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THE 1997 CLARK LANDFILL FAILURE AT INDIANA HARBOR WORKS LTV STEEL COMPANY, EAST CHICAGO, INDIANA

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ABSTRACT

A rapid failure of approximately 900,000 cubic yards of fill and lake bed foundation soil occurred sometime between 7:00 a.m. and 8:00 a.m., on August 6, 1997. The 45-acre landfill was approximately 100 feet higher than Lake Michigan and slid on a layer of weak silty clay located 55 to 60 feet below the water surface of Lake Michigan. The horizontally-translating slide mass nearly blocked the operating intake flume to the No. 2 Intake Pump House for the steel mill cooling water. Figure 1 shows the plan location of the slide mass and scarp location. The slide mass extended approximately 1,000 feet from the southwest fill area to a location east of the cofferdam that holds an oil boom along the south side of the landfill next to the intake flume. Pre-failure and failure conditions with estimated slide plane location and scarp geometry for failure sections A-A, B-B and C-C are shown on Figures 2 through 4.

The slide extended 200 to 300 feet north from the flume up to a 30- to 40-foot high scarp. The slide mass moved approximately a a 30- to 40-foot high scarp. The slide mass moved approximately 30 feet into the canal and moved the cofferdam structure at least 10 feet south. In fact, the slide mass filled more than 400 feet of the 25-foot-deep, by 140-foot-wide flume and nearly blocked the flume with only 3 to 4 feet of water flowing when the channel was 20 to 25 feet deep.

Clark Landfill Conditions Prior to Failure

According to project aerial photographs from 1939 to 1998, and 1971 plant drawings for Youngstown Sheet and Tube (predecessor to LTV Steel Company) reviewed by the authors, Lake Michigan waters formerly covered what is now called Clark Landfill. Designs for the intake flume containment dike were prepared in 1971. The 1973 aerial photograph showed most of the Clark Landfill footprint to have been filled above the lake level. In August 1989, LTV was granted an interim status permit to operate Clark Landfill as an on-site restricted steel mill waste landfill by the Indiana Department of Environmental Management (IDEM).

According to LTV records, there was steel mill waste up to and above El. 600 (NGVD) in the southwest fill area prior to 1991, as illustrated in Figure 5. It appears that the landfill was filled to El. 600 by 1975 with mill waste placed above that elevation beginning at the east corner and progressing toward the west. By July 1991 (LTV aerial survey), the north central portion of the landfill was filled from El. 600 up to 650 feet, as shown on Figure 5, with side slopes of approximately 1.5 horizontal to 1 vertical (l.5:lV) or steeper around the perimeter of the landfill.

Based on LTV records, the landfill in the southwest area was to be filled between 1993 and 2002 from El. 600 to 650 along the intake flume with perimeter slopes of 2H:1V and a 2 percent slope from El. 650 up to 656. Figures 6 through 9 depict the landfill closure design in 1993. No topographic survey, no explorations, no historic file review, and no stability analyses were performed as part of this 1993 closure design effort. Based on LTV records, the assumed the average filling rate in 1993 was approximately 108,000 cubic yards per year.

By August 1996 the landfill took on new dimensions. The maximum height was above El. 670 in the north central portion of the landfill, and at El. 600 to El. 620 in the southwest portion of the landfill. Figure 10 illustrates the existing topographic plan in July 1996. Figures 11 through 13 show the consultants 1997 revised proposed landfill cross-sections along the southwest portion of the landfill next to the flume, showing the top of the landfill was now proposed to close out at El. 720. Between 1991 and 1996, filling rates reportedly increased from 108,000 to 160,000 cubic yards per year.

With the landfill approaching its capacity, LTV and their consultant discussed options in October 1996 for closing the landfill. In November 1996 LTV accepted a landfill closure scheme titled, “Alternative 10” which involved continued filling and re-grading of the landfill through May 1998, using the current filling rate. The landfill was to attain a maximum El. 720 with 3H:V slopes along the intake flume in the southwest fill area and 3H:1V slopes. Figure 14 presents the 1997 landfill regrading plan.

Site Geology

The LTV site is dominated by the post glacial littoral silt and sand deposit over the Lake Border and Wheeler Till, consists of overconsolidated medium stiff silty clay.

The upper Lake Border Till and lower Wheeler sequences represent two different advances or phases of the Lake Michigan Glacier Lobe. The Wheeler Sequence consists of clayey silt and silty clay and is more granular and harder than the overlying Lake Border Sequence. The Wheeler Till Sequence is typically described as heavily overconsolidated hardpan clay. The Wheeler Till overlies the Silurian dolomite.

Generalized Subsurface Profile

Subsurface fill and soil conditions at the site are described below from the ground surface downward in terms of approximate elevation, geologic units and soil descriptions:

- El. 681 to 550 Waste Fill - Post 1971 fill comprised of loose to dense blast furnace (BF) slag, basic oxygen furnace (BOF) steel making dust, mixed with fly ash, and other steel mill waste.
- El. 552 to 548 Silt and Sand (Post Glacial Coastal Deposit - Natural, thin deposit of littoral drift consisting of medium dense to dense, gray silty fine sand, trace clay.
- El. 548 to 505 Silty Clay (Lake Border Till) - Natural, soft to stiff, gray, silty day, trace fine sand, trace gravel. This unit can be divided into three geologic sub-units based on strength, grain size, plasticity and water content properties.
- El. 505 to 446 Hardpan (Wheeler Till) - Natural, very stiff to hard, gray, silty clay and clayey silt.
- Below El. 446 - Silurian dolomite bedrock.

The Lake Michigan water level in 1997 at the time of the failure was approximately at El. 581 feet.

Pre-Failure Geotechnical Studies at Landfill Site

The authors reviewed copies of geotechnical studies performed by LTV's consultant at the Indiana Harbor Works for the Northwest fill area and previous landfill sites. There was an abundance of geotechnical data on the glacial lake bed sediments beneath the site. The authors reviewed 1964 reports by D'Appolonia & Associates, Inc. (D'Appolonia). These reports indicate there were 11 borings at six exploration stations located along the east, north, and west perimeter of the Clark Landfill site, shown on Figure 5 as 1964 defined stations A, B, AA, BB, SS and TT.

The borings included soil descriptions, water content measurements, tube sampling, vane shear tests, piezometer and inclinometer installations. The 1964 testing program consisted of isotropically consolidated undrained triaxial compression strength testing with pore water pressure measurements and one-dimensional consolidation testing. The undrained shear strength (S_u) data from field vane shear strength testing in silty clays below El. 550 averaged between 1,000 and 1,200 pounds per square foot (psf). The fill overlying this silty clay was no higher than El. 595 to 600.

D'Appolonia's laboratory triaxial testing program on the silty clay below El. 550 reported a friction angle of 28 degrees, with zero cohesion, for normal stresses in excess of 6,000 psf. Most of the triaxial tests demonstrate contractive behavior during shear. The one-dimensional consolidation tests show the silty clay to have a pre-consolidation stress ranging from 4,000 to 6,000 psf, depending upon water content, which ranged from 25 to 35 percent, (see Figure 15 for an example of this condition).

However, the D'Appolonia test results only include data from silty clays that were recently loaded with fill up to El. 600. These studies demonstrated that the silty clays were slightly over-consolidated in 1964 and relatively impermeable, with coefficients of consolidation for virgin compression ranging from 50 square feet per year ($c_{v,}$ ft^2/year) in the laboratory to greater than 100 ft^2/year in the field.

D'Appolonia also performed an undrained strength stability analysis (USA) for a 50-foot-thick (i.e., up to El. 600) fill over the silty clay and a 4,000 psf surcharge fill pressure (i.e., equivalent to 30 feet of waste fill up to El. 630) located 183 feet from the edge of fill next to the lake using a friction angle of 35 degrees for the waste fill and an undrained shear strength of 1,000 psf in the foundation silty clay. The computed factor of safety (FS) was 1.6, which is greater than the regulatory standard minimum FS of 1.3 to 1.5. The D'Appolonia 1964 reports recommended inclinometers and piezometers be installed along the edges of the fill to monitor fill performance by measuring lateral movements and excess pore pressure development. Even though the theoretical FS was greater than the minimum recommended, D'Appolonia recognized the need to monitor performance as a check on theory. In this way, if deformations or fill induced pore pressures in clay developed faster or greater than anticipated, indicating a reduced FS, filling could be adjusted and failure avoided. These reports provide insight into standard of practice for fill placement to make land in Lake Michigan for steel mill expansion during the 20th Century.

In 1988, LTV Steel Company (LTV) retained a local engineering firm to perform a landfill stability analysis of a separate industrial landfill southeast of Clark Landfill. The local firm performed three test borings with continuous sampling in the silty clay, ran drained triaxial shear strength testing, and conducted slope stability analyses on a surveyed cross section. This fill is located next to the Indiana Harbor channel and had approximately 1.5H:1V slopes, rising to El. 630. Triaxial tests performed on the silty clays had individual friction angles ranging from 20 to 31 degrees, with water contents ranging from 34 to 20 percent, respectively. The computed FS against sliding was 1.2 for a deep circular sliding surface passing through the foundation silty clay. As a result of this study, LTV stopped filling and installed vertical inclinometers to monitor the fill performance. LTV between 1989 through 1991, the landfill creeped 1- inch toward Indiana

Harbor during the three year monitoring period without filling. This study and data were apparently not directly used by the engineers working on Clark Landfill.

The Clark Landfill engineers did not specifically reference the work of local published studies such as Peck and Reed (1954), which is often referenced when evaluating the Chicago clay. Peck and Reed (1954) include explorations and contours for the thickness of compressible silty clay at the LTV site, as well as typical consolidation properties for the silty clay, as shown on Figure 16. The closest boring ("G") in the Peck and Reed study (Figure 17) even shows a high water content, lower strength clay layer that correlates well with data from the Clark Landfill southwest of the 1964 designated the Northwest fill area.

Review of 1996 Explorations and Stability Analyses for Landfill Closure

During the summer of 1996, engineers conducted abbreviated program of geotechnical explorations (3 holes) at the Clark Landfill site, as shown in Figure 6. The work was comprised of two standard penetration test (SPT) borings (B-1 and B-4) located at the southwest and east end of the landfill next to the intake flume side of the waste fill, and one cone penetrometer probe hole (CPT-4). The field program proposed in April 1996 was initially designed to have four sampled SPT holes and eight to nine CPT probes. Due to difficulties in drilling through the waste fill, the engineer proceeded with the analysis with the lesser number of borings. The field and laboratory tests included pocket penetrometer tests on SPT soil samples and grain size and Atterberg limit tests on four soil samples.

Silty clay data from Boring B-1 at the southwest fill area (ground surface El. 600.18) and CPT-4 near the flume represent geotechnical data from unfailed clay within the 1997 failure zone at the southwest fill area. Boring B-1 data showed minimum uncorrected blowcounts (N) in the silty clay of 6 blows per foot (bpf) and pocket penetrometer unconfined compression strengths of 0.25 to 0.5 tons per square foot (tsf). Boring B-4 located at the east end of the Clark Landfill has minimum uncorrected N values in the silty clay of 12 bpf. Test boring B-4 blowcounts were higher than B-1, possibly due to a higher and older filling at the east end of the site, which caused some consolidation and strength gain in the underlying silty clay. Figure 18 illustrates uncorrected blowcounts for Borings B-1 and B-4 versus elevation. Using Peck and Reed's (1954) "N/6" correlation of blowcount to unconfined compressive strength, the uncorrected blowcounts from B-1 and B-4 could have identified crude approximate unconfined compression strengths of 1.0 and 2.0 tons per square foot, as shown in Figure 19. This corresponds approximately to undrained shear strength ($S_u = Q_u/2$) of 1,000 and 2,000 psf.

Using 1996 industry standard correlation methods, as published in the often referenced soil mechanics book titled Soil Mechanics in Engineering Practice by Terzaghi, Peck and Mesri (1996), the cone penetrometer tip resistance data can be converted to equivalent unconsolidated undrained (UU) triaxial shear strength data. This correlation is presented on Figure 20 and represents undrained shear strength of 950 to 1,200 psf for unfailed silty clay beneath the future slide mass with the weakest clay near El. 516.

Using these empirical correlation procedures, the undrained shear strength range of 950 to 1,200 psf from B-1 and CPT-4 matches the undrained shear strengths of 1,000 to 1,200 psf presented in D'Appolonia's 1964 shear strength report for silty clay at the site not yet loaded above El. 600. Figure 20 very clearly shows a weaker layer close to El. 516 as noted by the arrow. This turned out to be the failure zone, as was later identified by inclinometer data, and could have been predicted with these 1996 data.

LTV's consultant performed total stress stability analyses in July 1996 using undrained shear strength of 1,500 psf for the 1994 landfill closure geometry. They reported safety factors against sliding (FS) for cross-sections taken perpendicular to the landfill at B-1 and B-4 of 1.1 (1.085 and 1.064), as summarized on Table 1. These computed FS were well below IDEM standards.

In December 1996, LTV's consultant questioned the use of undrained shear strength parameters and thereafter, adopted effective stress friction angles of 20 and 35 degrees, with zero cohesion, for the silty clay foundation clay and waste fill, respectively. LTV's consultant also assumed no excess pore water pressure development or completely drained behavior in their effective stress / strength slope stability analyses. The December 1996 showed FS less than 1.1, as shown on Figure 21. This computed FS was well below IDEM standard for landfill stability.

LTV's consultant also conducted stability analyses for the pre-failure 1997 regrading plan. These analyses assumed no fill-induced excess pore pressures would be developed during loading over the southwest fill area between August 1996 and the proposed closure in May 1998. Therefore, LTV's consultant assumed pore water pressures in the silty clay would be at Lake Michigan level or slightly higher beneath the fill. Table 1 summarizes LTV's consultant's documented 1996 analyses along the intake flume. It was interesting to note all failure surfaces were circular and not wedge block shaped.

Post-Failure Landfill Stability Studies by LTV's Landfill Design Consultant

Within a week after failure, LTV retained original designer to conduct post-failure subsurface explorations consisting of 18 test borings to perform soil sampling, standard penetration tests, and pneumatic piezometer and inclinometer installations. In total, there were five boring clusters within the failure mass (LTV-1 through LTV-5) and two (LTV-6 and LTV-7) outside the failure mass. Figure 22 illustrates where the "LTV" series boreholes and instrumentation were located.

In summary, a total of six inclinometers were installed, four inside and two outside the slide mass. A total of eight pneumatic piezometers were installed in the foundation silty clay. The inclinometers installed within the slide mass showed continued post-failure movement along a translational shear zone. Pneumatic piezometers showed excess pore water pressure within the silty clay foundation. In most cases the excess heads exceeded the ground surface elevation.

A soil sampling and laboratory testing program was performed by local engineers and at a local University. Testing included grain size, Atterberg limits, specific gravity, water content, UU and CIU triaxial compression, and drained direct shear strength testing on undisturbed tube samples of foundation silty clay and remolded landfill waste. Table 2 presents the soil strength models assigned by LTV's consultant to the silty clay layer controlling landfill stability. Two soil strength models were used by LTV's engineer to compute the pre-failure stability of the slide mass using the computer program PCSTABL5M. Both the effective stress analysis (ESA), including assumed excess pore pressures, and total stress analysis using undrained shear strength analysis (USA) models provided factors of safety against sliding below 1.0. Tables 3 and 4 present the results of LTV's design engineer post-failure foundation stability analysis using the USA and ESA models, respectively. The location of the post failure boring locations (LTV-1 through LTV-7) and stability analyses sections are shown on Figure 22.

Independent Post Failure Landfill Stability Studies by STS

LTV retained STS Consultants, Inc. (STS) to determine why the landfill failed. As a result STS performed three phases of explorations at the site. Phase 1explorations, comprised of 16 test borings and nine piezocone probes, were conducted for a general assessment of conditions around and through unfailed portions of the landfill and within the failure mass from April to June 1998. A Phase 2 exploration program was conducted from September to November, 1998 within the intake flume and through the failure mass to facilitate closure design. Nine borings were performed in the flume and three borings were drilled on land.

A third phase of drilling (Phase 3) was conducted in May and August of 1999 to obtain supplemental undisturbed tube samples, perform vane shear tests, and install an inclinometer though an unfailed portion of the landfill next to the intake flume. These explorations included performing piezocone penetrometer tests, drilling rotary drill holes with vane shear tests, and taking SPT samples, 2-inch Shelby tubes, and 3-inch and 5-inch Osterberg tubes for soil testing. The tests were intended to refine the soil model to explain why some landfill cross-sections failed and others did not. Most of the rotary drill holes were instrumented with piezometers or inclinometers.

Laboratory testing of the silty clay foundation soils was performed by STS, University of Illinois, and the University of Massachusetts. The goal of the test program was to measure physical properties and drained and undrained shear strength of the silty clay from various locations inside and outside the failure mass. Testing included water content, Atterberg limits, density, specific gravity, and grain size testing. Tube samples were tested for permeability, one-dimensional consolidation, undrained (UU, CIU, CAU compression and extension) and drained (CD) triaxial compression, drained direct shear (DS) , undrained direct simple shear (CK_oUDSS), and drained (DRS) and undrained rotational shear (URS). The goal of the program was to understand the site geology and develop a soil shear strength model that worked for failed and unfailed silty clay at the site for landfill closure design.

Groundwater Conditions

There is one unconfined surface water aquifer beneath the site. The upper aquifer/phreatic surface is controlled by Lake Michigan water levels (e.g., El. 581 during May 1998) in the flume and waste fill. Water level measurements taken during May 1998 in four open drill casings into the waste fill show water levels in the waste fill ranging from El. 581 to 586. Below the surface aquifer there are saturated but low permeability silty clay layers. LTV's consultants and STS engineers installed multiple level pneumatic piezometers in the silty clay layer between 1997 and 1998. No piezometers were installed by LTV's engineer in 1996. However, many multiple-level piezometers were installed across the Northwest Fill Area by D'Appolonia during 1964 and 1965. This fill area was north and immediately adjacent to Clark Landfill. This silty clay layer has significant elevated or excess pore water pressures induced by filling. The silty clay is slightly over-consolidated, normally consolidated and/or under-consolidated and has had insufficient time to dissipate its excess pore water pressure due to its low permeability (triaxial permeability tests by STS and by the University of Massachusetts have measured permeabilities in the range of $1x10^{-7}$ to $1x10^{-8}$ centimeters per second).

Cross-sections of the landfill at A-A, B-B, C-C and D-D, presented in STS'1998 studies, show piezometric head levels increasing with depth within the silty clay layer, indicating drainage upward toward the natural silty sand layer and waste fill, which serve as the single upward drainage blanket.

Figure 23 presents representative post-failure piezometric water levels in the silty clay versus time for the installed piezometer instruments. Nearly all of the piezometers show excess pore water pressure dissipating slowly with time. These figures were used to linearly extrapolate back in time and estimate the magnitude of the pore water pressures on August 6, 1997. These linear extrapolations of were used to model failure in our effective stress analysis of stability to be discussed later.

STS noted active landfill loading after the failure as dredging's from the filled intake flume and new BOF was being placed on the northeast side of Clark Landfill, as shown on Figure 22, and this filling confirmed the increased pore pressure response in the 1997 instrumentation. Two piezometers in the silty clay, located at LTV-5-SPT and LTV-

7-STP, showed increased piezometric heads between August 1997 and March 1998. Once this increased pore pressure problem was reported, LTV suspended all waste filling at the Clark landfill in early May 1998. According to LTV correspondence to the steel mill, dated May 13, 1998, Clark Landfill immediately stopped receiving the heavy BOF dust waste stream.

Figure 24 illustrates this important condition, since it demonstrates that the average 20-foot rise in fill being placed at the northeast side of Clark Landfill caused rapid pore water pressure increases in instruments 500 and 750 feet away from and along the south side of the flume and slide area. The fill area and “LTV” borings are shown on Figure 22.

Preliminary Stability Review

When STS was retained, we began a preliminary review of the cause of failure. To get a quick and simple feel for the problem, we used the stability chart developed by Taylor (1948) to assess the approximate height of fill that could be placed rapidly over the silty clay beneath the southwest fill area without causing a stability failure. We reviewed the blowcounts from B-1 and cone penetration data from CPT-4, and used vane shear test data from the adjacent steel mill across Indiana Harbor to assign a preliminary undrained strength of 1,200 psf to silty clays underlying the southwest fill area.

We evaluated whether LTV’s engineered 1993 fill using a 2H:1V slope up to El. 650 and LTV's engineered 1996 fill using a 3H:1V slope up to El. 720 could be achieved. Using Taylor's chart, shown as Figure 25, we computed the limiting fill elevation to be 630 and 633 (for Case 1loading conditions, as defined on Figure 25), for LTV's 1993 and 1996 design options, less than the proposed design top elevations. This simple method can be used as a screening tool to assess if the proposed design is readily achievable. The Taylor method is conservative and did not model the benefits of the granular waste fill strength or the time rate of filling. This quick screening method indicated potential problems, so we proceeded to a more formal review involving different soil models and stability analysis methods.

Shear Strength Models

Two soil strength models were used for this review. The first method used undrained shear strengths in an undrained shear stress analysis (USA) based on LTV's field (SPT, CPT and Q_p) data to assess the 1993 and 1996 Clark Landfill closure designs and the 1996 existing conditions. The adopted shear strength for the silty clay under the southwest fill area was assumed to be 1,200 psf. This represents an average strength for the site and matches historic information from D'Appolonia and from adjacent steel mills.

The other model considered soil friction and pore water pressures in an effective stress analysis (ESA). In this analysis, we adopted anisotropic shear strength properties for the silty clay stratum divided into three distinct layers. We assumed the critical failure surface passed along a near horizontal (about 5 degrees above horizontal) slide plane and assigned a friction angle of 21 degrees (with zero cohesion) for the 40-foot thick silty clay along this plane (Figure 26). For silty clay outside the 21 degree zone, we adopted a friction angle of 27 degrees. The 21 degree friction angle is slightly higher than the 20 degrees used by LTV’s engineer. Prof. Arthur Casagrande also measured a 20 degree friction angle using a drained triaxial test on similar Chicago clays, as referenced in Peck and Reed (1954). We used the pore pressure ratio, R_u, in the stability analyses by extrapolating piezometer data back to August 6, 1997 to reduce the shear strength along the sliding surfaces.

We adopted LTV’s consultant engineer’s selected friction angle of 35 degrees for the mill waste fill, BF and BOF slag. This is based on several direct shear tests commissioned by LTV. There may be some cementation in the waste fill; however, this may be offset by pockets of looser or softer waste fill deposits. The strength of the waste is not very critical at this site since the landfill failure was deep within the foundation silty clay, translational in nature, and all of the active sliding planes were steeply angled, thereby reducing the resisting forces within the active driving block.

Stability Analysis Methods

We utilized different stability analysis methods with two soil strength models in developing a working model that fits the stable 1996 geometry and the assumed non-failure geometry of the landfill on the morning of August 6, 1997, just prior to failure. The analysis methods and their results follow.

<u>Undrained Shear Strength Stability Analysis Using LTV's 1997 Geotechnical Data</u>

We first performed a total stress analysis using previously reported undrained shear strength values. We selected LTV cross-sections A-A and B-B, as shown in plan on Figure 10, to evaluate existing 1996 conditions and LTV’s proposed 1993 and 1997 landfill closure geometry. We adopted LTV’s engineer’s adopted frictional strength of 35 degrees for the waste fill and we adopted D’Appolonia’s undrained shear strength of 1,200 psf for the silty day strata at these locations since both sections were not previously filled above El. 600 to 610 prior to 1991. The analysis used the computer program XSTABL with wedge block failure surfaces to model failures through upper and lower portions of the silty clay strata.

The 1993 analysis was performed on section B-B only and is presented on Table 1 and Figure 27 using the 2H:l V slopes from El. 600 up to 650, and assuming rapid undrained loading. Computed FS were between 0.7 and 1.0, less than the 1.3 to 1.5 required by local engineering standards, indicating that LTV’s December 1993 design would not have been successful unless filling was slow enough to allow consolidation in the silty clay to increase the undrained shear strength the necessary amount for stability.

The analysis using actual 1996 grades and the LTVs accepted 1997 closure grades are shown on Figures 28 and 29 for cross-sections A-A and B-B. The computed factors of safety against sliding (FS) were less than unity (1.0) for surfaces passing through the lower portion of the silty clay stratum using the July 1996 landfill geometry, as summarized in Table 3. The computed FS are less than industry standard of 1.3 at end of construction and 1.5 for long term conditions (IDEM requires a minimum FS of 1.5, subject to possible reduction upon review and degree of certainty of soil strength).

Using the LTV proposed closure geometry, we computed a FS range of 1.0 or less for a failure surface passing through the upper portion of the silty clay stratum and approximately 0.8 for a failure surface passing through the lower portion of the silty clay stratum. This simple landfill stability model, based on pre-failure 1996 exploration data, the adjacent earlier D'Appolonia data (e.g., 1964 Northwest Fill Area design), and knowledge of the undrained shear strength in the silty clay under the man-made fill placed over Lake Michigan at the nearby steel mill , shows a computed FS less than unity. These results are far less than the minimum industry standard FS of 1.3 for end of construction conditions.

Because landfill failure was expected using the USA approach, and because an assumed uniform shear strength undrained analysis may be overly conservative since filling may have caused clay consolidation and drainage to occur, we next chose to perform ESA analyses to model excess pore pressures.

<u>Effective Stress Analysis to Match August 6, 1997 Instability Conditions</u>

We performed the effective stress stability analyses using the computer program XSTABL with clay strata friction angles and extrapolated pore pressure ratios. Our pore water pressure values were based on extrapolated data from LTV and STS installed piezometers. We defined 13 to 17 shear strength boxes to zone the silty clay layers and assign R_u coefficients to each zone in the four stability sections (A-A thru D-D). The material friction angles and pore water pressure coefficients (R_u) were assigned to each zone based on extrapolating piezometer readings from fall of 1997 and spring of 1998 back to August 6, 1997.

The ESA method of analysis, using LTV's estimated ground surface grading for August 6, 1997 yielded FS less than 1.0 for Sections A-A, B-B and C-C; and greater than 1.0 at Section D-D. Table 4 and Figures 30 through 33 show the results of the ESA analysis for Sections A-A through D-D, respectively. The ESA model fits the observed conditions reasonably well with FS near unity; we recognize that some small variations from this model are possible due to anisotropy in the silty clay, variation in waste strength properties, and variations in pore pressure in the silty clay. Furthermore, as indicated in Table 3, the less complex TSA analysis agreed well with the ESA analysis for failure at sections A-A and B-B.

Reasons for Landfill Failure

The landfill failed at the southwest fill area due to excess pore pressure induced by rapid loading over and adjacent to the southwest portion of Clark Landfill, at the north end of LTV section A-A. Between July, 1996 and August 6, 1997 the north end of LTV section A-A was raised approximately 40 feet in attempts to achieve the 3H:1V design fill slope. A lesser amount of fill was added over section B-B. Section C-C was actually slightly unloaded along the crest of the landfill (6 to 8 feet) prior to failure. The silty clays beneath sections A-A, B-B, C-C and D-D were normally or under-consolidated prior to failure, as determined by consolidation tests on unfailed samples of silty clay beneath and beside the Clark Landfill. The regrading and filling called for in the LTV 1997 closure plan caused positive pore pressure development in normally and under-consolidated, contractive, silty clay.

The 1997 filling rates at the southwest area were more rapid than during any earlier period of filling from 1971 to 1996. Figures 34 through 36 show this rapid filling condition at two locations along Sections A-A and B-B. Based on these figures, the average filling rate between 1991 and 1996 ranged from 2 to 6 feet per year in the southwest fill area. The average filling rate between 1996 and 1997 called for by the LTV closure plan ranged from 19 to 25 feet per year, as shown on Figures 35 and 36. Failure occurred in 1997 due to accelerated loading and higher driving stresses caused by increasing fill heights, resulting in high excess pore water pressure in the foundation silty clay without sufficient time for dissipation.

Consequences of Landfill Failure

As a result of the landfill failure, the silty clay had lower post-peak shear strengths that had to be considered in the final closure design. Figure 37 shows the expected reduced shear strengths in terms of the undrained soil strength model. For example, the average peak undrained shear strength prior to failure under the southwest fill area which was 1,000 to 1,300 psf was likely reduced to an undrained strength of 800 psf. As shown on Figure 26, the peak average friction angle was 21 degrees and the residual friction angle was measured to be approximately 13 degrees. Due to these reduced strengths, it would be impossible to excavate the slide in and out of the flume to establish original grades and achieve a minimum factor of safety of 1.3 during construction. The side slopes of the failed landfill portion needed to be flattened, with excess material removed from the top of landfill and placed along the north perimeter of the fill. During the closure regrading, piezometer and inclinometer monitoring were required to ensure acceptable foundation clay performance.

Landfill Closure and Monitoring

Between 1998 and November 2007 Clark landfill was re-graded and capped in accordance with an IDEM approved closure design prepared by the authors. The landfill was re-graded to achieve flatter slopes as shown in plan on Figure 38 and the re-graded north side slopes on Figure 39. Figure 40 shows cumulative inclinometer movements at section C-C are stable since inclinometer LTV 210 shows two years of no slide plane movement 66 feet below grade (El. 525). The final fill geometry and filling rate was designed to result in computed effective stress stability of the re-graded fill of FS of 1.5 at the end of final closure. A summary of deep piezometer pore water dissipation under south side failure sections A-A, B-B and C-C is shown on Figure 41 for the period of 1998 to 2010 with closure chronology events. Figure 42 shows a cross-section on the north side at landfill sections G-G and H-H for the period of 1998 through 2010 with chronology of activity at the site. These figures clearly show the slow rate of consolidation and upward draining behavior of the lake bed silty clay. The pore pressure dissipations behavior matches the single drainage upward type consolidation characteristics described by D'Appolonia in 1964. The Clark Landfill remediation and closure program was approved by IDEM and the fill remains stable today.

Summary and Conclusions

1. The landfill failure occurred in the upper Lake Border glacial lake bed silty clay till as a result of excess pore pressures caused by steeper fills due to landfill regrading, specifically new fill placed rapidly over the southwest corner of Clark Landfill.
2. The 1997 regrading plan could not have been safely completed in the one year time frame from early 1997 through May 1998. If Clark Landfill had been instrumented in early 1997 with inclinometers and piezometers in clay the resultant movements and excess pore water pressures could have warned the engineer and LTV of impending failure and could have stopped filling, or removed fill.

References

1. Peck, R.B. and Reed, W.C. (1954). "Engineering properties of Chicago Subsoils," University of Illinois Engineering Experiment Station Bulletin No. 423, 62 pp.
2. Terzaghi, K., Peck, R.B. and Mesri, G. (1996) Soil Mechanics in Engineering Practice, Third Edition, New York, John Wiley and Sons, 549 pp.
3. Taylor, D.W. (1948). Fundamentals in Soil Mechanics, New York, John Wiley and Sons, 700 pp.
4. LTV files containing Northwest Fill and Clark Landfill related references.