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WISCONSIN
HENRY A.
KOCH
E - 15273
MADISON,
WIS.
PROFESSIONAL ENGINEER

Henry A Koch

IN-FIELD CONDITIONS REPORT
PHYSICAL SITE DESCRIPTION
DELAFIELD SANITARY LANDFILL
SANITARY TRANSFER AND LANDFILL COMPANY, INC.
NORTHWEST 1/4, SECTION 27, T7N, R18,
TOWN OF DELAFIELD
WAUKESHA COUNTY, WISCONSIN

C 7091

WARZYN
ENGINEERING INC

Consulting Engineers • Civil • Structural • Geotechnical • Materials Testing • Soil Borings • Surveying

1409 EMIL STREET, P.O. BOX 9538, MADISON, WIS. 53715 • TEL. (608) 257-4848

January 19, 1979
C 7091

Mr. Roger Klett, Solid Waste Investigator
Southeast District
Department of Natural Resources
9722 W. Watertown Plank Road
Milwaukee, WI 53226

Re: In-Field Conditions Report-
Delafield Sanitary Landfill

Dear Mr. Klett:

Attached is our report discussing existing in-field conditions at the Delafield Sanitary Landfill. The investigation included a subsurface exploration program, laboratory testing, topographic survey, and analysis of data to aid in preparation of the text manuscript and associated drawings.

We will soon be submitting a report on water quality in and about the Delafield Sanitary Landfill assessing groundwater quality conditions. We anticipate the report will be completed during February, 1979.

We request that you review the attached report and provide us with your comments. For your convenience, we have forwarded copies of this report directly to the Bureau of Waste Management, DNR, Madison. If you have any questions or desire clarification or further information regarding any aspects of the report, please contact us.

Very truly yours,

WARZYN ENGINEERING INC.

Daniel R. Viste

Daniel R. Viste
Chief Hydrogeologist

DRV/dmf
Enclosure: Report - In-Field Conditions Report - Physical Site Description
cc: Bureau of Waste Management, Madison (2 copies)
Ron Nickel, Sanitary Transfer and Landfill Co.
James Morgan, Attorney

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NORTHWEST 1/4, SECTION 27, T7N, R18, TOWN OF DELAFIELD
WAUKESHA COUNTY, WISCONSIN

SUMMARY AND CONCLUSIONS

The following summary and conclusions regarding the Delafield Sanitary Landfill are presented based on our in-field investigation, analysis of data, and subsequent preparation of this report for the Sanitary and Transfer Landfill Company, Inc.:

1. The Delafield Sanitary Landfill encompasses approximately 35 acres and is located in the south one-half of the south one-half of Section 22 in the northwest one-quarter (OF THE NW 1/4) of Section 27, T7N, R18E, City of Delafield, Waukesha County, Wisconsin.

2. Approximately 2.2 million cubic yards of refuse has been disposed since 1955. Averge fill depth varies from 40 to 60'. The in-place waste is confined between Elevations 985', USGS Datum, to 1080', USGS Datum.

3. The site is underlain by 20 to greater than 100' of glacially-derived, unconsolidated sediments consisting of interbedded layers of lacustrine clays, glacial outwash, and glacial tills. The unconsolidated deposits generally exhibit permeabilities greater than 10^{-3} cm./sec. in the previously filled area.

4. The Niagara dolomite is the underlying bedrock and slopes downward to the northeast from an elevation of 985' USGS Datum in the southern portion of the site to 925' USGS Datum in the northeast corner of the site.

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5. Groundwater flow is downward (recharge) in the northern and southern sections of the site and lateral or horizontal in the central portion of the site. Groundwater flows away from the site in all directions but south, with approximately 90% of the groundwater flow exiting the site to the northwest. Groundwater elevations under the waste disposal area range from 970', USGS Datum, to 980', USGS Datum.

6. Groundwater flow directions are similar in the unconsolidated glacial aquifer and the Niagara dolomite aquifer indicating that the unconsolidated and bedrock aquifers are interconnected. Nine of the 25± private water supply wells within 1200' of the landfill are downgradient from the landfill.

7. A clay cap to be placed over the entire landfill upon abandonment is calculated to materially limit future generation of leachate.

INTRODUCTION

A. Site Location

The Delafield Sanitary Landfill property is located in the south 1/2 of the south 1/2 of Section 22 and the northwest 1/4 of Section 27, T7N, R18E, Town of Delafield, Waukesha County. The present active fill area is located in the southwest 1/4 of the northwest 1/4 of Section 27. By roadway description, the landfill is located approximately 1/2 mile east of State Highway 83, immediately adjacent and south of Interstate Highway I-94, approximately 1/2 mile west of County Trunk Highway "HE" and approximately 1/2 mile northwest of County Trunk Highway "E". A vicinity map, part of Drawing No. C 7091-1, indicates the relationship of the site to municipal and county boundaries. Surrounding land ownership, also shown on Drawing C 7091-1, outlines property owners within approximately 1 mile of the landfill. The regional topographic map, part of Drawing No. C 7091-1, indicates the site location in relation to the surrounding geomorphologic and man-made features.

B. Purpose and Scope

This report and the accompanying drawings (C 7091-1, -15, -17, -18, -19, -20, -21, -22, -23, -24 and -27) have been prepared for Sanitary Transfer and Landfill Company to document in-field conditions regarding existing subsoil, bedrock, surface water, and groundwater conditions. The results of the in-field conditions investigation are essential in evaluating the feasibility of continued waste disposal operations at the existing Delafield Sanitary Landfill.

The scope of work incorporated a subsurface exploration program, groundwater monitoring, laboratory soil testing, topographic mapping, and the analysis and preparation of a text manuscript and accompanying drawings presenting our conclusions and recommendations. Some specific work elements are included in the following descriptions.

1. A series of soil borings and test pit excavations were performed to aid in interpretting the stratigraphy of the investigation area.

2. The soil borings were instrumented with monitoring wells and piezometers to determine vertical flow gradients, the configuration of the water table surface, and the piezometric surface in the bedrock aquifer.

3. Field baildown tests were performed to determine the permeability of the glacial deposits and bedrock aquifer.

4. Laboratory testing of the soils, including Atterberg limits, soil gradations, soil density, natural moisture, and permeability tests were performed to determine various soil properties.

5. A topographic base map was periodically up-dated to provide information regarding existing waste disposal conditions, well elevations, and to aid in the development of engineering design recommendations.

6. The analysis of data and report compilation were done to evaluate site conditions and to prepare an in-field conditions report.

C. Previous and On-going Work

Numerous studies have been completed and submitted to the Wisconsin Department of Natural Resources (DNR) by Warzyn Engineering Inc. regarding conditions at the Delafield Sanitary Landfill. Portions of this report will reference previously submitted reports listed below which should be addressed if particular details are desired for specific topics referenced. Reports which have been completed since 1977 are listed below and are briefly described as to content.

1. Summary Report-July 20, 1977

   The Summary Report presented information regarding existing site conditions based on an analysis of all site investigations performed prior to 1977. Also included in the report was a discussion of up-dated topographic mapping and preliminary methane gas testing. An analysis of the existing hydrogeologic conditions was presented including accompanying drawings which summarized all available data and presented recommendations on additional work necessary to document the in-field conditions. The present in-field conditions report is a direct continuation of the original summary report recommendations and details conclusions about hydrogeologic conditions which could not be thoroughly addressed with data available at the time of the summary report.

2. Abandonment of On-Site Private Wells - August 18, 1977

   The report describes the abandonment of two on-site private water supply wells located in a proposed fill area. The abandonment of the wells was requested by the DNR. Well abandonment was performed by Warzyn Engineering Inc., and well abandonment reports submitted to the appropriate agencies as well as photodocumentation of the well abandonment program.

3. Documentation of Clay Liner - September 21, 1977

The report describes the basic construction of the then-in-place liner and tests performed upon specific portions of the liner. The report also outlined a method to determine liner effectiveness by placing a monitoring well above the liner, a suction lysimeter immediately below the liner, and a monitoring well in the upper portion of the water table as a nested installation.

4. Loss of Leachate Head - May 18, 1978

An evaluation was performed regarding the development of leachate head upon the liner in the area instrumented for documentation of the clay liner. The report indicates the expected and determined head build-up of leachate upon the liner and an analysis of flow through the liner.

5. Proposed Clay Capping, Southwestern Exterior Portion - September 25, 1978

The report describes the existing cover soils at the Delafield Landfill and proposes the placement of a specially designed impermeable cap to minimize the infiltration of precipitation. Also described, is the proposed method of drainage control and the development of a sedimentation basin.

6. Methane Gas Study - Delafield Sanitary Landfill - September 25, 1978

The report describes previous and present investigations regarding methane gas generation and migration. The study involved the instrumentation of numerous gas monitoring probes around the site and interpretation of conditions regarding gas production and migration at the site. Also presented were recommendations regarding proposed methods of gas control at the site.

7. Waste Leaching Investigation - September 29, 1978

This report evaluates the stage of decomposition of in-place wastes and the relative strength of leachate expected from such waste as compared to existing conditions at the Delafield Landfill. Conclusions are presented regarding the future generation potential of leachate.

8. Documentation of Clay Liner and Leachate Collection System - November 20, 1978

The report discusses and summarizes the documentation and construction of a clay liner extension and leachate collection system. The liner was constructed to minimize leachate migration and uncontrolled leachate seeps.

The above described reports have all been previously submitted to the DNR. Work presently on-going includes the analysis of available water quality data obtained since 1973 at the Delafield Sanitary Landfill and adjacent private wells. The report will describe existing conditions regarding water quality in and about the Delafield Landfill and will be submitted during February, 1979.

D. Site Operations Up-Date

A detailed description of general site operation practices has been previously presented within the Summary Report and will not be discussed here. However, additional information is available regarding the current landfill practices and is briefly described. In-place waste volumes at the site have been computed to be approximately 2.2 million yards. The area in which present waste fill operations are occurring is shown on Drawing No. C 7091-17. It is anticipated that future filling operations will continue over the recently placed clay liner, which is also outlined on the previously mentioned drawing. The present fill operations are primarily confined to an area underlain by a clay liner and leachate collection system to facilitate the collection of leachate from eminating through the base of the site.

SITE INVESTIGATION

A. Topographic Mapping

An up-dated topographic map, prepared during May, 1977, was submitted with the Summary Report, dated July 20, 1977, by Warzyn Engineering Inc. The drawing presented the location of active filling, previously filled areas of the landfill, soil borings, and then existing groundwater monitoring wells.

Additional wells installed during the course of the present investigation were located by stadia methods on January 6, 1978. The well elevations were obtained by level circuit. Topographic up-date information was also obtained to define the then existing limits of fill and existing borrow areas, and areas modified since the previous topographic mapping.

On May 11, 1978, Warzyn Engineering Inc. issued a drawing (Drawing C 7091-16) illustrating the field control monuments to be established at the site. The original proposed control was modified during the field survey. The present field control monuments and up-dated topographic survey have been combined on to the drawing entitled "Site Topography and Boring Locations." Original Drawing No. C 7091-16 has been voided in lieu of the new Drawing C 7091-17 showing the existing control monuments and up-dated topography. The additional topographic data was obtained during the period from May 15 through May 19, 1978, and May 23, 1978. The control monument program included the establishment of permanent horizontal and vertical control for future work at the site. In addition, horizontal controls were established in the active working area to aid in the construction of the liner, excavation of borrow areas, and other work associated with day-to-day site operations. Other horizontal control points were established to aid people on-site in identifying their location. These control points are useful in the gas testing program and in making visual observations on site.

The topographic base map was drawn to a 2' contour interval using USGS datum as was used in previous contour maps. To better define topographic conditions within the existing borrow area, additional topographic up-date information was obtained during June, 1978 and has been added to the site topography and boring location drawing (C 7091-17).

B. Field Exploration

1. Borings and Instrumentation

a. Previous Work

Prior to December, 1977, a total of 22 soil borings, 9 instrumented as monitoring wells, were performed by Wisconsin Testing Laboratories of Milwaukee, Wisconsin. All of the soil borings were compiled and presented on geologic cross sections within the Summary Report, previously submitted. Of the soil borings performed, B1 through B12 ranged in depth from 10 to 20' and were performed during the period from November 21 through November 25, 1974 and on February 4, 1974. The soil borings addressed the potential for future fill operations in the area south of the present fill area. During the period from January 23 through January 31, 1975, four monitoring wells were installed, P1, P2, P3, and P4, which were instrumented to monitor the direction of groundwater flow. The results of the soil boring programs during 1974 and 1975 were submitted to the DNR by EMCON and Associates of San Jose, California, in May, 1975.

During the period from May 12 through June 2, 1976 an additional 6 monitoring wells (E1, E2, E2B, E3, E4, and E5) were installed at the site. The wells were installed to monitor the water table and obtain water samples for water quality analysis. The soil boring logs for these wells were also submitted to the DNR by EMCON and Associates, during 1976.

b. Present Work

A review of available information by Warzyn Engineering Inc. and DNR indicated that additional subsoil, bedrock and groundwater information was needed to evaluate existing site conditions. The following work was performed: A.) Instrumentation of additional monitoring wells to define the groundwater mound beneath the landfill and quality of the groundwater in the shallow flow system, B.) Instrumentation of wells within the bedrock aquifer to determine vertical gradients and groundwater quality in the Niagaran aquifer, C.) Construct two specially sealed wells through refuse to determine the nature of soils underlying the site, D.) Instrument the liner with a well-suction lysimeter nest to evaluate the effectiveness of the liner. The information regarding liner effectiveness has been submitted with the "Documentation of Clay Liner" report.

To provide detailed subsoil, bedrock, and hydrogeologic data in and about the Delafield Landfill, 18 additional soil borings were performed during the period from December 1, 1977 through January 18, 1978. The borings were performed with truck-mounted and all-terrain mounted rotary rigs using the procedures outlined in Appendices A and B.

Of 19 borings performed, soil Borings E2B, E3B, E5B, E6, E7, E8, E9B, E10, E11, E12, E13, and E15 were standard penetration borings while the remaining borings E6B, E9, E13B, E13C, E14, E15, and E15L were auger borings. All of the borings with the exception of E13A, E13B, E13C, and E15L were instrumented with 2" ID PVC pipe with a PVC well screen bottom section.

Water table wells, wells which monitor the surface of the saturated zone, were installed at well locations E6, E7, E8, E9, E10, E11, E12, and E15. A suction lysimeter was installed at E15L to aid in documenting liner effectiveness. Leachate head monitoring wells were installed at Locations E14A and E15A to determine the build-up of leachate upon the liner. Piezometers, for purposes of this report, are defined as a well sealed so as to monitor groundwater potential at a specific point in the groundwater system. Piezometers were installed at Locations E2C, E3B, E5B, E6B, and E9B. Well construction details are shown on the geologic cross sections which include Drawings C 7091-19, -20, and -21.

Rock drilling was performed within the piezometer wells sealed in the upper bedrock. Rock cores were recovered in Wells E2C and E9B, while rock drilling for a distance from 5 to 15' into the bedrock was performed at the remaining piezometer installations.

2. Test Pit Excavations

On June 7, 1978, three test pits were performed at the Delafield Sanitary Landfill under the direction of a hydrogeologist. The test pits were designated TP1, TP2, and TP3, and are shown on Drawing C 7091-17. The test pits were performed to determine subsoils adjacent to and under the newly installed liner and also the depth to groundwater below the base of the liner at Locations TP1 and TP2. At TP3, the test pit was performed to determine the soil types existing at the surface adjacent to the south end of the western lobe of waste fill.

C. Groundwater Monitoring

Water levels were obtained by Warzyn Engineering Inc., on 11 occasions since instrumentation of the additional wells during December, 1977, and January, 1978, on the following dates: December 15, 1977, January 4, 1978, January 13, 1978, February 20, 1978, March 9, 1978, March 28, 1978, April 7, 1978, April 27, 1978, May 24, 1978, June 5, 1978, and August 30, 1978. Water levels were measured to ±0.01'. The results of elevations obtained during the water level monitoring program, well elevations, and well types are recorded in Appendix C. The wells are available for continued monitoring. A water table map was constructed from water elevations obtained March 9, 1978, see Drawing C 7091-23.

Baildown tests were also performed on selected wells on June 7, 1978 by Warzyn Engineering personnel. Three wells instrumented within the unconsolidated deposits, including E2B, E3, E4 and E5 were tested. The results of the baildown tests performed on Wells E2B, E4, and E5 are included in Appendix D.

D. Laboratory Testing

The soil samples obtained in the field investigation program were classified in the laboratory by a hydrogeologist. Representative samples of the unconsolidated sediments were analyzed in our laboratory in order to determine hydraulic conductivity, soil gradations, and to assist in soil classification. The soil tests which were performed included falling head permeability tests, grain size analyses, Atterberg Limits, and modified Proctor compaction tests. The modified Proctor compaction tests as well as various permeability tests were performed on the clay liner. These tests where applicable are included in the In-Field Conditions Report and were presented in the "Documentation of Clay Liner and Leachate Collection System" report.

Approximately 35 samples were tested for soil gradations including washed sieves and hydrometers. Atterberg limits were also performed on 16 representative samples to determine plasticity characteristics. The soil gradations and Atterberg Limits tests are summarized in Table 2, Physical Characteristics of Unconsolidated Deposits, and presented where applicable on the geologic cross sections, see Drawings C 7091-19 through C 7091-21. In addition, all of the soil gradation curves are shown on Drawings C 7091-A4 through A9; C 7091-A25, -A26, -A27, -A37, -A38, -A39, and C 7091-A52 through -A67, see Appendix E.

SITE DESCRIPTION

A. Regional and Local Setting

The Delafield Sanitary Landfill lies within the central portion of Waukesha County and within the Kettle Moraine area. On a regional basis, this is an area of irregular topography with many hills, swales and depressions. This rugged topography was formed by glacial actions. A number of major lakes are located to the northeast and northwest of the landfill including Pewaukee, Nagawicka, Pine, Okauchee, Oconomowoc, upper and lower Nemahbin, etc. Because of the numerous lakes within this area and the resulting urbanization of rural areas, significant portions of land associated with the regional setting of the landfill are under development. Located immediately west of the landfill is the Fairfield Addition while the Cherokee Woods Development is located south and east of the landfill. The landfill site is located within the boundaries of the Kettle Moraine State Forest. Three small drainage courses eminate from the vicinity of the landfill and are discussed in a following section dealing with surface water hydrology. Located approximately 1/2 mile southeast of the landfill is a large peat bog approximately 1/2 square mile in area.

On a more local basis the physical characteristics of the existing landfill site are a function of the Interlobate Moraine features and the resulting activities of sand and gravel extraction and landfilling operations. Originally the landfill existed as a hill in a series of hills running in a north-south direction. Resulting graveling operations at the landfill created a depression which is subsequently being filled via landfill operations. Partial filling of the depression has occurred with the majority of the waste being placed in an L-shaped mound along the north and west borders of the site.

Access to the site is obtained from Silver Nail Road, which is a frontage road along the south side of Interstate Highway I-94. Continued development within the Fairfield and Cherokee Woods developments has resulted in a number of houses being constructed in close proximity to the landfill. The closest home, located west of the landfill, is approximately 100' from the property boundary and approximately 325' from the present active fill operations. Approximately 25 private water supply wells are within 1200' of the landfill, having been installed after commencement of waste operation and during the period since waste disposal operations began. That portion of the landfill site located in the southern portion of Section 22 is utilized for equipment storage, garage facilities and office space.

B. Topography

Topography of the Delafield Sanitary Landfill is depicted on Drawing C 7091-17. Topography south of a line through Borings B12, B8 and B3 has not been changed by the landfill operations or the previous graveling operations and is representative of the topography as it previously existed in the area of the landfill. The southern portion of the landfill site is composed of numerous hills and swales with elevations varying from 1020 to 1060, USGS Datum. Slopes in this undisturbed area vary from 8% to 30%. That portion of the landfill located in the central area of the site reflects topography that has been primarily affected by the landfilling operations. The basic land feature present is an L-shaped mound. The top of the mound, the toe of the slope on the northern face, and the toe of the slope within the southern boundary are at Elevations 1070, 1000 and 1020, respectively, USGS Datum. The slope along the northern and eastern exterior portion of this mound varies from 10% to 25% while the interior slope along the southern and eastern portions of the mound varies from 25% to 50%. The main access road traverses the mound from east to west, and then southward to the active fill area. The active borrow area is located due south of the haul road near Boring B7. The base of the borrow area is at approximate Elevation 980, USGS Datum, while the land surface to the south varies from 1030 to 1050 USGS Datum. To the interior of the landfill site the topography is varied reflecting the stockpiling of gravel and clay materials and the excavation of borrow. Clay materials stockpiled for the construction of a liner are located between Monitoring

Well E7 and Boring B13 and appear as a flat-topped hill at approximate Elevation 1040, USGS Datum. A pile of gravel materials exists northeast of Boring 6 approximately 20' in height and approximately 1/2 acre in area. Clay stripping operations have been conducted in the area immediately east of the newest liner installation.

Topography along the northern boundary of the landfill site and in the vicinity of the maintenance and office buildings reflects construction activities for Interstate Highway I-94 and activities associated with the construction of the office facilities. It is our understanding that no waste materials have been placed north of a line between Monitoring Wells E2 and E3. Portions of the land vacated in the vicinity of the garage maintenance and office facilities areas are leveled and graveled while that portion of the site located in the northwest corner is quite rugged reflecting the original topography of the area. A large cut, for the eastbound lane of Interstate I-94 is present along the northern property line.

C. Subsoils

1. General Stratigraphy

Subsoils at the Delafield Landfill site are glacially derived unconsolidated deposits which range in thickness from 30' at Boring P4 to approximately 105' near Monitoring Well E1. The thinnest accumulation of glacial deposits occurrs under the topographic low in the central portion of this site, primarily as a result of the removal of sand and gravel. The thickest deposits are found under the topographic highs located in the northeastern and southern (undisturbed) sections of the site. The unconsolidated deposits consist of glacial till, glacial outwash (ice-contact deposits), and lacustrine type deposits which extend to bedrock. Glacial till is an unsorted mixture of gravel, sand, silt and clay whereas the on-site glacial outwash is a poorly sorted to well-sorted, heterogeneous mixture composed primarily of sand and gravel with little or no fines. The lacustrine deposits are lake deposits of silt and clay.

The soils were deposited within the interlobate areas between the Green Bay and Lake Michigan ice lobes of Pleistocene glaciation. The till type deposits were deposited directly by glacial ice and are not sorted by particle size. During warmer climactic regimes, rapid melting of glacial ice resulted in the deposition of the coarser outwash deposits. In areas of ponded waters the lacustrine silts and clays were deposited.

Due to the diversity of soils within the landfill area, the glacial deposits have been categorized into three groups, based on their relative permeabilities. Permeable deposits are defined as soils exhibiting permeabilities greater than 10^{-3} cm./sec. and include the coarser sand and gravel deposits. Semi-permeable deposits include the glacial tills and silty outwash deposits having permeabilities between 10^{-3} and 10^{-6} cm./sec. The lacustrine silts and clays are classified as impermeable with permeabilities of less than 10^{-6} cm./sec.

A plan view map of the predominant soil permeability types, below the zone of saturation, is shown on Drawing No. C 7091-18. The geologic cross sections indicate the interpretted soil permeability types in and between borings. The geologic cross-section locations are also shown on Drawing No. C 7091-18. Geologic cross-sections are presented on Drawing Nos. C 7091-19, -20, and -21.

The general stratigraphy of the unconsolidated deposits in the landfill area is as follows:

TABLE 1

GENERAL STRATIGRAPHY OF ON-SITE UNCONSOLIDATED DEPOSITS

STRATUM	THICKNESS	ORIGIN
Refuse	0 - 80'	Waste Disposal
Topsoil	0 - 4'	Residual
Clay Liner	0 - 6'	On-Site Clay
Permeable Units	0 - 120'	Glacial Outwash
Semi-permeable Units	0 - 70'	Glacial Till-Poorly Sorted Outwash Deposits
Impermeable Units	0 - 30'	Lacustrine

a. Topsoils

Due to previous sand and gravel mining operations and present day landfilling practices, the topsoil over much of the area has been removed. Where present, it ranges in thickness from 2" to 4'. The lithology of the topsoil can be described as a brown to black silty clay which has developed from the uppermost glacial deposit immediately underlying the soil.

b. Waste

Based on soil borings and visual observations, the waste at the Delafield Landfill site consists of municipal type refuse. The areal extent of the waste is shown on Drawing C 7091-17. The surface area covered by waste is approximately 35 acres, reaching a maximum thickness of 80' near Boring E10 and consisting of a volume equal to 2.2 million yards.

c. Clay Liner

The implacement and physical characteristics of Clay Liners I through IV have been described in the report, "Delafield Sanitary Landfill, Documentation of Clay Liner", dated September 21, 1977. The following discussion is a summary and update of the information presented in that report.

The areal extent of the clay liner is shown on Drawing C 7091-17. The thickness of the liner, where present, generally ranges from 2' to 6' in thickness. The source of the earthen material used to construct the liner is from on-site lacustrine silt and clay, excavated from the area in the vicinity of Boring B6. The liner soils have been tested for Atterberg limits, grain size, permeability, and moisture content. The results of the soils analyses, excepting permeability and moisture content, are listed in Table 2.

The average soil gradations for clay liner materials based on four liner samples is 2% gravel, 6% sand, 61% silt, and 31% clay. The average Atterberg limits were 38.3 for a liquid limit and 17.9 for a plastic index. The USCS classification of the clay liner material is

CL. CL type materials have an estimated permeability ranging from 10^{-6} to 10^{-8} cm./sec. Falling head permeability tests of the liner material resulted in permeabilities ranging from 10^{-7} to 10^{-8} cm./sec. The moisture content of the liner material from four samples ranged from 18% to 21%.

d. Permeable Units

The permeable units are defined as having a permeability greater than 10^{-3} cm./sec., generally consisting of sand and gravel outwash deposits. The deposits range from a brown to gray, fine to coarse sand, some to little gravel, little silt, trace clay, (SW-SM); to a brown, fine to medium sand, trace to some gravel, little to trace silt, (SP-SM, SP); to a brown to gray, fine to large gravel and sand, to some sand, trace silt, (GW). The permeable outwash deposits are the most dominant type of soil present at the Delafield Landfill site, ranging in thickness from 0 to 120'. The deposits are thickest under topographic highs and thinnest under topographic lows.

A field bail down test was performed at Well E4, which is screened in the permeable material, with a resultant permeability (K) value of 8×10^{-4} cm./sec. While a permeability of 8×10^{-4} cm./sec. is not greater than 10^{-3} cm./sec., it is very close to the permeable range. The soil gradation tests in the upper part of Boring E4 resulted in very low P200 values indicating permeable soils. No soil analyses were performed below the zone of saturation. Since soil samples are not available for additional analysis, the subsoils at Boring E4 are classified as permeable. A field test was also attempted in Well E3 but the well could not be bailed fast enough to develop a significant drawdown. Based on the average soil gradation presented below, and using Hazen's Approximation, we estimate the average permeability of the permeable subsoils to be $\pm$ 5 x 10^{-3} cm./sec. Most of the soil samples tested have P200 values in the range of 10 to 12%. Grain size analyses were performed on 15 samples from borings within the permeable material. The resultant grain sizes ranged from 0 to 83% gravel, 19 to 90% sand, and 1 to 12% P200 material, see Table 2. The average grain size was 25% gravel, 66% sand, and 9% P200 material.

TABLE 2

PHYSICAL CHARACTERISTICS OF UNCONSOLIDATED DEPOSITS

CLAY LINER

BORING	DEPTH	GSA[1]	P200[2]	LL[3]	PI[3]	USCS[4]
E-15	13.5'	8 /10 /53 /29	82	38.5	16.4	CL
--	Liner III S-2	1.9/ 8.6/58.6/30.9	89.5	35.2	15.5	CL
--	Liner III S-4	0 / 1 /66.4/32.6	99	38.4	18.7	CL
--	Liner III S-5	0 / 3.3/66.5/30.2	96.7	40.9	20.8	CL

PERMEABLE UNITS

BORING	DEPTH	GSA[1]	P200[2]	LL[3]	PI[3]	USCS[4]
E-1	0 - 25'	83 /16 / 1 / 0	1	--	--	GW
E-3	50'	0 /90 /10	10	--	--	SP-SM
E-4	13'- 39'	45 /53 / 2 / 0	2	--	--	SP
E-3	7'- 20'	6 /83 /11 / 0	11	--	--	SP-SM
E-6	55'	25 /67 / 8	8	--	--	SP-SM
E-7	40'	38 /52 /10	10	--	--	SM-SP
E-7	50'	0 /89 /11	11	--	--	SM-SP
E-7	60'	1 /89 /10 / 0	10	--	--	SP-SM
E-8	20'	30 /59 / 9 / 2	11	--	--	SW-SM
E-8	60'	17 /73 / 8 / 2	10	--	--	SP-SM
E-9	20'	16 /78 / 6	6	--	--	SW-SM
E-9	35'	33 /59 / 8	8	--	--	SW-SM
E-10	70'	44 /45 /11	11	--	--	SW-SM
E-11	75'	43 /46 /11	11	--	--	SW-SM
E-12	30'	0 /88 /12	12	--	--	SP-SM

SEMI-PERMEABLE UNITS

BORING	DEPTH	GSA[1]	P200[2]	LL[3]	PI[3]	USCS[4]
E-2	10'	0 /82 /18	18	--	--	SM
E-2	30'	28 /45 /27	27	--	--	SM
E-2	35'	43 /42 /15	15	--	--	SM
E-2	50'	17 /44 /39	39	--	--	SC
E-4	10'- 12'	15 /39 /39 / 7	46	--	--	SM
E-5	10'	30 /38 /21 /11	32	19.7	5.5	SC-SM
E-5	45'	13 /53 /30 / 4	34	--	--	SM
E-6	40'	24 /31 /31 /14	45	12.4	1.6	SM
E-10	100'	0 /85 /15	15	--	--	SM
E-11	80'	42 /37 /14 / 7	21	--	--	SM
E-15	16'	0 /52 /28 /20	48	--	--	SM
E-6	65'	18 /41 /22 /20	42	12.7	3.4	SM
E-5	60'	53 /31 /13 / 3	16	--	--	GM
E-6	15'	49 /37 /12 / 2	14	--	--	GM
E-13	15.5'	36 /30 /21 /13	34	24.8	9.0	SC

Table 2 (Cont.)

IMPERMEABLE UNITS

BORING	DEPTH	GSA[1]	P200[2]	LL[3]	PI[3]	USCS[4]
E-2	5'	8 / 9 /59 /24	83	39.6	18.2	CL
E-2	20'	0 / 1 /72 /27	99	21.5	6.2	CL-ML
E-7	10'	--	--	44.0	19.7	CL
E-11	100'	13 /14 /41 /32	73	18.2	4.2	CL-ML
E-5	4'- 6'	7 /17 /65 /11	76	--	--	CL
E-4	2'- 4'	3 /11 /56 /30	86	--	--	CL
FD-A	--	1 / 8 /65 /27	91	30.9	11.4	CL
FD-B	--	4 /10 /58 /28	86	35.3	14.9	CL
FD-C	--	0 / 0 /72 /28	100	34.5	14.0	CL
ST	--	0 /19 /51 /30	81	38.6	18.4	CL

LEGEND:

GSA[1] = Grain Size Analysis (e.g.)

% gravel / % sand / % silt / % clay
20 / 30 / 40 / 10

% gravel / % sand / % silt and clay
20 / 60 / 20

P200[2] = Percent finer than No. 200 sieve (silt and clay)

LL[3] = Liquid Limit

PI[3] = Plastic Index

USCS[4] = Unified Soil Classification System

FD = Field Density test location

ST = Shelby tube sample

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e. Semi-Permeable Deposits

The semi-permeable deposits are defined as having permeabilities between 10^{-3} and 10^{-6} cm./sec. These deposits are generally ice-contact deposits, containing appreciable amounts of fines, or glacial till and range from fine to coarse sand, some silt, clay and gravel, (SM); to a fine to coarse sand, some silt, little gravel and clay, (SM); to a fine to medium sand, some silt, well-sorted, (SM); to a brown gravel and sand or gravel, some sand, little to some silt, (GM). The semi-permeable deposits range in thickness from 0 to 70' and are typically interbedded with the permeable units previously described. Two field baildown tests were performed in Wells E-2B and E-5, which are screened in semi-permeable material. The resultant permeabilities were 3.2×10^{-5} cm./sec. and 6.4×10^{-5} cm./sec. at Wells E-2B and E-5, respectively.

Grain size analyses were performed on 15 samples of semi-permeable material with the resulting grain size ranges of 0 to 53% gravel, 30 to 85% sand, and 15 to 46% P200 material. The average gradation was 24% gravel, 46% sand, and 30% P200 material.

f. Impermeable Deposits

The impermeable deposits, for the purpose of this report, are those deposits with permeabilities of less than 10^{-6} cm./sec. The deposits are lacustrine clays and silts ranging from a brown silty clay, thinly laminated to massive bedding, (CL); to gray silty clay, trace fine sand, (CL); to a gray clayey silt, some gravel, (CL-ML); to dense brown silt with thin layers of fine to coarse silty sand (CL-ML). The impermeable deposits range in thickness from 0' to 30'.

To determine the physical properties of the impermeable deposits, grain size analyses, and Atterberg limits tests were performed on 9 samples. The grain sizes ranged from 0 to 13% gravel, 0 to 17% sand, 51 to 72% silt, and 11 to 30% clay. The average gradation for the 9 samples is 4% gravel, 9% sand, 61% silt, and 26% clay. The liquid limits ranged from 18.2 to 44 with an average value of 32.8. The plastic index ranged from 4.2 to 19.7 with an average value of 13.4. The Atterberg limits indicate the clays to have a low to medium plasticity with a low shrink-swell potential.

The previous subsoil sections were designed to outline the physical characteristics of the subsoils present at the Delafield Landfill site. Due to the complexity of the glacial stratigraphy present at the site, adequately describing the field relationships between soil types is very difficult. To more clearly illustrate soil stratigraphy, the following discussion will describe each of the 7 geologic cross-sections presented with this report, followed by a general discussion of the glacial deposits. The geologic cross-sections are shown on Drawings C 7091-19 through -21 and the location of each cross-section is indicated on Drawing C 7091-18. Table 3 is a summary of the cross section locations and borings used to compile each section.

2. Description of Geologic Cross-Sections

a. Cross Section A-A

Section A-A is primarily composed of permeable sands, ranging up to 120' in thickness near the center of the section. The southern portion of the section shows an impermeable clay capping the permeable

TABLE 3

SUMMARY OF CROSS-SECTION LOCATIONS

SECTION	BORINGS	LOCATION
A-A	E-2, E-2B, E-2C, E-1, E-8, E-7, P-3, B-12, B-13	North-south trending section along eastern margin of site.
B-B	E-12, E-11, B-5, E-9, E-9B, B-6, P-4, B-6, B-6B, B-4	North-south trending section through center of site.
C-C	E-3, E-3B, E-10, E-4, D-9, D-15, E-5, E-5B	North-south trending section along western margin of site.
D-D	E-2, E-2B, E-2C, E-12 E-3, E-3B	East-west trending section along northern margin of site.
E-E	E-1, E-11, E-10	East-west trending section through north-central portion of site.
F-F	E-10, E-14A, E-9, E-9B, B-11, E-7, P-3	Northwest-southeast trending section through center of site.
G-G	E-3, E-3B, E-10, E-4, D-9, D-15, E-5, E-5B	East-west trending section through center of site.

sands, ranging up to 16' in thickness near E-7. Borings E-2, E-2B, and E-2C at the northern end of the section indicate interbedded semi-permeable and impermeable deposits extending down to bedrock. The total thickness of the interbedded soils at E2C is approximately 80'.

b. Cross Section B-B

The northern half of Section B-B is underlain by permeable sand with interbedded till/silty outwash and clay at Boring E-11. The central half of the section is capped with a clay liner. The southern half of the section is dominated by a thick till/silty outwash deposit up to 60' in thickness. A mound of sand and gravel overlies the till/silty outwash at E-6.

c. Cross Section C-C

Section C-C is primarily outwash deposits with interbedded clay and glacial till/silty outwash at E-3, E-10, and E-4. The clays and tills are not believed to be continuous over the length of the section. There is an abrupt stratigraphic change near E-5. At this location, the massive sand and gravel deposit grades into a glacial till/silty outwash. The southern extent of the till/silty outwash has not been defined.

d. Cross Section D-D

Section D-D is characterized by a large sand and gravel unit located in the central portion near E-12. The eastern end of the section is dominated by a thick accumulation of interbedded till/silty outwash and clay. The sand and gravel at the western end is capped by a large mound of glacial till/silty outwash.

e. Cross Section E-E

Section E-E is primarily sand at the eastern end of the section up to 110' thick. The sand becomes interbedded with till/silty outwash and clay in the central and western portions of the site near E-10. There is a clay layer approximately 8' in thickness directly underlying the refuse at E-10. The semi-permeable units at E-11 and E-10 have not been correlated due to the large difference in grain size.

f. Cross Section F-F

The southern section of F-F is characterized by a large sand deposit capped by an 8' to 10' clay layer. The sand unit grades into a glacial till/silty outwash near B-11. Although no borings actually penetrated the till/silty outwash on the section, it is believed to be present due to its presence in other nearby borings. In the vicinity of Boring E-9B, the till/silty outwash grades into another sand unit in a northerly direction. The sand unit becomes interbedded with a till/silty outwash unit at E-10. The till/silty outwash unit is at least 20' in thickness. A clay layer overlies the sand at E-10, in turn, overlain by refuse. The northern extent of the clay and till/silty outwash units at E-10 are not well defined, though they are believed to be localized deposits because they are not present at E-3B.

g. Cross Section G-G

The glacial stratigraphy depicted by Section G-G is quite straight-forward. The eastern half of the section is a sandy type deposit approximately 80' in thickness. In the central portion of the section near E-15, the sandy deposit grades into a semi-permeable till/silty outwash which is the dominant deposit over the western half of the section. The till unit increases in thickness in a westerly direction, being approximately 35' thick near E-15 and 70' in thickness near E-5B.

3. Summary of Subsoils

The subsoils at the Delafield Landfill are quite complex as is evident from the preceding discussion of the geologic cross-sections, however, a few generalizations can be made. The bulk of the subsoils at the site can be classified as permeable sand type deposits. The sands underlie a majority of the northern half of the site. In the vicinity of E-10 and E-11, there is a glacial till/silty outwash deposit at Elevation 980 USGS Datum. The till may be continuous between E-10, E-11, and E-2. Associated with this till are clays, with the clays being directly beneath the till/silty outwash at E-11, overlying the till at E-10 and interbedded with the till at E-2. The clay is the uppermost deposit at B-10, directly underlying the refuse, however the aerial extent of the clay unit is small.

Another till/silty outwash unit is present in the southwest quarter of the site and is defined by Borings E-15, E-5, B-4, E-6B, and E-13C. This second large till/silty outwash unit extends down to bedrock over this area, with an average thickness of approximately 40'. The till is capped by a sand mound near E-6. The southeastern quarter of the site is underlain by a thin clay unit, approximately 5 to 10' in thickness. The clays in the southeast quarter are generally underlain by sandy type deposits.

D. Bedrock Geology

The consolidated rock underlying Waukesha County includes Precambrian to Silurian age strata. A brief description of the underlying bedrock presented in this report is based on References 1 and 2. The Precambrian basement complex underlies the entire state of Wisconsin and is found at a depth of approximately 2000' in the vicinity of the Delafield Landfill site. The basement complex is composed of crystalline granites, quartzites and slates and does not constitute an aquifer in southern Wisconsin.

The Cambrian aquifer lies unconformably on top of the Precambrian basement complex. The Cambrian aquifer consists of Cambrian age sandstones and Ordovician age dolomites and sandstones, having a total thickness of approximately 1500'. The units within the Cambrian aquifer in descending order are: the Galena Dolomite-Platteville formation, St. Peter Sandstone, Prairie du Chien group, Trempealeau formation, Franconia sandstone, Galesville sandstone, Eau Claire sandstone, and Mt. Simon sandstone. The Cambrian aquifer is capped by the relatively impermeable Maquoketa shale, which is Ordovician in age. The Cambrian aquifer is a source of groundwater for most of the large commercial, industrial and municipal users throughout the area.

The Maquoketa shale is a dolomitic, blue-gray shale which hydraulically separates the Cambrain aquifer from the overlying water table aquifers. The shale is not an aquifer. The total thickness of the shale in the vicinity of the site is approximately 200' and is present at a depth of 150 to 200' below the land surface.

The Niagara dolomite aquifer overlies the Maquoketa shale. This aquifer generally thickens to the east and commonly exceeds 200' in thickness in the eastern quarter of Waukesha County. The permeability of the dolomite is a secondary permeability due to fractures and solution cavities within the dolomite. Many of the fractures on the upper surface of the dolomite were formed by the loading and unloading of the tremendous weight of glacial ice during the various glacial and interglacial stages of Pleistocene glaciation and isostatic rebound since glaciation.

In the landfill site, various borings encountered the dolomite. Table 4 is a summary of the borings in which bedrock was encountered, the elevation of the bedrock surface, and the depth to the bedrock surface.

TABLE 4

DEPTH TO BEDROCK

BORING	DEPTH TO BEDROCK	ELEVATION OF BEDROCK SURFACE (USGS DATUM)
P-3	68'	961.3
P-4	40'	960.0
E-2C	82'	922.9
E-3B	68'	940.5
E-5B	75'	947.6
E-7	65'	961.3
E-9B	41'	947.3
E-6B	75'	987.0

The bedrock surface elevations compiled in Table 4 have been used to construct a bedrock surface map (See Drawing C 7091-22). The bedrock surface map indicates that the dolomite surface slopes gently downward to the northeast at an angle of approximately 2%.

The permeability of the dolomite has been roughly determined using specific capacity pumping tests of nearby private wells obtained from drillers' logs. The specific capacity is defined as the ratio of pumping rate to draw down. William C. Walton, Groundwater Resource Evaluation[3], has constructed a series of graphs which allows one to estimate permeability from a known specific capacity. The permeability of the dolomite calculated in this manner ranges from 10^{-3} to 10^{-5} cm./sec. The permeability is dependent, primarily, on the degree to which the dolomite is fractured (secondary permeability).

The dolomite was cored in Borings E-2C and E-9B. Two parameters were used to determine the physical nature of the rock; rock recovery (RR) and rock quality designator (RQD). The RQD is defined as the percentage of the rock core 4" in length or longer. The RQD values determined were 43% at E9B and 90% at E-2C with corresponding rock recovery values of 81% and 100%. The values indicate that the dolomite is quite competent at E-2C. At E-9B the rock is quite fractured in the upper 10' of the bedrock system. Significant problems were encountered in drilling Well E-9B due to water losses in the upper dolomite bedrock.

Unconsolidated glacial drift overlies the Niagara dolomite and is present at the land surface. The characteristics of the glacial material have been previously described in the subsoil section. The glacial material is an aquifer and is hydraulically connected with the underlying Niagara dolomite. The combined glacial drift and Niagara dolomite aquifer is the primary aquifer for domestic, commercial, small municipal, and subdivision water supplies in the area.

E. Surface Water Hydrology

No intermittent streams or permanent drainage ways carry runoff off-site from the landfill area. Due to the high permeability of the surficial soils and the irregular topography with numerous localized depressions, the majority of the precipitation that falls on the site infiltrates into the subsoils with little generation of surface water runoff. Precipitation falling on the northern slope of the landfill collects in a depression near E-12 (See Drawing C 7091-17). The depression is approximately 1 1/4 acres in size and has no outlet. The runoff which collects in this depression either infiltrates into the groundwater system or is returned to the atmosphere through the process of evapo-transporation. The depression acts as a settling basin for any suspended material carried by surface water runoff from the northern slope of the previously landfilled area. A minor component of runoff generated along the northern margin of the property may reach an intermittent stream that eventually flows to the west end of Lake Pewaukee.

Surface water runoff west of the site enters a number of small, flat depressions which eventually lead to a large glacial drainage way in the eastern half of Section 28. An intermittent stream does not exist in this channel in the vicinity of the landfill site. Man-made improvements further south along the channel have created an artificial drainage way which eventually leads to Skuppernung Creek. Flow in Skuppernung Creek is southwesterly toward Dutchman Lake.

Precipitation falling on the interior slopes of the landfill, west margin of the fill area, and central portions of the property is ponded in a number of depressions, but generally flows to a depressional area north of Boring B7. The runoff from this area will soon be redeveloped to direct surface water runoff to a proposed settling-infiltration basin in the area between Borings B7 and B11, reference previously submitted Clay Capping Report. Surface runoff from the southern portion of the landfill site, south of a line through Borings B-3, B-8, and B-12, is ponded in depressions near Borings B-13, B9, and P2. A small portion of surface water eminating from the southeastern portion of the landfill property may eventually reach a large peat bog located approximately 1500' from the southeast corner of the property. Surface water from the peat bog enters Brandy Brook and flows in a southerly direction. Surface water from the eastern portion of the previously landfilled area is channelized along the access road and eventually reaches small depressions near Boring E-1.

F. Groundwater Hydrology

1. Regional Flow in Consolidated Aquifer

The Interlobate moraine, in which the Delafield Landfill is located, with its extensive, coarse, granular deposits and irregular drainage systems, supplies significant recharge to the Niagara dolomite and sand and gravel combined aquifers within the Delafield area.

A regional groundwater map, Drawing C 7091-27, has been developed using water level data collected by the Private Water Supply Section of the DNR during 1977 and modified by Warzyn Engineering personnel utilizing on-site bedrock monitoring wells. The water levels shown reflect piezometric head in the Niagara aquifer. The direction of groundwater flow is determined from water elevations within wells sealed at varying depths (vertical gradients) and at various locations (horizontal gradients). The movement of groundwater occurs as a result of potential differences within the saturated soil or rock material, with groundwater migrating from areas of high potential (E-6B) to areas of low potential (downgradient wells). Potential is defined as the amount of potential energy exhibited at a specific point within the groundwater system as monitored by a well sealed at a specific depth. Losses of potential energy occur through friction dissipated as heat energy as the groundwater moves through the soil horizon. The hydraulic gradient is a measure of potential loss per unit distance of travel.

The most dominant feature of regional groundwater flow system in the vicinity of the Delafield Landfill site is a major groundwater divide within the water table and bedrock aquifers. A groundwater divide can be imagined as a vertical plane through an aquifer where groundwater flows away from the plane at 90°. Groundwater divides often times result in topographically high areas of high recharge in permeable subsoils. Groundwater usually flows outward and downward away from the divide towards areas of lower water table elevations. The groundwater flow in the vicinity of the landfill site is not unlike a divide, but with one exception. As stated, a groundwater divide is characterized by groundwater flow outward from a divide in two directions. As evident from the regional groundwater map (Drawing C 7091-27), the flow system resembles a cross between a groundwater mound and groundwater divide, with flows radiating outward in all directions but south.

The regional map indicates the generalized groundwater flow pattern but is not sufficiently detailed to indicate potential migration directions for groundwater within the disposal area boundaries. A more detailed map of on-site bedrock piezometric elevations has been developed to more thoroughly delineate groundwater flow directions.

2. On-Site Flow in Consolidated Aquifer

An on-site bedrock aquifer flow map was constructed on the basis of water elevations obtained from on-site wells sealed in the bedrock aquifer, reference Drawing No. C 7091-24. On-site flow in the bedrock aquifer is primarily northward into the disposal area from Boring B-6B at a hydraulic gradient of 0.3 ft./ft. As groundwater flows under the disposal area, the hydraulic gradient is reduced to 0.007 ft. per ft. At the margins of the site, groundwater gradients steepen and resume the regional gradients of approximately .03 ft. per ft., with the exception of flow to the northeast away from Well E-2B.

The gradient between Well E-2B and the landfill private well (No. 5) is 0.1 ft. per ft. The water table surface then flattens out and resumes the regional gradient to the northeast. It appears that the water elevation may be higher than that indicated at No. 5. We would expect similar gradients to the northeast as exist to the north and west of the site.

The bulk of the bedrock flow under the disposal area leaves the site to the northwest with a lesser component of flow to the northeast. As evidenced from the regional groundwater map, a number of private wells are located downgradient from the landfill area. In a westerly direction, Well Nos. 1, 2, 3, 6, 7, 8, 9, 10, 11, 12, 21, and 67 are downgradient with Wells 1, 2, 3, 6, 7, 8, 9, 10, and 12 being within the 1200' limit. Downgradient wells to the north and east include Wells 5, 91, 92, 94, 95, and 96. It should be noted that there are a number of private wells situated parallel to downgradient flow but laterally displaced. The actual number of downgradient wells may vary with time as flow directions change with abnormally high or low precipitation.

3. <u>Flow in Unconsolidated Deposits</u>

On-site water table flow is depicted on Drawing C 7091-23. Groundwater elevations ranged from a high of 998.0' at Well E-6B in the southern portion of the site to a low of 970.2, USGS Datum at Well E-3 on March 9, 1978. Water levels obtained during the period from January through August, 1978, are shown in Appendix C.

It appears that Wells E-2A and E-6A monitor perched water table conditions. At the E-2 well site, the shallow well E-2A consistently registers a water level approximately 20' higher in elevation than the adjacent two deeper wells. Well E-2A was installed in a clay layer sandwiched between coarse layers of sand and gravel, giving rise to the perched water table conditions. Water levels in E-2B and E-2C correlate more closely with the water levels obtained in the remaining water table wells. Well E-6A was installed in a coarse sand unit which is interbedded with a till unit. Groundwater tends to accumulate within the more permeable sands thereby producing a perched condition. Due to the perched conditions at Wells E-2A and E-6A, the water table map, Drawing C 7091-23, was based upon water level elevations in Wells E-2B and E-6B.

The horizontal groundwater gradient in the vicinity of Well E-6B is approximately 0.035 ft./ft. to the north. This value correlates favorably with the regional bedrock gradient of 0.031 ft./ft. As groundwater flows from the southern section of the site out across the site in a northerly direction, the gradient is reduced to a value of 0.007 ft./ft. The reduction in gradient appears due to the permeability contrast within the subsoils. Drawing C 7091-18 shows the dominant soil types below the water table to illustrate the soil types through which groundwater flow actually occurs. The subsoils in the southern portion of the site are semi-permeable till/outwash type deposits whereas the central portion of the site is underlain by permeable outwash deposits. A solution of

Darcy's Law, assuming constant discharge per unit area, indicates that less permeable materials will have higher gradients than materials of greater permeability, other conditions being held equal. The hydraulic gradient is an indicator of the rate of head (potential) loss. We would expect greater head losses in a semi-permeable soil than in permeable soils. Groundwater flowing north from E6 encounters the sandy deposits typical of the central portion of the site and hydraulic gradients are reduced. As groundwater leaves the northern site boundary, the hydraulic gradients again increase gradually to approach the regional hydraulic gradients of approximately 0.03 ft./ft.

4. Vertical Flow

Vertical gradients determined from piezometer nests within the site are generally downward in the less permeable soils to the north and nearly horizontal in the permeable soils in the central portion of the site. Vertical gradients in the southern portion of the site at E6 and E-6B were not determined because of perched water table conditions. Piezometer Nests E5-E5B and E9-E9B oscillate between upward and downward flow. The reason for the oscillation between upward and downward flow is not specifically known, however the net flow direction can be considered nearly horizontal since the deflections in gradients remain very close to zero.

Downward groundwater movement or recharge is the dominant mode of flow in the northern site area as defined by piezometer nests E2B-E2C and E3-E3B. On August 30, 1978, the downward hydraulic gradients were 0.06 and 0.16 ft./ft., respectively. Well E3B is sealed in a very "tight" bedrock zone and, subsequently, recharges quite slowly. Some erratic water levels were recorded in WEll E3B due to its long recharge time after baildown for sampling. Given the hydrogeologic setting of a groundwater mound or divide, downward flow in this area is to be expected.

5. Summary

The on-site bedrock piezometric surface map, Drawing C 7091-24, was constructed using water level data from Wells E-3B, E-2C, E-5B, E-9B, and E-6B, all of which are screened in the bedrock. The water table map incorporated the shallower water table wells and indicates that flow directions and gradients are virtually identical in the bedrock and unconsolidated aquifers. The similarity of groundwater flow patterns depicted by the two maps indicates that the glacial material is hydraulically connected to the dolomite aquifer.

Precipitation which infiltrates in the site area migrates to the water table and is deflected northward through the site to exit the site to the north and west, primarily. Groundwater flow is downward, as well as northward, such that groundwater passes vertically from the unconsolidated to consolidated aquifer while flowing in a generally northward direction.

G. Water Budget

1. Introduction of Methods

Four water budget analyses have been performed as an aid in evaluating the potential impact of the Delafield Landfill on the existing hydrogeologic environment. The first two budgets analyze insitu field conditions to determine the relationship of leachate volume generated within the waste fill versus groundwater flow rates. The second two budgets analyze the effects of the proposed clay cap on future leachate generation rates. A detailed discussion of methods used in computing the water budget are contained in Appendix F.

2. Existing Conditions

Presently, the landfill is covered primarily by a 2' layer of outwash type soils, sand with some silt and gravel. Approximately 18% of the site consists of flat slopes (±2%) with 82% of the site with steep slopes (10-25%), see Drawing No. C 7091-23. Under these conditions, the runoff coefficients for the sand layer used in the water budget analysis are 0.5 for the flat areas and 0.11 for the steep areas. As shown in Appendix F, the precipitation (P), runoff (RO), infiltration (I), Actual Evapotranspiration (AET), and percolation (PERC) values for existing conditions with flat slopes and steep slopes are summarized below:

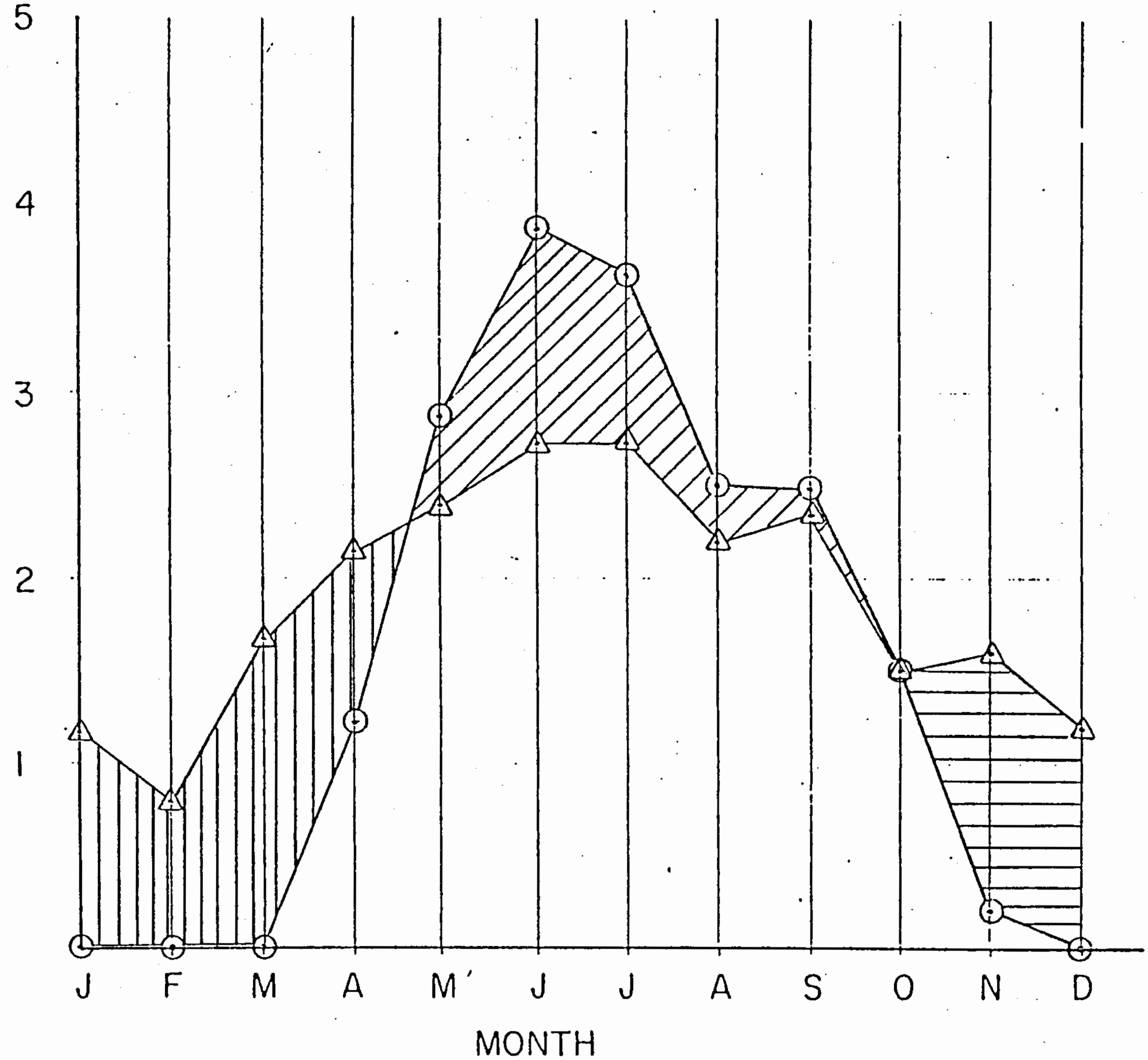
WATER BUDGET - EXISTING CONDITIONS

PARAMETER	FLAT SLOPE	STEEP SLOPE	COMPOSITE
P	30.7"	30.7"	
RO	1.5"	3.3"	
I	29.2"	27.4"	
AET	21.8"	21.1"	
PERC	7.4"	6.3"	6.5"

With an annual precipitation rate of 30.7", 7.4" percolates through the flat areas and 6.3" through the steep slopes, resulting in an average of 6.5" annual percolation on the entire site. The results of the water budget analysis for the existing cover are presented on Drawings C 7091-A69 and -A70.

The water balance methods were originally developed for natural soil conditions. The calculated results generally indicate the relationship between various climatic and soil factors but should not be construed as an exact indicator of leachate generation but as a general guide to determine leachate generation rates.

WATER BUDGET



6.3" ||||||| PERCOLATION

SOIL MOISTURE RECHARGE

SOIL MOISTURE UTILIZATION

21.1 —o— ACTUAL EVAPOTRANSPIRATION

27.4 —△— INFILTRATION

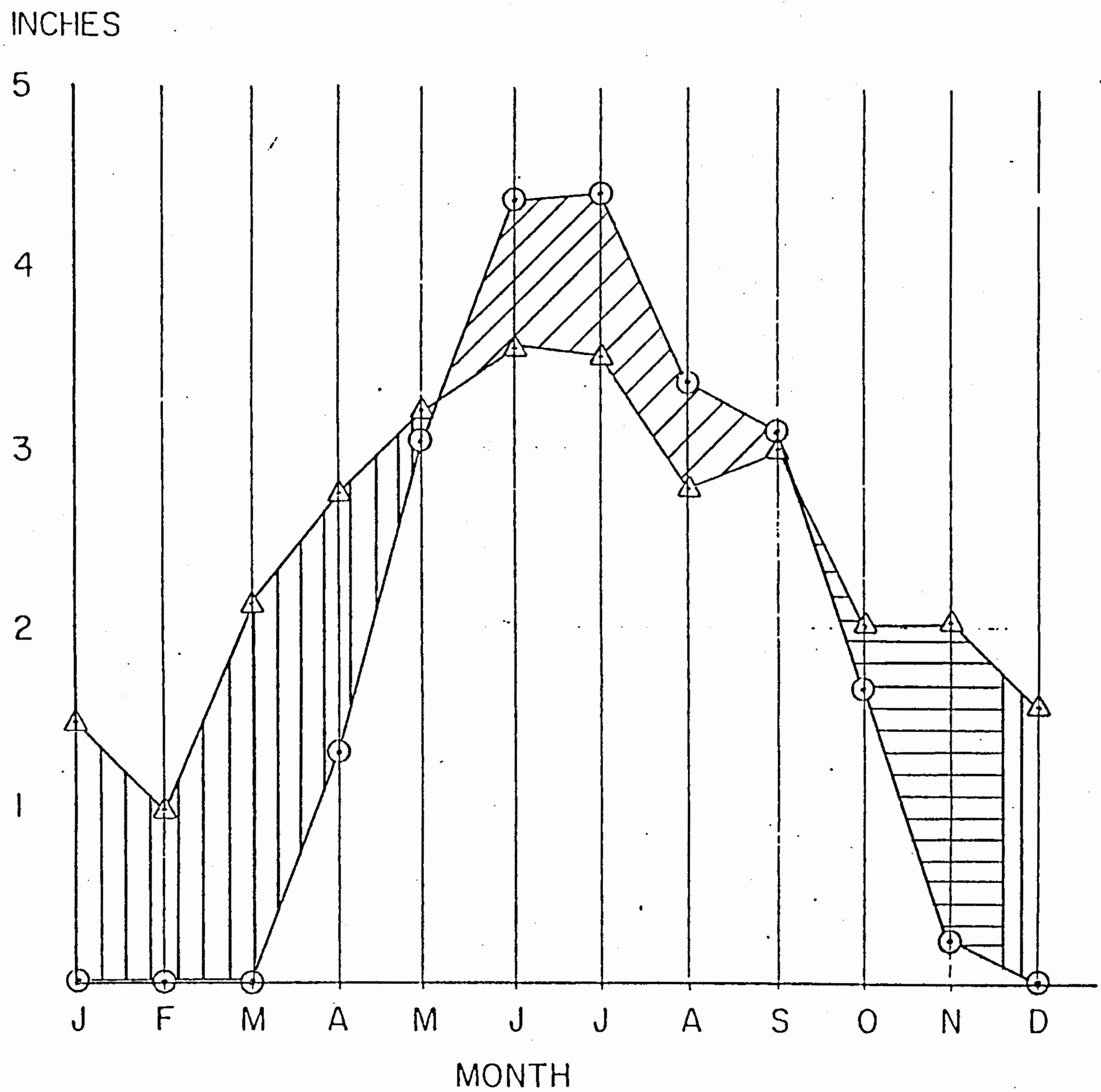
WARZYN ENGINEERING AND SERVICE COMPANY, INC.
Consulting Engineers

EXISTING COVER—STEEP SLOPES

DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

DWN TDH | CHK'D DAW | APP'D | DATE 1/19/79 | C7091-A69

WATER BUDGET



7.4" ||||||| PERCOLATION

═══ SOIL MOISTURE RECHARGE

////// SOIL MOISTURE UTILIZATION

21.8" —o— ACTUAL EVAPOTRANSPIRATION

29.2" —Δ— INFILTRATION

W WARZYN ENGINEERING AND SERVICE COMPANY, INC.
MADISON Consulting Engineers WISCONSIN

EXISTING COVER — FLAT SLOPES

DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

DWN TDH	CHK'D DAW	APP'D Daniel R. Viste	DATE 1/19/79	C7091-A70

Based on the water budget results, the greatest percolation (infiltration-actual evapotransporation) occurs during the spring thaw and rapidly decreases in April to essentially 0 inches during May. During late April - early May, the evapotransporation rate exceeds percolation and a moisture recharge deficit develops. From May through early September, the existing soil moisture is reduced due to high evapotransporation in the summer months. By September, evapotransporation rates decrease and percolation increases such that soil moisture recharge begins. Essentially all infiltration into the cover at that time is used to bring soil moisture to field capacity. Once field capacity is reached in mid to late fall, percolation again develops into the waste and continues through the winter and into the following spring until evapotransporation again exceeds infiltration.

The leachate production volume on an annual basis is controlled by the rate of percolation (6.5"/yr. with existing conditions) over the area of disposed wastes into which percolation occurs (approximately 35 acres).

$$V_L = PERC \times A$$

Where:

V_L = Leachate production rate

PERC = Percolation

A = Area of disposed waste

$$V_L = 6.5\text{"/yr.} \times 35 \text{ acres}$$

$$V_L = 2250 \text{ cubic feet/day}$$

The resulting leachate volume production averaged to a daily basis is approximately 2250 cubic feet per day.

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3. Groundwater Flow Volume

From the water table map, Drawing C 7091-23, groundwater leaves the site in predominantly two directions, northwest and east. The volume of groundwater flow leaving the site margins within the unconsolidated aquifer (sand and gravel) and the Niagaran formation were calculated on the basis of Darcy's Law, shown below:

$$Q = KiA$$

Where:

Q = Discharge per cross sectional area perpendicular to flow.

K = Hydraulic conductivity (permeability).

i = Hydraulic Gradient

A = Cross-sectional area through which flow occurs.

The horizontal hydraulic gradients from the regional groundwater map are approximately 0.03 ft./ft. to the northwest and 0.1 ft./ft. to the east. The estimated average permeabilities of the two units are 5×10^{-3} cm./sec. (14.2 ft./day) for the glacial units and 10^{-4} (0.28 ft./day) for the dolomite aquifer. The average saturated thicknesses of the sand and gravel unit and the dolomite are 30' and 100', respectively. The width of the flow paths to the northwest and east are 1600' and 900', respectively. The calculations for groundwater flow leaving the site to the northwest are shown below:

a. Unconsolidated Deposits

$$Q = KiA$$

K = 14.2 ft./day
i = 0.03 ft./ft.
A = 30 ft. x 1600 ft. = 48,000 sq. ft.

Q = approx. 20,000 cubic feet per day.

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b. Niagaran Formation

Q = approx. 1400 cubic feet per day.

Groundwater flow volumes leaving the site to the east are shown below:

a. Unconsolidated Deposits

Q = approx. 38,000 cubic feet per day.

b. Niagaran Formation

Q = approx. 2500 cubic feet per day.

From the above data, the calculated horizontal flow volume leaving the property boundaries is approximately 62.000 cubic feet per day with 58,000 cubic feet per day flowing through the sand and gravel and 3900 cubic feet per day flowing through the dolomite. Flow rates within the glacial aquifer are approximately 15 times greater than in the bedrock aquifer.

The relationship between groundwater flow directions and refuse location is such that the majority of the groundwater flow which occurs under the refuse will exit the site to the northwest, therefore the major focus of the water budget analysis will be on the flow to the northwest. It is estimated that approximately 90% of the flow under the existing waste will exit to the northwest and, subsequently, that 90% of any leachate generated, which enters the groundwater system, will also leave the site to the northwest. From the preceeding calculations of groundwater flow volumes, approximately 21,000 cubic feet per day flow under the refuse and out away from the site to the northwest with 20,000 cubic feet per day flowing within the sand and gravel aquifer and 1400 cubic feet per day in the dolomite. Based on the leachate generation rates, approximately 2000 cubic feet per day (90% of total) of leachate will also exit the site in a northwesterly direction upon entering the groundwater system or approximately 9% of the northwesterly groundwater flow volume. Similar calculations indicate that only 0.5% of the groundwater flow volume to the east is leachate.

4. Clay Cap - Proposed

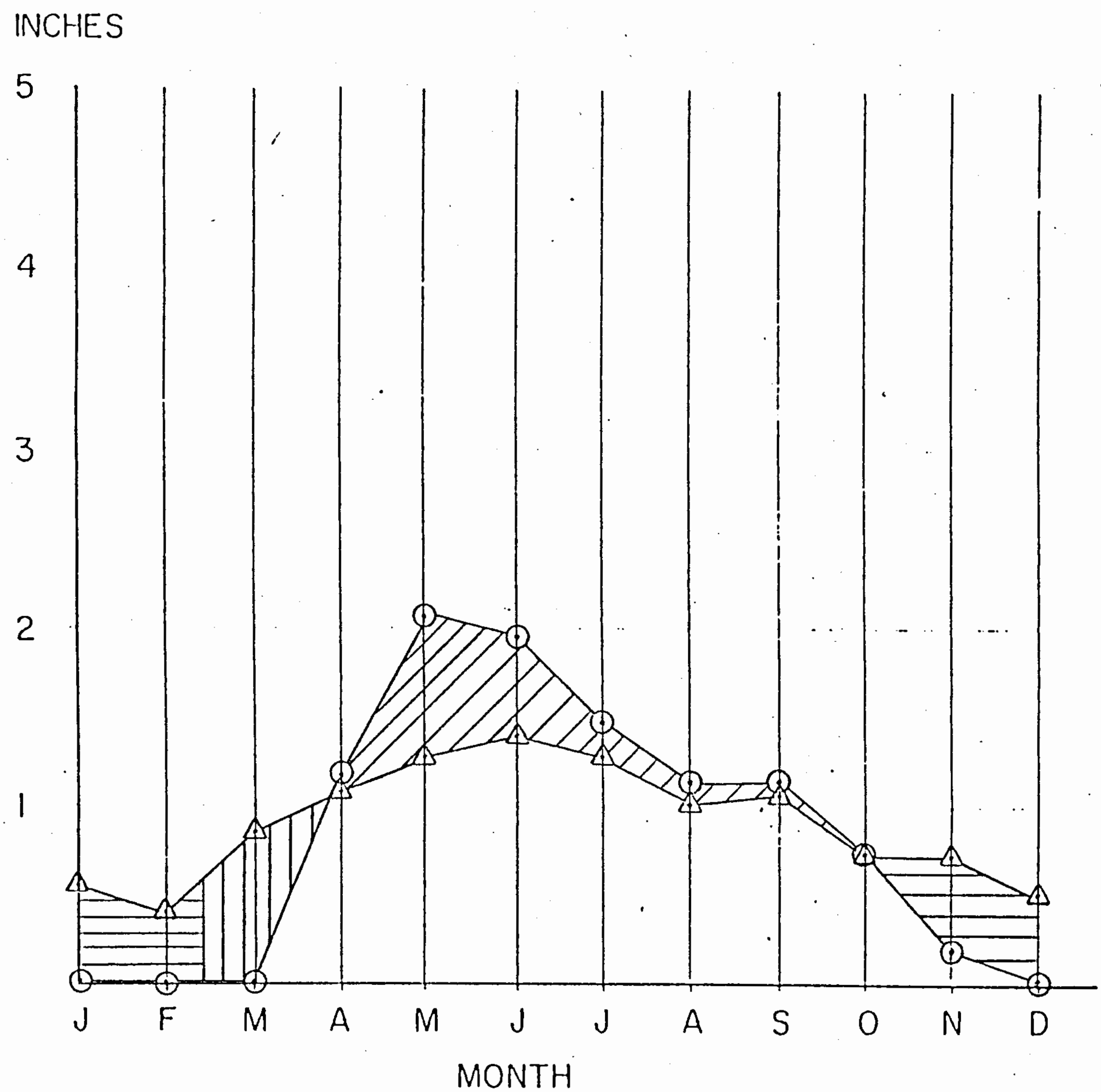
The second two water budgets were prepared as a predictive tool for assessing the effectiveness of a proposed impermeable cap to be implaced on top of the existing sandy cover. The cap would consist of a 1' to 1.5' thick layer of clay placed directly on top of the existing cover. The clay would in turn be covered by a minimum 4" thick layer of sandy soil. The entire site would then be covered by a thin veneer of vegetative supporting soil to facilitate plant growth and retard surface erosion. The steep slopes present at the site would be maintained to maximize surface runoff. The clay cap is intended to be an impermeable barrier to precipitation infiltrating into the in-place waste. The sandy covering is designed to: 1) protect the clay from drying out, and 2) minimize the formation of desiccation cracks in the clay. It is felt that the sand will retain some moisture throughout the majority of the year and minimize drying of the underlying clay layer.

The clay cover will be the controlling soil in regard to runoff, even though it is covered by a sandy layer. Precipitation will readily infiltrate through the sand layer and come in contact with the clay. Due to the impermeable nature of the clay, the water will collect on top of the clay and runoff downslope through the overlying sand, with only a very small amount of water actually infiltrating through the clay. The second water budget was performed to predict the amount of water which will actually percolate through the clay into the waste and be available for leachate generation. The budget was performed as previously described, with a calculated runoff coefficient of 0.27 for the flat areas and 0.65 for the steep slopes. The results of the water budget are represented on Drawings C 7091-A71 and C 7091-A72. The calculations are presented in Appendix F.

From Appendix F, the total amount of percolation entering the waste would be 1.4" annually, or 4.5% of the total annual precipitation. Percolation of precipitation into the waste would occur in March. During the months of April through September, evapotransporation rates exceed precipitation and moisture is withdrawn from storage in the soil. During the fall months, precipitation would exceed evapotransporation rates and soil moisture is replaced. It is not until spring that significant percolation through the clay cap would actually develop.

The clay cap would tend to reduce percolation rates and, subsequently, leachate generation rates, by a factor of approximately 6. The annual leachate generation rate would be reduced to approximately 350 cubic feet per day as compared to 2250 cubic feet per day being generated under existing conditions. It is felt that the use of a clay cap is an environmentally sound practice.

WATER BUDGET



0.8" PERCOLATION

SOIL MOISTURE RECHARGE

SOIL MOISTURE UTILIZATION

8.0" —o— ACTUAL EVAPOTRANSPIRATION

10.8" —△— INFILTRATION

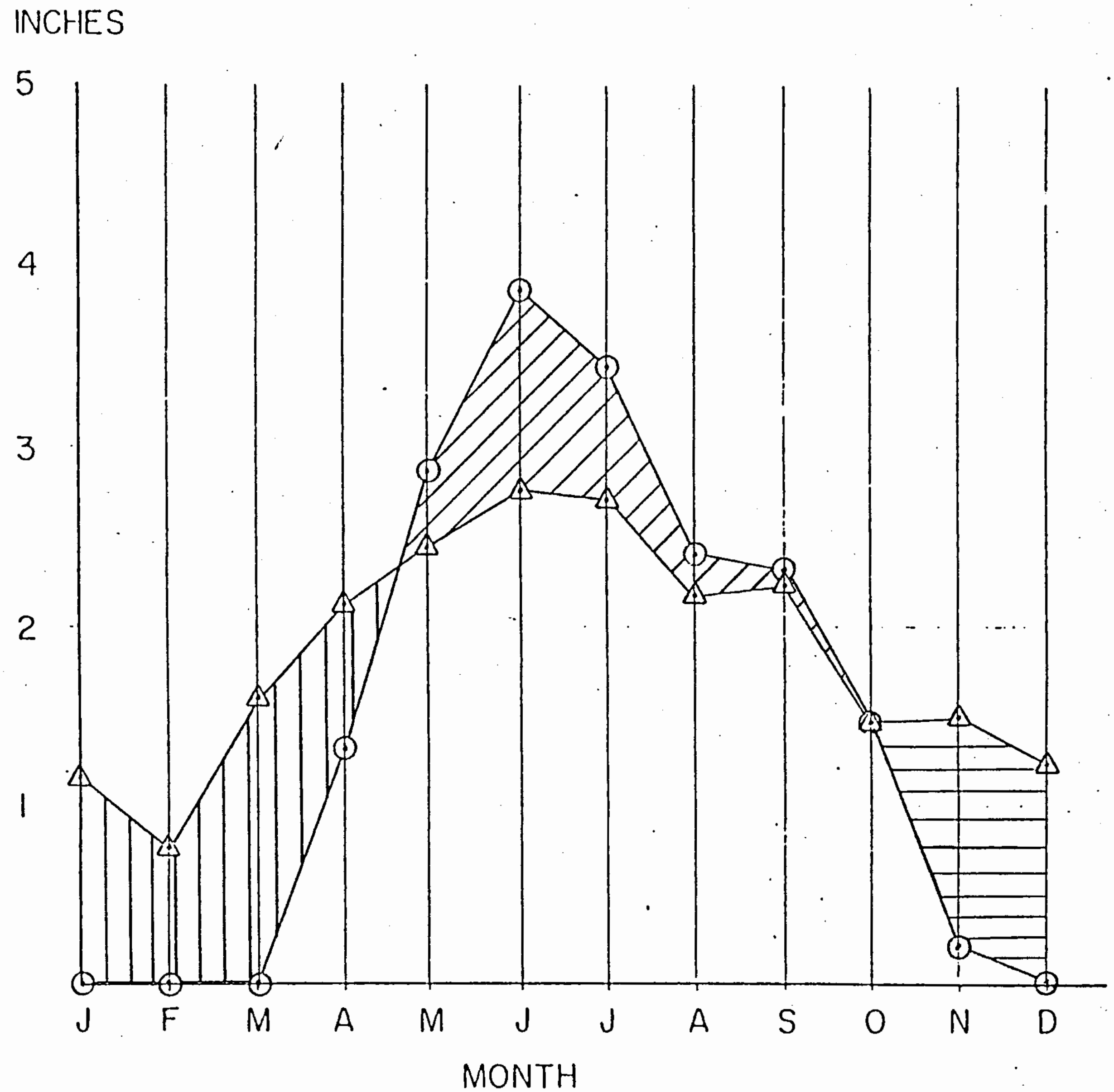
WARZYN ENGINEERING AND SERVICE COMPANY, INC.
Consulting Engineers

PROPOSED CLAY CAP-STEEP SLOPES

DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

DWN TDH	CHK'D DAW	APP'D Daniel R. Viste	DATE 1/19/79	C7091-A71

WATER BUDGET



4.3" |||||||| PERCOLATION

═══ SOIL MOISTURE RECHARGE

////// SOIL MOISTURE UTILIZATION

18.1" —o— ACTUAL EVAPOTRANSPIRATION

22.4" —△— INFILTRATION

WARZYN ENGINEERING AND SERVICE COMPANY, INC.
Consulting Engineers

PROPOSED CLAY CAP-FLAT SLOPES

DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

DWN TDH	CHK'D DAW	APP'D Daniel R. Viste	DATE 1/19/79	C7091-A72

RECOMMENDATIONS

We have prepared these recommendations subsequent to the results of analyses of the in-field conditions at the Delafield Sanitary Landfill. The recommendations outline future steps to be taken regarding investigative reports and interim operations at the landfill.

1. The Report on Water Quality in and about the Delafield Sanitary Landfill should be completed and submitted to the DNR for review and comment.
2. Interim filling operations should continue on the lined portion of the landfill in accordance with accepted engineering practices.
3. Abandonment procedures should be implemented in those areas previously filled, in a manner designed to best protect the environment. A plan should be developed on a design capacity basis to facilitate a final, environmentally sound, site abandonment, based on the current state of the art.

CLOSING REMARKS

We have completed the Physical Site Description, In-Field Conditions Report as outlined in the initial work scope, modified by subsequent discussions with the Department of Natural Resources, and through re-evaluation of additional data continuously being obtained. The study was confined to a physical site description of the Delafield Sanitary Landfill to provide a basis for the forthcoming analysis and discussion of water quality data obtained in and about the Delafield Sanitary Landfill. The report on water quality is currently under preparation and will be submitted to the DNR in the near future.

Respectfully submitted,

WARZYN ENGINEERING INC.

Douglas A Wierman

Douglas A. Wierman
Hydrogeologist

Daniel R. Viste

Daniel R. Viste
Chief Hydrogeologist

Henry A Koch

Henry A. Koch
Professional Engineer

DAW/DRV/HAK/dmf

Appendices

REFERENCES

1. Cotter, R.D., et al., Water Resources of Wisconsin, Rock-Fox River Basin, Hydrogeologic Investigations Atlas HA 360, 1969.

2. Gonthier, J.D., Groundwater Resources of Waukesha County, Wisconsin, Info. Circular No. 29, Wisconsin Geological Survey, 1975.

3. Walton, W.C., Groundwater Resource Evaluation, 1970.

4. Fenn, D.G., et al., Use of the Water Balance Method for Predicting Leachate Generation from Solid Waste Disposal Sites, EPA Publication, 1975.

5. Thornthwaite, C.W., and Mather, J.R., Instructions and Tables for Computing Potential Evapotransporation and the Water Balance, 1957.

6. American Society of Civil Engineers, Design and Construction of Sanitary and Storm Sewers, Manual and Report on Engineering Practices No. 37, 1969.

7. U.S. Weather Bureau Atlas Technical Paper #40, 1961.

8. U.S. Department of Agriculture, Soil Conservation Service, Engineering Field Manual, 1971.

APPENDIX "A"

Subsurface Investigation

GENERAL REMARKS

We have endeavored to evaluate subsurface conditions and physical properties of the subsoil as revealed by the borings and laboratory testing. A problem inherent in this evaluation is the variability in engineering properties within soil strata involved, and specifically in any location variation in the soil which is located between borings. Due to natural or man-made causes, subsurface conditions may change with time.

Conclusions drawn and recommendations given in this report are for a specific proposed use of this site. They are our opinions and are based upon conditions that existed at the boring locations and such parameters as proposed site usage, soil loading, elevations, etc..

Since subsurface conditions depend on seasonal moisture variations, frost action, construction methods, and the inherent natural variations, careful observations must be made during construction. These should be brought to our attention as it may be necessary to modify the conclusions and recommendations presented herein.

APPENDIX "B"

FIELD METHODS
for
EXPLORATION AND SAMPLING SOILS

A. Boring Procedures Between Samples

The bore hole is extended downward, between samples, by a continuous flight auger, driven and washed-out casing, or rotary boring with drilling mud or water.

B. Standard Penetration Test and Split-Barrel Sampling of Soils (ASTM* Designation: D 1586)

This method consists of driving a 2" outside diameter split barrel sampler using a 140 pound weight falling freely through a distance of 30 inches. The sampler is first seated 6" into the material to be sampled and then driven 12". The number of blows required to drive the sampler the final 12" is recorded on the log of borings and known as the Standard Penetration Resistance. Recovered samples are first classified as to texture by the driller. Later, in the laboratory the driller's classification is reviewed by a soils engineer who examines each sample.

C. Thin-walled Tube Sampling of Soils (ASTM* Designation: D 1587)

This method consists of forcing a 2" or 3" outside diameter thin wall tube by hydraulic or other means into soils, usually cohesive types. Relatively undisturbed samples are recovered.

D. Soil Investigation and Sampling by Auger Borings (ASTM* Designation: D 1452)

This method consists of augering a hole and removing representative soil samples from the auger flight or bucket at 5'0" intervals or with each change in the substrata. Relatively disturbed samples are obtained and its use is therefore limited to situations where it is satisfactory to determine approximate subsurface profile.

E. Diamond Core Drilling for Site Investigation (ASTM* Designation: D 2113)

This method consists of advancing a hole in hard strata by rotating downward a single tube or double tube core barrel equipped with a cutting bit. Diamond, tungsten carbide, or other cutting agents may be used for the bit. Wash water is used to remove the cuttings. Normally a 2" O.D. by 1 3/8" I.D. coring bit is used unless otherwise noted. The rock or hard material recovered within the core barrel is examined in the field and laboratory. Cores are stored in partitioned boxes and the length of recovered material is expressed as a percentage of the actual distance penetrated.

*American Society for Testing and Materials, Philadelphia, Pennsylvania

APPENDIX C

GROUNDWATER LEVEL MONITORING RESULTS

WARZYN
ENGINEERING INC

WELL NO.	ELEVATION TOP OF CASING	TYPE OF WELL	ELEVATION OF SEAL	WATER ELEVATIONS 12-15-77	1-4-78	1-13-78
E-1	1040.46	Water Table		997.56		
*E-2	1007.00	Water Table		997.20		997.41
E-2B	1007.57	Water Table		978.07	1001.44	977.81
E-2C	1007.42	Piezometer	918	976.32	976.00	975.79
E-3	1018.21	Water Table		970.97	970.75	970.66
E-3B	1011.67	Piezometer	936	987.33	984.85	977.33
E-4	1037.26	Water Table		979.51		976.84
E-5	1024.62	Water Table		979.12	977.06	977.13
E-5B	1025.58	Piezometer	943	979.12	977.17	977.13
*E-6	1065.68	Water Table	1057		1009.14	1008.94
E-6B	1065.32	Piezometer	982		998.82	998.82
E-7	1029.63	Water Table	1020	977.93	977.69	977.63
E-8	1032.77	Water Table	1023	1006.67	977.60	977.45
E-9	991.81	Water Table	984	977.93	977.43	977.33
E-9B	990.79	Piezometer	945			976.31
E-10	1074.11	Water Table	984		982.10	977.35
E-11	1072.42	Water Table	983			977.00
E-12	999.70	Water Table	986	974.90	975.05	974.83
*E-14A	995.30	Leachate Head	991			
*E-15A	1000.44	Leachate Head	990			
E-15	1001.40	Water Table	969			
P-3	1625.00	Piezometer				

*Monitors Perched Water Table or Leachate Head

WELL NO.	ELEVATION TOP OF CASING	TYPE OF WELL	ELEVATION OF SEAL	WATER ELEVATIONS 2-20-78	3-9-78	3-28-78
E-1	1040.46	Water Table		977.06	976.77	976.90
*E-2	1007.00	Water Table		996.39	995.97	1000.32
E-2B	1007.57	Water Table		977.46	977.19	977.89
E-2C	1007.42	Piezometer	918	975.81	975.96	976.11
E-3	1013.21	Water Table		970.53	970.25	970.15
E-3B	1011.67	Piezometer	936	977.36	970.12	970.32
E-4	1037.26	Water Table		976.62	976.29	976.46
E-5	1024.62	Water Table		976.74	976.47	976.70
E-5B	1025.58	Piezometer	943	976.67	976.44	976.60
*E-6	1065.68	Water Table	1057	1013.00	1012.70	1012.18
E-6B	1065.32	Piezometer	982	997.40	998.00	998.02
E-7	1029.63	Water Table	1020	977.43	977.17	977.20
E-8	1032.77	Water Table	1023	977.25	976.99	977.06
E-9	991.81	Water Table	984	977.08	976.81	976.89
E-9B	990.79	Piezometer	945	977.11	976.85	977.02
E-10	1074.11	Water Table	984	977.07		
E-11	1072.42	Water Table	983	972.00	976.50	976.58
E-12	999.70	Water Table	986	974.22	973.83	974.81
*E-14A	995.30	Leachate Head	991			988.07
*E-15A	1000.44	Leachate Head	990	984.77 Dry		986.78
E-15	1001.40	Water Table	969	976.04	976.23	976.31
P-3	1625.00	Piezometer				976.32

*Monitors Perched Water Table or Leachate Head

WELL NO.	ELEVATION TOP OF CASING	TYPE OF WELL	ELEVATION OF SEAL	WATER ELEVATIONS 4-7-78	4-27-78	5-24-78
E-1	1040.46	Water Table		977.00	977.36	977.95
*E-2	1007.00	Water Table			1000.29	998.99
E-2B	1007.57	Water Table			978.43	978.79
E-2C	1007.42	Piezometer	918		976.67	977.30
E-3	1013.21	Water Table		970.10	970.43	971.00
E-3B	1011.67	Piezometer	936		948.33	963.30
E-4	1037.26	Water Table		976.55	976.84	977.39
E-5	1024.62	Water Table		976.82	977.11	977.69
E-5B	1025.58	Piezometer	943	976.79	977.08	977.66
*E-6	1065.68	Water Table	1057	1012.24	1015.89	1018.01
E-6B	1065.32	Piezometer	982	997.14	999.18	1001.98
E-7	1029.63	Water Table	1020	977.30	977.64	978.29
E-8	1032.77	Water Table	1023	977.17	977.50	978.11
E-9	991.81	Water Table	984	977.06	977.35	977.95
E-9B	990.79	Piezometer	945	977.09	977.39	978.00
E-10	1074.11	Water Table	984			
E-11	1072.42	Water Table	983	976.69	977.03	977.58
E-12	999.70	Water Table	986	975.88	973.94	976.56
*E-14A	995.30	Leachate Head	991	988.44	988.38	988.16
*E-15A	1000.44	Leachate Head	990	986.93	986.62	986.89
E-15	1001.40	Water Table	969	976.48	976.76	977.35
P-3	1625.00	Piezometer		976.18	976.50	977.51

*Monitors Perched Water Table or Leachate Head

WELL NO.	ELEVATION TOP OF CASING	TYPE OF WELL	ELEVATION OF SEAL	WATER ELEVATION 6-5-78	8-30-78	12/12/78	2/13/79	6/25/79
E-1	1040.46	Water Table		978.21	980.41	980.42		
*E-2	1007.00	Water Table		997.97	997.40			
E-2B	1007.57	Water Table		978.73	980.70			
E-2C	1007.42	Piezometer	918	975.60	977.96			
E-3	1013.21	Water Table		971.36	973.56			
E-3B	1011.67	Piezometer	936	963.52	970.52			
E-4	1037.26	Water Table		977.59	979.70			
E-5	1024.62	Water Table		977.85	979.86	979.91	979.21	
E-5B	1025.58	Piezometer	943	977.83	979.82			
*E-6	1065.68	Water Table	1057	1018.28	1015.09			
E-6B	1065.32	Piezometer	982	1002.21	1000.31			
E-7	1029.63	Water Table	1020	978.60	981.05	981.04		985.38
E-8	1032.77	Water Table	1023	978.36	980.60	980.70		984.87
E-9	991.81	Water Table	984	978.16	980.35	980.40		
E-9B	990.79	Piezometer	945	978.22	980.37			
E-10	1074.11	Water Table	984					
E-11	1072.42	Water Table	983	977.79	980.04	980.08	979.41	984.14
E-12	999.70	Water Table	986	976.51	976.42			
*E-14A	995.30	Leachate Head	991	988.02	988.96			
*E-15A	1000.44	Leachate Head	990	986.69	985.63			
E-15	1001.40	Water Table	969	977.56	979.68	979.79		
P-3	1625.00	Piezometer		977.76				

*Monitors Perched Water Table or Leachate Head

APPENDIX D

WELL BAILDOWN TEST RESULTS

WARZYN
ENGINEERING INC

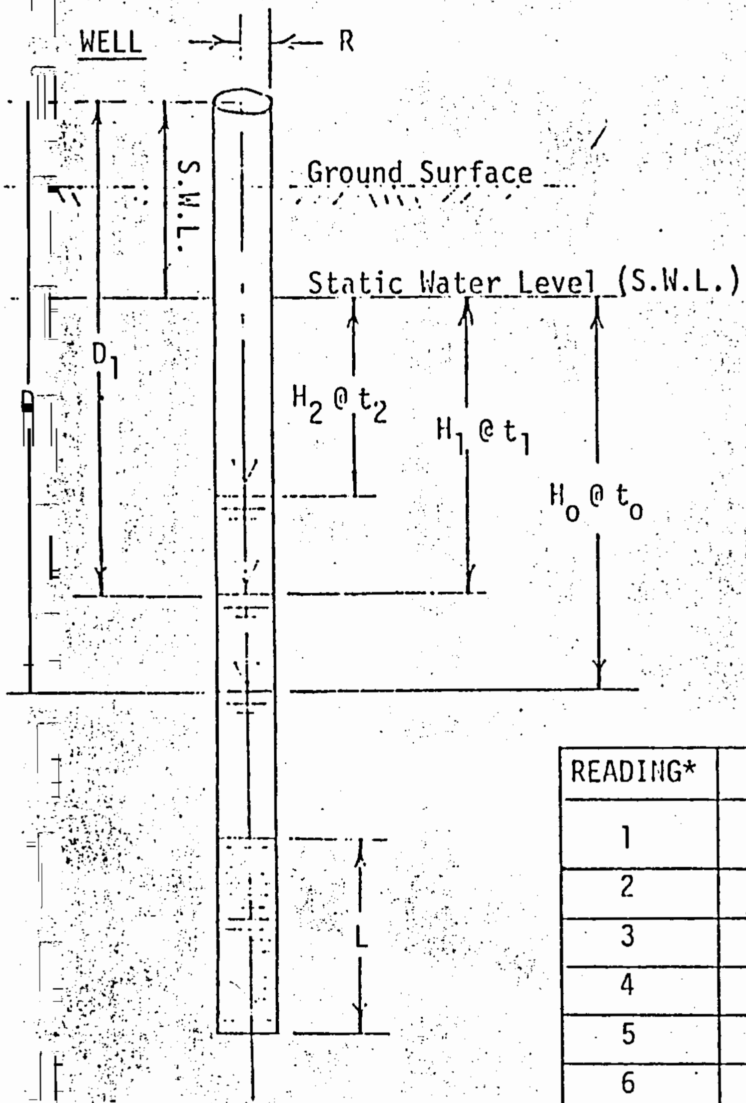
1) Project: DELAFIELD SANITARY LANDFILL

2) Location: DELAFIELD, WISCONSIN 4) Job No: C7091

3) Client: SANITARY TRANSFER AND LANDFILL CO. 5) Date: 6/7/78

6) Well or Boring No: E-2B

7) Personnel: CGK



8) R=Radius of well= 1.5 (in.)

9) L=Length of Screen 10 (ft.)
(from well detail sheet)

10) Static Water Level 28.83 (ft.)
(Depth to Water)

11) Baildown Data (From Test)-Record Information in Minutes and Feet

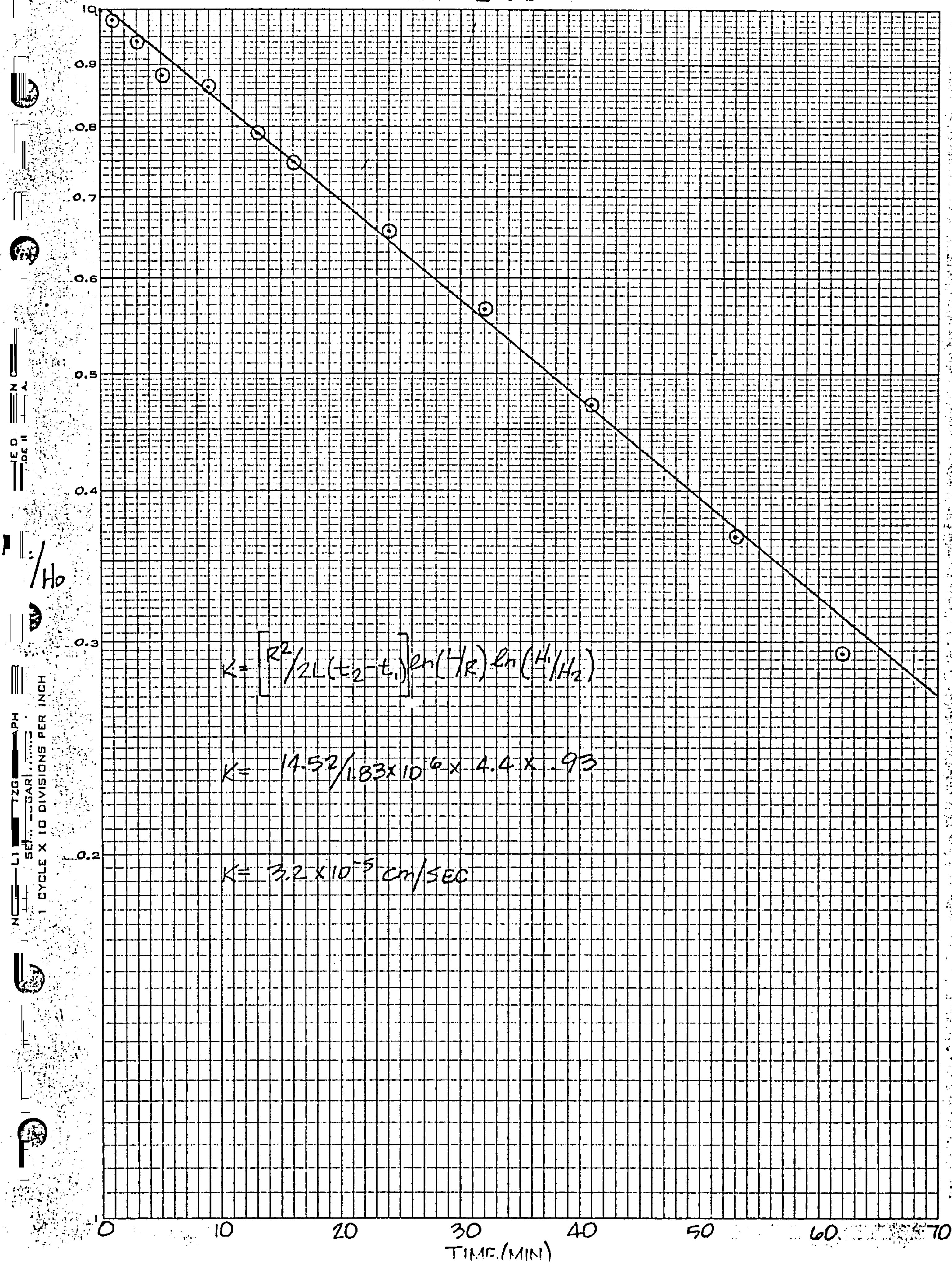
READING*	TIME (Start)	DEPTH TO WATER (After Baildown) D_t	2+ D_t - SWL=Ht	3+ Ht/H_o
1	t_o 0	D_o 39.67	H_o 10.84	1.0
2	t_1 1	D_1 39.50	H_1 10.67	.984
3	t_2 3	D_2 39.15	H_2 10.22	.943
4	t_3 5	D_3 38.78	H_3 9.55	.881
5	t_4 9	D_4 38.09	H_4 9.26	.854
6	t_5 13	D_5 37.41	H_5 8.58	.792
7	t_6 16	D_6 36.94	H_6 8.11	.748
8	t_7 24	D_7 35.93	H_7 7.10	.655
9	t_8 32	D_8 34.95	H_8 6.12	.565
10	t_9 41	D_9 33.95	H_9 5.12	.472
11	t_{10} 53	D_{10} 32.81	H_{10} 3.98	.367
12	t_{11} 62	D_{11} 32.02	H_{11} 3.19	.294
13	t_{12} 74	D_{12} 31.10	H_{12} 2.27	.209
14	t_{13} 83	D_{13} 30.55	H_{13} 1.72	.159

* Take readings until well is stabilized, if tight soils - test may be stopped prior to stabilization as necessary

+Disregard Columns 2 and 3 during baildown test. They are for office calculations.

$$K = \frac{R^2}{2L(t_2 - t_1)} \ln(L/R) \ln(H_1/H_2) = 3.2 \times 10^{-5} \text{ cm/sec}$$

E-2B



1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
.1
/Ho
1 CYCLE X 10 DIVISIONS PER INCH
K= [R²/2L(t₂-t₁)] ln(L/R) ln(H₁/H₂)
K= 14.52/1.83x10⁶ x 4.4 x .93
K= 3.2 x 10⁻⁵ cm/SEC
0
10
20
30
40
50
60
70
TIME (MIN)

BAILDOWN TEST - FIELD FORM

1) Project: DELAFIELD SANITARY LANDFILL

2) Location: DELAFIELD, WISCONSIN

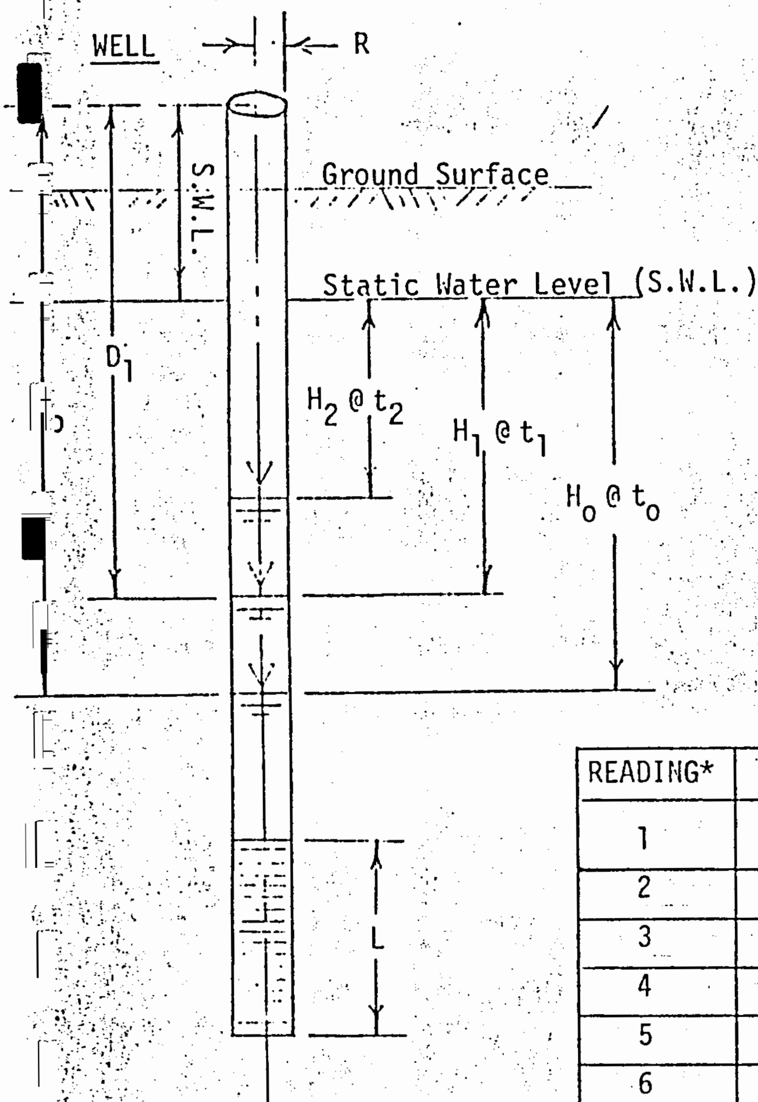
3) Client: SANITARY TRANSFER AND LANDFILL CO.

4) Job No: 7091

5) Date: 6/7/78

6) Well or Boring No: E-4

7) Personnel: CGK



8) R=Radius of well= 1.5 (in.)

9) L=Length of Screen 5 (ft.)
(from well detail sheet)

10) Static Water Level 59.62 (ft.)
(Depth to Water)

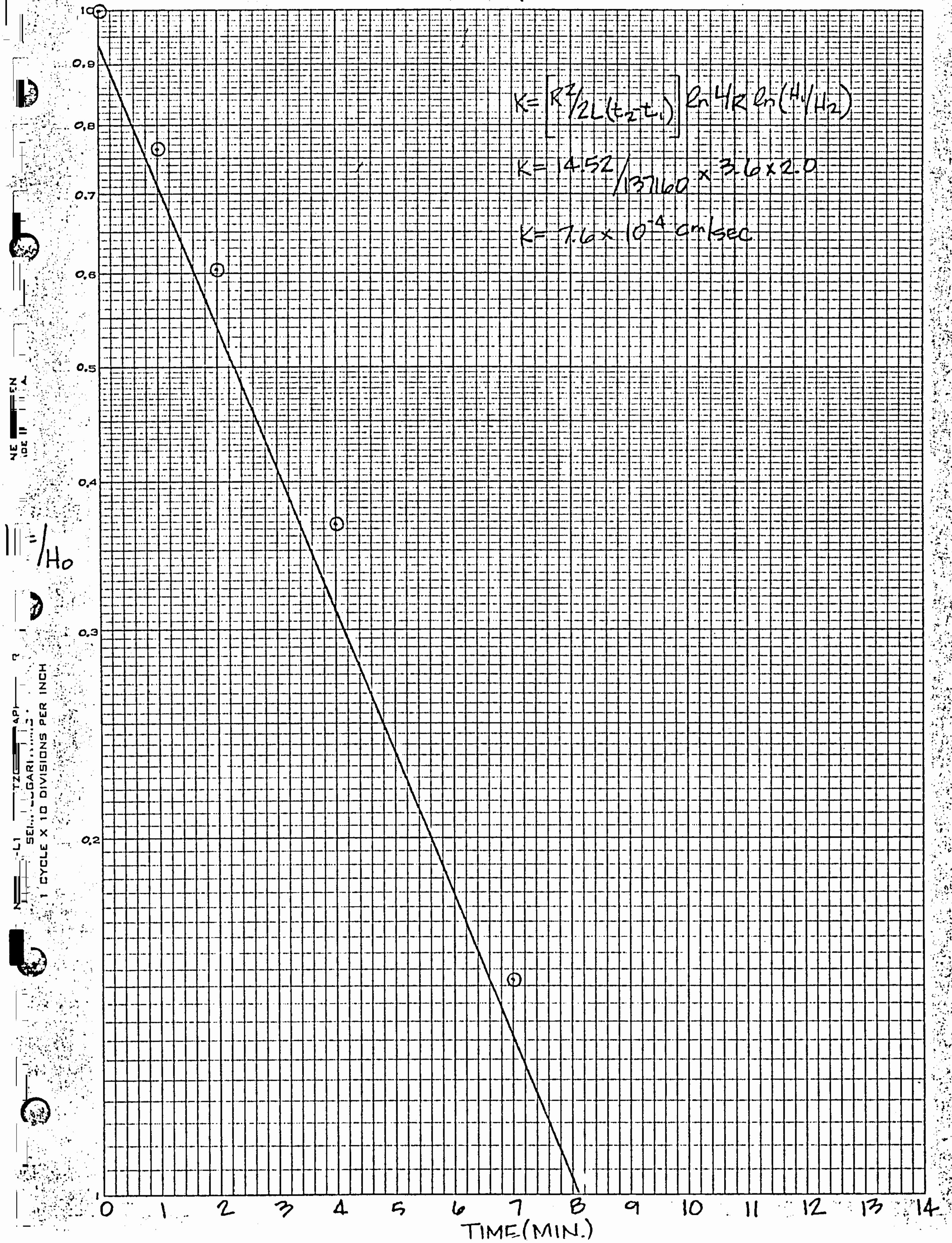
11) Baildown Data (From Test)-Record Information in Minutes and Feet

READING*	TIME (Start)	DEPTH TO WATER (After Baildown) D_t	2+ D_t - SWL = H_t	3+ H_t/H_o
1	t_o 0	D_0 65.06	H_0 5.44	1.0
2	t_1 1	D_1 63.76	H_1 4.14	.761
3	t_2 2	D_2 62.90	H_2 3.28	.603
4	t_3 4	D_3 61.62	H_3 2.00	.368
5	t_4 7	D_4 60.44	H_4 .82	.151
6	t_5 9.5	D_5 59.96	H_5 .34	.063
7	t_6	D_6	H_6	
8	t_7	D_7	H_7	
9	t_8	D_8	H_8	
10	t_9	D_9	H_9	
11	t_{10}	D_{10}	H_{10}	
12	t_{11}	D_{11}	H_{11}	
13	t_{12}	D_{12}	H_{12}	
14	t_{13}	D_{13}	H_{13}	

* Take readings until well is stabilized, if tight soils - test may be stopped prior to stabilization as necessary

+Disregard Columns 2 and 3 during baildown test. They are for office calculations.

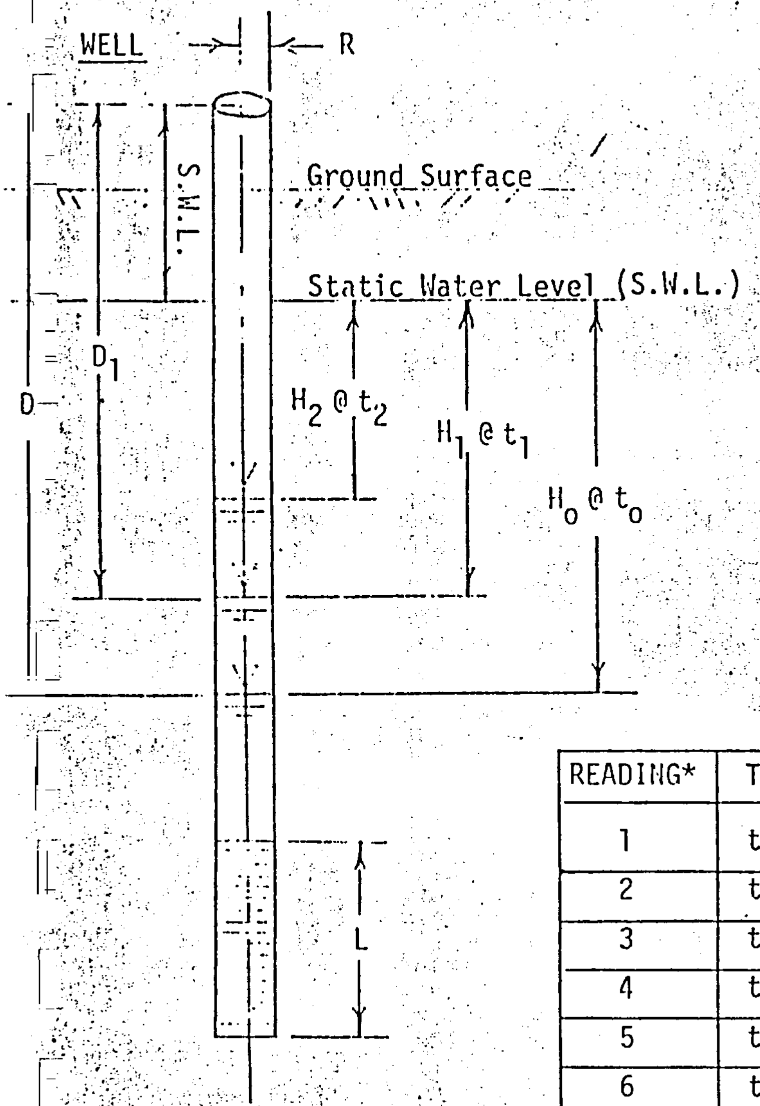
$$K = \frac{R^2}{2L(T_2-T_1)} \ln(L/R) \ln(H_1/H_2) = 7.6 \times 10^{-4} \text{ cm/sec}$$


K= [R²/2L(t₂-t₁)] ln 4/R ln(H₁/H₂)
K= 14.52/137160 x 3.6 x 2.0
K= 7.6 x 10⁻⁴ cm/sec
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
.1
/Ho
1 CYCLE X 10 DIVISIONS PER INCH
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14
TIME (MIN.)

1) Project: DELAFIELD SANITARY LANDFILL

2) Location: DELAFIELD, WISCONSIN 4) Job No: 7091

3) Client: SANITARY TRANSFER AND LANDFILL CO. 5) Date: 6/7/78



6) Well or Boring No: E-5

7) Personnel: CGK

8) R=Radius of well= 1.5 (in.)

9) L=Length of Screen 10 (ft.)
(from well detail sheet)

10) Static Water Level 46.74 (ft.)
(Depth to Water)

11) Baildown Data (From Test)-Record Information in Minutes and Feet

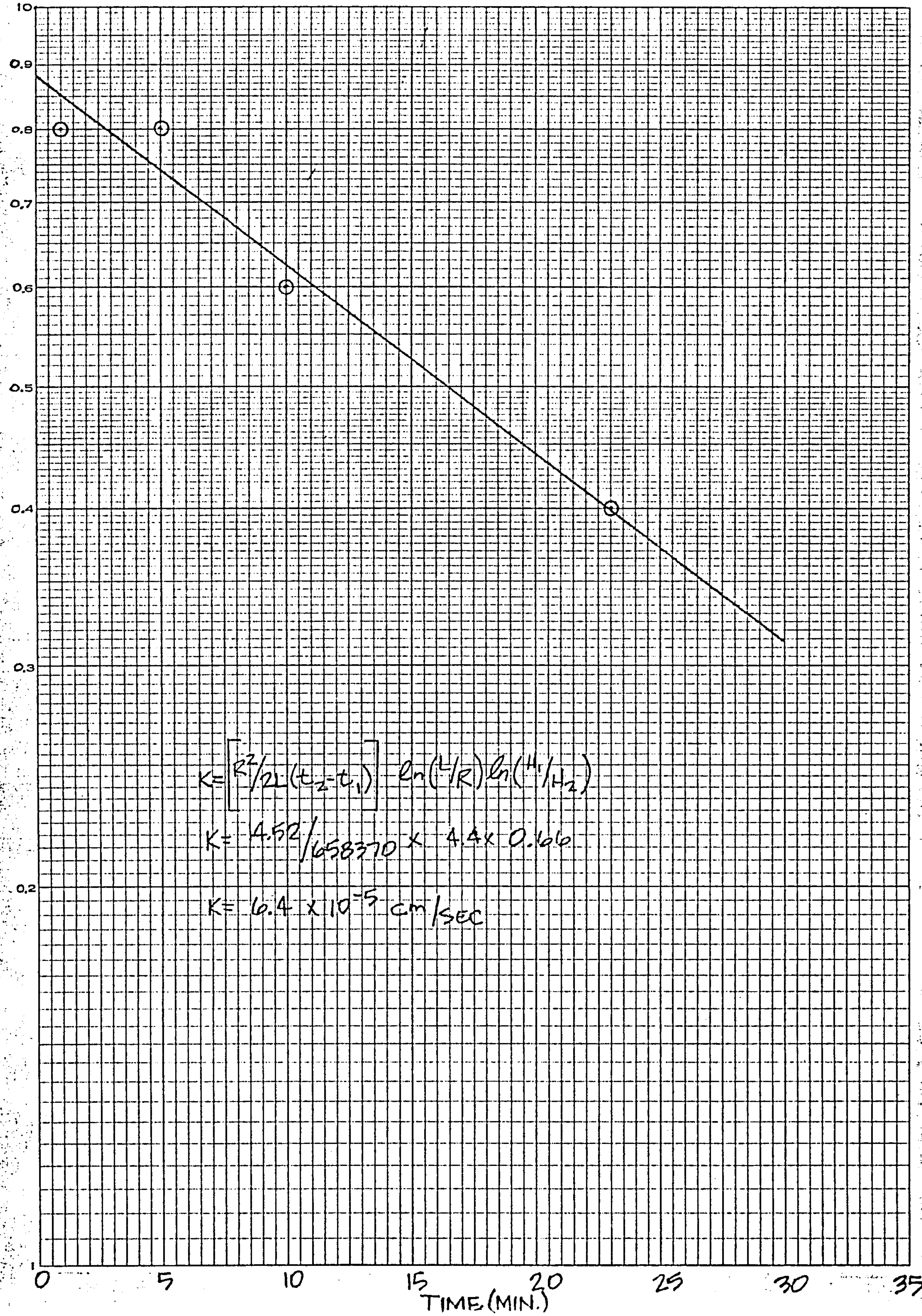
READING*	TIME (Start)		DEPTH TO WATER (After Baildown) D_t		2+ D_t - SWL = Ht		3+ Ht/H_0
1	t_0	0	D_0	46.79	H_0	.05	1
2	t_1	1	D_1	46.78	H_1	.04	.80
3	t_2	5	D_2	46.78	H_2	.04	.80
4	t_3	10	D_3	46.77	H_3	.03	.60
5	t_4	23	D_4	46.76	H_4	.02	.40
6	t_5		D_5		H_5		
7	t_6		D_6		H_6		
8	t_7		D_7		H_7		
9	t_8		D_8		H_8		
10	t_9		D_9		H_9		
11	t_{10}		D_{10}		H_{10}		
12	t_{11}		D_{11}		H_{11}		
13	t_{12}		D_{12}		H_{12}		
14	t_{13}		D_{13}		H_{13}		

* Take readings until well is stabilized, if tight soils - test may be stopped prior to stabilization as necessary

† Disregard Columns 2 and 3 during baildown test. They are for office calculations.

$$K = \frac{R^2}{2L(T_2 - T_1)} \ln(L/R) \ln(H_1/H_2) = 6.4 \times 10^{-5} \text{ cm/sec}$$

E-5


1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
1
/Ho
1 CYCLE X 10 DIVISIONS PER INCH
K= [R²/2L(t₂-t₁)] ln(L/R) ln(H₁/H₂)
K= 4.52/658370 x 4.4x 0.66
K= 6.4 x 10⁻⁵ cm/SEC
0
5
10
15
20
25
30
35
TIME (MIN.)

APPENDIX E

SOIL LABORATORY TEST RESULTS

Drawings C 7091-A4 through -A9,
" " " -A25 through -A27,
" " " -A37 through -A39,
" " " -A52 through -A67

WARZYN
ENGINEERING INC

LOG OF TEST BORING

General Notes

Descriptive Soil Classification

GRAIN SIZE TERMINOLOGY

Soil Fraction	Particle Size	U.S. Standard Sieve Size
Boulders	Larger than 12″	Larger than 12″
Cobbles	3″ to 12″	3″ to 12″
Gravel: Coarse	¾″ to 3″	¾″ to 3″
Fine	4.76 mm to ¾″	#4 to ¾″
Sand: Coarse	2.00 mm to 4.76 mm	#10 to #4
Medium	0.42 mm to 2.00 mm	#40 to #10
Fine	0.074 mm to 0.42 mm	#200 to #40
Silt	0.005 mm to 0.074 mm	Smaller than #200
Clay	Smaller than 0.005 mm	Smaller than #200

Plasticity characteristics differentiate between silt and clay.

GENERAL TERMINOLOGY

Physical Characteristics
Color, moisture, grain shape, fineness, etc.

Major Constituents
Clay, silt, sand, gravel

Structure
Laminated, varved, fibrous, stratified, cemented, fissured, etc.

Geologic Origin
Glacial, alluvial, eolian, residual, etc.

RELATIVE DENSITY

Term	"N" Value
Very Loose	0-4
Loose	4-10
Medium Dense	10-30
Dense	30-50
Very Dense	Over 50

RELATIVE PROPORTIONS OF COHESIONLESS SOILS

Proportional Term	Defining Range By Percentage of Weight
Trace	0%- 5%
Little	5%-12%
Some	12%-35%
And	35%-50%

CONSISTENCY

Term	q_u-tons/sq. ft.
Very Soft	0.0 to 0.25
Soft	0.25 to 0.50
Medium	0.50 to 1.0
Stiff	1.0 to 2.0
Very Stiff	2.0 to 4.0
Hard	Over 4.0

ORGANIC CONTENT BY COMBUSTION METHOD

Soil Description	Loss on Ignition
Non Organic	Less than 4%
Organic Silt/Clay	4-12%
Sedimentary Peat	12-50%
Fibrous and Woody Peat	More than 50%

PLASTICITY

Term	Plastic Index
None to Slight	0-4
Slight	5-7
Medium	8-22
High to Very High	Over 22

The penetration resistance, N, is the summation of the number of blows required to effect two successive 6″ pentrations of the 2″ split-barrel sampler. The sampler is driven with a 140 lb. weight falling 30″ and is seated to a depth of 6″ before commencing the standard penetration test.

Symbols

DRILLING AND SAMPLING

CS—Continuous Sampling
RC—Rock Coring: Size AW, BW, NW, 2″ W
RQD—Rock Quality Designator
RB—Rock Bit
FT—Fish Tail
DC—Drove Casing
C—Casing: Size 2½″, NW, 4″, HW
CW—Clear Water
DM—Drilling Mud
HSA—Hollow Stem Auger
FA—Flight Auger
HA—Hand Auger
COA—Clean-Out Auger
SS—2″ Diameter Split-Barrel Sample
2ST—2″ Diameter Thin-Walled Tube Sample
3ST—3″ Diameter Thin-Walled Tube Sample
PT—3″ Diameter Piston Tube Sample
AS—Auger Sample
WS—Wash Sample
PTS—Peat Sample
PS—Pitcher Sample
NR—No Recovery
S—Sounding
PMT—Borehole Pressuremeter Test
VS—Vane Shear Test
WPT—Water Pressure Test

LABORATORY TESTS

q_p—Penetrometer Reading, tons/sq. ft.
q_u—Unconfined Strength, tons/sq. ft.
W—Moisture Content, %
LL—Liquid Limit, %
PL—Plastic Limit, %
SL—Shrinkage Limit, %
LI—Loss on Ignition, %
D—Dry Unit Weight, lbs./cu. ft.
pH—Measure of Soil Alkalinity or Acidity
FS—Free Swell, %

WATER LEVEL MEASUREMENT

▽—Water Level at time shown
NW—No Water Encountered
WD—While Drilling
BCR—Before Casing Removal
ACR—After Casing Removal
CW—Caved and Wet
CM—Caved and Moist

Note: Water level measurements shown on the boring logs represent conditions at the time indicated and may not reflect static levels, especially in cohesive soils.

UNIFIED SOIL CLASSIFICATION SYSTEM

COARSE-GRAINED SOILS

(More than half of material is larger than No. 200 seive size.)

GRAVELS	**Clean Gravels** (Little or no fines)	
	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
	GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
	Gravels with Fines (Appreciable amount of fines)	
	GM $^{d}_{u}$	Silty gravels, gravel-sand-silt mixtures
	GC	Clayey gravels, gravel-sand-clay mixtures
SANDS	**Clean Sands** (Little or no fines)	
	SW	Well-graded sands, gravelly sands, little or no fines
	SP	Poorly graded sands, gravelly sands, little or no fines
	Sands with Fines (Appreciable amount of fines)	
	SM $^{d}_{u}$	Silty sands, sand-silt mixtures
	SC	Clayey sands, sand-clay mixtures

FINE-GRAINED SOILS

(More than half of material is smaller than No. 200 sieve.)

SILTS AND CLAYS Liquid limit less than 50%	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
	OL	Organic silts and organic silty clays of low plasticity
SILTS AND CLAYS Liquid limit greater than 50%	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
	CH	Inorganic clays of high plasticity, fat clays
	OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS	PT	Peat and other highly organic soils

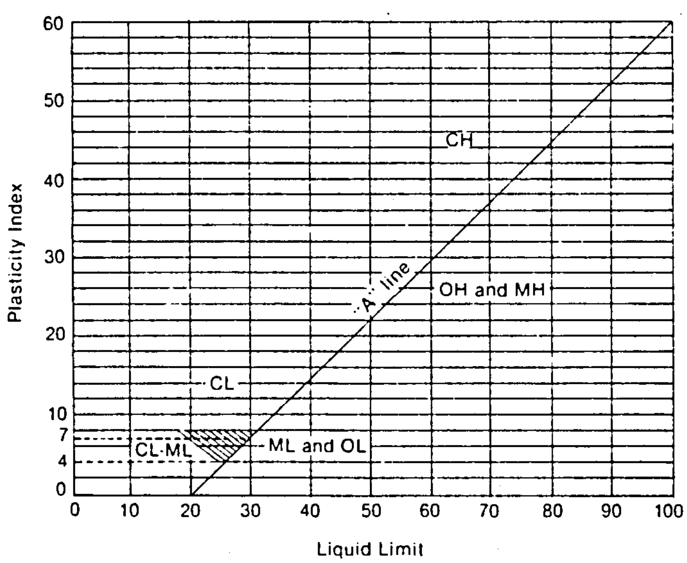
LABORATORY CLASSIFICATION CRITERIA

GW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} X D_{60}}$ between 1 and 3	
GP	Not meeting all gradation requirements for GW	
GM	Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
GC	Atterberg limits above "A" line with P.I. greater than 7	
SW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} X D_{60}}$ between 1 and 3	
SP	Not meeting all gradation requirements for SW	
SM	Atterberg limits below "A" line or P.I. less than 4	Limits plotting in hatched zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols.
SC	Atterberg limits above "A" line with P.I. greater than 7	

Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows:

Less than 5 per cent	GW, GP, SW, SP
More than 12 per cent	GM, GC, SM, SC
5 to 12 per cent	Borderline cases requiring dual symbols

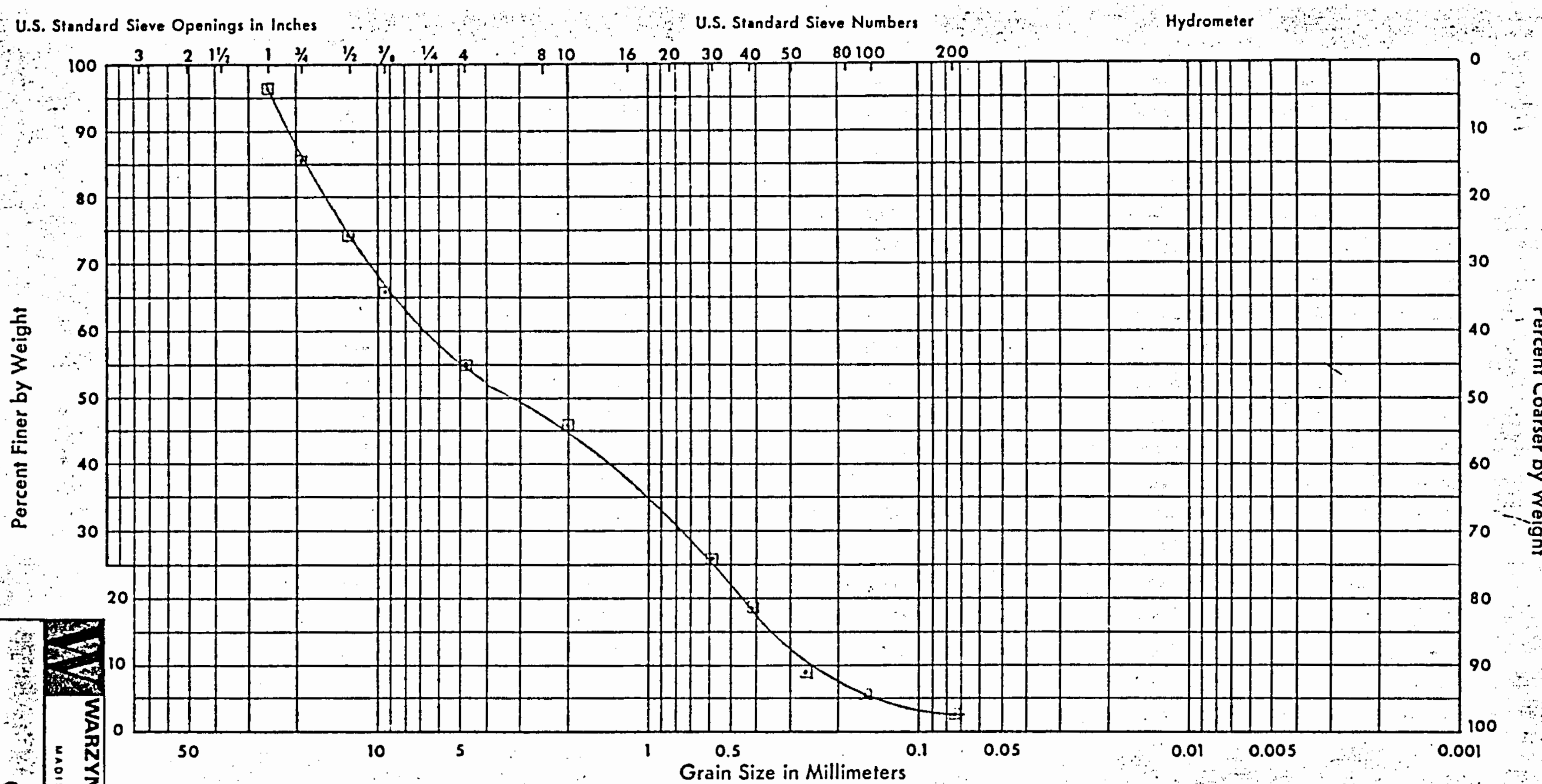
PLASTICITY CHART



For classification of fine-grained soils and fine fraction of coarse-grained soils.

Atterberg Limits plotting in hatched area are borderline classifications requiring use of dual symbols.

Equation of A-line: PI = 0.73 (LL - 20)



COARSE GRAVEL	FINE GRAVEL	COARSE SAND	MEDIUM SAND	FINE SAND	SILT	CLAY

Unified Classification System (ASTM D2487)

LABORATORY ANALYSIS PERFORMED B
WISCONSIN TESTING LABORATORIES

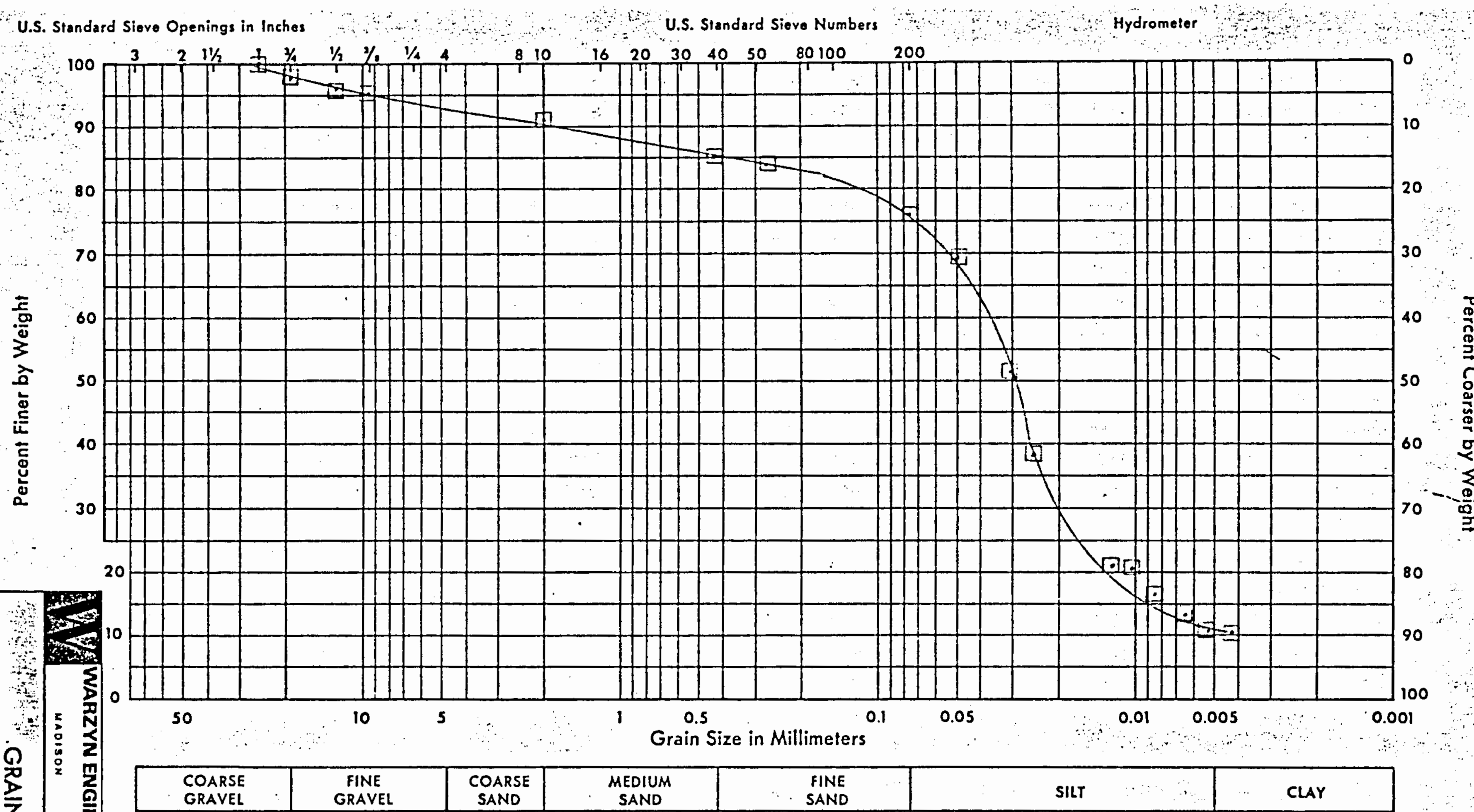
Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
⊡	E4	13-39'				45	53	2	0	SAND AND GRAVEL, TRACE SILT. (SP)

WARZYN ENGINEERING AND SERVICE COMPANY, INC.
Consulting Engineers
MADISON WISCONSIN

GRAIN SIZE ANALYSIS

DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

DWN JLC
CHK'D TCP
APP'D
DATE 6/30/77
C7091-A4



Unified Classification System (ASTM D2487)

LABORATORY ANALYSIS PERFORMED BY WISCONSIN TESTING LABORATORIES

Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
⊡	E5	4-6'				7	17	65	11	SILT, SOME SAND, LITTLE CLAY, LITTLE GRAVEL (CL)

DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

GRAIN SIZE ANALYSIS

WARZYN ENGINEERING AND SERVICE COMPANY, INC.
Consulting Engineers
MADISON WISCONSIN

DWN JLC
CHK'D TCP
APP'D Dennis R. Vick
DATE 6-30-77
7091-A5

DWN JLC

CHK'D TCP

APP'D [illegible]

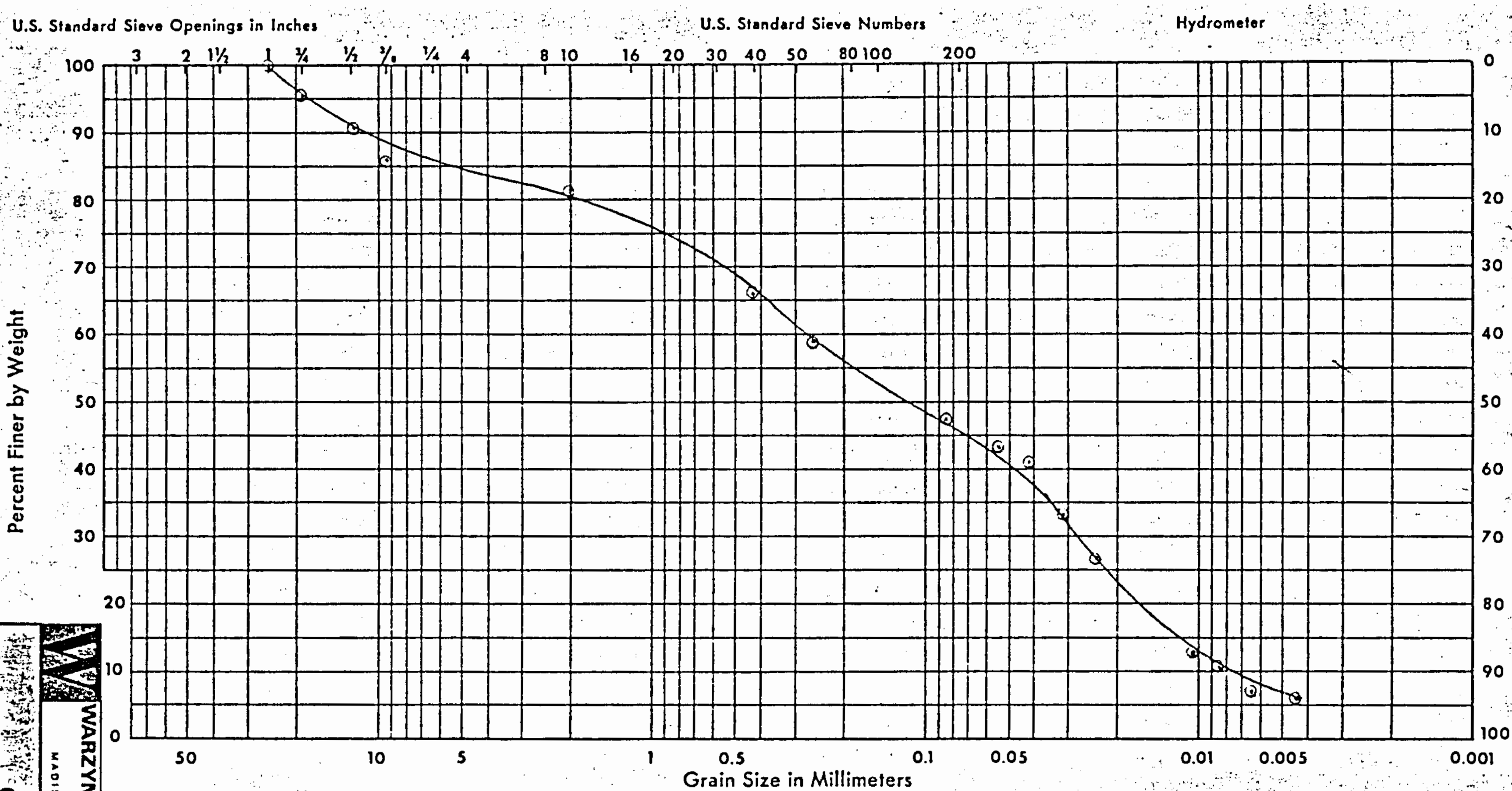
DATE 6-30-77

C7091-A6

DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

GRAIN SIZE ANALYSIS

WARZYN ENGINEERING AND SERVICE COMPANY, INC.
MADISON *Consulting Engineers* WISCONSIN

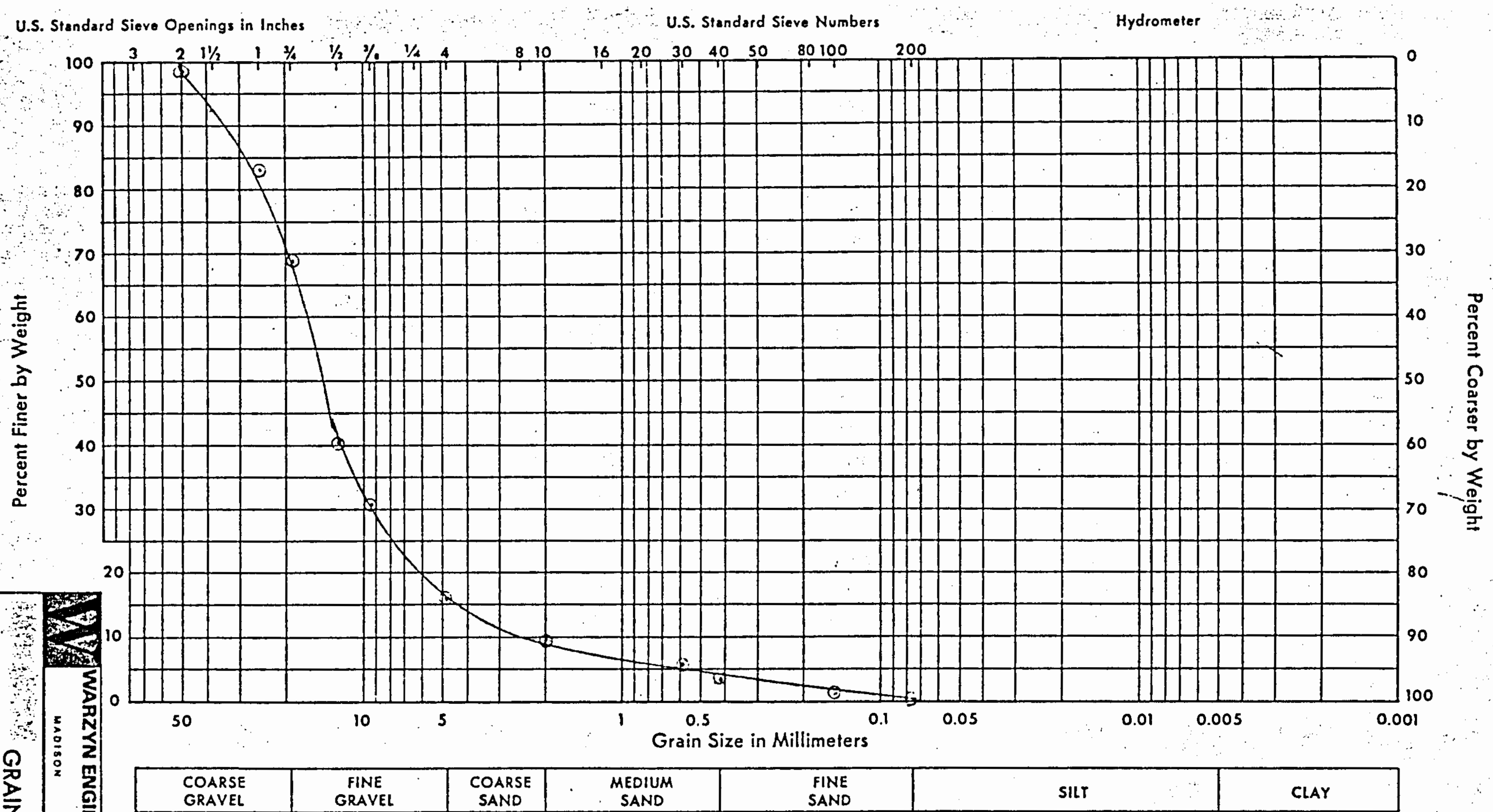


COARSE GRAVEL	FINE GRAVEL	COARSE SAND	MEDIUM SAND	FINE SAND	SILT	CLAY

Unified Classification System (ASTM D2487)

LABORATORY ANALYSIS PERFORMED B
WISCONSIN TESTING LABORATORIES

Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
⊙	E4	10-12'				15	39	39	7	SAND AND SILT, SOME GRAVEL, TRACE CL
										(SM)



Unified Classification System (ASTM D2487)

LABORATORY ANALYSIS PERFORMED BY WISCONSIN TESTING LABORATORIES

Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
⊙	E 1	0-25				83	16	1	0	GRAVEL, SOME SAND, TRACE SILT (GW

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Consulting Engineers
MADISON WISCONSIN

GRAIN SIZE ANALYSIS

DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

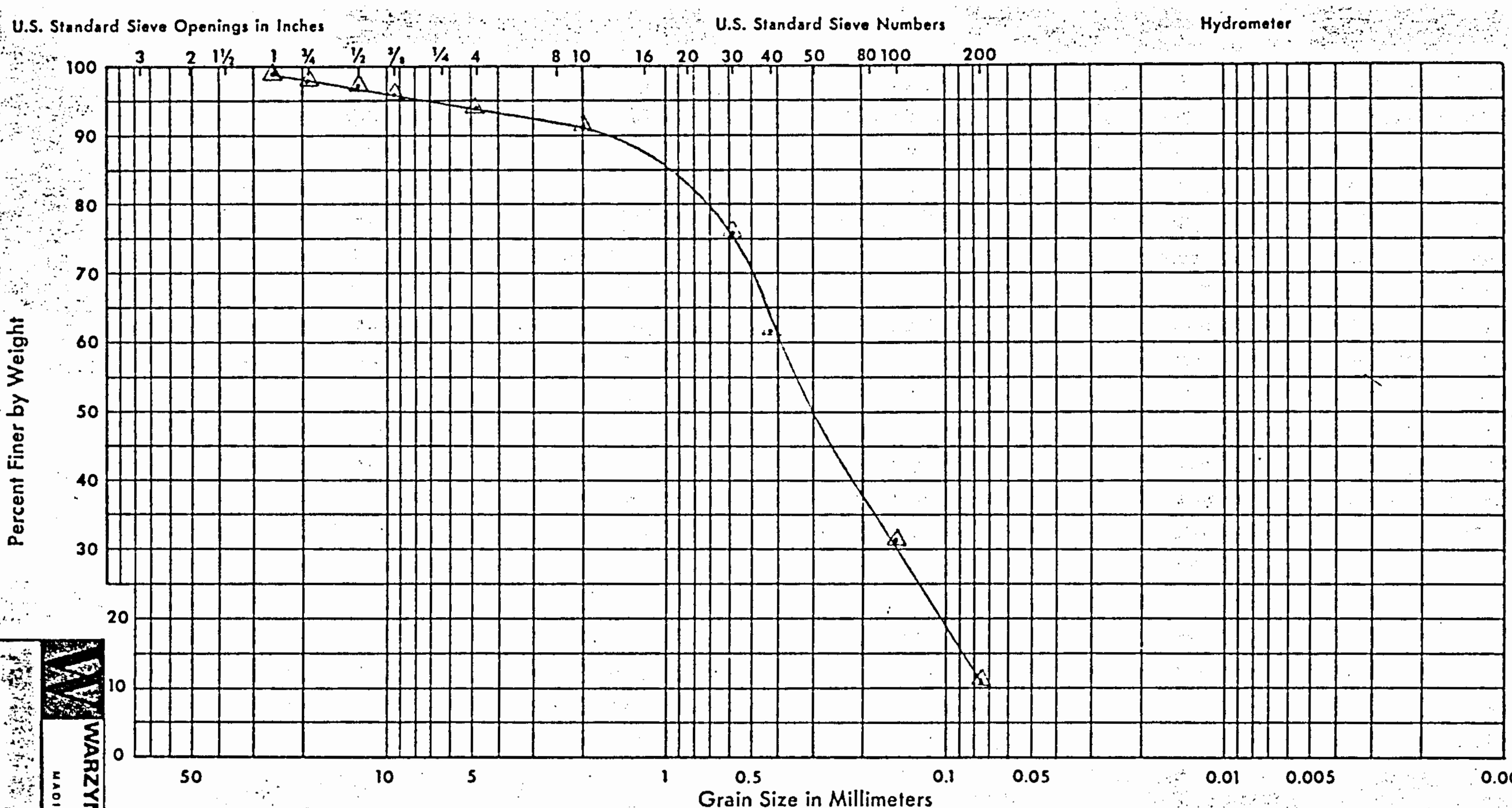
DWN JLC
CHK'D JCP
APP'D Daniel R. Vite
DATE 6-30-77
C7091-A-7

GRAIN SIZE ANALYSIS

WARZYN ENGINEERING AND SERVICE COMPANY, INC.
Consulting Engineers
MADISON WISCONSIN

DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

DWN JLC
CHK'D TCP
APP'D Daniel L. Vite
DATE 6-30-77
C7091-A8

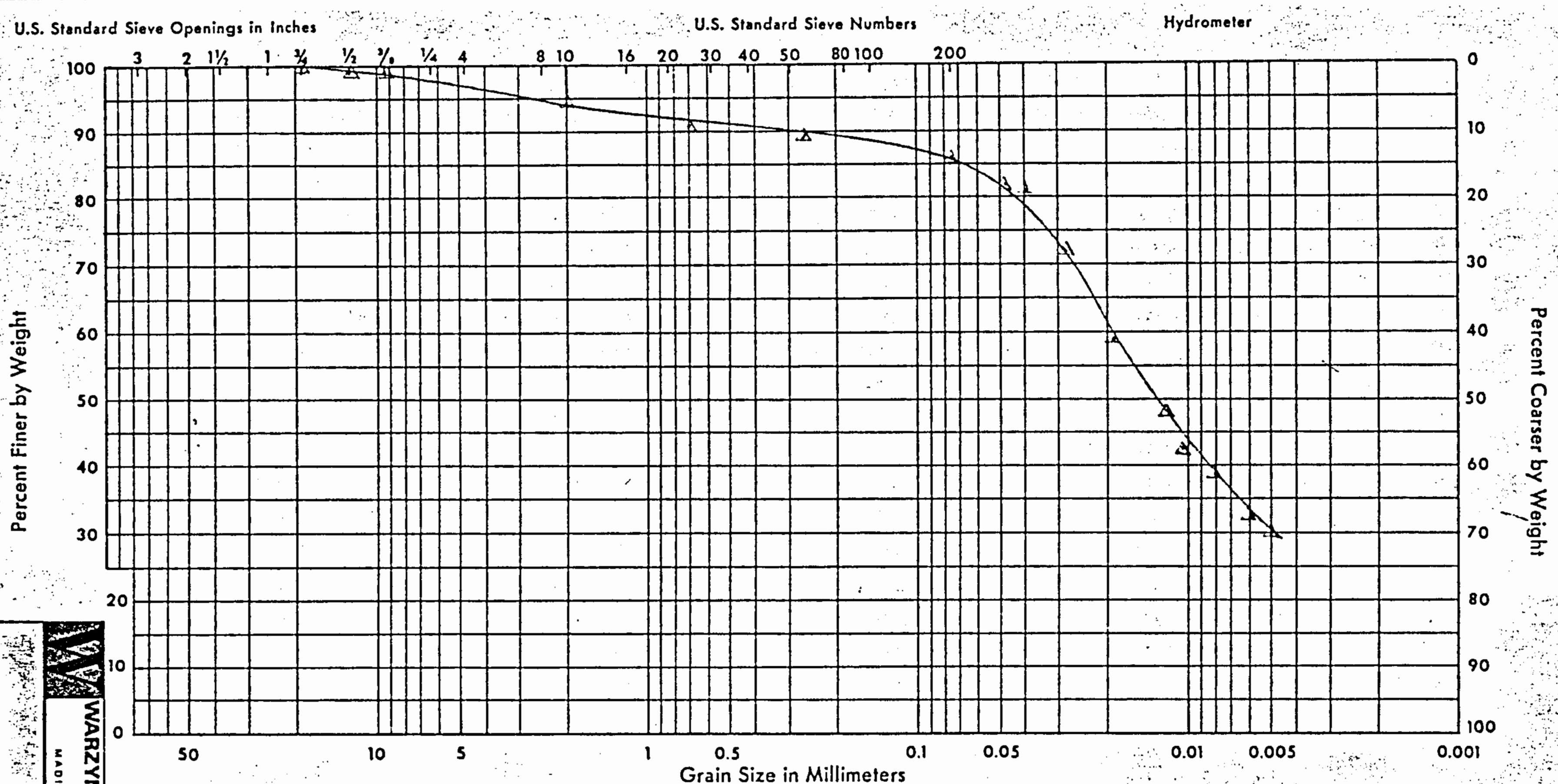


COARSE GRAVEL	FINE GRAVEL	COARSE SAND	MEDIUM SAND	FINE SAND	SILT	CLAY

Unified Classification System (ASTM D2487)

LABORATORY ANALYSIS PERFORMED BY WISCONSIN TESTING LABORATORIES

Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
△	E3	7-20'	—	—	—	6	83	11	0	SAND, LITTLE GRAVEL, LITTLE SILT (SP-SM)



COARSE GRAVEL	FINE GRAVEL	COARSE SAND	MEDIUM SAND	FINE SAND	SILT	CLAY

Unified Classification System (ASTM D2487)

LABORATORY ANALYSIS PERFORMED BY WISCONSIN TESTING LABORATORIES

Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
△	E 4	2-4'				3	11	56	30	SILT, SOME CLAY, LITTLE SAND, TR. GRAV (CL)

DWN JLC

CHK'D TCP

APP'D Daniel R. Viste

DATE 6-30-77

C7091-A91

WARZYN ENGINEERING AND SERVICE COMPANY, INC.
Consulting Engineers
MADISON WISCONSIN

GRAIN SIZE ANALYSIS

DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

DWN TCP

CHK'D HAK

APP'D Henry A Kock

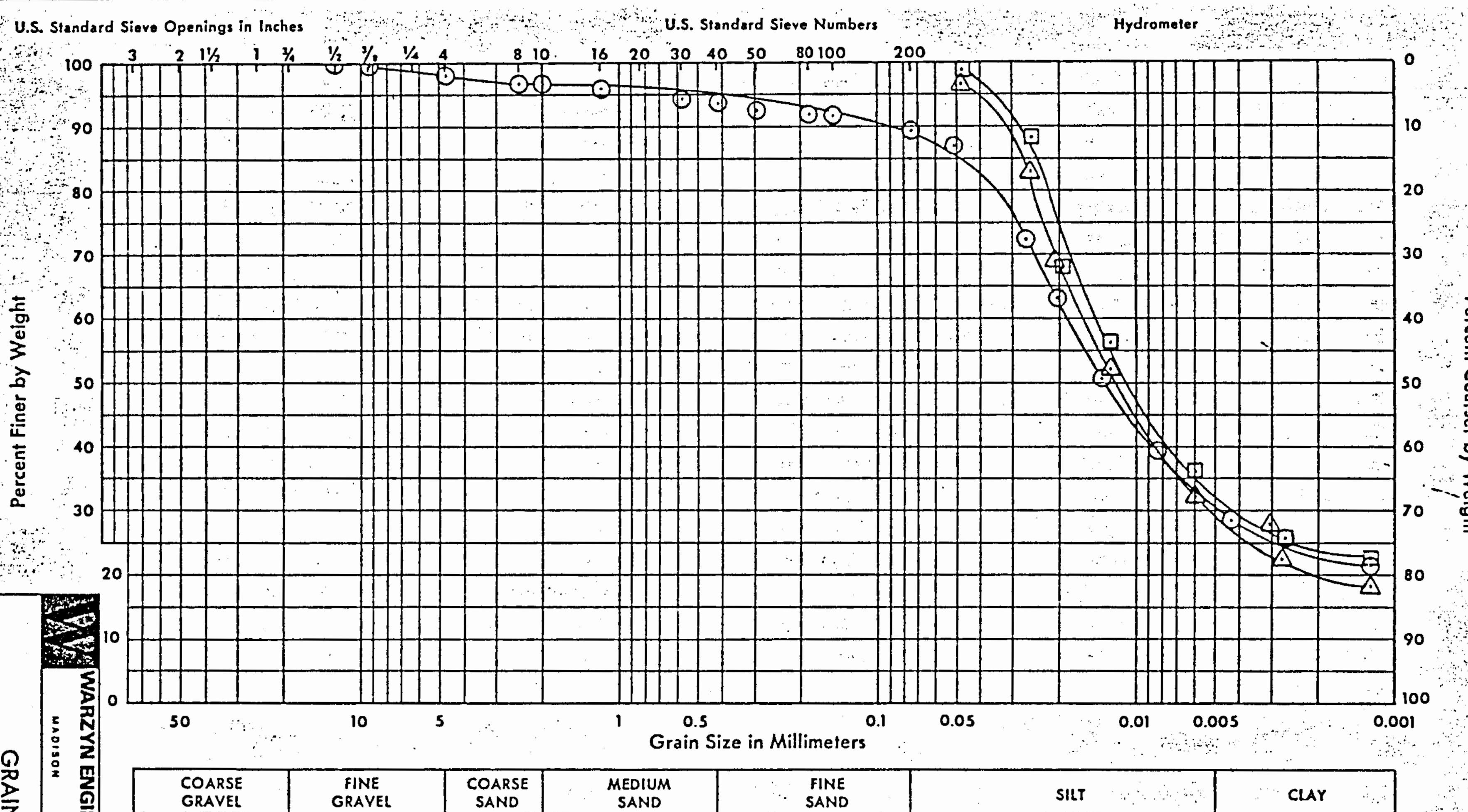
DATE 9/19/77

C7091-A25

DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

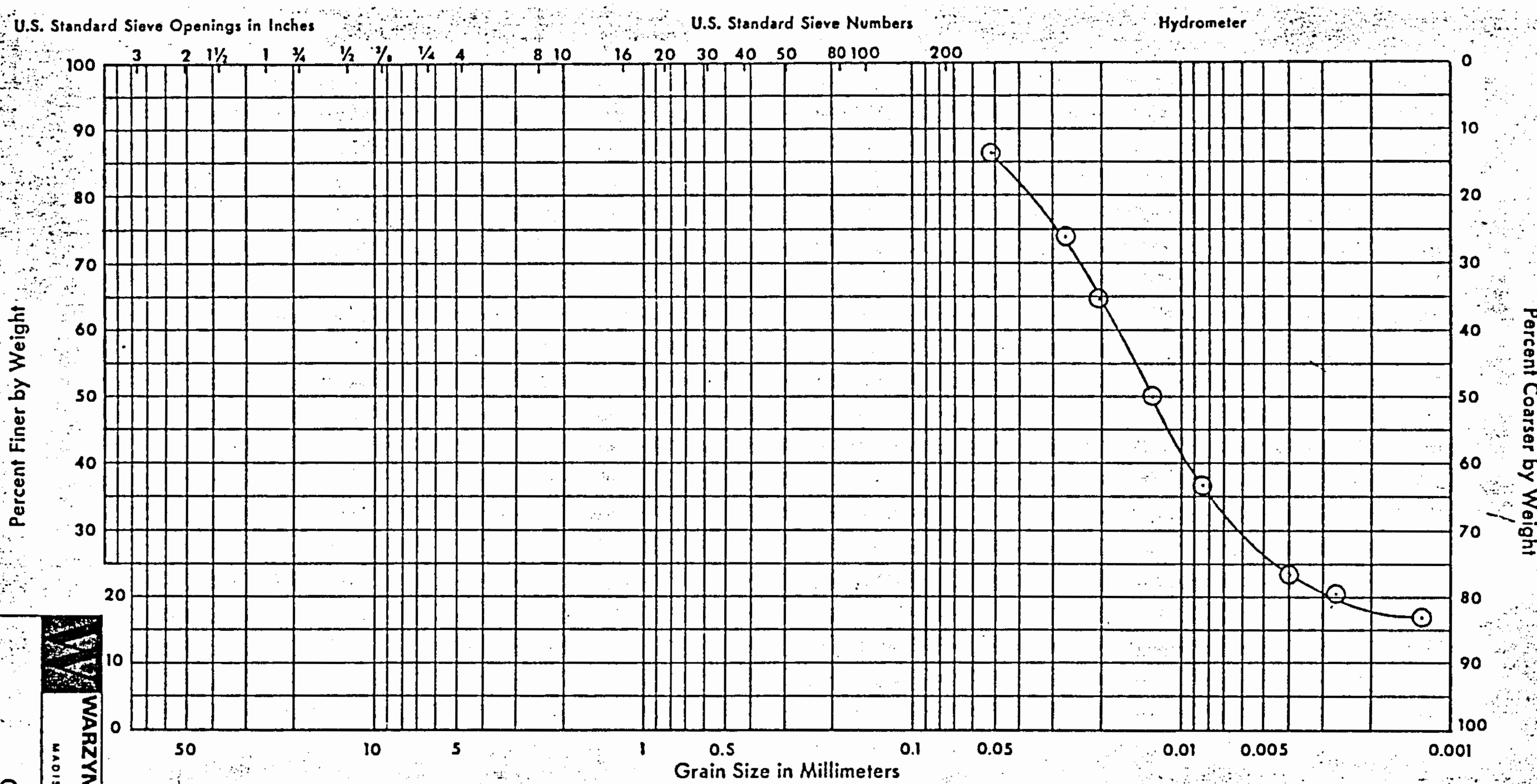
GRAIN SIZE ANALYSIS

WARZYN ENGINEERING AND SERVICE COMPANY, INC.
MADISON Consulting Engineers WISCONSIN



Unified Classification System (ASTM D2487)

Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification (USCS)
⊙	S#2	Liner III	23.6	35.2	15.5	1.9	8.6	58.6	30.9	Samples 2, 4 and 5 - An Inorganic Silty Clay of Low to Medium Plasticity (CL) With An Estimated Permeability of 10^{-6} to 10^{-8} cm/sec.
⊡	S#4	Liner III	30.9	38.4	18.7	0	1.0	66.4	32.6	
△	S#5	Liner III	32.9	40.9	20.8	0	3.3	66.5	30.2	



COARSE GRAVEL	FINE GRAVEL	COARSE SAND	MEDIUM SAND	FINE SAND	SILT	CLAY

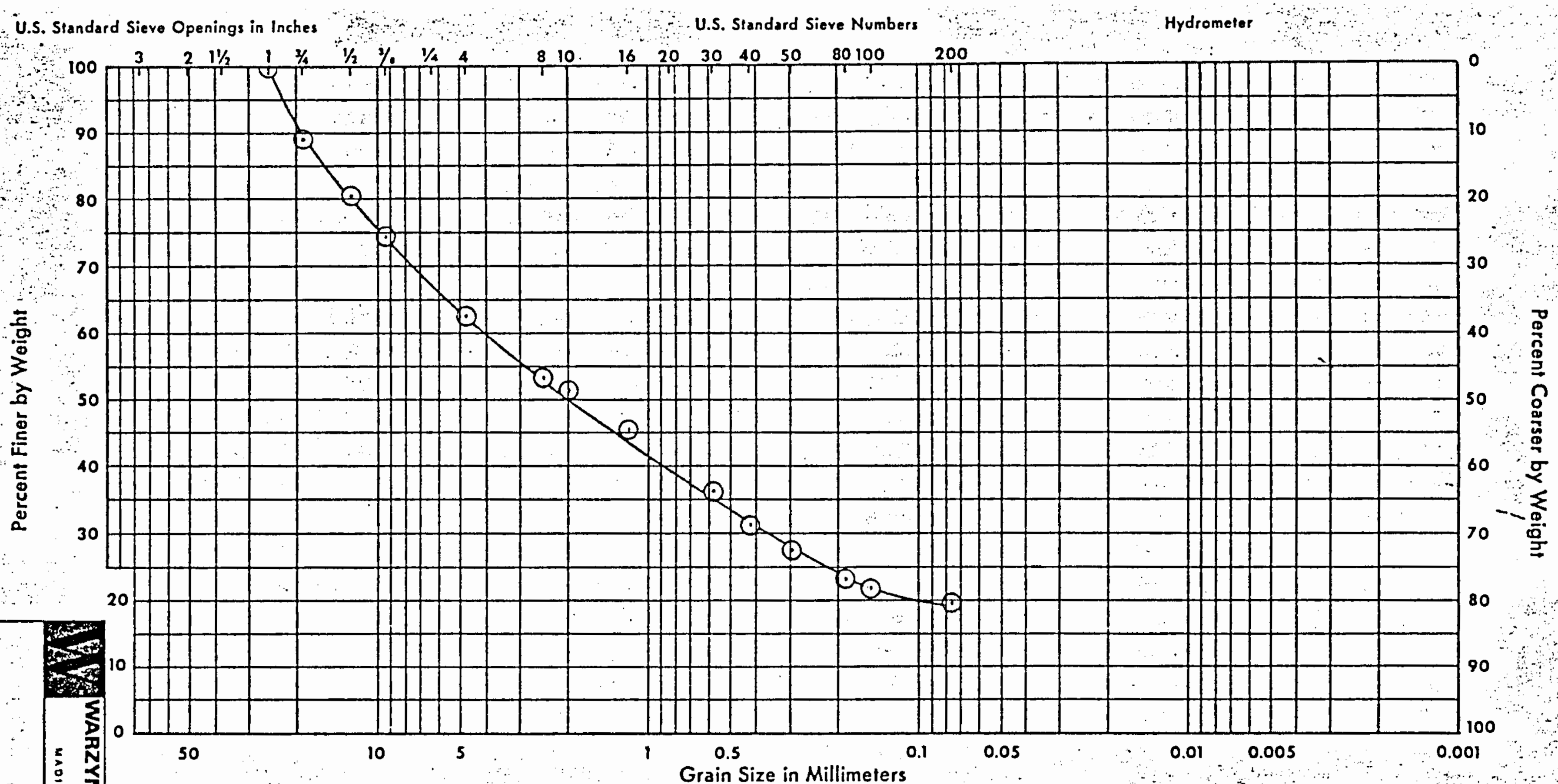
Unified Classification System (ASTM D2487)

Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification (USCS)
⊙	S#6		36.8	35.5	15.9	0	1.4	71.6	27.0	A WATERLAID DEPOSIT CLASSIFIED AS AN INORGANIC SILTY CLAY OF LOW TO MEDIUM PLASTICITY (CL) WHICH OVERLAYS PORTIONS OF LINER II WITH A DEPTH OF FROM 1 TO 6 INCHES. EST. PERMEABILITY 10^{-6} TO 10^{-8} cm/sec

GRAIN SIZE ANALYSIS
DELAFIELD SANITARY LANDFILL
WATER LAID DEPOSIT
DELAFIELD WISCONSIN

WARZYN ENGINEERING AND SERVICE COMPANY, INC.
Consulting Engineers
MADISON WISCONSIN

DWN TCP
CHK'D HAK
APP'D Henry A Koch
DATE 9/19/77
C7091-A26



COARSE GRAVEL	FINE GRAVEL	COARSE SAND	MEDIUM SAND	FINE SAND	SILT	CLAY

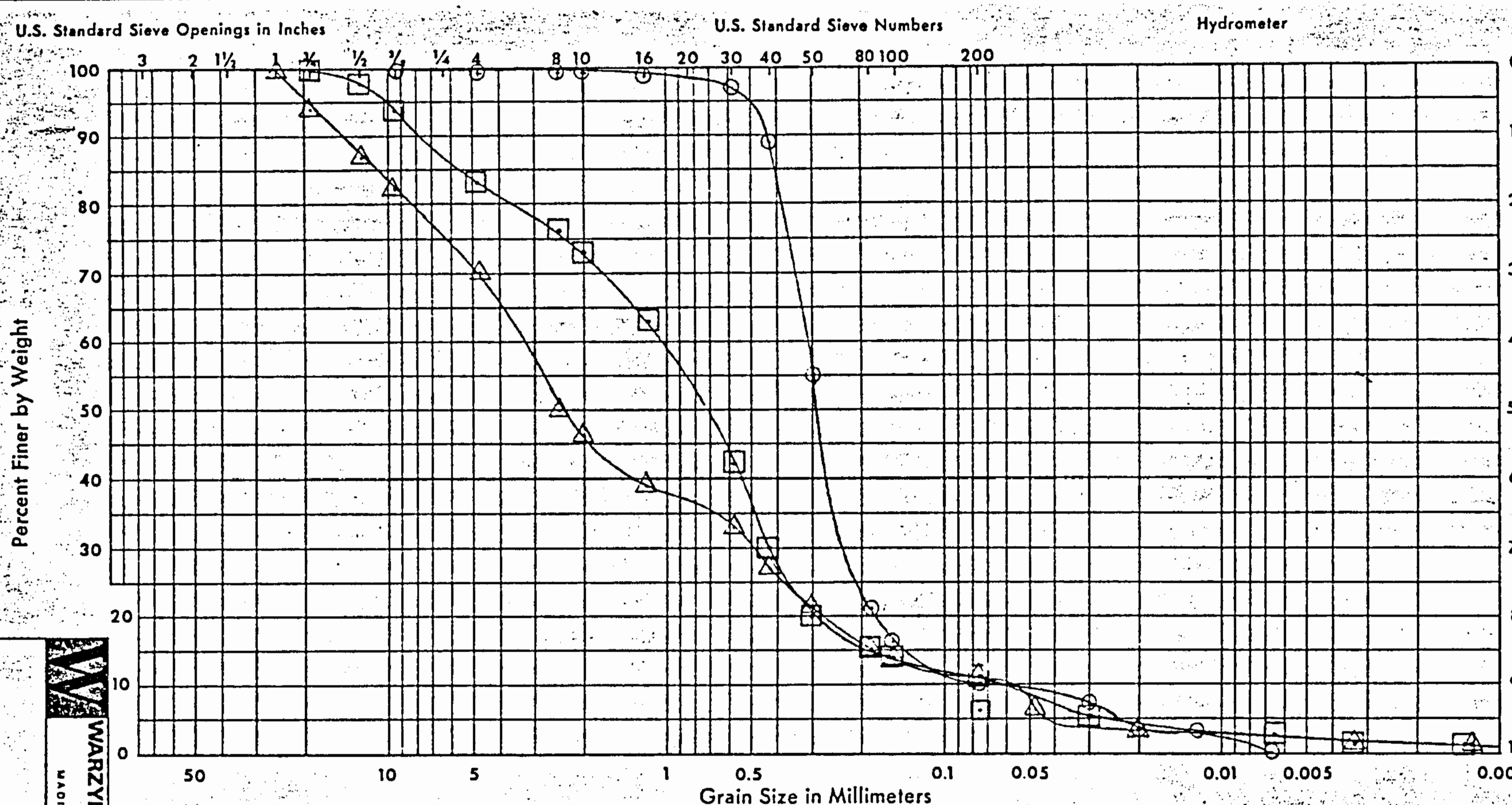
Unified Classification System (ASTM D2487)

Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification (USCS)
⊙	S#7	---	---	---	---	37.2	43.0	19.8	---	A WELL GRADED FINE TO COARSE SAND AND GRAVEL WITH SOME SILT (GM-SM). ESTIMATED PERMEABILITY 10^{-3} TO 10^{-6} cm/sec. THIS SOIL UNDERLAYS THE LINER.

GRAIN SIZE ANALYSIS
DELAFIELD SANITARY LANDFILL
LINER SUBBASE
DELAFIELD, WISCONSIN

WARZYN ENGINEERING AND SERVICE COMPANY, INC.
Consulting Engineers
MADISON WISCONSIN

DWN TCP
CHK'D HAK
APP'D Harry A. Kock
DATE 9/19/77
C7091-A27



COARSE GRAVEL	FINE GRAVEL	COARSE SAND	MEDIUM SAND	FINE SAND	SILT	CLAY

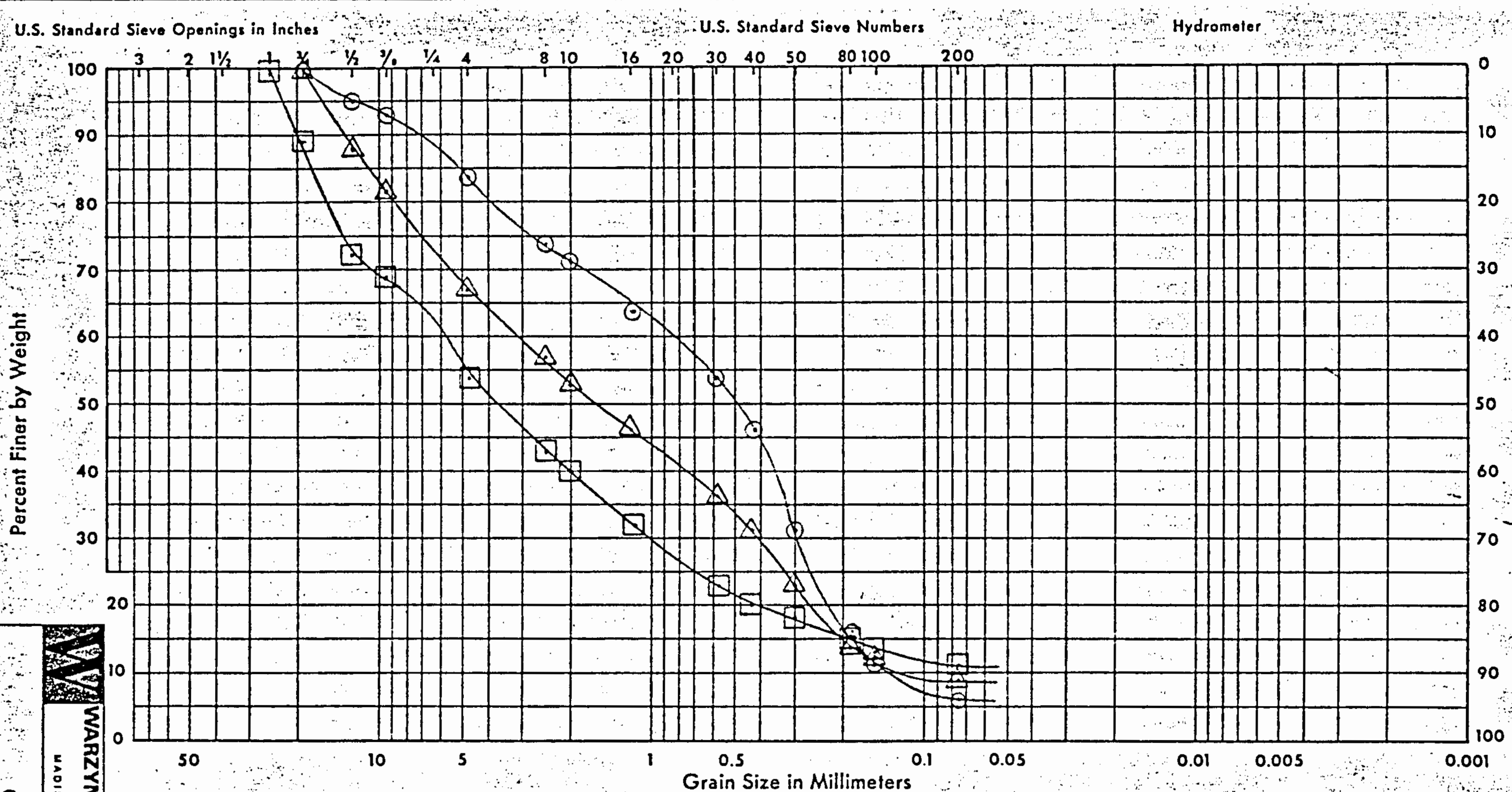
Unified Classification System (ASTM D2487)

Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
⊙	E7-12	60.0				1	89	10	0	SAND, LITTLE SILT, TRACE GRAVEL (SP-SM)
△	E8-4	20.0				30	59	9	2	SAND, SOME GRAVEL, LITTLE SILT, TRACE CLAY (SW-SM)
⊡	E8-12	60.0				17	73	8	2	SAND, SOME GRAVEL, LITTLE SILT, TRACE CLAY (SP-SM)

DWN RP
CHK'D DRY
APP'D Daniel E. Viste
DATE 1/19/79
C7091-A37

GRAIN SIZE ANALYSIS
DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

WARZYN ENGINEERING AND SERVICE COMPANY, INC.
Consulting Engineers
MADISON WISCONSIN



COARSE GRAVEL	FINE GRAVEL	COARSE SAND	MEDIUM SAND	FINE SAND	SILT	CLAY

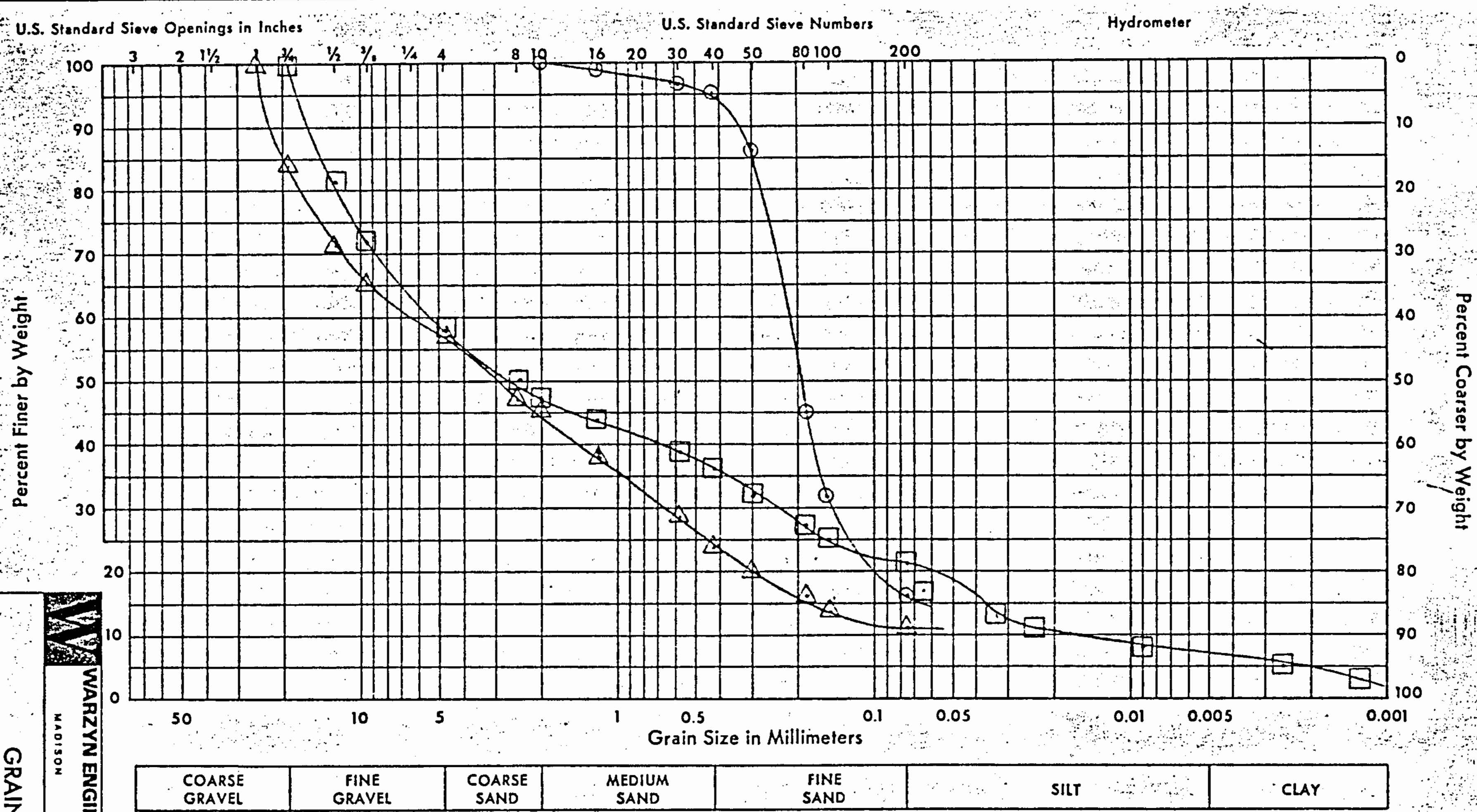
Unified Classification System (ASTM D2487)

Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
⊙	E9-4	20.0'				16	78	6		SAND, SOME GRAVEL, LITTLE SILT
										(SW-SM)
△	E9-7	35.0'				33	59	8		SAND, SOME GRAVEL, LITTLE SILT
										(SW-SM)
⊡	E10-3	70.0'				44	45	11		SAND AND GRAVEL, LITTLE SILT
										(SW-SM)

DWN RP
CHK'D DRV
APP'D Daniel E. Vick
DATE 1/19/79
C7091-A38

GRAIN SIZE ANALYSIS
DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

WARZYN ENGINEERING AND SERVICE COMPANY, INC.
Consulting Engineers
MADISON WISCONSIN



Unified Classification System (ASTM D2487)

Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
⊙	E10-9	100.0				0	85	15		SAND, SOME SILT AND (SM)
△	E11-3	75.0				43	46	11		SAND AND GRAVEL, LITTLE SILT (SW-SM)
⊡	E11-4	80.0		NON-PLASTIC		42	37	14	7	GRAVEL AND SAND, SOME SILT, LITTLE CLAY (SM)

DWN RP

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APP'D Daniel E. Viste

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C7091-A39

GRAIN SIZE ANALYSIS

DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

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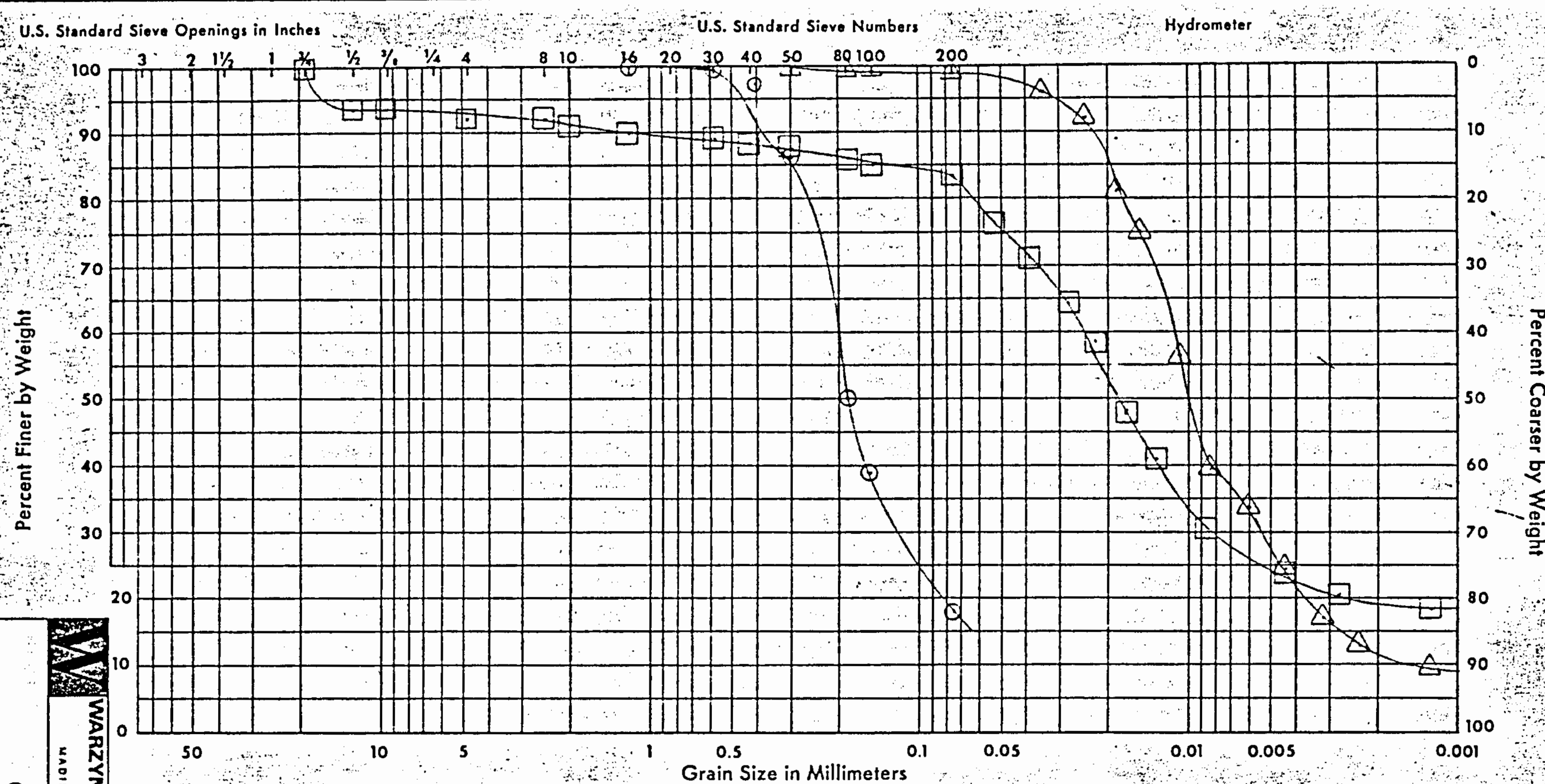

COARSE GRAVEL	FINE GRAVEL	COARSE SAND	MEDIUM SAND	FINE SAND	SILT	CLAY

Unified Classification System (ASTM D2487)

Curve	Sample	Depth	N.M.	LL.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
⊡	E2C-1	5'		39.6	18.2	8	9	59	24	SILTY CLAY, TRACE SAND AND GRAVEL (CL)
⊙	E2C-2	10'					82	18	—	SAND LITTLE SILT AND CLAY (SM
△	E2C-4	20'		21.5	6.2		1	72	27	CLAYEY SILT (CL-ML)

DWN RP
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DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

GRAIN SIZE ANALYSIS

WARZYN ENGINEERING AND SERVICE COMPANY, INC.
Consulting Engineers
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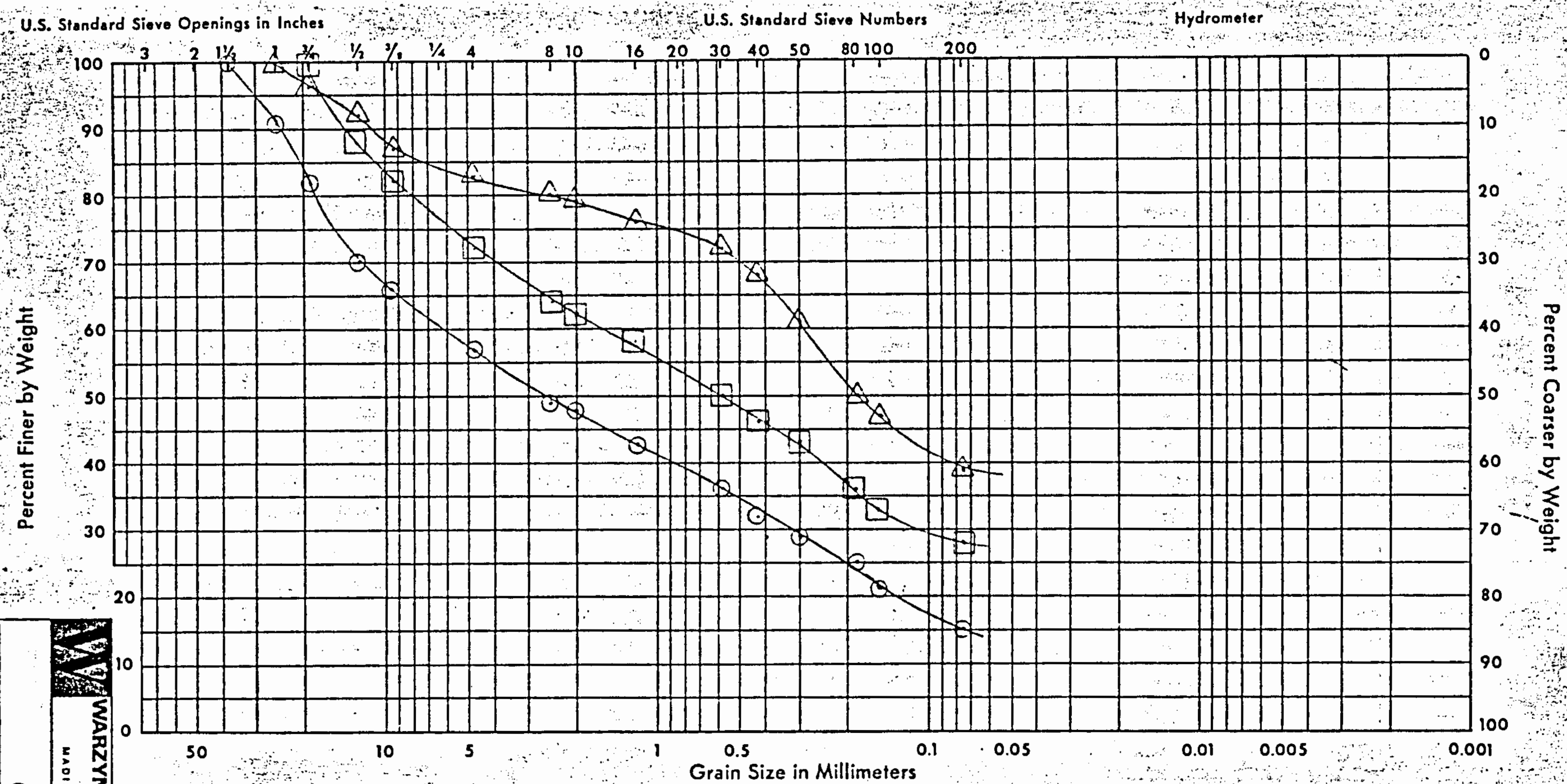

COARSE GRAVEL	FINE GRAVEL	COARSE SAND	MEDIUM SAND	FINE SAND	SILT	CLAY

Unified Classification System (ASTM D2487)

Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
⊡	E2C-6	30'		14.1	0	28	45	27	—	SAND, SOME GRAVEL AND SILT (SM)
⊙	E2C-7	35'				43	42	15	—	GRAVEL AND SAND, LITTLE SILT (SM)
△	E2C-10	50'				17	44	39	—	SAND, SOME SILT, CLAY, AND GRAVEL (SC)

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GRAIN SIZE ANALYSIS

DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

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APP'D Daniel C. Viete
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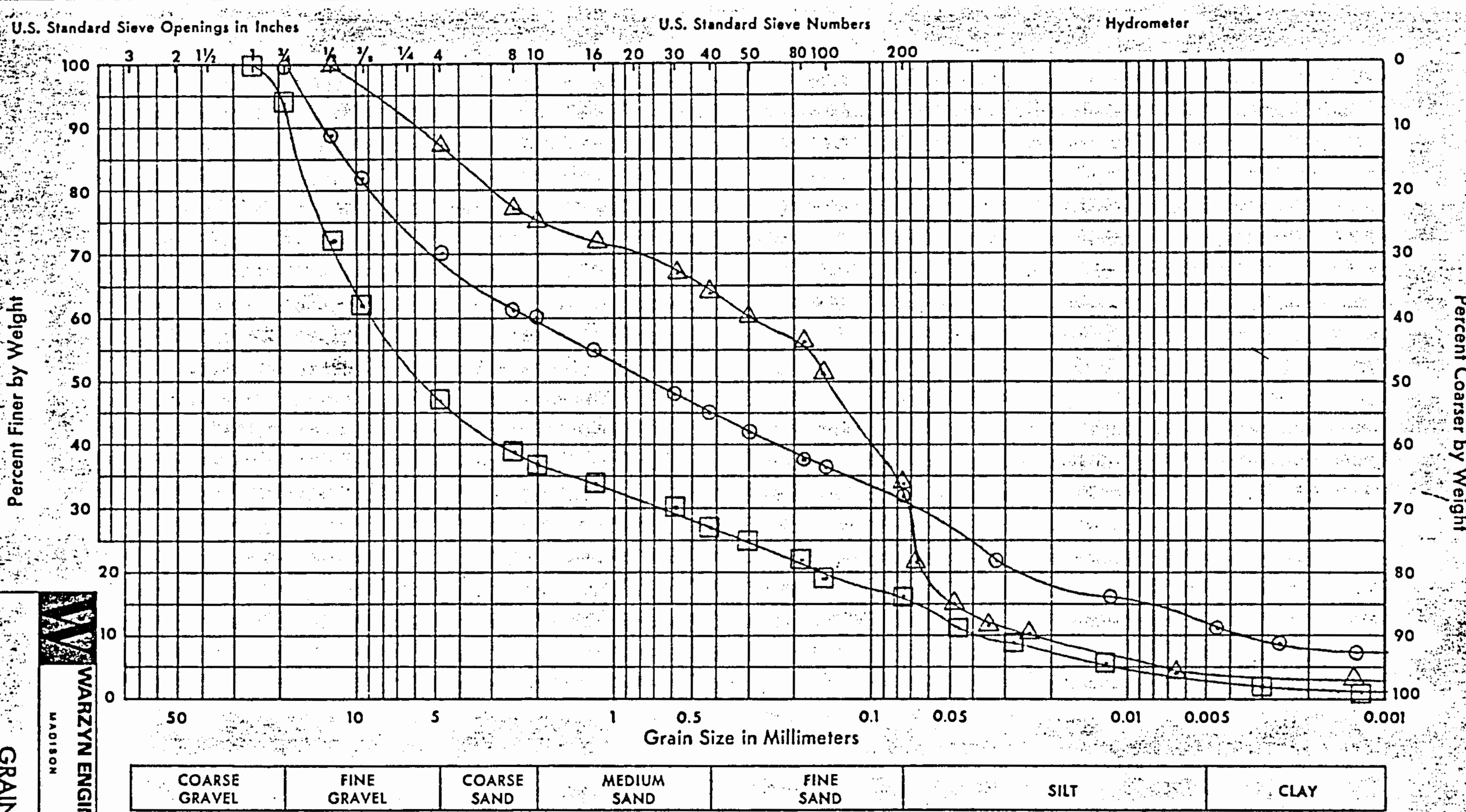
C7091-A53



Unified Classification System (ASTM D2487)

Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
⊙	E5B-2	10.0		19.7	5.5	30	38	21	11	SAND, SOME GRAVEL AND SILT, LITTLE CLAY (SC-SM)
△	E5B-9	45.0		NON PLASTIC		13	53	30	4	SAND, SOME SILT AND GRAVEL, TRACE CLAY (SM)
⊡	E5B-12	60.0				53	31	13	3	GRAVEL, SOME SAND AND SILT, TRACE CLAY (GM)

GRAIN SIZE ANALYSIS
DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

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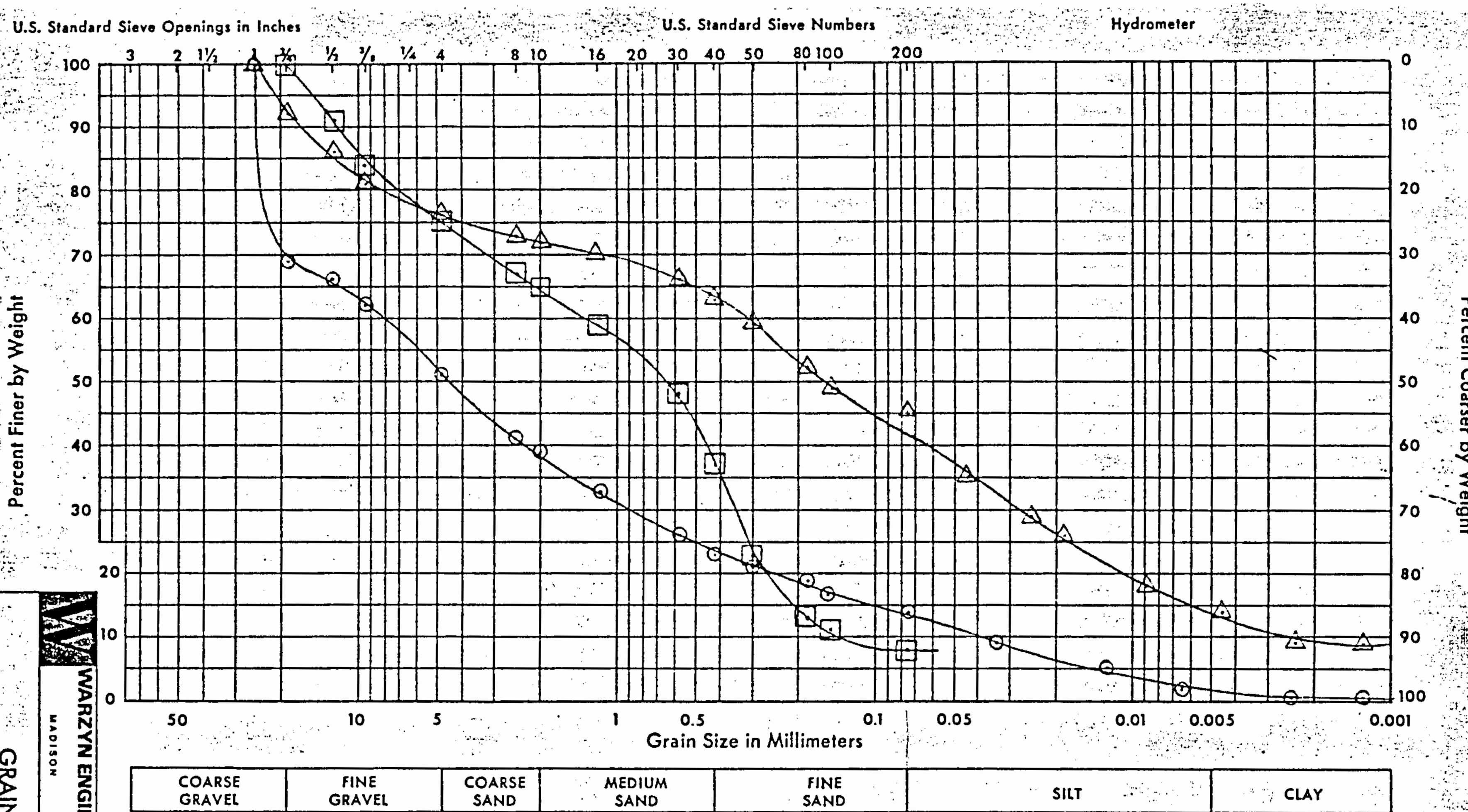

Unified Classification System (ASTM D2487)

Curve	Sample	Depth	N.M.	LL	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
⊙	EGB-3	15.0'				49	37	12	2	GRAVEL AND SAND, SOME SILT, TRACE CLAY (GM)
△	EGB-8	40.0'		12.4	1.6	24	31	31	14	SILTY SAND, SOME GRAVEL, AND CLAY (SM)
⊡	EGB-11	55.0'				25	67			SAND, SOME GRAVEL, LITTLE SILT (SP-SM)

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GRAIN SIZE ANALYSIS
DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

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CHK'D RCC
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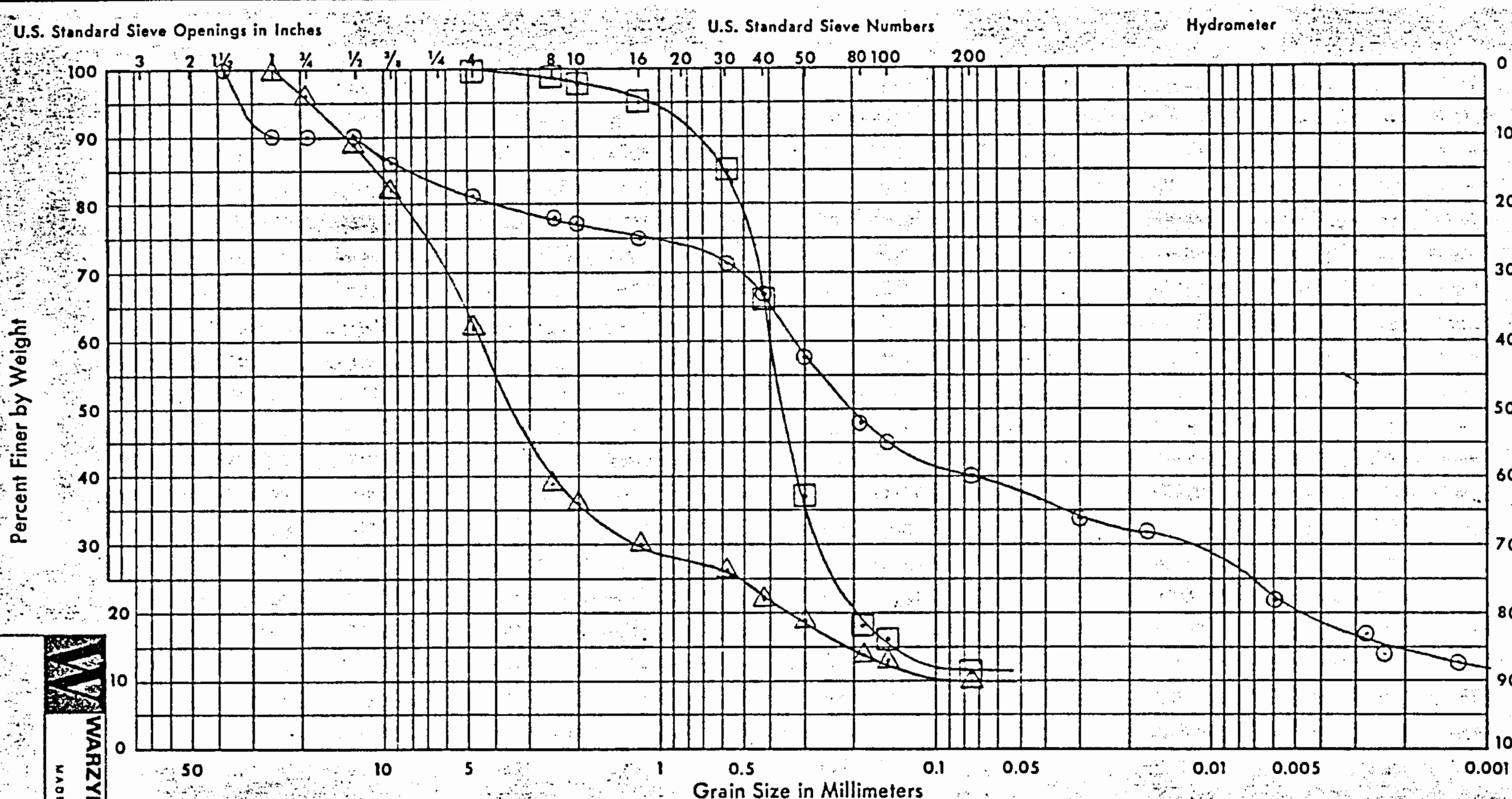
C7091-A56



COARSE GRAVEL	FINE GRAVEL	COARSE SAND	MEDIUM SAND	FINE SAND	SILT	CLAY

Unified Classification System (ASTM D2487)

Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
⊙	E6B-13	65.0'		12.7	3.4	18	40	22	2	SAND, SOME SILT, CLAY, AND GRAVEL (SM)
△	ET-8	40.0'				38	52	10		SAND AND GRAVEL, LITTLE SILT
										(SM-SP)
⊡	ET-10	50.0'				0	89	11		SAND, LITTLE SILT (SM-SP)

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GRAIN SIZE ANALYSIS
DELAFIELD SANITARY LANDFILL
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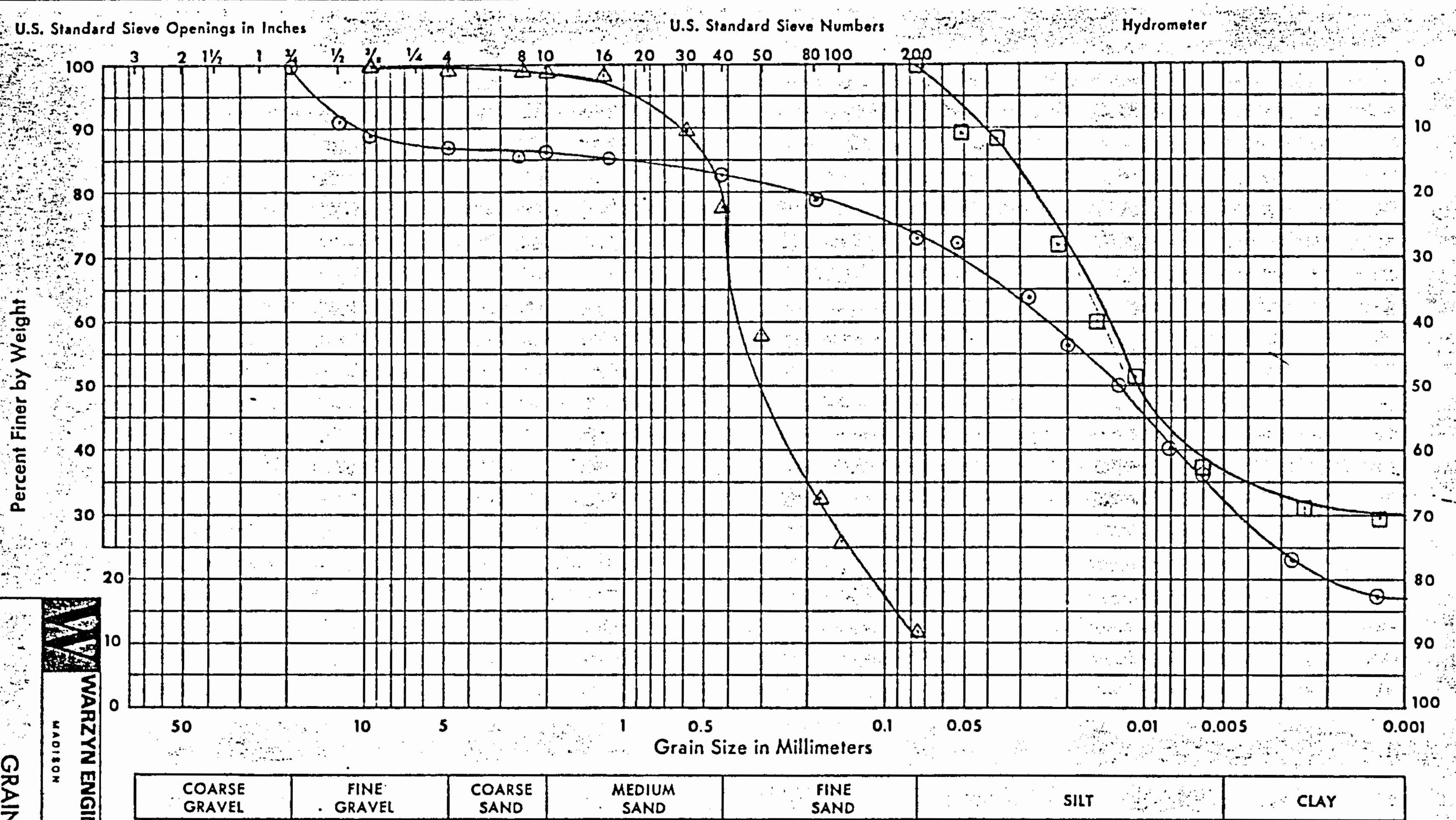
GRAIN SIZE ANALYSIS

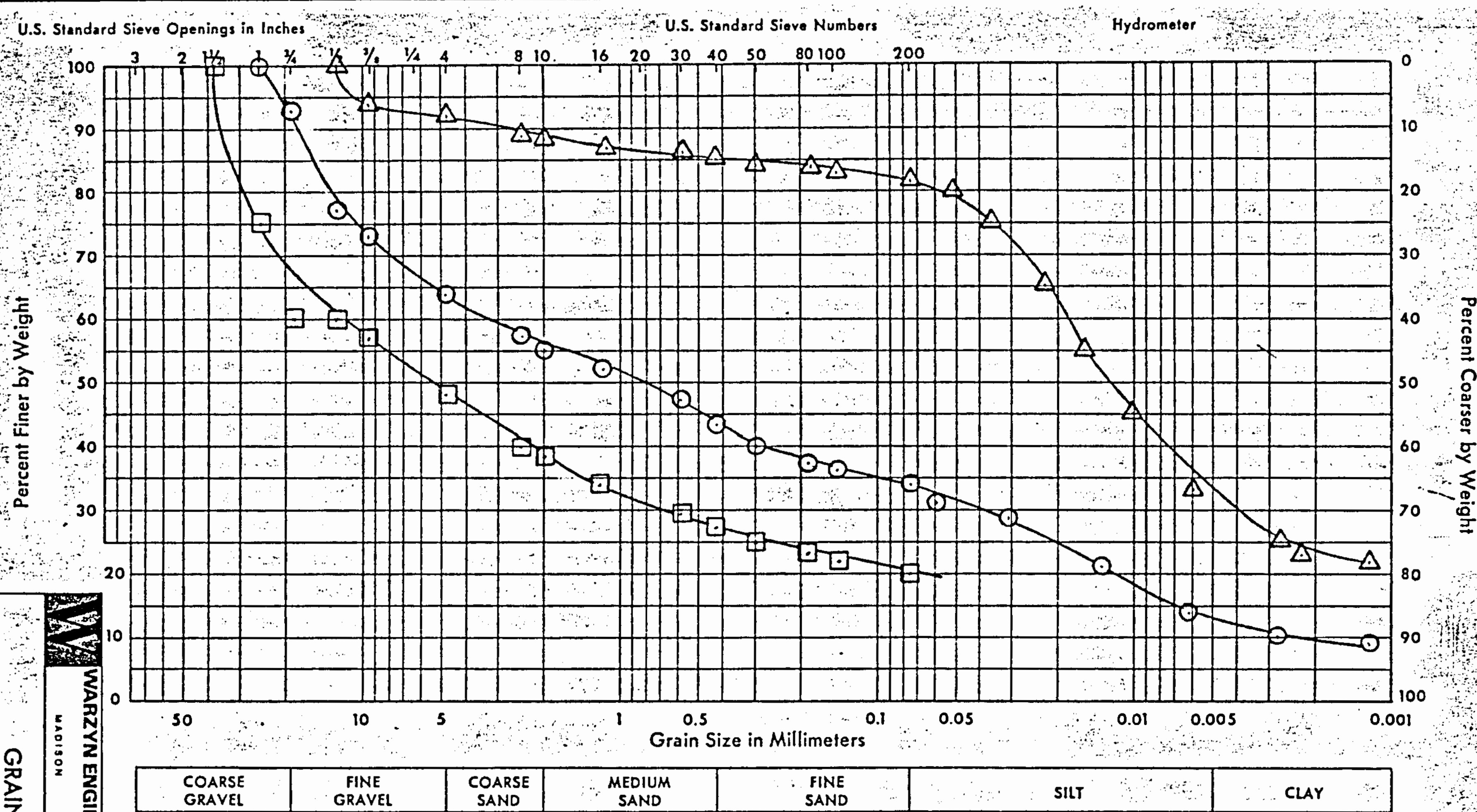
DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

DWN RP
CHK'D CJK
APP'D Daniel R. Vieto
DATE 1/19/79
C7091-A58



Unified Classification System (ASTM D2487)

Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
⊙	E11-7	100.0'		18.2	4.2	13	14	41	32	CLAYEY SILT, SOME SAND, AND GRAVEL (CL-ML)
△	E12-6	30.0				0	88	12		FINE TO MEDIUM SAND, LITTLE SILT AND CLAY (SP-SM)
⊡	E7-10							63	37	



Unified Classification System (ASTM D2487)

Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
⊙	E13	15.5'		24.8	9.0	36	30	21	13	GRAVEL, SOME SAND, SILT AND CLAY
										(CLAY LINER)
△	E15L	13.5'		38.5	16.4	8	10	53	29	SILT SOME CLAY LITTLE SAND AND
										GRAVEL (CLAY LINER) (CL)
⊡	E15L-2	16.0'				52	28	20		GRAVEL, SOME SAND AND SILT
										(SM)

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GRAIN SIZE ANALYSIS
DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

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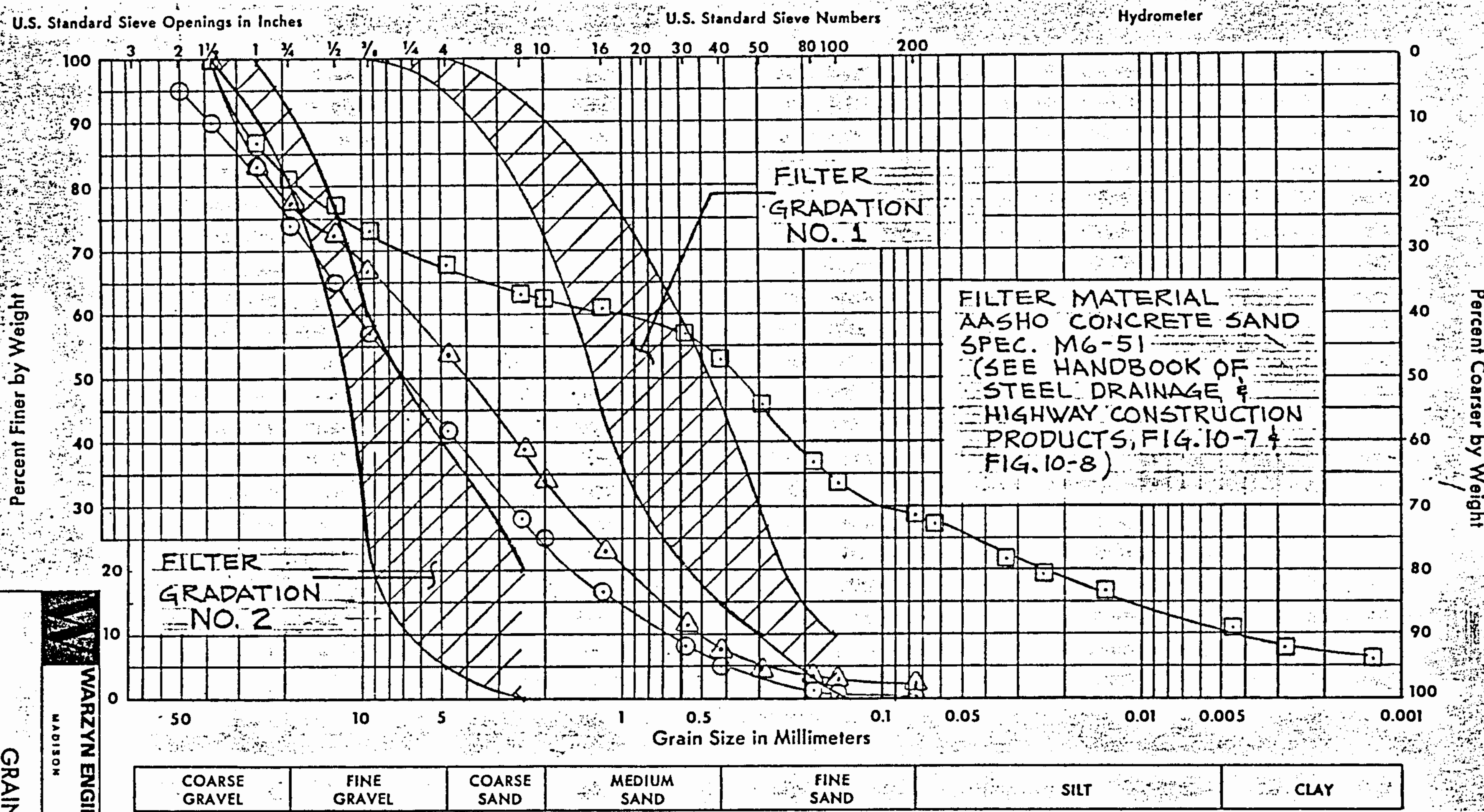

Unified Classification System (ASTM D2487)

Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
⊙	S#1	BANK				58.0	41.9	0.1		GRAVEL AND SAND, TRACE SILT AND CLAY (GW)
⊡	S#3	BANK	1.7	10.6	0.9	32.3	47.9	9.2	10.6	SAND, SOME GRAVEL, LITTLE CLAY AND SILT (SM)
△	S#1	NEW BANK				46.4	51.3	2.3		SAND AND GRAVEL, TRACE SILT AND CLAY (SP)

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DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

GRAIN SIZE ANALYSIS

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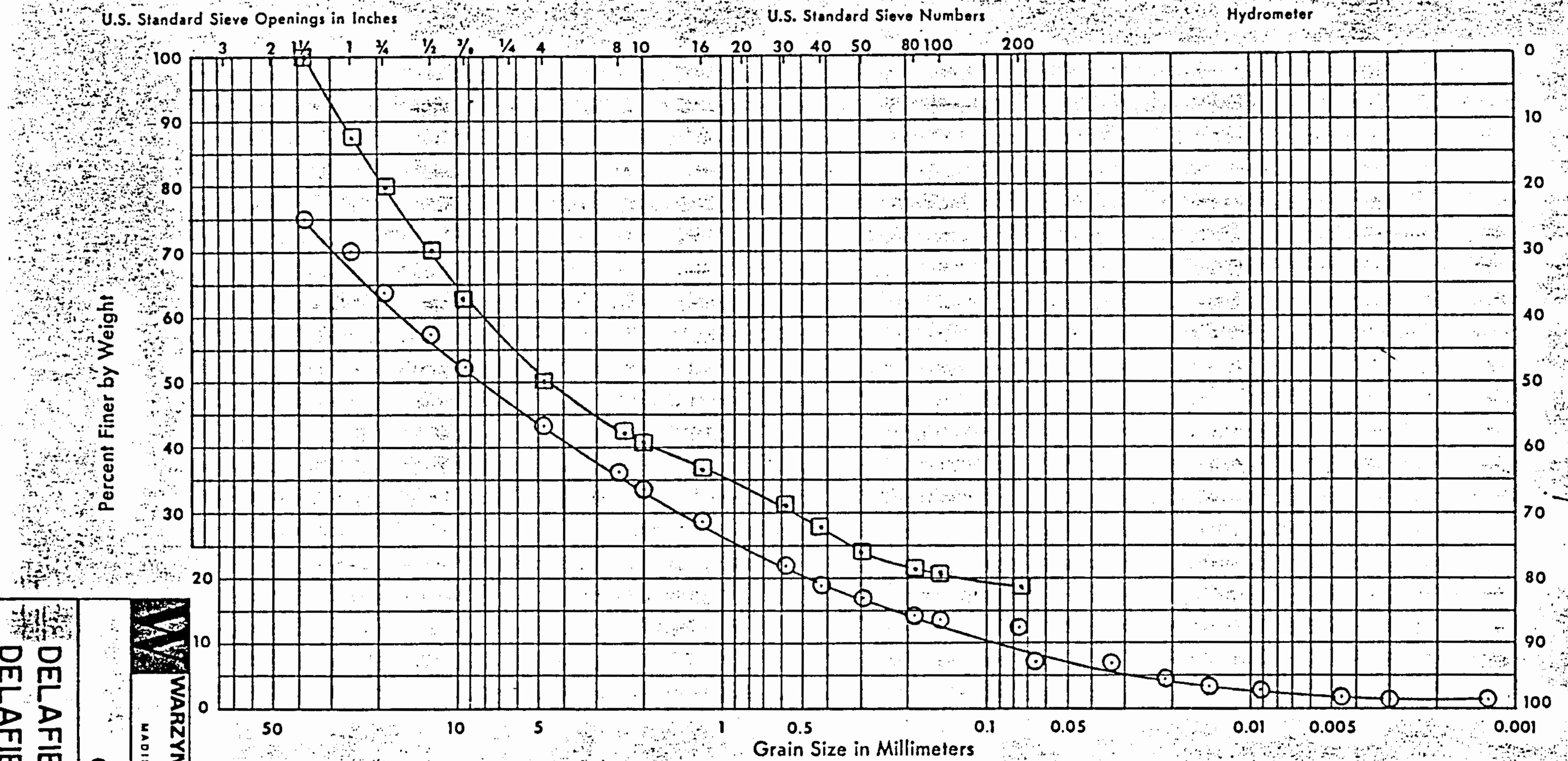

COARSE GRAVEL	FINE GRAVEL	COARSE SAND	MEDIUM SAND	FINE SAND	SILT	CLAY

Unified Classification System (ASTM D2487)

Curve	Sample	Depth	N.M.	LL	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
⊙	S#9	BANK LINER	4.9			56.7	30.8	10.4	2.1	GRAVEL, SOME SAND, LITTLE SILT, TRACE CLAY (GM)
⊡	BL#1	BANK LINER		17.2	15.8	49.8	31.9	18.3		GRAVEL, SOME SAND, SILT AND CLAY (GC)

DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

GRAIN SIZE ANALYSIS

WARZYN ENGINEERING AND SERVICE COMPANY, INC.
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DWN TCP
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C7091-A61

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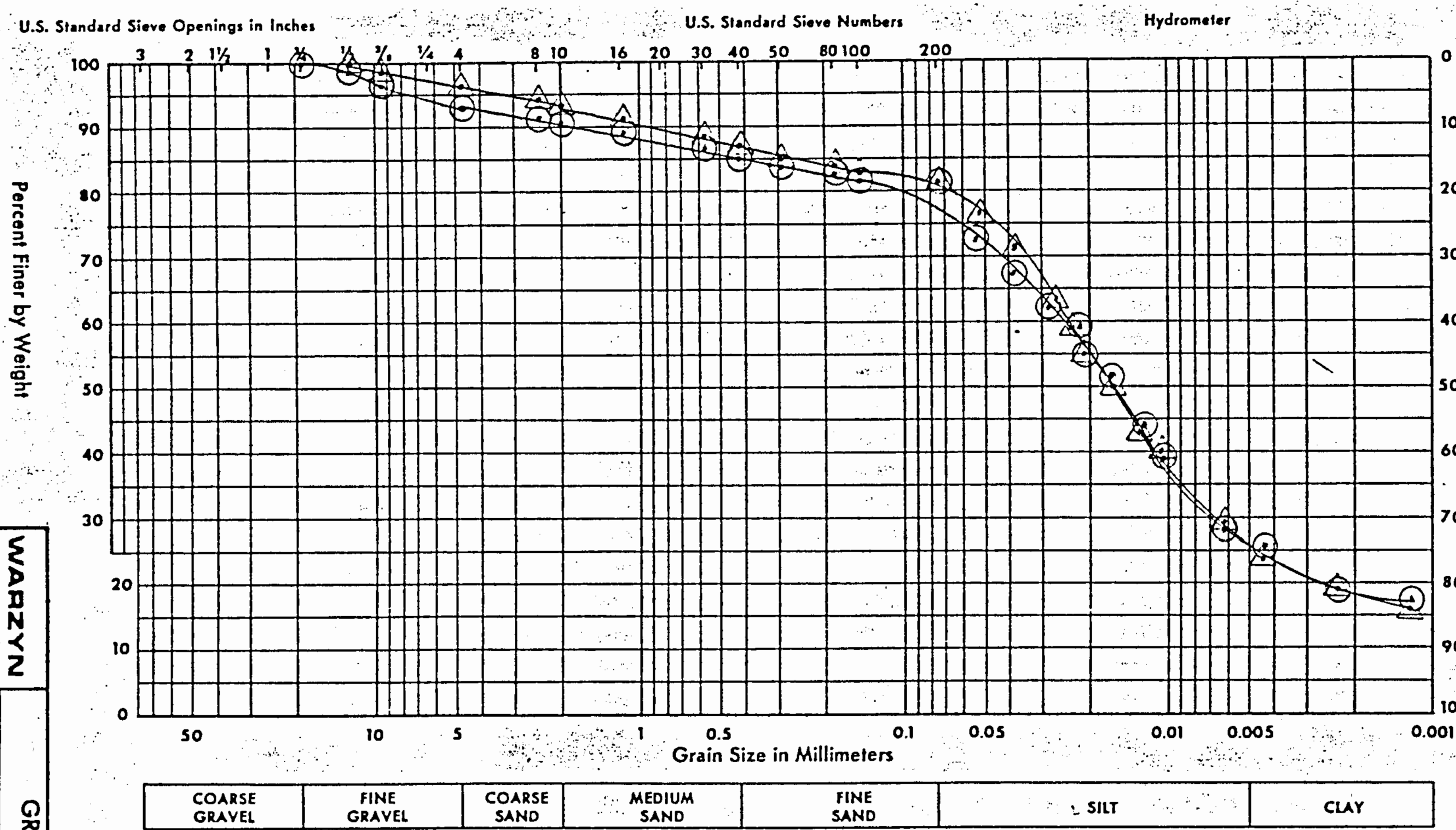

Unified Classification System (ASTM D2487)

Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
△	L1		29.53	35.28	15.08	4	14	56	26	SILT, SOME CLAY & SAND, TRACE GRAVEL (CL)
⊙	L2		26.96	36.37	15.77	7	16	51	26	SILT, SOME CLAY & SAND, LITTLE GRAVEL (CL)

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GRAIN SIZE ANALYSIS

DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

DWN CGK, TMG
CHK'D CGK
APP'D David R. Vait
DATE 11/20/78
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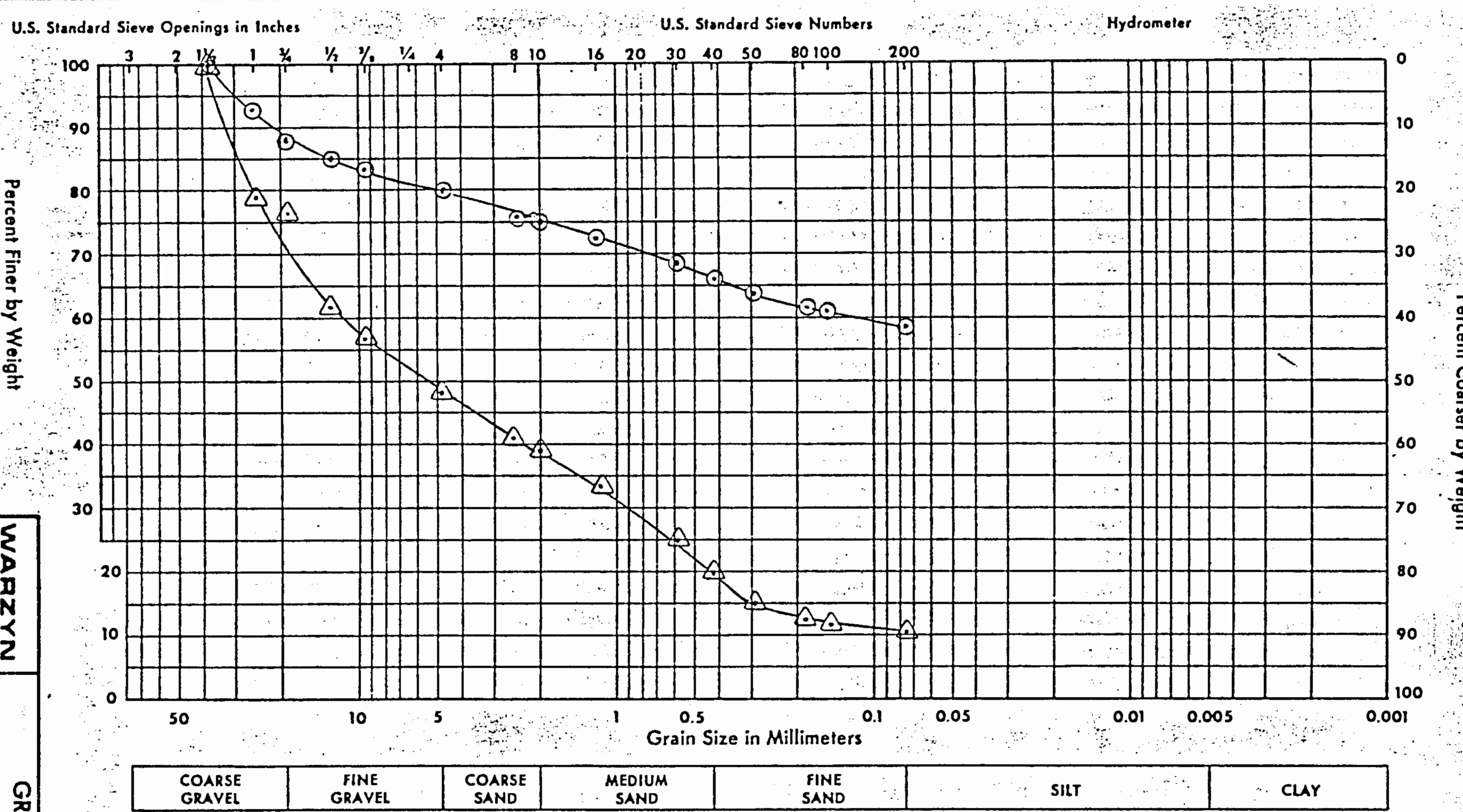

Unified Classification System (ASTM D2487)

Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
△	C1		NON-PLASTIC			52	38	10		GRAVEL AND SAND, LITTLE SILT AND CLAY (GP-GM)
⊙	C2			29.3	19.6	20	22	58		SILT AND CLAY, SOME SAND AND GRAVEL (CL)

DWN PJER
CHK'D CGK
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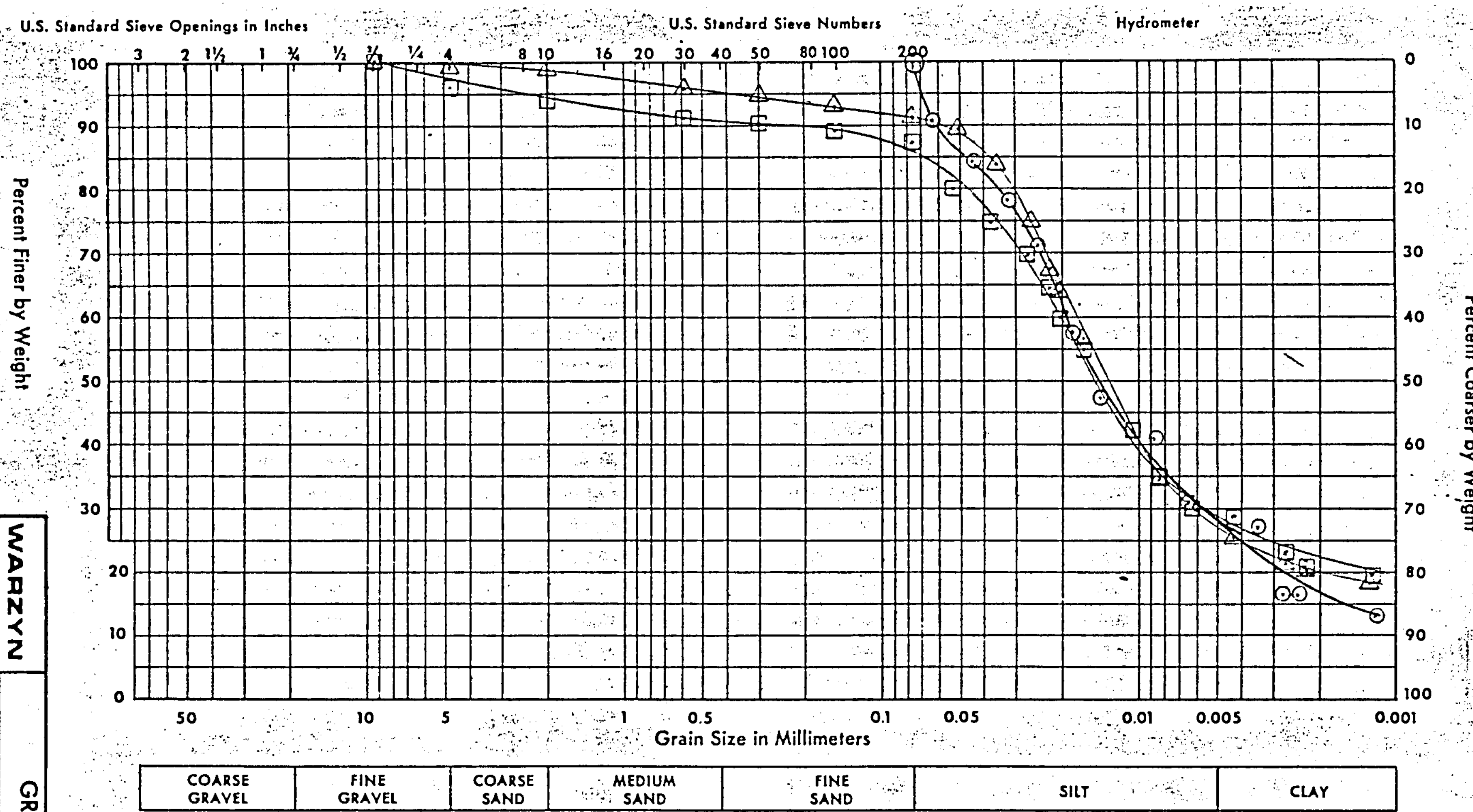
GRAIN SIZE ANALYSIS
DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN



Unified Classification System (ASTM D2487)

Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
△	FIELD DENSITY TEST "A"		23.2	30.9	11.4	I	8	65	27	SILT, SOME CLAY, LITTLE SAND, TRACE GRAVEL (CL)
⊡	FIELD DENSITY TEST "B"		23.2	35.3	14.9	4	10	58	28	SILT, SOME CLAY, LITTLE SAND, TRACE GRAVEL (CL)
⊙	FIELD DENSITY TEST "C"		24.7	34.5	14.0	0	0	72	28	SILT, SOME CLAY (CL)

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GRAIN SIZE ANALYSIS

DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

DWN JLC
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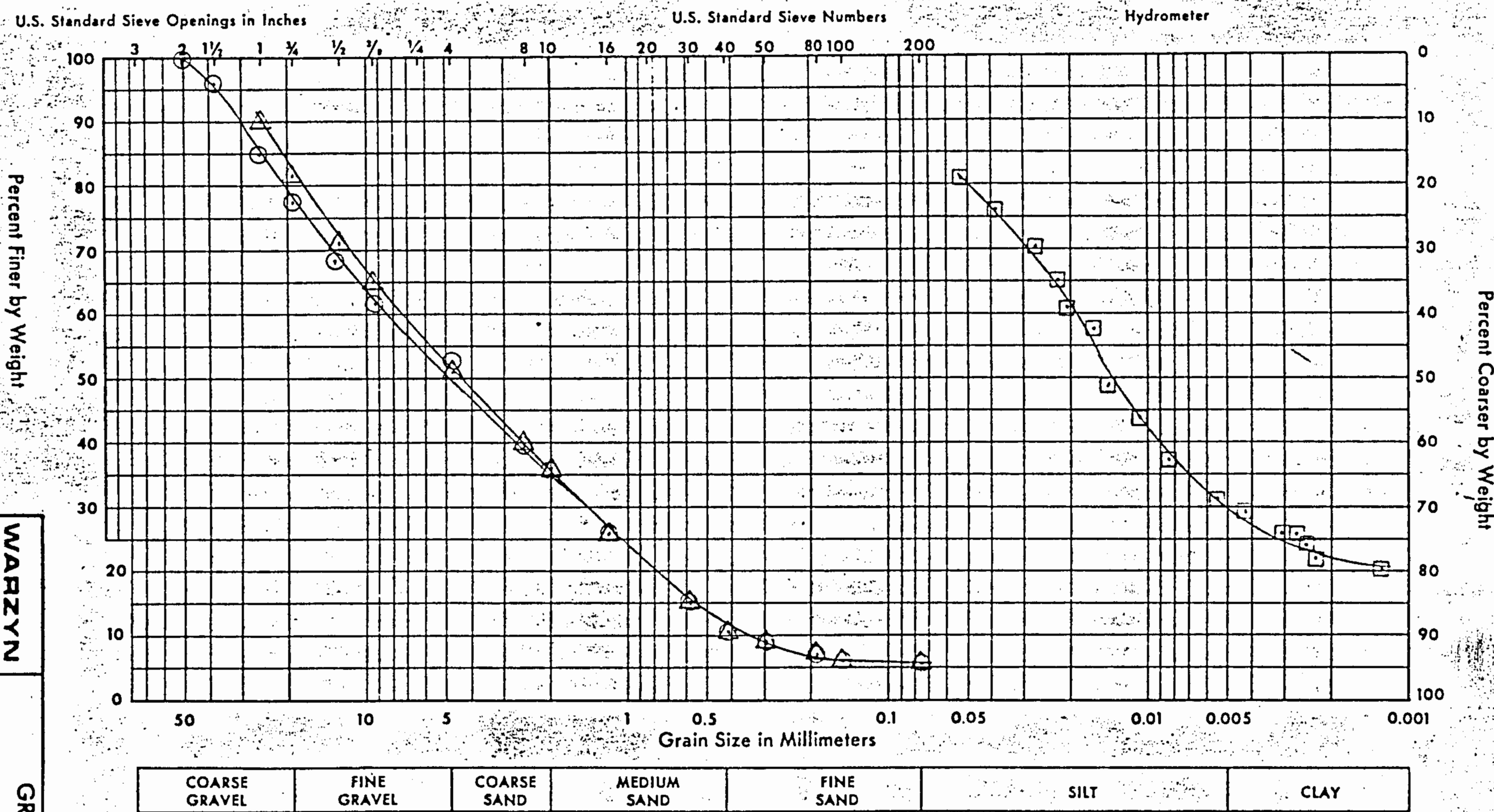
C7091-A64



Unified Classification System (ASTM D2487)

Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
△	SAND FILTER "A" MATERIAL					49	45	6		GRAVEL AND SAND TRACE SILT & CLAY (GW-GM)
⊙	SAND FILTER "B" MATERIAL					47	47	6		GRAVEL AND SAND TRACE SILT & CLAY (GP-GM)
⊡	SHELBY TUBE		22.3	38.6	18.4		19	51	30	SILT, SOME CLAY & SAND (CL)

DWN JLC

CHK'D BER.

APP'D Daniel R. Wick

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GRAIN SIZE ANALYSIS

DELAFIELD SANITARY LANDFILL

DELAFIELD WISCONSIN

DATE 7-5-78

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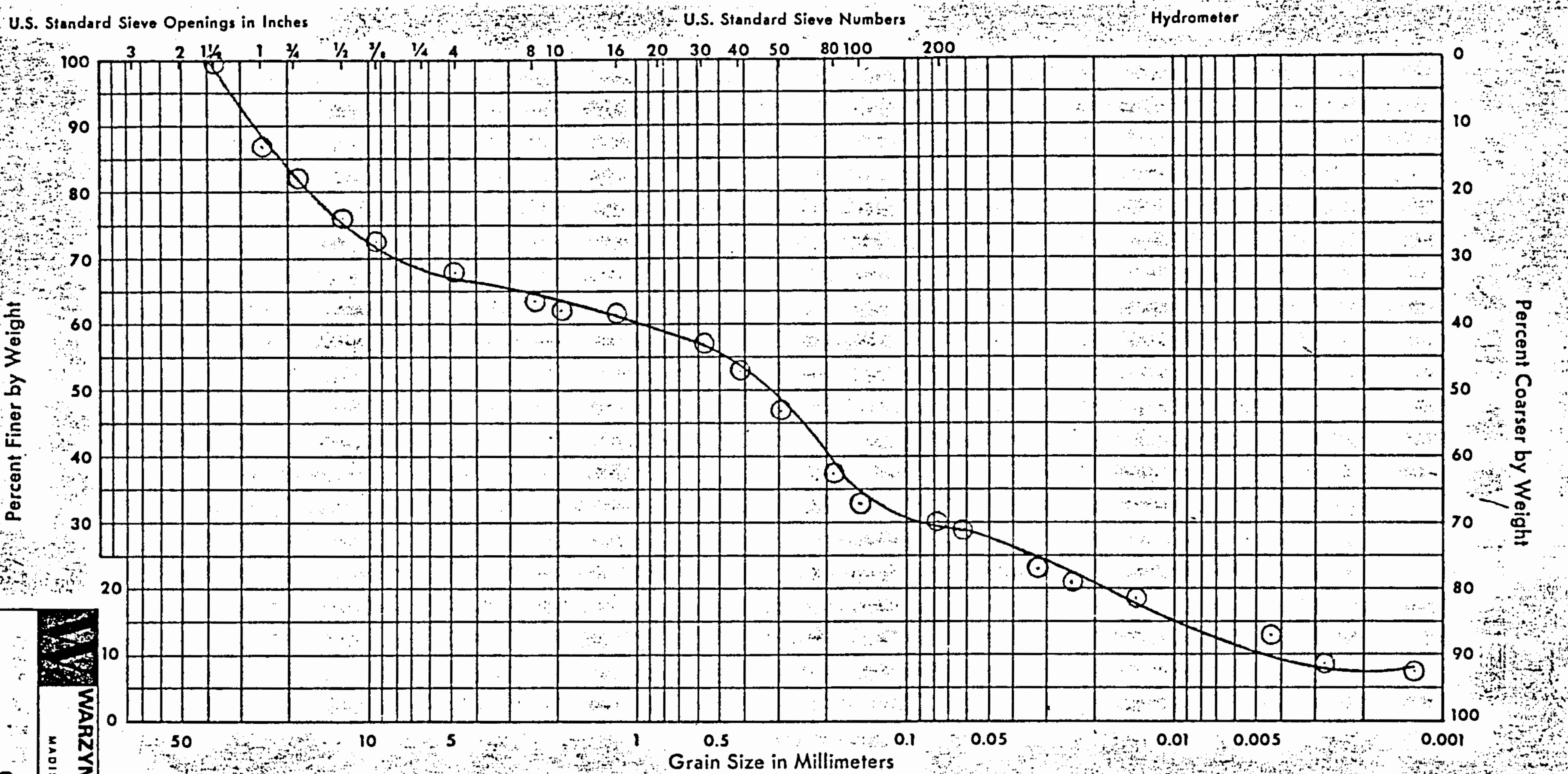

COARSE GRAVEL	FINE GRAVEL	COARSE SAND	MEDIUM SAND	FINE SAND	SILT	CLAY

Unified Classification System (ASTM D2487)

Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
⊙	B #3	BANK	1.7	10.6	0.9	32	39	18	11	SAND, SOME GRAVEL & SILT, LITTLE CLAY (SM)

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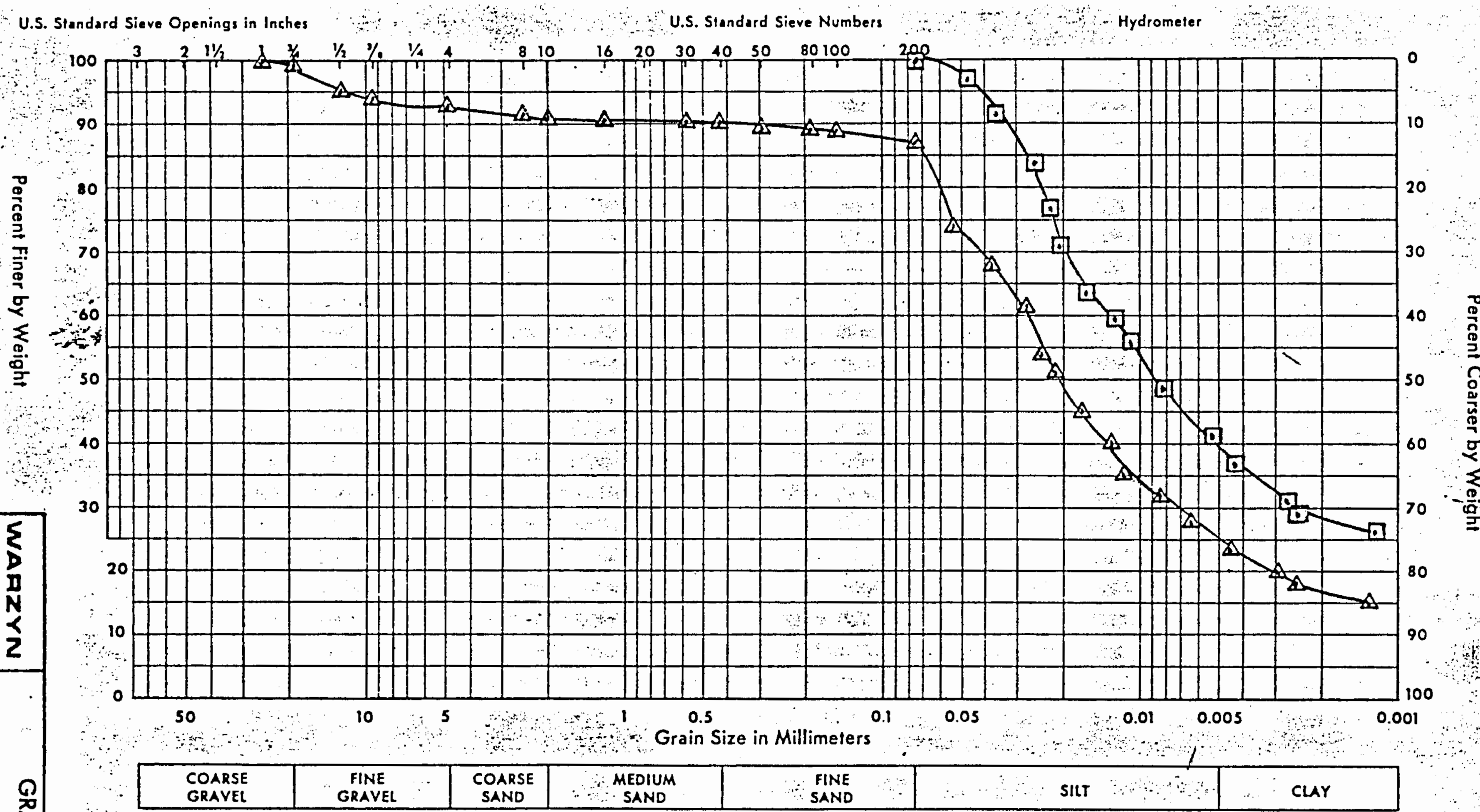
GRAIN SIZE ANALYSIS

DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

DWN GHK
CHK'D RCC
APP'D HAK
DATE 9/25/78
C7091-A62



Unified Classification System (ASTM D2487)

Curve	Sample	Depth	N.M.	L.L.	P.I.	% Grav	% Sand	% Silt	% Clay	Soil Classification
△	BAG A		16.4	40.0	19.9	7	7	61	25	SILTY CLAY, LITTLE SAND AND GRAVEL (CL)
⊡	BAG B		22.3	40.1	20.2	0	0	61	39	SILTY CLAY (CL)

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GRAIN SIZE ANALYSIS

DELAFIELD SANITARY LANDFILL
DELAFIELD WISCONSIN

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APPENDIX F

WATER BUDGET CALCULATIONS

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DESCRIPTION OF METHODS

The general water balance methods, which determine the percolation rates of precipitation through the soil and existing waste, are described in the United States Environmental Protection Agency (EPA) report entitled Use of the Water Balance Method for Predicting Leachate Generation from Solid Waste Disposal Sites[4]. The water balance accounts for precipitation, evapotransporation rates, soil moisture, vegetation type, and surface water runoff. Annual precipitation and temperature data used in the analysis were obtained from the nearest National Oceanic and Atmospheric Administration's recording station located in Waukesha, Wisconsin, approximately 10 miles east of the landfill area. The temperature data was reduced to potential evapotransporation rates using the tables derived by C.W. Thornthwaite and J.R. Mather in Instructions and Tables for Computing Potential Evapotransporation and the Water Balance[5]. Back-up data regarding water budget calculations, climatic data, and runoff curves are included in Appendix F.

The values presented in the EPA's report for surface water runoff coefficients is dependent on basically three parameters: vegetation, slope, and soil type. The EPA's values are quite limited in the various combinations of vegetation soil types and slopes which are assigned runoff coefficients. It was felt that the EPA values do not accurately reflect the present and future field conditions at the landfill with the limited vegetative, soil, and slope combinations given, particularly for slopes greater than 7%. To obtain more definitive runoff coefficients, a series of publications were consulted.

The coefficient of runoff is defined as the ratio of peak runoff to the average rainfall intensity for a given frequency and duration of storm. The American Society of Civil Engineer's Manual and Report on Engineering Practices No. 37[6], contains average runoff coefficient values determined for 5 and 10 year frequency storms (the EPA uses these runoff coefficients). The duration of the storm used in determining coefficients is 24 hours. The intensity of 5 and 10 year frequency 24 hour storms for the Delafield area was obtained from the U.S. Weather Bureau Atlas Technical Paper No. 40[7].

For the calculation of runoff coefficients, the U.S. Department of Agriculture, Soil Conservation Service's (SCS) Engineering Field Manual[8], chapter 2, entitled "Estimating Runoff", was consulted. It has compiled a series of graphs which utilize rainfall intensity, area of watershed, slope, vegetation, and soil type to determine peak runoff. The method is appropriate for small watersheds, i.e., 5 acres or larger. For practical purposes, the landfill can be considered as a small watershed.

In the SCS methods, the parameters of vegetation and soil type are incorporated into a curve number (CN). Curve numbers are determined by selecting a specific hydrologic soil group hydrologic condition, and land use (vegetation). The SCS Engineering Field Manual utilizes four different soil groups based on the soil's ability to allow surface water infiltration, see Appendix F for classifications. For the two soil types considered in this report, Group B is deemed representative of the existing sand cover (permeable) and Group D would be representative of a clay cap (impermeable). Soil conditions for the two soil types was considered to be "good", typical of a humid area such as Wisconsin. The vegetation type listed by SCS most closely resembling actual field conditions at the Delafield Landfill is "meadow". The curve numbers selected, based on the beforementioned soil group, hydrologic condition, and vegetation type, are 60 and 80 for the existing sand cover and the proposed clay cap, respectively.

The SCS Manual considers three categories of slope ranges. The categories are flat slope (0-3% slope), moderate slope (3-8% slope) and steep slope (greater than 8% slope). For many combinations of slope categories and curve numbers, the SCS has compiled a series of graphs for determining peak runoff for any size watershed between 5 and 2000 acres. The relevant graphs for the water budgets presented in this report are included in Appendix F.

Once the peak runoff was determined, it was inserted into the definition of runoff coefficient along with the storm intensity. Subsequently, the runoff coefficient was determined. This method considers a larger range of soil types, vegetative regimes, and slopes than the EPA method such that, it is felt, the above methods provide a realistic evaluation of site conditions.

Since the Delafield Landfill is not homogeneous in respect to slope, runoff coefficients were determined for each slope regime (flat and steep). The resulting annual percolation rates are weighted by the area of each slope type and combined to give the average percolation rate over the entire site. For example, if one quarter of the site has steep slopes with an associated percolation rate of 4" per year, and the remaining three quarters of the site has flat slopes with a percolation of 6" annually, the percolation rates are weighted by factors of .25 and .75, respectively, yielding a percolation rate for the entire site of 5.5" on an annual basis. This procedure has been utilized in the two groundwater budgets presented in this report.

HYDROLOGIC SOIL GROUPS

Over 8,000 soils have been classified into four hydrologic soil groups as shown in Exhibit 2-1. The hydrologic soil groups, according to their infiltration and transmission rates, are:

A. (Low runoff potential). Soils having high infiltration rates even when thoroughly wetted. These consist chiefly of deep, well to excessively drained sands or gravels. These soils have a high rate of water transmission in that water readily passes through them.

B. Soils having moderate infiltration rates when thoroughly wetted. These consist chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

C. Soils having slow infiltration rates when thoroughly wetted. These consist chiefly of soils with a layer that impedes downward movement of water or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.

D. (High runoff potential). Soils having very slow infiltration rates when thoroughly wetted. These consist chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

Table 9.1.--Runoff curve numbers for hydrologic soil-cover complexes

(Antecedent moisture condition II, and $I_a = 0.2\ S$)

Land use	Cover: Treatment or practice	Cover: Hydrologic condition	Hydrologic soil group A	B	C	D
Fallow	Straight row	----	77	86	91	94
Row crops	"	Poor	72	81	88	91
	"	Good	67	78	85	89
	Contoured	Poor	70	79	84	88
	"	Good	65	75	82	86
	" and terraced	Poor	66	74	80	82
	" " "	Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
	" and terraced	Poor	61	72	79	82
		Good	59	70	78	81
Close-seeded legumes 1/ or rotation meadow	Straight row	Poor	66	77	85	89
	" "	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	"	Good	55	69	78	83
	" and terraced	Poor	63	73	80	83
	" and terraced	Good	51	67	76	80
Pasture or range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	"	Fair	25	59	75	83
	"	Good	6	35	70	79
Meadow		Good	30	58	71	78
Woods		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads		----	59	74	82	86
Roads (dirt) 2/		----	72	82	87	89
(hard surface) 2/		---	74	84	90	92

1/ Close-drilled or broadcast.
2/ Including right-of-way.

EXISTING COVER CONDITIONS - FLAT SLOPES

	JAN.	FEB.	MAR.	APR.	MAY	JUNE	JULY	AUG.	SEPT.	OCT.	NOV.	DEC.	
PE	0	0	0	1.34	3.02	4.61	5.42	5.04	3.12	1.71	.24	0	
P	1.57	1.04	2.23	2.90	3.37	3.75	3.66	2.99	3.20	2.13	2.16	1.70	30.7
$C_{R/O}$	.05	.05	.05	.05	.05	.05	.05	.05	.05	.05	.05	.05	
RO	.08	.05	.11	.15	.17	.19	.18	.15	.16	.11	.11	.09	1.5
I	1.49	.99	2.12	2.75	3.20	3.56	3.48	2.84	3.04	2.02	2.05	1.61	29.2
I-PE	1.49	.99	2.12	1.41	.18	-1.05	-1.94	-2.20	-.08	.31	1.81	1.61	
Eneg. I-PE						-1.05	-2.99	-5.19	-5.27				
ST	3.0	3.0	3.0	3.0	3.0	2.08	1.06	.50	.48	.79	2.60	3.0	
AST	0	0	0	0	0	-.92	-1.02	-.56	-.02	.31	1.81	.40	
AET	0	0	0	1.34	3.02	4.48	4.50	3.40	3.06	1.71	.24	0	21.8
PERC.	1.49	.99	2.12	1.41	.18	0	0	0	0	0	0	1.21	7.4

EXISTING COVER CONDITIONS - ~~FLAT~~ Steep SLOPES

	JAN.	FEB.	MAR.	APR.	MAY	JUNE	JULY	AUG.	SEPT.	OCT.	NOV.	DEC.	
PE	0	0	0	1.34	3.02	4.61	5.42	5.04	3.12	1.71	.24	0	30.7
P	1.57	1.04	2.23	2.90	3.37	3.75	3.66	2.99	3.20	2.13	2.16	1.70	
$C_{R/O}$	.11	.11	.11	.11	.11	.11	.11	.11	.11	.11	.11	.11	
RO	.17	.11	.25	.32	.37	.41	.40	.33	.35	.23	.24	.19	3.3
I	1.40	.93	1.98	2.58	3.00	3.34	3.26	2.66	2.85	1.90	1.92	1.51	27.4
I-PE	1.40	.93	1.98	1.24	-.02	-1.27	-2.16	-2.38	-.27	.19	1.68	1.51	
Eneg. I-PE					-.02	-1.29	-3.45	-5.83	-6.10				
ST	3.0	3.0	3.0	3.0	2.98	1.92	.92	.41	.36	.55	2.23	3.0	
AST	0	0	0	0	-.02	-1.06	-1.00	-.51	-.05	.19	1.68	.77	
AET	0	0	0	1.34	3.02	4.40	4.26	3.17	2.90	1.71	.24	0	21.1
PERC.	1.40	.93	1.98	1.24	0	0	0	0	0	0	0	.74	6.3

.18 (7.4) + .82(6.3) = 6.5" annual percolation under present cover conditions.

PROPOSED CLAY CAP - FLAT SLOPES

	JAN.	FEB.	MAR.	APR.	MAY	JUNE	JULY	AUG.	SEPT.	OCT.	NOV.	DEC.	
PE	0	0	0	1.34	3.02	4.61	5.42	5.04	3.12	1.71	.24	0	
P	1.57	1.04	2.23	2.90	3.37	3.75	3.66	2.99	3.20	2.13	2.16	1.70	30.7
$C^{R/O}$	.27	.27	.27	.27	.27	.27	.27	.27	.27	.27	.27	.27	
RO	.42	.28	.60	.78	.91	1.01	.99	.81	.86	.58	.58	.46	8.3
I	1.15	.76	1.63	2.12	2.46	2.74	2.67	2.18	2.34	1.55	1.58	1.24	22.4
I-PE	1.15	.76	1.63	.78	-.56	-1.87	-2.75	-2.86	-.78	-.16	1.34	1.24	
Eneg. I-PE				(-.27)	-.83	-2.70	-5.45	-8.31	-9.09	-9.25			
ST	3.0	3.0	3.0	2.73	2.26	1.18	.45	.17	.12	.11	1.45	2.69	
AST	.31	0	0	-.27	-.47	-1.08	-.73	-.28	-.05	-.01	1.34	1.24	
AET	0	0	0	1.34	2.93	3.82	3.40	2.46	2.39	1.56	.24	0	18.1
PERC.	.84	.76	1.63	1.05	0	0	0	0	0	0	0	0	4.3

PROPOSED CLAY CAP - STEEP SLOPES

	JAN.	FEB.	MAR.	APR.	MAY	JUNE	JULY	AUG.	SEPT.	OCT.	NOV.	DEC.	
PE	0	0	0	1.34	3.02	4.61	5.42	5.04	3.12	1.71	.24	0	
P	1.57	1.04	2.23	2.90	3.37	3.75	3.66	2.99	3.20	2.13	2.16	1.70	30.7
$C^{R/O}$	.65	.65	.65	.65	.65	.65	.65	.65	.65	.65	.65	.65	
RO	1.02	.68	1.45	1.88	2.19	2.44	2.38	1.94	2.08	1.38	1.40	1.11	19.9
I	.55	.36	.78	1.02	1.18	1.31	1.28	1.05	1.12	.75	.76	.59	10.8
I-PE	.55	.36	.78	-.32	-1.84	-3.30	-4.14	-3.99	-2.00	-.96	.52	.59	
Eneg. I-PE			(-1.13)	-1.45	-3.29	-6.59	-10.73	-14.72	-16.72	-17.68			
ST	1.68	2.04	2.03 2.82	1.83	.96	.31	.10	.05	.03	.02	.54	1.13	
AST	.55	.36	-.01 .78	-.20 .99	-.87	-.65	-.21	-.05	-.02	-.01	.52	.59	
AET	0	0	0	1.22	2.05	1.96	1.49	1.10	1.14	.76	.24	0	10.0
PERC.	0	0	.79	0	0	0	0	0	0	0	0	0	.8

.18(4.3) + .82(.8) = 1.4" predicted annual percolation for proposed clay cap.

.09

CONTERMINOUS UNITED STATES

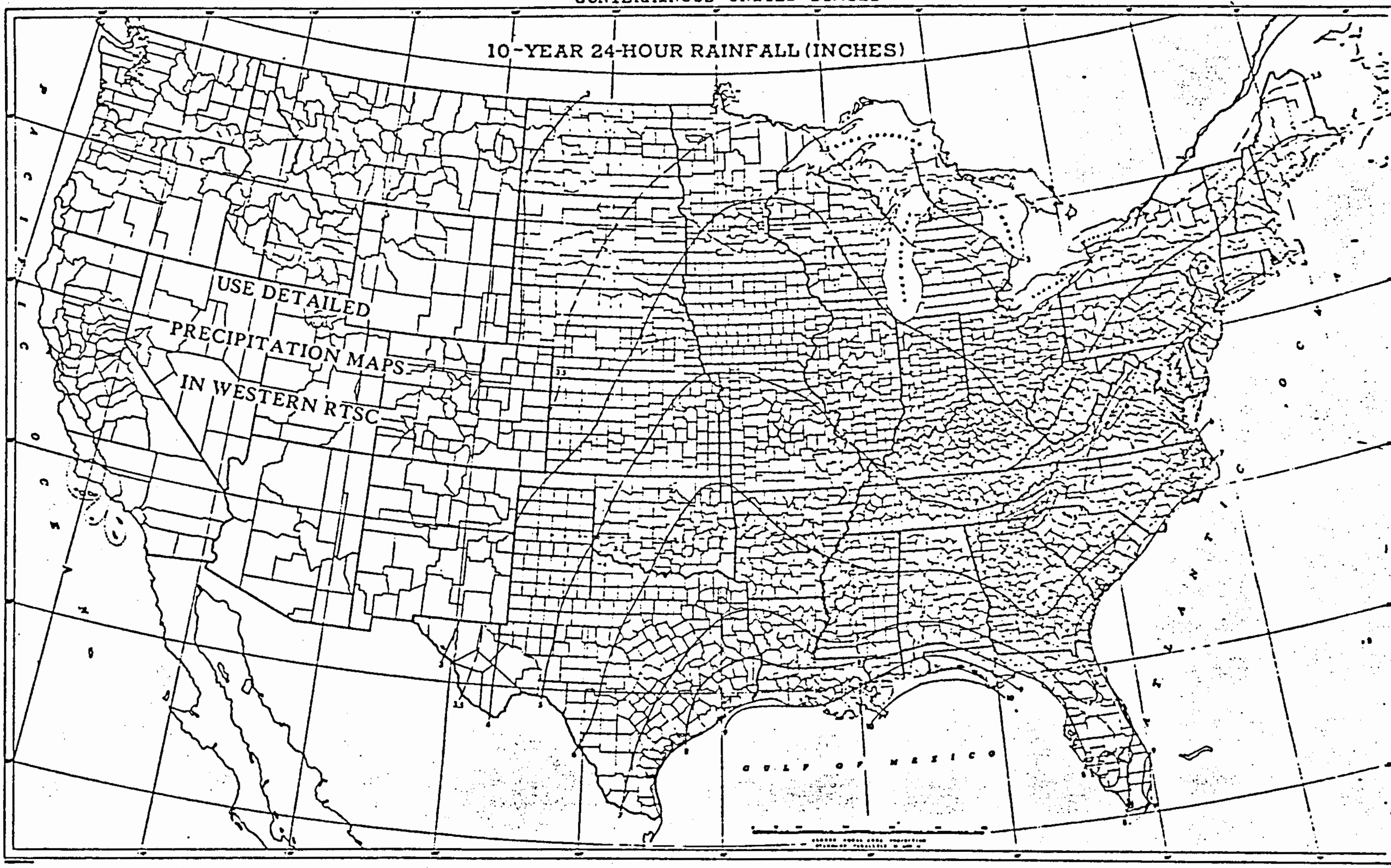

Exhibit 2-3
Sheet 3 of 5

Prepared by U. S. Weather Bureau

CONTERMINOUS UNITED STATES

Exhibit 2-3
Sheet 2 of 5

Prepared by U. S. Weather Bureau

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - STEEP

CURVE NUMBER - 60

24 HOUR RAINFALL FROM US WB TP-40

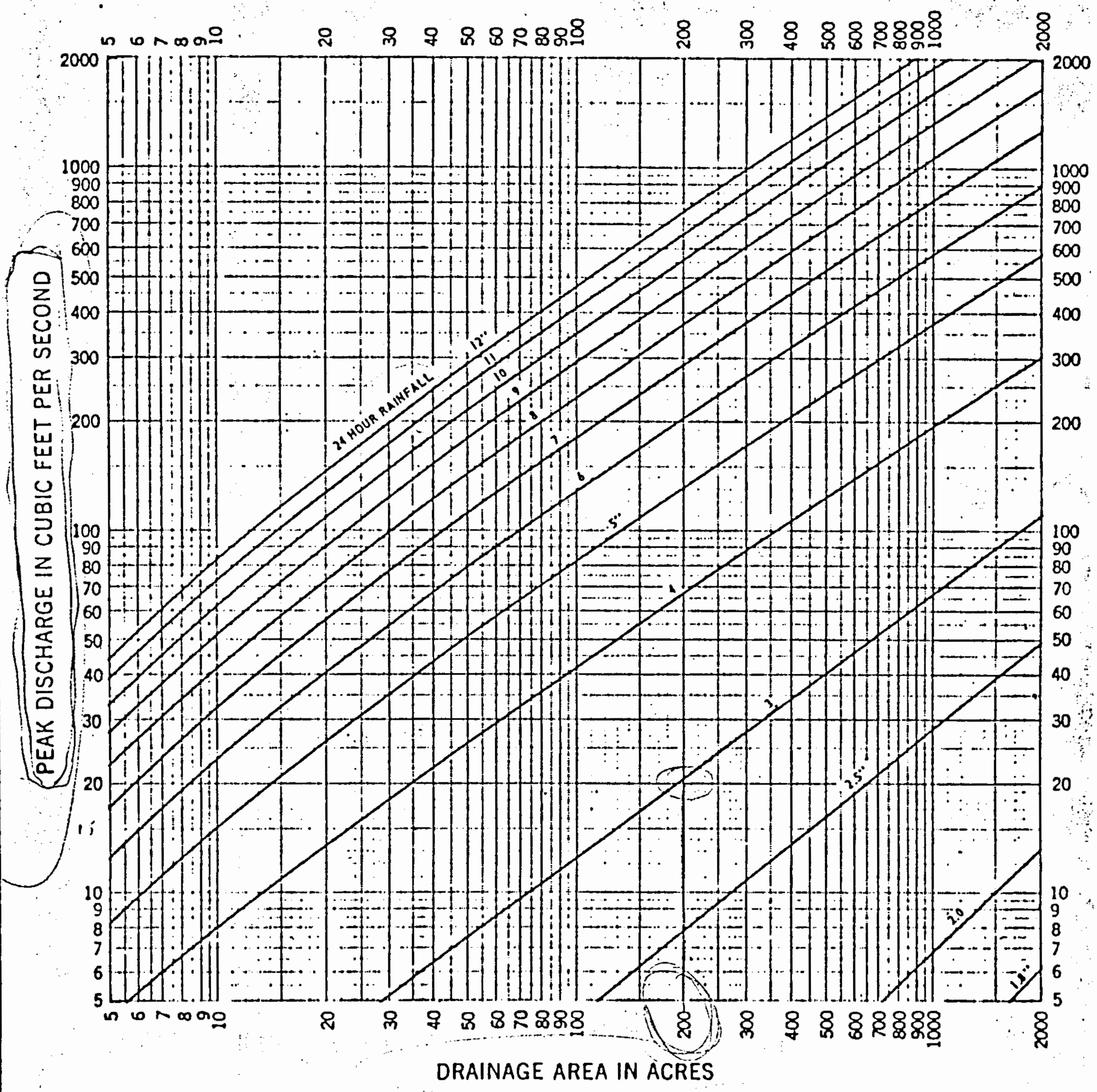

Exhibit 2-10

EFM Notice-4, 5/71

REFERENCE		STANDARD DWG. NO.
"Chapter 2, Engineering Field Manual for Conservation Practices"	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - HYDROLOGY-BRANCH	ES-1027 SHEET 15 OF 21 DATE 2-15-71

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - STEEP
CURVE NUMBER - 80

24 HOUR RAINFALL FROM US WB TP-40

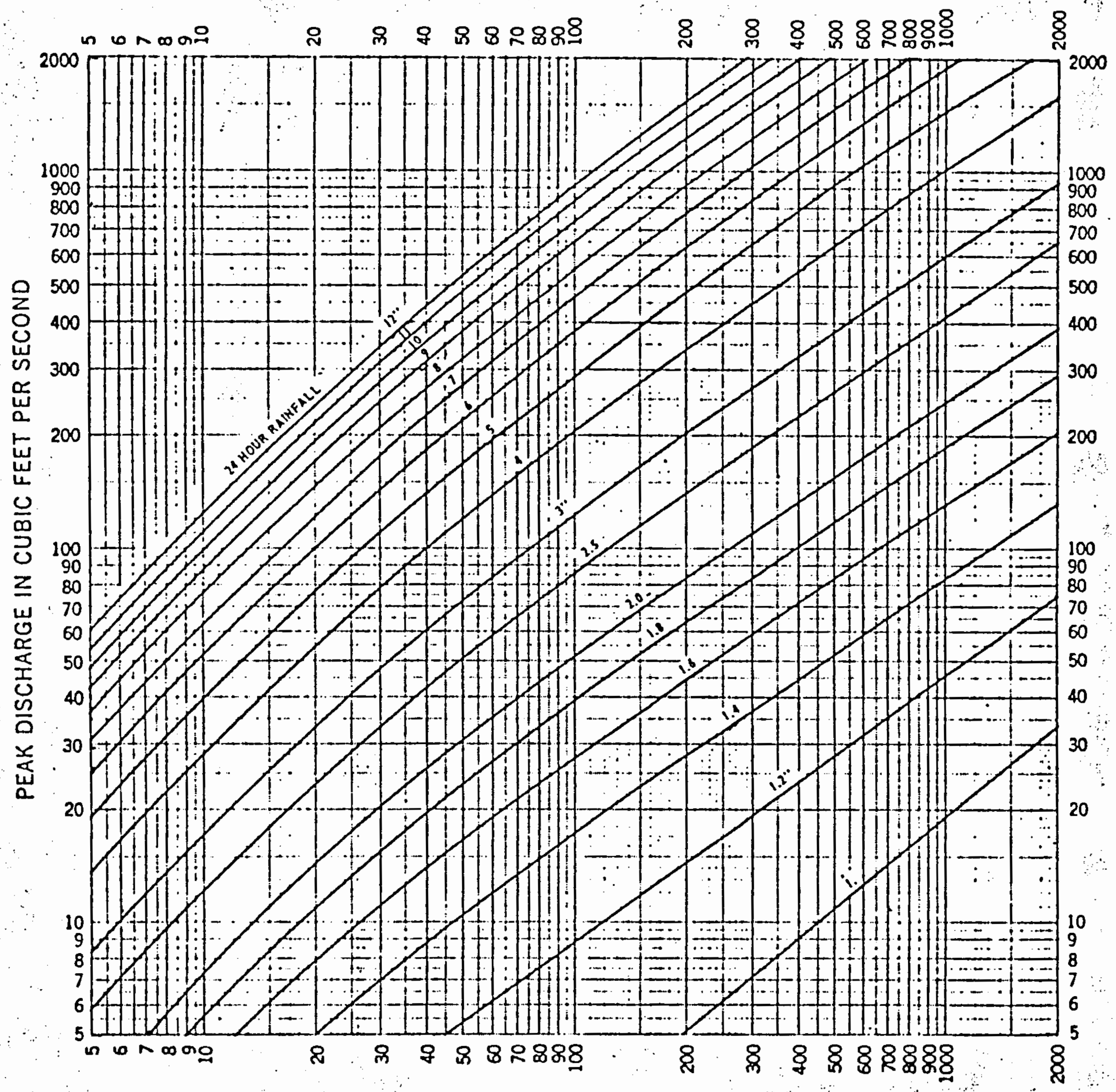

Exhibit 2-10

EFM Notice-4, 5/71

REFERENCE		STANDARD DWG. NO.
"Chapter 2, Engineering Field Manual for Conservation Practices"	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - HYDROLOGY BRANCH	ES-1027 SHEET 19 OF 21 DATE 2-15-71

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - FLAT
CURVE NUMBER - 60

24 HOUR RAINFALL FROM US WB TP-40

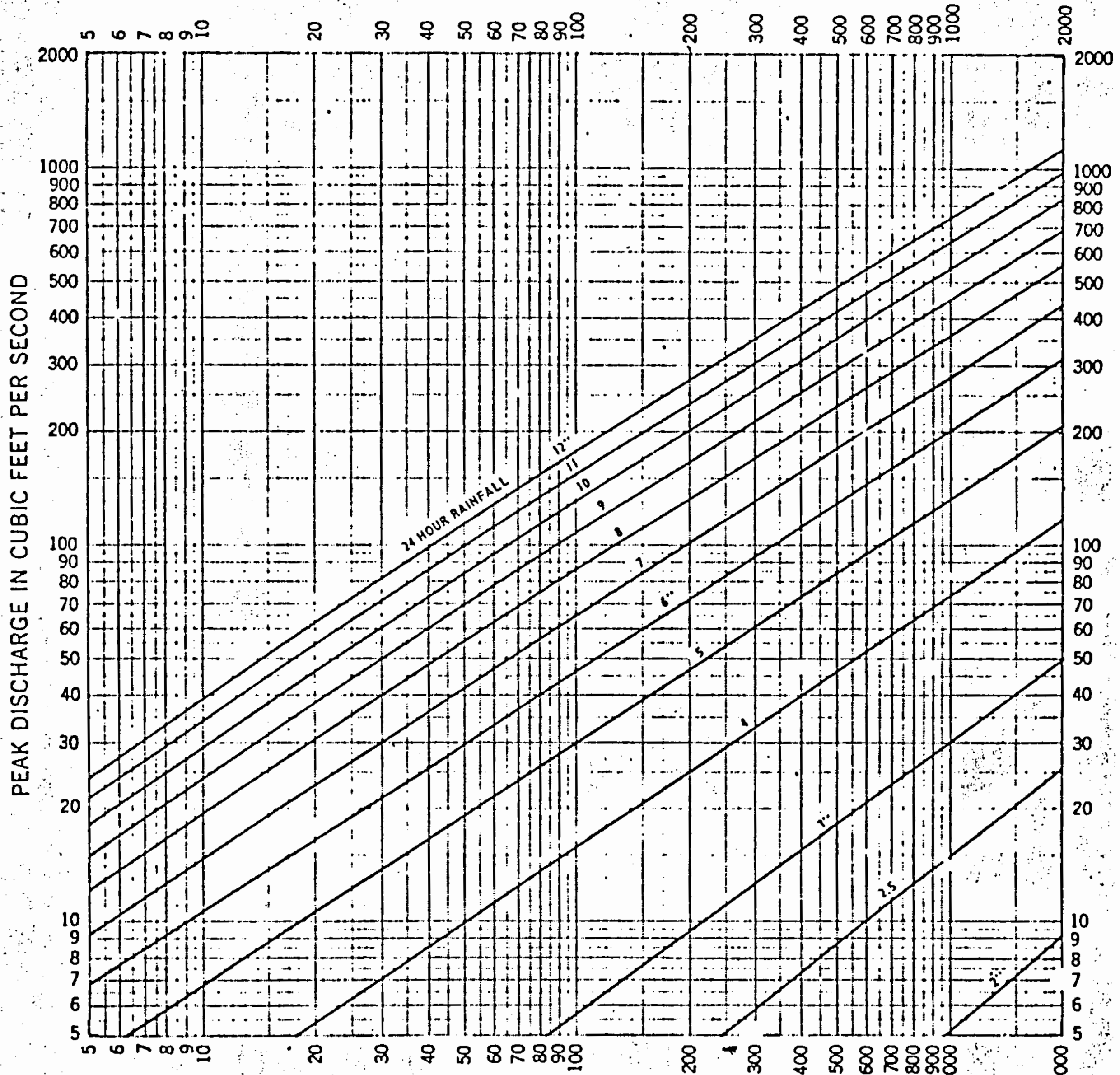

Exhibit 2-10

EFM Notice-4, 5/71

REFERENCE		STANDARD DWG. NO.
"Chapter 2, Engineering Field Manual for Conservation Practices"	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - HYDROLOGY BRANCH	ES-1027 SHEET 1 OF 21 DATE 2-15-71

PEAK RATES OF DISCHARGE FOR SMALL WATERSHEDS
TYPE II STORM DISTRIBUTION

SLOPES - FLAT
CURVE NUMBER - 80

24 HOUR RAINFALL FROM US WB TP-40

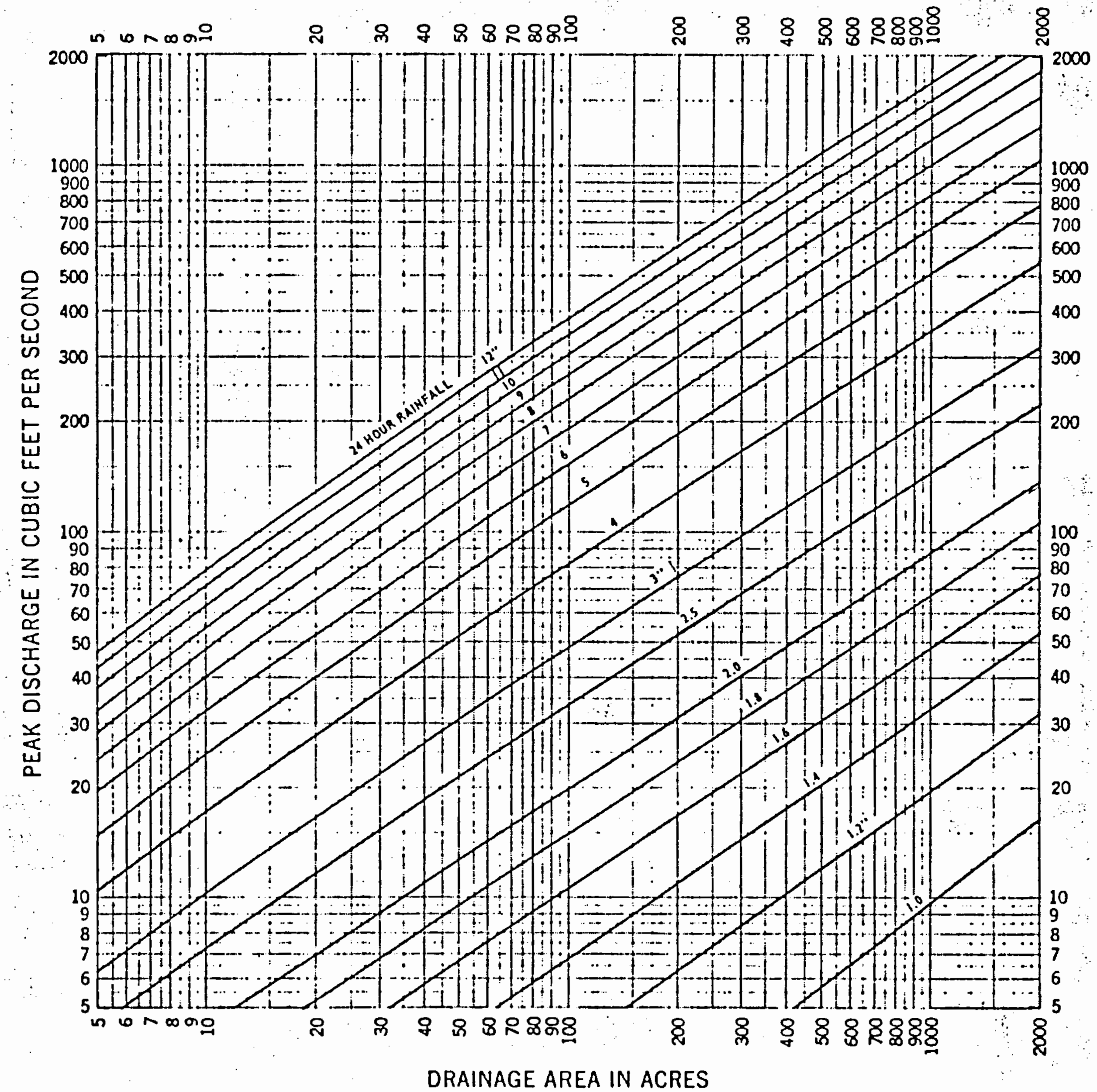

Exhibit 2-10

EFM Notice-4, 5/71

REFERENCE		STANDARD DWG. NO.
"Chapter 2, Engineering Field Manual for Conservation Practices"	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - HYDROLOGY BRANCH	ES-1027 SHEET 5 OF 21 DATE 2-15-71