01 Jun 1988, 1:00 pm - 5:00 pm

Performance of Lightweight Waste-Impoundment Dikes

Peter J. Bosscher
University of Wisconsin-Madison, Madison, Wisconsin

Tuncer B. Edil
University of Wisconsin-Madison, Madison, Wisconsin

Proceedings: Second International Conference on Case Histories in Geotechnical Engineering, June 1-5, 1988, St. Louis, Mo., Paper No. 1.17

Performance of Lightweight Waste-Impoundment Dikes

Peter J. Bosscher
Assistant Professor of Civil and Environmental Engineering, University of Wisconsin-Madison, Madison, Wisconsin

Tuncer B. Edil
Professor of Civil and Environmental Engineering, University of Wisconsin-Madison, Madison, Wisconsin

SYNOPSIS: The containment dikes of two sludge disposal lagoons were founded on low strength, highly compressible wetland soils in Madison, Wisconsin. These lagoons, constructed in 1942 and 1967 respectively, encompass 130 acres of digested sludge produced at the sewage treatment plant. The dikes have experience two previous failures in 1970 and 1973. A dike rehabilitation program was initiated in 1976 to prevent additional failures. New dikes were built using wood chips as a lightweight fill. Non-woven synthetic filter fabric was used to prevent soil intrusion into the chips and to provide resistance to lateral spreading. An investigation was initiated in 1984 to assess the current and long term stability and settlement of the dikes, to determine the fate of the wood chip fill, and to develop recommendations for ways to stabilize the dikes, if necessary. This paper presents the results of the stability and settlement analyses, and the attendant interpretations. The investigation indicated better than marginal stability, predicted minor loss of freeboard between 1987 and the year 2000, and found only minor changes in the wood chips after 10 years of service.

INTRODUCTION

Wood chips have been used to rehabilitate sludge lagoon dikes constructed over wetland soils in Madison, Wisconsin. The previous dikes required rehabilitation because of two dike failures. The Madison Metropolitan Sewerage District authorized rehabilitation work to be carried out in two phases because of the risk involved. A 600-ft long demonstration project was built and successfully completed in August, 1976. The remaining rehabilitation was done in 1977. Because of site constraints, rehabilitation had to be made near the alignment of existing dikes. The dike rehabilitation program was described by Schneider and Roth (1977). A light-weight fill was made using wood chips from diseased elm trees and a non-woven filter fabric to prevent soil intrusion into the wood chips and to provide restraint against lateral spreading. A soil cover was provided to protect the wood chips and to retain the sludge. This paper describes the evaluation of the state of stability of the dikes nearly 10 years after their rehabilitation. The evaluation program was designed to answer the following questions:

1. What is the state of stability of the dikes 10 years after rehabilitation? Is shear failure similar to the previous failures likely to take place?

2. Is there a significant deterioration of the dike constructed of wood chips? What is the projected dike competency?

3. What is the projected settlement of the dikes? Where is the current procedure of maintaining freeboard leading to in terms of additional settlements and dike stability?

BACKGROUND

The two lagoons are located on the western edge of Nine Springs Marsh, an extensive grass-sedge wetland area, and east of the Madison Metropolitan Sewerage District (MMSD) Nine Springs sewage treatment plant as shown on Figure 1. Lagoon 1 is a 45-acre lagoon constructed in about 1942 and Lagoon 2 is an 85-acre lagoon installed in 1967. Digested sludge produced at the treatment plant is currently discharged and held in these two lagoons.

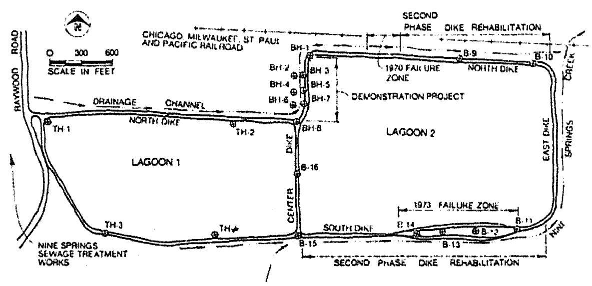

Figure 1. General Plan of the Dikes (from Schneider and Roth, 1976).

Soils underlying the lagoons are of glacial drift and post glacial drift origin. Surface soils are organic and range in thickness from about 10 to 40 feet. The organic soils overlie silt, sand, and gravel till. The till extends to depths of approximately 250 ft where sandstone bedrock is found. Since the water table lies at about the ground surface, the wetland area is often subject to flooding during spring runoff.

Nine Springs Creek flows along the south and east perimeter of the lagoons, and the drainage channel is located along the north edge of the lagoons. They converge at the northeast corner of Lagoon 2 and flow together about one-half mile to the Yahara River. The Chicago, Milwaukee, St. Paul, and Pacific Railroad crosses Nine Springs Marsh just north of the lagoons on a railroad embankment installed in 1854 using displacement methods.

The dikes for Lagoon 1 were built with fill hauled in over a 10-year period. The dikes of Lagoon 2 were constructed by drag-lining peat from the wetland to form embankments around the lagoon area. As the dikes consolidated and displaced the wetland soils, miscellaneous fill consisting mainly of silty sand and gravel, was periodically imported and placed to maintain embankment heights. Existing embankments ranged from 5 to 7 ft in height, with crest widths of 15 to 22 ft and side slopes ranging from 1:1 to 1.5:1.

Dikes around Lagoon 1 have been relatively stable although they require periodic maintenance to compensate for settlement. Dikes surrounding Lagoon 2, however, require continual maintenance and have failed twice since construction. The two areas of embankment failure are shown on Figure 1. Both failures were similar in nature beginning with relatively rapid subsidence of portions of the embankment. As the dikes settled, imported fill was placed to try to maintain freeboard and prevent discharge of supernatant into the adjacent surface waters. The dikes generally continued to settle rapidly, and mud waves formed on both inboard and outboard sides. The failure zones spread longitudinally along the dikes as repair filling proceeded. The failure zone at the north dike eventually extended approximately 300 ft; the failure zone at the south dike extended approximately 1,300 ft.

At the time the north dike failed, sludge and supernatant escaped into the drainage channel. After this failure, more fill was placed and the failure zone eventually stabilized. A similar failure occurred in 1973 in the south dike. The dike crest settled below lagoon level, but the associated mud waves rose high enough to prevent discharge of supernatant.

Presently, both sludge lagoons are essentially filled to capacity. In order to permit continued discharge of sludge to the lagoons, settled sludge is excavated from the lagoons and applied to farmlands. To maintain freeboard, supernatant is also returned to the plant. Disposal of the liquid sludge produced at the treatment works and of the sludge currently held in the lagoons was studied as part of a pollution control facilities plan for MMSD. This study proposed emptying the sludge from Lagoon 2 and the eastern half of Lagoon 1 over a period of 9 to 14 years. The western half of Lagoon 1 would be kept operable for seasonal sludge storage, but the remainder of the lagoons would be abandoned after the sludge was removed. Based on past performance, it was apparent that rehabilitation of the unstable dikes would be necessary to minimize risk of failure until the sludge was completely removed.

Several significant constraints were placed on the design of the rehabilitation of the dikes including the inability to empty the sludge, the inability to construct new dikes on the inboard side without increasing dike height, and the inability to expand the lagoons by building new dikes on the outboard side because of the proximity of Nine Springs Creek, the drainage channel, and the railroad. For these reasons, it was necessary to design the dike rehabilitation as closely as possible to the existing dike alignments. Based on a field exploration and laboratory testing program, Schneider and Roth (1977) evaluated the existing conditions and possible rehabilitation schemes. The rehabilitation methods selected by them use wood chips from trees infected with Dutch elm disease. The City of Madison, working with an independent contractor, was starting to reduce the logs of diseased elm trees to wood chips. The presence of this regularly available supply of wood chips made dike rehabilitation with the wood-chip fill economically attractive. Use of the light-weight wood chips reduced loads to a sufficiently low level so that excess settlement and ultimate general shear failure caused by placement of additional fill could be avoided. Non-woven synthetic filter fabric was used to provide restraint against spreading failures and to prevent intrusion of soil into the wood-chip fill.

In 1979, the Madison Metropolitan Sewerage District initiated an active program of applying the sludge to farmland as a fertilizer. However, this program, which was aimed at removing the sludge from the lagoons, was retarded by the discovery of PCB's in portions of the sludge and by regulatory restrictions on the allowable concentration of PCB's in the farmland fertilizers. Therefore, it is expected that the lagoons could be holding sludge for another 15 to 20 years, nearly doubling the initial design life of the rehabilitated dikes. Since another uncontrolled release of sludge and supernatant caused by continued subsidence of existing dikes and/or shear failure is not acceptable, an evaluation of the current status of stability of the dikes was undertaken nearly 10 years after their construction.

SUBSURFACE CONDITIONS AND SOIL PROPERTIES

Twenty test borings were drilled at various locations around the lagoons by Schneider and Roth (1977) as shown in Figure 1. Based on results of those explorations, the general soil conditions at the sludge lagoons were determined to be as given in Figure 2. Three new boreholes and a number of cone penetrometer soundings were made initially as part of the current evaluation program. Another 8 boreholes were made later as part of the installation of instrumentation for the field monitoring program. The subsurface conditions revealed in these boreholes generally supported the subsurface conditions described in Figure 2.

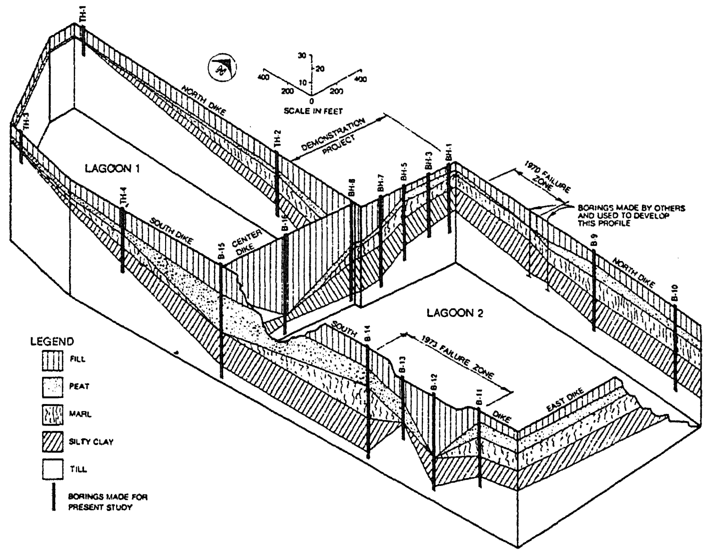

Figure 2. Subsurface Fence Diagram (from Schneider and Roth, 1976).

Some of the boreholes were made very close to the boreholes made in the investigation of Schneider and Roth. For instance, a new borehole located near boring B-14 (see Figure 1) in the 1973 failure zone revealed the presence of 7 ft of new fill of gravel/wood chips combination underlain by 14 ft of old fill material consisting of gray-brown sandy-silty material. The native material below the fill is a high organic content (89%) peat with water contents of up to 400%. The thickness of the peat was 11.5 ft in the recent boring compared to 13 ft in boring made by Schneider and Roth. Under the peat, an organic material with lower water contents (about 200 to 300%) and a lower organic content (28%) was encountered. This 22.5 ft thick layer corresponds, perhaps, to the 24 ft thick deposit referred to as "silt, brownish-gray with occasional shell fragments" in B-14 of Schneider and Roth. This layer was underlain with another peat deposit of lower organic content (44%) compared to the upper fibrous peat. This is referred to as "amorphous peat" herein. This deposit is about 5 ft thick. At a depth of 55 ft there is a gray silty clay of lower water content (about 68%). This latter material is classified as a "highly plastic silt" according to the Unified Soil Classification System. It has a low organic content of about 5%. The thickness of the peat in the underlying organic material is about 1.5 ft less than the values given in the boring of 1975, perhaps indicating the amount of compression of the layers within the last 10 years.

Laboratory tests were performed to characterize the wetland soils and to determine the shear strength and compressibility of the subsurface materials. The testing program included 11 consolidated-undrained triaxial compression tests with pore pressure measurements on undisturbed samples of the old fill, peats, and organic silt. Additionally, 4 consolidated-drained direct shear tests on samples of wood chips obtained from depth of 2.5 ft and 9 ft, respectively, were performed to provide a comparison of the strengths of the young and old wood chips. Laboratory consolidation tests were performed on samples from the major compressible soil units encountered, i.e., the peat, the organic soil called "marl", and the silty clay. These tests provided an updating of the extensive laboratory tests performed by Schneider and Roth (1977).

Strength Properties of Wood Chips and Soils

The effective strength parameters of the various soil units were estimated based on the recent tests and compared with the values reported and used by Schneider and Roth.

Wood Chips

The direct shear tests performed on wood chip samples retrieved from 2.5 and 9 ft depths, indicated friction angles of 53° to 57° as compared to 49° reported by Schneider and Roth. This may be a result of the difference in densities as specimens were reconstituted in the laboratory. The most recent specimens were denser than the Schneider and Roth specimens. The insitu density was determined from an undisturbed shelby tube sample first, and the chips were reconstituted at this density for the strength test. Furthermore, the higher friction angles were determined using a drained direct shear test, whereas Schneider and Roth performed triaxial tests.

An interesting observation is the difference of 4° in the friction angles of the wood chips obtained from 2.5 ft and 9 ft. The deeper (older) wood chips exhibited the lower friction angles. Apparently, some degradation due to decomposition has taken place, but not to a great extent over 10 years. It is believed that there has not been a major change in the source of wood chips during this time. The older wood chips have smaller size than the younger wood chips.

Old Fill

The low plasticity silty clay material found in the old fill of the north bank indicated an effective friction angle of about 29° which compared well with the values in the analysis by Schneider and Roth. There is an effective cohesion intercept of about 400 lb/ft^2.

Organic Soils

The tests of the top fibrous peat gave an effective friction angle of 53°, which is decidedly higher than 40.5° reported by Schneider and Roth. The stability of the dikes was checked with both 42° and 53° in the analyses. A cohesion intercept of 200 lb/ft^2 was also measured.

It was noticed that the 22.5 feet of soil under the fibrous peat revealed in the boring is a rather complex material with a range of organic contents. The first 17.5 feet of this material consists of "organic silt with shell fragments". The 2 samples fitting this description were tested giving 40° for the effective friction angle with a cohesion intercept of 400 lb/ft^2. Schneider and Roth reported two tests of friction angles of 36° to 38° for this material. However, they chose to use 29° in their stability analyses.

The bottom 5 ft of this organic layer is an amorphous peat, which gave an effective friction angle of 5.5° and a cohesion intercept of about 830 lb/ft^2. These values are considered somewhat unusual. However, they did not influence stability analyses since most of the critical failure surfaces were confined to the zone above this layer.

Highly Plastic Silt

This is a deposit of very low organic content encountered in the recent borings as well as the borings of Schneider and Roth. There were no strength tests on samples from this layer. Most failure surfaces were confined to the weak peat deposits above this layer in the analyses.

Compression Properties of Soils

The marl had an organic content of about 28% and exhibited a compression behavior very similar to that of the peat samples (organic content 23% to 89%). The silty clay had a markedly lower organic content (about 5%) and exhibited much less compressibility compared to the others as shown in Figure 3. This figure also implies that the soils encountered had been compressed under a stress of about 1,000 lb/ft^2 or less. The one-dimensional laboratory compression versus time curves were represented using an equation first proposed by Gibson and Lo (1961) and applied to peats and organic soils by Edil and Dhowian (1979). This equation has been found to be quite useful in representing the compression of peats from numerous sites and has the following form:

$$\varepsilon(t) = \Delta\sigma\left\{\mathbf{a} + \mathbf{b}\left[1 - e^{-\left(\frac{\lambda}{\mathbf{b}}\right)t}\right]\right\} \tag{1}$$

where $\varepsilon(t)$ = vertical strain,

$\Delta\sigma$ = stress increment,

t = time,

a = primary compressibility,

b = secondary compressibility,

$\lambda/\mathbf{b}$ = rate factor for secondary compression.

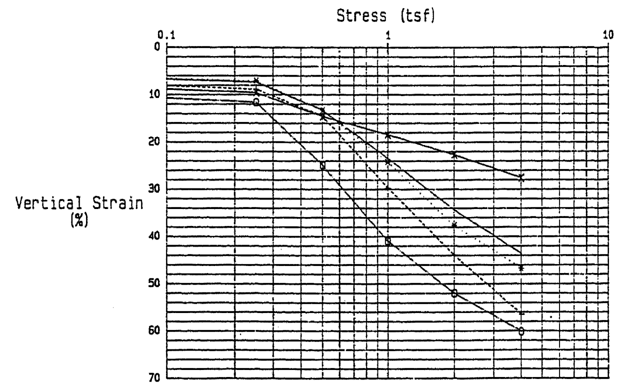

Figure 3. Compression Curves of Wetland Deposits

A convenient method of analysis of a given set of either laboratory or field vertical strain versus time data was described by Edil and Dhowian (1979). This method is used in determining the empirical compression parameters (**a**,**b**, and $\lambda/\mathbf{b}$). The settlement is calculated by multiplying the vertical strain by the initial thickness of the soil (in the laboratory or in the field). The compression parameters depend on the type of peat and the stress level. Often there is a difference between the measured laboratory values and those seen to be governing the field compression.

The laboratory compression curves (vertical strain versus time) under different increments of stress have been evaluated using the compression equation given above to obtain the parameters resulting in the best possible fit of the equation to the measured data. Figures 4 and 5 provide a plot of parameters **a** and **b** as a function of the stress level used in the laboratory tests. These curves show trends noted before for similar soils from other sites (Edil and Mochtar, 1984). There is a decrease of compressibility with increasing stress. Figure 6 shows the rate factor for secondary compression ($\lambda/\mathbf{b}$) as a function of the average strain rate ($\dot{\varepsilon}$). Average strain rate is defined as the final measured strain divided by the lapsed time under a given increment of stress applied.

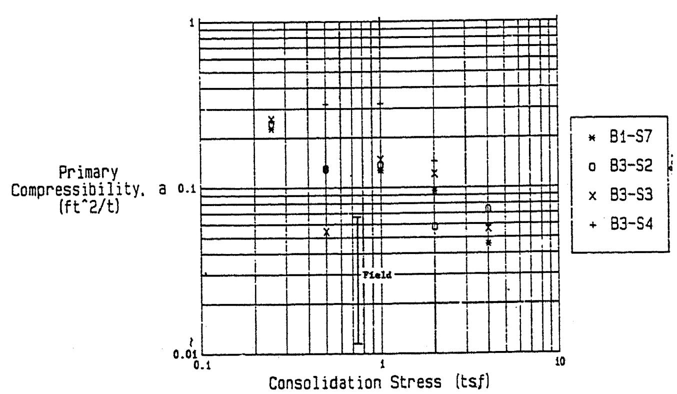

Figure 4. Primary Compressibility versus Consolidation Stress

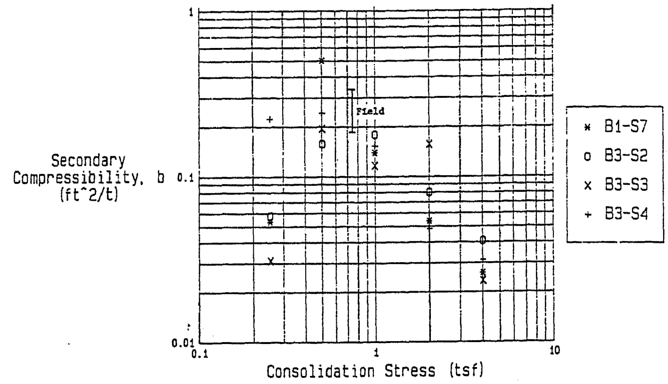

Figure 5. Secondary Compressibility versus Consolidation Stress

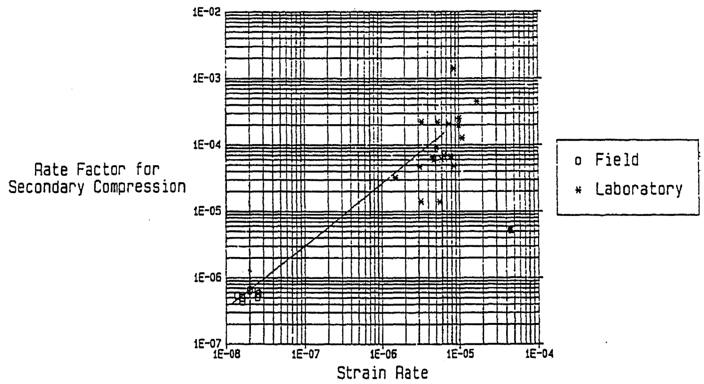

Figure 6. Rate Factor for Secondary Compression versus Strain Rate

STABILITY ANALYSES

A series of stability analyses for a cross section (see Figure 7) through the south dike of Lagoon 2 near the west end of the 1973 failure zone was performed. This section was considered to be a critical section based on the thicknesses of the soft soil deposits and the geometry of the embankment. Furthermore, the most detailed account of soil properties and the stratigraphy was available at this location. Initially, it was assumed that all of the excess pore pressure in the peat underlying the fill had completely dissipated. In effect, the pore pressures in the underlying soils were set equal to the hydrostatic pressures produced by the lagoon and canal water elevations. Other analyses were made in which the pore pressures in the peat were higher than the hydrostatic pressure. There were also analyses performed in which the soil properties were varied within the perceived range of uncertainty. All of the analyses used an effective stress approach and were performed using the STABL computer program (Siegel, 1975) which is based on the limiting equilibrium method of slices for circular and non-circular failure surfaces. A summary of safety factors as a function of amount of excess pore water pressure assumed in the peat layer (0,100, and 200 lb/ft^2) is given in Figure 8 for different assumptions about the soil properties and the choice of analysis. These analyses indicated the following (Edil and Bosscher, 1985):

1. Under the hydrostatic pore pressure regime, the cross section was quite stable.

2. The slope on the lagoon side was found to be more unstable than the slope on the canal side.

3. The presence of excess pore pressures in the underlying peat layer is very significant relative to stability.

4. All of the critical failure surfaces were confined basically to the upper peat layer.

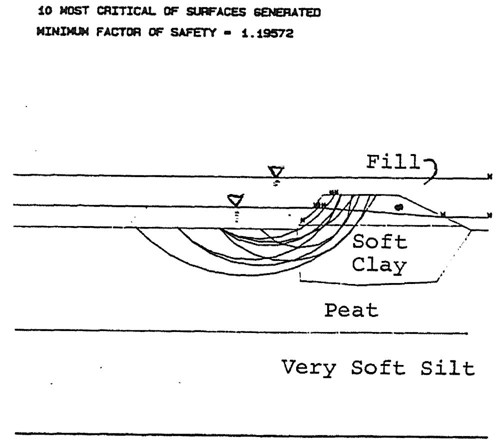

Figure 7. Stability Analysis Section and Critical Failure Surfaces

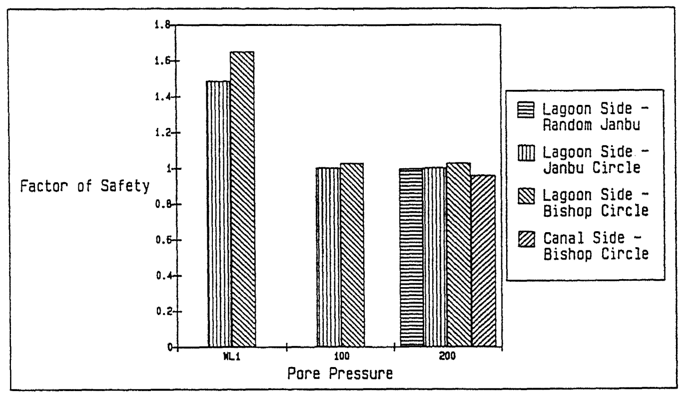

Figure 8. Safety Factors for Different Pore Pressures
[WL1=Hydrostatic Pressure; 100,200=Excess Pore Pressure (lb/ft^2)]

It is clear from this analysis that the presence of residual excess pore pressures induced by the weight of the fill (dike material) and additional pore pressures due to upward ground water discharge can reduce the stability of the dike slope to near critical levels. Since the lagoons lie within a groundwater discharge zone (as indicated in the previous studies), upward seepage should be investigated relative to stability.

Since it is clear that the dike stability is dependent on the magnitude of the excess pore water pressure in the underlying soils, it became imperative to make an assessment of the amount of excess pressure. For this purpose, a piezometer and a well were installed near boring B-14. The porous tip of piezometer was at a depth of 25 ft below the top of the dike and sealed in the underlying peat deposit. A cased open well was placed in the middle of the dike crest at the same location and extended 15 ft into the fill material. The water levels observed in the piezometer indicated that the peat deposit is not experiencing higher pore pressures than would be expected. It is, however, higher than the level indicated in the open well in the dike material. This may be due to the lag in water level response to the changes in the water levels of both the lagoon and the nearby stream channel. Using this new information, additional stability analyses were performed. These analyses indicated better than marginal stability of the dike under present conditions of strength and pore water pressure. However, because of the limited hydrologic record, further periodic observations of the pore water pressures was needed to ensure the maintenance of this stability.

SETTLEMENT ANALYSES

During construction, settlement plates were installed under the reconstructed dikes, six settlement plates were placed under the southeastern half of Lagoon 2 dikes in 1977, another 3 settlement plates were installed later along the southwestern half of Lagoon 2 dikes and 6 settlement plates along the northeastern portion in 1979. Some of the settlement plates were damaged due to traffic on the dikes and the data related to them was discarded. However, there were about 7 settlement plates giving useful data. These 7 sets of settlement data were analyzed by assuming a stress increase of 420 lb/ft^2 (corresponding to a 7 ft high wood-chip embankment above water) and estimating the compressible layer thickness from the borings. The field settlement data collected by the MMSD personnel over the years since the rehabilitation of the dikes was evaluated using the same compression equation given above (equation 1). A range of values for parameters **a** and **b** as obtained from the field settlement data are also marked on Figures 4 and 5 at a stress level of 0.75 ton/ft^2 (estimated to be the average stress level in the field). The field values of **λ/b** are plotted at the field average strains on Figure 6. The grouping of the field and laboratory λ/b values and difference between them is noted. This conforms with the observations made at other sites (Edil and Mochtar, 1984).

An interpretation of the laboratory and the field measured compression values provides some insight into the settlement behavior of the dikes in the future. Table 1 gives the average values and the ranges of the three compression parameters as obtained from the field and the laboratory data. The estimated average values from the empirical curves reported by Edil and Mochtar (1984) based on numerous case records are also listed. It should be noted here that there are some uncertainties in the field analysis of the data stemming from the assumed equal stress increase of 420 lb/ft^2 at each settlement plate and the assumed compressible layer thicknesses. Furthermore, all compressible layers are combined in the back analysis of the field settlement data. The field settlement records, themselves, could be suspect as damage to the settlement plates is likely due to the traffic on the dikes. Nevertheless, some trends do emerge from an examination of all sources of information available.

Table 1. Compression Parameters

Parameter	Field Measured Average	Field Measured Range	Lab Measured Average	Lab Measured Range	From Empirical Curves (Edil and Mochtar, 1984)
a (ft^2/t)	0.0263	0.0113-0.0673	At σ = 0.5 t/ft^2: 0.1583	0.1296-0.3188	0.2735
			At σ = 1.0 t/ft^2: 0.1828	0.1271-0.3209	0.0840
b(ft^2/t)	0.2492	0.1911-0.3259	At σ = 0.5 t/ft^2: 0.2739	0.1577-0.5008	0.0874 (0.2466)*
			At σ = 1.0 t/ft^2: 0.1471	0.1164-0.1794	0.0536 (0.4824)*
λ/b (1/day)	7.9×10^{-4}	$(6.4-9.3)\times10^{-4}$	1.8×10^{-1}	$(1.4-6.5)10^{-1}$	5.5×10^{-4}

* Corrected to the field as recommended by Edil and Mochtar (1984).

The laboratory measured values of parameter **a** are higher than the field values. The empirical values provide a range comparable to the laboratory and field values of **a**, depending on the stress level. The higher laboratory values of **a** will make the prediction of field settlements larger than given by the settlement plates. However, the rate of settlement after a number of years would not be affected as much by the value of parameter **a** as by the values of **b** and **λ/b**. The values of parameter **b** are quite consistent between the field and the laboratory. The empirical values corrected to the field as suggested by Edil and Mochtar (1984) also compare well with the Nine Springs values. It is noted here that the type of laboratory/field corrections suggested by Edil and Mochtar is not required for the Nine Springs data. The values of **λ/b** in the field are much lower than obtained in the laboratory. This a well-established trend. The value given by the empirical curves for the average field strain is very comparable to the average value of **λ/b** obtained directly from the field data.

The presence of uncertainties and unresolved inconsistencies, i.e., discrepancy between the field and laboratory **a** values, and the lack of the expected discrepancy between the field and the laboratory **b** values, precludes highly conclusive predictions of future settlements. However, an attempt can be made in establishing some bracketing values.

If we assume that the settlement plate data are a reliable indicator of the actual settlement trends, we can use the lowest and highest combinations of the compression parameters obtained from the field data and estimate upper and lower bounds of future settlements. Table 2 presents such a prediction using the compression equation. The compressible layer thicknesses used were 34 and 23 ft for the upper and lower bounds estimates, respectively. The settlement data as given by the settlement plate SP2 is also included in Table 2. This settlement plate gave the largest settlement of all the plates considered to be undamaged. At the site of this plate, which is near boring B-14, the compressible layers are perhaps the thickest, measuring about 34 ft. The upper bound estimates compare well with the SP2 record. Extrapolation of the settlement plate data to the present (3,010 days) and to the year 2,000 (8,054 days) indicates that less than 2 inches of additional settlement is to be expected.

Table 2. Settlement Prediction

Date	Time (days)	Settlement Upper Bound Magnitude (ft)	Settlement Upper Bound Change (in)	Estimate Lower Bound Magnitude (ft)	Estimate Lower Bound Change (in)	Settlement at SP2 Magnitude (ft)	Settlement at SP2 Change (in)
12/12/1977	0						
4/12/1978	121	0.73		0.11		0.55	
			10.4		2.8		10.6
11/16/1979	704	1.60		0.34		1.46	
			2.6		0.7		2.9
6/18/1980	919	1.82		0.40		1.70	
			7.4		2.6		9.1
6/1/1983	1997	2.44		0.62		2.46	
			2.8		1.3		4.3
3/10/1986	3010	2.67		0.73		2.82	
			1.6		1.4		
1/1/2000	8054	2.81		0.85			

If estimates for settlement are based on the largest values of parameters **a**, and **b**, considering both the field and the laboratory data, 4 inches of settlement between now and the year 2,000 can be expected. These estimates, while not very firm, are quite encouraging with respect to future settlement performance of the dikes. Either of these estimates results in a minor loss of freeboard and is quite tolerable.

DIKE INSTRUMENTATION

Justification

Based on this evaluation of the rehabilitated dikes nearly 10 years after their construction (Edil and Bosscher, 1986), the following observations could be made:

1. The stability analyses indicate better than marginal stability of the dikes under present conditions of strength and pore water pressure. However, because of the limited nature of the hydrologic record collected, further periodic observations of the pore water pressures are needed to ensure the maintenance of the stability.

2. Wood chips placed in the dikes 10 years ago seem to exhibit strength properties close to those recently placed indicating relatively low amounts of degradation. Based on a careful survey of the literature, common estimates of useful life of wood chips are found to extend to at least 15 years (Nelson and Allen, 1974; Jackson, 1979). No published studies of chip durability have extended these estimates beyond 15 years even though it is expected that wood chips could provide acceptable performance for longer periods.

3. Settlement estimates indicate a minor loss of freeboard can be expected between the present and year 2000. These estimates, while not very firm, are quite encouraging with respect to future settlement performance of the dikes.

Based on these observations, the Madison Metropolitan Sewerage District was advised as follows:

1. No evidence has been found to warrant additional major construction to improve dike stability. This recommendation is based, however, on an adoption of a continual monitoring program relative to dike settlement and stability.

2. The monitoring program should consist of multilayer settlement and piezometric measurements. In addition, any regrading or other significant construction activity should be noted and evaluated for the potential effect on dike stability.

3. The levels of supernatant in the lagoon and the water level in the stream channels should be recorded periodically. The levels of these fluids are important relative to the stability of the dikes. Safe levels for these fluids may be established, with any variation outside of these levels to be allowed only after an evaluation of the effect on dike stability.

Following these recommendations, Madison Metropolitan Sewerage District authorized installation of an instrumentation system during the Fall of 1986. The instrumentation system was designed to monitor the following elements of the lagoon dikes:

1. The settlement under the weight of dike fill materials of various compressible layers in the dike foundation.

2. The lateral movements within relatively soft and weak soil layers.

3. The excess pore water pressures in the substrata created by dike fill.

4. The phreatic water levels within the dike due to the levels of supernatant in the lagoon and water in channels outside the dikes.

Field Instrumentation

The Sondex settlement system and the slope indicator system by Slope Indicator Company were chosen to monitor settlement and lateral movements at various depths in the substrata below the dike fill materials. The settlement system consists of 3 inch I.D. flexible corrugated plastic pipe with metal rings attached at 5 ft intervals. An electical probe is used to determine the vertical location of the metal rings within ±0.01 feet. A 2.75 inch O.D. grooved plastic casing, used for the inclinometer system, was typically installed inside the Sondex plastic pipe. The inclinometer, when lowered into the casing and oriented with the casing grooves, provides a rapid means of assessing the deviations of the casing from vertical as a function of depth. These readings may then be compared with readings taken at an earlier time to determine the amount of lateral movement as a function of depth and time.

Geonor M206 piezometers were used to monitor total pore pressures (hydrostatic plus excess) for this project. The piezometer point consists of a hollow stem approximately 1-ft long with 3 bronze filters mounted on the stem. Plastic tubing (1/4 in-I.D.) is attached to the porous point and threaded first through a 5 ft length of E-rod and then through a 1 inch diameter steel water pipe. After the piezometer is pushed or driven into the ground, the water level in the plastic tubing is measured with a small diameter water level indicator especially designed for this purpose. Phreatic water levels in the lagoon dikes were measured using conventional groundwater monitoring wells consisting of 2-in diameter, Schedule 40 PVC slotted well screen and riser pipe. The 10 ft screens were enclosed by No. 30 flint sand to be used as filter material around the screen and as backfill. A seal of dry granular bentonite was placed over the sand to prevent surface water infiltration. Water levels are measured with a conventional electronic water level indicator.

Eight locations along the dikes were chosen for the placement of instrumentation clusters (Weber, 1987). These positions are located at sites circling Lagoon 2 and at other points near Lagoon 1. Criteria used in determining the instrument locations included:

1. areas of observed prior dike failures,
2. areas of known deep wetland soils based on early investigations,
3. areas of unknown subsurface conditions or gaps in the soil profile, and
4. budget constraints.

At 7 of the 8 cluster locations, the Sondex and inclinometers were installed in the same borehole. At each cluster location settlement instrumentation, a lateral movement instrumentation, multiple piezometers, and a well were installed. In all, 8 Sondex and inclinometer systems, 6 monitoring wells, and 10 piezometers were installed. Instrument depths were established by review of an initial boring at each cluster location which was sampled at 2.5-ft intervals in the upper 10 ft, and at 5-ft intervals thereafter with a split-spoon sampler using standard penetration test (SPT) procedures and extended on the order of 5 to 10 ft into sand soil below the compressible wetland deposits. Additional boreholes within a cluster were drilled to permit instrument installation without additional sampling. The drilling methods included 4-in diameter continuous flight augering and rotary wash boring.

Since the completion of the installation of the instrumentation system by the end of 1986, a program for periodic measurement of the instruments was initiated. A computer program for the database management, reduction and presentation of the data was developed.

FIELD MONITORING DATA

The data obtained from the field consists of:
1. water levels in the open wells and the piezometers,
2. depths of the Sondex rings, and
3. inclinometer readings.

The data has been collected since late February, 1987 upon completion of the instrument installation. The water level data has been used to support the assumptions made in the dike stability analysis. The results as obtained from the monitoring instrumentation near boring B-14 are presented in Figure 9. The measured piezometric levels are similar to the values assumed during the analysis phase. It can also be noted that the previous assumptions of the region being a groundwater discharge zone is substantiated (see piezometric levels in Piezometers 2 and 3 compared to the surface well) The dramatic dip in the piezometric levels around early August in Piezometers 2 and 3 is due to pumping of these piezometers to determine the response of the system. It should also be noted that the surface well water elevation lies between the creek elevation and the sludge elevation, the two boundary elevations.

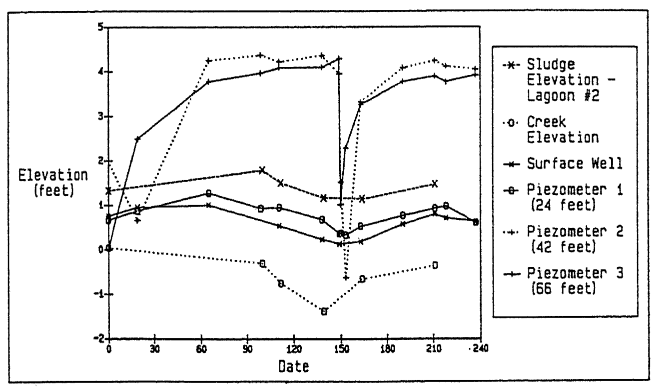

Figure 9. Ground Water Level and Pore Pressure Measurements

The settlement data has been reduced and plotted in Figure 10. This figure indicates the small amount of settlement which can be measured using the Sondex system. The data is presented with the amount of settlement assumed to be zero at the time of installation. Even with only 240 days of settlement, the results show that the settlement is zero in the mineral soils at +68 feet, and the settlement occurring mainly in the soils in the 40-68 ft depths. Additional settlement can be seen in the 15-20 ft depth. The Sondex pipe was riveted to the inclinometer pipe at the ground surface which is thought to account for the reduction in measured settlement at the ground surface. Based upon the measured rate of 0.07 feet occurring in 240 days, the additional total settlement expected between now and the year 2000 is under 1.5 inches assuming a logarithmic decline in the settlement rate. This value compares well with the previous prediction of less than 2.0 inches.

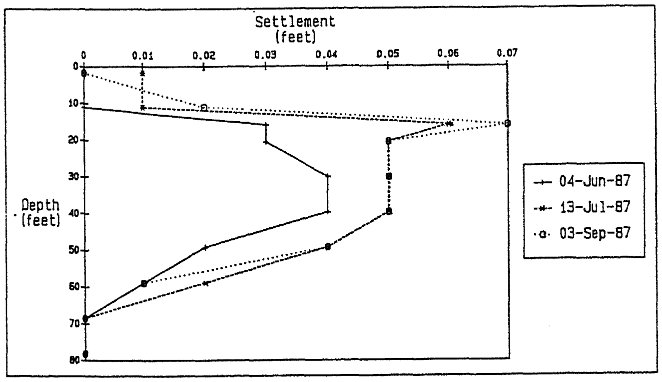

Figure 10. Settlement Measurements

The inclinometer data indicated negligible movements in the tubes since installation. This supports the stability analyses which indicated that the dikes should be stable in their present configuration.

Further monitoring of this site is continuing on an ongoing basis. The level of effort in data acquisition will likely taper off as the data is analyzed and annual trends in data are noted and shown to support the previous conclusions.

CONCLUSIONS

Sludge lagoon dikes rehabilitated using wood chips and founded on highly compressible and weak wetland deposits have provided satisfactory performance over the past 10 years in a difficult environment for the use of conventional stabilization techniques. The present evaluation of stability and settlement based both in analysis and field data collected to date indicates that the dikes are likely to continue to provide satisfactory service for another 15 to 20 years. Due to rather marginal stability of the dikes and the environmental risks associated with a dike failure, a field monitoring program has been initiated. This program has provided an initial verification of certain assumptions used in the analyses. The instrumentation is expected to serve as a continuing control and warning system during the service life of the dikes.

ACKNOWLEDGEMENTS

The authors wish to express their appreciation to the Madison Metropolitan Sewerage District and Mr. James L. Nemke, Director, for their support of this work. CH2M Hill Engineers designed the dike rehabilitation and Warzyn Engineering, Inc. installed the field monitoring instrumentation. Messrs. N. Paschke and G. Sachs made the field measurements. Mr. N. H. Severson's help in the field and the laboratory is gratefully acknowledged.

REFERENCES

Edil, T.B. and P.J. Bosscher, (1985), "Geotechnical Investigation of Nine Springs Sludge Lagoon Dikes," An Interim Report to the Madison Metropolitan Sewerage District, Madison, Wisconsin.

Edil, T.B. and P.J. Bosscher, (1986), "Geotechnical Investigation of Nine Springs Sludge Lagoon Dikes," The Final Report to the Madison Metropolitan Sewerage District, Madison, Wisconsin.

Edil, T.B. and A.W. Dhowian, (1979), "Analysis of Long-Term Compression of Peats," Geotechnical Engineering, Southeast Asian Society of Soil Engineering, Vol. 10, No. 2, p.159-178.

Edil, T.B. and N.E. Mochtar, (1984), "Prediction of Peat Settlement," Proc., Sedimentation/Consolidation Models, American Society of Civil Engineers, p.411-424.

Gibson, R.E. and K.Y. Lo, (1961), "A Theory of Soils Exhibiting Secondary Compression," Acta Polytechnica Scandinavica, Ci 10296, p.1-15.

Jackson, N.C., (1979), "A Summary of the Use of Sawdust for Highway Fills," Materials Office Report No. 157, Washington State Department of Transportation.

Nelson, D.S. and W.L. Allen, Jr., (1974), "Sawdust as Lightweight Fill Material," Report No. FHWA-RD-74-502, Federal Highway Administration, Offices of Research and Development, Washington, D.C.

Schneider, J.R. and L.H. Roth, (1976), "Geotechnical Investigation Sludge Lagoon Dike Rehabilitation, Phase II," CH2M Hill Report L-9617.7 to Madison Metropolitan Sewerage District, Madison, Wisconsin.

Schneider, J.R. and L.H. Roth, (1977), "Dike Rehabilitation Using Wood Chips," Proc. Conference on Geotechnical Practice for Disposal of Solid Waste Materials, American Society of Civil Engineers, Ann Arbor, Michigan, p.661-679.

Siegel, R.A., (1975), "STABL User Manual," Joint Highway Research Project JHRP-75-9, Purdue University and Indiana State Highway Commission, West Lafayette, Indiana.

Weber, R.H., (1987), "Sludge Lagoon Dike Instrumentation," Warzyn Engineering Inc. Report C12764 to Madison Metropolitan Sewerage District, Madison, Wisconsin.