Management of Confined Disposal Facilities

APPENDIX D1

CDF FIELD INVESTIGATION TO EVALUATE CONSOLIDATION POTENTIAL

U.S. Army Engineer District, Buffalo • 1776 Niagara Street • Buffalo New York 14207

US Army Corps
of Engineers
Waterways Experiment
Station

October 1995

Laboratory and Field Investigation to Evaluate Toledo Confined Disposal Facility for Consolidation Potential

by Paul A. Gilbert

Prepared for US Army Engineer Buffalo District

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Preface

The study reported herein was conducted by the Geotechnical Laboratory (GL), U.S. Army Engineer Waterways Experiment Station (WES), Vicksburg, MS, for the U.S. Army Engineer District, Buffalo (CENCB). The study was part of an investigation to determine recoverable storage capacity from confined disposal facilities at Toledo, OH. Funds for the investigation were authorized by MIPR #NCB-MR-94-37E7 dated 18 May 1994.

The project was conducted under the general supervision of Dr. W. F. Marcuson III, Director, GL, WES; Dr. Don Banks, Chief, Soil & Rock Mechanics Division, and Mr. David Bennett, Chief, Soils Research Center (SRC). The WES principal investigator was Mr. Paul A. Gilbert, SRC, who also prepared this report. Point of contact at CENCB was Mr. Wiener Cadet, Plan Formulation and Technical Management Section. Special thanks are extended to Mr. Jonathan E. Kolber, Coastal Geotechnical Section, and Mr. John A. McCarthy, Toledo Area Office, for assistance with field investigations and soil sampling.

During the conduct of this investigation Dr. Robert W. Whalin was the Director of WES. COL Bruce K. Howard, EN, was the Commander.

1 Introduction

General

The Port of Toledo is located on Lake Erie in northern Ohio at the mouth of the Maumee River (see Figure 1). It is a major transportation hub in the industrial and agricultural center of North America. An average of 15 million tons of cargo is handled each year at the Port of Toledo, including coal, ore, and agricultural products, as well as general cargo (U.S. Army Engineer District, Buffalo, 1993). Sediment carried into the lake by the Maumee River causes silting and shallowing of Toledo Harbor and causes problems with navigation and ship movement. The Army Corps of Engineers has maintained the Port since the late 19th century through a yearly dredging program, however, environmental concerns have been expressed since about 1985 regarding the practice of open lake disposal of dredged material. There is an additional concern regarding the potential loss of shallow water habitat as a result of the expansion of Confined Disposal Facilities (CDFs) for storage of dredged material in the nearshore area.

Activities associated with the Port of Toledo generate over 500 million dollars each year with over 5,000 jobs depending on Port operations. For this reason, action to maintain viable navigability of the Port is vital. Five activities will be undertaken to address environmental concerns as well as ensure viability of the Port;

a. Continued dredging of the Maumee River for a distance of 7 miles inland from the mouth and 19 miles into Lake Erie along the Lake Approach Channel. Dredged material from 7 miles of the River and the first 5 miles of the Lake Approach Channel would be placed in the CDF (see Map 1 for details).

b. Conservation tillage, where a certain amount of crop residue is left on farm fields to minimize soil washed into the Maumee River.

c. Recycling dredged material, after it has been processed and treated to produce good quality top soil.

d. CDF management, where the life of the existing CDFs Cell #1 and Cell #2 would be extended using dewatering and consolidation measures as well as raising the CDF to recover and add storage capacity in existing CDFs.

e. Continuous evaluation of the above measures (and other measures including a new CDF) taken to assess their effectiveness and feasibility.

Objectives

The objectives of this investigation are to assess the potential for consolidation and storage recovery at Toledo Cell #1 within the existing CDF in Toledo Harbor and to assess the potential for additional storage capacity by raising the CDF to help achieve the broader goals of the long-term sediment management study (U.S. Army Engineer District, Buffalo, 1993). These objectives are accomplished by making measurements in, and acquiring suitable soil specimens from the existing CDF, as well as performing laboratory tests and analyses to allow estimation of the amounts of consolidation and soil strength that are expected.

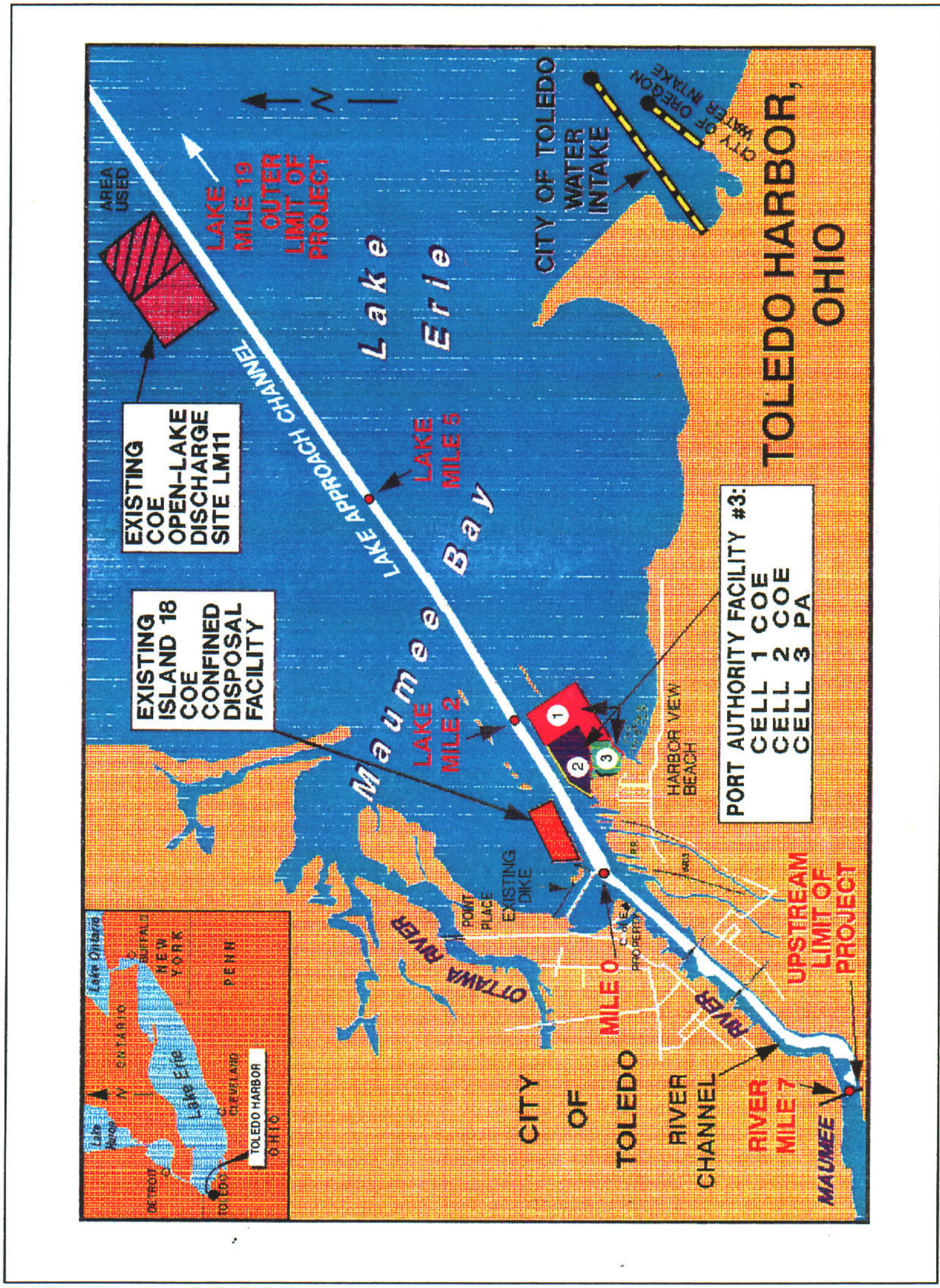

Figure 1. Toledo Harbor on Lake Erie

2 Site Description and Investigation Procedure

Site Description

A plan of the existing Toledo CDF Comprising Cell #1 and Cell #2 is shown in Figure 2. Since Cell #2 is new, it can be consolidated as it is filled using sound management techniques. Cell #1 on Figure 2 is the tract that is analyzed for storage recovery. It is about 220 acres in area and contains dredged material that is about 18 ft in average depth. Three locations designated Site 1, Site 2, and Site 3 are shown on Figure 2. Soil samples for characterization tests and soil properties tests were taken from all three sites. Piezometers were installed at Site 1 and Site 2 so that level of excess pore water pressure could be determined to assess the potential for consolidation.

Background and Procedure

Erosion as the result of conventional agricultural practices causes sediment to be carried into streams and rivers. The sediment remains suspended in the waters of stream and rivers near the source and point of entry because of high velocity and the associated turbulence and circulation. However, as rivers widen or empty into lakes or other larger bodies of water, velocity decreases and sediment precipitates and accumulates on the bottom of the waterway. Sediment is continuously carried into navigation channels, lakes, and harbors in this manner to the extent that some harbors and shipping lanes would be silted up and rendered useless unless this material is periodically removed.

One technique to maintain navigability is to remove accumulated sediment (called maintenance dredge material) from waterway and harbor bottoms using a suction cutter dredge. This dredging process is effective and efficient (Houston, 1986), however, the material removed has low strength, high water content, and may have elevated levels of toxic contaminants present. Moreover, once the material is removed from its in situ location, a suitable and environmentally sound site must be available for storage of the maintenance dredged material.

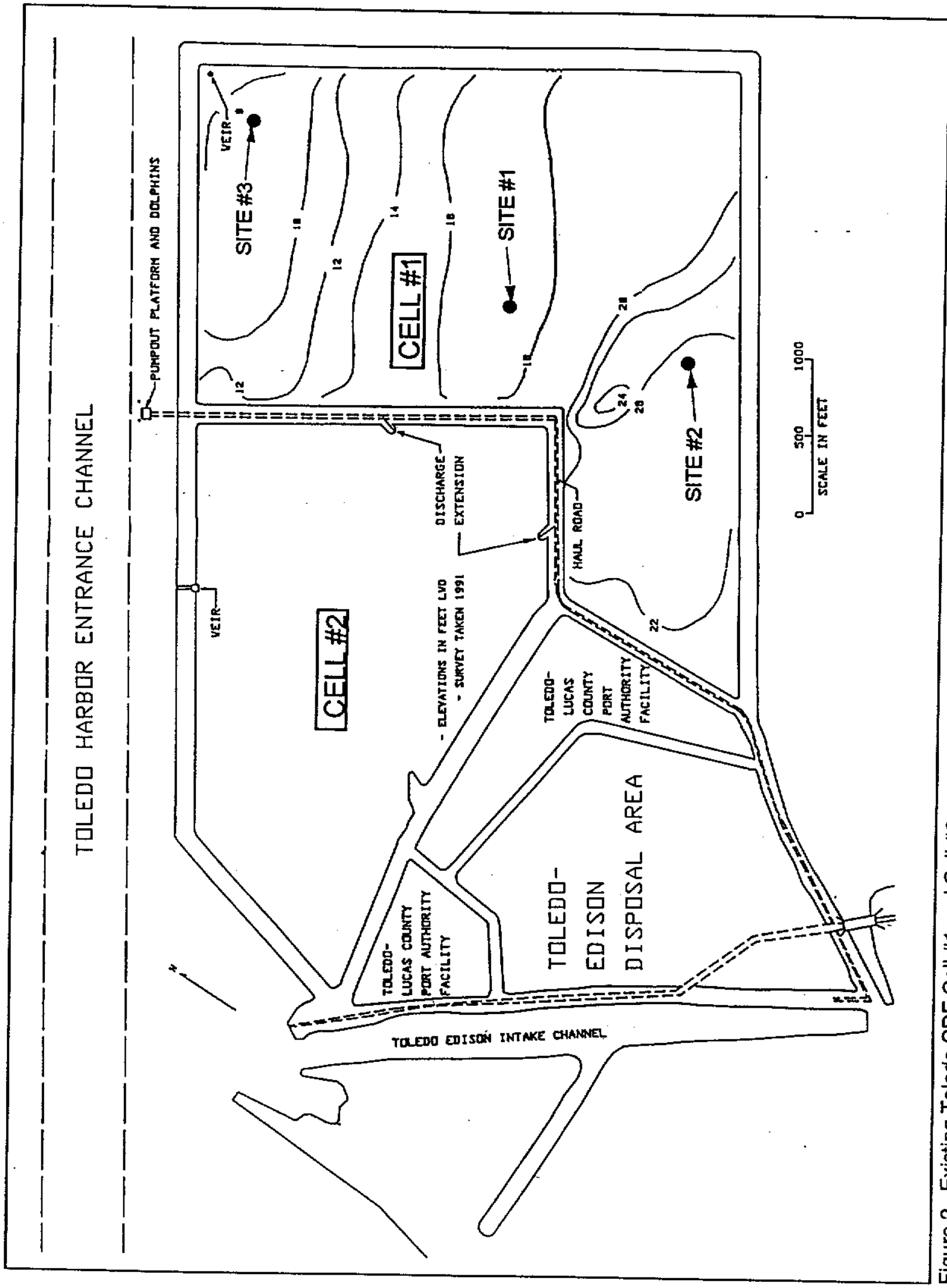

Figure 2. Existing Toledo CDF Cell #1 and Cell #2

Using a suction cutter dredge, sediment along with water for transport is removed at the head of a pipeline. The soil-water mixture may then be stored in the hopper of a bottom dump scow for transport to a deep-water location for disposal, or drawn into a powerful pump and sent through a pipeline that could range in length from a few hundred feet to several miles and placed in the CDF that has been specially designed and constructed to receive maintenance dredged material. If open water disposal is prohibited for environmental or other reasons, existence of a CDF is a crucial element in the system for sound management of waterways and harbors with respect to maintenance dredged material.

Deep water disposal is a tenable technique for disposal of maintenance dredged material, however, it has come under increasing pressure in recent years because of environmental concerns. Moreover, there are costs associated with deep water disposal in terms of potential damage to the waters by resuspension of fine grained maintenance dredged material in the water column. It should be noted that in salt water environments, precipitation of fine sediment occur readily because salt in the water accelerates flocculation. However, in fresh water environments, precipitation is slow and sedimentation may require extended periods of time, particularly for fine grained soils.

Disposal by placement in an upland CDF may be a more environmentally desirable alternative because the maintenance dredged material is contained and confined; however, it is also a more expensive alternative in that it requires space (land), that must be semi-permanently dedicated to the facility. Additionally, the facility must be specifically designed to receive the dredged material and the material, itself, must be processed and dewatered after deposition. Management, processing, and dewatering dredged material require equipment and man power.

Maintenance dredged material is sent through a dredge pipe often at solids contents less than 15 percent, by volume, which means that 85 or more percent of the dredged material is water. Water is required to facilitate transportation through the dredge pipe, however, once in the disposal facility, water becomes a problem because it occupies valuable storage space and is difficult to remove. Sound management of CDFs is important because they have limited life; with time, their storage capacity is exhausted and no more material may be placed in the facility. If a CDF is managed poorly, valuable and possibly irreplaceable storage capacity may be wasted. Techniques are available to recover misused storage capacity, but these techniques are expensive to implement and may not always recover the storage capacity desired.

After placing dredged material in a CDF, it is allowed to stand for a period of time so that soil solids settle and clear water remaining at the surface is drained through weirs. After the decanted water is removed, further dewatering may be achieved by exposing the surface to evaporation effected by sun and wind, then digging/trenching ditches in the soft wet soil to allow remaining water to seep out of the mass under the force of gravity. A network of trenches may be designed and constructed in such a manner that water can be directed to on-site weirs and consequently removed from the CDF. Maintenance dredged material processed in this manner may be dewatered to a water content that is approximately the liquid limit (Spigalon and Fowler, 1987). Lowering the water content substantially below the liquid limit in a CDF is generally expensive and difficult.

Several techniques are investigated below to assess the materials in Cell #1 for consolidation potential, and to process the material in Cell #2 for consolidation potential. Laboratory tests performed on soils from Toledo CDF Cell #1 include Atterberg limits tests, grain-size analysis, oedometer tests, and specific gravity tests. Field tests include field vane shear strength tests and measurement of excess pore water pressure. Certain innovative procedures and techniques to facilitate consolidation were also considered and evaluated such as electro-osmosis, and strip drain analysis.

3 Piezometer Description and Installation

Piezometers were installed at Site 1 and Site 2 to measure hydrostatic excess pressure in the CDF from which potential for consolidation was determined. The piezometers installed at Toledo CDF Cell #1 were a type known as diaphragm piezometers that are characterized by two pneumatic tubes leading to the piezometric porous tip. A (flexible rubber) membrane flapper is forced against the end of one of the tubes by in situ water pressure after the piezometer is installed and the water pressure around the instrument has come to equilibrium. To make a pore pressure measurement from the observation station, air pressure from a pneumatic pressure indicator is introduced into the pneumatic tube that has the membrane flapper against it until the pore water pressure acting on the opposite side is slightly exceeded. Consequently, air is forced past the membrane flapper, escapes through the opposite tube, and is detected with a bubble chamber at the observation station. The air pressure in then reduced until bubbling stops; this air pressure in the line, as measured with a sensitive bourdon gauge on the pneumatic pressure indicator, is therefore assumed to equal the pore water pressure, and is recorded. A photograph of one of the piezometer tips installed at Toledo CDF Cell #1 is shown in Figure 3. Three piezometers at three depths below the ground surface, 8 ft, 16 ft, and 21 ft, were installed so that the profile of excess pore water pressure could be defined with depth. The piezometer tips were installed on specially prepared riser pipes and pushed into the soft ground by hand at Toledo CDF Cell #1. The pneumatic tubes coming from the piezometer tips extend through the riser pipes to the surface where observations of pore water pressure are made. A photograph of the riser pipes and the associated pneumatic tubes at Site 2 is shown in Figure 4.

After Installation (on 28 April 1994), the piezometers were observed[1] on 29 April, 4 May, 11 May and 19 May. By 19 May 1994, the instruments had come to equilibrium with the pore water environment in Cell #1, and showed a roughly triangular pattern of excess pore water pressure beginning at about 16 ft below ground level and increasing to about 5 ft at 21 feet below ground level.

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[1] Piezometric level was observed by personnel of the Toledo Area Office, U.S. Army Engineer District, Buffalo (CENCB)

Figure 3. Piezometer tip installed in CDF Cell #1

Figure 4. Riser pipe and pneumatic tube at Site 2

4 Material Characterization by Atterberg Limits

Atterberg limits are indices used to classify soils and to describe them in terms of consistency and texture. Additionally, Atterberg limits are useful in characterizing dredged materials that are placed in a CDF because they can be an indicator of how rapidly materials may be expected to settle through water and ultimately, how compressible the resulting soil layer might be. For example, at low water contents, clay soils are brittle solids and become plastic as water content increases. The term 'plastic' refers to the ability of a soil to be molded into various shapes without fracturing or cracking. At high water contents, a clay soil will flow like a viscous liquid. The solid, plastic and liquid states reflect the consistency or stiffness of the soil. Atterberg limits are the water contents at which soil consistency changes from one state to another. For example, the liquid limit (LL) is the water content at which soil changes from the liquid to the plastic state, and the plastic limit (PL) is the water content at which soil changes from the plastic to the solid state. These limits are arbitrary and based on the tests from which they are determined. Moreover, consistency of soil changes gradually from one state to another instead of an abrupt change at the limit. However, although arbitrary, these tests are now standardized (ASTM, 1995, U.S. Army Engineer Waterways Experiment Station, 1970), so that they may be used for soil classification and provide reliable indices to soil properties and behavior.

The plastic limit is defined as the water content at which soil crumbles when it is rolled down to a thread one-eight of an inch in diameter. In the standard liquid limit test, a soil-water mixture is placed in a metal cup of specified dimensions and mass, and a groove of precise size and shape cut in the soil with a tool. Liquid limit is the water content (expressed as a percent) at which the groove will close for a distance of ½ inch when the cup is dropped from a height of 1 centimeter twenty-five times, impacting on a hard rubber base (of specified hardness). The numerical difference between the liquid and plastic limits is called the plasticity index (I_p).

Atterberg limits used in conjunction with the plasticity chart devised primarily by Casagrande (1948) are used to classify fine grained soils. The plasticity chart is shown in Figure 5. On the plasticity chart, high compressibility soils are those with liquid limits above 50; low compressibility soils have liquid limits less than 50. If a soil falls below the A-line (See Figure 5), it is classified as silt, whereas soils falling above the A-line are classified as clay. For example, a soil with LL = 75 and PI = 47 is classified, CH, that is, clay of high compressibility, whereas a soil with LL = 46 and PI = 23 is classified CL, a clay of low compressibility. The notions of high and low compressibility have significant implications for materials placed in a CDF, in that these classifications are good indicators of the ease with which a soil may be processed and dewatered. Generally, soil with high compressibility (as indicated by the plasticity chart) require longer periods for sedimentation/settlement from a slurry and are more difficult to drain and dewater. High compressibility and propensity to retain water are characteristics associated with certain clay minerals present in the fine fraction of soils that Atterberg limits may help identify. The fine fraction of soils is that fraction of particles sizes less than 0.074 mm in diameter. For example, soils of high plasticity often contain significant amounts of the clay mineral montmorillonite, a mineral associated with extremely small particle size (and therefore slow settlement through water), low shear strength, low permeability, high compressibility and a high propensity to retain significant amounts of water. Conversely, soils of low plasticity are likely to contain non-plastic minerals such as quartz, feldspar, or clay minerals associated with low plasticity, such as kaolinite. These minerals are characterized by large particle sizes (and therefore rapid settlement through water), high permeability, low compressibility, and a propensity to not retain significant amounts of water. Therefore, Atterberg limits will allow assessment of the amount of difficulty expected in handling, processing, and dewatering materials placed in a CDF.

Atterberg limits tests, as well as all other laboratory tests performed in this investigation, are performed in accordance with EM 1110-2-1906 (U.S. Army Engineer Waterways Experiment Station, 1970).

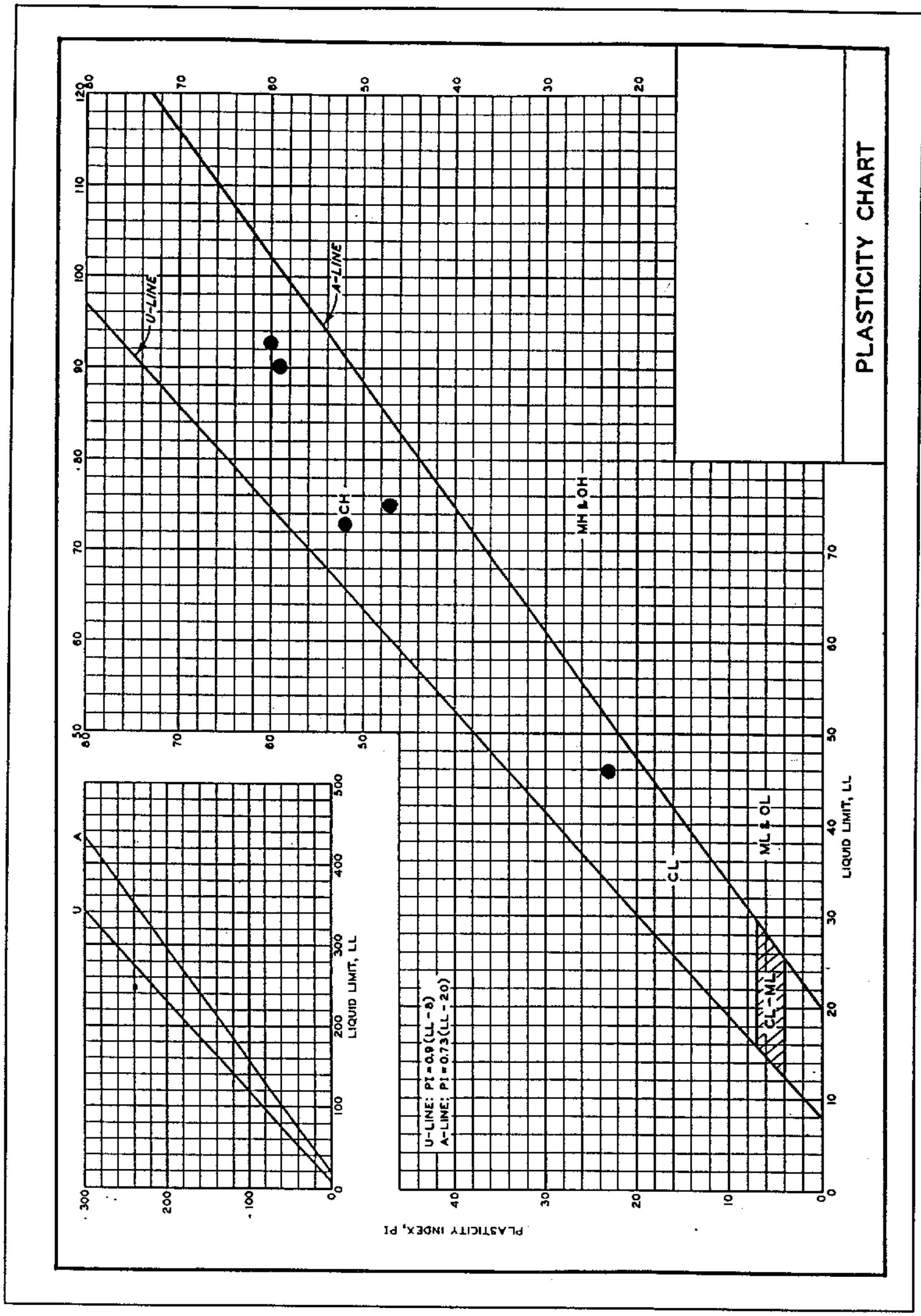

Figure 5. Plasticity chart

5 Recovery and Handling of Soil Samples

Soil was recovered from Site 1, Site 2 and Site 3 (shown in Figure 2) for laboratory tests and site characterization. Samples were taken from depths up to 13 ft deep by excavating with a back hoe at Site 1 and Site 2. Large intact chunks of soil were removed using the bucket of the back hoe and placed in O-ring sealed, five gallon plastic buckets with as little disturbance as possible. The material recovered from Site 3 was above the liquid limit and therefore in a liquid state. The soil samples were stored in an air conditioned environment at the Toledo Area Office of the U.S. Army Engineer District, Buffalo (CENCB), until they were shipped to the Waterways Experiment Station (WES). Once received at the WES the samples were stored in a controlled temperature and humidity environment until they were tested. However, because of the method of recovery and handling of these samples, they cannot be considered undisturbed.

Water content tests were performed on samples from all buckets of material and Atterberg limits were performed on material from selected buckets. Results of these tests are presented in Table 1.

Table 1 Water Content Variation				
Location	**Depth, ft**	**Water Content, Percent**	**Liquid Limit, Percent**	**Plasticity Index, Percent**
Site #1.	0-2	46.65		
	3½	54.44	73.0	52.0
	6½	62.31		
	8	50.84	46.0	23.0
	8-10	50.77		
Site #2.	2	114.31	93.0	60.0
	5-7	55.70		
	10	73.90	75.0	47.0
	12	75.89		
	13	65.30/63.49		
Site #3.	2	150.67	90.0	59.0

Referring back to Table 1, the water content variation were taken with depth at sites 1,2, and 3 along with liquid limit and plasticity index data. Three specific gravity tests were performed on materials at different depths; the value typical of the site is $G_s = 2.70$.

The Atterberg limits data are plotted on a plasticity chart shown in Figure 5. Four of the five samples tested are high compressibility clay CH soils and were typical. One soil plotted as a low plasticity CL soil and was selected for Atterberg limits testing because is appeared different from the others. The appearance of this low plasticity soil points out the random nature of materials placed in the Cell #1. If the soil samples recovered from Site 1, Site 2 and Site 3 are representative, most soils contained in Toledo Cell #1 are highly plastic clays, probably containing montmorillonite. However, some materials of lower plasticity are present in the site as confirmed by the CL soil identified at Site 1. It is likely that material plasticity is higher in the vicinity of the weir since less plastic soils are characterized by larger particles that will settle through water and be deposited at greater distances from the weir.

6 Grain-Size Analysis

Soil samples taken from the Toledo CDF consisted of fine-grained cohesive material. A combination of sieve and hydrometer tests was used to determine the grain-size distribution. A grain-size determination was performed for each of the three sites as identified in Figure 2.

Results of the grain-size analysis for material taken from Site 1, Site 2, and Site 3 are shown in Figures 6, 7, and 8, respectively. Although basically fine-grained, material from Site 1 contained about 8 percent sand; this is consistent with the fact that one of the Atterberg limits from Site 1 indicated a material of somewhat low plasticity. Material from Site 2 contained about 1 percent sand, and material from Site 3, which is near the weir, contained only about ½ percent sand.

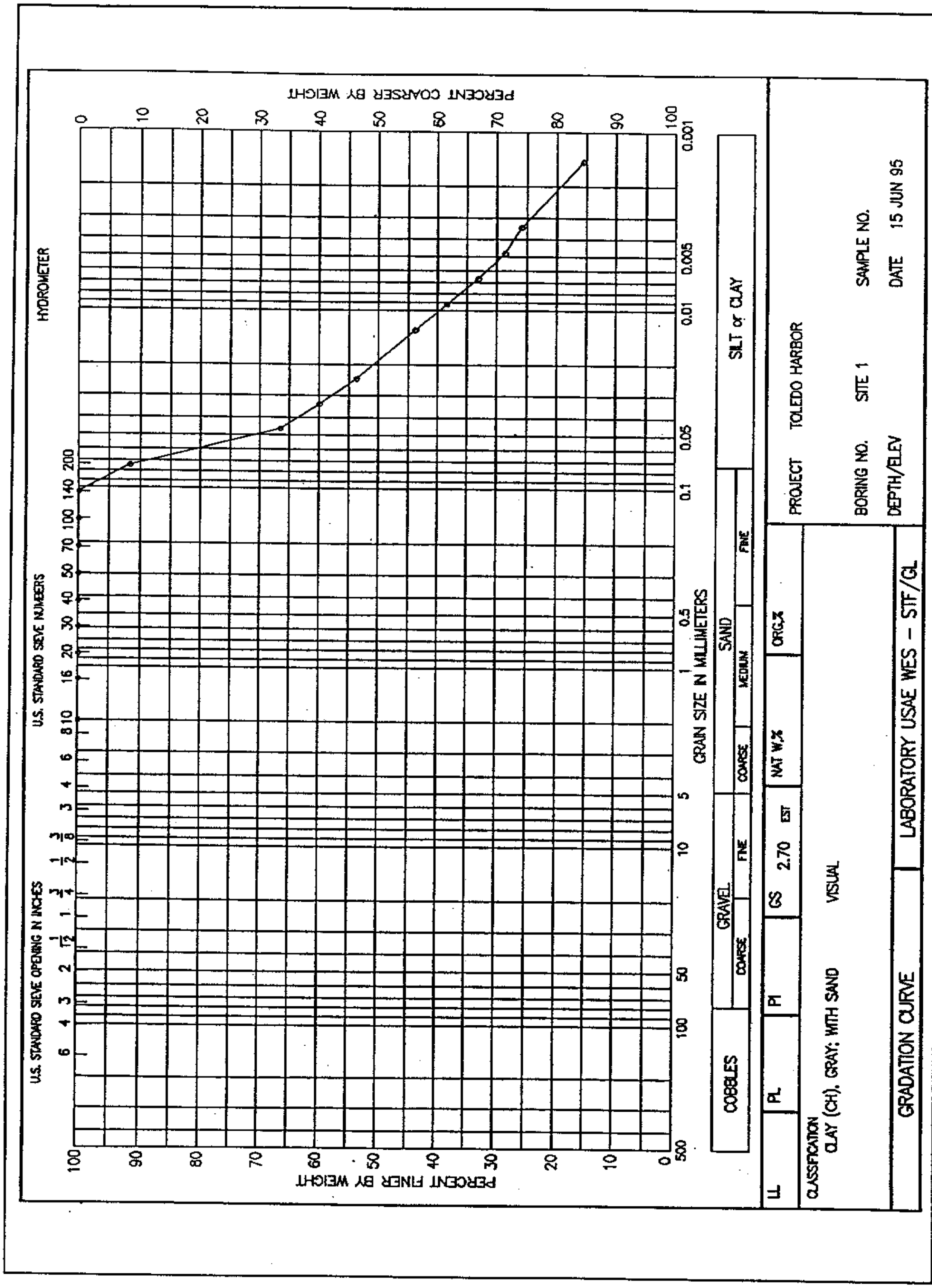

Figure 6. Grain-size analysis of Site 1 soil

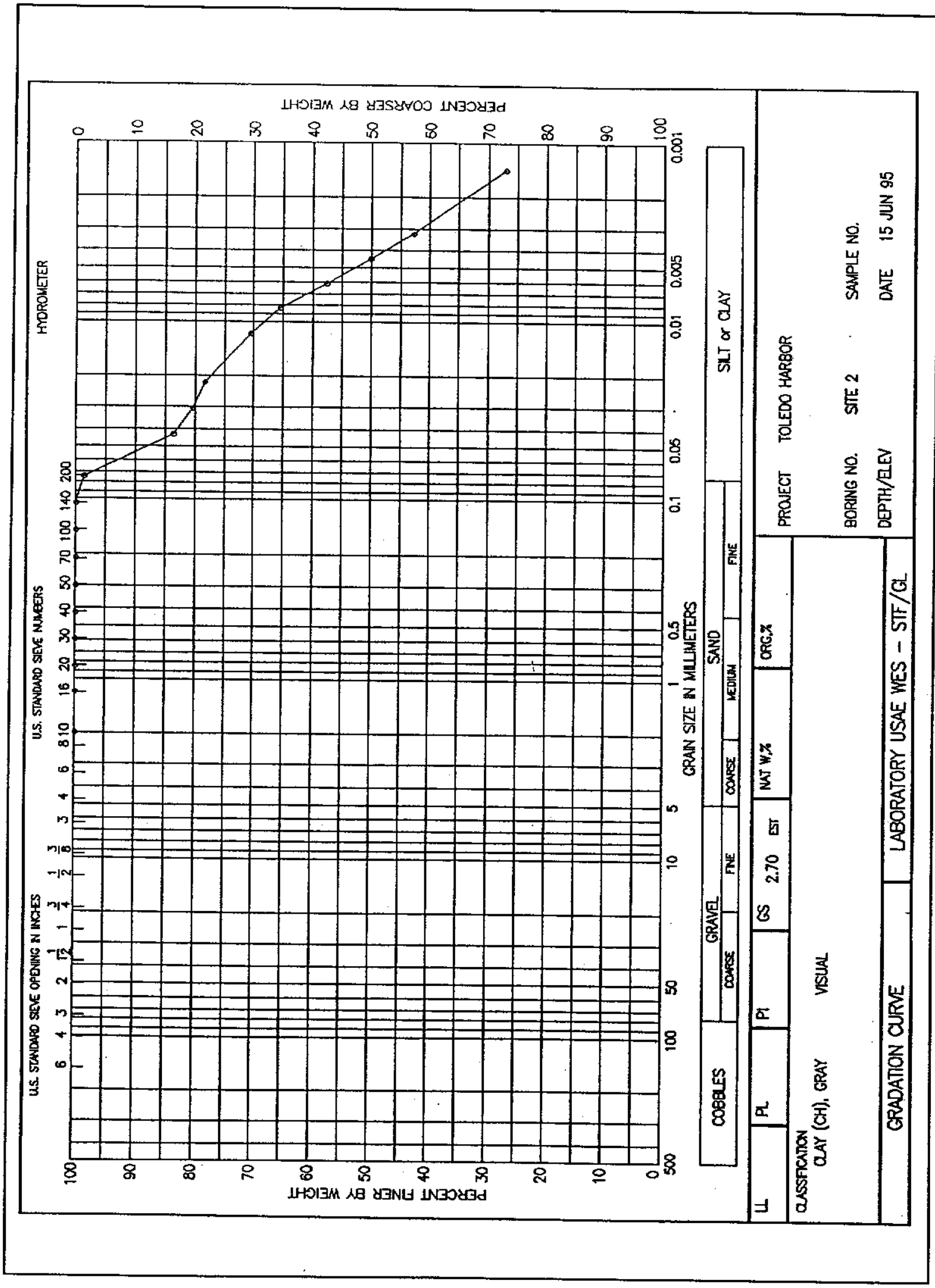

Figure 7. Grain-size analysis of Site 2 soil

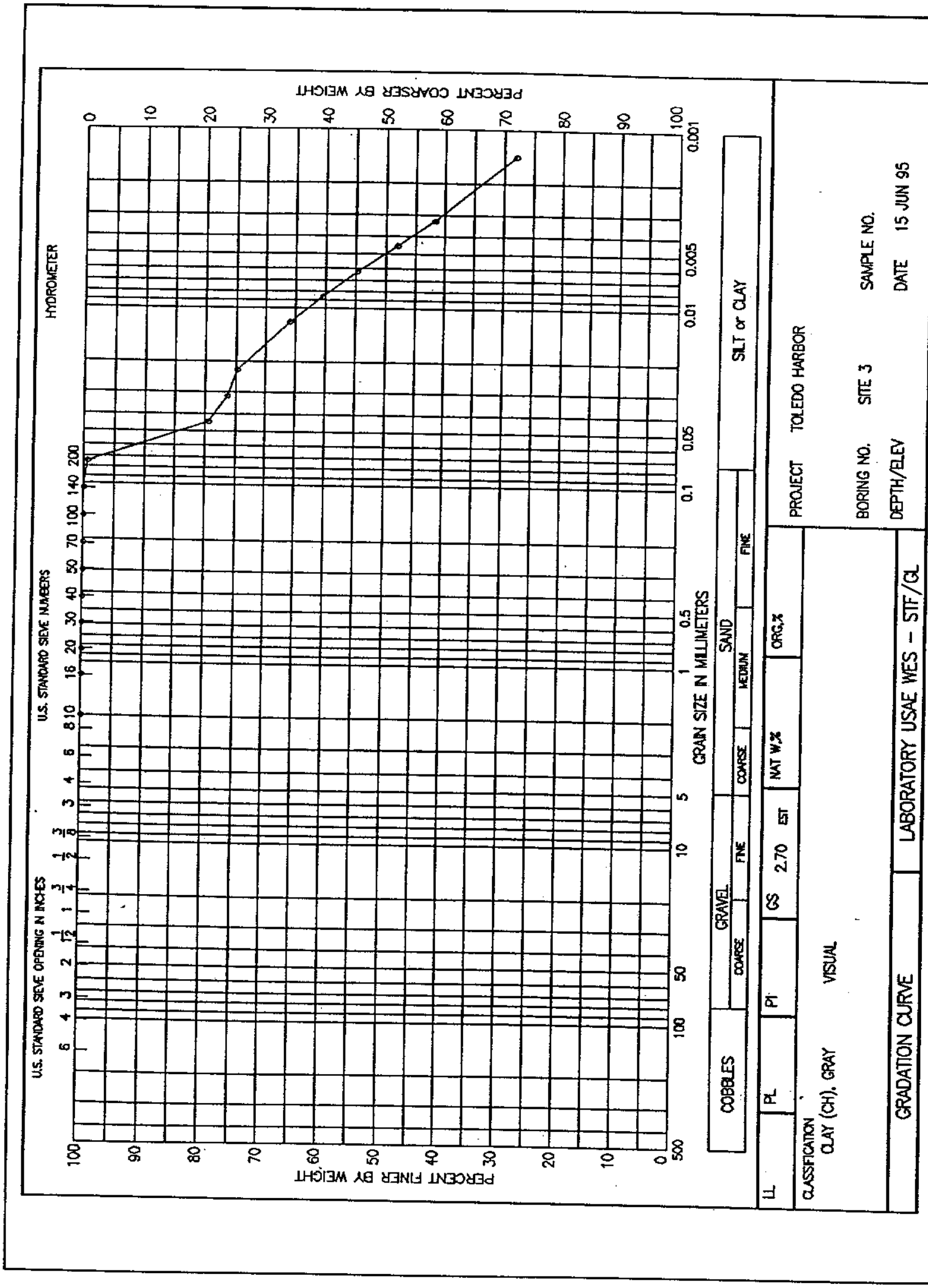

Figure 8. Grain-size analysis of Site 3 soil

7 Soil Strength Measurement Using Vane Shear Device

Many deposits of cohesive soil are too sensitive, too soft, too heterogeneous, or the intended use of strength information does not warrant the recovery of undisturbed samples and use of the triaxial apparatus to determine undrained strength. Additionally, the site in which a soil strength profile is desired may be so soft as to make the use of equipment required to recover undisturbed soil samples impossible or extremely expensive. Even if 'undisturbed' samples are recovered from sites where the soil is extremely soft, actual disturbance due to sampling, then trimming and installing the specimen into a triaxial apparatus will often result in measured strength that is significantly different from the in-situ strength, particularly if the subject soil is near the liquid limit. In such cases, the field vane shear test (VST) can be used quite effectively to determine shear strength profile.

A vane shear device generally consists of four thin blades fixed to and extending radially from a central rod as shown in the drawing of Figure 9. The vane is inserted into the soil at a selected location, then pushed to the depth at which shear strength measurement is required. The device is then slowly rotated while measuring the torque required to cause rotation. The accepted standard for the ratio of vane height to diameter is 2 to 1 (American Society for Testing and Materials, 1956); shear occurs in the soil around the surface area of a cylinder with the height and diameter of the vane, and the undrained shear strength computed from the dimensions of the cylinder and the torque measurement. No useful stress-deformation properties are obtained from the vane shear device, only a value of undrained strength.

The vane shear test is relatively quick and easy to perform, but several drawbacks limit its use. The major disadvantages of the test are, 1). drainage conditions are not well known or controlled, and, 2). the failure plane is controlled by the vane's movement and may not be the critical plane. Because the materials in the present investigation are soft and saturated, the disadvantages outlined above are not significant.

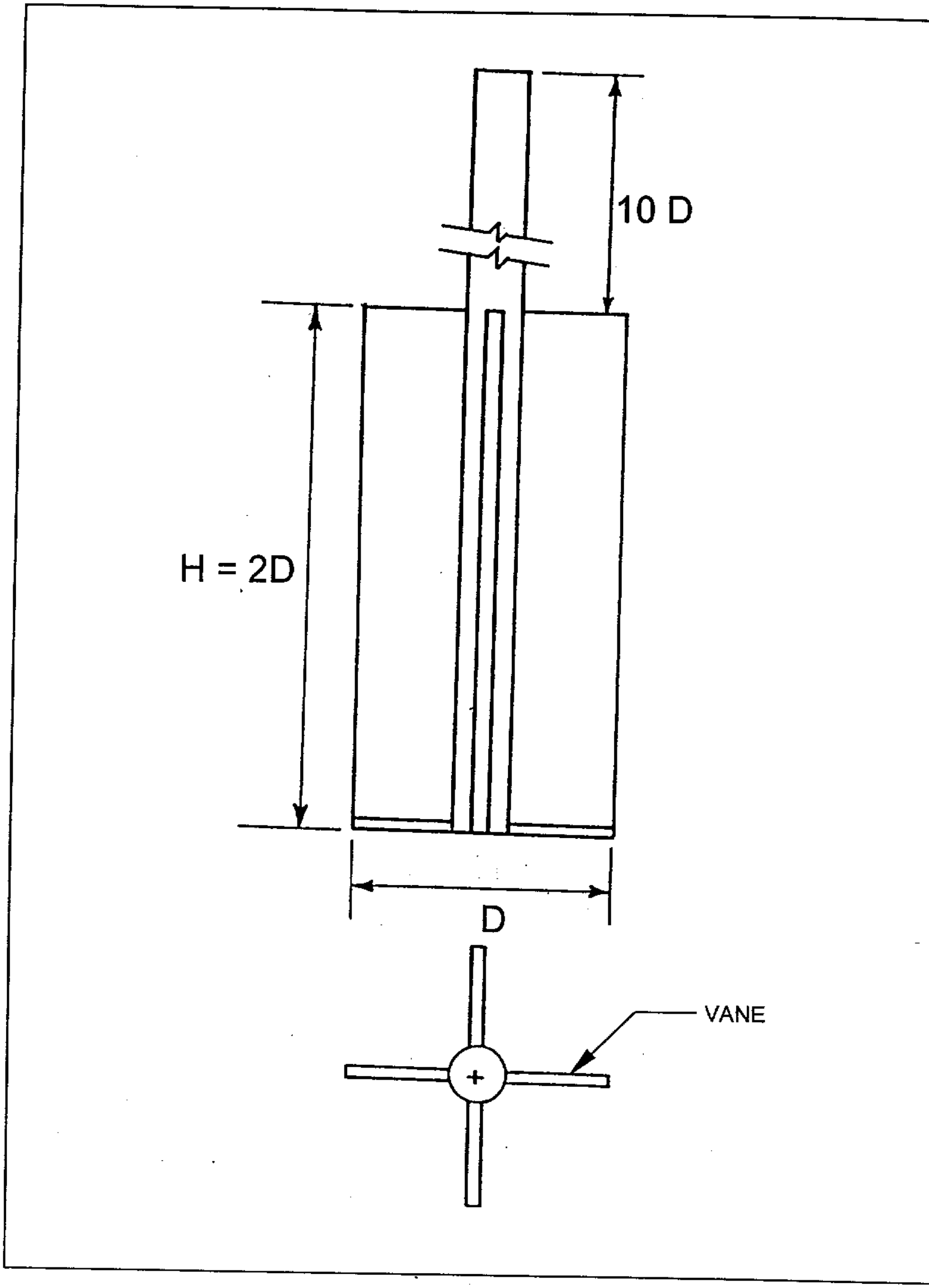

Figure 9. Drawing of vane shear device

A major advantage of the test is that it is an insitu test where disturbance is minimal and the effects of confining pressure are correctly represented. Additionally, the test does not require sophisticated equipment and is not difficult to perform.

The rate at which the vane is rotated affects the test results. Considerable effort has been spent determining an appropriate rate of rotation for strength measurement with the vane shear device; that rate is reported to be 6 °/min (American Society for Testing and Materials, 1956).

Three vanes (that are 2, 3, and 4 in.in diameter) were taken to the Toledo Cell #1, since the vane size that would prove most effective and advantageous for strength measurement at this site was unknown. All vanes in the set are constructed with a height to diameter ratio of 2 to 1. A vane measuring 3 in. in diameter was initially selected. This instrument produced satisfactory results in that the torque required to rotate a 3-in.vane was in a suitable range for the torque sensing device used and the vane was easy to handle and install. Consequently, the 3-in.vane was used for all subsequent strength measurement. Data were observed at different depths within a selected location; the vane was positioned in the soil mass by pushing with a downward force limited to about 170 pounds, although penetration was always achieved with a much lower force. When a depth was reached where strength measurement was desired, the vane was rotated at the prescribed rate, data for the peak and residual strengths were acquired at that depth, then the vane driven deeper for the next observation. Residual strength is the strength measured on remolded soil observed after ten or more rotations of the vane.

The configuration of the torque wrench used in this investigation is basically that of a cantilever beam that is deflected by a force applied at a well controlled distance (moment arm). Deflection of the beam is measured with a dial gage having a scale that read torque directly. However, a careful calibration showed that it is necessary to apply a factor to readings from the torque wrench to obtain true/correct values of torque. Torque is converted to average shear stress using a relationship discussed in ASTM Publication 193 (American Society for Testing and Materials, 1956),

$$S = \frac{6\,T}{7\,\pi\,D^3} \tag{1}$$

where

T = resisting torque
D = diameter of the vane

Derivation of equation 1 is based on the fact that the height of the vane is twice the diameter, D.

Strength Profile Determined with Vane Shear Measurements

In April, 1994, when the water table was very near the ground surface, measurement of the shear strength profile with depth using the vane shear device described above yielded the results presented in Table 2. A specialist observing strength data with the vane shear device is shown in the photograph of Figure 10. At Site 1 and Site 2, shear strength appears to increase to a maximum value at about one-half to one-third the total depth, then decrease to a constant value. Peak strength of materials in Site 2 is slightly higher than that of Site 1. Materials present in the site are only slightly sensitive, as the ratio of peak strength to residual strength is typically less than 4.

Table 2
Shear Strength Variation with Depth at Sites 1 and 2

Location	Depth, ft.	Peak Torque (in-lb)	Peak Strength (PSF)	Residual Torque (in-lb)	Residual Strength (PSF)
Site 1	7.00	185	269	109	158
	7.75	434	632	109	158
	9.83	412	600	109	158
	12.83	304	442	109	158
	16.33	304	442	109	158
	18.17	271	395	109	158
Site 2	2.75	217	315	54	79
	5.67	521	758	141	205
	7.88	293	426	76	82
	11.00	282	410	109	158
	12.79	271	395	141	205
	16.38	271	395	109	158
	17.92	326	474	163	237

Data observed 4/28/94 with 3 inch diameter, 6 inch high vane shear device.
Water table was at the surface on 4/28/94.

Figure 10. Field use of vane shear device

8 Consolidation and Oedometer Tests

Consolidation

Consolidation of soil consists of compression of a soil mass and expulsion of water as the result of applied pressure. Movement of water through a soil mass requires time. The time rate of soil compression is controlled principally by permeability of the soil; the coefficient of permeability is that property which indicates the ease with which pore fluids move through the interstices of a particular soil. An extensive mathematical description of the consolidation process has been developed, beginning with Terzaghi (1925). The basic theory as developed by Terzaghi (1925), is used to estimate settlement due to dissipation of excess pore water pressure within a soil mass.

One of the assumptions made in development of the theory of consolidation is that the material is completely saturated. That assumption is reasonable in this investigation because the soil under consideration was first taken from a submerged environment, delivered through a dredge pipe in a suspension at a water content that is much higher than the saturated water content, deposited in a CDF and allowed to settle through water. Decant water is then removed and the surface allowed to dry. Even though some drying at the surface produces less than complete saturation, analysis of specimens recovered from Cell #1 indicates that, except for the first 2 to 3 ft, soil in the CDF is water saturated.

Oedometer Tests

To estimate soil settlement, compression tests must be performed on suitable specimens and soil parameters measured that are used in conjunction with consolidation theory. In the oedometer test, stress/pressure is applied to soil which is placed inside a metal ring. Specimens tested were 4.44 in. in diameter and about 1.1 in. high. Pressure is applied along the vertical axis through a porous stone; the metal ring is stiff enough to prevent strain in the horizontal direction. A porous stone is placed on top of the specimen to allow the escape of water as the specimen compresses. Measurement of specimen vertical deflection is observed (using a dial gage) as a function of time.

Oedometer tests were performed on four representative specimens, two from Site 1 and two from Site 2. The specimens were loaded with vertical pressures of 0.0625, 0.125, 0.25, 0.5 and 1.0 tons per square foot. The loads were left on for a period of 24 hours.

Deflection versus time curves and void ratio versus pressure relationships are included in Appendix A. Water content of the specimens ranged from about 60 to about 70 percent and saturation from about 95 to 100 percent. The relationships between vertical deflection and time, and void ratio and pressure as presented in the data of Appendix A are typical for relatively soft cohesive soils.

Determination of Coefficient of Consolidation

The value of C_v, the coefficient of consolidation, is determined using curve fitting procedures applied to time-compression curves from one dimensional consolidation tests. The logarithm of time-fitting method for determining coefficient of consolidation is used in this investigation. The assumption is made that the initial shape of the consolidation curve is a parabola. From that assumption, the initial reading, d_0, is extrapolated. Deflection at 100 percent primary consolidation, d_{100}, is found by extending the straight-line portions of the middle of the curve and the end of the curve to a point of intersection. Deflection at 50 percent primary consolidation d_{50} is halfway between the initial reading, d_0, and d_{100}. Time for 50 percent consolidation, t_{50}, is read on the time axis at the point where a horizontal line through d_{50} intersects the consolidation curve. The coefficient of consolidation, C_v, is then determined from the equation,

$$C_v = \frac{T_{50}\ (H_{dp})^2}{t_{50}} \qquad (2)$$

where

T_{50} = fifty percent consolidation time factor, = 0.197
H_{dp} = length of drainage path
t_{50} = time for fifty percent consolidation.

The coefficient of consolidation, C_v, changes with stress level.

The small specimen tested in the oedometer is a model or element from which behavior of the greater mass is estimated. It should be noted that the accuracy with which 'true' behavior of the mass can be predicted depends on how homogeneous the mass is, that is, how well the mass is represented by the small model, and how disturbed the test specimen is. Compressibility from oedometer tests on 'undisturbed'[2] soil specimens increases as actual disturbance increases. Perloff and Barron (1976) discuss the effect of sampling disturbance on settlement, and conclude, that the difference between the observed compressibility curve and the in-situ relationship increases as the amount of disturbance increases. Figure 11 is reproduced directly from Perloff and Barron (1976); it illustrates (for normally consolidated clay) that for a given effective consolidation stress, σ'_c, the associated change in void ratio decreases (settlement increases) as the soil condition goes from the insitu (undisturbed) condition to a completely remolded (disturbed) state. The terms e_o, σ'_o, and σ'_p are the insitu void ratio, effective vertical insitu stress and effective preconsolidation pressure, respectively, of the soil sample as shown in Figure 11.

As Figure 11 depicts, compressibility and therefore settlement from consolidation increases as disturbance increases. The soil samples recovered from Cell #1 were not completely remolded, but did contain the effects of notable sampling and trimming disturbance. Because of acknowledged disturbance, the magnitude of settlement predicted/calculated from the consolidation analysis is greater than what will actually occur in the field in undisturbed soil.

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[2] The term 'undisturbed' soil specimen means that special procedures have been used to recover the soil and disturbance has been minimized. Disturbance to the soil, however, is inevitable, simply due to the fact that the soil has been removed from its insitu environment. Some 'undisturbed' soil samples are more disturbed than others (Gilbert, 1992), depending on soil type, depth, and equipment and techniques used to recover the material.

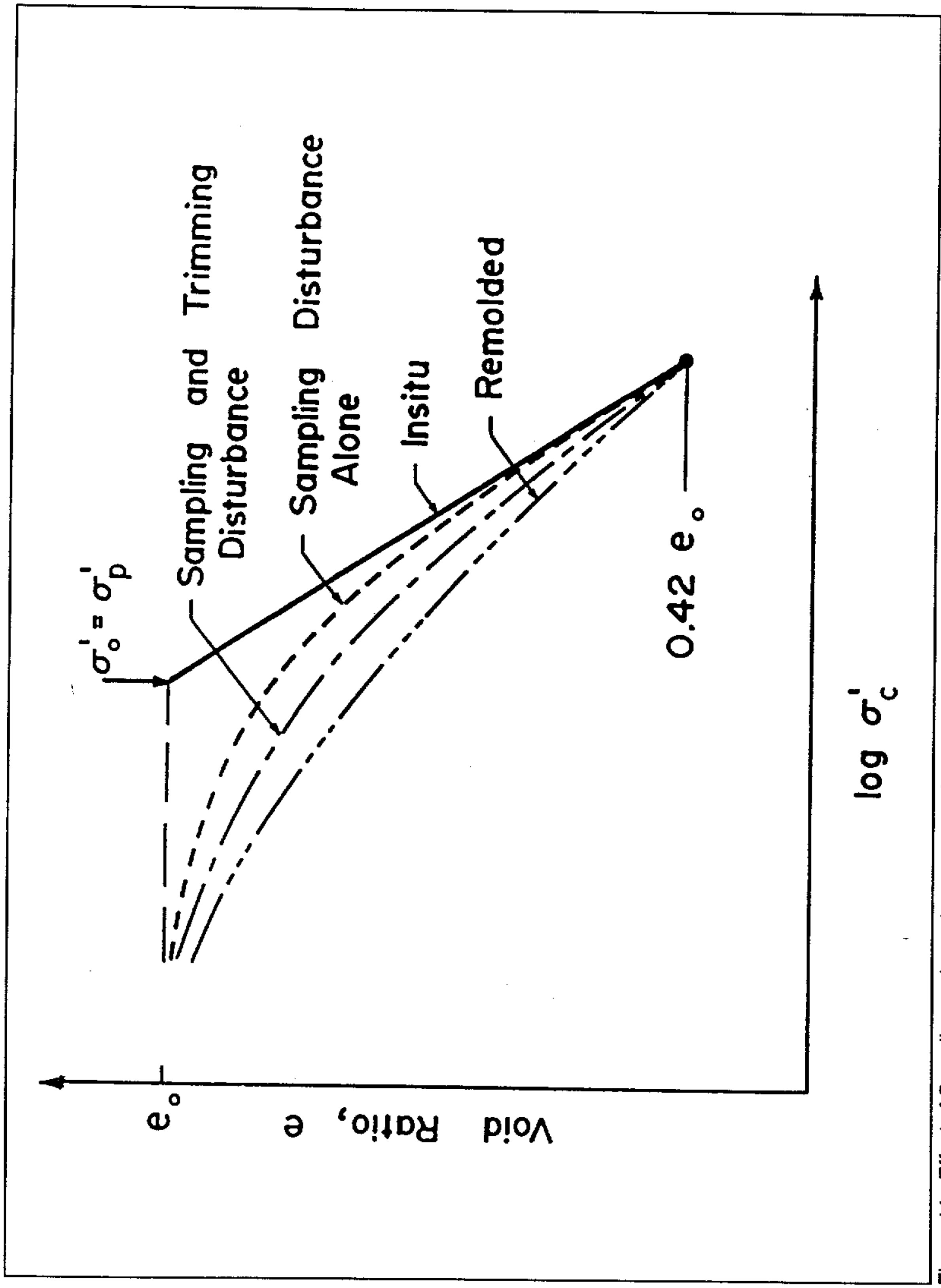

Figure 11. Effect of Sampling and specimen preparation on the laboratory e-log σ_c' curve

9 Use of Strip Drains to Accelerate Consolidation Experience at Craney Island

Stark (1994) describes a demonstration at Craney Island, a 2400 acre CDF at the mouth of Chesapeake Bay, near Norfolk, Virginia, where the use of vertical strip drains resulted in a rapid increase in storage capacity. Vertical strip drains are constructed of a flexible plastic core with channels or studs for water flow, encased in a nonwoven geotextile. Channels in the plastic core carry water to the surface (or other drainage layer) to relieve excess pore water pressure, and the surrounding geotextile prevents soil from entering and clogging the core. Because of ease of installation and reliability, vertical strip drains have replaced conventional sand drains in the last 5 to 10 years, and have been used in many projects to accelerate consolidation in soft, fine grained soils. Stark (1994) states that the cost of installed strip drain is $0.40 to $1.00 per linear ft, depending on the quantity installed, whereas the cost of installed conventional sand drain is $3.50 to $6.50 per linear ft. Therefore, vertical strip drains offer considerable cost advantage over conventional sand drains.

Strip drains are stored on large rolls at construction sites until they are needed for installation. They are installed by encasing them inside a hollow mandrel, pushing the mandrel and strip drain into the soil, then withdrawing the mandrel while the strip drain remains in place. Before installation, the strip drain is threaded through and attached to a metal plate that is held at the end of the mandrel. The metal plate prevents soil from entering the mandrel during installation. When the mandrel is pushed into place, soil flows around the metal plate and holds it in place when it reaches the bottom of the opening, therefore the strip drain is not pulled out as the mandrel is withdrawn. The strip drain insertion cycle can vary from 1 to 5 min, depending on the depth.

Strip drains accelerate consolidation by providing a path for the escape of pore water to relieve excess pore water pressure. Excess pore water pressure in a soil mass is pressure that is greater than gravity produced hydrostatic pressure. <u>If there is no excess pore water pressure in a soil deposit, then vertical strip drain</u>

will produce no consolidation. If there is excess pore water pressure in a soil deposit, time rate of consolidation can be controlled by installing vertical strip drains. For example, to estimate the time for 90 percent consolidation, equation 1 can be rewritten

$$t_{90} = \frac{T_{90}\,(H_{dp})^2}{C_v} \tag{3}$$

where

T_{90} = ninety percent consolidation time factor = 0.848.

All the factors on the right hand side of equation 3 are constant except H_{dp}, the length of the drainage path through which excess pore water must flow to escape. Therefore the time for 90 percent consolidation, t_{90}, may be controlled by manipulating the spacing of vertical strip drains installed in a soil mass to shorten the length of the drainage path, H_{dp}. Obviously, control of the rate of consolidation with strip drains comes at the price required to purchase and install strip drains.

Factors Affecting Amount of Consolidation

The amount of consolidation that can be achieved by dissipation of excess pore pressure is a function of the amount of excess pore water pressure and the thickness of the soil deposit. Little excess pore water pressure and/or thin soil deposits will produce small amounts of consolidation.

Craney Island is unique among CDFs in that it was constructed over soft, underconsolidated marine clays with excess pore pressure measured to be 40 to 50 feet in some locations within the site. Additionally, the underlying marine clay stratum is of the order of 70 feet thick, and the placed dredged material is 30 feet thick, for a total thickness of approximately 100 feet. If dissipation of excess pore water pressure produces a volumetric strain of 10 percent, the result is 10 ft of settlement, which is substantial. However, geology and foundation conditions at each CDF are unique, so consolidation potential will likely be different from that at Craney Island. For example, the Toledo CDF is underlain by hardpan, which is a general term used to describe a hard cemented soil layer that does not soften when wet. Therefore, it is unlikely that any significant consolidation of the foundation material of Cell #1 will occur.

10 Settlement Analysis of Cell #1 of the Toledo CDF

Consolidation tests were performed on material from Site 1 and Site 2. The material selected for the settlement analysis was a hybrid material having the highest density of all materials tested and the greatest compressibility. Such a material will produce the largest estimate of settlement because the highest possible pressure is applied to the most compressible material. Density for the hybrid material is that from Site 1 from a depth of 10 feet below ground level, compressibility characteristics are from Site 2 at a depth of 13 feet below ground level. A void ratio of 1.371 was used to determine a saturated wet density of 108.4 PCF. A capillary zone 2 ft thick at the soil surface is assumed. Assumed also is that 6 ft of excess pore water pressure exists from 16 ft below grade down to 21 ft, which is the bottom of the cell. Pore pressure actually measured with piezometers indicate that the excess pore water pressure is actually about 5 feet at 21 feet, increasing from hydrostatic at perhaps 19 feet. Based on consolidation characteristics of Site 2 material from a depth of 13 ft, total settlement from dissipation of 6 ft of excess pore water pressure in a five foot thickness (from 16 to 21 ft) will amount to about 2.5 inches.

It is the intention of this analysis to consider the worst case conditions in estimates of settlement. The triangular distribution of excess pore water pressure that was actually measured by the installed piezometers is small. The assumed 6 ft of excess pore water pressure over a five foot thickness at the bottom of the cell will result in a liberal estimate of settlement. All assumptions made for the settlement analysis will produce more settlement than what will occur in the CDF, including the fact that the material on which consolidation tests were performed, was undisturbed. Therefore, the amount of settlement is estimated from the analysis, 2.5 inches, is larger than settlement likely to occur in the field.

Additionally, if the layer of soil analyzed above is surcharged with a thickness of material that applies 5.0 PSI of pressure, analysis shows that the settlement is 1.36 ft. However, if no adjustment is made to the drainage path length (the drainage path, H_{dp}, is 20 ft because there is an impermeable layer of hardpan at the bottom of the cell which is 20 ft deep), time required to achieve this amount of settlement is computed to be about 66 years.

Cost Estimates for Strip Drain Installation

If vertical strip drains are installed on a 12 ft triangular pattern described by Stark (1994), then the time required to achieve 1.36 ft of settlement is reduced to 6 years. If a single strip drain covers an effective area of 125 square feet, and the area of Cell #1 is 220 acres with an average thickness of 18 feet, then about 1,380,000 linear feet of strip drain are required to cover the site. The installed cost of strip drains under these conditions is $966,000 or $552,000 if the installed cost of strip drain is $0.70 or $0.40 per linear ft, respectively. These costs do not include the cost of a horizontal drainage system, or the cost of placing 6 ft (about 5 PSI) of surcharge over the site.

11 Electro-Osmosis

If a direct current (DC) voltage is applied across a wet soil mass, cations are attracted to the cathode and anions to the anode. There is usually an excess of cations in a soil-water mixture and as cation migrate toward the cathode, they drag water molecules along with them, causing a net water flow toward the cathode. This water may be collected at the cathode and removed.

Applying a voltage across a thickness of soil in the manner described produces an electric potential gradient in soil that is directly analogous to the hydraulic gradient produced by a water pressure differential in a soil mass. The electric potential gradient can be hundreds of times stronger than a hydraulic gradient, and will produce flow as long as the electric potential (voltage) is applied. The major disadvantage in using this technique is expense; the generation of electricity is cost and energy intensive. In spite of associated high costs, the technique can be effective, advantageous, and economical in dewatering wet dredged material, and theory as well as field observation (Mitchell, 1976) suggests that electro-osmosis is more effective and efficient in dewatering clay than fine sand. For example, Casagrande (1953) describes the successful use of electro-osmosis on several field projects involving fine-grained clays. However, the efficiency of electro-osmosis for dewatering is sensitive to the conductivity of the pore water in the soil. Mitchell (1976), produces analysis to show that power requirements vary directly as pore water conductivity. Mitchell (1976) also presents data from field case studies to demonstrate that electro-osmosis is effective in dewatering soils saturated with pore water having conductivity in the range 200 to 300 μmhos, whereas electro-osmosis was not effective when conductivity of the pore water was 2500 μmhos. It is interesting to note that although the Maumee River and Lake Erie are fresh water sites, conductivity of the pore water taken from soils in the CDF is not in the range for effective and efficient dewatering with electro-osmosis. Conductivity of the pore water collected from electro-osmotic consolidation of soil taken from the interior of Cell #1 (Site 2) is measured to be 1100 μmhos, and pore water taken from wet, high water content ($w = 150$ percent) soil from Site 3 near the weir is determined to have conductivity of 1800 μmhos. Measurements of the conductivity of the pore water were made with a YSI Model 30 salinity/conductance/temperature meter manufactured by Yellow Springs Instruments of Yellow Springs, Ohio. The instrument is specified to have a precision of 1 μmhos and an accuracy of ±25 μmhos over the range required to measure conductivity of pore water samples from Cell # 1.

Bench Scale Electro-Osmosis Test

Electro-osmosis was performed on a fairly large specimen consisting of about 34 pounds of wet material to determine the effectiveness of electro-osmosis on a material taken from the Toledo CDF. The material tested was taken from Site 2 from a depth of 10 ft, with the water table at 6 ft. Total density (wet unit weight) of the soil was determined to be 1.546 g/cc (96.5 PCF), and the water content measured to be 73.9 percent for a dry unit weight of 55.5 PCF; this specimen was determined to be water saturated. Total dry weight of soil in the specimen is 8842 g. An electric potential of 10 volts DC was passed through electrodes in the soil 6.25 inches apart for a period of about 49 days. Water expelled versus time relationship is shown in Figure 12, which shows that water removal is most rapid at the beginning of energy application; the rate of expulsion decreases rapidly with time and appeared to reach a point of diminishing returns where little water is expelled. The process produced the expulsion of 356 grams of water for a change in water content of 4.2 percent. This represents a change in water content from 73.9 to 69.7 percent, which is a modest decrease in water content, and a negligible 1.26 percent decrease in total volume, for the expenditure of substantial electrical energy. For example, the total energy expenditure was about 0.6 KWH for the recovery of 356 g water; this amounts to 6.3 KWH per gallon of water recovered. If electricity costs $0.10 per KWH, then the cost to recover water is $0.63 per gallon of water, which is a prohibitively high cost if projected to the prototype. Power applied versus time is shown in Figure 13 and shows a similar trend where power decreases to an approximately constant level at about 14000 minutes (10 days), which is approximately the point of diminishing return. If it were decided to arrest power application at 14000 minutes, cost at that time is $0.22 per gallon of water recovered if this bench scale test can be projected to the field. This cost is only that of electricity at $0.10 per KWH and does not include the cost of equipment, field preparation, or installation of electrodes. For these expenditures, a modest 3.6 percent decrease in water content would be achieved and a decrease in total volume of 1.08 percent. For these soil and water conditions, electro-osmosis cannot be justified as a viable economic technique for dewatering dredged material.

Conductivity of Cell #1 material is larger than what is optimal for electro-osmosis, causing the efficiency of the process to be low and energy costs prohibitively high. However, the technique of electro-osmosis should not always be summarily dismissed. Under circumstances of low pore water conductivity, the process could prove economical and could be used to considerable advantage for dewatering high water content, fine grained soils. Obviously, since the efficiency of electro-osmosis for dewatering fine grained soils decreases with salinity, the process will not be useful in environments containing saline or brackish water where conductivity can be greater than 30,000 μmhos.

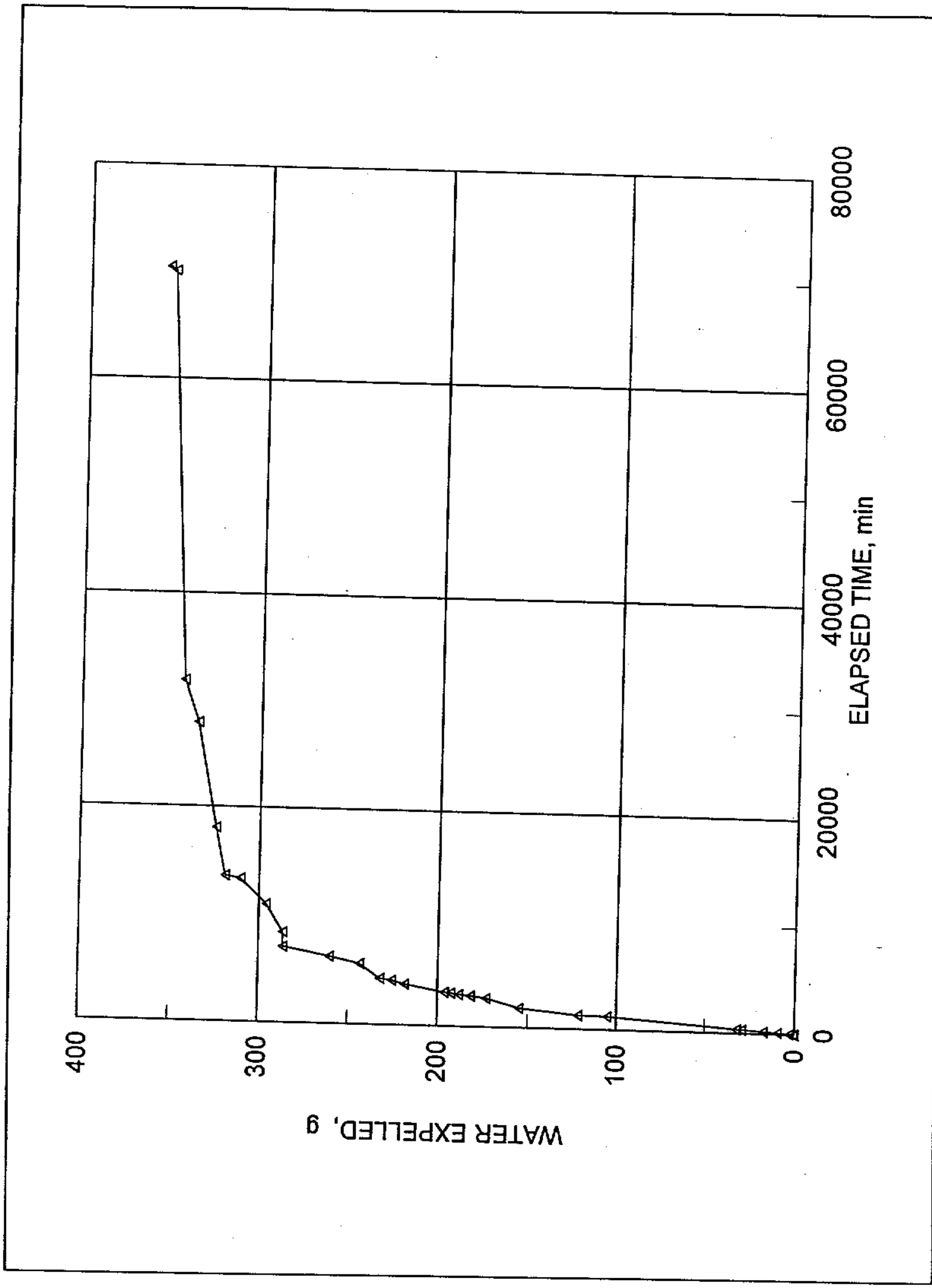

Figure 12. Water expelled by electro-osmotic pressure versus time

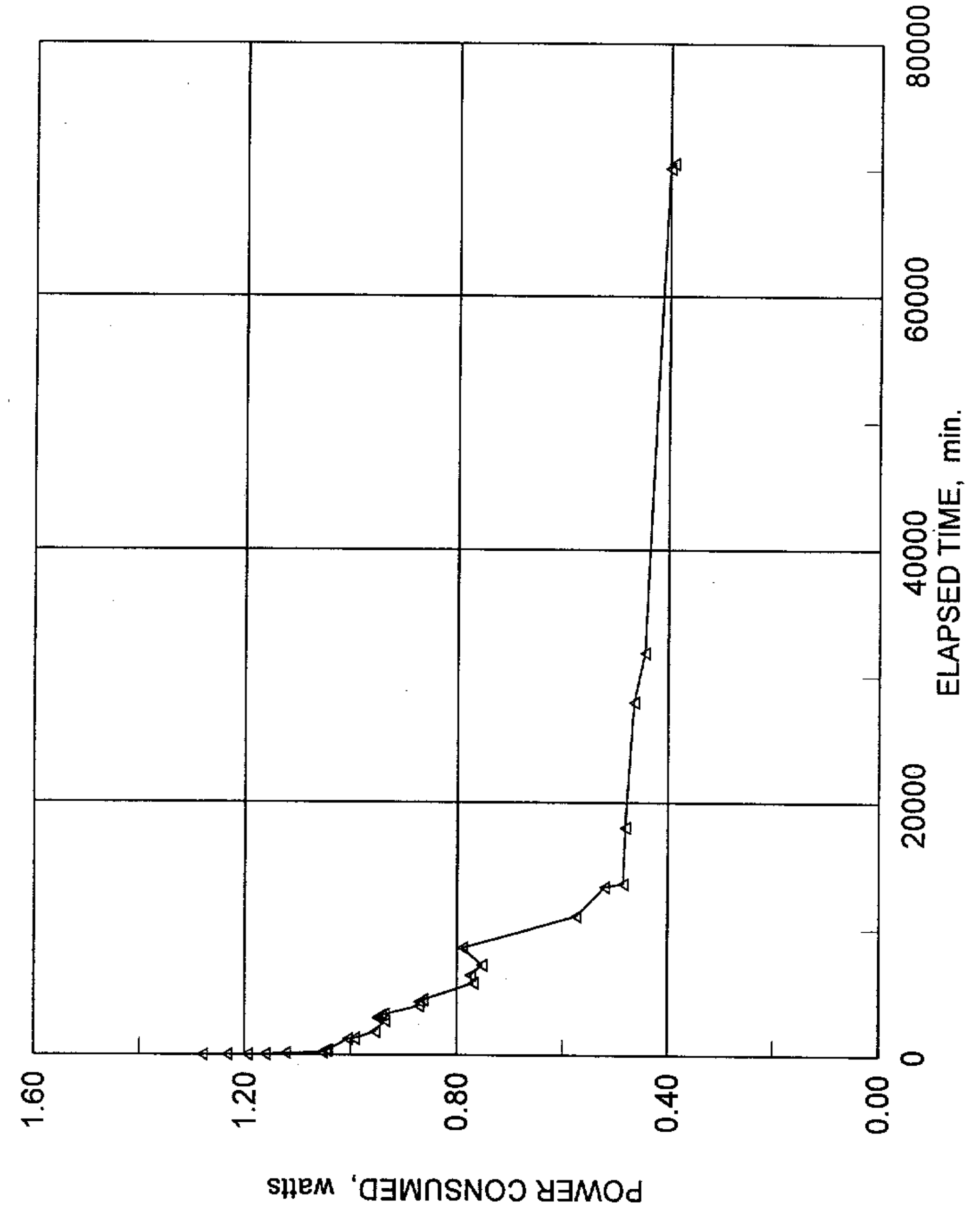

Figure 13. Power consumed versus time

12 Observations and Conclusions

Observations

The water contents of most of the soil at Site 1 and Site 2 are very near the liquid limit, and some water contents are lower than the liquid limit. This observation is the result of direct comparison of water content data with Atterberg limits data shown in Table 1. Additionally, the measured strength profile suggests that materials (at Site 1 and Site 2) are at, or lower than the liquid limit. For Example, Casagrande (1938) reports that the strength of any soil at the liquid limit is about 57 PSF. All strengths measured and reported in Table 2, including residual strengths, are greater than 57 PSF, implying that soil water contents are lower than the liquid limit.

Additionally, the method of material sampling suggests significant soil strength. Soil samples were recovered from depths up to 13 ft below ground level at Site 1 and Site 2 in Cell #1 by excavating with a back hoe; it is important to note that the excavations opened for soil recovery remained open during the sampling operation. The fact that excavations from 10 to 13 ft in depth did not collapse is an indication that the soil possesses substantial strength. For example, a value, H, given by the equation

$$H = \frac{2\,c}{\gamma} \qquad (4)$$

is the theoretical depth at which sloughing takes place in a vertical cut of purely cohesive soil that has no bracing. In equation 4, c and γ are the cohesion and total density, respectively, of the associated soil. If 10 ft is substituted for depth, H, in equation 4, along with a total density, γ, of 108.4 Pounds per Cubic Foot (PCF)[3], then cohesive strength, c (in PSF), of the soil is determined to be 542 PSF, which is consistent with peak strength values reported in Table 2. After the soil samples were taken, the excavations were deliberately refilled for reasons of safety. It is

[3] Total density estimated from a water content of 55 percent, a specific gravity of 2.70 and the assumption of complete water saturation.

highly likely that, with time, the excavations would have spontaneously failed by sloughing if they had been left open. The important point is that unbraced excavations opened to a depth from 10 to 13 ft, did remain open for a long enough period to allow recovery of soil samples, and that fact is indicative of substantial soil strength.

If Site 1 and Site 2 are typical of the entire CDF, water removal (and strength enhancement) has occurred during the life of the facility by some combination of natural drying from sun and wind, water removal by willow trees and other vegetation[4] on the site, or by consolidation. These things could not have occurred without deliberate management and effort that includes maintaining the site in the dry by regularly and periodically draining decant water through the weir. Deliberate management of the site is reflected and supported if the low excess pore water pressures, relatively high strengths, and water contents very near the liquid limit measured at Site 1 and Site 2 are characteristic of most of the site. However, experience has shown that reduction of water content in CDFs substantially below the liquid limit may be difficult.

At Site 3 near the weir, the material is very wet (measured water content beneath the surface crust is about 150 percent) with low strength. Materials at this location may be more difficult to dewater because soil plasticity at this location is likely higher than that in materials elsewhere in the site. Appropriate measures to decrease water content and increase storage capacity at this location include those described by Leshchinsky and Fowler (1993), notably,

a. maintaining a dry surface to the greatest extent possible by draining decanted water through the weir.

b. perimeter and/or internal cell trenching

c. periodic removal of crust material

d. deep disking (using low ground pressure equipment) to promote drying by exposing lower layers to sun and wind.

Conclusions

Based on data observed at Cell # 1 of the Toledo CDF, laboratory test results, and analysis of data, the following conclusions are believed reasonable:

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[4]Vegetation will rapidly cover the nutrient rich dredged soil placed in the CDF. However, there is evidence that, in spite of water removal by transpiration, it may be more advantageous to remove surface vegetation to maximize water evaporation as the result of radiant energy absorption from the sun and the drying effect of air circulation (Brutsaert, 1988).

a. Soils within the CDF are typically fine grained, high plasticity soils of high compressibility, probably containing montmorillonite. However, lower plasticity materials are present within the site with some layers of soil containing as much as 8 percent sand by dry weight.

b. Soils not in the vicinity of the weir have water contents measured to be at or near the liquid limit.

c. Insitu measured strength of soils not in the vicinity of the weir is fairly substantial, indicating that the materials are dry and the site has received deliberate management and attention.

d. Piezometer measurements indicate little, if any, excess pore water pressure at locations not in the vicinity of the weir.

e. Consolidation analysis indicates that negligible storage capacity is gained by dissipating excess pore water pressure that exists within the soil mass.

f. Surcharging the site with 5 PSI will produce consolidation of the order of 1 foot. Time required for this settlement is about 70 years but can be reduced to 6 years by installation of vertical strip drains.

g. The cost of installing vertical strip drains in a 12 foot triangular pattern over the 220-acre CDF for the purpose of accelerating consolidation varies from about \$500,000 to \$1,000,000.

h. The use of electro-osmosis to produce consolidation in the CDF is inefficient because of relatively high conductivity (1100 to 1800 μmhos) of the pore water in soils throughout the Toledo CDF. Fairly large scale laboratory tests indicate that modest dewatering is achieved for high energy expenditures. On the basis of cost and efficiency, the procedure is not recommended for use under conditions where pore water conductivity is more than 200 to 300 μmhos.

i. Underneath a dry crust that was measured to be from 8 to 12 in. in thickness, soil in the vicinity of the weir (Site 3) was observed to be well above the liquid limit and therefore in a fluid state. A crust, once formed at the surface of a CDF, prevents further soil drying with the result that soil underneath a surface crust will remain in a high water content, low strength, fluid state indefinitely.

13 Recommendations for Cell Management

Based on the results of, and conclusions drawn from the studied described above to assess the potential for consolidation, recovery, and addition of storage capacity at Cell # 1 and Cell #2 of the Toledo CDF, the following recommendations are made for the Toledo long term management study:

a. Decant water should be expeditiously drained from the cells as soon as practicable after slurry placement in the cell and sedimentation has occurred. Maintaining a dry soil surface will allow sun and wind to remove the maximum amount of water.

b. Growth/accumulation of vegetation on the surface of the cells should be removed from existing Cell #1, and prevented from occurring in the new Cell #2. Although the presence of vegetation promotes the removal of some water by transpiration, a greater amount of water is removed by radiant energy from the sun and the circulation of air at the surface. A canopy of vegetation over and on the surface of a CDF prevents penetration of sunlight, air circulation, and impedes surface water runoff.

c. Use perimeter and/or internal cell trenching to allow gravity drainage of dredged materials in both Cell #1 as well as the new Cell #2. Runoff from the trenches must be directed to a weir for expeditious removal from the sites. The efficiency of gravity drainage increases as the depth of trenches increases, however, the practical maximum depth of trenches is limited by the strength of the soil in which the trenches are placed; that is, if the depth of the trenches is too great, the trenches will collapse and close under their own weight. Because strength and consistency properties of materials in the site are unpredictable, the maximum practical depth of trenches must be determined by experience. It is highly likely that low ground pressure equipment will be required to operate on newly placed dredged material after it has been decanted and allowed to dry by exposure to sun and wind. Perimeter and internal cell trenching is highly recommended for the new Cell #2 because it is extremely cost effective in that trenching allows consolidation of newly placed material and provides drainage and overall dewatering of the cell.

d. Disking may be used to facilitate soil drying by exposing material at depth to the drying influences of sun and air flow. However, after disking and drying, it will be necessary to, a) compact the disturbed soil with a rubber tired roller or sheeps-foot roller (to prevent re-wetting and deep water percolation), or b) remove it from the cell for beneficial use. On-site beneficial use of recovered material for the construction of cross-dikes is highly desirable because it increases cell storage while decreasing site management costs. The use of recovered soil to raise perimeter dikes or cross dikes is one viable and practical means of increasing CDF storage, however, stability analyses (that are beyond the scope of this report) must be performed to ensure the structural stability of the CDF dikes. Such analyses must be based on the measured strength of underlying material.

e. Electro-osmosis is not recommended at the Toledo CDF Cell #1 because relatively high (electrical) conductivity of water in the soil makes the process unacceptably inefficient. The amount of storage recoverable from consolidation by electro-osmosis is too small to justify the high cost.

f. Installation of vertical strip drains to produce consolidation and recovery of storage is not recommended at the Toledo CDF because too little excess pore water pressure exists within the existing Cell #1 to justify the high cost of vertical strip drains against the small amount of storage recoverable from consolidation, even if the area is surcharged.

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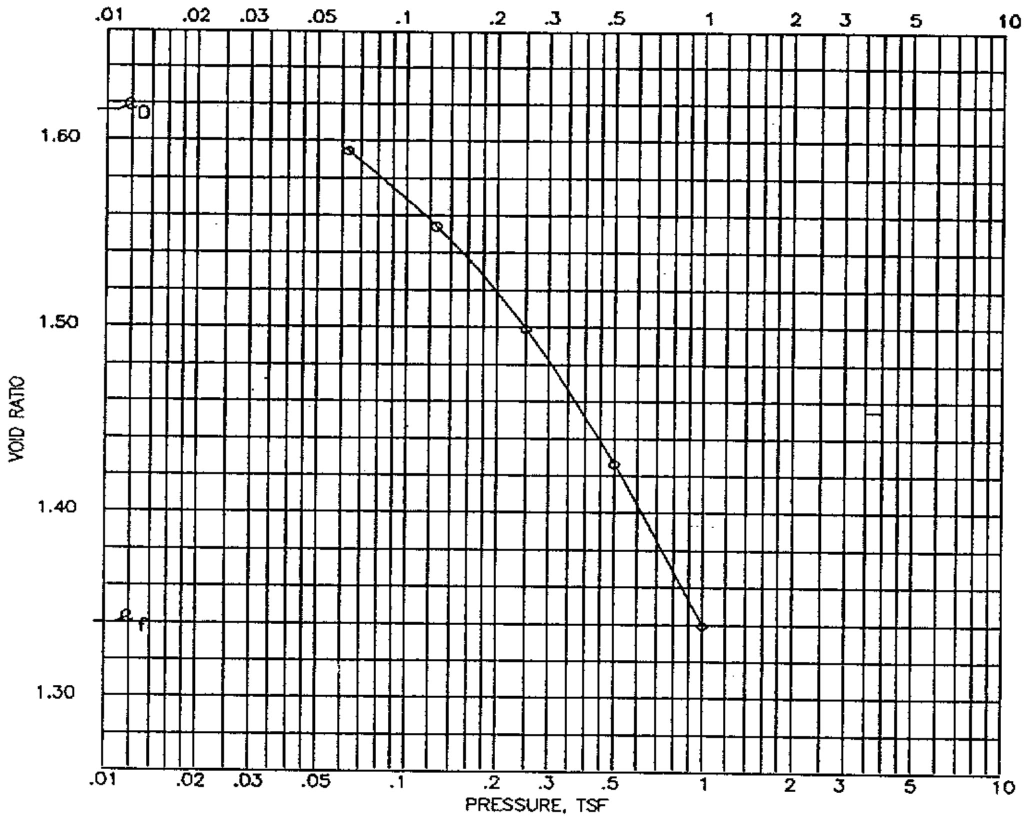
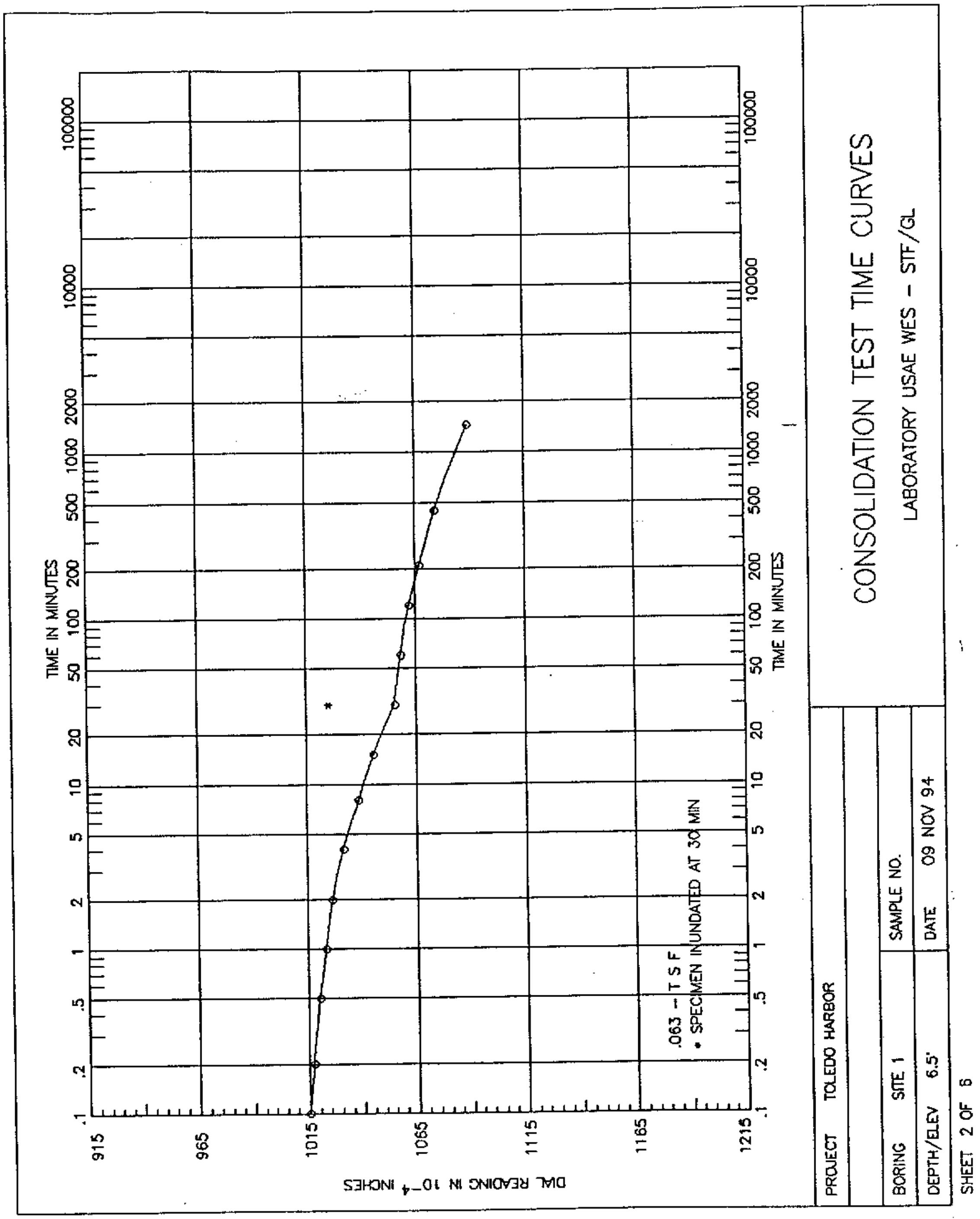
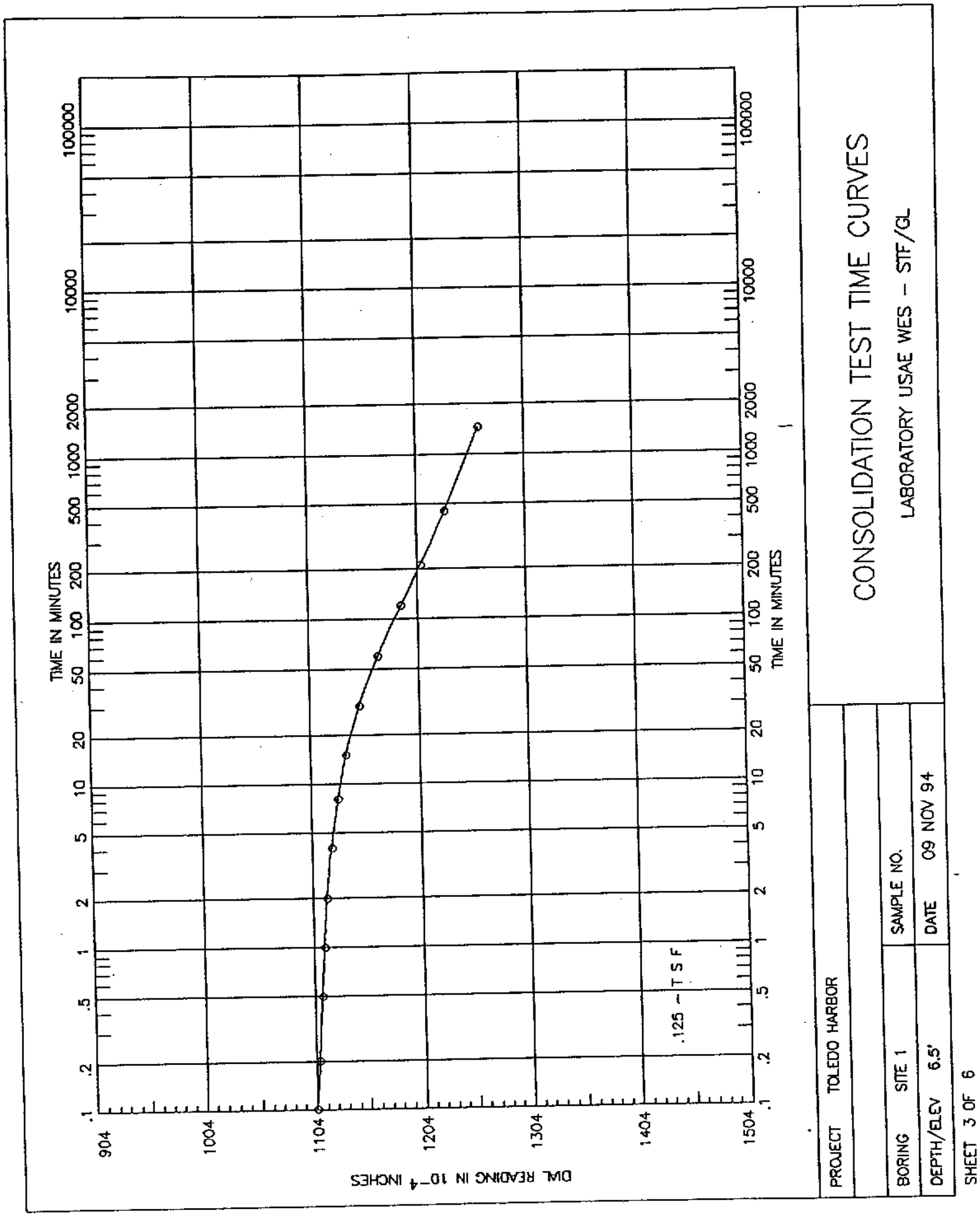
Appendix A
Oedometer Tests



				BEFORE TEST	AFTER TEST
OVERBURDEN PRESSURE, TSF			WATER CONTENT, %	56.8	48.0
PRECONSOL. PRESSURE, TSF			DRY DENSITY, PCF	64.5	72.1
COMPRESSION INDEX			SATURATION, %	95.0	96.8
TYPE SPECIMEN	UNDISTURBED		VOID RATIO	1.615	1.339
DIA. IN 4.44	HT. IN 1.120		BACK PRESSURE, TSF		
CLASSIFICATION DREDGE MATERIAL					
LL	PL	PI	PROJECT TOLEDO HARBOR		
GS 2.70 (EST)	D_{10}				
REMARKS:			BORING NO. SITE 1		SAMPLE NO.
			DEPTH/ELEV 6.5'		TECH. US
			LABORATORY USAE WES – STF/GL		DATE 09 NOV 94
			CONSOLIDATION TEST REPORT		

SHEET 1 OF 6


CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES - STF/GL
TIME IN MINUTES
DIAL READING IN 10^-4 INCHES
.063 - T S F
• SPECIMEN INUNDATED AT 30 MIN
PROJECT TOLEDO HARBOR
BORING SITE 1
SAMPLE NO.
DEPTH/ELEV 6.5'
DATE 09 NOV 94
SHEET 2 OF 6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LABORATORY USAE WES - STF/GL
TIME IN MINUTES
DIAL READING IN 10^{-4} INCHES
.125 - T S F
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1004
1104
1204
1304
1404
1504
.1
.2
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1
2
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50
100
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BORING SITE 1
SAMPLE NO.
DEPTH/ELEV 6.5'
DATE 09 NOV 94
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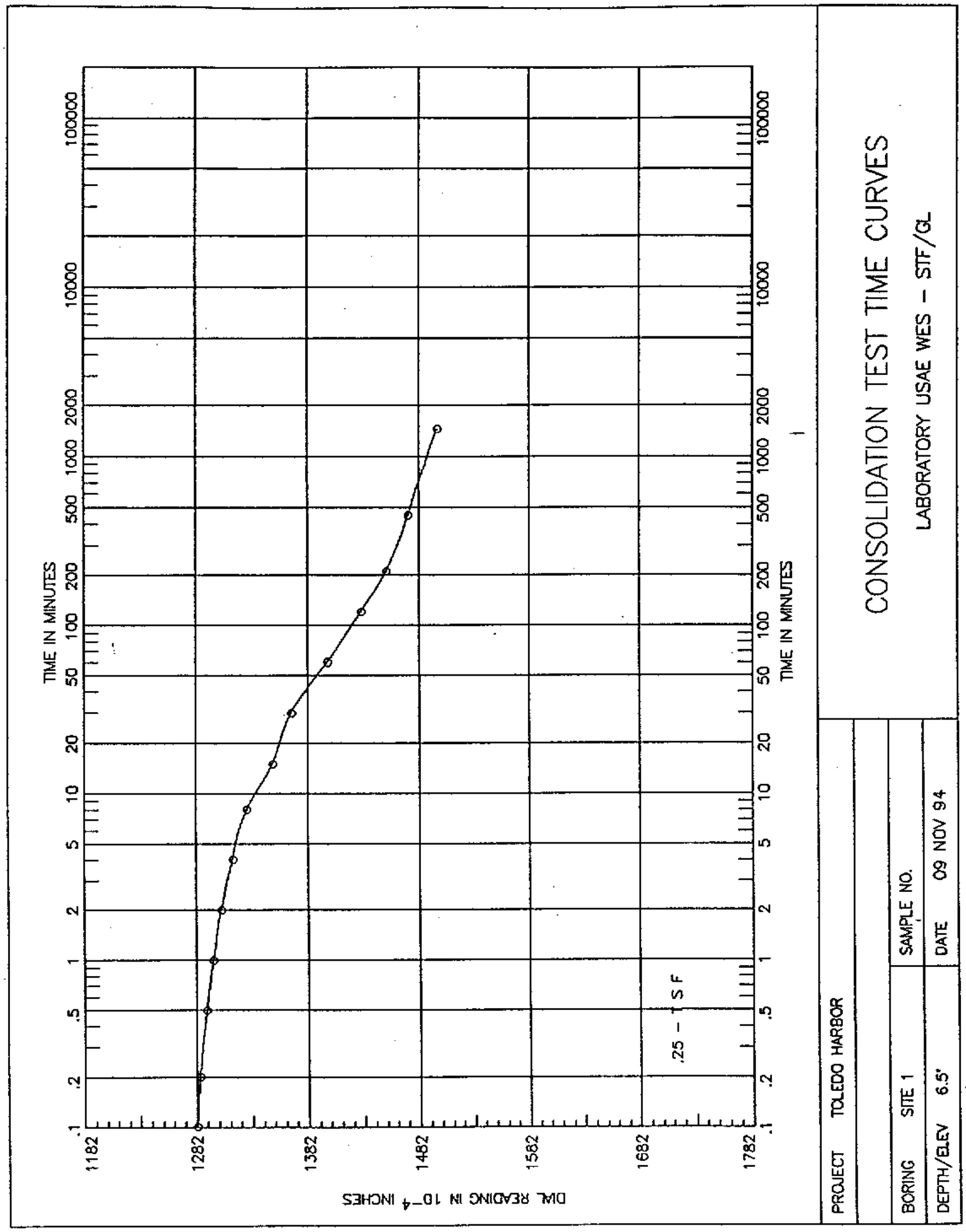
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DIAL READING IN 10^-4 INCHES
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SAMPLE NO.
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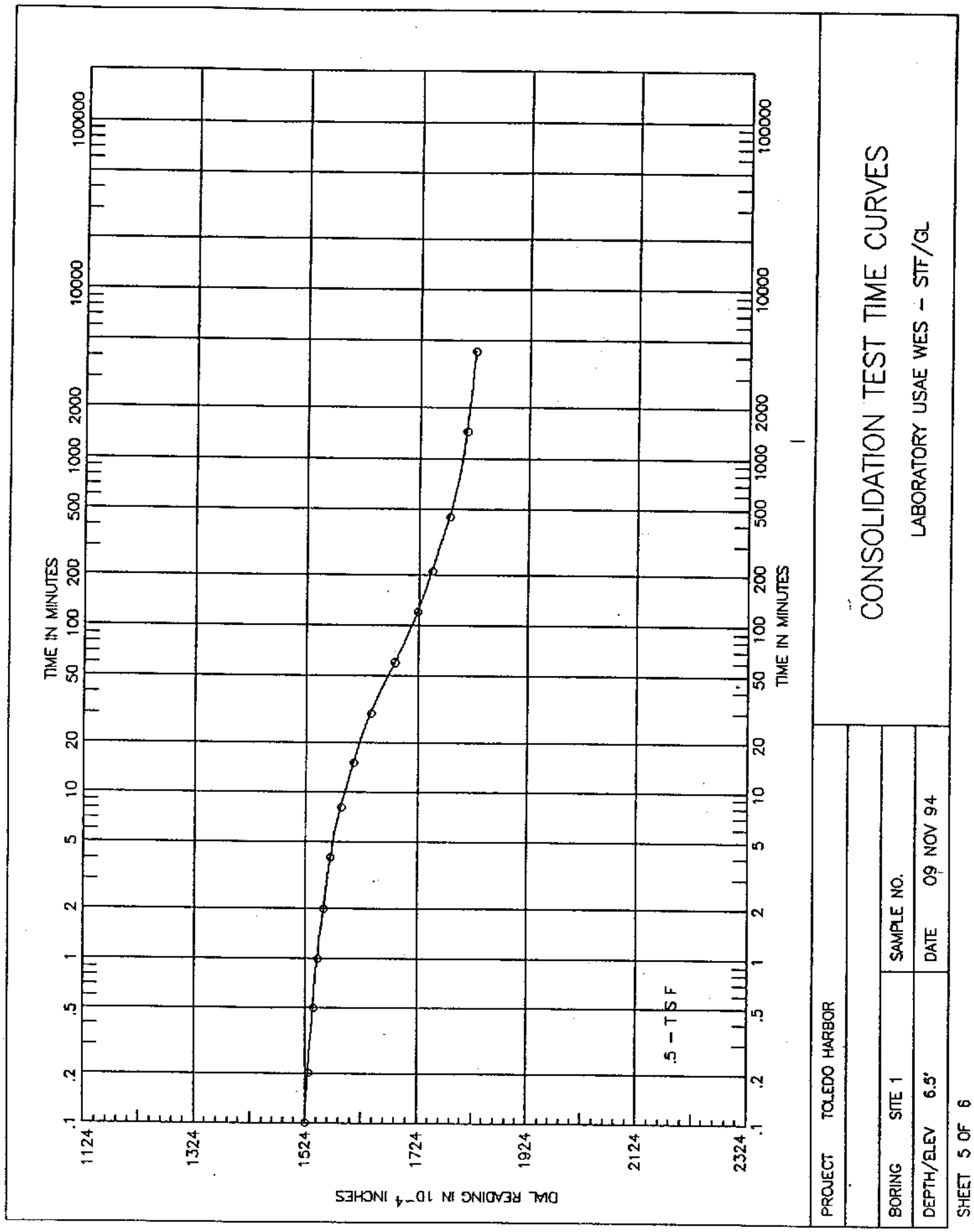
DATE 09 NOV 94
SHEET 4 OF 6



CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES - STF/GL
TIME IN MINUTES
.1 .2 .5 1 2 5 10 20 50 100 200 500 1000 2000 10000 100000
DIAL READING IN 10^-4 INCHES
1124 1324 1524 1724 1924 2124 2324
.5 - TSF
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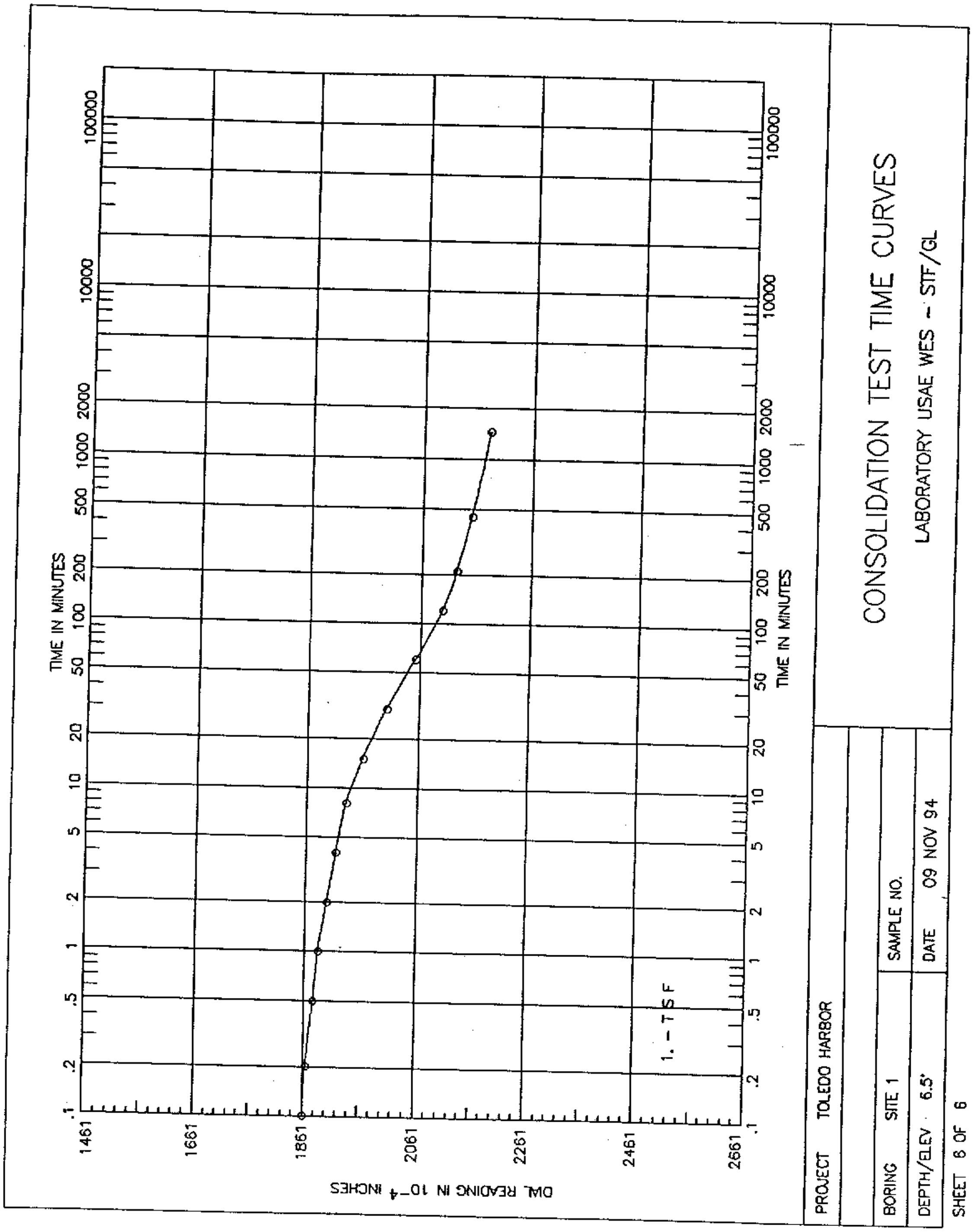
BORING SITE 1
SAMPLE NO.
DEPTH/ELEV 6.5'
DATE 09 NOV 94
SHEET 5 OF 6


CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES -- STF/GL
TIME IN MINUTES
DIAL READING IN 10^-4 INCHES
1. - TSF
.1 .2 .5 1 2 5 10 20 50 100 200 500 1000 2000 10000 100000
1461 1661 1861 2061 2261 2461 2661
PROJECT TOLEDO HARBOR
BORING SITE 1
SAMPLE NO.
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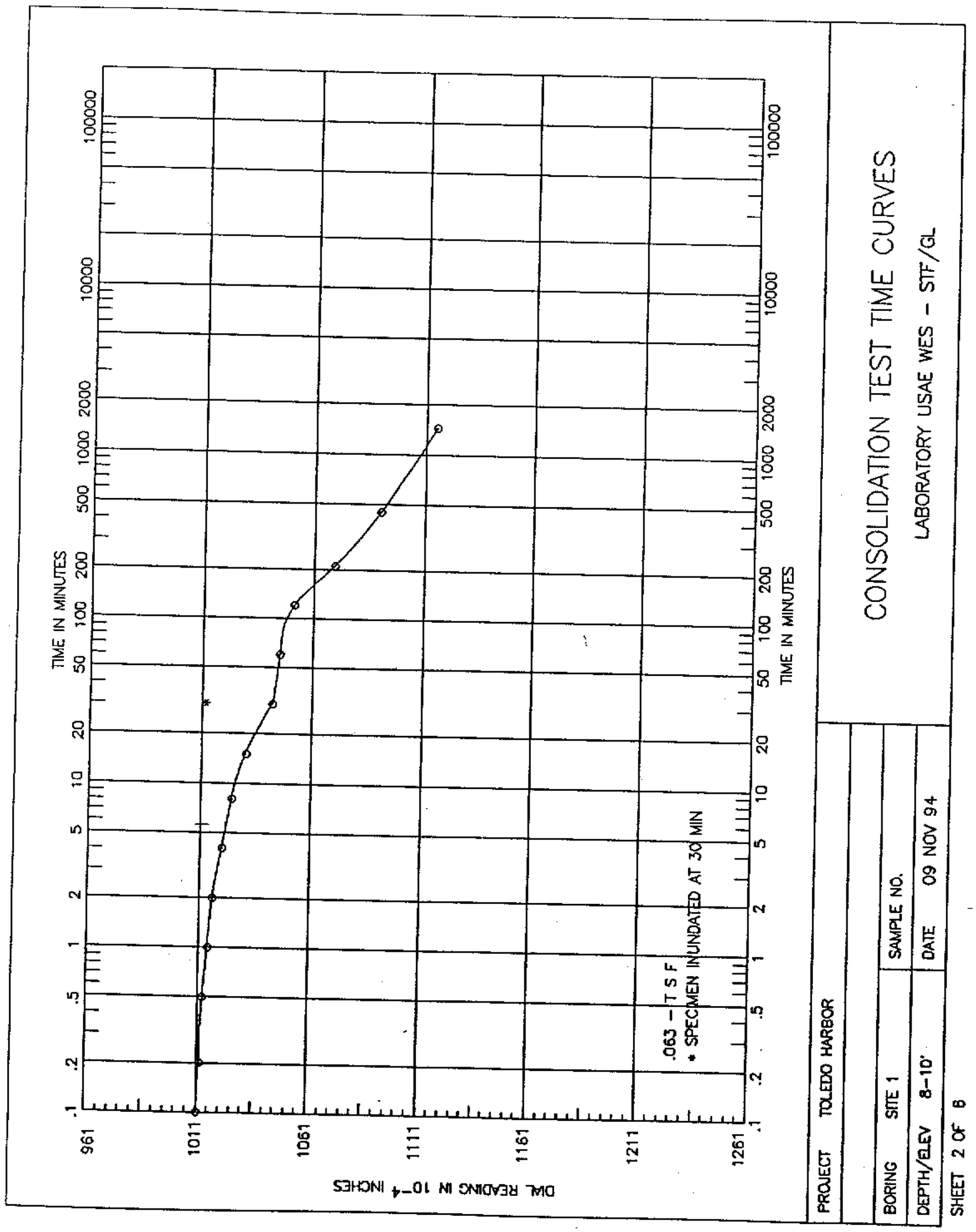
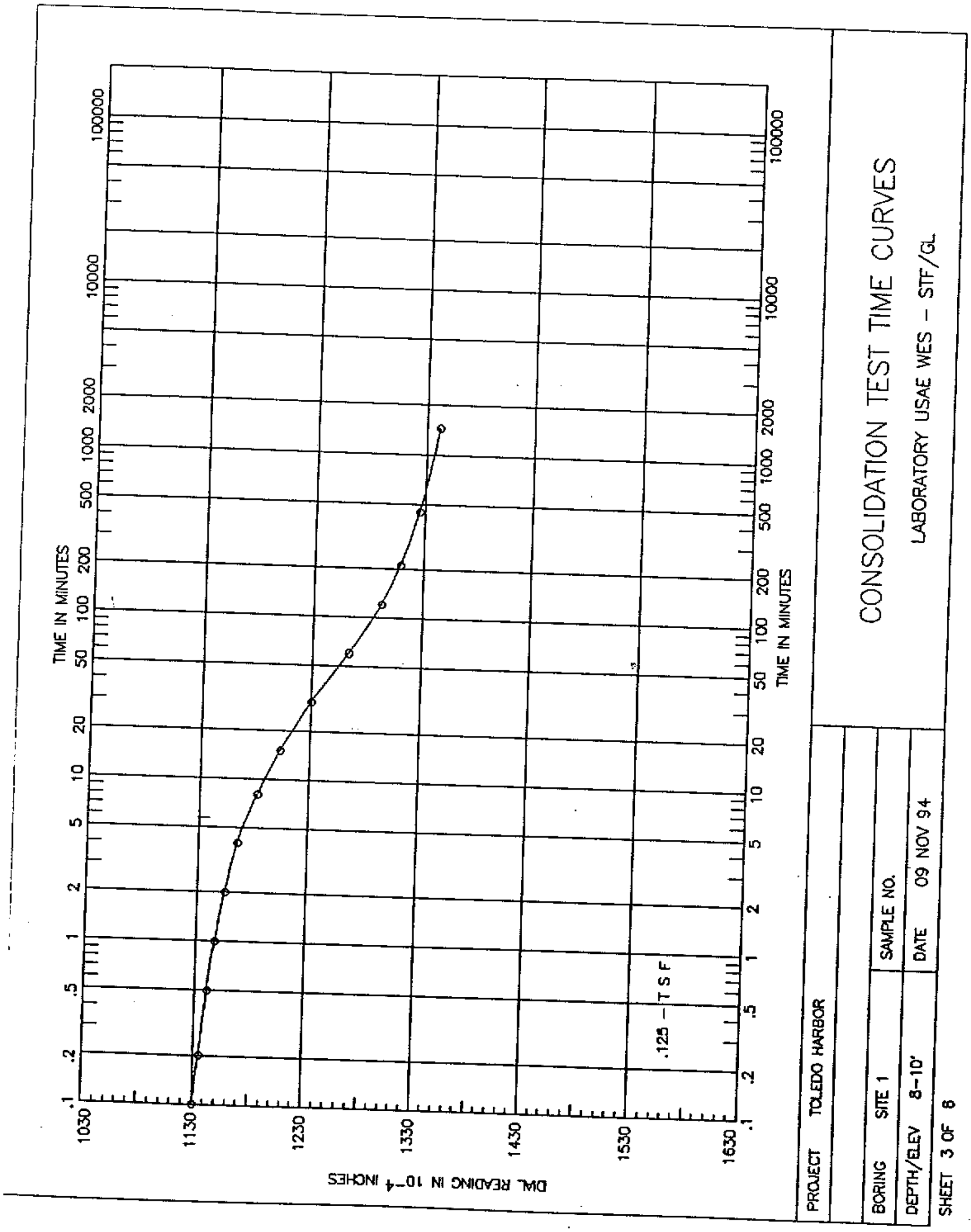
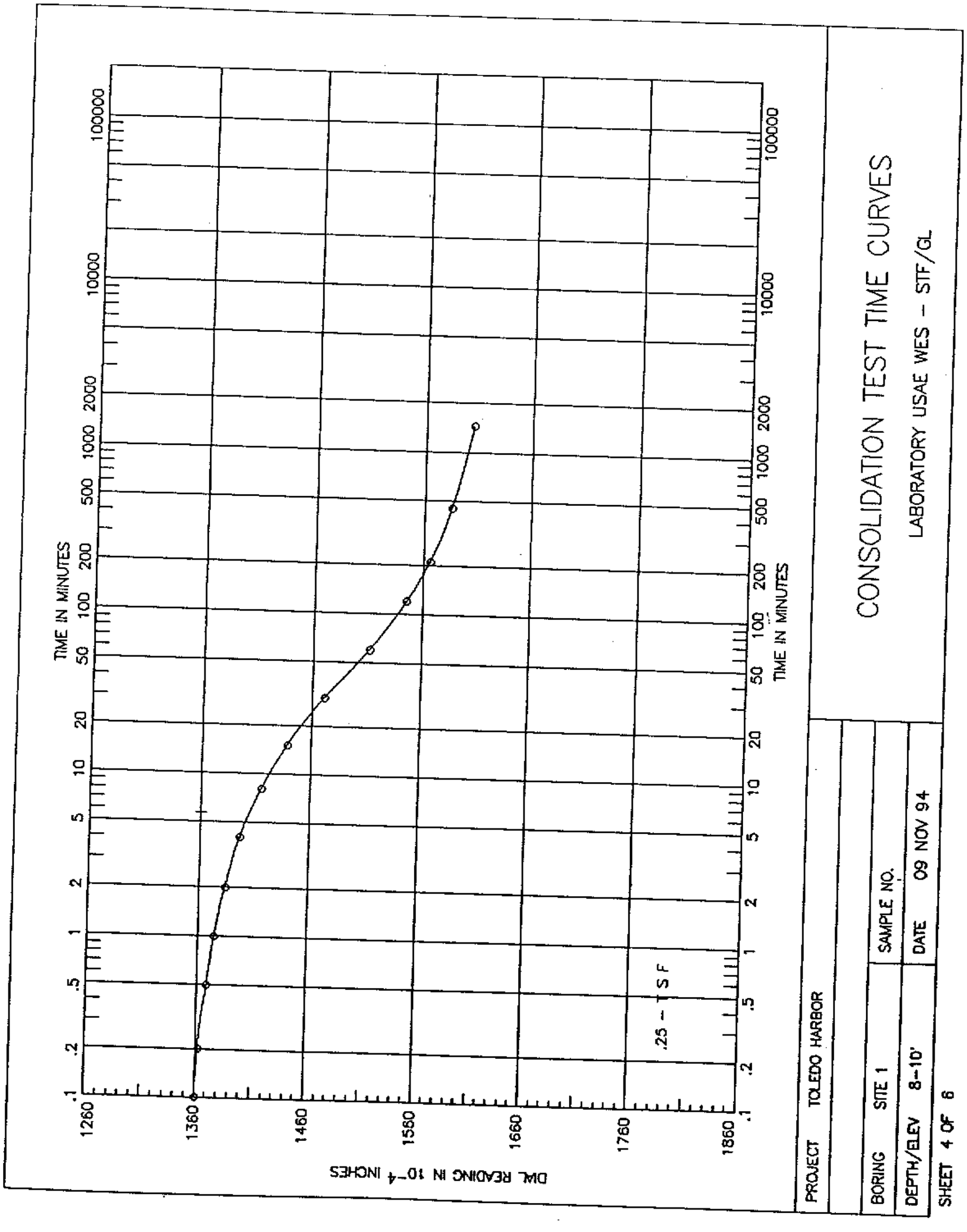
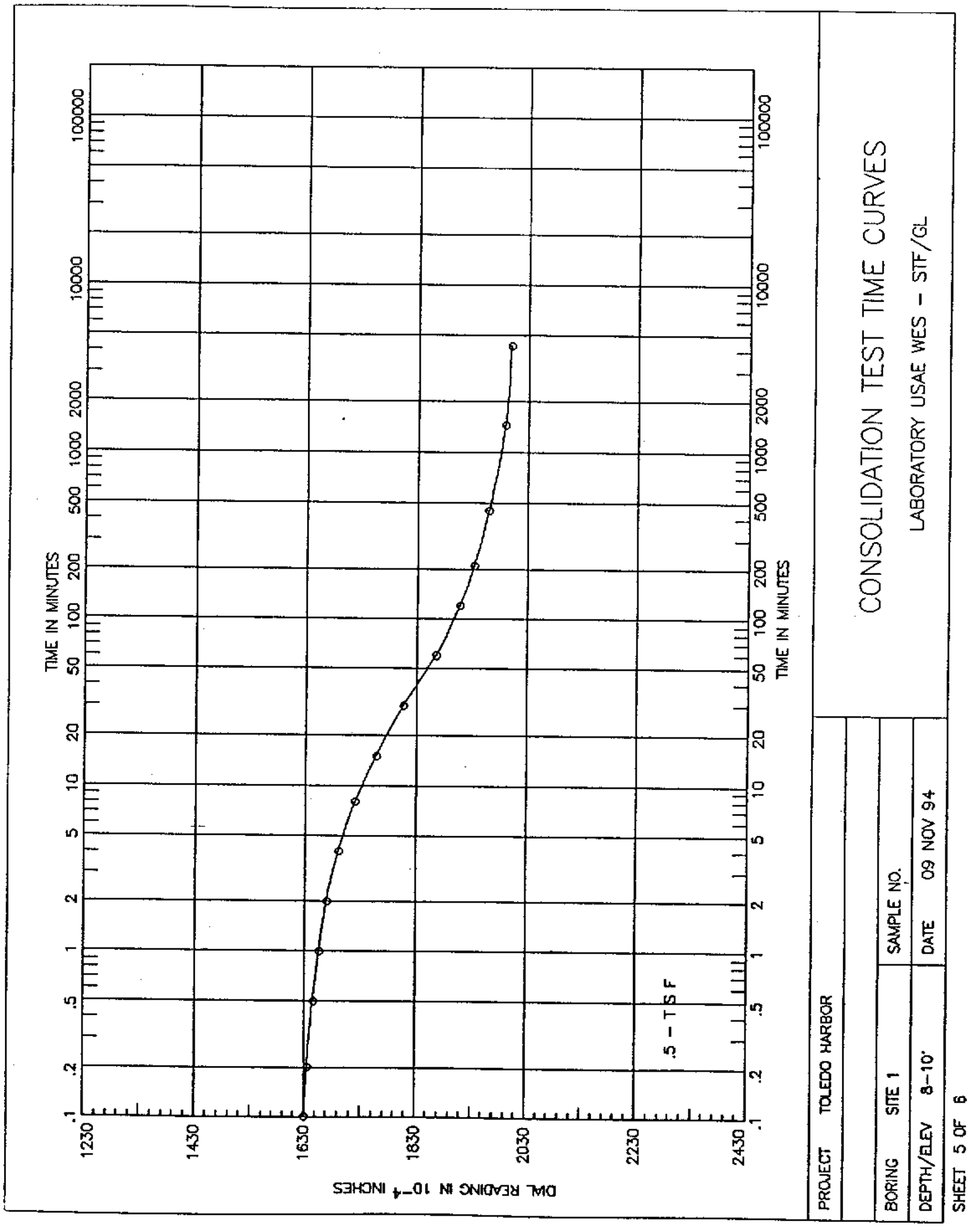
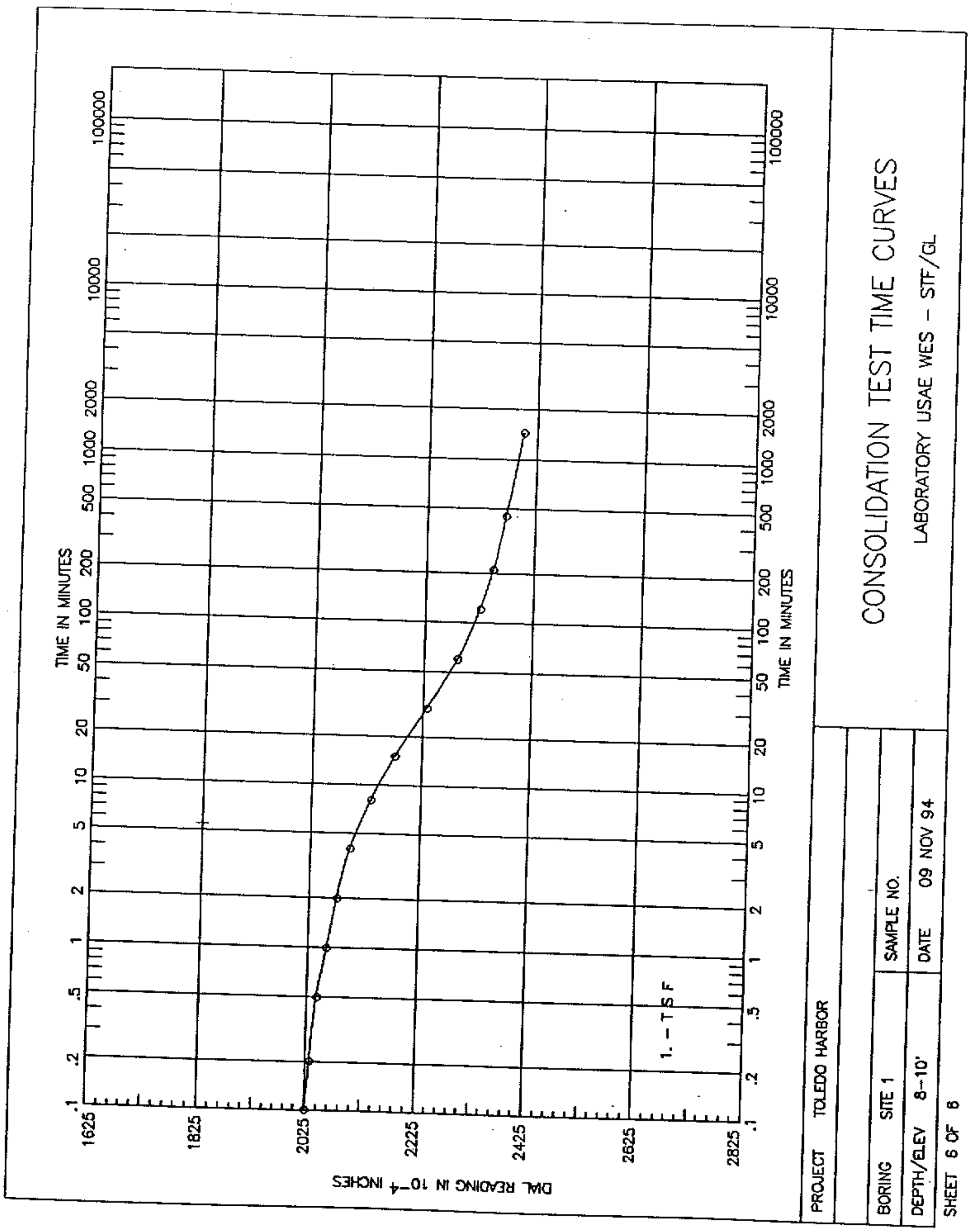
DATE 09 NOV 94
SHEET 6 OF 6


CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES - STF/GL
TIME IN MINUTES
DIAL READING IN 10-4 INCHES
.063 - T S F
* SPECIMEN INUNDATED AT 30 MIN
PROJECT TOLEDO HARBOR
BORING SITE 1
SAMPLE NO.
DEPTH/ELEV 8-10'
DATE 09 NOV 94
SHEET 2 OF 8

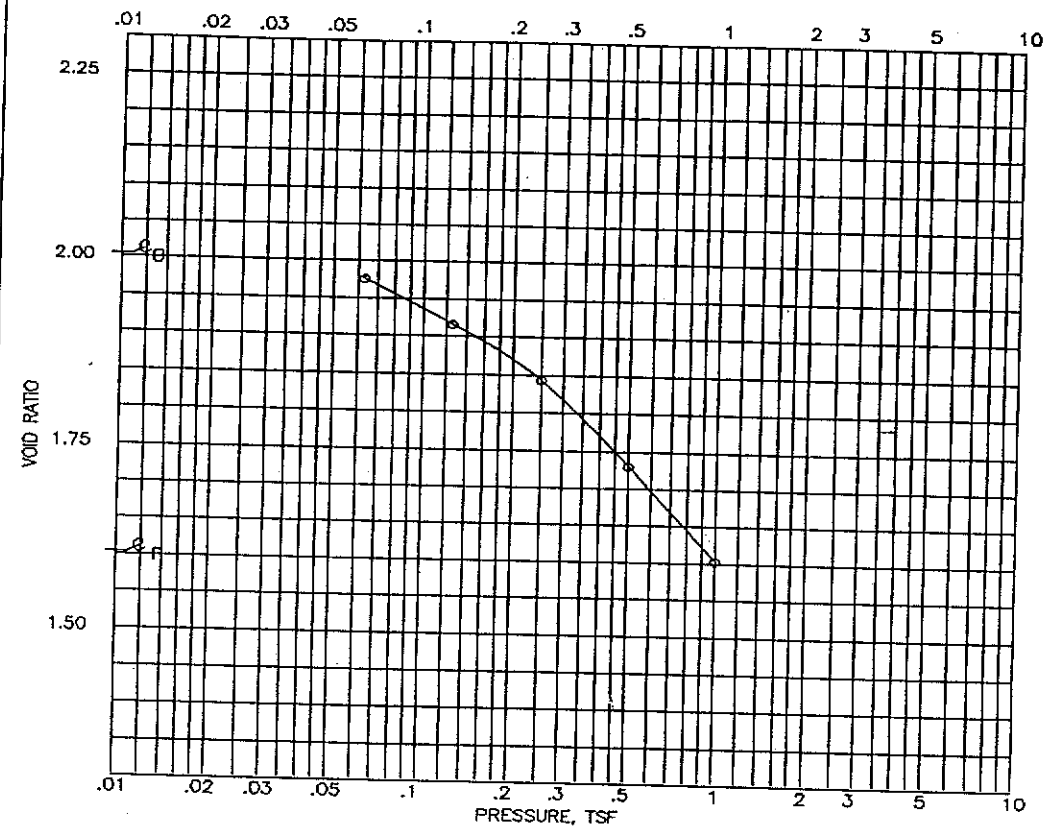

CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES - STF/GL
TIME IN MINUTES
DIAL READING IN 10^-4 INCHES
.125 - T S F
PROJECT TOLEDO HARBOR
BORING SITE 1
SAMPLE NO.
DEPTH/ELEV 8-10'
DATE 09 NOV 94
SHEET 3 OF 8


CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES - STF/GL
TIME IN MINUTES
DIAL READING IN 10^-4 INCHES
.25 - TSF
PROJECT TOLEDO HARBOR
BORING SITE 1
SAMPLE NO.
DEPTH/ELEV 8-10'
DATE 09 NOV 94
SHEET 4 OF 6


CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES - STF/GL
TIME IN MINUTES
DIAL READING IN 10⁻⁴ INCHES
.1 .2 .5 1 2 5 10 20 50 100 200 500 1000 2000 10000 100000
1230 1430 1630 1830 2030 2230 2430
.5 - T S F
PROJECT TOLEDO HARBOR
BORING SITE 1
SAMPLE NO.
DEPTH/ELEV 8-10'
DATE 09 NOV 94
SHEET 5 OF 6



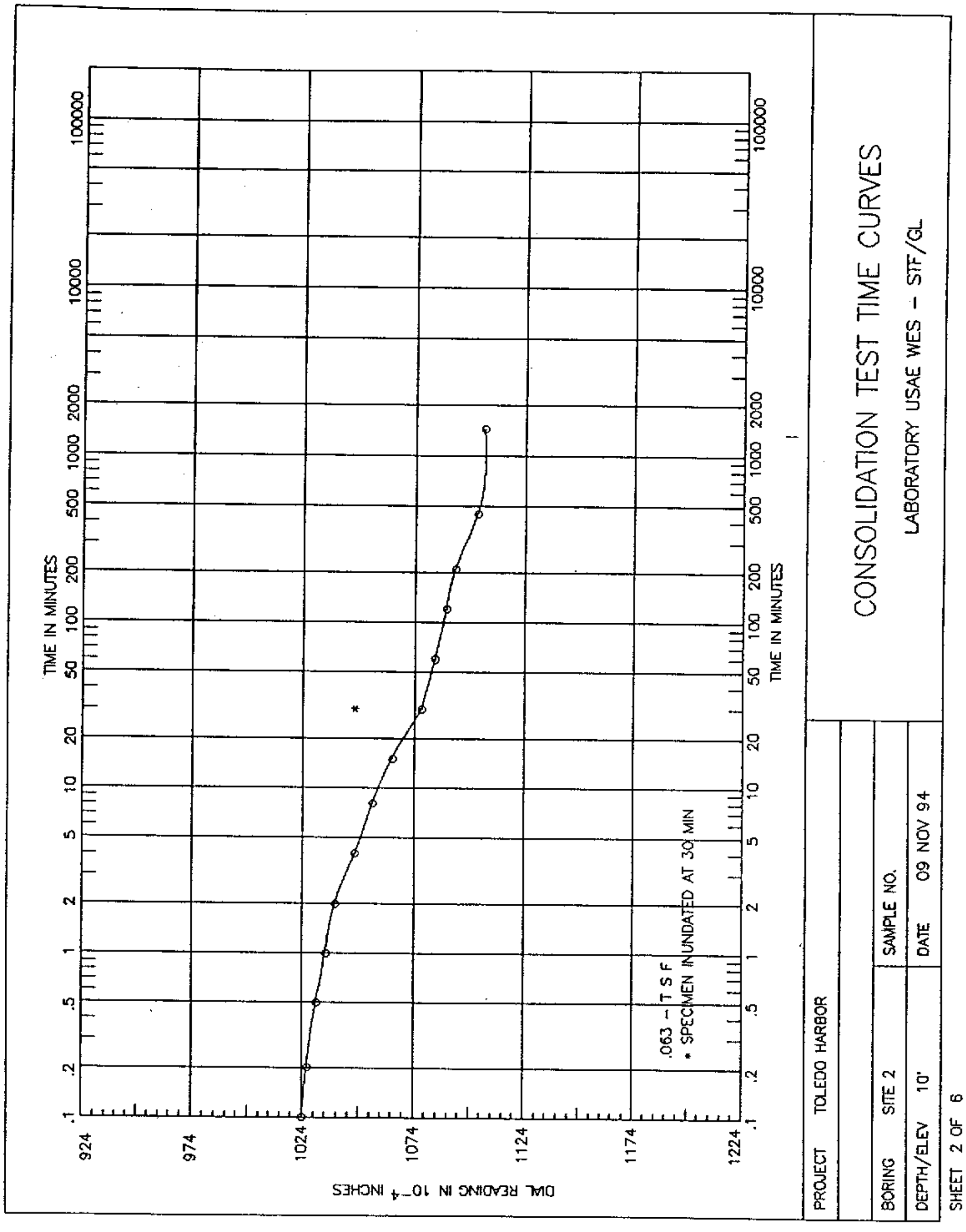
CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES - STF/GL
TIME IN MINUTES
DIAL READING IN 10^-4 INCHES
1. - T S F
PROJECT TOLEDO HARBOR
BORING SITE 1
SAMPLE NO.
DEPTH/ELEV 8-10'
DATE 09 NOV 94
SHEET 6 OF 8



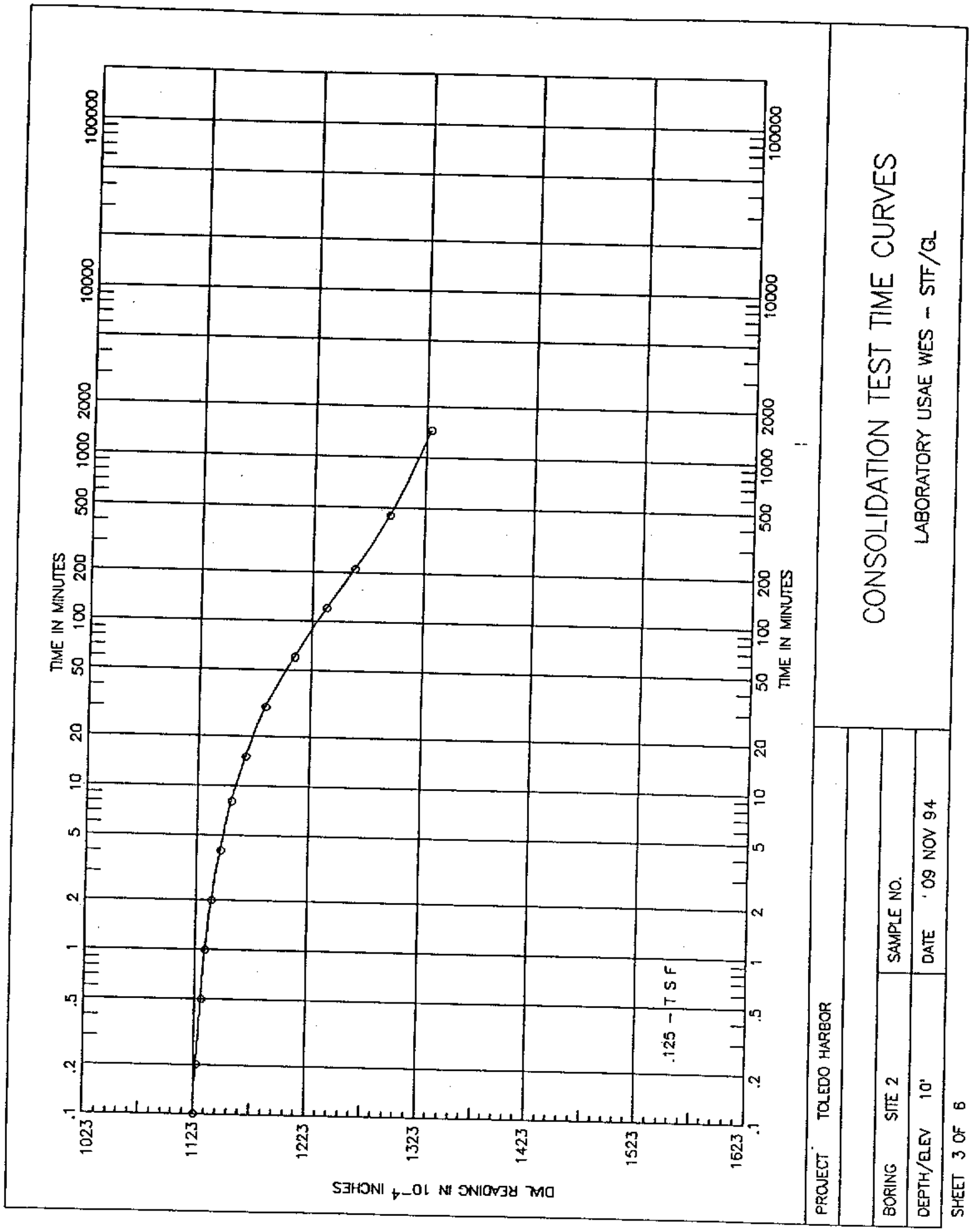
		BEFORE TEST	AFTER TEST
OVERBURDEN PRESSURE, TSF		WATER CONTENT, %	70.3	56.8
PRECONSOL. PRESSURE, TSF		DRY DENSITY, PCF	56.1	64.8
COMPRESSION INDEX		SATURATION, %	94.7	95.8
TYPE SPECIMEN	UNDISTURBED	VOID RATIO	2.003	1.601
DIA. IN 4.44	HT. IN 1.117	BACK PRESSURE, TSF		

CLASSIFICATION DREDGE MATERIAL			
LL	PL	PI	PROJECT TOLEDO HARBOR
GS 2.70 (EST)	D_{10}		
REMARKS:		BORING NO. SITE 2	SAMPLE NO.
		DEPTH/ELEV 10'	TECH. US
		LABORATORY USAE WES – STF/GL	DATE 09 NOV 94
		CONSOLIDATION TEST REPORT	

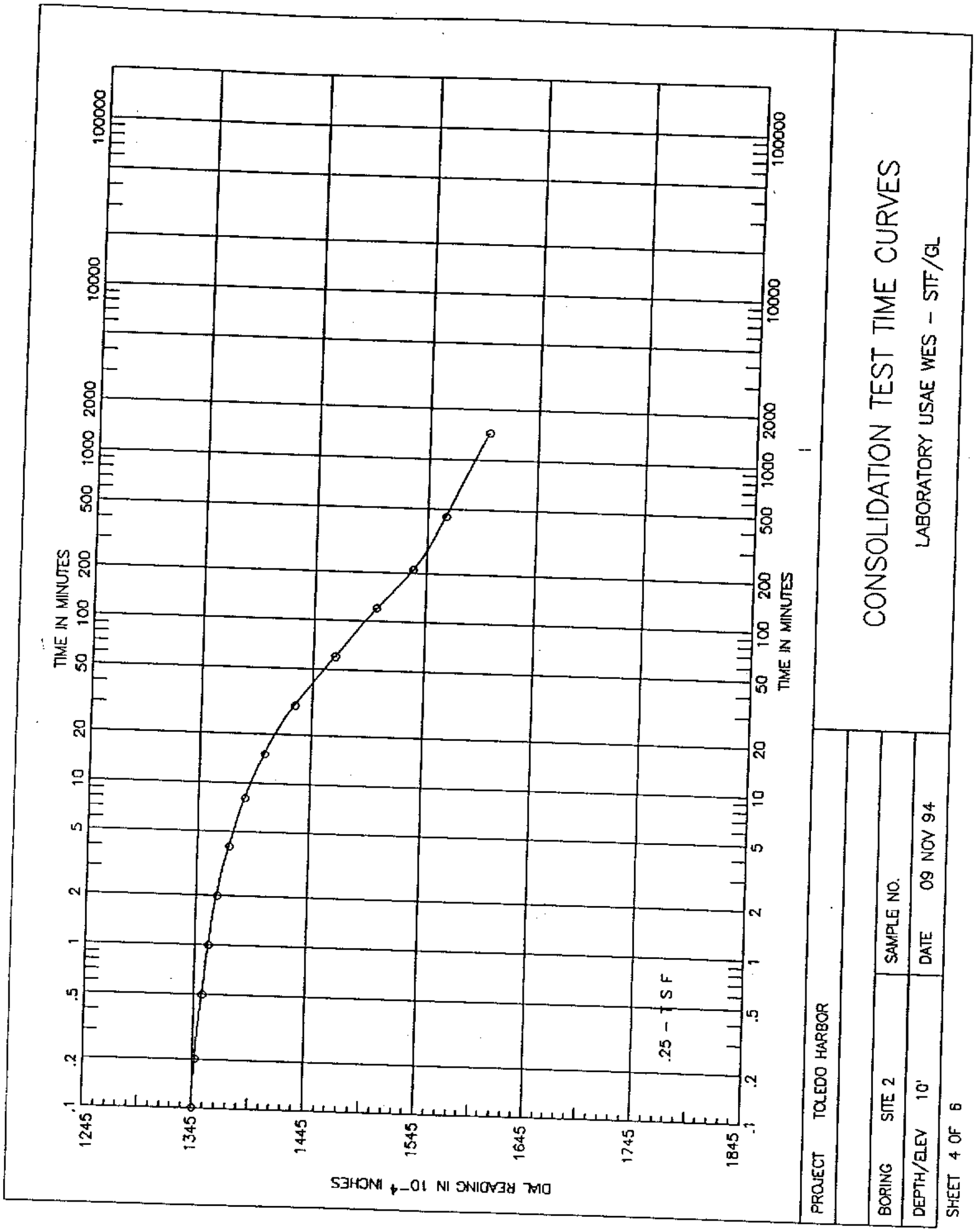
SHEET 1 OF 6

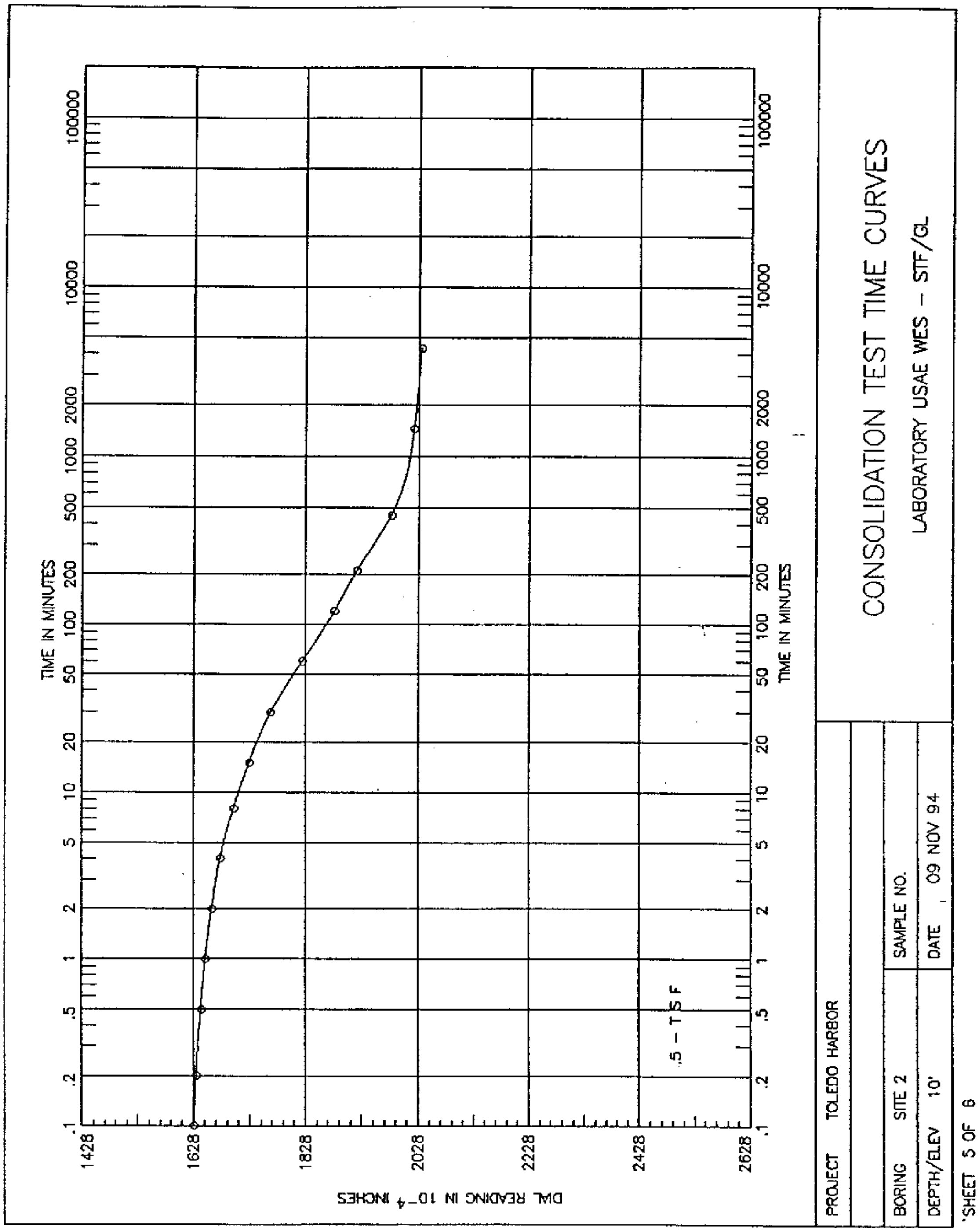


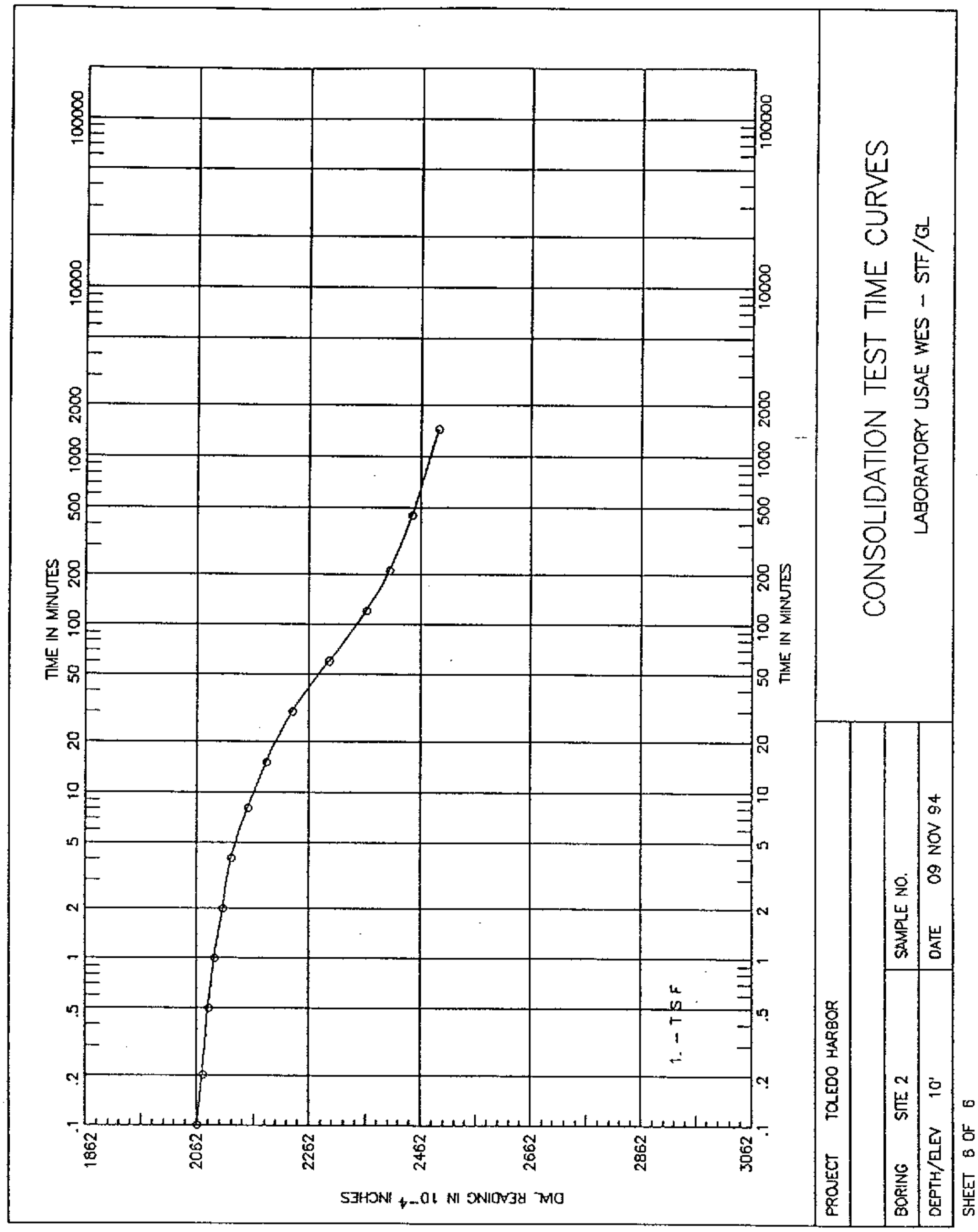
CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES - STF/GL
.063 - T S F
* SPECIMEN INUNDATED AT 30 MIN
TIME IN MINUTES
.1
.2
.5
1
2
5
10
20
50
100
200
500
1000
2000
10000
100000
DIAL READING IN 10^-4 INCHES
924
974
1024
1074
1124
1174
1224
PROJECT TOLEDO HARBOR
BORING SITE 2
SAMPLE NO.
DEPTH/ELEV 10'
DATE 09 NOV 94
SHEET 2 OF 6

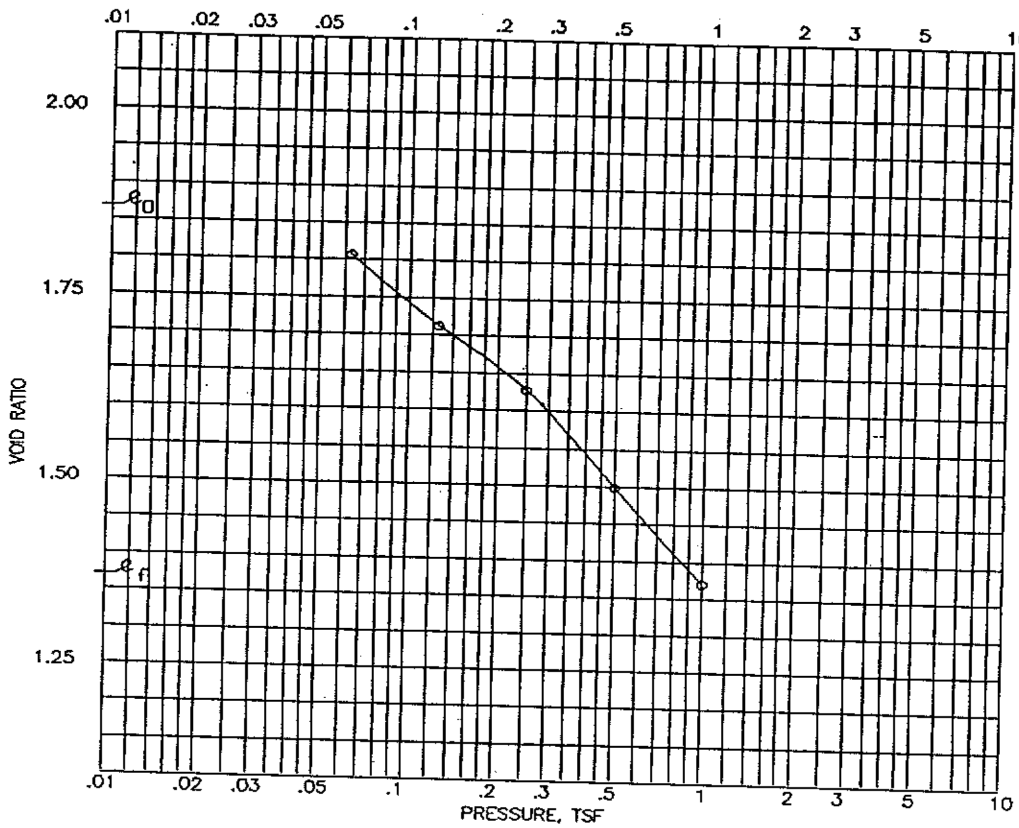


CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES - STF/GL
TIME IN MINUTES
.1 .2 .5 1 2 5 10 20 50 100 200 500 1000 2000 10000 100000
DIAL READING IN 10^-4 INCHES
1023 1123 1223 1323 1423 1523 1623
.125 - T S F
PROJECT TOLEDO HARBOR
BORING SITE 2
SAMPLE NO.
DEPTH/ELEV 10'
DATE '09 NOV 94
SHEET 3 OF 6


CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES - STF/GL
TIME IN MINUTES
DIAL READING IN 10⁻⁴ INCHES
.25 - 1 S F
PROJECT TOLEDO HARBOR
BORING SITE 2
SAMPLE NO.
DEPTH/ELEV 10'
DATE 09 NOV 94
SHEET 4 OF 6

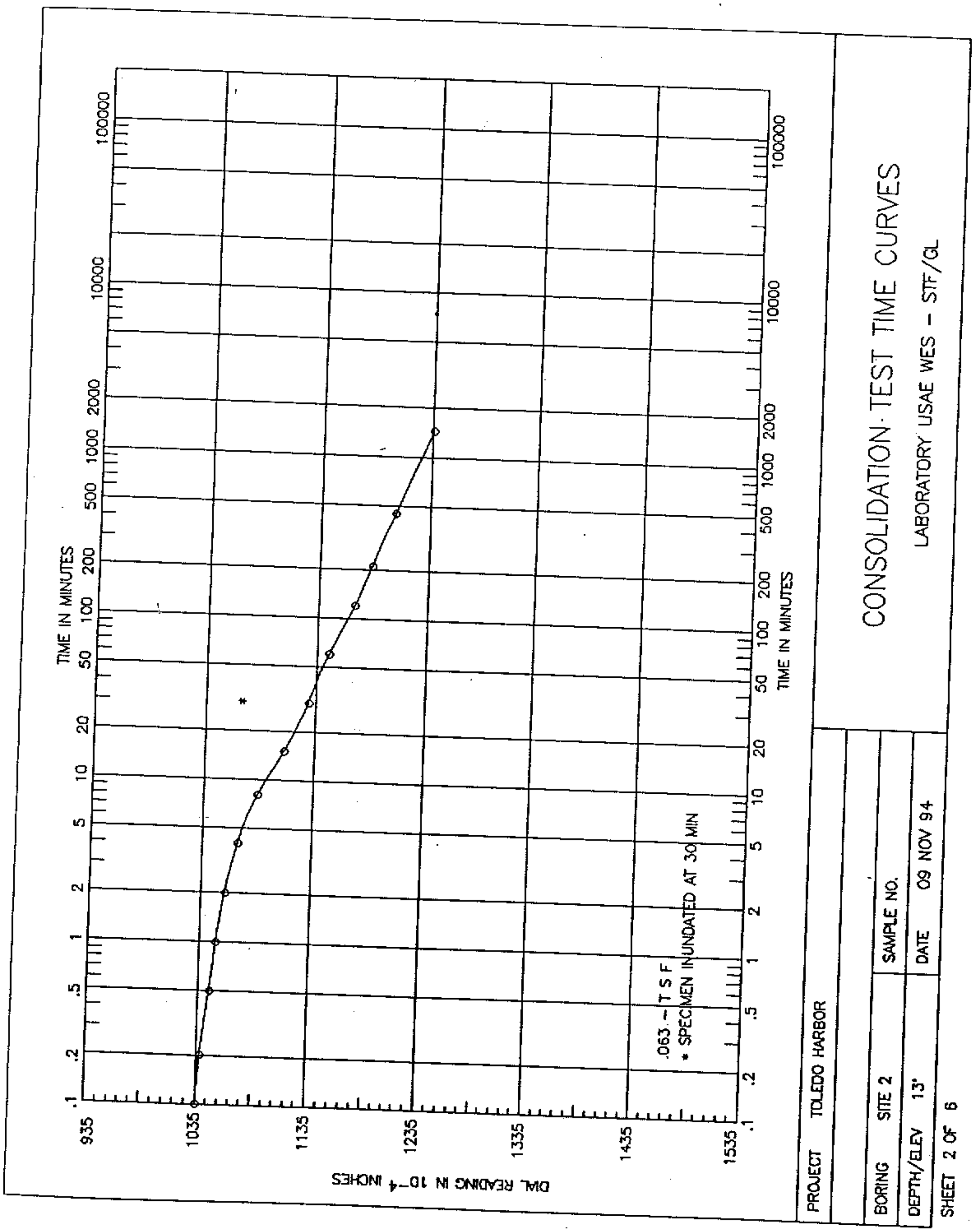

CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES - STF/GL
TIME IN MINUTES
.1 .2 .5 1 2 5 10 20 50 100 200 500 1000 2000 10000 100000
DIAL READING IN 10^{-4} INCHES
1428 1628 1828 2028 2228 2428 2628
.5 - TSF
PROJECT TOLEDO HARBOR
BORING SITE 2
SAMPLE NO.
DEPTH/ELEV 10'
DATE 09 NOV 94
SHEET 5 OF 6


CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES - STF/GL
TIME IN MINUTES
.1 .2 .5 1 2 5 10 20 50 100 200 500 1000 2000 10000 100000
DIAL READING IN 10^-4 INCHES
1862 2062 2262 2462 2662 2862 3062
1. - TSF
PROJECT TOLEDO HARBOR
BORING SITE 2
SAMPLE NO.
DEPTH/ELEV 10'
DATE 09 NOV 94
SHEET 6 OF 6

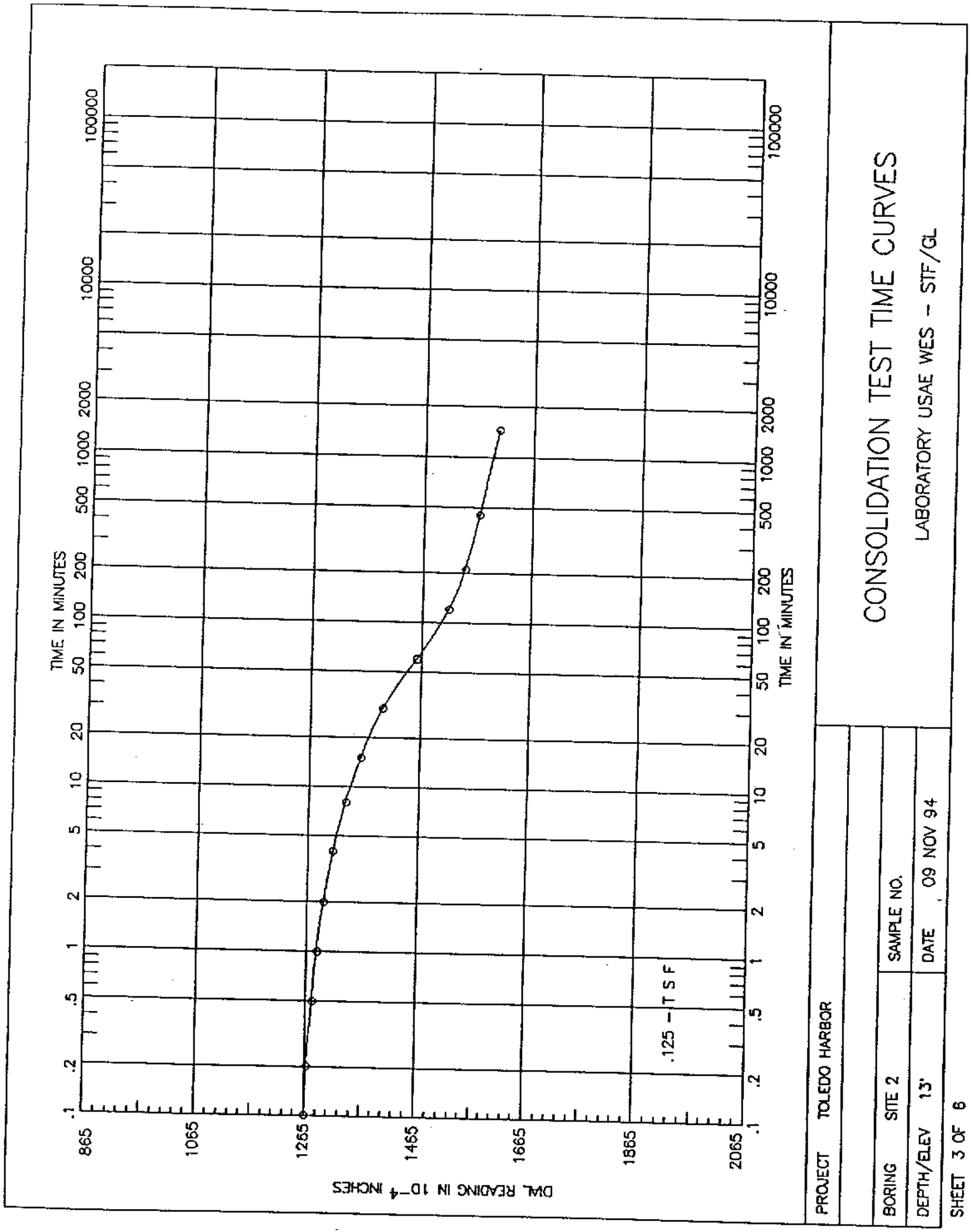


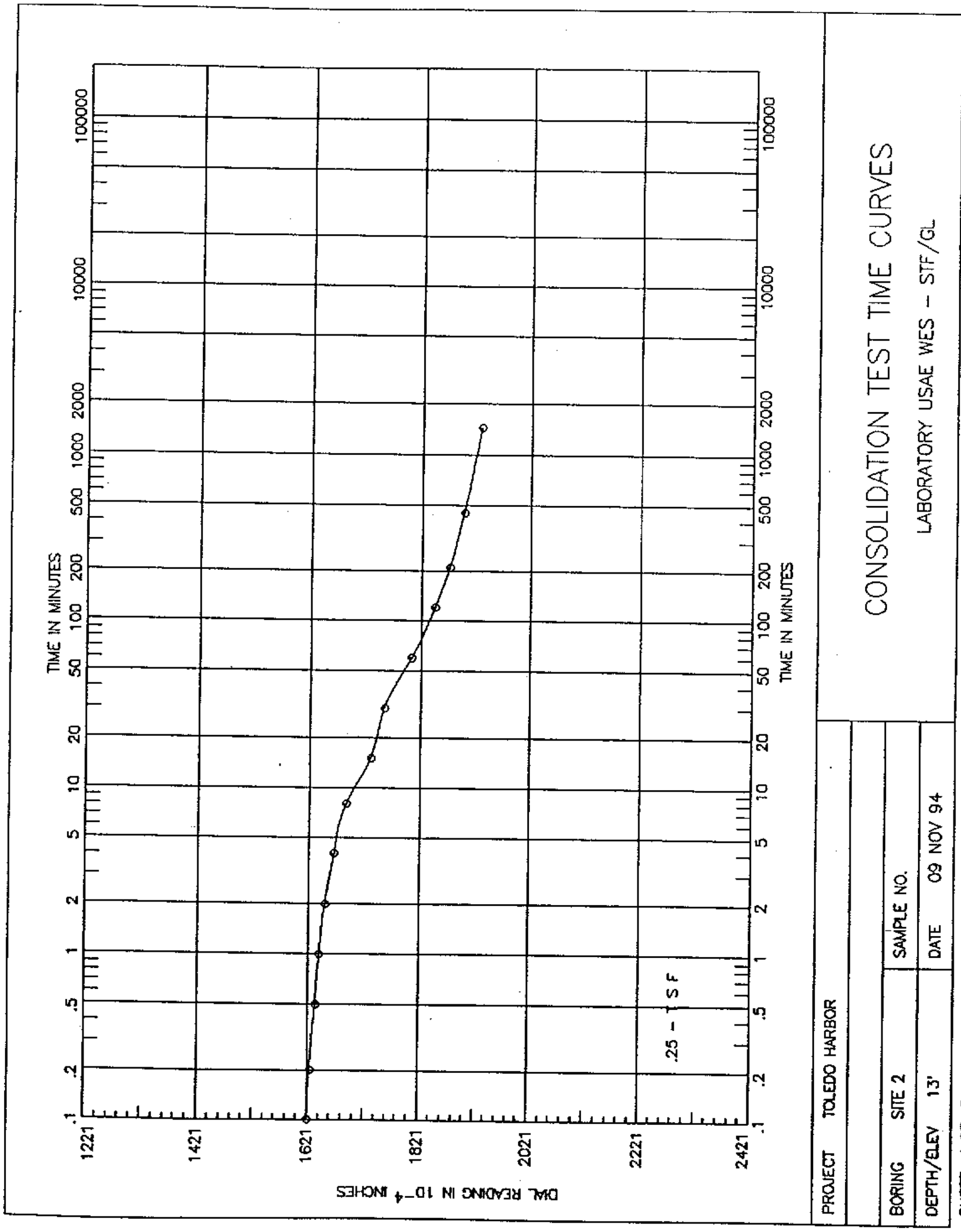
			BEFORE TEST	AFTER TEST
OVERBURDEN PRESSURE, TSF		WATER CONTENT, %	66.1	50.3
PRECONSOL. PRESSURE, TSF		DRY DENSITY, PCF	58.8	71.1
COMPRESSION INDEX		SATURATION, %	95.7	99.2
TYPE SPECIMEN	UNDISTURBED	VOID RATIO	1.867	1.369
DIA. IN 4.44	HT. IN 1.106	BACK PRESSURE, TSF		
CLASSIFICATION	DREDGE MATERIAL			
LL PL	PI	PROJECT TOLEDO HARBOR		
GS 2.70 (EST)	D_{10}			
REMARKS:		BORING NO. SITE 2	SAMPLE NO.	
		DEPTH/ELEV 13'	TECH. US	
		LABORATORY USAE WES – STF/GL	DATE 09 NOV 94	
		CONSOLIDATION TEST REPORT		

SHEET 1 OF 6

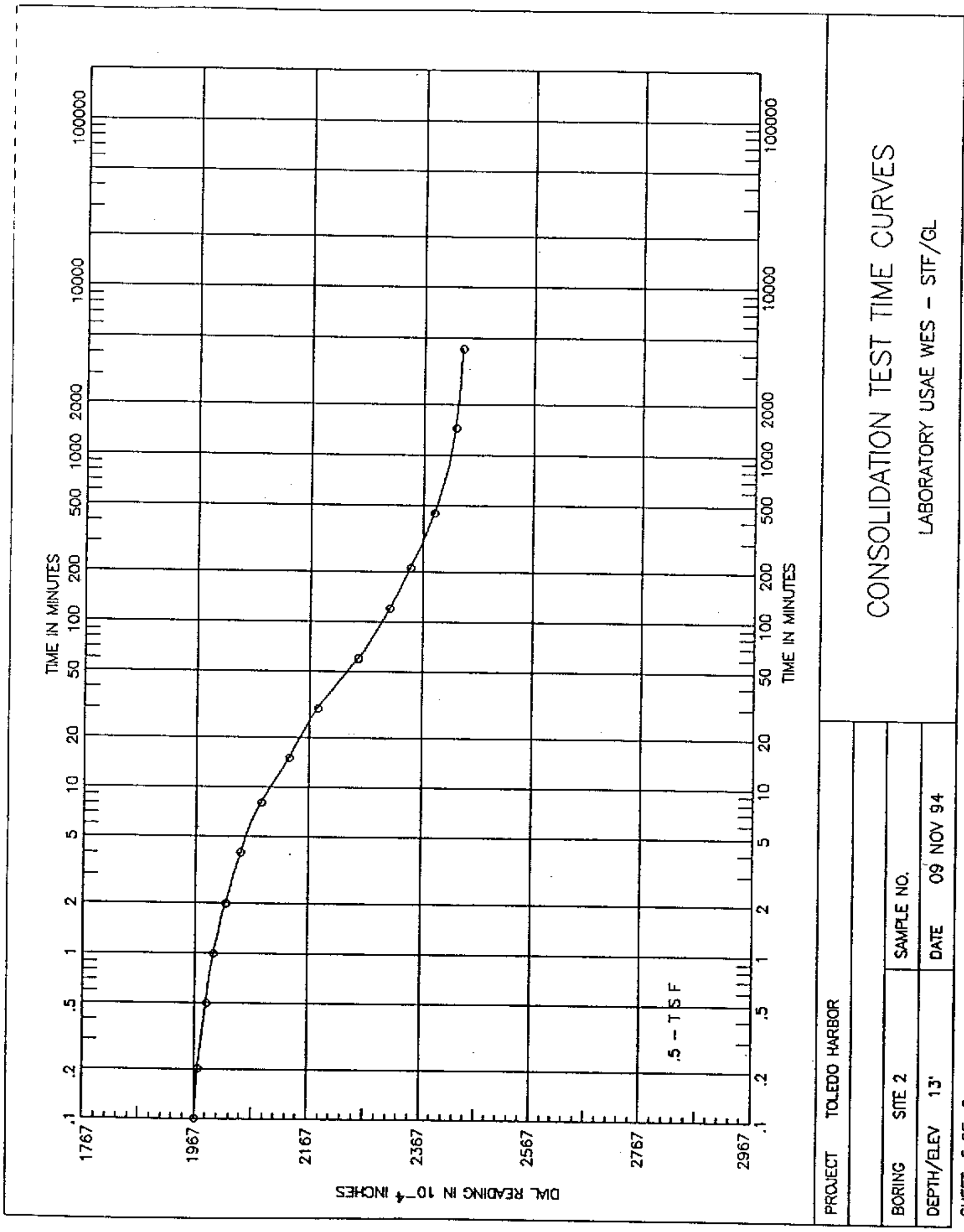


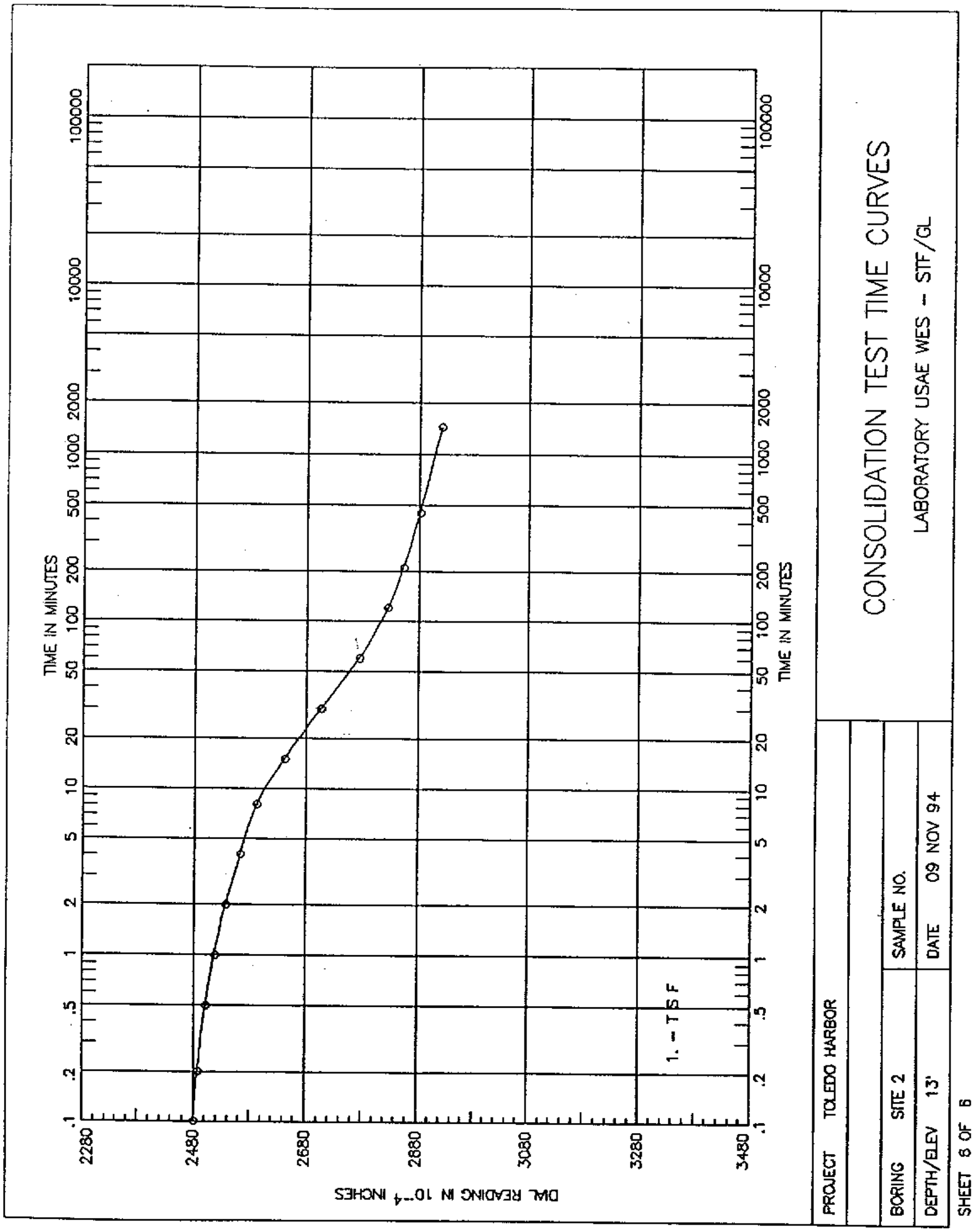
CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES - STF/GL
TIME IN MINUTES
DIAL READING IN 10^-4 INCHES
.063 -- T S F
* SPECIMEN INUNDATED AT 30 MIN
PROJECT TOLEDO HARBOR
BORING SITE 2
SAMPLE NO.
DEPTH/ELEV 13'
DATE 09 NOV 94
SHEET 2 OF 6


CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES - STF/GL
TIME IN MINUTES
DIAL READING IN 10^-4 INCHES
.125 - T S F
PROJECT TOLEDO HARBOR
BORING SITE 2
SAMPLE NO.
DEPTH/ELEV 13'
DATE 09 NOV 94
SHEET 3 OF 6



CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES - STF/GL
TIME IN MINUTES
.1 .2 .5 1 2 5 10 20 50 100 200 500 1000 2000 10000 100000
DIAL READING IN 10^{-4} INCHES
1221 1421 1621 1821 2021 2221 2421
.25 - TSF
PROJECT TOLEDO HARBOR
BORING SITE 2
SAMPLE NO.
DEPTH/ELEV 13'
DATE 09 NOV 94
SHEET 4 OF 6


CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES - STF/GL
TIME IN MINUTES
DIAL READING IN 10^-4 INCHES
.5 - TSF
.1 .2 .5 1 2 5 10 20 50 100 200 500 1000 2000 10000 100000
1767 1967 2167 2367 2567 2767 2967
PROJECT TOLEDO HARBOR
BORING SITE 2
SAMPLE NO.
DEPTH/ELEV 13'
DATE 09 NOV 94
SHEET 5 OF 6



CONSOLIDATION TEST TIME CURVES
LABORATORY USAE WES -- STF/GL
TIME IN MINUTES
DIAL READING IN 10^-4 INCHES
2280
2480
2680
2880
3080
3280
3480
.1
.2
.5
1
2
5
10
20
50
100
200
500
1000
2000
10000
100000
1. -- T S F
PROJECT TOLEDO HARBOR
BORING SITE 2
SAMPLE NO.
DEPTH/ELEV 13'
DATE 09 NOV 94
SHEET 6 OF 6