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ABSTRACT

The McClusky Canal, located in central North Dakota, crosses the divide between the Hudson Bay and Gulf of Mexico drainages. The 74 mile (120 km) long canal has a maximum cut depth of approximately 115 feet (35.1 m) and an average cut depth of 40 ft (12.2 m). Slope failures in deep cuts of high plasticity clay have occurred sporadically since its construction in the 1970s. In addition, a large number of deep failures occurred during construction and recently. The slope failures are examples of the classic problems of construction slope instability and progressive failure. This paper chronicles half a century of geotechnical engineering related to the McClusky Canal beginning with feasibility design investigation in the mid 1950s. It includes discussion of stability methods, stability investigations, and remediation efforts of the 1950s, 60s, 70s, 80s and 2000.

INTRODUCTION

The Bureau of Reclamation constructed McClusky Canal, located in north central North Dakota, in the 1970s as an irrigation waterway. Numerous slope failures have occurred throughout its 30-year history. Many occurred in clay cuts during, or shortly after construction and provide good examples of the classic problem of construction induced slope instability. Subsequent slides typically occurred during wet periods of the year or after large rainfall events. Although the frequency of slides is undocumented, long-employed maintenance personnel suggest a general trend towards an increasing number of slides per year, with many slides being associated with a large storm event which occurred in 1993. These latter slope failures are examples of progressive failure.

Background information related to the canal is presented. This includes a description of the geologic setting and a brief history of canal subsurface investigations and slope stability analyses. Next, a recently completed subsurface soil and groundwater investigation for a short, deep and unstable cut section of the canal is summarized. This is followed by a discussion of the results of recent slope stability analyses.

BACKGROUND

McClusky Canal was constructed in the early 1970s. Hundreds of slides, concentrated at numerous locations, have occurred in cut-slopes. An approximately 1.6 mile (2.6 km), 80 ft (24 m) deep canal section, referred to herein as Reach 2A, has experienced extreme slope stability problems and was the focus of a recent investigation. Reach 2A is located between canal stations 1120+00 and 1206+00[1] and will be used herein to exemplify stability problems.

The project, as originally designed, had the potential to irrigate in excess of one million acres of land. Water pumped from Lake Sakakawea into Audubon Lake would flow by gravity through the McClusky Canal to Lonetree Reservoir and then through two other canals to the principal areas to be irrigated. The 74 mile McClusky Canal has a design capacity of 1950 ft^3/sec with a water depth of approximately 17 ft (5.2 m), a bottom width of approximately 25 ft (7.6 m), and maximum water surface width of approximately 94 ft (2.9 m). The maximum and average depths of excavation for the canal are approximately 115 ft (35.1 m) and 40 ft (12.2 m) respectively. Slopes were constructed and/or cut at a slope of two horizontal to one vertical (2:1) along the canals entire length.

A representative section of the canal is presented on Fig. 1. The design canal invert elevation in Reach 2A is approximately 1829 ft (557.5 m); the O&M Road elevation is approximately elevation 1852 ft (564.5 m); and the original ground surface elevation varied between 1875 ft (571.5 m) and 1910 ft (582.2 m). A waste bank consisting of soil excavated during canal construction is located adjacent to the canal and has a maximum height of approximately 40 ft (12.2 m).

[1] Canal stationing reflects the distance from the canal inlet in units of feet. For example, Station 1206+00 is approximately 120,600 ft (3.84 km) from the canal inlet.

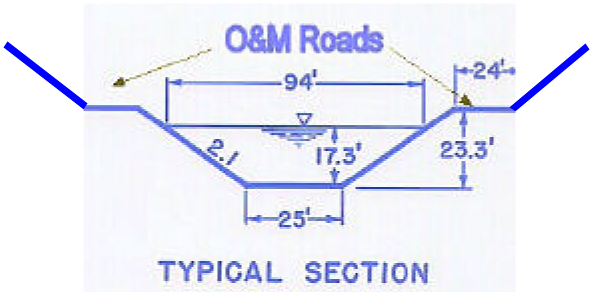

Fig. 1. Representative Section of the Canal.

Figure 2 portrays a centerline profile of Reach 2A showing the waste bank, original ground surface elevation, top of cut after construction, canal invert information. The stratigraphy shown in this figure will be discussed later.

Numerous slope failures have occurred in Reach 2A throughout its approximate 30 year history. Some slides occurred during and shortly after construction, however, most occurred later. Often the latter slides occurred during periods of rapid snowmelt or after large precipitation events. Longitudinal tension cracks along the crest of the slope or in the waste bank were often observed for months and/or years preceding slides. In the winter these cracks would fill with ice. Perched water seeps into the canal cuts at several locations. Figure 3 presents an aerial photograph taken in May 1999. Many existing slides are apparent. Continued landslides and slope modifications have altered the shape of the canal since the photograph was taken.

General Geology

North Dakota was covered with glacial ice during the Pleistocene epoch. Advancing glaciers and meltwater, associated with multiple glacial and interglacial stages, transported rock and soil material. In North Dakota four main types of glacial material were deposited: tills, glaciallacustrine (lake-related) sediment, glacialfluvial (outwash), and eolian (wind-blown) sediment. The action of the glaciers and melting ice tended to form a glacial sediment referred to as till which consists of boulders, cobbles, pebbles, sand, silt, and clay. Lake sediment consists chiefly of silt and clay. Glacial river and stream outwash sediment consists chiefly of sand and gravel and wind-blown sediment consists of sand and silt (Knodel, 1977).

The McClusky Canal, Reach 2A area is located in a physiographic region called the Missouri Coteau. Thick and extensive glacial deposits and features unconformably mantle Paleocene and older bedrock (Knodel, 1977). Bluemle (1991) describes the Missouri Coteau as "hummocky, glaciated irregular plains that resulted from collapse of superglacial sediment; gentle slopes characterize 50 to 80 percent of the area and local relief ranges from 100 (30 m) to 300 feet (91 m). The Missouri Coteau also forms a topographic high that divides Hudson Bay and Gulf of Mexico drainages and therefore is not transected by streams.

History of Slope Stability Investigations

The following history summarizes literature obtained by a search of Bureau of Reclamation records.

The first study undertaken to determine safe side slopes for the

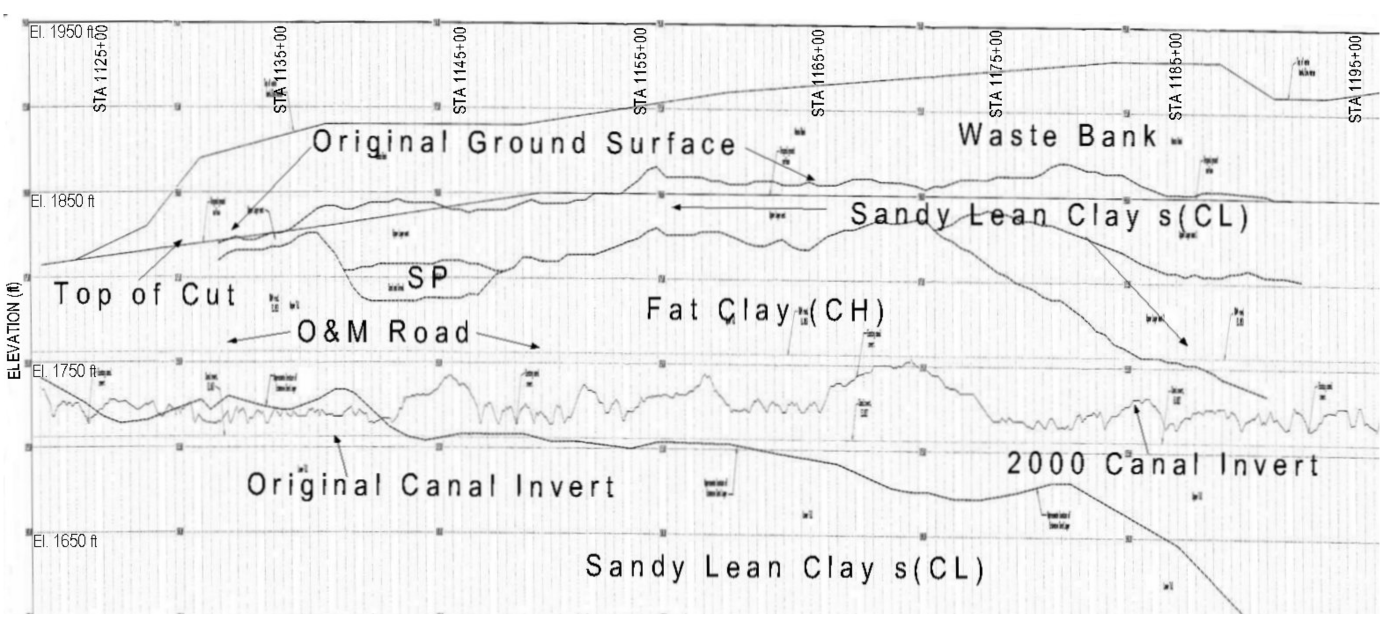

Fig. 2. Canal Centerline Profile and Stratigraphy

McClusky Canal was reported in 1956. It documented feasibility-level investigation of soils along the proposed canal alignment. Unconfined compression, triaxial shear, and index property tests (particle size analyses, liquid limit, and plastic limit) were performed on undisturbed samples. A series if slope stability analyses were performed that considered side

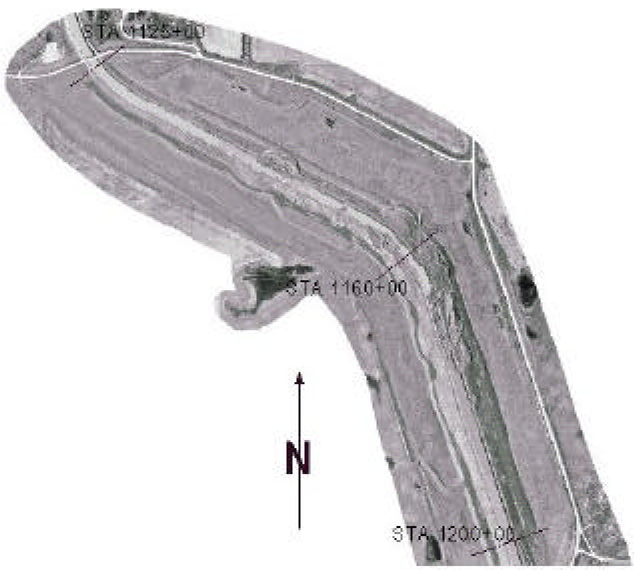

Fig. 3. 1999 Aerial Photograph of Reach 1

slopes of 2:1, 2 ½ :1, 3:1, and 4:1 and cut-slope depths of 50, 60, 75, and 100 ft (15, 18, 23 and 30 m) using the "Swedish Slip Circle method". A safety factor was determined for each combination of depth and slope using unconfined compressive strengths of 10, 15, 20 and 25 lb/in^2 (69, 103, 138 and 172 kN/m^2) to represent soil strength. A safety factor of 1.3 was considered acceptable. Comparing the results of these analyses with the results of unconfined compression tests resulted in the suggestion that, for feasibility estimates, 1-1/2:1 cut-slopes may be used for slopes with heights 40 ft or less; for deeper cuts a 2:1 slope would be adequate. Stability was evaluated for 14 canal sections. Three of these analyses suggested the need for flatter slopes. Hence, the investigation concluded "There will be many relatively short reaches in deep cuts which will undoubtedly require much flatter slopes, depending not only on the strength of the materials, side slopes, and cut depths but also on the method and speed of construction." (USBR, 1956).

Two 1968 reports summarize the result of preconstruction investigations and analyses conducted between 1962 and 1968 (USBR 1968a, USBR 1968b). Drill holes were spaced one mile apart along the approximately 74 mile proposed canal alignment. Soils were visually classified and field moisture and unit weight determinations were made. Numerous vane shear, direct shear, triaxial shear, and consolidation tests were performed.

Slope stability was evaluated using both the Bishop and Fellenius[2] methods. Total and effective stress analyses were performed. The first report concludes that 2:1 slopes are generally acceptable. However, the second, using data representing a proposed deeply cut canal segment between stations 2974+00 and 3091+00, predicted marginally acceptable slope stability at a 2:1 slope.

The canal was constructed in the 1970s using a 2:1 slope for all cut sections. It is not known why a 2:1 slope was selected for all location given the documented concerns regarding slope stability. It may have been perceived that high safety factors were over-protective and only a few slides would actually occur. Thinking this, it may have been considered economical to remediate a few slides after canal construction rather than flatten the slopes at the time of construction.

Major slope failures were noted between stations 1661+30 and 1673+50 shortly after excavation in the fall of 1973. In the fall of 1974, a large slide occurred on the left side of the canal prism between stations 1206+61 and 1216+40; a short distance beyond the southern end of Reach 2A. A photograph of this latter slide is provided on Fig. 4. An ensuing investigation, summarized in a 1977 report, concluded "The results of the analysis indicates that canal slopes should be cut back to either a 3:1 or 4:1 slope" (USBR 1977). In addition it was recommended that the Waste bank be moved farther from the canal. The study used the Spencer Method for slope stability analyses. Pore water pressures for effective stress analyses were estimated using Ru values (Ru is the ratio of

Fig. 4. 1974 Slide Located Beyond the South End of Reach 2A. Reach 2A is visible in the background.

[2] The Fellenious Method is also known as the Ordinary Method of Slices and the Swedish Circle Method (Lamb, Whitman 1969)

pore water pressure to overburden pressure) to represent different soil types.

A month after that report was written, in July of 1977, a large landslide occurred between stations 3291+00 and 3293+50. A study ensued and the subsequent 1979 report of investigation demonstrated the effect of water table elevation on slope stability. The water table elevation within the slope was varied in the reported slope stability analyses. The water table elevation, which best predicted a safety factor equal to 1.0 (imminent failure), was selected to represent the condition possible at the time of failure (USBR 1979). However, there was no direct evidence to suggest such a water table did, or even could, exist.

In the late 1970s and early 1980s more than 53,000 ft of horizontal wells were installed to reduce pore pressure in the slope. Very few of the wells produced flowing water. Nearly all of those that did became encrusted with precipitates and stopped flowing within a few years.

Slides continued to occur at a rate estimated to exceed 10 each year. Other memorandums and project notes found in Bureau of Reclamation archives highlight efforts to understand the causes of slope failures. References to ground water and aerial studies were found, however, documents reporting these activities were not located. Surface and shallow subsurface horizontal drains were suggested in some internal memorandums as a method to control groundwater thereby reducing risk of slope failure. Several stabilization measures suggested were implemented including flattening slopes, installing surface drains, and eliminating ponded water behind the waste banks. However, slope failures continued, including some in sections where remedial measures aimed at controlling groundwater were undertaken.

It was noted by operation and maintenance personnel that slide activity was correlated to precipitation events. The Bureau of Reclamation annual 1998 Operation and Maintenance Review makes the following statement regarding recent slides; "Most of the (more recent) slides along the McClusky Canal occurred in 1993, after a very large rainfall event, which local landowners estimated to be 12 to 14 inches of rain within a 24 hour period." Aerial photographs, taken a day after the storm, show large pools of standing water in farmland located adjacent to the canal. Figure 5 shows a recent 90 ft high slide near the center of Reach 2A at Station 1160+00.

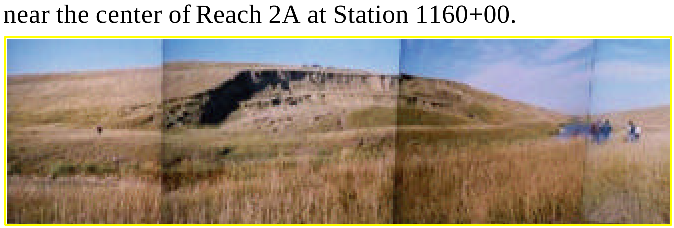

Fig. 5. Recent Slide on Right Side of Reach 2A Near Station 1160+00.

Operation and maintenance personnel have noted a frequent occurrence of slide remobilization. This is due, in part, to the practice of excavating the toe of slides to eliminate canal blockage.

Slides continue to occur in canal cut-slopes. In 2000, an investigation was performed that focused on Reach 2A, a section that has experienced serious slope stability problems. While previous investigations focused primarily on construction-induced failure and the hypothetical effects of water table fluctuations on slope stability, this investigation focused on the progressive slope failure mechanism. The remainder of this paper discusses this later study.

REACH 2A SUBSURFACE CHARACTERISTICS

It is well known that the long-term stability of cut-slopes in fissured, high plasticity clay is commonly less than would be predicted by stability analyses using peak shear strength parameters. Stark and Eid (Stark *et al*. 1997) state: "A possible explanation for this result is that softening, which is the reduction in the available drained strength resulting from an increase in water content under constant effective stress, reduces the shear strength of the mass to the fully softened value. After reaching the fully softened shear strength, progressive failure reduces the average shear strength along the failure surface to a value between the fully softened and residual shear strength values." Stark and Eid have proposed a method for performing a slope stability analysis that accommodates the concerns of clay softening and progressive failure. This method was implemented to evaluate several existing Reach 2A slides and predict the stability of the remaining intact Reach 2A cut-slopes. A subsurface investigation was performed to collect appropriate geologic and geotechnical information for this purpose. This consisted of cone-penetrometer tests, drilling and sampling, laboratory analyses and groundwater characterization.

Reach 2A Subsurface Investigations

Seventy-three cone penetrometer holes were pushed to depths of 50 to 100 ft to characterize soil type, soil strength, stratigraphy, and ground water conditions in Reach 2A. A typical cone log is provided on Fig. 6

This log represents a location where a cone was pushed through the center of a slide. In this instance the disturbed slide material is readily distinguished from the underlying, stiffer, undisturbed soil. This type of information was very helpful in establishing slide geometries.

A soil sampling and testing program was performed immediately after the field portion of the cone penetrometer investigation was completed. The program consisted of 28 drill holes; twenty-five of these holes were selected for observation well installations. Soil samples, obtained using hollow-stem auger methodology, were visually classified. Push-tube samples were tested in the laboratory to classify soils and determine unit weights. Load-displacement behavior and shear strength was determined by consolidated undrained triaxial shear, direct shear, and one-dimensional consolidation tests.

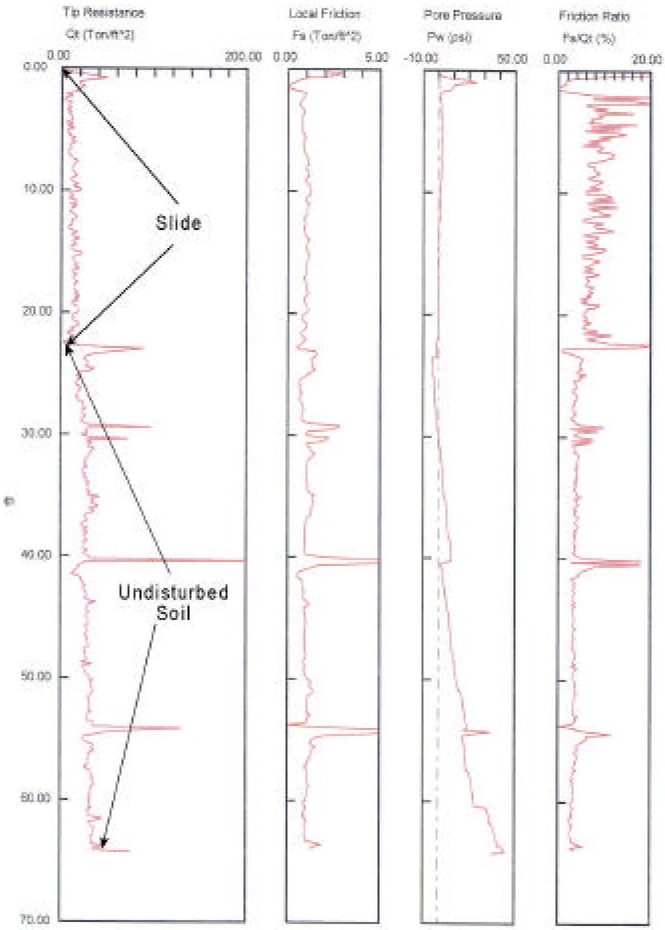

Fig. 6. Example Cone Penetrometer Log

<u>Reach 2A Stratigraphy</u>

Local soil strata are approximately planer and dip to the south at about 1 degree. The canal is directed east through the first half of Reach 2 and then turns slightly to the south near the midpoint as seen on Fig. 3. Consequently, soil layers observed on the cut face of the canal appear horizontal through the first half of Reach 2, and appear to dip in the later half. Figure 2 portrays the general stratigraphy along the Reach 2A canal alignment using an exaggerated vertical scale.

Investigations revealed that a 5- to 60-ft thick surficial deposit, consisting primarily of Sandy Lean Clay, s(CL), overlays a 40- to 50-ft thick layer of Fat Clay (CH). The upper layer supports deep tension cracks along the crest of the canal that have been observed to fill with ice in the winter. Below the Fat Clay lies a thicker glacial deposit composed primarily of Sandy Lean Clay s(CL). Figure 7 demonstrates the difference in plasticity and residual shear strength between the Fat Clay (CH) and the underlying Sandy Lean Clay s(CL). A layer of Poorly Graded Sand (SP), typically less than 0.5 ft thick, separates the Fat Clay (CH) and the lower Sandy Lean Clay s(CL). This layer is thin and is not shown on Figure 2. A Poorly Graded Sand (SP) is also exposed in the canal cut in the vicinity of canal station 1145+00 and appears to be a buried steam bed.

The Sand (SP) layer separating the Fat Clay (CH) and the underlying Sandy Lean Clay s(CL)can be approximated as an undulating planer feature having several feet of relief and dipping to the south at about 1 degree. It intersects the canal invert near station 1148+00. Observation well data indicates that water flows from the canal into this sand layer at this location. Stability analyses suggest that water pressure fluctuations within this sand that result from changes in canal water level elevations may significantly influence slope stability.

A thin soft layer of Fat Clay (CH) (typically less than 6-inches thick) is located just above the Sand (SP). This layer is thought to be a superglacial sediment. Cone penetrometer data, interpretation of slide geometries from surface topography and cone data, and the observation that several PVC wells have

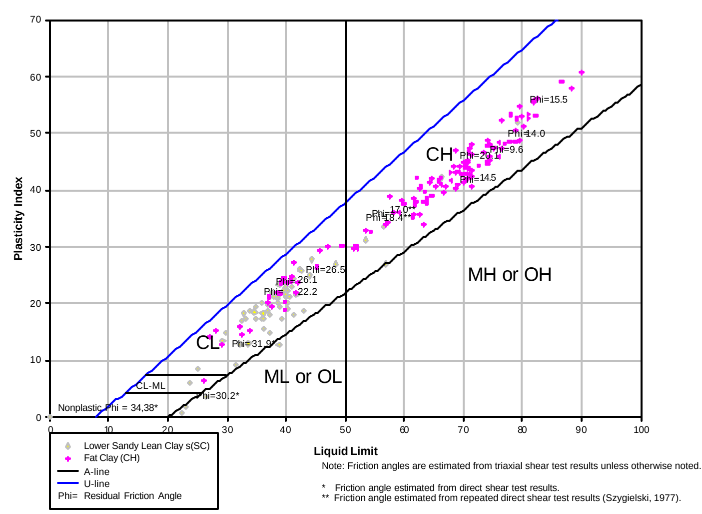

Fig. 7. Plasticity Chart showing differences between Fat Clay (CH) and Lower Sandy Lean Clay s(CL) layers.

sheared just above the sand layer suggests that shear surfaces have a tendency to develop in this thin soft clay layer.

Laboratory testing focused on acquiring data needed to analyze Reach 2A slopes using the method proposed by Stark and Eid. By this method, progressive slope failure is presumed to occur on a slip surface predicted by the stability analysis that yields the lowest safety factor calculated using a soil peak strength model. Once the failure surface is defined, soil strength at the time of failure is represented by a value between the fully softened and residual strength. Hence, estimates of peak, fully softened, and residual strength were required.

Consolidated drained triaxial shear tests were performed on relatively undisturbed specimens of Fat Clay (CH) and specimens of the overlying and underlying Sandy Lean Clays s(CL). Figure 8 depicts bilinear strength models for the Fat Clay (CH).

The peak shear strength and residual shear strength models represent bilinear models fitted to test results. Also presented on Figure 8 is an empirically derived bilinear model representing estimates of fully softened (FS) strength predicted using liquid limit, plasticity index, and particle size information (Stark *et al*. 1997).

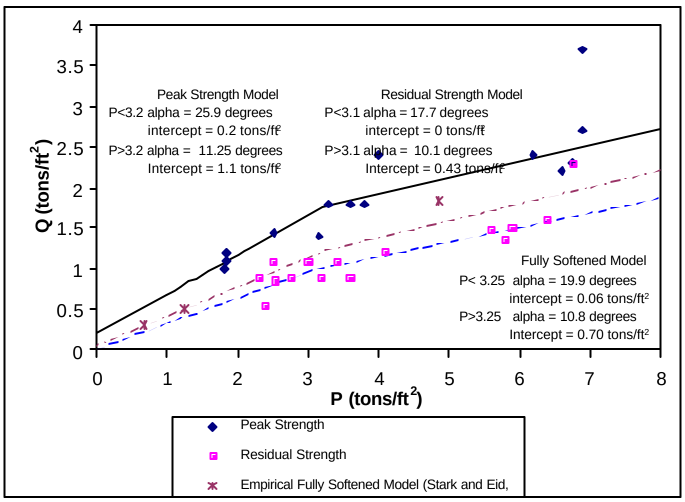

Fig. 8. Soil Bilinear Shear Strength Models.

A linear strength model (Phi = 24.2, c = 1000 lb/ft^2), estimated from the peak shear stress in triaxial shear tests, was used in stability analyses to represent the upper Sandy Lean Clay s(CL). A deep tension crack was always assumed in stability analyses, hence the material strength of the waste bank had little impact on the outcome. The waste bank strength was modeled with Phi = 33 and c = 0. Because all slip surfaces occurred above the lower Sandy Lean Clay s(CL) layer, a strength model for this material was not needed for stability analyses.

<u>Reach 2A Groundwater</u>

Regional groundwater is located far below the canal invert in the vicinity of Reach 2A. Locally, water is perched in sand lenses contained in the upper Sandy Lean Clay s(CL) and flows horizontally towards the canal. Long periods of precipitation and periods of snowmelt cause the perched water level to rise. Observation well data indicates that sand lenses near the surface respond rapidly to rainfall events while deeper sand layers are unaffected. The exception to this is the sand layer between the Fat Clay and lower Sandy Lean Clay s(CL) that is recharged by the canal near the canal invert. Water pressure in this layer responds to canal water surface elevation, which, to a small extent, responds to precipitation events.

A typical groundwater model for Reach 2A, characterized by equipotential lines, is shown on Fig. 9. Models were created using the computer program SEEP/W^{TM} (SEEP/W 1999). Figure 9 represents expected groundwater flow in a Reach 2A canal slope section during periods of normal periods of rainfall. Several models representing different cross-sections of Reach 2A were created. Similar models were made to represent periods of extended precipitation. Pore pressures for effective stress slope stability analyses were obtained directly from these groundwater models.

RESULTS OF REACH 2A STABILITY ANALYSES

The computer program SLOPE/W^{TM} was used to perform stability analyses (SLOPE/W 1999) for numerous locations in Reach 2A. Circular and noncircular slip surfaces were analyzed by the Spencer method. The top of the waste bank and original ground surface varied with the cross-section being considered, having maximum values of 1935 ft (590 m) and 1920 ft (585 m) respectively. The toe of the Waste Bank was always located 40 ft (12 m) from the crest of the slope. The face of the Waste Bank was given the same slope as the canal cut. Twenty feet (6.0 m) wide O&M roads and the canal invert were fixed at elevations 1853 ft (565 m), and 1827 ft.(557 m) respectively.

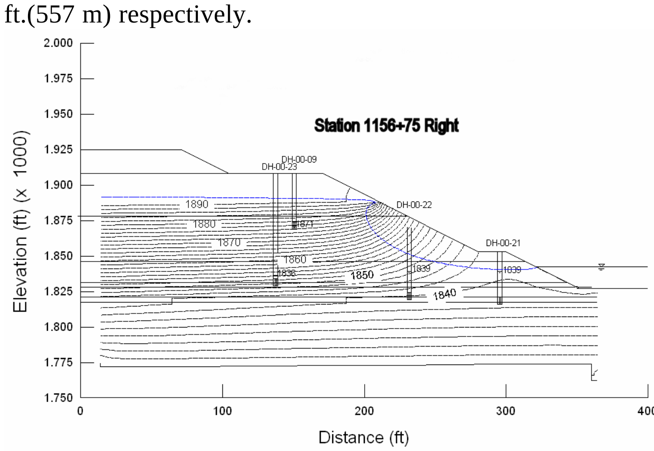

Fig. 9. Equipotential Lines and Groundwater Elevations Representing Hypothesized and Measured Groundwater Conditions Respectively.

Stability was analyzed for many canal sections and many conditions. Groundwater conditions considered included rapid drawdown of the canal, changes in precipitation rate and duration, tension crack depth and fill, and the proximity of the soft clay layer above the sand relative to canal invert The analyses also addressed intact and existing slides. One existing slide, entirely in Fat Clay (CH), having its base at the elevation of the O&M road was remobilized when O&M crews removed soil to clear the road for the field investigation. This simple remobilized condition was also modeled to help validate the modeling effort.

All intact slope evaluations resulted in safety factors greater than one when peak shear stress model was used to represent the Fat Clay (CH). Conversely, all intact slope evaluations yielded safety factors less than one when using the method suggested by Stark and Eid (1997).

In nearly all cases in which the Stark and Eid method was applied it was observed that the failure surface resulting in the lowest safety factor, rounded to the nearest hundredth was not unique. Typically, when using the soil peak shear strength model, there was a family of slip surfaces having a common base elevation that yielded nearly the same minimum safety factor. This could explain the numerous shapes of observed failure surfaces and also suggests that Reach 2A slides might occur in a progressive manner with small failures unloading the toe of the slope and thereby initiating the movement of larger soil masses.

The models effectively demonstrated that slopes where soil softening and movement has progressed to the extent that a slide is impending are very sensitive to changes in groundwater caused by precipitation, snowmelt, and/or canal water surface fluctuations. Small changes in groundwater conditions could initiate the slide. However, the models also suggest that the slopes will inevitably fail due to clay softening even if these events do not act to trigger them.

One of the striking features of the Reach 2A slides is that they become larger in a downstream direction and then stop altogether near the lower end of the Reach. This is, at least in part, due to the location of the sand layer relative to the canal invert. The base of most existing slides is the top of the sand layer where there exists a soft layer of Fat Clay (CH). This layer dips to the south. Consequently the soft layer is above the invert at the upstream end of Reach 2A and below the invert and the downstream end. Slides become deeper and wider with increased canal stationing. Near the downstream end of the reach, the opposite canal bank interferes with the development of the shear surface thereby increasing the calculated safety factor. Despite this improvement, the safety factor for progressive failure remains less than one suggesting a slide is inevitable. Tension cracks have been recently observed at the crest of the left slope at the downstream end of Reach 2A.

The effect of ice in tension cracks is unknown. Repeated thawing and freezing, and/or migration of moisture towards the crack followed by freezing, are mechanisms that could induce pressure on the crack face and promote slope movement. Investigating the effect of ice formation in tension cracks on slope failures was beyond the scope of the subject investigation.

Another mechanism that may accelerate slope failure in the subject section of the canal is associated with heave of the underlying clay due to excavation. Reduced stresses in the underlying Sandy Lean Clay s(CL), caused by canal excavation, will cause the clay volume to increase. The vertical heave resulting from this volume change will decrease with increasing distance from the canal centerline. This type of movement is expected to cause a horizontal strain within the slope acting in a manner that would tend to reduce stress. Consequently, slope instability is promoted because the frictional component resisting slide action is reduced and the increase in volume and moisture content associated with reduced stress would result in clay softening.

SUMMARY

The McClusky Canal is located in central North Dakota and crosses the divide between the Hudson Bay and Gulf of Mexico drainages. The 74 mile (120 km) long canal has a maximum cut depth of approximately 115 feet (35.1 m) and an average cut depth of 40 ft (12.2 m). Numerous slope failures have occurred throughout its 30-year history. Many occurred in clay cuts shortly after construction and provide a good example of the classic problem of construction induced slope instability. More recent slides provide a good example of progressive failure.

A Feasibility Study for the proposed McClusky Canal was performed in the mid-1950s. Slope stability analyses indicated potential problems in deep-cut canal sections. Design-level investigations performed in the 1960s approved the use of 2:1 cut-slopes, however, stability analyses still indicated some canal sections may experience slides. The canal was constructed with 2:1 slopes in the 1970s and construction induced slides were common. Slides continued well past the construction period and studies ensued. Remedial measures such as flattening slopes, improving surface drainage, dewatering using horizontal drains were implemented, yet slides continued. A major storm event in 1993 triggered a large number of slides, and in 2000 an investigation of the worst section of canal, Reach 2A, was performed.

Reach 2A consists of an approximately 1.6 mile, 45 to 80 ft deep canal cut with an adjacent 40 ft high waste bank. Reach 2A initially tracks east but turns south at its midpoint. The mobilize mass of slides generally increases with increased canal stationing until the canal turns south. Shortly after the canal turns south the incidence of slides nearly ceases. Large tension cracks, sometimes filled with ice, are observed to precede slides. Some slides have been remobilized by the practice of excavating the toe of the slide to clear the canal. Concerns also included slides induced by rapid drawdown of the canal, major precipitation event, and snowmelt.

The 2000 investigation focused on progressive slope failure as the cause of recent slope instability. The investigation began with a detailed subsurface investigation that included 73 cone penetrometer holes, 28 drill holes, and 25 observation well installations. Laboratory testing consisted of triaxial, direct shear, consolidation, and index properties tests. The cone penetrometer provided detailed stratigraphy and laboratory tests provided soil strength information necessary for stability analyses. Groundwater models were created to reflect conditions associated with normal precipitation and extended precipitation events.

A method proposed by Stark and Eid (1997) was implemented to evaluate slope stability. The results of stability analyses suggest that all remaining 2:1 slopes within Reach 2A will eventually fail as the available shear strength of high plasticity clay is reduced by a progressive failure mechanism. The analyses also provides explanations for the observed increase in the size of slides with canal stationing followed by a near total absence of recent slides near the southeastern end of the reach. It is also suggested that ice formation in tension cracks and time-dependent heave of underlying clays may adversely impact slope stability.

REFERENCES

Bluemle, J.P. [1991]. "The Face of North Dakota", revised edition, Educational Series 21, North Dakota Geological Survey, Bismark, ND.

Bureau of Reclamation [1956]. "*Laboratory Studies of Subsurface Materials, McClusky Canal – Garrison Division, Missouri River Basin Project*", Earth Laboratory Report No. EM-451, Division of Engineering Laboratories, Denver, CO, April 25, 1956.

Bureau of Reclamation [1968a]. "*Laboratory Studies of Foundation Materials from McClusky Canal – Garrison Diversion Unit, Missouri River Basin Project, North Dakota*", Report No. EM-751, Division of Research, 17 Denver, CO, January 3, 1968.

Bureau of Reclamation [1968b]. *Laboratory Studies of Foundation Materials from Reach 3C - Station 2974+00 to 3091+00, McClusky Canal – Garrison Diversion Unit, Missouri River Basin Project, North Dakota*, Earth Laboratory Report No. EM-762, Division of Research, Denver, CO, July 1968.

Bureau of Reclamation [1977]. "*Stability Analyses of Landslides at Stations 1206+61 (376+78 m) to 1216+40 (370+76 m) and Stations 1661+30 (506+36 m) to 1673+50 (510+08 m) - McClusky Canal, Reach 2 - Garrison Diversion Unit, Pick-Sloan Missouri River Basin Program, North Dakota*", Earth Sciences Reference No. 77-42-19, Division of Design and Division of Research, Denver, CO, June 9, 1977.

Bureau of Reclamation [1979]. "*Results of Laboratory Testing of Soil Samples and Stability Analysis of Landslide at Station 3291+00 to 3293+50 - McClusky Canal, Reach 4A - Garrison Diversion Unit, Pick-Sloan Missouri Basin Project, North Dakota*", Geotechnical Branch Reference No. 79-24, Division of Design and Division of Research, Denver, CO, October 1, 1979.

Bureau of Reclamation [1998]. Gemperline, M.C. and S. Robertson, "*Participation in the McClusky Canal Modified Review of Operation and Maintenance (RO&M)*", Bureau of Reclamation Travel Report, Denver, CO, October 30, 1998.

Knodel, P.K. [1977]. Slide Presentation Titled "Stability of Deep Cut Sections at McClusky Canal", Reclamation Geotechnical Group Files, Denver, CO.

Lambe, W.T. and RV. Whitman [1969]. "*Soil Mechanics*", John Wiley & Sons, Inc., New York, NY.

SEEP/W [1999]. Version 4.23, Computer Program Documentation, GeoSlope International, http://www.geo-slope.com.

SLOPE/W [1999]. Version 4.23, Computer Program Documentation, GeoSlope International, http://www.geo-slope.com.

Stark, T.D., T.E Hisham. Stark *et al*. [1997] "Slope Stability Analysis in Stiff Fissured Clays", *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 123, No. 4, April, 1997, pp. 335-343.