173-303-650 (i.e. Scove and Piping Potential, Soil/Bentonite Liner Permeability, Seismic Stability, etc).

SLP/sjm

KEH 0001.00 (12/89) (EF) KEF003
Document Transmittal

9112172072

Chen-Northern, Inc.

April 18, 1991

Kaiser Engineers Hanford Company
P.O. Box 888
Richland, Washington 99352

ATTENTION: Mr. Steve Peterson

SUBJECT: Additional Information for Project W-105
Part B Permit Application
Compliance with Washington Annotated Codes,
(WAC) 173-303-650

Gentlemen:

In accordance with your request of April 17, 1991, we have reviewed previously transmitted information and have prepared additional information regarding compliance of the W-105 geotechnical design with WAC 173-303-650. The new information includes:

- Scour and piping potential for the soil-bentonite liner.

We have reviewed the following information previously transmitted to Kaiser Engineers Hanford Company (KEH):

- Soil-Bentonite Liner Permeability (Chen-Northern letter of March 11, 1991 to KEH).
- Shear strength, dike stability, settlement, subsidence, and uplift stresses on the gravel dikes and soil-bentonite liner (Chen-Northern letter of March 26, 1991 to KEH). In these analyses, each basin liner was assumed to consist of two High Density Polyethylene liners and a tertiary soil-bentonite system. The soil-bentonite liner was considered to be part of the dikes in regard to structural integrity.
- Piping and scour potential of the gravel dikes (Chen-Northern letter of April 10, 1991 to KEH).

The results of our review and recent analysis indicates that:

1. The W-105 dikes, including the gravel basins and soil-bentonite liner, are expected to withstand the hydraulic pressures exerted by the liquid waste in the impoundment.

2. The geotechnical design of the W-105 project, including the factors listed above, complies with the requirements set forth in WAC 173-303-650.

If you have any questions regarding this letter, or if we can be of further service, please contact us.

Respectfully Submitted,
CHEN-NORTHERN, INC.

Brian J. Williams, P.G.
Geotechnical Engineer

Dee J. Burrie, P.E.
Division Manager

REGISTERED PROFESSIONAL ENGINEER

9112170723

PIPING AND SCOUR

Piping through a soil-bentonite liner may occur when the liner is penetrated by some conduit (hole or leakage path), and water is allowed to pass unimpeded through the conduit. In the design of the W-105 soil-bentonite liner, a non-woven geotextile (Polyfelt TS 750 ®) was specified for placement between the gravel dike materials and the soil-bentonite liner. Our analysis indicates that the geotextile will perform as an effective retention barrier, thus minimizing the potential for soil-bentonite liner piping.

Scour of a soil-bentonite liner is a function of flow type and velocity of flow adjacent to the soil-bentonite liner. Under normal operating conditions of hydrostatic pressure, a pinhole-type or seam-type leak is the normal mode of leakage. This type of leakage is typically low velocity and low volume. In this case, scour is not expected to occur. Scour of the soil-bentonite liner is only expected to occur under conditions of high velocity turbulent flow, such as a hose directed at unprotected section of the soil-bentonite, or a large-scale pipe failure leaking high-pressure fluid directly onto the soil liner. Since no piping penetrates the soil-bentonite liner, this situation is not expected to occur.

9112130724

REPORT
OF
PERMEABILITY TESTING

TO
KAISER ENGINEERS HANFORD COMPANY
RICHLAND, WASHINGTON

KEH W-105 SOIL-BENTONITE TEST FILL
PERMEABILITY TESTING

PROJECT NO. 86-1905

PREPARED
BY
CHEN-NORTHERN, INC.
CONSULTING GEOTECHNICAL ENGINEERS

TRI-CITIES, WASHINGTON

MARCH, 1991

Chen-Northern, Inc.

March 11, 1991

Kaiser Engineers Hanford Company
P.O. Box 888
Richland, Washington 99352

ATTENTION: Mr. Steve Peterson,
KEH W-105 Project Manager

SUBJECT: Final Report of Soil-Bentonite Liner Test Fill Permeability Testing; Project W-105

Gentlemen:

In accordance with our agreement, we have conducted in-place and laboratory permeability tests on three soil-bentonite test fill sections at the W-105 Project. The report which follows describes our investigations and presents our results.

We have discussed our findings with personnel from Kaiser Engineers Hanford Company (KEH), Westinghouse Hanford Company (WHC), Richland Operations of the U.S. Department of Energy (DOE), the Washington Department of Ecology (WDOE), and the LOESS Group, an independent consultant to KEH. The results presented herein have been previously presented in a meeting on February 26, 1991.

If you have any questions regarding this report, please call us at your convenience.

Respectfully Submitted,
CHEN-NORTHERN, INC.

Brian J. Williams, P.G.
Geotechnical Engineer

Dee J. Burrie, P.E.
Division Manager

DEE J. BURRIE
STATE OF WASHINGTON
REGISTERED
PROFESSIONAL ENGINEER

cc: Mr. Larry Gaddis, P.E

HIH

PURPOSE

This testing project was conducted to evaluate the permeability characteristics of three soil-bentonite combinations proposed for use as an amended soil liner. Sealed Double Ring Infiltrometer (SDRI) permeability testing was used as the primary test method for evaluation of permeability. The SDRI testing, in conjunction with other materials tests, forms the basis for acceptance of the full-scale amended soil liner.

PROJECT DESCRIPTION

The W-105 Project is located near the northeast corner of the 200 East area on the Hanford Federal Reservation. The project includes construction of three lined liquid retention basins and a pipeline connecting these basins to other facilities on the Hanford Site. Prior to the beginning of test fill construction, the subgrade for the retention basins was constructed in the native gravel soils. After test fill construction and testing is completed, subsequent phases of construction will include placement of the liner and cover system.

The specific work described by this project is the construction of test fills located north of the W-105 construction area. In the test fill area, several test fill pads were constructed using various pugmill-mixed soil-bentonite combinations. Pugmill mixing was performed in two series. During the first series, the soil-bentonite product exhibited out-of-specification variations in bentonite and moisture content. These materials were deemed unsuitable for construction of both the test fill areas and the full-scale liners, and a different contractor was selected to mix the soil-bentonite material for the test fill areas. The second series of soil-bentonite combinations were found to comply with the specified tolerances for bentonite and water content, and three test fill sections were constructed.

TEST FILL CONSTRUCTION

Three test fill areas were constructed in December, 1990, to evaluate permeability of the proposed soil-bentonite combinations. The materials were mixed at two different nominal bentonite contents. Curing times ranged from out-of-mill construction to a minimum of 24 hours stockpile time after blending and prior to construction. Test fill bentonite content and curing time is listed below.

TEST FILL NUMBER	NOMINAL BENTONITE CONTENT, %	STOCKPILE CURING TIME, HOURS	DATE PLACED
3	11 - 14	Test Fill Constructed Out of Pugmill	12/4-12/5
6	11 - 14	24	12/6, 12/10

TEST FILL NUMBER	NOMINAL BENTONITE CONTENT, %	STOCKPILE CURING TIME, HOURS	DATE PLACED
7	13 - 16	24	12/6, 12/10

The test fill sections were constructed using the equipment and placement methods proposed for construction of the full-scale liner. The proposed test fill site was stripped of organic material and compacted. A layer of non-woven geotextile was then laid on the compacted subgrade. Test fill material was then placed on the geotextile and compacted. The first lift of soil-bentonite was placed to a nominal thickness of 12 inches, with subsequent lifts placed at a nominal thickness of 6 inches. With the exception of the final lift, the lifts were compacted by six passes of a static (non-vibratory) dual-drum tamping-foot roller (Caterpillar 825). Each pass of the roller consisted of full drum width coverage. The final lift was compacted with two passes of the tamping foot roller, and then four passes of an Ingersoll-Rand smooth-drum static roller. Subsequent to final lift placement, plastic sheeting was placed on the finished surface to minimize drying of the surface. After the plastic sheeting was placed, six relatively undisturbed ring-type samples were obtained from each of the test fills for preliminary laboratory permeability testing.

LABORATORY PERMEABILITY TESTING

Six samples were obtained from each test fill. The six samples consisted of two samples from three random locations. Dual samples were taken to minimize lost time from re-sampling if one of the samples were to be lost or damaged during handling or shipment.

The samples were obtained by pushing a lined split-tube sampler into the test fill. The liners consisted of brass tubes 1 inch and 6 inches in length, and 2.5 inches in diameter. The sampler was pushed and withdrawn using a front-end loader.

After the sampler was withdrawn from the test fill, the sampler was disassembled, and the sample rings were removed and examined for possible defects in the enclosed soil sample. The sample rings were visually examined by our engineer, and the 6 inch length of sample that appeared to be in the best condition was selected for shipment and testing. Each brass tube was individually capped and sealed, wrapped in bubble wrap, and then placed in another capped and sealed tube for transportation to our Pasco laboratory. At the laboratory, the samples were packed in foam packing materials and shipped via air freight to our laboratory in Billings, Montana for laboratory permeability testing.

At the Billings laboratory, the samples were again visually examined by the senior laboratory technician. The samples selected for testing were extruded from the brass tubes, trimmed, measured, weighed, placed in a double latex membrane, and then placed in a triaxial cell for permeability testing. The samples were back-pressure saturated until the "B" coefficient was 0.99, indicating practical saturation. The samples were then placed under a nominal hydraulic gradient of 10 and tested for a period of seven days. The tests were performed in general accordance with the Corps of Engineers Manual EM 110-2-1906, Appendix VII. The results are summarized below and are presented in Appendix A.

TEST FILL SAMPLE #	PERMEABILTY, CM/SEC
TF#3, 1A	1x10 -8
TF#3, 3A	3x10 -8
TF#3, 3B	2x10 -8
TF#6, 1A	2x10 -8
TF#6, 2A	1x10 -8
TF#6, 3A	2x10 -8
TF#7, 1A	2x10 -8
TF#7, 2B	5x10 -9
TF#7, 3A	1x10 -8

SEALED DOUBLE RING INFILTROMETER TESTING

The primary permeability evaluation of the test fills was performed using the Sealed Double Ring Infiltrometer (SDRI). The SDRI test is a derivation of several types of infiltration tests adapted specifically for in-place permeability testing of soil liners. This test method models leakage of containment fluid onto a soil liner. The procedure was established by Trautwein at the University of Texas (Austin) in 1986. A copy of this original paper is included as Appendix B. Since 1986, the SDRI has become one of the standard test methods for in-place permeability determinations, and is currently referenced under the ASTM Test Method D5093.

The SDRI tests performed on the W-105 project are an adaptation of the present ASTM Test Method. The tests at the W-105 project were started prior to complete acceptance of the method by ASTM, and thus contain several minor variations from the ASTM procedure.

The SDRI apparatus consists of an aluminum inner and outer ring, soil tensiometers for measuring soil suction (and, indirectly, wetting front penetration), and flexible intravenous fluid (IV) bags and tubing for flow measurement into the inner ring. (See Figure 1.)

As fabricated for the W-105 project, the outer ring consisted of four aluminum panels 7 feet 2 inches in length and 30 inches in height. When bolted together with a rubber gasket between panels, the panels form a square box 7 feet in square dimension. The inner ring consisted of a welded aluminum box 30 inches by 30 inches in plan dimension, varying from 10 inches to 14 inches in height. The variation in height allowed air entrapped during testing to rise to one end of the box to be withdrawn. The inner ring has two ports, one of which has a valve for bleeding the air referenced above, and the other port is used as the inlet from the two flexible bags filled with water.

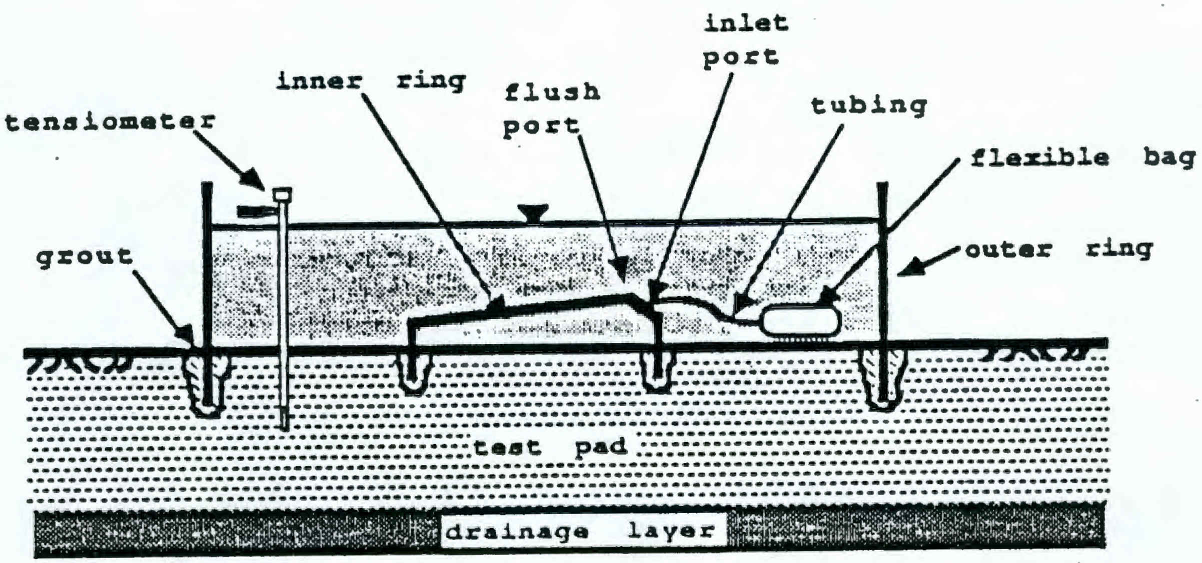


FIGURE 1. Schematic of a Sealed Double Ring Infiltrometer Installed On Test Pad.

The outer ring of the SDRI was installed by machine excavating a trench approximately 16 to 18 inches in depth and 4 inches in width into the soil liner. The trench was then filled with a commercial grout (VOLCLAY grout), and the outer ring was pushed into the grout to a depth of 16 to 18 inches. The inner ring was installed by hand excavating a trench 4 inches in depth and 2 inches in width. Grout was placed in the trench, and the inner ring was pushed into place.

After the rings were placed, the soil tensiometers were installed. (See Figures 2 and 3). The tensiometers were installed in three sets of three tensiometers. Each set of tensiometers was installed with the top of the porous tips at depths of 6, 12, and 18 inches. After installation of the tensiometers, the outer ring was filled with water to a depth of 12 inches when measured at the centerline of the inner ring. During initial filling, the inner ring ports were left open to allow filling concurrent with the outer ring to eliminate any differential head during initial filling. The apparatus was then left for a period of at least one day prior to closing the inner ring ports and attaching the flow measurement bags.

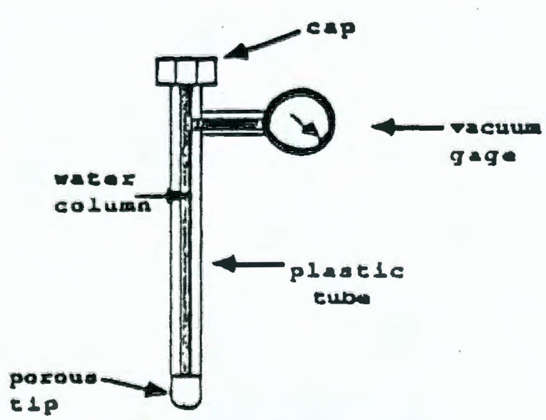


FIGURE 2. Schematic Of a Tensiometer

9 1 1 2 1 7 1 0 7 3 0

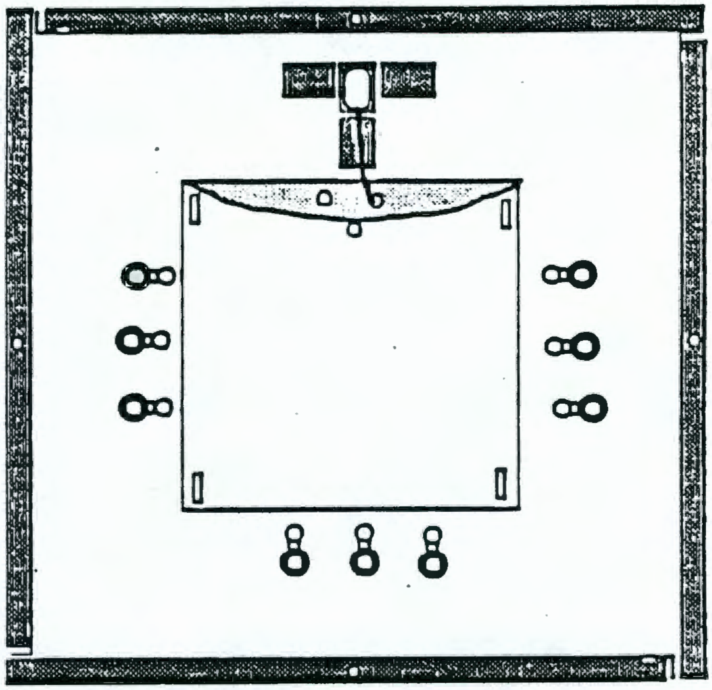


FIGURE 3. Top View of SDRI Apparatus

Since the SDRI tests were performed in temperatures below freezing, the apparatus were enclosed in small, insulated wooden buildings. These buildings were heated by a portable electric heater, which maintained the temperature of the apparatus and water above freezing.

Initially, the tensiometers and bags were read every working day. After initial readings established the expected range of permeabilities, the reading period was extended to periods of several days. Temperatures inside the SDRI enclosures were also recorded at each reading period.

PERMEABILITY CALCULATION

Measurement of inner ring infiltration was determined by weighing the IV bags full of de-aired water prior to starting flow, then re-weighing the bags after a minimum period of 24 hours. The weight of water which flowed out of the bags was converted to a volume of flow. Each time the bags were removed or re-attached, the time of day and date were recorded to establish the time component of flow. The infiltration rate is then calculated by the following equation:

$I = Q/(Axt)$, where

I = infiltration rate in milliliters per second,

Q = flow in milliliters,

A = the area of the inner ring, or 5806 square centimeters, and

t = elapsed time from start of the flow measurement to removal of bag from the apparatus

After the infiltration rate is determined, permeability can be calculated. Using Darcy's law, permeability is calculated by dividing the infiltration rate by the gradient of the flow. The gradient is generally defined as the change in head divided by the flow path; or, in the case of the infiltrometer,

$$i = (H + D\,(+Hs))/L, \text{ where}$$

i = the gradient; a dimensionless number,

H = pressure head from depth of water in the outer ring,

D = pressure head from depth of water penetration into the test fill,

Hs = suction head inside the wet side of the wetting front, and

L = length of the flow path or depth of water penetration into the test fill.

For calculation of gradient at the W-105 project, a strictly conservative approach was used. First, no amount of suction head was attributed to the equation. Suction head was not used in the equation for the following reasons:

1. The amount of suction head measured by the tensiometers is theoretically greater than the suction head "Hs" inside the wetting front. Therefore, an assumed suction head inside the wetting front could be overestimated and may result in unconservative calculation of permeability.

2. The amount of suction head inside the wetting front is variable depending upon the degree of saturation, and is practically impossible to measure under field conditions.

Secondly, the depth of water penetration (D and L) for all calculations was assumed to be a maximum of 6 inches. This assumption was made since some depth had to be assumed. (as of February 28, 1991), the tensiometer readings had not gone to zero, indicating penetration of the wetting front had not progressed to a depth of 6 inches. Using the assumed 6 inch wetting front depth, the calculated gradient is:

$$i = (12" + 6")/6"$$

$$= 3, \text{ a dimensionless number}$$

The assumed gradient of 3 was used in all calculations of permeability for the W-105 project. The reading times, infiltration rates, and calculated permeabilities are presented in Appendix C.

Given the above equations, it can be seen that with a large gradient the calculated permeability is less than that calculated with a small gradient. By eliminating the suction head factor from the gradient calculation, the calculated gradient decreases, conservatively increasing the calculated permeability. In addition, by assuming a 6 inch wetting front penetration, the gradient decreases from actual (since the wetting front had not actually reached 6 inches), and the calculated permeability increases. Therefore, the permeability calculated by these equations is inherently conservative.

RESULTS

The results of the SDRI tests are presented on Figures 4, 5, and 6, and are combined on Figure 7. The plots show some variation, but the actual range of variation is extremely small. The graphic presentation somewhat exaggerates the variations. The most obvious variation is shown before day 20 on Test Fill No. 6. This variation, a dramatic decrease in infiltration, is attributed to test fill soil degassing. In this case, as water penetrates the initially unsaturated soil, the entrapped air migrates along the path of least resistance upward. This is a phenomena typical of initial saturation of an unsaturated soil, where the entrapped air is driven out of the enclosing soil pore spaces and migrates upward. As the trapped air migrates upward, infiltration rate decreases.

CONCLUSIONS

The results of the field and laboratory testing indicate that the permeability rates are expected to be less than 4×10^{-8} cm/sec for the nominal 11 to 14 percent bentonite content (Test Fills No. 3 and 6) and less than 2×10^{-8} cm/sec for the nominal 13 to 16 percent bentonite content (Test Fill No. 7).

It is our opinion that the permeability of the test fill materials will be less than the maximum EPA-recommended permeability of 1×10^{-7} cm/sec. Liner materials, placed using construction procedures utilized during test fill construction, can be expected to exhibit comparable permeability characteristics.

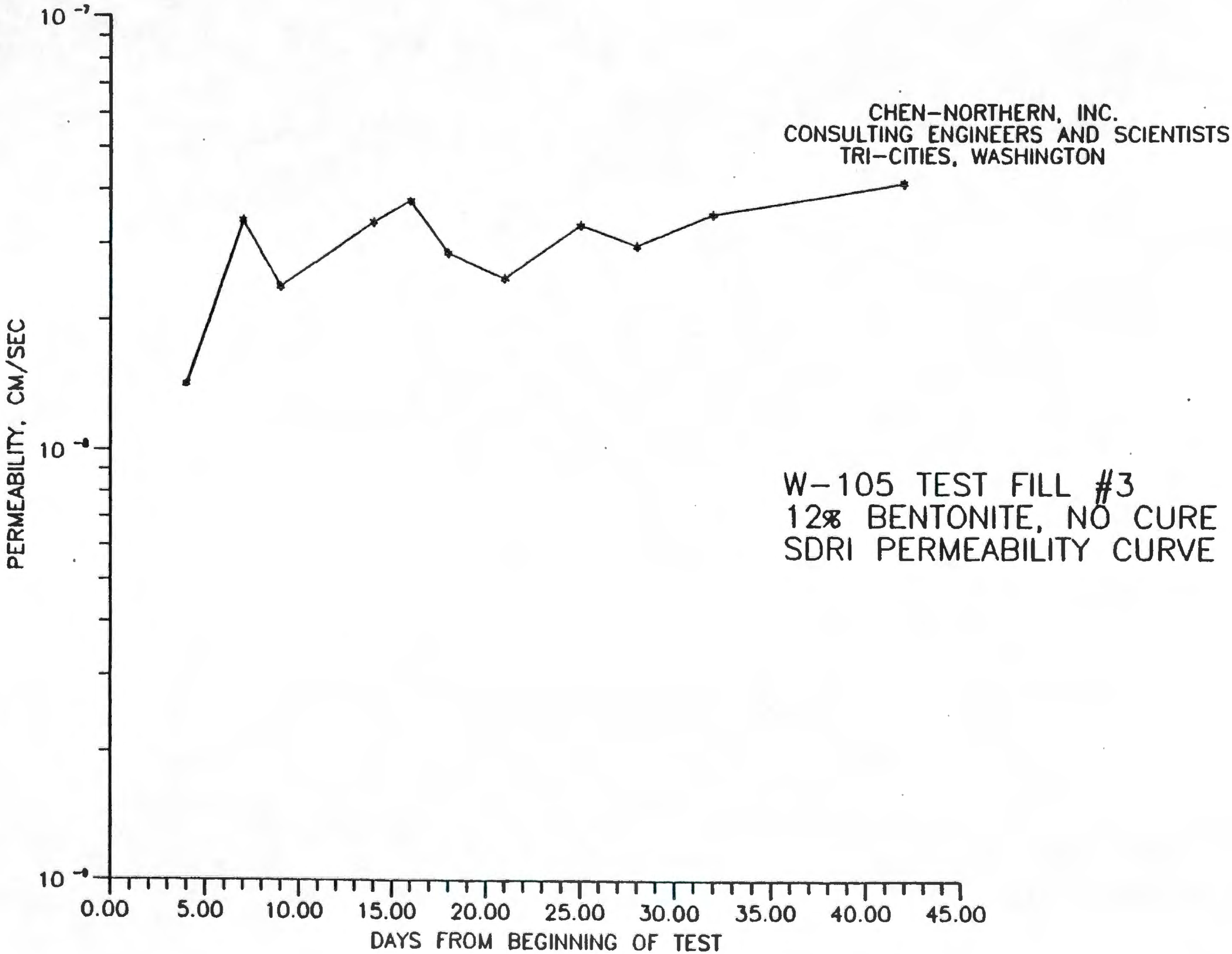


FIGURE 4. Permeability Vs. Time; Test Fill No. 3

91121330735

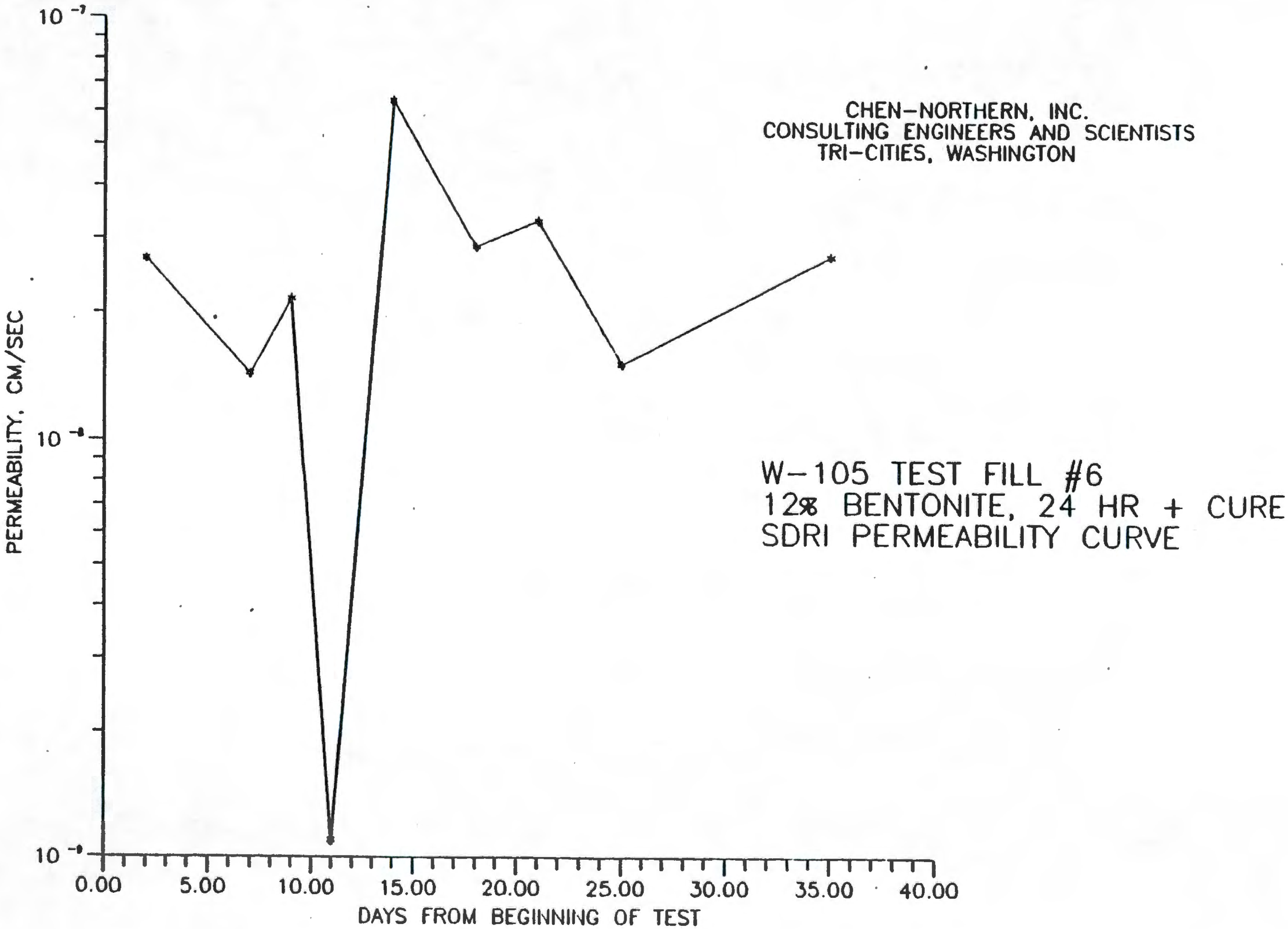


FIGURE 5. Permeability Vs. Time; Test Fill No. 6

9 1 1 2 1 7 3 0 7 3 6

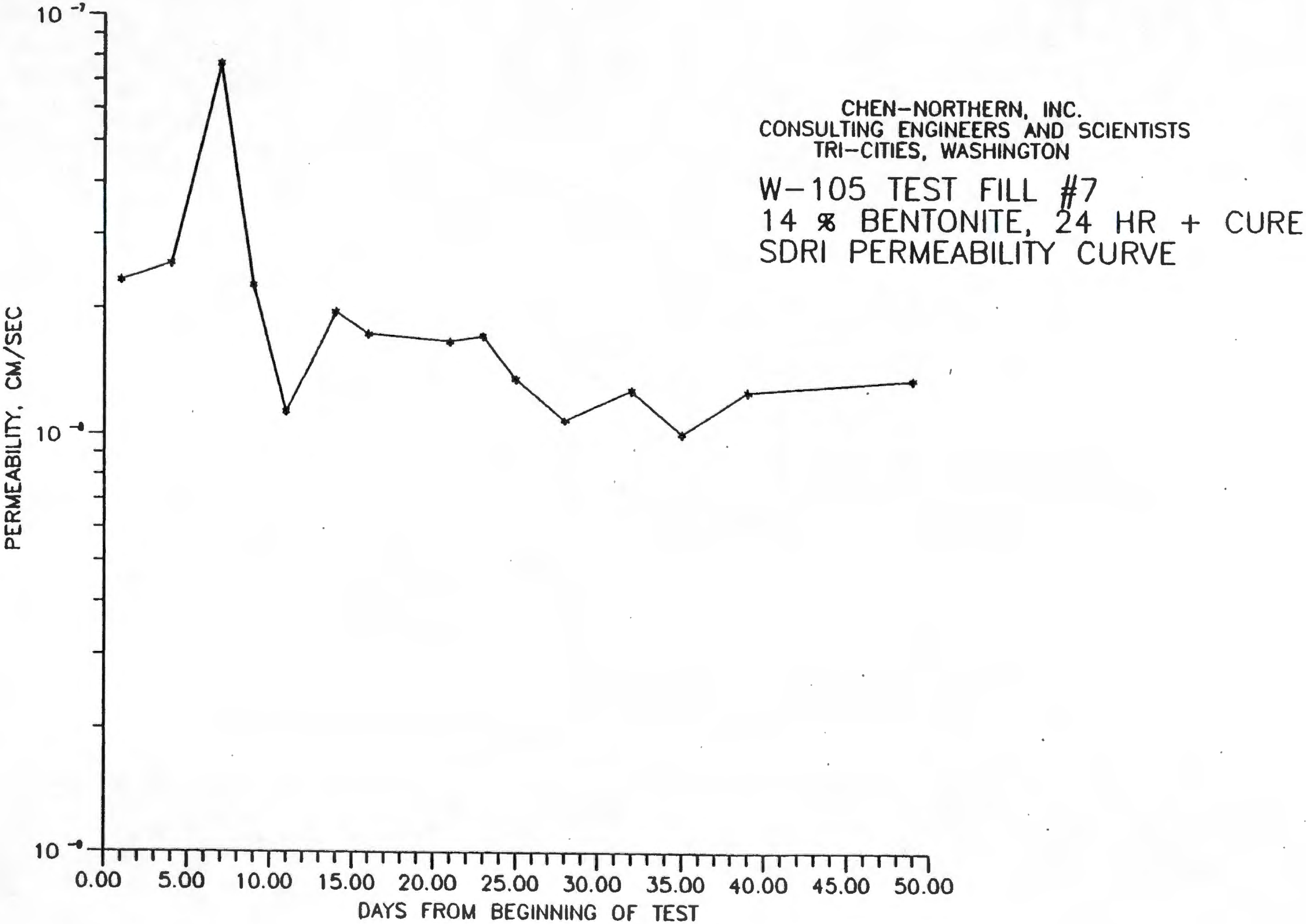


FIGURE 6. Permeability Vs. Time; Test Fill No. 7

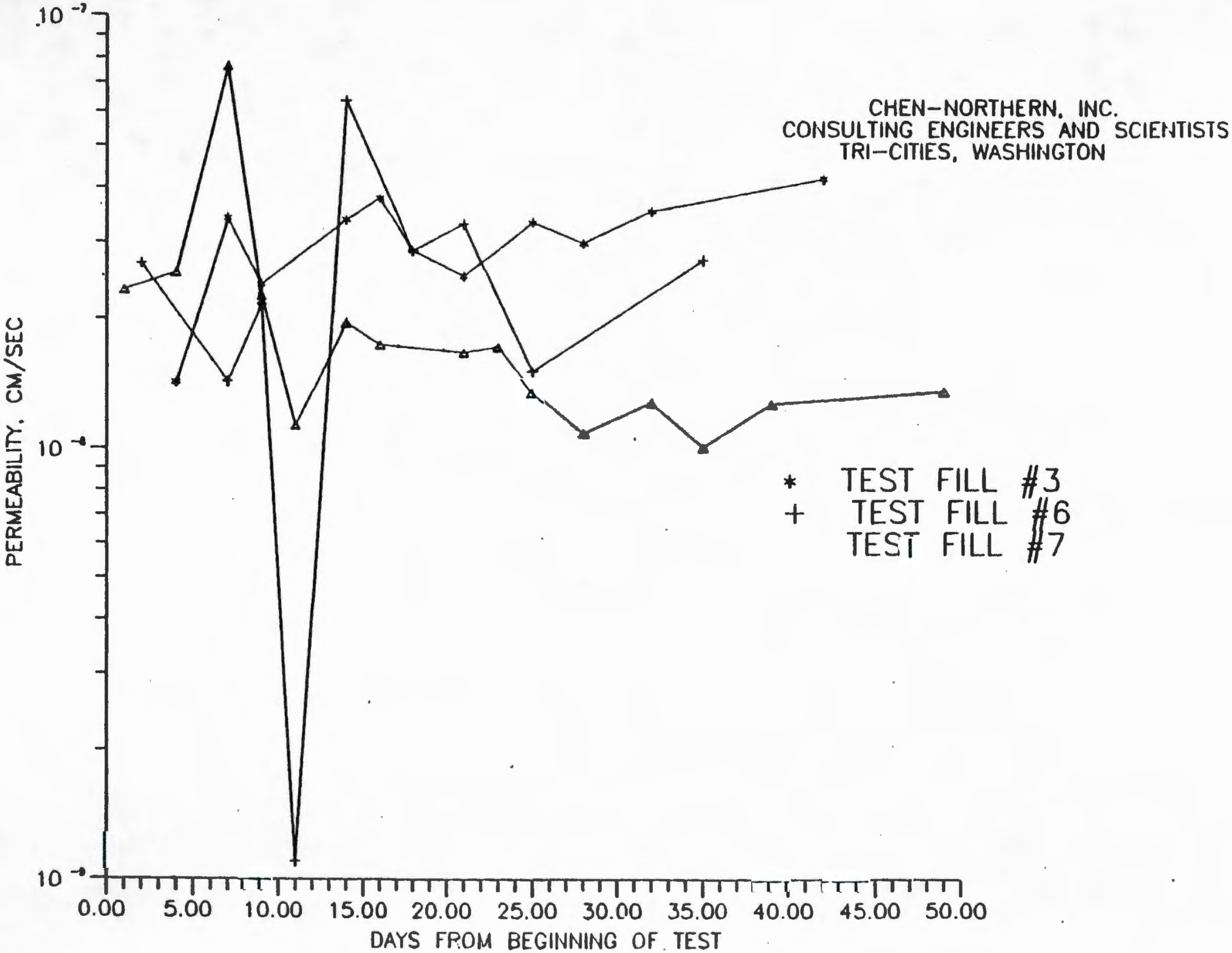


FIGURE 7. Permeability Vs. Time: Test Fills No. 3, 6, and 7

APPENDIX A
LABORATORY PERMEABILITY RESULTS

Chen-Northern, Inc.
A member of the HIH group of companies
600 SOUTH 25TH STREET
P. O. BOX 30615
BILLINGS, MT 59107
(406) 248-9161
FAX (406) 248-9282

TECHNICAL REPORT

REPORT TO:

ATTN: BRIAN WILLIAMS
CHEN-NORTHERN, INC.
2214 NORTH 4TH AVE
P O BOX 2601
TRI-CITIES, WA 99302

DATE: December 18, 1990
JOB NUMBER: 87-601
SHEET: 1 **OF** 1
INVOICE NO.: 108149

REPORT OF: Triaxial Permeability Test - KEH - W105 (Job No. 86-1905)

9 3 7 0 0 1 2 1 9 9

Sample Identification:

On December 10, 1990, we received three soil samples from the subject project, with instructions to perform a triaxial permeability test on each sample.

The tests were prepared and performed in general accordance with Corp of Engineers Manual EM 110-2-1906 Appendix VII.

TEST RESULTS:

Sample Identification	Dry Density (pcf)	Moisture Content (%)	Coefficient of Permeability (cm/sec)
TP 3 No. 1A	102.1	21.2	1×10^{-8}
TP 3 No. 3B	101.8	20.3	2×10^{-8}
TP 3 No. 3A	102.9	21.2	3×10^{-8}

Reviewed by

clz

Chen-Northern, Inc.

A member of the HIH group of companies

600 SOUTH 25TH STREET
P. O. BOX 30615
BILLINGS, MT 59107
(406) 248-9161
FAX (406) 248-9282

TECHNICAL REPORT

REPORT TO: ATTN: BRIAN WILLIAMS
CHEN-NORTHERN, INC.
2214 NORTH FOURTH AVENUE
P O BOX 2601
TRI-CITIES, WA 99302

DATE: January 29, 1991
JOB NUMBER: 87-601
SHEET: 1 **OF** 1
INVOICE NO.: 108518

REPORT OF: Triaxial Permeability Test - Kaiser Engineers Hanford W-105 (Job No. 86-1905)

Sample Identification:

On December 21, 1990, we received six soil samples from the subject site with instructions to perform a triaxial permeability test on each sample. The tests were prepared and performed in general accordance with the Corp of Engineers Manual EM 110-2-1906, Appendix VII.

TEST RESULTS:

Lab No.	Sample Identification	Dry Density (pcf)	Moisture Content (%)	Coefficient Of Permeability (cm/sec)
9001	TP 1 No. 1A	103.8	20.7	2×10^{-8}
9002	TP 1 No. 2A	101.8	21.6	1×10^{-8}
9003	TP 1 No. 3A	102.3	20.9	2×10^{-8}
9004	TP 7 No. 1A	101.8	22.1	2×10^{-8}
9005	TP 7 No. 2B	101.1	22.5	5×10^{-9}
9006	TP 7 No. 3A	103.4	22.2	1×10^{-8}

Reviewed by: [signature]

rl

Note: Samples identified as "1" weremislabelled in the laboratory; these samples are from Test Fill #6.

APPENDIX B
1986 TRAUTWEIN PAPER
UNIVERSITY OF TEXAS, AUSTIN

FIELD MEASUREMENT OF INFILTRATION RATES USING A SEALED DOUBLE-RING INFILTROMETER

by

Stephen. J. Trautwein

Notes for Short Course Entitled

"CLAY LINERS AND COVERS
FOR
WASTE DISPOSAL FACILITIES"

Presented
by

College of Engineering
The University of Texas at Austin

INTRODUCTION

Most evaluations of the hydraulic conductivity of earthen liners have been based upon laboratory permeability tests. Recently, the reliability of laboratory permeability tests for earthen liners has been questioned. Field tests have yielded much higher values of hydraulic conductivity than laboratory tests. For this reason, many regulating agencies are putting emphasis on field tests.

One method of field testing that is gaining wide spread use involves the use of a Sealed Double-Ring Infiltrometer(SDRI). The purpose of these notes is to explain: 1) how the need for field testing arose; 2) the development of the SDRI; 3) installation and use of the SDRI; 4) methods of data reduction; and 5) ways to minimize factor which influence the data.

FIELD TESTING VS LABORATORY TESTING

In the past, the accepted practice for verifying that the hydraulic conductivity an earthen liner or cover met the design requirements was to perform a laboratory test on a small diameter sample. Sometimes the tests were performed on an undisturbed sample obtained from the liner but mostly recompacted samples of the same material were tested. Unfortunately, many of the landfills designed in this manner leaked. This led several investigators to perform field tests . It was found that hydraulic conductivities determined in the field were sometimes orders of magnitude greater than those determined in the laboratory. Some of the reasons for this difference were obvious. Visual inspection revealed desiccation cracks, the presence of deleterious materials such as roots and twigs, and zones of materials such as silt and sand. All these provided preferential pathways for water to flow more rapidly through the liner.

To explain the difference between laboratory and field values of hydraulic conductivity it is convenient to define the following terms, micropermeability and macropermeability, . Micropermeability refers to the flow of permeant through the void spaces between soil particles, most of which are in contact with other soil particles. Macropermeability refers to flow through these void spaces as well as flow through larger void spaces such as desiccation cracks, fissures, root holes, etc. Flow through small samples such as those tested in the laboratory is controlled by the micropermeability of the soil whereas flow through earthen liners or covers is controlled by the macropermeability.

Because of the difference found between hydraulic conductivity measured on the lab and the field, many regulatory agencies started to require field testing to verify that liner and covers met specifications.

TYPES OF FIELD TESTS

When field testing was first required, there was little guidance available for selecting what type of test to perform. Regulations typically included a statement similar to the following: "The hydraulic conductivity must be verified with a field test performed according to accepted civil engineering practice". The problem was that there was no standard or accepted technique for determining thehydraulic conductivityof a liner or cover.

Techniques used for measuring hydraulic conductivity in the field included borehole tests, piezometer or porous probes, and infiltration tests. In evaluating what technique might be best adapted for testing for liners and covers, infiltration testing seemed the logical choice. Large areas could be tested and the test modelled the case for which the liner or cover was designed, i.e. water ponded on the surface. This allowed for the direct measurement of the rate at which water passed through the soil.

While the other techniques offered the advantages of simplicity and relatively short testing times, they were considered to have severe shortcomings. These shortcomings included small sample size, measurement of horizontal rather than vertical flow, and uncertainty in accounting for unsaturated soil conditions.

INFILTRATION TESTING

Infiltrometers were used mainly to ensure that a soil had a high enough hydraulic conductivity so that it would drain adequately, not a low enough hydraulic conductivity so that it could serve as a moisture barrier. When an attempt was made to use a simple open ring infiltrometer(Fig. 1) to measure hydraulic conductivity of a liner, two major problems became apparent. The first was a large component of lateral flow beneath the ring. The second was the inability to measure small changes in the water level.

Lateral flow is a problem because it can not be separated from the vertical, one-dimensional component which is needed to determine k. Fortunately, the problem of lateral flow can be accounted for by installing a second ring centered inside the outer ring so that it encompasses the area in which the flow is one-dimensional(Fig.2). This is known as a double-ring infiltrometer. Measurements of flow are made from the inner ring. Equal water levels must be maintained in both rings to ensure that water does not flow from one ring to the other.

Large diameter double-ring infiltrometers have been used in an attempt to measure the hydraulic conductivity of liners. To account for evaporation, a second set of rings of the same diameter but with a sealed bottom were used as control rings. The drop in the water level in the control rings was subtracted from the drop in water level in the test rings. However, in many instances evaporation was significantly greater than infiltration. For example the drop in the water level of one foot of water ponded on a soil with a hydraulic conductivity of 1x10-7 cm/sec is about 0.0035 in. per day(Fig. 3). Evaporation can be as much as 0.25 in. per day. Subtracting two large numbers to obtain a small number without introducing significant error is questionable. Also, it is questionable that the water level can be measured to that degree of accuracy because the water surface is not still in large rings.

Fortunately, the problem of measuring small changes in elevation can be overcome by using a special inner ring. This ring is described in the next section.

SEALED DOUBLE-RING INFILTROMETER

The problem of measuring a small change in elevation is eliminated by using a Sealed Double-Ring Infiltrometer(SDRI). With this device(Fig. 4), rather than measuring a drop in elevation, the amount of water flowing into the ground is measured directly. The inner ring is sealed and submerged in the water in the outer ring. Measurement of flow is made by connecting a flexible bag, filled with a known weight of water, to a port on the inner ring. As water infiltrates into the ground from the inner ring, and equal amount of water flows into the inner ring from the flexible bag. After a known interval of time, the bag is removed and weighed. The weight loss, converted to a volume, is equal to the amount of water that has infiltrated the ground. An infiltration rate is then determined from this volume of water, the area of the inner ring, and the interval of time.

The design of the SDRI offers several other advantages over open ring systems. Evaporation is eliminated because the inner ring is sealed. The head at any elevation in the inner or outer ring is the same so there is no gradient to cause water to flow from one ring to the other. Also, since the head is the same, the pressure difference across the wall of the inner ring is constant, hence the inner ring will not expand or contract even though the water level in the outer ring may change.

The first version of the SDRI consisted of two circular, fiberglass rings. The outer ring was 7 ft. in diameter and 28 in. high. The inner ring was five ft. in diameter, was dome shaped with an upper and lower port. The size of the rings was selected to be large enough to measure the macropermeability and to ensure flow was one-dimensional. The inner ring was embedded 4 in. in the ground while the outer ring was embedded 6 in. Flow measurements were made using Mariotte tubes. A separate Mariotte tube for each ring was used to maintain constant and equal heads.

While some success was achieved using the first version of the SDRI, several problems arose that led to changes in the rings. These problems included: (1) leaks beneath the outer ring; (2) leaks beneath the outer ring affecting measurements made in the inner ring; and (3) the inability of the Mariotte tubes to measure small amounts of flow.

The first two problems were solved by making the outer ring taller so that it could be embedded deeper. This also had the added advantage of forcing flow in both rings to be one-dimensional to a greater depth. The outer ring was also made wider so that the distance between the inner and outer ring was increased. This necessitated making the ring square so that it could be disassembled for transport. The outer ring is made of metal panels that bolt together at the corners(Fig 5). The ease of digging straight trenches as opposed to circular trenches was also a determining factor in changing the shape of the ring. The inner ring was also made square for this reason(Fig. 6).

The last problem was overcome by replacing the Mariotte system with a flexible bag. As it turned out, the flexible bag was not only more accurate, it was simpler to use and less expensive.

INSTALLATION OF SDRI

Outer Ring

The outer ring is embedded 12 to 18 in. in the soil. A trenching machine, the type used for laying wire and pipe, is used to excavate the trench. These machines are readily available at equipment rental stores. It is best to use a machine that cuts a narrow trench, usually 4 in. in order to minimize the amount of grout needed to fill the trench.

The rings are sealed in placed with grout. A bentonite grout, such as Volclay Grout manufactured by American Colloid works best. This grout sets to the consistency of peanut butter, so leaks can always be repaired by re-packing the grout. Several methods have been used to mix the grout including a cement mixer, a centrifugal pump, and a grout mixer. The most convenient method seems to be the grout mixer. The larger the mixer the better. After mixing, the grout is poured into a wheel barrow and then dumped in the trench. until it is filled to the top. The outer ring is then lifted and centered over the trench and pushed into place. A berm(Fig. 7) is then built around the outer ring to keep the panels from bowing when the ring is filled with water. The berm also provides overburden on the grout to reduce the chance of a blowout.

Inner Ring

The inner ring is embedded in a narrow trench, 1 to 2 in. wide and 4 to 6 in. deep, centered within the outer ring. If excavating the trench by hand, a masons or brick hammer works best. A more convenient method of excavating a trench is by using a chain saw. A special chain, one with carbide tips brazed on the chain, is needed. Once the trench is excavated, it is filled with grout then the inner ring is centered over the trench and then pushed into place.

Tensiometers

The position of the wetting front during the test is needed when calculating hydraulic conductivity from the infiltration rate. Tensiometers can be used for this purpose. A tensiometer is a device used to measure soil suction. It consists of a sealed plastic tube with a porous tip on one end(Fig. 8). When the porous tip is placed in unsaturated soil, water will be drawn out of the tube and the gage will register a suction. When the wetting front passes the tip , the soil will become saturated and the suction will go to zero. The depth of the wetting front can be determined then by noting when the gage reading goes to zero.

Tensiometers are widely used in the agricultural area and the typical method of installation involves diving a pipe into the ground to form a hole and then pushing the tensiometer in place. This installation procedure may cause cracking in a compacted soil and a preferred method of installation involves pre auguring a hole with a device such as that shown in Fig. 9.

Typically nine tensiometers are used per test, three at each depth of 6 in. ,12 in., and 18 in..

Filling the Rings

Both rings are filled simultaneously. A splashboard is placed on the ground between the inner and outer ring and water from a hose is directed on it. The ports on the inner ring are left open. As the water level in the outer ring rises to the level of the ports in the inner, it flows through the ports until the inner ring is filled. The outer ring is filled to a depth of 12 in. Once filled, the inner ring is tapped to dislodge air bubbles trapped inside so that they rise and pass out the top vent port.

Connecting the Fittings

Barbed fittings that make it convenient to connect and disconnect tubing are installed in the ports. A length of tubing that is plugged on one end is attached to the top flush port. This port is used to bleed air that may accumulate inside the inner ring during the test. A second piece of tubing is attached to the inlet port. The flexible bag is attached to the other other end of this tube. A top view of the SDRI installed along with tensiometers is shown in Fig. 10.

Covering the Rings

Temperature changes can cause the inner ring to expand and contract thereby introducing error into the flow measurement. Covering the rings with a tarp will serve to insulate the ring and reduce temperature changes in the water. Blocking sunlight from the water in the rings also inhibits the growth of algae which can clog the tubing and restrict flow into the inner ring.

A frame made from 2x4's is placed on top of the outer ring to support the tarp. If temperature fluctuations continue to be a problem, panels of insulating material can be placed of the top of the 2x4's.

FLOW MEASUREMENTS

As mentioned previously, flow measurements are made with the flexible bag. The bag is filled water, weighed, connected to a port on the inner ring, and submerged in the water of the outer ring. Any water that flows out of the inner ring into the ground will be replaced by an equal amount of water from the bag. Periodically, the bag is removed and weighed to determine the amount of water that was lost. This weight loss, converted to a volume, is equal to the volume of water that infiltrated the ground while the bag was connected to the inner ring.

The flow measurement data is used to construct a plot of infiltration versus time. For unsaturated soils such as compacted clay liners and covers, infiltration decreases with time at first, changing rapidly at the beginning of the test, and then eventually becoming constant with time as the soil becomes saturated. Consequently, more frequent readings are needed at the beginning of the test and less frequent readings are need as the flow rate becomes steady.

It should be noted that it is not necessary to have the bag connected to the inner ring continuously throughout the test. The bag only need be connected to the inner ring to obtain enough points to define the infiltration versus time curve. The infiltration rate is not affected by the presence or absence of the bag.

FACTORS THAT AFFECT FLOW MEASUREMENTS

Two factors that can introduce cause error in the measured flow rate are temperature and swelling of the soil. Methods to account for these factors are discussed below.

Temperature

A change in temperature will cause the inner ring and the water contained within it to expand and contract. The net effect of of these volume changes is that approximately 50 cc of water will flow in or out of the bag for each degree Celsius change in temperature.

There are several ways to minimize the problem of temperature changes. First, the amount of temperature change that occurs can be reduced by insulating the rings. Second, the bag can be connected and disconnected when the water in the rings is at the same temperature. Even though the temperature may have fluctuated while the bag was connected to the ring, if the system returns to the same temperature, the net flow of water in and out of the bag due to temperature will be zero Finally, the bag can be left connected to the inner ring long enough so that the volume of flow that occurs due to temperature change will be small compared to that due to infiltration. For example, if the bag is left connected until 2000 cc of flow occurs and the temperature change is 4 ^{o}C, than the error is only 10%, an acceptable amount for this type of test.

Swell

Many earthen liners and covers are constructed of clays with high shrink-swell potential. If the soil being tested is initially unsaturated, it will swell as water infiltrates it. The flow measured from the flexible bag includes both the amount taken up by swell and the amount passing through the soil. A means for separating the two is needed in order to calculate k.

One method that can be used is to measure the elevation of the inner ring. Once the wetting front passes below the bottom of the inner ring, the ring will rise as the soil beneath it swells. A plot of swell versus time can be constructed. If it is assumed that the swell occurs in the saturated soil, then the amount of flow that went into swell is equal to the elevation change of the inner ring that occurred while the bag was connected multiplied by the area of the inner ring. Subtracting this amount from the total flow measured leaves the amount which flowed through the soil. This is the amount that should be used to calculate k.

CALCULATION OF INFILTRATION

The calculation of infiltration (I) is straight forward and is determined as follows:

$$I = Q/(At)$$

where:

I = infiltration (cm/sec)
Q = volume of flow (cm3)
A = area of flow (cm2)
t = time interval in which Q was determined (sec)

CALCULATION OF HYDRAULIC CONDUCTIVITY

The calculation of hydraulic conductivity (k) is also straight forward and can be determined as follows:

$$k = Q/(iAt)$$

where:

Q = volume of flow (cm3)
A = area of flow (cm2)
t = time interval in which Q was determined (sec)
i = gradient
$= \Delta h/\Delta s$
Δh = head loss
Δs = length of flow path for which Δh is measured

since:

$$I = Q/(At)$$

then:

$$k = I/i$$

While the calculation of hydraulic conductivity seems relatively simple, the difficulty lies in the determination of the gradient. The problem is one of saturated - unsaturated flow. At the present time there is controversy about what effect soil suction has on the infiltration rate. Listed below are three methods that have been used to calculate k. These methods differ because of the procedure used to calculate the gradient.

Apparent Hydraulic Conductivity

The assumption made when using this method is that the soil layer being tested is saturated. The head loss through the layer is equal to the difference in the levels in piezometers placed at the top and bottom of the layer as illustrated in Fig. 11a. The water level in the top piezometer would rise to the level of the water in the outer ring. The water level in the lower piezometer would be at the bottom of the soil layer. The head loss is equal to:

where:

$$\Delta h = H + D$$

H = depth of water in outer ring

D = thickness of layer being tested

The gradient is:

$$i = (H + D)/D$$

For a test on a typical liner, H = 1 ft. and D = 2 ft., the gradient is 1.5. If the layer being tested is not saturated, this is a conservative estimate of hydraulic conductivity since the gradient will be larger than 1.5.

Suction Head Method

The parameters used in this method are shown in Fig. 11b. , where D is the depth to the wetting front and H_s is the suction head. The suction head is obtained from tensiometers installed below the wetting front. The gradient is:

$$i = (H + D + Hs)/D$$

The appeal of this method is the high gradients that can be obtained, and hence low hydraulic conductivities. It is not unusual to measure suction heads in excess of 20 ft. In a compacted clay liner. Suction heads of this magnitude can yield gradients as high as 80.

The problem with this method lies in the assumption that the suction head can be added to the other gravity head terms in the equation for the gradient. To properly account for the effect of suction on the infiltration rate, a non-linear analysis is required in which the moisture characteristic curve as well as the relationship between the degree of saturation and hydraulic conductivity for the soil being tested is used.

The suction head that exists beneath the wetting front may cause some increase in the infiltration rate, however, the full impact of it is not felt because hydraulic conductivity in the unsaturated zone is less than that in the saturated zone. The effect of the high gradient is offset by the fact that flow into the unsaturated zone is restricted by the lower k. Also, as water enters the unsaturated zone, the suction head is decreased.

The author considers this method to be unconservative and does not recommend its use.

Wetting Front Method

The parameters used the Wetting Front Method are shown in Fig. 11c. The equation for the gradient using this method is the same as that used in the Apparent Hydraulic Conductivity Method except that D in this case is the depth to the wetting front. It is assumed that the suction head at the wetting front is zero. The range in gradient for a test on a two foot thick liner with one foot of ponded water on it is shown in Fig. 12.

Most of the reduction in the infiltration rate for tests the author has performed can be accounted for by the reduction in the gradient shown in Fig. 12. This observation lends support to the opinion that the effect of suction on the infiltration rate is minimal. The Wetting Front Method is the recommended method to use when calculating hydraulic conductivity from infiltration rate.

TESTING TIMES

In most cases, the soil being tested will be unsaturated and the initial flow will be transient. It is recommended that a plot of infiltration rate vs. time be made. Calculated hydraulic conductivity will be correct only when the infiltration rate is reasonably steady. From experience, infiltration rate decreases with time and becomes reasonably steady after 1 - 4 weeks of seepage.

The length of testing will depend on the type of information desired. If steady-state infiltration is required, the testing time can be a month or more. However, if the test is being performed only to confirm that the infiltration rate of hydraulic conductivity is below a specified value, then the testing time may be as short as several days. For instance, if the initial infiltration rates are below the required value, and the infiltration rate is decreasing with time, it is not necessary to wait for steady flow to occur to conclude that the infiltration rate is below the specified value. Conversely, if the initial infiltration rates are much larger than the required infiltration rate, and the infiltration rate is not decreasing significantly with time, then one can conclude that the infiltration rate exceeds the required value.

SUMMARY

Many regulatory agencies require field testing to demonstrate that the hydraulic conductivity of an earthen liner or cover is below the required value. Infiltration tests offer compelling advantages over other types of field test. These advantages include the ability to test large areas and the fact that an infiltration test models the case for which the liner or cover is designed, i.e. water ponded on the surface.

The SDRI is a double-ring infiltrometer that has been modified to overcome the problems associated with testing of liner and covers. These problems include a large component of lateral flow and measurement of small changes in the water level due to low infiltration rates. The large size and deep embedment of the outer ring eliminates the problem of lateral flow. The sealed inner-ring eliminates the need to measure small changes in the water level and makes it possible to measure the volume of water that infiltrates the ground directly.

Two factors that affect flow measurements are temperature changes and swell. The problem of temperature change can be minimized by insulating the rings, connecting and disconnecting the bag when the system is at the same temperature, and leaving the bag connected to the inner ring long enough so that the flow due to infiltration is significantly greater then that due to temperature changes.

Swell can be accounted for by measuring the change in elevation of the inner ring. This change in elevation multiplied by the area of the inner ring is equal to the flow that went into swell.

Several method of calculating hydraulic conductivity were discussed. The "Apparent Hydraulic Conductivity Method" is the most conservative but does not require information on the location of the wetting front. The "Suction Head Method" is unconservative and there are serious questions about its validity. The "Wetting Front Method" accounts for the position of the wetting front and is the recommended method for determining k.

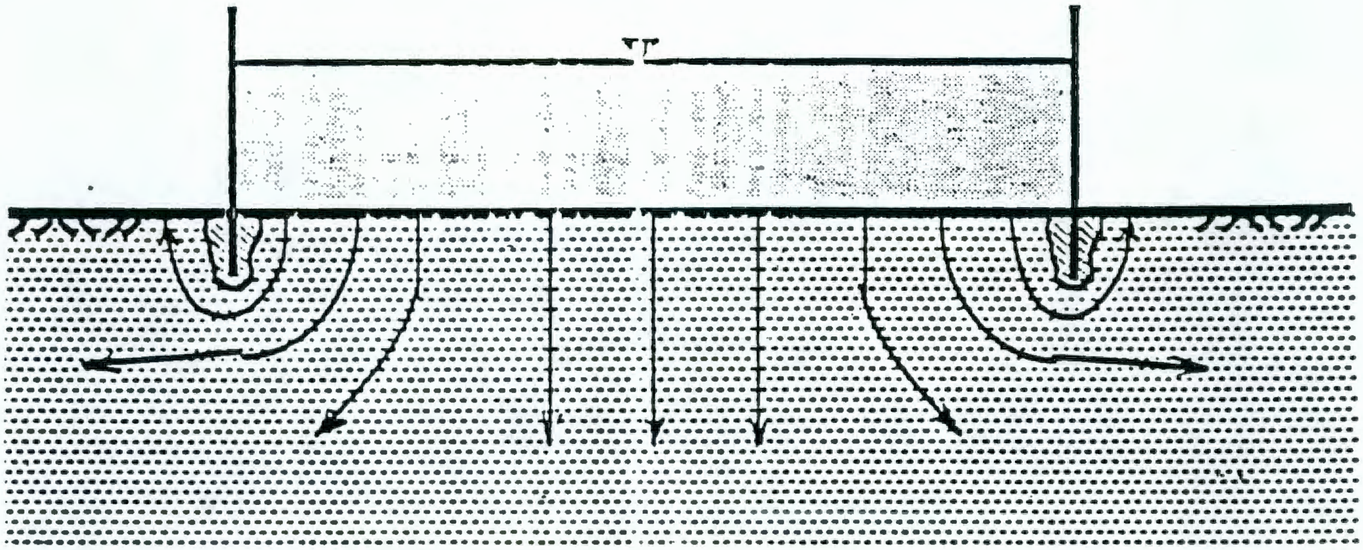


Figure 1. Open Single-Ring Infiltrometer

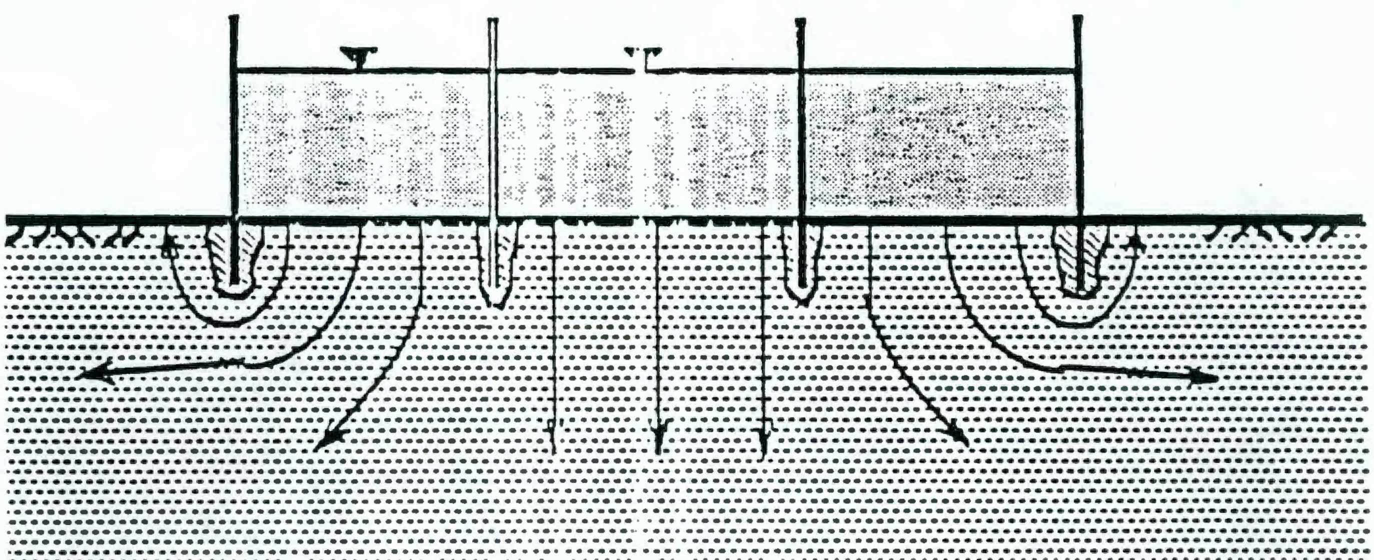


Figure 2. Open Double-Ring Infiltrometer

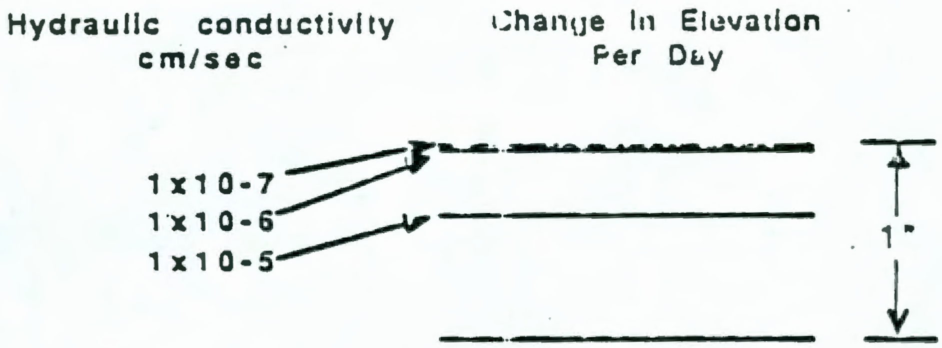


Figure 3. Change In Elevation Per Day Of 1 Foot Of Water Ponded On The Surf

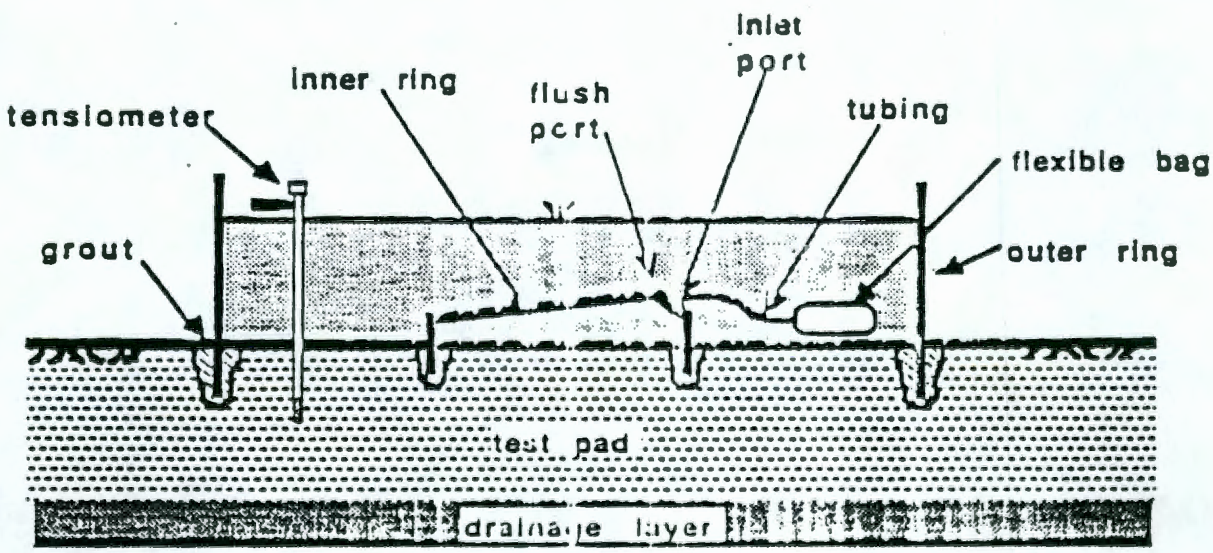


Figure 4. Schematic Of A Sealed-Double Ring Infiltrometer Installed On Test Pad

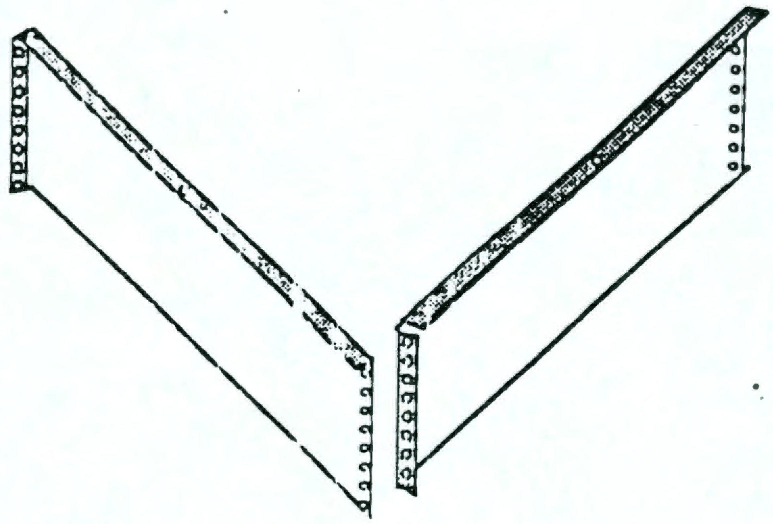


Figure 5. Outer Ring Panels.

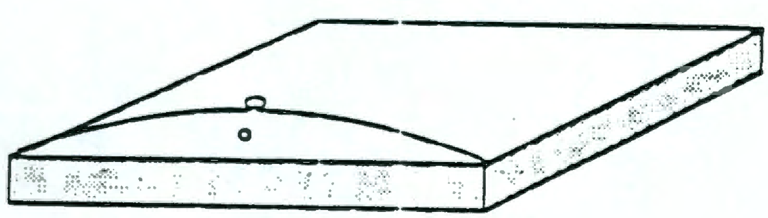


Figure 6. Inner Ring

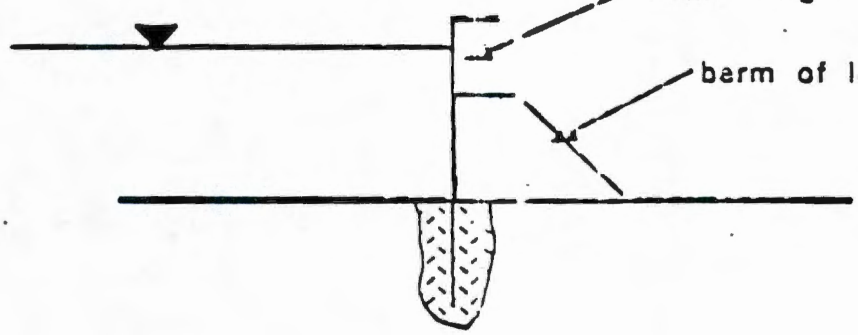


Figure 7. Berm Of Soil On Outside Of Outer Ring

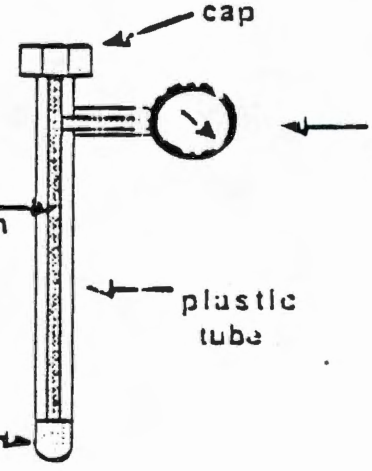


Figure 8. Schematic Of A Tensiometer

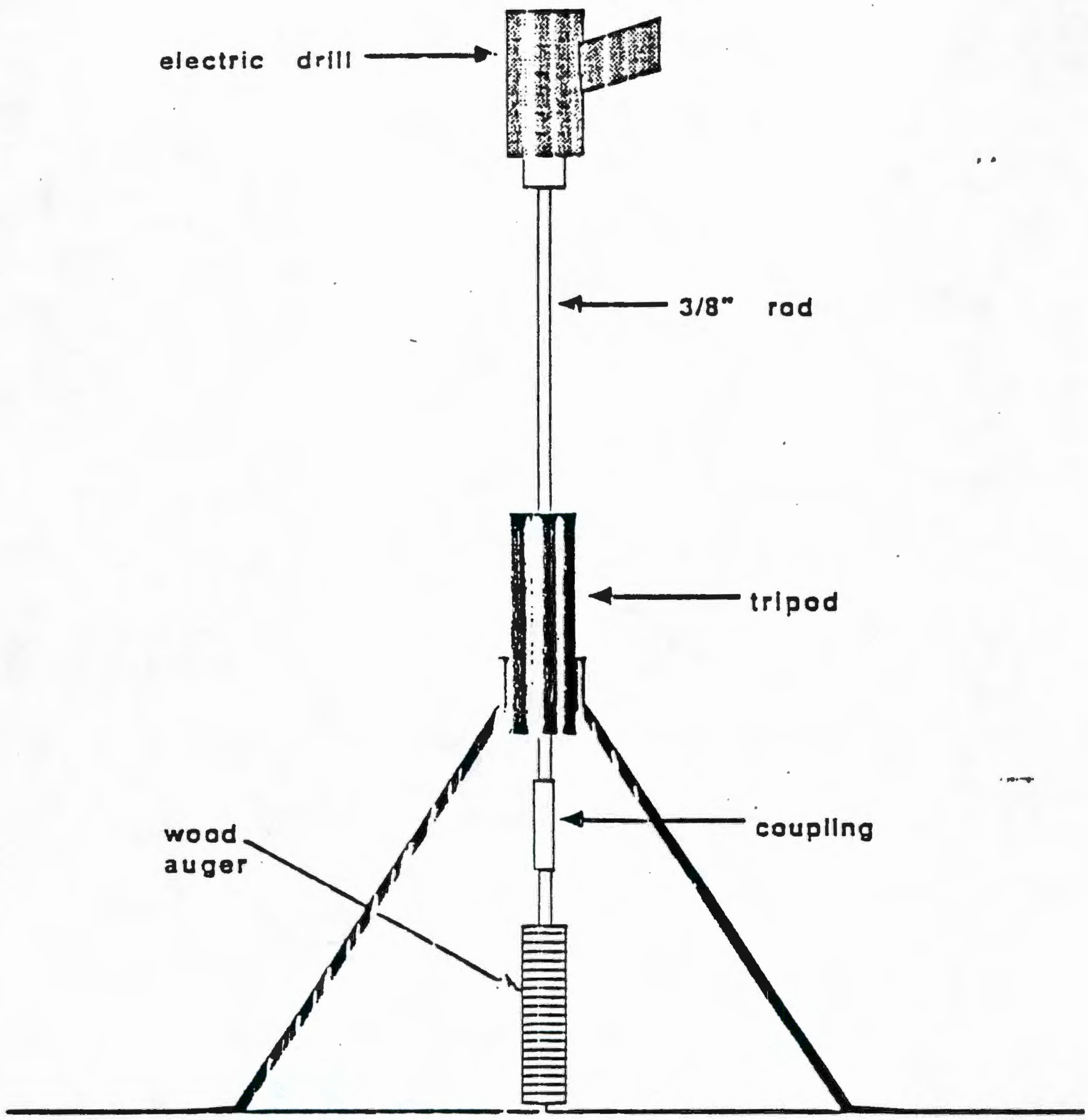


Figure 9. Set-up For Installing Tensiometer

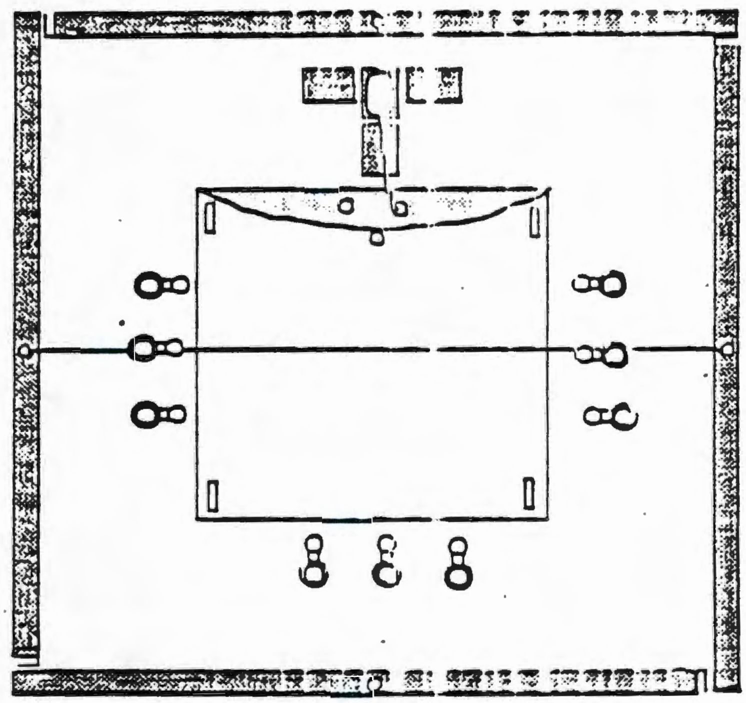


Figure10. Top View Of SDRI With Tensiometers Installed

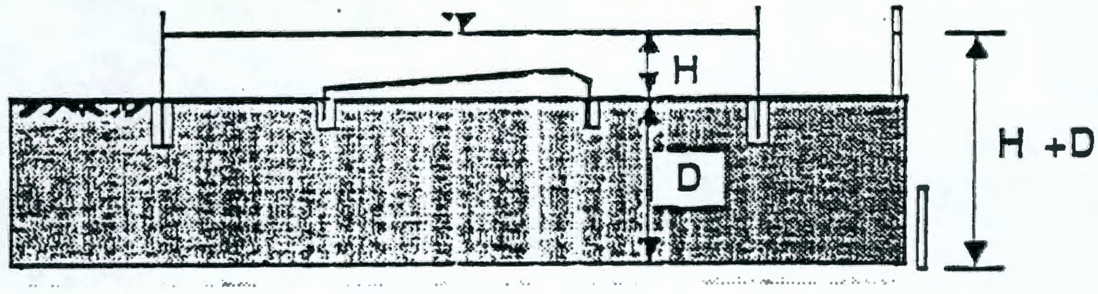
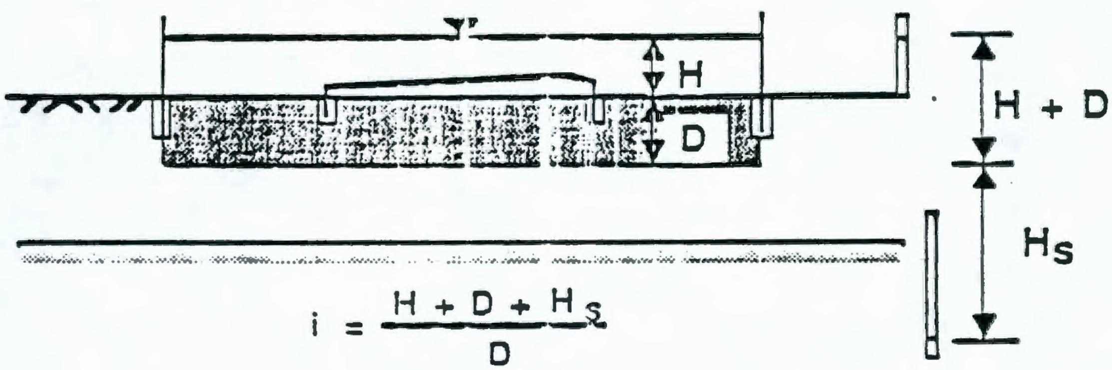
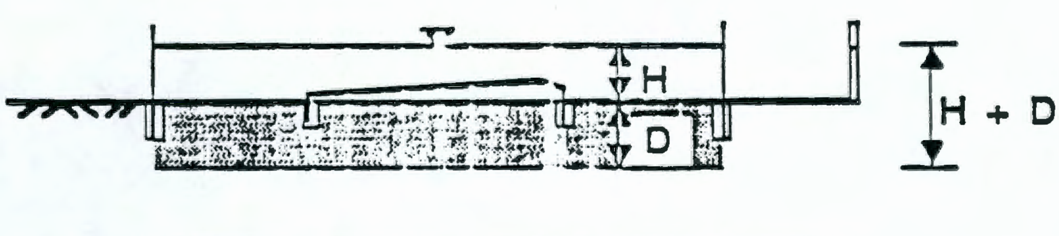


Figure 11a. Apparent Hydraulic Conductivity Method



$$i = \frac{H + D + H_S}{D}$$

Figure 11b. Suction Head Method



$$i = \frac{H + D}{D}$$

Figure 11c. Wetting Front Method

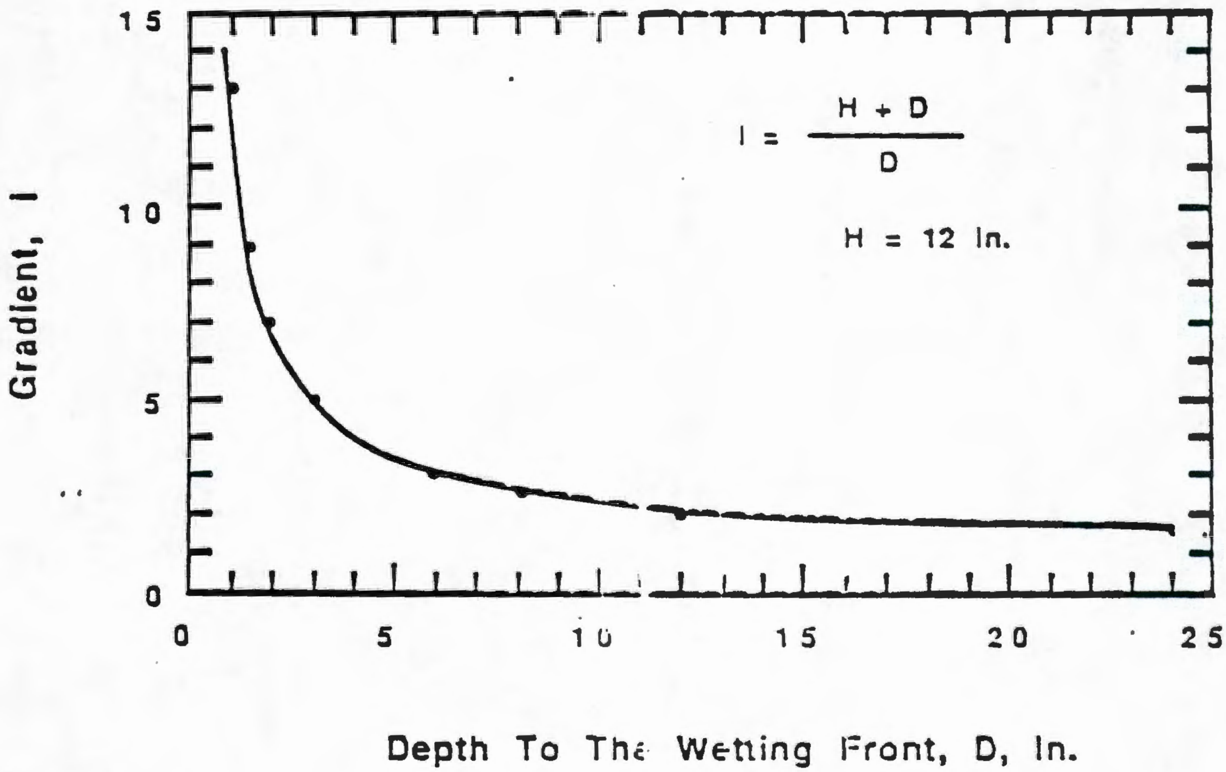


Figure 12. Relationship Between Gradient And Depth To Wetting Front

APPENDIX C
SDRI INFILTRATION AND PERMEABILITY DATA

TEST FILL FLOW SUMMARY, TEST FILL #3

DAYS	DATE	COMBINED BAG WEIGHTS, GRAMS	WEIGHT LOSS, GRAMS	TIME INTERVAL, SECONDS	INFILTRATION RATE, CM/SEC.	PERMEABILITY, CM/SEC.
2	1/16/91	6938.4	+338	-	-	-
4	1/18/91	6897.9	40.5	163,980	4.25x10-8	1.42x10-8
7	1/21/91	6745.9	152	257,400	1.02x10-7	3.39x10-8
9	1/23/91	6679.4	66.5	159,780	7.17x10-8	2.39x10-8
14	1/28/91	6427.1	252.3	431,340	1.01x10-7	3.36x10-8
16	1/30/91	6312.9	114.2	174,600	1.13x10-7	3.76x10-8
18	2/1/91	6232.1	80.8	162,900	8.54x10-8	2.85x10-8
21	2/4/91	6120.7	111.4	257,340	7.46x10-8	2.49x10-8
25	2/8/91	5921.8	198.9	341,700	1.00x10-7	3.33x10-8
28	2/11/91	5791.0	130.8	253,800	8.88x10-8	2.96x10-8
32	2/15/91	5581.5	209.5	344,160	1.05x10-7	3.50x10-8
42	2/25/91	4942.1	639.4	887,400	1.24x10-7	4.14x10-8

TEST FILL FLOW SUMMARY, TEST FILL #6

DAYS	DATE	COMBINED BAG WEIGHTS, GRAMS	WEIGHT LOSS, GRAMS	TIME INTERVAL, SECONDS	INFILTRATION RATE, CM/SEC.	PERMEABILITY, CM/SEC.
0	1/21/91	6020.2	-	-	-	-
2	1/23/91	5945.7	74.5	159,780	8.03x10-8	2.68x10-8
7	1/28/91	5838.1	107.6	431,340	4.30x10-8	1.43x10-8
9	1/30/91	5772.8	65.3	174,600	6.44x10-8	2.15x10-8
11	2/1/91	5769.7	3.1	162,900	3.28x10-9	1.10x10-9
14	2/4/91	5487.0	282.7	257,340	1.89x10-7	6.30x10-8
18	2/1/91	6232.1	80.8	162,900	8.54x10-8	2.85x10-8
21	2/11/91	5172.5	144.9	253,800	9.80x10-8	3.28x10-8
25	2/15/91	4980.9	191.6	344,160	4.50x10-8	1.50x10-8
35	2/25/91	4570.5	410.4	887,400	7.97x10-8	2.66x10-8

TEST FILL FLOW SUMMARY, TEST FILL #7

DAYS	DATE	COMBINED BAG WEIGHTS, GRAMS	WEIGHT LOSS, GRAMS	TIME INTERVAL, SECONDS	INFILTRATION RATE, CM/SEC.	PERMEABILITY, CM/SEC.
0	1/7/91	6855.2	-	-	-	-
1	1/8/91	6825.0	30.2	73,800	7.00x10-8	2.33x10-8
4	1/11/91	6716.1	108.9	244,800	7.66x10-8	2.55x10-8
7	1/14/91	6364.9	351.2	264,600	2.29x10-7	7.62x10-8
9	1/16/91	6296.8	68.1	172,800	6.79x10-8	2.26x10-8
11	1/18/91	6264.6	32.2	163,980	3.38x10-8	1.13x10-8
14	1/21/91	6177.1	87.5	257,340	5.86x10-8	1.95x10-8
16	1/23/91	6128.9	48.2	159,780	5.20x10-8	1.73x10-8
21	1/28/91	6004.5	124.4	431,340	4.97x10-8	1.66x10-8
23	1/30/91	5952.4	52.1	174,600	5.14x10-8	1.71x10-8
25	2/1/91	5914.1	38.3	162,900	4.05x10-8	1.35x10-8
28	2/4/91	5865.6	48.5	257,340	3.25x10-8	1.08x10-8
32	2/8/91	5790.2	75.4	341,700	3.80x10-8	1.27x10-8
35	2/11/91	5746.3	44.2	253,800	3.00x10-8	9.99x10-9
39	2/15/91	5670.5	75.8	344,160	3.79x10-8	1.26x10-8
49	2/25/91	5463.3	207.2	887,400	4.02x10-8	1.34x10-8

Chen Northern, Inc.

April 10, 1991

Kaiser Engineers Hanford Company
P.O. Box 888
Richland, Washington 99352

ATTENTION: Mr. Steve Peterson

SUBJECT: Additional Information
W-105 Part B Permit Application

Gentlemen:

In accordance with your request of April 9, 1991, we have reviewed the potential for scour and piping in the gravel dikes of the W-105 project.

Our analysis indicates that, under all liner leakage conditions (excluding total loss of the liner), piping or scour are not expected to impact the stability of the gravel dikes.

If you have any questions regarding this letter, or if we can be of further service, please call us.

Respectfully Submitted,
CHEN-NORTHERN, INC.

Brian J. Williams

for Brian J. Williams, P.G.
Geotechnical Engineer

Don J. Burrie, P.E.
Division Manager

91121930752

PIPING AND SCOUR

Piping is a progressive erosion and transport mechanism which may occur when seepage forces through a water-retaining embankment cause erosion at the downstream face of the embankment. The erosion progresses upgradient from the face of the embankment and eventually encounters the impounded fluid, precipitating a massive loss of fluid. The primary factors controlling piping are embankment grain size and the exit velocity of seepage water through the embankment.

Scour is an open-surface erosion mechanism which may occur when free-field water velocities are of sufficient velocity to erode and transport particles, in accordance with Stokes law. The primary factors controlling scour are water velocity and grain size.

The basic assumption behind potential piping or scour is that a phreatic surface has formed through or below the water-retaining embankment, and that the seepage forces along, inside, or downstream (outside) of the embankment are sufficient to erode the embankment soils.

At the W-105 project, under all conditions except complete loss of the liner, no phreatic surface is expected to develop through the embankment which exits outside (downstream) of the embankment. The reasons for this include:

- o Groundwater at the project site is more than 150 feet below the ground surface.
- o The in-place permeability of the native soils is relatively high, ranging from about 5.5x10-4 centimeters per second to 1 centimeter per second (Chen-Northern, 1990).

Considering the relatively deep groundwater and relatively high rate of subsurface permeability, pond leakage (through the liner system) will tend to migrate vertically downward. In the case of this vertical flow, the basic mechanisms precipitating scour and piping cannot occur, and therefore neither piping or scour is expected to impact the stability of the gravel dikes at the W-105 project.

9117210763

RREFERENCES

Chen-Northerrn, Inc., 1990, "Report of Geotechncial Investigation, W-105 242-A Evaporation and Purex Interim Rentention Basins", Report for Kaiser Engineers Hanford Company.

Chen-Northern, Inc.

[illegible]

[illegible]
PO Box 2601
Tri-Cities, Washington 99302

509 547-1671
509 547-1673 Facsimile

March 26, 1991

Kaiser Engineers Hanford Company
P.O. Box 888
Richland, Washington 99352

ATTENTION: Mr. Steve Peterson

SUBJECT: Additional Information for Project W-105, Part B Permit Application

Gentlemen:

In accordance with your request of March 19, 1991, we have prepared the enclosed information. We understand that this information will be used to assist in your compilation of the W-105 Part B permit application. The requested information included the following:

- o Shear strength of the soil-bentonite liner
- o Dike stability using the program STABLE or equivalent
- o Settlement, subsidence, and uplift stresses on the liner

If you have any questions regarding this letter, or if we can be of further service, please contact us.

Respectfully Submitted,
CHEN-NORTHERN, INC.

Brian J. Williams, P.G.
Geotechnical Engineer

Dee J. Burrie, P.E.
Division Manager

SHEAR STRENGTH OF SOIL-BENTONITE

At the direction of Kaiser Engineers Hanford Company, direct shear tests were performed on soil-bentonite composites at 16 percent and 18 percent bentonite content. These tests were performed in October, 1990. The results are summarized below, and are presented in Appendix A, "Shear Strength Test Results".

% Bentonite	Mohr-Coulomb Shear Strength Parameter, Degrees	Cohesion, Kips per Square Foot
16	28	0.55
18	22	0.50

Golder Associates (June, 1990) established a Mohr-Coulomb shear strength parameter of 36 degrees for the soil-bentonite composite using 8 percent bentonite.

A soil-bentonite composite using a nominal 12 percent bentonite was chosen for construction. Based on the above results, our engineers estimated that a conservative <u>minimum</u> Mohr-Coulomb Shear Strength parameter of 30 degrees was appropriate for the nominal 12 percent soil-bentonite combination.

DIKE STABILITY

Dike stability analysis was performed using the program PCSTABL5 (Purdue University, 1986). Stability analysis was performed on the dike slopes prior to and subsequent to soil composite application, under both static and dynamic conditions. The analyses assumed the most critical pond condition of no impoundment fluid. The results of the analyses are summarized below. Assumptions and parameters used in the calculations are presented in Appendix B, "Dike Stability Analysis".

Minimum Factor of Safety for W-105 Dike Slopes Under Static and Dynamic (Horizontal Acceleration) Conditions

Static	0.10 g	0.15 g	0.20 g
1.77	1.33	1.17	1.04

Generally, a minimum factor of safety of 1.5 for static conditions and 1.1 for dynamic conditions is considered appropriate for the type of proposed construction.

The dynamic conditions of 0.1g to 0.15g acceleration have been presented by several sources (Blume, 1971, Dames and Moore, 1986; Golder Associates, 1990, and WSDOE 1990 and 1991). The dynamic condition of 0.2g acceleration is presented by the Uniform Building Code, for the general area of Category 2 Seismic Risk Region.

Based on the results of the calculations, it appears that the W-105 dike slopes will be stable under static conditions, and under the maximum locally anticipated earthquake.

SETTLEMENT, SUBSIDENCE, AND UPLIFT STRESSES ON THE LINER

Uplift Stresses

Based on the design of the soil-bentonite liner, no structural uplift stresses are present.

Uplift stresses from natural sources are expected to have negligible impact on the liner. This conclusion is based, in part, upon the following:

1. The groundwater table is greater than 100 feet below the proposed construction site (Chen-Northern, 1990), mean annual rainfall is less than 6.25 inches (Battelle, 1983), and the average unsaturated permeability of the soils near the basin bottoms ranges from about 5.5x10-4 cm/sec to about 1 cm/sec (Chen-Northern, 1990). Therefore, no hydrostatic uplift forces are expected to develop in the soils underlying the basins.

2. The primary soils under the basins are gravel and sand with no organic componeents (Chen-Northern, 1990). These soils are generally not considered capable of producing gasses through reaction with water or degradation of organic materials. Therefore, uplift from natural gaseous sources is not anticipated.

Subsidence

Subsidence is a process attributed to ground loss into a void such as old mine workings or karst (erosive limestone) topography. Neither of these conditions is present at the W-105 site. Therefore, subsidence is anticipated to be of no consequence for this project.

Settlement

Settlement was calculated for the gravel foundation soils and for the soil-bentonite composite, under the condition of hydrostatic loading from 21 feet of fluid depth. The combined settlement for the soils is expected to be about 1 inch. This amount of settlement is expected to have minimal impact on overall liner or basin stability. Settlement calculations are included in Appendix C, "Calculation of Settlement".

Energy Contract DE-AC06-76 RLO 1830.

2. Blume, John A. and Associates, 1971, "Seismic Analysis of Underground Waste Storage Tanks 241-AZ-101 and -102, Hanford, Washington", Report for Atlantic Richfield Hanford Company.

3. Bowles, J.E., 1988, "Foundation Analysis and Design, 4th ed.", McGraw-Hill Publishing Company.

4. Chen-Northern Inc., 1990, "Report of Geotechnical Investigation, W-105 242-A Evaporation and Purex Interim Retention Basins", Report for Kaiser Engineers Hanford Company.

5. Dames & Moore, 1986, "Report of Geotechnical Investigation, Proposed Shallow Land Disposal Site, Hanford, Washington", Report for U.S. Department of Energy.

6. Fox, E.N., 1948, "The Mean Elastic Settlement of a Uniformly Loaded Area at a Depth Below the Ground Surface", 2nd International Conference on Soil Mechanics and Foundation Engineering; Rotterdam, Holland.

7. Golder Associates, 1990, "Design Report, Project W-025, Radioactive Mixed Waste (RMW) Land Disposal Facility, Non-Drag-Off", report to U.S. Department of Energy.

8. Steinbrenner, W. "Tafeln zur Setzungsberechnung", in Die Strasse (English Translation), Volume 1.

9. Timoshenko, S. and Goodier, J.N., "Theory of Elasticity, 2nd Edition", McGraw-Hill Book Company.

10. Washington State Department of Ecology, Dam Safety Section. Personal conversations with Mr. W. David Cummings, P.E., May 17, 1990, and March 25, 1991.

APPENDIX A

SHEAR STRENGTH TEST RESULTS

(406) 248-9161
FAX (406) 248-9282

REPORT TO:

CHEN-NORTHERN, INC.
ATTN: DEE BURRIE
2214 NORTH 4TH AVENUE
P O BOX 2601
TRI-CITIES, WA 99302

DATE: October 5, 1990
JOB NUMBER: 87-601
SHEET: 1 **OF** 3
INVOICE NO.: 104380

REPORT OF: Direct Shear Tests Kaiser Engineers Project W105 (Job No. 86-1905)

Sample Identification:

On September 28, 1990, we received two soil samples from the subject projects with instructions to perform six direct shear tests. The test specimens were performed at the unit weights, moisture conditions and normal leads specified in your instructions. The tests were prepared and performed in general accordance with ASTM, D-3080 Test procedures. The results of the tests are included on the attached plates.

Reviewed by Larry G. O'Dell

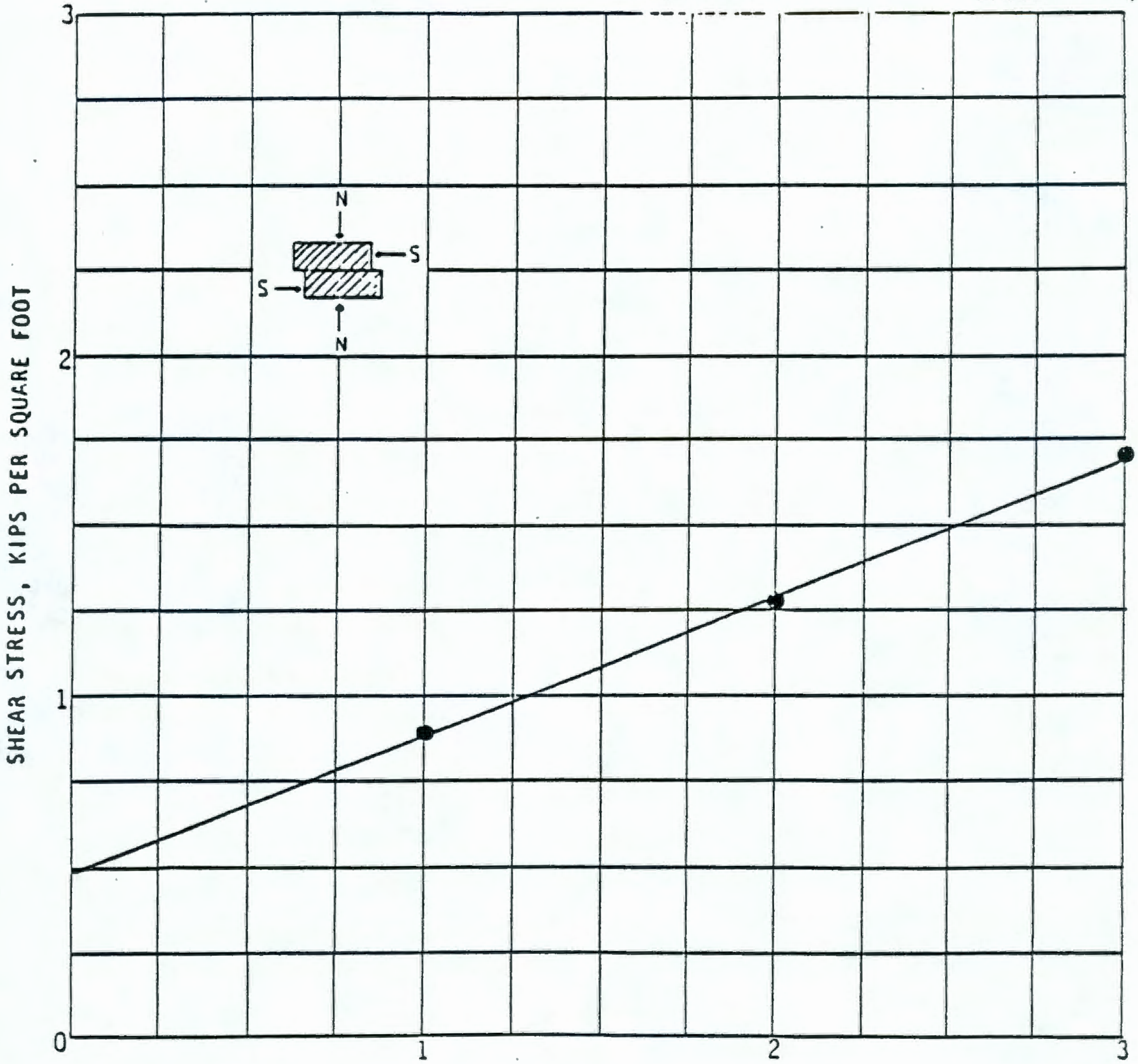
clz

Attachment 3

9112...0770

SAMPLE NO.: [illegible]

MOISTURE CONTENT : 26.3%
CLASSIFICATION : Silty sand with 18% benton
FRICTION ANGLE : 22°
COHESION INTERCEPT: 0.50 Ksf
SHEAR RATE : 0.0072 inch/minute



SATURATED
FIELD MOISTURE CONTENT

KAISER ENGINEERS PROJECT W105	Chen-Northern, Inc.
CHEN-NORTHERN, INC.	Attachment 3-8 JOB NO. 87-601 PLATE NO. 1

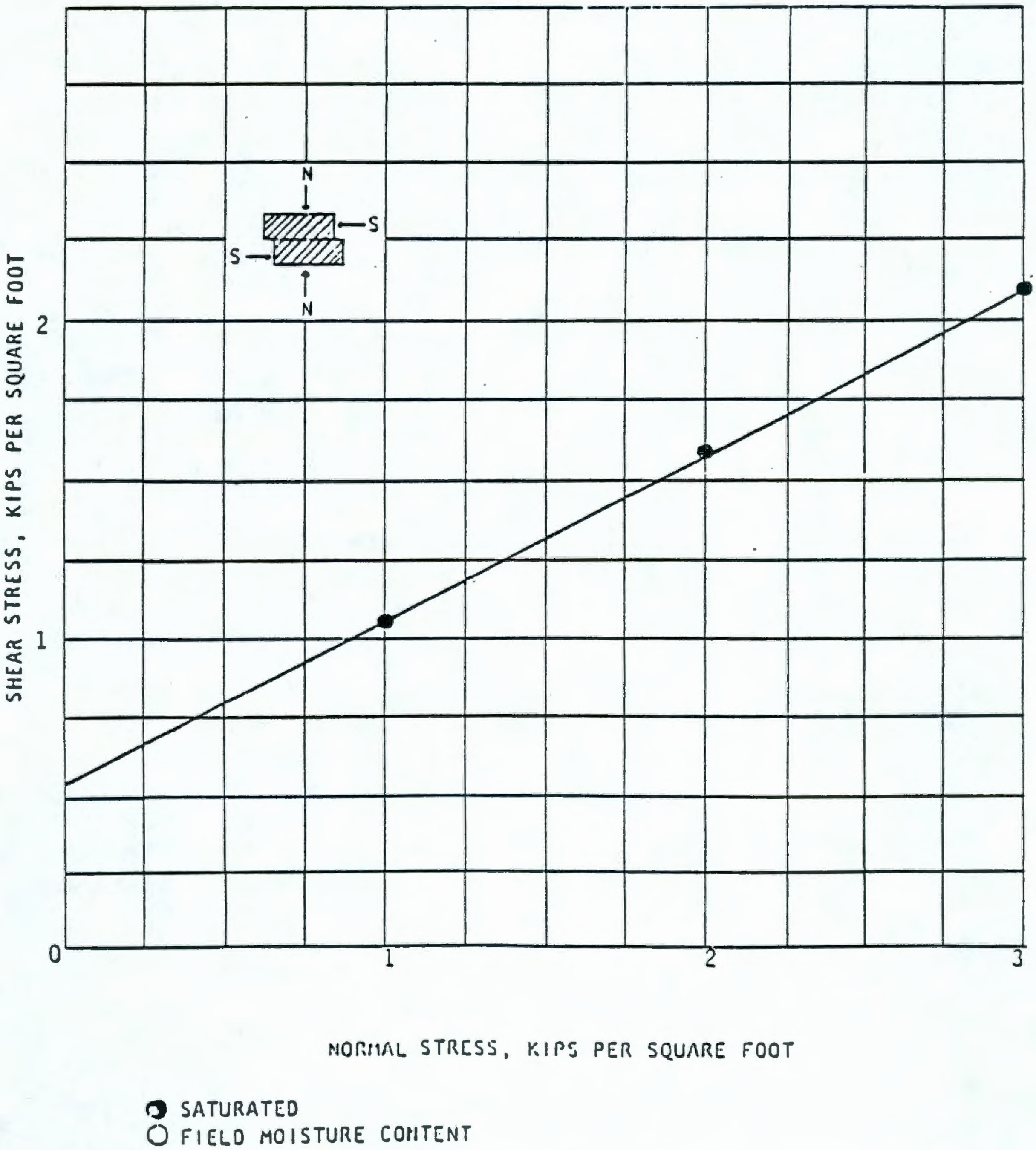
NET 159

9112170771

REMOLDED
DIRECT SHEAR TEST

DRILL HOLE:
DEPTH:
SAMPLE NO.: 39917

MOIST UNIT WEIGHT: 112.9 pcf
DRY UNIT WEIGHT : 92.7 pcf
MOISTURE CONTENT : 21.7%
CLASSIFICATION : Silty sand with 16% bentonit
FRICTION ANGLE : 28°
COHESION INTERCEPT: 0.55 Ksf
SHEAR RATE : 0.0072 inch/minute



KAISER ENGINEERS PROJECT W105	Chen-Northern, Inc.
CHEN-NORTHERN, INC.	Attachment 3-9 JOB NO. 87-601 PLATE NO. 2

NET 159

9 1 1 2 1 7 3 0 7 7 2

APPENDIX B

DIKE STABILITY ANALYSIS

Attachment 3-10

Unit Weight: 135 pounds per cubic foot minimum; Maximum Dry Density as determined by ASTM D1557 Method A was 144.5 pounds per cubic foot (Chen-Northern, 1990)

Mohr-Coulomb Shear Strength Angle: assumed Minimum of 33 degrees.

Soil-Bentonite

Unit Weight: 100 pounds per cubic foot, as determined during test fill construction, 1990 Kaiser Engineers Hanford Company in-house report.

Mohr-Coulomb Shear Strength Angle: assumed Minimum of 30 degrees.

All analyses were performed using 3 horizontal : 1 vertical slopes.

Analyses were performed at an assumed critical state of no fluid in impoundment; fluid in the lined impoundment would tend to act as buttressing effect for the slopes.

Analyses were performed with soils at in-place moisture, and not at saturated conditions. Since the basins are to be lined with a double-composite system, the assumption was made that any leakage through the liner would tend to be minimized by leachate collection system, and would be localized. Localized leakage is not anticipated to affect liner stability.

** PCSTABL5 **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

Run Date: 26 MARCH 1991
Time of Run: 13:45
Run By: B. WILLIAMS
Input Data Filename: W105
Output Filename: W105.OUT

PROBLEM DESCRIPTION KEH W-105 COMPLETED BASIN SLOPES

BOUNDARY COORDINATES

4 Top Boundaries
6 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	.00	20.00	60.00	20.00	1
2	60.00	20.00	150.00	50.00	1
3	150.00	50.00	158.00	50.00	1
4	158.00	50.00	210.00	50.00	2
5	.00	17.00	60.00	17.00	2
6	60.00	17.00	158.00	50.00	2

ISOTROPIC SOIL PARAMETERS

2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	101.0	101.0	.0	30.0	.00	.0	1
2	135.0	135.0	.0	33.0	.00	.0	1

Attachment 3-12

A Critical Failure Surface Searching Method, Using A Random

10 Surfaces Initiate From Each Of 10 Points Equally Spaced
Along The Ground Surface Between X = 60.00 ft.
and X = 100.00 ft.

Each Surface Terminates Between X = 10.00 ft.
and X = 150.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = .00 ft.

5.00 ft. Line Segments Define Each Trial Failure Surface.

1

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Examined. They Are Ordered - Most Critical
First.

* * Safety Factors Are Calculated By The Modified Janbu Method * *

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	77.78	25.93
2	82.75	26.43
3	87.46	28.11
4	91.84	30.52
5	93.30	31.10

*** 1.773 ***

Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	100.00	33.33
2	104.97	33.88
3	109.94	34.40
4	114.81	35.55
5	119.42	37.48
6	123.92	39.67
7	128.22	42.21
8	128.44	42.81

Attachment 3-13

*** 1.780 ***

Failure Surface Specified By 4 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	73.33	24.44
2	78.27	25.21
3	82.96	26.95
4	83.64	27.88

*** 1.802 ***

Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	91.11	30.37
2	96.11	30.30
3	100.96	31.52
4	105.70	33.12
5	109.65	36.18
6	109.98	36.66

*** 1.816 ***

Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	60.00	20.00
2	64.98	19.58
3	69.57	21.57
4	74.10	23.70
5	78.49	26.08
6	78.54	26.18

*** 1.825 ***

Failure Surface Specified By 4 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	60.00	20.00
2	64.99	20.25
3	69.56	22.29

Attachment 3-14

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	95.56	31.85
2	100.50	31.12
3	105.24	32.70
4	109.63	35.10
5	113.06	37.69

*** 1.862 ***

Failure Surface Specified By 4 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	100.00	33.33
2	105.00	33.42
3	109.46	35.67
4	110.02	36.67

*** 1.885 ***

Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	100.00	33.33
2	104.94	32.55
3	109.70	34.08
4	114.48	35.54
5	118.15	38.94
6	118.25	39.42

*** 1.889 ***

Failure Surface Specified By 7 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)

2	82.77	25.71
3	87.77	25.92
4	92.48	27.60
5	97.18	29.29
6	101.11	32.39
7	103.57	34.52

*** 1.975 ***

1

```
                    Y            A      X      I      S            F      T

                   .00       26.25      52.50      78.75      105.00     131.25
 X         .00  +------*-*-+----------+----------+----------+----------+
                -
                -
                -
                -
                -
        26.25   +
                -
                -
                -
                -
                -
 A      52.50   +
                -
                -      * *
                -    ..56
                -   ....56
                -  ......3
 X      78.75   + . .......1
                -. ........13
                -..........01
                -...........01
                - ..........04
                -... ........42
 I     105.00   ..............72.                  . .
                -...   ........24
                -....    .......2.
                -. ... ........29
                - . ..   ........2.
                - .....  .........2
 S     131.25   +   ...... . .....
                -     ............
                -        . ........
                -         . .......
                -               . .*
                -
       157.50   +                  *
                -
                -
                -
                -
                -
 F     183.75   +
                -
                -
                -
```


91121[illegible]30779

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

Run Date:	26 MARCH 1991
Time of Run:	15:42
Run By:	B. WILLIAMS
Input Data Filename:	W1051
Output Filename:	W1051.OUT

PROBLEM DESCRIPTION KEH W-105 COMPLETED BASIN SLOPES

BOUNDARY COORDINATES

4 Top Boundaries
6 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	.00	20.00	60.00	20.00	1
2	60.00	20.00	150.00	50.00	1
3	150.00	50.00	158.00	50.00	1
4	158.00	50.00	210.00	50.00	2
5	.00	17.00	60.00	17.00	2
6	60.00	17.00	158.00	50.00	2

ISOTROPIC SOIL PARAMETERS

2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	101.0	101.0	.0	30.0	.00	.0	1
2	135.0	135.0	.0	33.0	.00	.0	1

Attachment 3-17

A Horizontal Earthquake Loading Coefficient

1	100.00	33.33
2	104.99	33.59
3	109.56	35.63

Of .100 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

Cavitation Pressure = 2100.0 psf

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Irregular Surfaces, Has Been Specified.

100 Trial Surfaces Have Been Generated.

10 Surfaces Initiate From Each Of 10 Points Equally Spaced
Along The Ground Surface Between X = 60.00 ft.
and X = 100.00 ft.

Each Surface Terminates Between X = 110.00 ft.
and X = 150.00 ft.

Unless Further Limitations Were Imposed. The Minimum Elevation
At Which A Surface Extends Is Y = .00 ft.

5.00 ft. Line Segments Define Each Trial Failure Surface.

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Janbu Method * *

Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	82.22	27.41
2	87.19	27.95
3	92.17	28.47
4	97.03	29.62
5	101.64	31.56
6	106.14	33.74
7	110.45	36.29
8	110.67	36.89

*** 1.325 ***

Point No.	X-Surf (ft)	Y-Surf (ft)
1	64.44	21.48
2	69.11	19.67
3	73.59	17.45
4	78.48	18.45
5	83.47	18.86
6	88.32	20.06
7	93.20	21.16
8	97.93	22.79
9	102.37	25.08
10	106.96	27.07
11	111.12	29.85
12	114.46	33.56
13	114.85	38.28

*** 1.673 ***

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	82.22	27.41
2	87.10	26.29
3	91.97	25.17
4	96.97	25.28
5	101.97	25.42
6	106.75	26.86
7	110.55	30.11
8	114.38	33.32
9	117.79	36.98
10	119.38	39.79

*** 1.674 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	95.56	31.85
2	99.22	28.45
3	104.22	28.46
4	108.47	31.09
5	112.77	33.64
6	117.45	35.39
7	122.35	36.38

10	134.72	44.66
11	135.03	45.01

*** 1.708 ***

1

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	73.33	24.44
2	77.08	21.14
3	82.08	20.97
4	87.04	21.58
5	90.52	25.18
6	95.03	27.34
7	99.92	28.38
8	104.68	29.91
9	109.51	31.18
10	112.21	35.39
11	116.00	38.64
12	116.01	38.67

*** 1.743 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	82.22	27.41
2	86.49	24.80
3	91.25	23.27
4	96.24	22.95
5	101.19	23.64
6	105.38	26.37
7	109.15	29.66
8	112.98	32.86
9	116.84	36.04
10	120.31	39.64
11	120.50	40.17

*** 1.746 ***

1

```
              Y           A     X     I     S           F     T

             .00     26.25     52.50     78.75     105.00     131.25

 X      .00 +-----*-*-+---------+---------+---------+---------+
```

Attachment 3-20

```
          26.25 +
                -
                -
                -
                -
                -
   A      52.50 +
                -
                -         * *
                -        ...6
                -       ...6..
                -      . ..6.9
   X      78.75 + .  ...69..
                -.......69.1
                -. .....6041
                -.......60413
                - .......6713
                -........06412
   I     105.00 ..........6831
                -..........6531
                -............636
                -............875
                -  .. ........80.
                - . ... .......8.
   S     131.25 +    .. .........8.
                -      .. . ......8
                -      . .. .........
                -         . . . ......
                -                 . .*
                -
         157.50 +                    *
                -
                -
                -
                -
                -
   F     183.75 +
                -
                -
                -
                -
                -
   T     210.00 +                    *
```

A Horizontal Earthquake Loading Coefficient
Of .150 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

Cavitation Pressure = 2100.0 psf

A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Irregular Surfaces, Has Been Specified.

Attachment 3-21

100 Trial Surfaces Have Been Generated.

Along The Ground Surface Between X = 60.00 ft.
and X = 100.00 ft.

Each Surface Terminates Between X = 110.00 ft.
and X = 150.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = .00 ft.

5.00 ft. Line Segments Define Each Trial Failure Surface.

1

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Janbu Method * *

Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	82.22	27.41
2	87.19	27.95
3	92.17	28.47
4	97.03	29.62
5	101.64	31.56
6	106.14	33.74
7	110.45	36.29
8	110.67	36.89

*** 1.169 ***

Failure Surface Specified By 4 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	100.00	33.33
2	104.99	33.59
3	109.56	35.63
4	111.07	37.02

*** 1.202 ***

Failure Surface Specified By 7 Coordinate Points

6	88.22	20.06
7	93.20	21.16
8	97.93	22.79
9	102.37	25.08

Point No.	X-Surf (ft)	Y-Surf (ft)
1	91.11	30.37
2	96.11	30.57
3	101.10	30.75
4	106.02	31.68
5	110.02	34.67
6	113.93	37.78
7	114.08	38.03

*** 1.311 ***

Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	82.22	27.41
2	87.20	26.95
3	92.19	27.22
4	96.90	28.92
5	101.82	29.78
6	106.64	31.12
7	109.96	34.86
8	112.42	37.47

*** 1.364 ***

Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	95.56	31.85
2	100.52	31.23
3	105.46	31.95
4	110.40	32.76
5	113.73	36.49
6	116.48	38.83

*** 1.393 ***

Failure Surface Specified By 13 Coordinate Points

No.	(ft)	(ft)
1	73.33	24.44
2	77.08	21.14
3	82.08	20.97
4	87.04	21.58
5	90.52	25.18
6	95.03	27.34
7	99.92	28.38
8	104.68	29.91
9	109.51	31.18
10	112.21	35.39
11	116.00	38.64
12	116.01	38.67

***   1.548   ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	82.22	27.41
2	86.49	24.80
3	91.25	23.27
4	96.24	22.95
5	101.19	23.64
6	105.38	26.37
7	109.15	29.66
8	112.98	32.86
9	116.84	36.04
10	120.31	39.64
11	120.50	40.17

***   1.548   ***

```
                Y            A      X      I      S             F      T

                  .00     26.25     52.50     78.75    105.00    131.25

X         .00 +-----*-*-+---------+---------+---------+---------+
              -
              -
              -
              -
              -
        26.25 +
              -
              -
              -
              -
              -
A       52.50 +
              -
              -      * *
```

Attachment 3-24

```
X      78.75 + .....69..
             - .....69.1
             -.......6041
             -.......60413
             -........6713
             .........06412
I     105.00 ..........6831
             -..........6531
             -............636
             -. ..........875
             -    ..........80.
             - .  ...  .......8.
S     131.25 +    ..  .........8.
             -        .   . ......8
             -        .  ..  .........
             -            .   .   .  ......
             -                         .  .*
             -
      157.50 +                             *
             -
             -
             -
             -
             -
F     183.75 +
             -
             -
             -
             -
             -
T     210.00 +                             *
```

A Horizontal Earthquake Loading Coefficient
Of .200 Has Been Assigned

A Vertical Earthquake Loading Coefficient
Of .000 Has Been Assigned

Cavitation Pressure = 2100.0 psf

A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Irregular Surfaces, Has Been Specified.

100 Trial Surfaces Have Been Generated.

10 Surfaces Initiate From Each Of 10 Points Equally Spaced
Along The Ground Surface Between X = 60.00 ft.
and X = 100.00 ft.

Each Surface Terminates Between X = 110.00 ft.
and X = 150.00 ft.

Unless Further Limitations Were Imposed. The Minimum Elevation
At Which A Surface Extends Is Y = .00 ft.

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Janbu Method * *

Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	82.22	27.41
2	87.19	27.95
3	92.17	28.47
4	97.03	29.62
5	101.64	31.56
6	106.14	33.74
7	110.45	36.29
8	110.67	36.89

*** 1.041 ***

Failure Surface Specified By 4 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	100.00	33.33
2	104.99	33.59
3	109.56	35.63
4	111.07	37.02

*** 1.071 ***

Failure Surface Specified By 7 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	91.11	30.37
2	96.11	30.57
3	101.10	30.75
4	106.02	31.68
5	110.02	34.67
6	113.93	37.78
7	114.08	38.03

Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	82.22	27.41
2	87.20	26.95
3	92.19	27.22
4	96.90	28.92
5	101.82	29.78
6	106.64	31.12
7	109.96	34.86
8	112.42	37.47

*** 1.217 ***

Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	95.56	31.85
2	100.52	31.23
3	105.46	31.95
4	110.40	32.76
5	113.73	36.49
6	116.48	38.83

*** 1.243 ***

Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	64.44	21.48
2	69.11	19.67
3	73.59	17.45
4	78.48	18.45
5	83.47	18.86
6	88.32	20.06
7	93.20	21.16
8	97.93	22.79
9	102.37	25.08
10	106.76	27.07

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	82.22	27.41
2	87.10	26.29
3	91.97	25.17
4	96.97	25.28
5	101.97	25.42
6	106.75	26.86
7	110.55	30.11
8	114.38	33.32
9	117.79	36.98
10	119.38	39.79

*** 1.323 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	95.56	31.85
2	99.22	28.45
3	104.22	28.46
4	108.47	31.09
5	112.77	33.64
6	117.45	35.39
7	122.35	36.38
8	126.64	38.96
9	131.10	41.21
10	134.72	44.66
11	135.03	45.01

*** 1.376 ***

Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	86.67	28.89
2	91.47	27.51
3	96.35	26.41
4	101.34	26.68
5	106.23	27.75
6	110.76	29.86
7	113.61	33.97
8	115.76	38.48
9	115.77	38.54

Failure Surface Specified By 16 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	60.00	20.00
2	63.89	16.86
3	68.89	17.01
4	73.85	16.36
5	78.82	15.81
6	83.80	16.26
7	88.78	16.61
8	93.78	16.57
9	98.49	18.27
10	103.35	19.42
11	106.29	23.47
12	109.78	27.05
13	113.72	30.12
14	117.09	33.82
15	118.65	38.57
16	118.99	39.66

*** 1.387 ***

```
              Y           A      X      I      S           F      T

             .00      26.25     52.50     78.75    105.00    131.25

X     .00 +------*-*-+---------+---------+---------+---------+
          -
          -
          -
          -
          -
    26.25 +
          -
          -
          -
          -
          -
A   52.50 +
          -
          -      * *
          -     .0.6
          -    ..06..
          -   ...06..
X   78.75 +  ....06...
          -  ....06..1
          -.....0.6.41
          -.....0.6.413
          -........6713
           .......0..6412
I  105.00 .......0.06831
```

Attachment 3-29

```
                 -   ..........8..
                 - . ... .......8.
S      131.25 +   .. .........8.
                 -     .  . ......8
                 -     . .. ........
                 -       . . . ......
                 -               . .*
                 -
       157.50 +                  *
                 -
                 -
                 -
                 -
                 -
F      183.75 +
                 -
                 -
                 -
                 -
                 -
T      210.00 +                  *
```

APPENDIX C

CALCULATION OF SETTLEMENT

Settlement of Gravel Foundation Soils

At low stresses and strains, settlement of granular soils can be modelled using an elastic response. The settlement of a rectangular base with dimensions B and L on an elastic half-space can be computed using an equation satisfying the Theory of Elasticity (Timoshenko and Goodier, 1951; Steinbrenner, 1934; and Fox, 1948):

$\Delta H = (qo) * B * ((1-\mu**2)/Es) * Is * If$, where

ΔH = settlement, in units of feet

qo = intensity of contact pressure

Since the impounded fluid is essentially water, the unit weight of the impounded fluid is approximately 62.4 pounds per cubic foot. Contact pressure on the liner then equals fluid unit weight multiplied by depth of fluid (21 feet), or 62.4 pounds per cubic foot * 21 feet = 1310 pounds per square foot.

B = least lateral dimension of contact area

The minimum planned width of the basin bottoms is 120 feet, with length L = about 180 feet.

μ = Poisson's Ratio

Poisson's Ration was estimated as 0.4 (Dames & Moore)

Es = Elastic (Young's) Modulus of the soil

Elastic modulus was conservatively estimated as 1,440,000 pounds per square foot based on Bowles, 1988.

Is = Steinbrenner Shape Factor (Steinbrenner)

Settlement was calculated to a depth of 100 feet below the basin bottom. H/B then = 100'/120' = .833, L/B = 180/120 = 1.5. Referring to the attached chart, I1 = 0.132, and I2 = 0.1. Is then = $I1 + ((1-2\mu)/1-\mu)) * I2$, or 0.1653.

= $I1 + ((1-2\mu)/(1-\mu)) * I2$ (see attached sheet)

If = Fox Depth Ratio Factor (see attached chart; equals approximately 0.825).

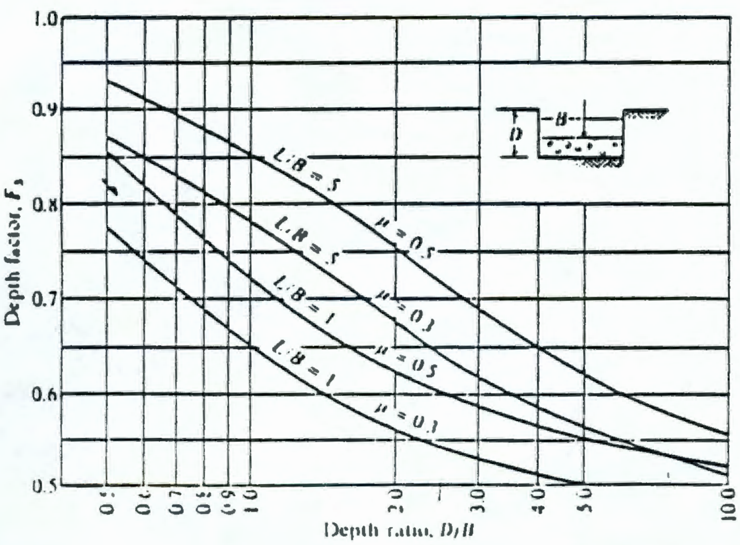
Inserting the above figures into the original equation gives

ΔH =.0125 feet, or about 0.15 inches settlement in the elastic gravel layer below the soil-bentonite layer.

Values of I_1 and I_2 to compute the Steinbrenner influence factor I_s

H/B'		L/B = 1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0
0.2	I_1 =	0.009	0.008	0.008	0.008	0.008	0.008	0.007	0.007	0.007	0.007	0.007
	I_2 =	0.041	0.042	0.042	0.042	0.042	0.042	0.043	0.043	0.043	0.043	0.043
0.4		0.033	0.032	0.031	0.030	0.029	0.028	0.028	0.027	0.027	0.027	0.027
		0.066	0.068	0.069	0.070	0.070	0.071	0.071	0.072	0.072	0.073	0.073
0.6		0.066	0.064	0.063	0.061	0.060	0.059	0.058	0.057	0.056	0.056	0.055
		0.079	0.081	0.083	0.085	0.087	0.088	0.089	0.090	0.091	0.091	0.092
0.8		0.104	0.102	0.100	0.098	0.096	0.095	0.093	0.092	0.091	0.090	0.089
		0.083	0.087	0.090	0.093	0.095	0.097	0.098	0.100	0.101	0.102	0.103
1.0		0.142	0.140	0.138	0.136	0.134	0.132	0.130	0.129	0.127	0.126	0.125
		0.083	0.088	0.091	0.095	0.098	0.100	0.102	0.104	0.106	0.108	0.109
1.5		0.224	0.224	0.224	0.223	0.222	0.220	0.219	0.217	0.216	0.214	0.213
		0.075	0.080	0.084	0.089	0.093	0.096	0.099	0.102	0.105	0.108	0.110
2.0		0.285	0.288	0.290	0.292	0.292	0.292	0.292	0.292	0.291	0.290	0.289
		0.064	0.069	0.074	0.078	0.083	0.086	0.090	0.094	0.097	0.100	0.102
3.0		0.363	0.372	0.379	0.384	0.389	0.393	0.396	0.398	0.400	0.401	0.402
		0.048	0.052	0.056	0.060	0.064	0.068	0.071	0.075	0.078	0.081	0.084
4.0		0.408	0.421	0.431	0.440	0.448	0.455	0.460	0.465	0.469	0.473	0.476
		0.037	0.041	0.044	0.048	0.051	0.054	0.057	0.060	0.063	0.066	0.069
5.0		0.437	0.452	0.466	0.477	0.487	0.496	0.503	0.510	0.516	0.522	0.526
		0.031	0.034	0.036	0.039	0.042	0.045	0.048	0.050	0.053	0.055	0.058
6.0		0.457	0.474	0.489	0.502	0.514	0.524	0.534	0.542	0.550	0.557	0.563
		0.026	0.028	0.031	0.033	0.036	0.038	0.040	0.043	0.045	0.047	0.050
7.0		0.471	0.490	0.506	0.520	0.533	0.545	0.556	0.566	0.575	0.583	0.590
		0.022	0.024	0.027	0.029	0.031	0.033	0.035	0.037	0.039	0.041	0.043
8.0		0.482	0.502	0.519	0.534	0.549	0.561	0.573	0.584	0.594	0.602	0.611
		0.020	0.022	0.023	0.025	0.027	0.029	0.031	0.033	0.035	0.036	0.038
9.0		0.491	0.511	0.529	0.545	0.560	0.574	0.587	0.598	0.609	0.618	0.627
		0.017	0.019	0.021	0.023	0.024	0.026	0.028	0.029	0.031	0.033	0.034
10.0		0.498	0.519	0.537	0.554	0.570	0.584	0.597	0.610	0.621	0.631	0.641
		0.016	0.017	0.019	0.020	0.022	0.023	0.025	0.027	0.028	0.030	0.031
20.0		0.529	0.553	0.575	0.595	0.614	0.631	0.647	0.662	0.677	0.690	0.702
		0.008	0.009	0.010	0.010	0.011	0.012	0.013	0.013	0.014	0.015	0.016
500.0		0.560	0.587	0.612	0.635	0.656	0.677	0.696	0.714	0.731	0.748	0.763
		0.000	0.000	0.000	0.000	0.000	0.000	0.001	0.001	0.001	0.001	0.001

H/B'		L/B = 2.5	4.0	5.0	6.0	7.0	8.0	9.0	10.0	25.0	50.0	100.0
0.2	I_1 =	0.007	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006
	I_2 =	0.043	0.044	0.044	0.044	0.044	0.044	0.044	0.044	0.044	0.044	0.044
0.4		0.026	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024
		0.074	0.075	0.075	0.075	0.076	0.076	0.076	0.076	0.076	0.076	0.076
0.6		0.053	0.051	0.050	0.050	0.050	0.049	0.049	0.049	0.049	0.049	0.049
		0.094	0.097	0.097	0.098	0.098	0.098	0.098	0.098	0.098	0.098	0.098
0.8		0.086	0.082	0.081	0.080	0.080	0.080	0.079	0.079	0.079	0.079	0.079
		0.107	0.111	0.112	0.113	0.113	0.113	0.113	0.114	0.114	0.114	0.114
1.0		0.121	0.115	0.113	0.112	0.112	0.112	0.111	0.111	0.110	0.110	0.110
		0.114	0.120	0.122	0.123	0.123	0.124	0.124	0.124	0.125	0.125	0.125
1.5		0.207	0.197	0.194	0.192	0.191	0.190	0.190	0.189	0.188	0.188	0.188
		0.118	0.130	0.134	0.136	0.137	0.138	0.138	0.139	0.140	0.140	0.140
2.0		0.284	0.271	0.267	0.264	0.262	0.261	0.260	0.259	0.257	0.256	0.256
		0.114	0.131	0.136	0.139	0.141	0.143	0.144	0.145	0.147	0.147	0.148
3.0		0.402	0.392	0.386	0.382	0.378	0.376	0.374	0.373	0.368	0.367	0.367
		0.097	0.122	0.131	0.137	0.141	0.144	0.145	0.147	0.152	0.153	0.154
4.0		0.484	0.484	0.479	0.474	0.470	0.466	0.464	0.462	0.453	0.451	0.451
		0.082	0.110	0.121	0.129	0.135	0.139	0.142	0.145	0.154	0.155	0.156
5.0		0.543	0.554	0.552	0.548	0.543	0.540	0.536	0.534	0.522	0.519	0.519
		0.070	0.098	0.111	0.120	0.128	0.133	0.137	0.140	0.154	0.156	0.157
6.0		0.585	0.609	0.610	0.608	0.604	0.601	0.598	0.595	0.579	0.576	0.575
		0.060	0.087	0.101	0.111	0.120	0.126	0.131	0.135	0.153	0.157	0.157
7.0		0.618	0.653	0.658	0.658	0.656	0.653	0.650	0.647	0.628	0.624	0.623
		0.053	0.078	0.092	0.103	0.112	0.119	0.125	0.129	0.152	0.157	0.158
8.0		0.643	0.688	0.697	0.700	0.700	0.698	0.695	0.692	0.672	0.666	0.665
		0.047	0.071	0.084	0.095	0.104	0.112	0.118	0.124	0.151	0.156	0.158
9.0		0.663	0.716	0.730	0.736	0.737	0.736	0.735	0.732	0.710	0.704	0.702
		0.042	0.064	0.077	0.088	0.097	0.105	0.112	0.118	0.149	0.156	0.158
10.0		0.679	0.740	0.758	0.766	0.770	0.770	0.770	0.768	0.745	0.738	0.735
		0.038	0.059	0.071	0.082	0.091	0.099	0.106	0.112	0.147	0.156	0.158
20.0		0.756	0.856	0.896	0.925	0.945	0.959	0.969	0.977	0.982	0.965	0.957
		0.020	0.031	0.039	0.046	0.053	0.059	0.065	0.071	0.124	0.148	0.156
500.0		0.832	0.977	1.046	1.102	1.150	1.191	1.227	1.259	1.532	1.721	1.879
		0.001	0.001	0.002	0.002	0.002	0.003	0.003	0.003	0.008	0.016	0.031



Influence factor I_F for footing at a depth D. Use actual footing width and depth dimension for D/B ratio.

Settlement of Soil-Bentonite Layer

Golder Associates (1990) originally performed consolidation testing of the proposed W-105 borrow sand, at a bentonite content of 8 percent. Consolidation at approximately 1300 psf was about 2.5 percent. Moisture content of the as-tested material was 19.6 percent, which was approximately the same as the composite determined by Chen-Northern (1990). Dry unit weight of the soil prior to consolidation testing was 104.2 pounds per cubic foot, as compared to nominal densities of 100 pounds per cubic foot obtained during construction of the liner test fill (Kaiser Engineers Hanford Company in-house report). Although the bentonite in the Golder consolidation test was 8 percent, and the proposed nominal bentonite content is 12 percent, it is our opinion that consolidation of the two soils will be similar. Therefore, consolidation of the proposed 36-inch thick soil-bentonite bottom liner can be calculated as:

$$\Delta H = (2.5/100) * 36 \text{ inches, or approximately } 0.9 \text{ inches.}$$

ENGINEERING CHANGE NOTICE

Page 1 of 3

1. ECN ~~152746~~
Proj. ECN W105-88

2. ECN Category (mark one)	3. Originator's Name, Organization, MSIN, and Telephone No.		4. Date
Supplemental ☒ Direct Revision ☐ Change ECN ☐ Temporary ☐ Supersedure ☐ Discovery ☐ Cancel/Void ☐	J. A. Shipman, KEH, E6-24, 6-8532		4-11-91
	5. Project Title/No./Work Order No. 242A EVAP & Purex Interim Retnn Basin/W105/ER156	6. Bldg./Sys./Fac. No. 200E/242AL	7. Impact Level 3
	8. Document Number Affected (include rev. and sheet no.) See Below	9. Related ECN No(s). N/A	10. Related PO No. N/A

11a. Modification Work	11b. Work Package Doc. No.	11c. Complete Installation Work	11d. Complete Restoration (Temp. ECN only)
☐ Yes (fill out Blk. 11b) ☒ No (NA Blks. 11b, 11c) Unknown	~~N/A~~ Unknown	N/A Cog. Engineer Signature & Date	N/A Cog. Engineer Signature & Date

12. Description of Change

1) H-2-79590 SH 1 REV 2

a) Revise Drawings as shown on Page 3.

RONALD G. HOLLENBECK
STATE OF WASHINGTON
22750
REGISTERED
PROFESSIONAL ENGINEER

Ronald G. Hollenbeck 4-12-91

13a. Justification (mark one)	13b. Justification Details
Criteria Change ☒ Design Improvement ☐ Environmental ☐ As-Found ☐ Facilitate Const. ☐ Const. Error/Omission ☐ Design Error/Omission ☐	Client request based on a letter from WSDOE (dated April 3, 1991 from T. L. Nord, WSDOE to S Wisness, DOE). Drawing was modified to clarify the thickness of soil/bentonite in the bottom of the basin. The thickness of the soil/bentonite was not changed.

14. Distribution (include name, MSIN, and no. of copies)

KEH DISTRIBUTION
Const Doc Cntl E2-50
Engrg Doc Cntl E6-52

WHC DISTRIBUTION
Project Files R1-28
J. K. Epperley S0-05
L. R. Hall S1-54
STATION 10 A3-87

E. R. Hamm S4-65
M. N. Islam R3-08
J. L. Rusk R3-30
D. R. Shreve R3-30
L. R. Tollbom (PE) R3-30
T. S. Vail R1-51
J. D. Williams H4-57
R. B. Wurz S5-15

DOE
A. G. Lassila A5-18

RELEASE STAMP

OFFICIAL RELEASE (13)
BY WHC
DATE APR 12 1991
STATION 12

1. ECN (use no. from pg. 1): W105-88

15. Design Verification Required	16. Cost Impact ENGINEERING	16. Cost Impact CONSTRUCTION	17. Schedule Impact (days)
☐ Yes	Additional ☒ $ 270	Additional ☐ $ N/A	Improvement ☐ N/A
☒ No	Savings ☐ $	Savings ☐ $	Delay ☐

18. Change Impact Review: Indicate the related documents (other than the engineering documents identified on Side 1) that will be affected by the change described in Block 12. Enter the affected document number in Block 19.

N/A

SDD/DD	☐	Seismic/Stress Analysis	☐	Tank Calibration Manual	☐
Functional Design Criteria	☐	Stress/Design Report	☐	Health Physics Procedure	☐
Operating Specification	☐	Interface Control Drawing	☐	Spares Multiple Unit Listing	☐
Criticality Specification	☐	Calibration Procedure	☐	Test Procedures/Specification	☐
Conceptual Design Report	☐	Installation Procedure	☐	Component Index	☐
Equipment Spec.	☐	Maintenance Procedure	☐	ASME Coded Item	☐
Const. Spec.	☐	Engineering Procedure	☐	Human Factor Consideration	☐
Procurement Spec.	☐	Operating Instruction	☐	Computer Software	☐
Vendor Information	☐	Operating Procedure	☐	Electric Circuit Schedule	☐
OM Manual	☐	Operational Safety Requirement	☐	ICRS Procedure	☐
FSAR/SAR	☐	IEFD Drawing	☐	Process Control Manual/Plan	☐
Safety Equipment List	☐	Cell Arrangement Drawing	☐	Process Flow Chart	☐
Radiation Work Permit	☐	Essential Material Specification	☐	Purchase Requisition	☐
Environmental Impact Statement	☐	Fac. Proc. Samp. Schedule	☐		☐
Environmental Report	☐	Inspection Plan	☐		☐
Environmental Permit	☐	Inventory Adjustment Request	☐		☐

19. Other Affected Documents: (NOTE: Documents listed below will not be revised by this ECN.) Signatures below indicate that the signing organization has been notified of other affected documents listed below.

Document Number/Revision	Document Number/Revision	Document Number/Revision

20. Approvals

OPERATIONS AND ENGINEERING	Signature	Date
Cog./Project Engineer	WHC/CE Daniel R. Shiree for L. R. Tollbom WHC/PE	4-12-91
Cog./Project Engr. Mgr.	S. P. Bailey	4/12/91
QA	Larry R. Hall	4-12-91
Safety		
Security		
Proj.Prog./Dept. Mgr.		
Def. React. Div.		
Chem. Proc. Div.		
Def. Wst. Mgmt. Div.		
Adv. React. Dev. Div.		
Proj. Dept.		
Environ. Div.		
IRM Dept.		
Facility Rep. (Ops)		
Other		

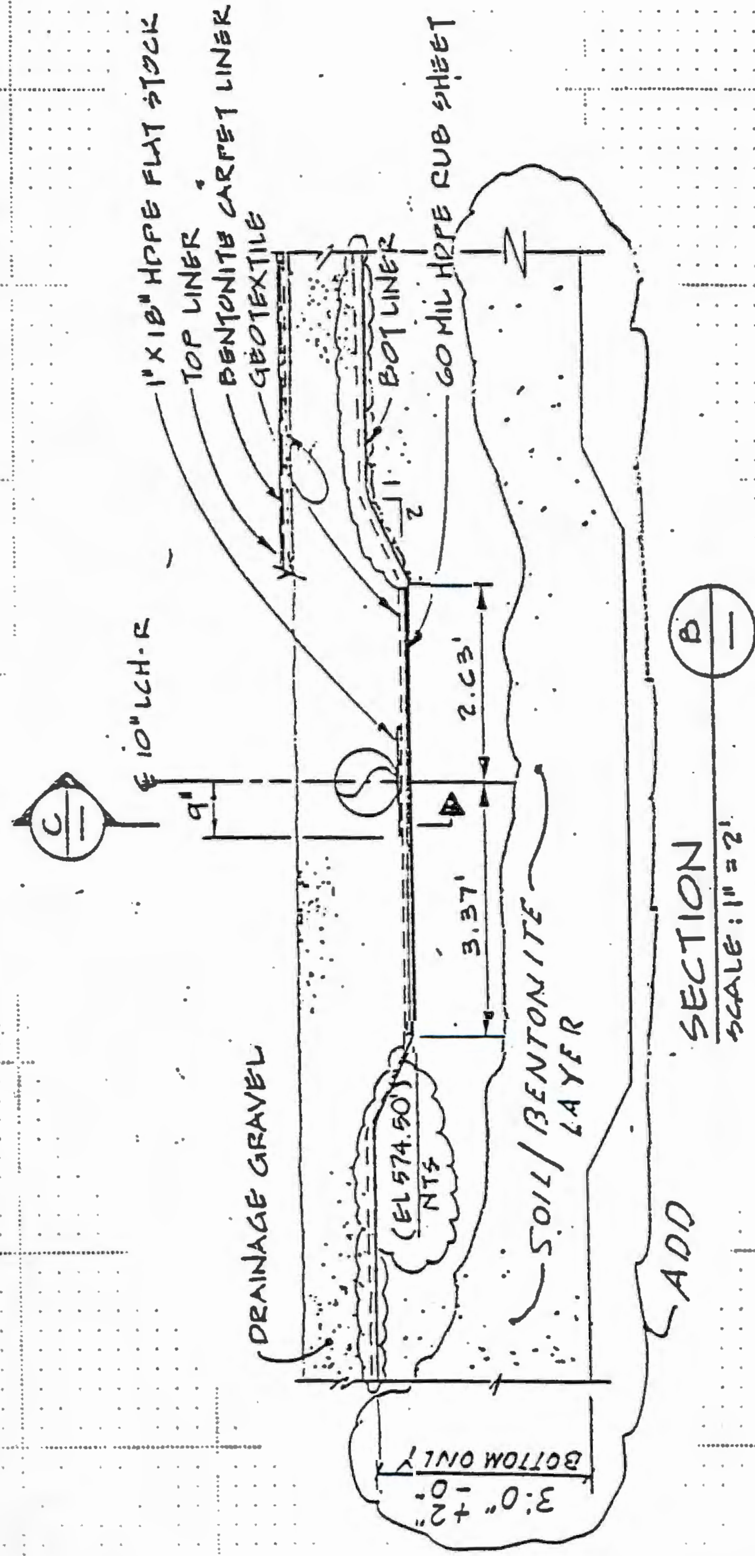
ARCHITECT-ENGINEER	Signature	Date
PE	S. Petts	4/12/91
QA	TD Hamp / Fd Fedi	~~4-12-91~~ 4-12-91
Safety	D. Lundgren	4-11-91
Design	R. A. Hollenbeck	4-11-91
ENV ~~Other~~	R. A. Hollenbeck	4-11-91

DEPARTMENT OF ENERGY

ADDITIONAL

9 1 1 2 1 7 0 7 9 9

KAISER ENGINEERS HANFORD	ENGINEERING CHANGE NOTICE SKETCH					
Ref. Dwg.	**Sh.**	**Rev.**	**Prepared By**	**Checked By**	**ECN No.**	**Page**
H-2-79590	1	2	J.A. SHIPMAN	E.A. Graley	ECN-W105-88	3/3



Justification (continued)

- **Treatment and disposal of land disposal restricted waste in interim storage is required by Tri-Party Agreement Milestones M-26-03 and M-26-04**

M-26-03	**Cease discharge of 242-A Evaporator process condensate to LERF Units**	**December 1994**
M-26-04	**Remove all hazardous waste residues from the 242-A Evaporator LERF Units**	**June 1995**

DISTRIBUTION COVERSHEET

Author	Addressee	Correspondence No.
SH Wisness/DOE-RL	TL Nord/Ecology	Incoming: 9102148

Subject Response to April 3, 1991 Request, Regarding the Liquid Effluent Retention Facility (LERF)

Internal Distribution

Approval	Date	Name	Location	w/att
		Correspondence Control	A3-01	*X
		RJ Bliss		
		WH Hamilton, Jr. (Assignee)		X
		DE Kelley	R1-48	
		RE Lerch		X
		HE McGuire		
		RD Morrison	B2-35	
		LL Powers	B2-35	
		TB Veneziano		
		EDMC	H4-22	X

*Copies of the attachment may be obtained from either EDMC H4-22 or Correspondence Control A3-01.

9 1 1 2 1 7 3 0 8 0 2