SLOPE INSTABILITY STUDY
SOUTH SASKATCHEWAN RIVER BANKS
SASKATOON, SASKATCHEWAN

MEEWASIN VALLEY AUTHORITY

FINAL REPORT

FILE S134 23 DECEMBER 1985

The City of Saskatoon is making available for your information a general study of parts of the east riverbank area conducted in 1985 by the Meewasin Valley Authority and the City of Saskatoon, as well as studies conducted for the City of Saskatoon in 2012 and 2013 for parts of the area between 11th Street East and Saskatchewan Crescent.

These reports are provided as a courtesy only. There have been significant changes in ground water levels as well as actual slope failures in recent years; therefore, the information contained in all studies must be regarded with caution and with the assistance of external experts. The City makes no representation that these reports reflect the current condition of the area.

City of Saskatoon

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23 December 1985
File S134

Meewasin Valley Authority
345 Third Avenue South
SASKATOON, Saskatchewan
S7K 1M6

ATTENTION: MR. GLEN GRISMER
RESOURCE CONSERVATION COORDINATOR

Dear Sir:

SUBJECT: Slope Instability Study
South Saskatchewan River Banks
Saskatoon, Saskatchewan

Please find enclosed our report on the above project. I trust that we have addressed the issues raised in the Terms of Reference. Please do not hesitate to contact me should you have any questions.

Yours truly,

CLIFTON ASSOCIATES LTD.

A. WAYNE CLIFTON, P. ENG.

AWC/jmd

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GLOSSARY

aquifer A water bearing body of permeable sand or gravel that is capable of yielding significant amounts of groundwater to wells or springs.

bedrock Strata deposited in geologic time before the Glacial epoch, i.e. before the ice ages.

boulders The coarsest particles of soil. Boulders are the fraction of soil larger than 300 mm.

clay Very fine grained mineral soil possessing the property of plasticity. Clay particles are smaller than 2 microns (0.002 mm).

cobble The coarse fraction of mineral soil intermediate between gravel and boulders, ranging in size from 75 to 300 mm.

electric logging A method of recording the formations traversed by a drill hole. An electrical log is normally based on the measurements of two electrical parameters namely the spontaneous potential (SP) generated between the borehole fluid and soil pore water; and the electrical resistivity (R) of the strata to the flow of electric currents.

factor of safety A measure of the degree of stability of a slope. The factor of safety is generally taken as the ratio of forces tending to resist landsliding, to the forces tending to cause landsliding. A slope must have a factor of safety of greater than 1.0 to be stable.

geotechnical investigation A investigation to determine the thickness and distribution of the soil strata, groundwater conditions, and soil engineering properties such as strength, permeability and compressibility, to provide information for analysis and design of structures built on or with natural materials.

glacial drift Sediments transported and deposited by glaciers.

gravel The coarse fraction of soil, intermediate between sand and cobble. An accumulation of mineral rock particles ranging from 4.76 to 76 mm in diameter.

groundwater discharge Discharge of water from the soil. This is commonly called seepage. If enough flow is present, a spring may occur.

groundwater recharge The flow of water into the ground to replenish the groundwater table.

headscarp The steep slope created at the landward extent of a landslide where the landslide mass breaks away from the headland.

hydrogeology The study of the occurrence, flow and behavior of water within the ground, i.e. in soil or rock.

instability Slumping or landsliding that occurs when the factor of safety with respect to stability reaches 1.0 indicating that the shear resistance of the soil in a slope has been exceeded.

intertill stratified drift Sorted drift (clay, silt, sand or gravel) that occurs between two till sheets of different geologic ages.

intratill stratified drift Stratified drift, generally sand or gravel, that has been deposited within an identifiable till stratum.

lacustrine Formed or deposited in lakes.

landslides A mass of earth which perceptibly moves, or slides, or falls down a slope.

monitoring Observation by instruments such as slope movement indicators, or by surveys, or with piezometers, to measure changes in water levels or shear movement in slopes.

outcrop The exposure of a soil strata where it projects through the overburden. Outcrops are commonly found along the valley wall or ravines.

phreatic surface Commonly called the groundwater table or piezometric surface, it is the upper limit of the zone of saturation in the soil. Below this imaginary surface, all of the soil pores are filled with water.

physical environment The total of all external conditions which act upon the soil. This may include climatic influences (temperature, precipitation, evapotranspiration), stresses in the soil, applied loads, biologic influences and other factors which can effect the engineering behavior of the soil.

piezometer An observation well, usually of small diameter, installed and sealed in a stratum to measure the groundwater level (pore water pressure) in the stratum.

piezometric level The static level of water in a soil or aquifer. Synonymous with groundwater table.

piezometric pressure The pressure head generated in the soil pores by a water head equivalent to the piezometric level.

piping The removal of soil particles by percolating groundwater. This removal of soil particles can lead to creation of underground tunnels which increase gradients and lead to more piping. The process can rapidly accelerate, leading to the rapid erosion of a slope.

plasticity The property of a dry soil that allows it, when mixed with water, to be rolled into a thin thread. Soils that require a large amount of water to allow plastic deformation are called highly plastic, while those requiring only a small amount of water are said to possess low plasticity. Only clay soils and some organic soils exhibit plasticity.

preglacial Referring to sediments and strata deposited before the ice ages.

sand Mineral soil grains intermediate in size between silt and gravel. The size range for sand is 0.74 to 4.76 mm.

seepage gradients The slope of the piezometric surface (water table) in the soil. It is normally expressed as a drop of feet vertically in a horizontal run of a given number of feet.

shear surface Also called the slide plane, it is the surface on which the landslide mass moves over the underlying intact soil. All shear displacement takes place across this surface.

silt The soil grain size intermediate between clay and sand. Silt is a fine grained mineral soil that possesses no plasticity. The mean particle diameter for silt ranges from 0.002 to 0.076 mm.

slope movement indicators Instruments installed in boreholes or on the surface of slopes to measure shear displacement (landslide movement) in the slope.

slope movement Downslope movement of soil in a slope. Synonymous with landslide movement.

slumping Material that has moved down the slope. Synonymous with landsliding.

springs A place where groundwater discharges naturally from the soil to the ground surface in observable amounts.

stability analysis An engineering analysis comparing the strength of the soil with the stresses imposed on the soil. A stability analysis will calculate the factor of safety of the slope and determine its relative susceptibility to landslide movement.

stabilize slopes To revise the topography, lower the groundwater levels or use other means to raise the factor of safety to sufficient levels where landslide movement ceases.

stable slopes Slopes not subject to landslide movements.

stratified drift Drift (glacial debris) that has been sorted by water or wind into particles of similar sizes, i.e. clay, silt, sand, gravel, cobble or boulders. The material is sorted into strata of similar grain sizes giving a stratified or layered appearance.

stratigraphy The arrangement and sequence of various strata in the earth's crust.

subcrop The occurrence of a stratum on the underside of another stratum. The elevation of the top of the lower stratum.

test hole A small diameter hole drilled or bored into the ground for the purpose of obtaining information on the soil stratigraphy, and to obtain representative samples and install piezometers.

till Unsorted glacial drift. Till is an assortment of all particle sizes ranging from clay to boulders in association with each other and not sorted according to particle size.

toe of landslide The lower extremity of a landslide. The toe is commonly the lowermost point where thrusting movement can be observed.

1.0 INTRODUCTION

The Meewasin Valley Authority (MVA) has regulated development within the valley of the South Saskatchewan River since 1978. The jurisdiction of the Authority includes assessment and approval for all structures and other types of developments on or near the river banks. Instability exists in many areas along the valley slopes in Saskatoon. A significant amount of damage to property has resulted from slope movements over the past 70 years. Some remedial measures have been implemented, however slope movement continues in many other areas.

The objectives of this study are outlined in the Terms of Reference included in Appendix A. Not all of the objectives were addressed; specifically the tasks related to cataloguing of geotechnical information (Items 1 and 2 of Phase I) were excluded for budgetary reasons. The specific objectives were to:

- Summarize the geologic history of the Saskatoon area.
- Provide a brief history of slumping in the City of Saskatoon.
- Briefly explain the slumping mechanisms.
- Identify factors affecting slope instability.
- Present an overview of hydrogeologic conditions in the study area.
- Identify procedures commonly used to stabilize slopes.
- From available information, evaluate the risks and effects of slumping within the study area
- Consider the impact on stability of a range of potential improvements and activities permitted by the City of Saskatoon bylaws.

- Identify geotechnical investigation requirements for stability analysis.
- Examine the role of MVA in enforcing standards associated with geotechnical investigations.
- Outline the types of monitoring systems and how they might be used to assess the safety of slopes.
- Propose limiting factors of safety for design purposes.
- Propose a system for cataloguing geotechnical reports and borehole information, and propose a home for this data base.

The study area includes the river banks of the South Saskatchewan River within the Conservation and Buffer Zones of the Meewasin Valley Authority as shown in Figure 1. The east side of the river is of primary concern.

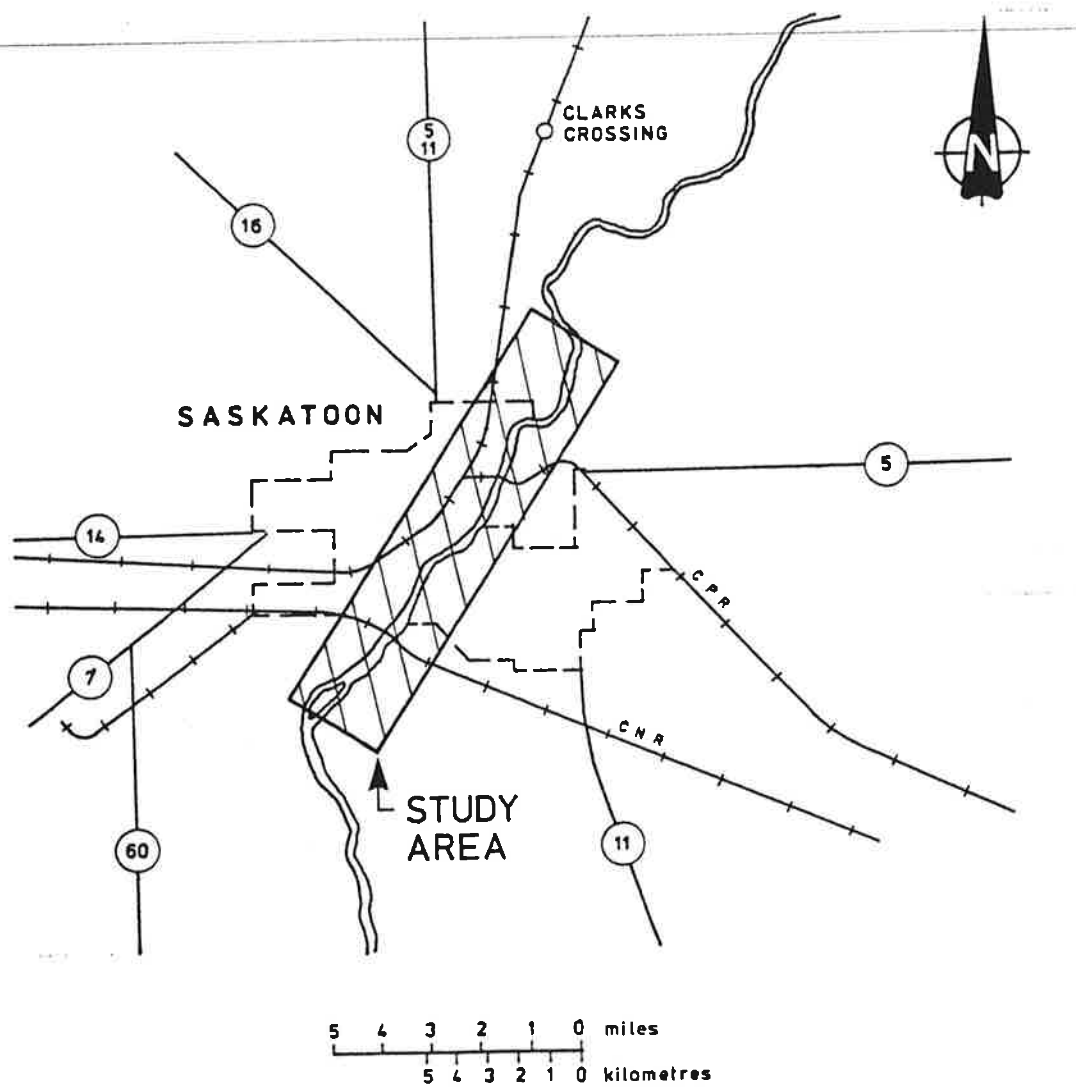


FIGURE 1

LOCATION OF STUDY AREA

2.0 GEOLOGIC HISTORY

The geologic history of the study area was covered in detail by Clifton (1979). That presentation covered an area well beyond that of concern in this study.

The lowest stratum of concern from a stability perspective is the bedrock under the South Saskatchewan River which varies in elevation from about 396 m to 427 m ASL (Christiansen, 1970). River soundings (Clifton Associates Ltd., 1984d) indicate that present river bottom within the study area varies from about elevation 465 m to 471 m ASL. Since the river bottom is about 40 m above the bedrock subcrop, the bedrock plays no role in the existing instability and the drift is of primary interest in the consideration of the stability of the river banks.

The primary land forming processes within the study area were multiple glaciations followed by erosion of the South Saskatchewan River Valley. The present landscape is an expression of many millions of years of deposition, erosion and other physical processes. Although the present landscape features are formed in only the surficial 30 m or so of sediments, the deposition and shaping of these materials has been going on for many thousands of years.

Preglacial History

The bedrock sediments were deposited in preglacial seas which covered the central plains area. The youngest of these sediments in the Saskatoon area consists of a series of marine and non-marine strata that were left behind as the Cretaceous seas slowly retreated and the landscape was uplifted. Well integrated drainage developed as weathering and erosion occurred. Present day knowledge of preglacial valleys (Christiansen, 1979c) indicates that, prior to glaciation, drainage was well developed and probably towards the Arctic Ocean. These bedrock valleys were subsequently infilled with stratified sediments of the Empress Group and with till.

Christiansen, (1967) reported that up to 600 ft. of evaporites consisting of various soluble salts and precipitates were deposited in the Saskatoon area during the Devonian era. Although these strata occur about 4,000 ft. below the surface, subsequent solution has removed much of them from under a section of the City. This has caused down-dropping of a portion of the overlying sediments resulting in the "Saskatoon Low". This structural depression has been infilled by a great thickness of till and stratified drift deposited during repeated glaciations of the area.

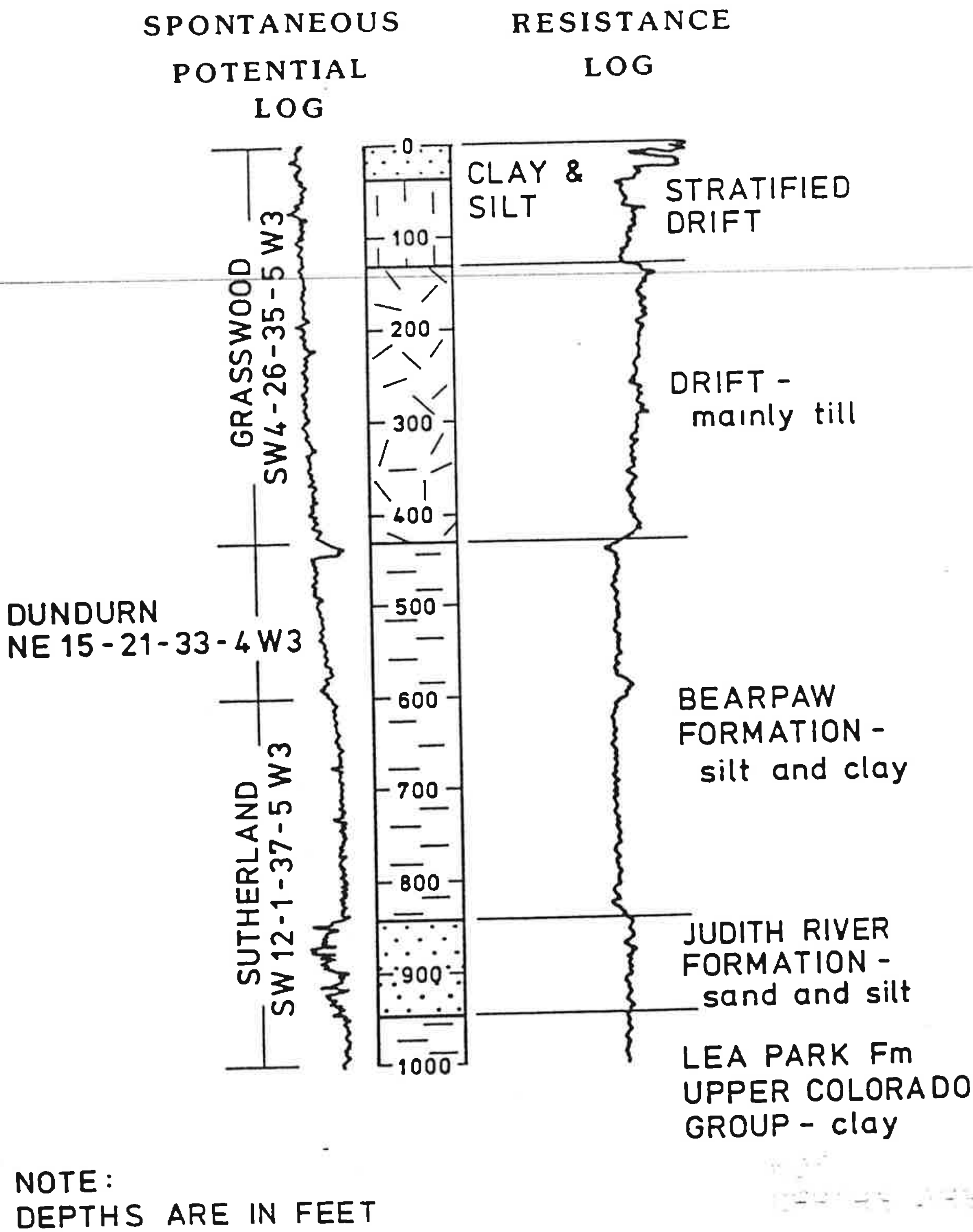
A considerable depth of sediments of various geologic ages is missing from much of the Central Plains. The youngest preglacial strata beneath the Saskatoon area are estimated to be about 70 million years old (Caldwell, 1968; McLean, 1971). The Quaternary or glacial period began about 3 million years ago. Therefore, more than 60 million years of geologic history has been removed by erosion, weathering and the action of glaciers. However, as discussed above, there is no evidence of any of the bedrock sediments being involved in any slope instability within the study area. Rather, the observed instability is occurring within the Quaternary sediments.

Glacial History

Major climatic changes initiated the Glacial epoch about 1 million years ago. During this period there were at least five major glacial advances through the Saskatoon area. The advance of the huge continental ice sheets during each glaciation resulted in erosion of the preglacial landscape. The eroded materials were mixed and transported by the advancing ice and later deposited as the ice fields retreated. A period of weathering and erosion followed each major retreat of the ice. These processes resulted in a sequence of identifiable till units. These units are well known in the Saskatoon area (Christiansen, 1968a) and are commonly referred to, oldest to youngest, as:

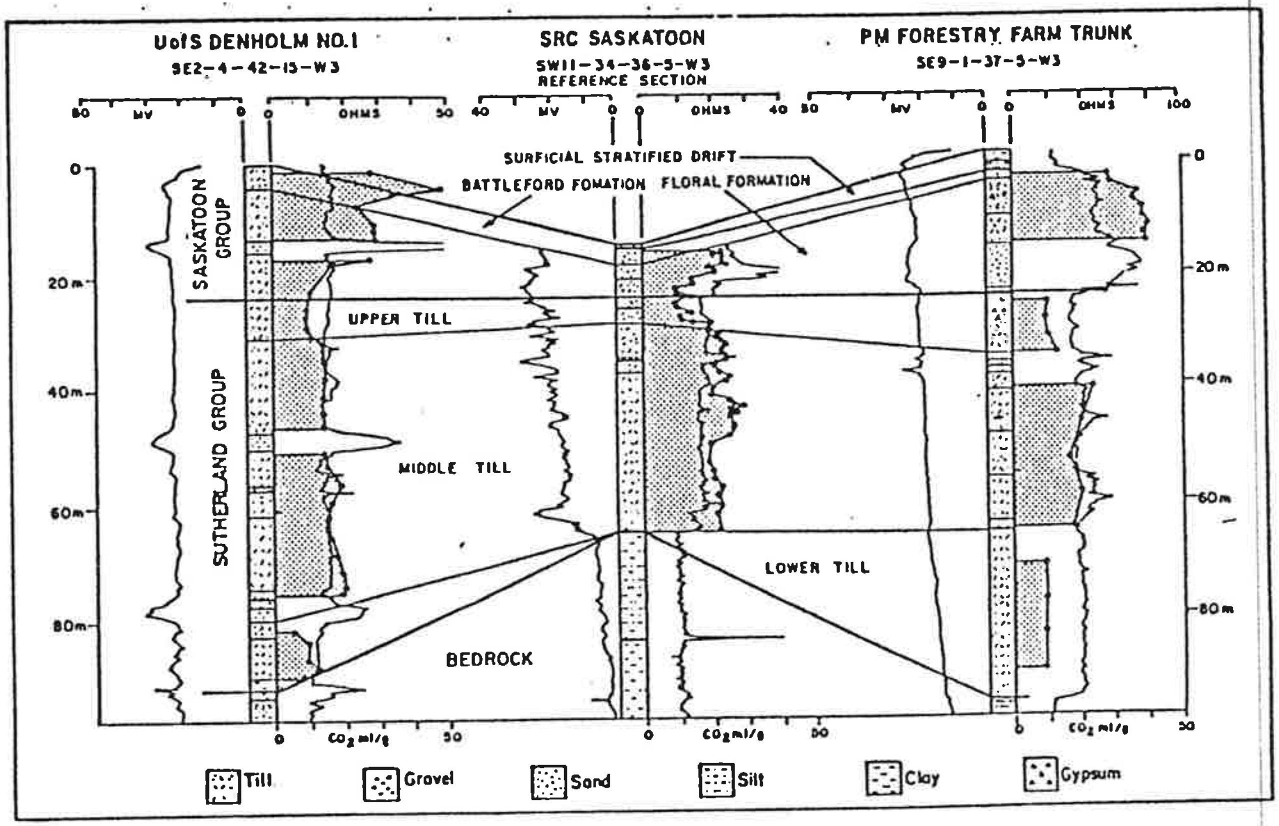
- the Sutherland Group, consisting of:
    - Lower Sutherland Formation.
    - Upper Sutherland Formation.
- the Saskatoon Group consisting of:
    - Floral Formation.
    - Battleford Formation.

The general bedrock-drift stratigraphic sequence is illustrated in Figure 2 while the inter-relationship of the till units is illustrated in Figure 3. The till units are primarily a heterogeneous mixture of all particle sizes



COMPOSITE GEOLOGIC LOG IN THE SASKATOON AREA (AFTER CHRISTIANSEN, 1970)

FIGURE 2



STRATIGRAPHY OF THE DRIFT IN THE SASKATOON AREA
(AFTER CHRISTIANSEN, 1983)

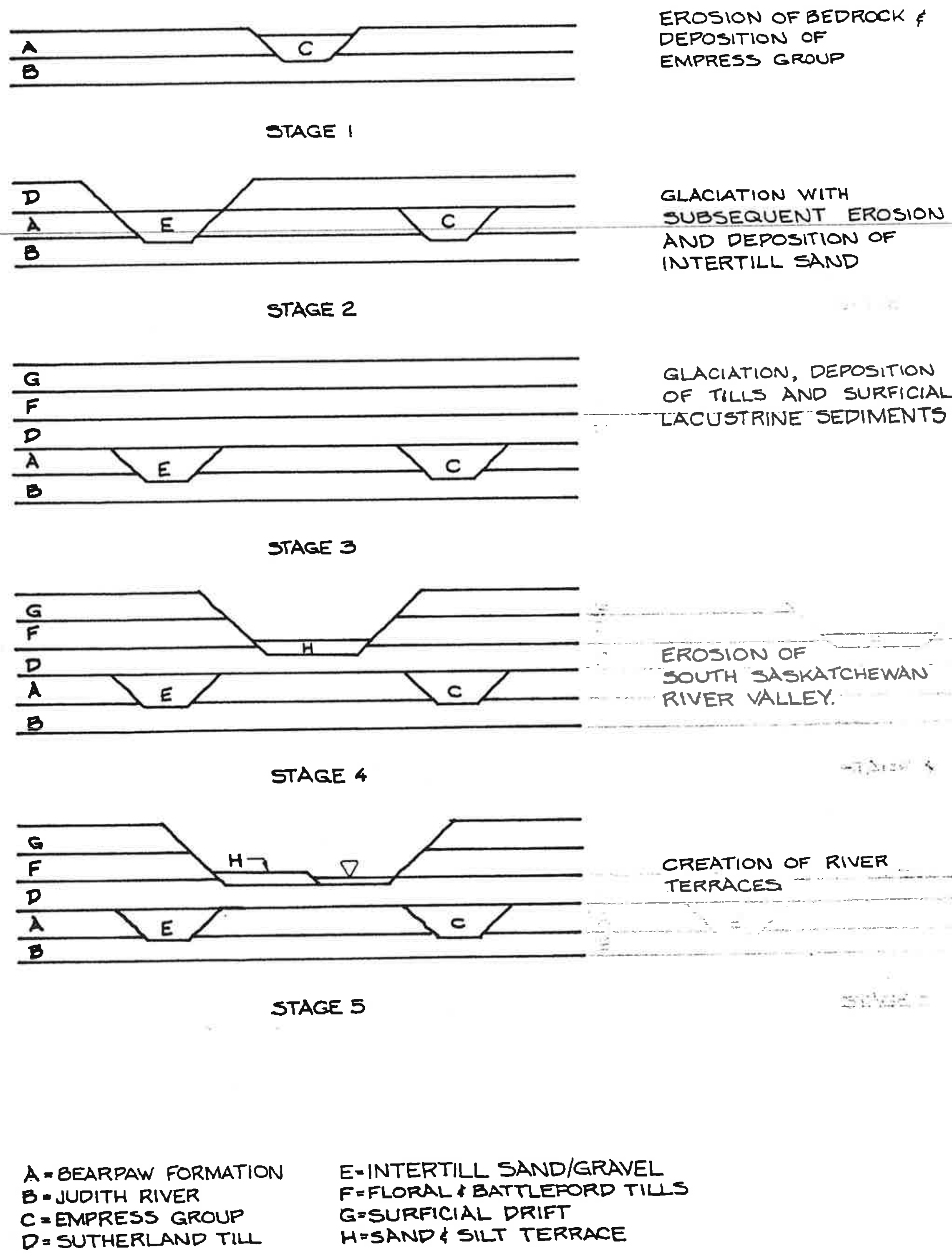
FIGURE 3

from clay to boulders. However, they generally have a silty or sandy clay matrix of low plasiticity. Tills of both the Saskatoon Group and the Sutherland Group are exposed on the river bank. The Battleford Formation till of the Saskatoon Group is thin and less dense, while the Floral Formation till is a very dense material that provides foundation support for high capacity foundations in Saskatoon.

Large deposits of sand and gravel were formed, both within the till sheets (intratill stratified drift), and between them (intertill stratified drift). The method of formation of these is shown schematically in Figure 4. These strata now form major aquifers within the study area. Christiansen (1979c) interprets the recession of the last (Wisconsinian) glacier as being about 12,000 years before the present (Figure 5). This glacier deposited the Battleford Formation sediments and was the source of meltwater that shaped the landscape in the post glacial period.

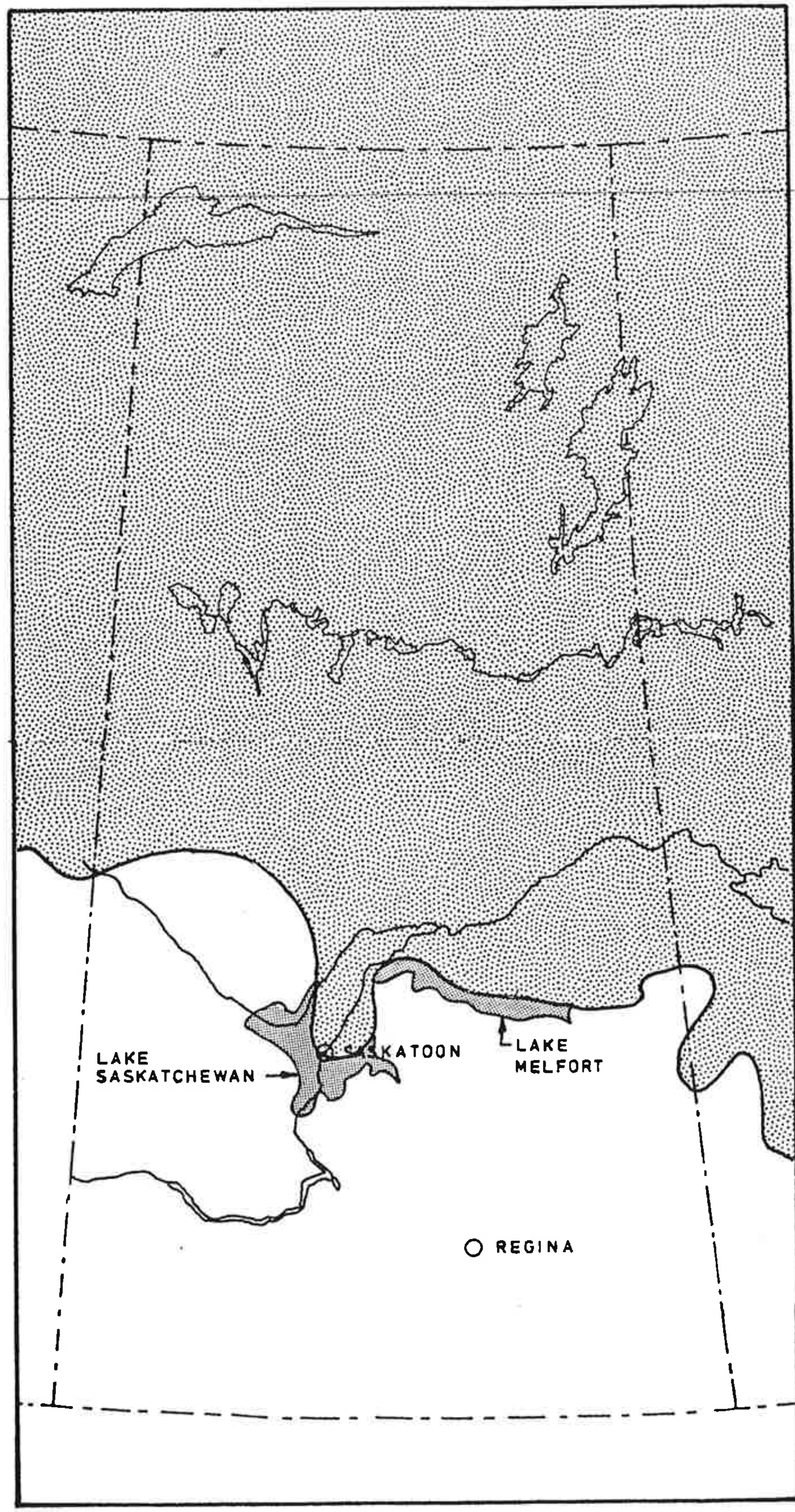
Post Glacial History

A temporary lake (Lake Saskatchewan) was formed between the retreating glacier ice and higher ground to the west during the final retreat of the Wisconsin glacier. Rivers flowed into the lake from the west, forming deltas close to the retreating shoreline (Figure 6). Radiocarbon dates (Koster, 1978) indicate that this lake persisted for about 3,000 years and was likely drained by about 9,000 years before the present. While it persisted, over 60 metres of stratified clay and silt were deposited on some areas of the land it covered. The lowermost sediments in the lacustrine sequence are generally medium to highly plastic



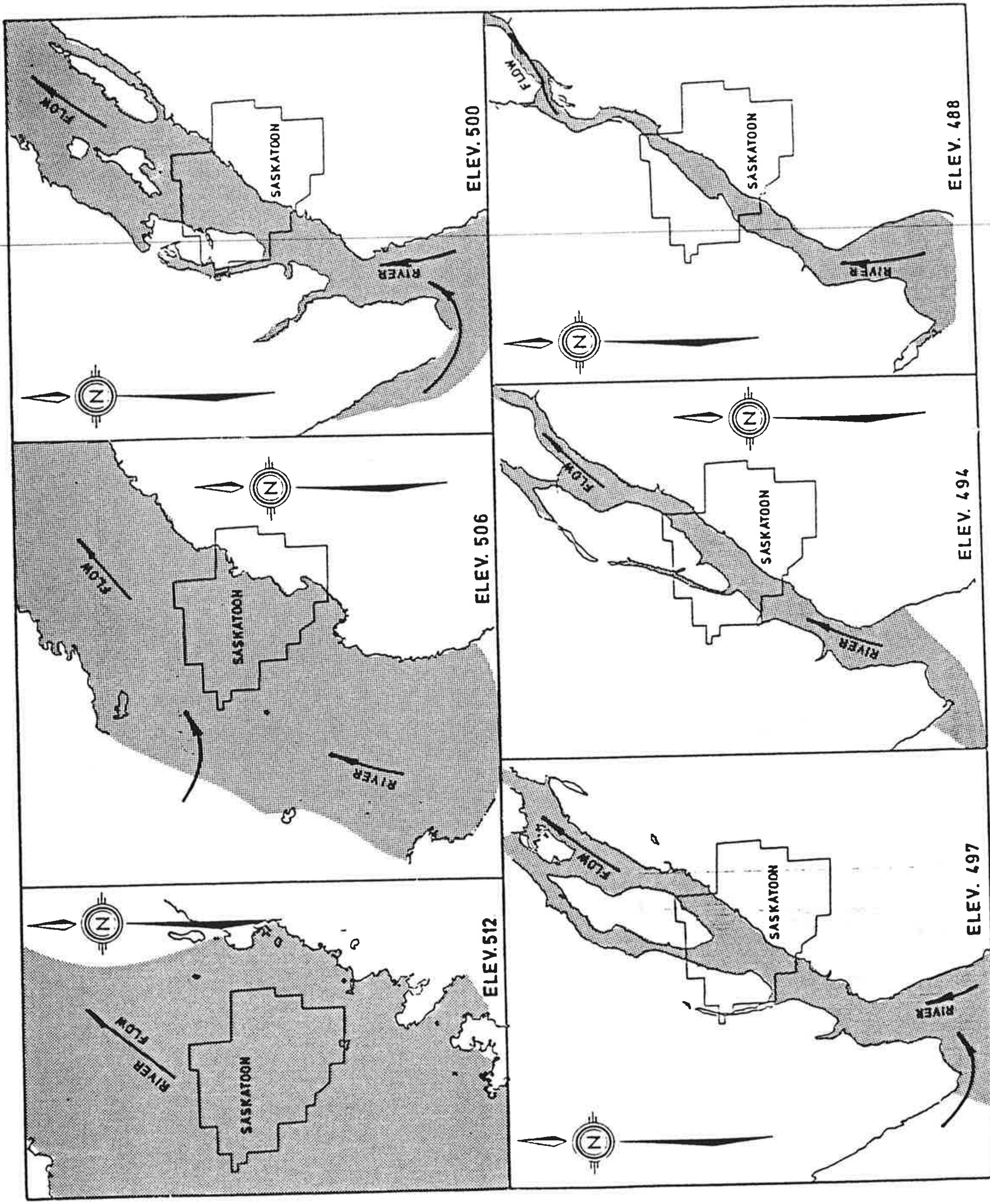
DEPOSITIONAL/EROSIONAL SEQUENCE AT SASKATOON
(AFTER CHRISTIANSEN, 1980)

FIGURE 4



ICE FRONTAL POSITION 12,000 YEARS AGO
(AFTER CHRISTIANSEN 1979c)

FIGURE 5



STAGES IN THE FORMATION OF THE SOUTH SASKATCHEWAN RIVER VALLEY IN SASKATOON

FIGURE 6

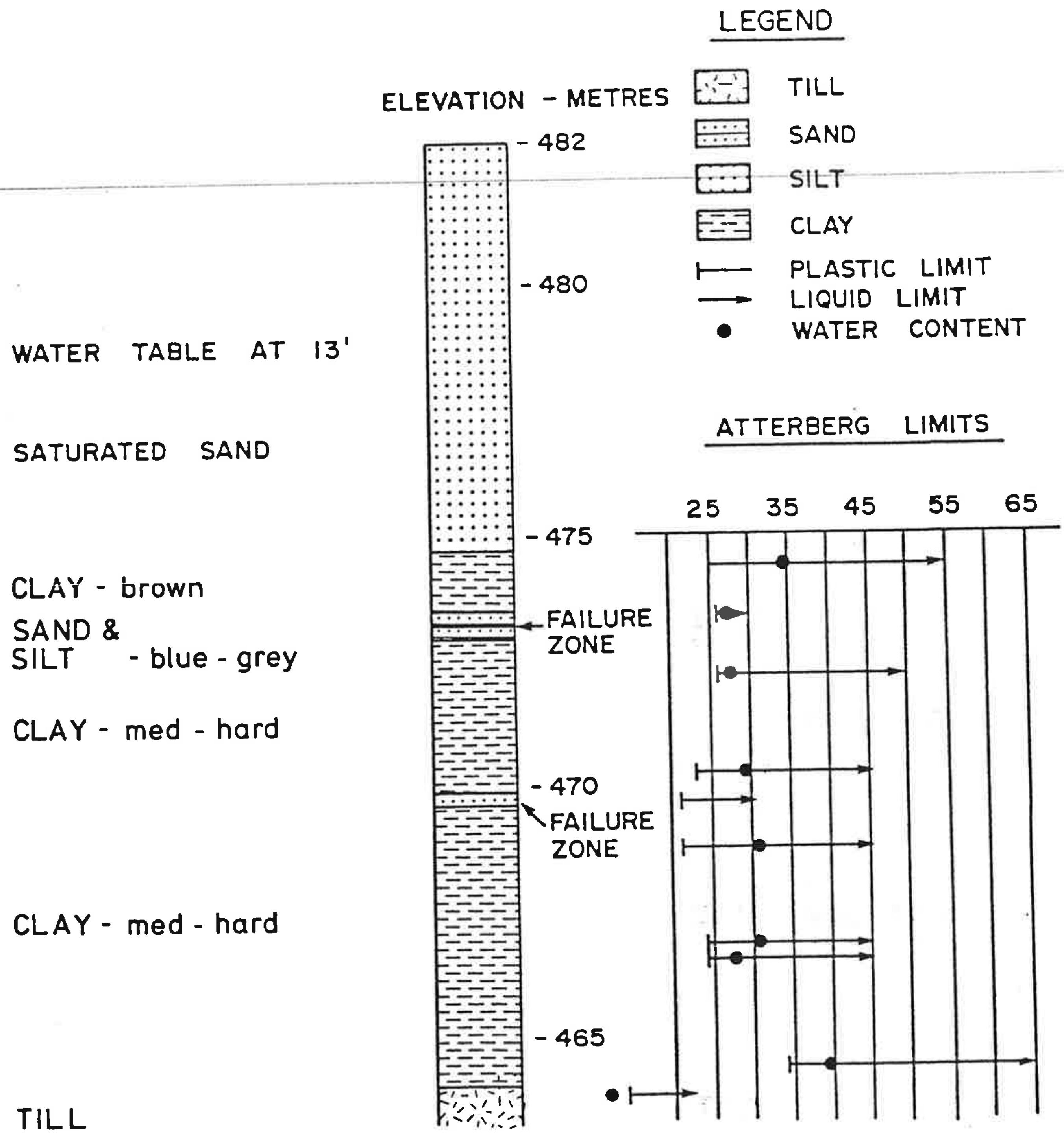
clays which are the weakest materials in the stratum and form the base of much of the landsliding that occurs in the river valley.

The depositional sequence of lacustrine sediments is complex. Sand was deposited, probably as a delta where the South Saskatchewan and other drainage channels emptied into Lake Saskatchewan, on top of the finer grained lake sediments in the southerly part of the basin. Farther north, finer grained lacustrine clay was deposited over the coarser lacustrine sediments. This depositional sequence accounts for the present stratigraphic variations which include sand overlying clay in the southerly part of the study area (Figures 7 and 8) and the typical clay-silt-clay sequences encountered further north (Figure 9).

When the ice had retreated sufficiently to expose an outlet, drainage of the glacial lake occurred rapidly through a spillway along the ice margin. This channel rapidly eroded to form the valley of the South Saskatchewan River (Figure 6). During its rapid development, the valley incised through the unconsolidated lacustrine sediments, and less quickly though the more resistant glacial debris. The lacustrine sediments were removed over an extensive area on the west bank and to a lesser extent on the east bank, particularly north of the University of Saskatchewan. As the river eroded to near its present level, the flow persisted for a longer period at each elevation, and depositional terraces were formed on the west side of the river. Radiocarbon dates reported by Turchenek et al, (1974) suggests that the Saskatoon terrace, on which the city core has developed, was eroded to its present elevation about 8,000 years ago and was abandoned by the river perhaps 3,000 years ago. Other dates (Koster, 1978) suggests the river level was stabilized at Frenchman's Flats about 35 kilometres south of Saskatoon about 3,000 years before present. Erosion, and subsequent building of point bars, has proceeded relatively slowly from that date, leaving a river valley up to 30 metres in depth, and with young slopes, probably only a few hundred years old, on the actively eroding river banks.

Summary

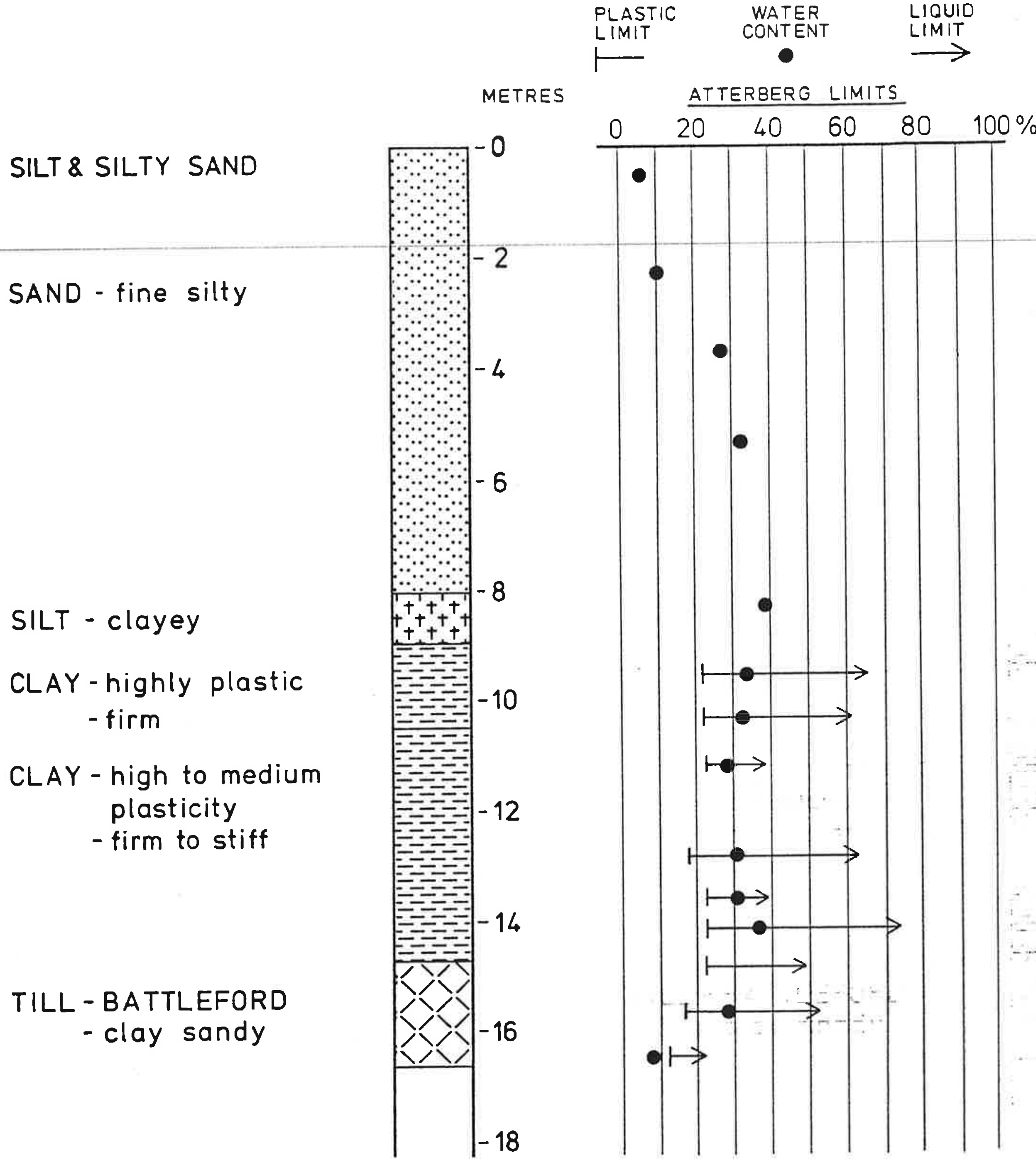
A complex history of erosion and deposition has resulted in a unique sequence of sediments which form the present river valley through Saskatoon. Figure 4 schematically illustrates how various geologic processes have resulted in identifiable, mappable sequences of till and stratified drift which make up the present valley walls. Variations in the stratigraphy are illustrated by Figures 7 to 10 which are typical test hole logs from within the study area. Figure 7 is a test hole from the vicinity of Beaver Creek illustrating surficial sand overlying basal clay, typical of an onlapping delta. Figure 8 is a test hole log from Diefenbaker Park area showing typical surficial sandy silty sand, intermediate sand, and basal clay overlying Battleford till. Figure 9 is a test hole log from the vicinity of the Geology building on the University of Saskatchewan campus, illustrating that the materials are finer grained and the basal clay much thicker. Isopachs of the surficial clay in this area are shown on Drawing S134-1.



Flight auger hole #2 show failure zones

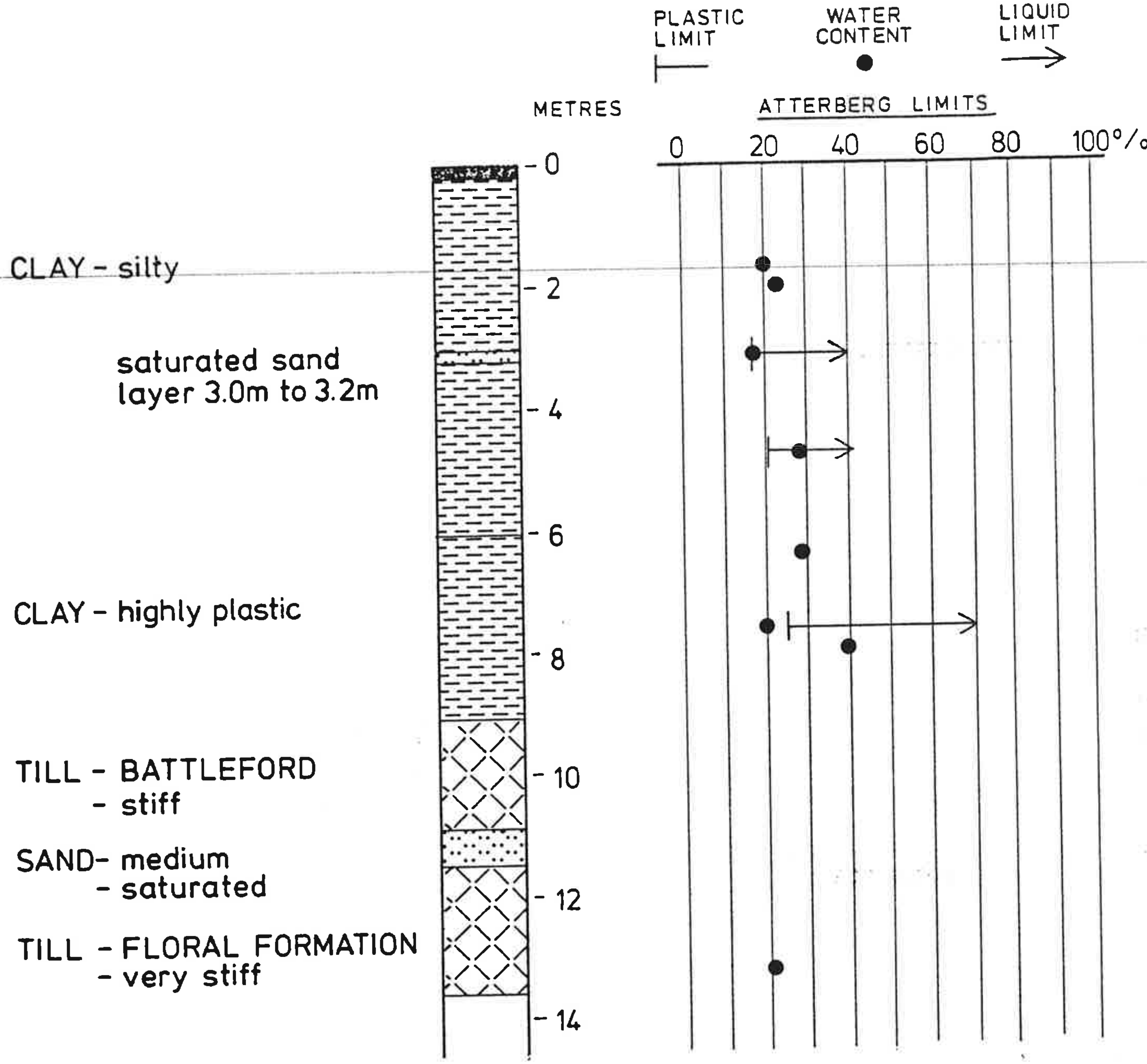
STRATIGRAPHY AND SOIL PROPERTIES AT BEAVER CREEK
(AFTER YOSHIDA, 1981)

FIGURE 7

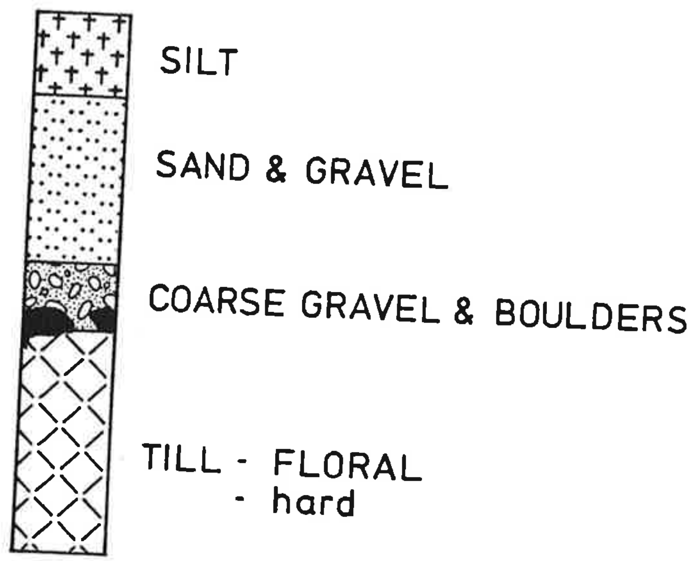


STRATIGRAPHY AND SOIL PROPERTIES AT DIEFENBAKER PARK
(AFTER CAL, 1984d)

FIGURE 8



STRATIGRAPHY AND SOIL PROPERTIES, GEOLOGICAL BUILDING, UNIVERSITY OF SASKATCHEWAN (AFTER CAL, 1985) FIGURE 9



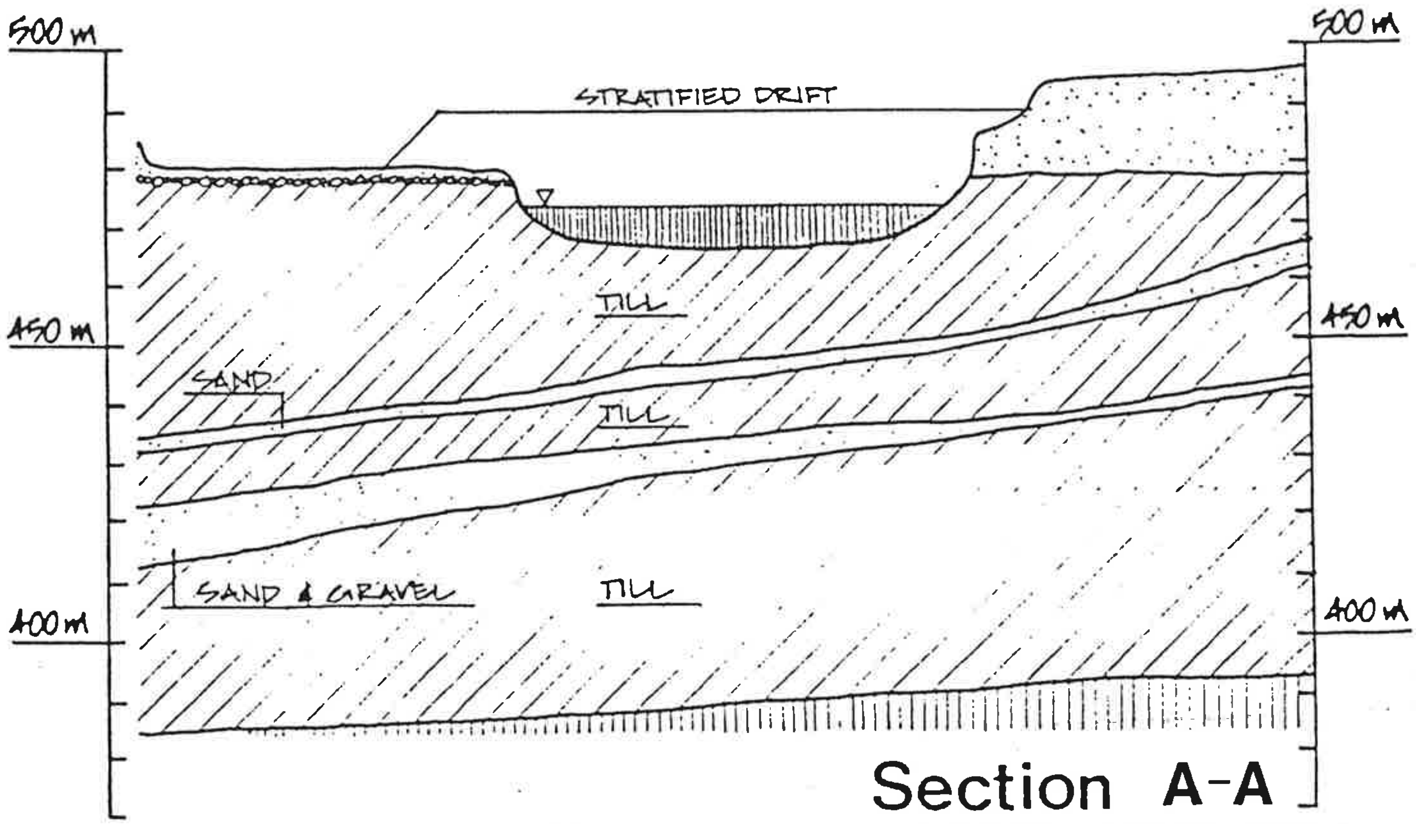
TYPICAL TESTHOLE LOG ON THE SASKATOON TERRACE LANDFORM

FIGURE 10

Figure 10 is a typical test hole log from the Saskatoon terrace illustrating the absence of lacustrine sediments and Battleford Formation till on the west river bank. On the lower part of the terrace, the terrace sand and gravel lie directly on eroded till of the Floral Formation.

The geologic setting of the east bank differs significantly from that of the west bank. With the exception of a small terrace upstream of Rotary Park, the east bank is over-steepened and comprised of tills of the Saskatoon and Sutherland Groups capped by weaker lacustrine sediments. Most present day landsliding is occurring in the lacustrine sediments. A few slides have been caused by seepage from stratified drift units within the tills. By contrast, post glacial erosion and deposition have developed a terrace along the west bank. In these locations, the weaker lacustrine sediments and Battleford Formation till have been eroded leaving much stronger sediments, less prone to landsliding.

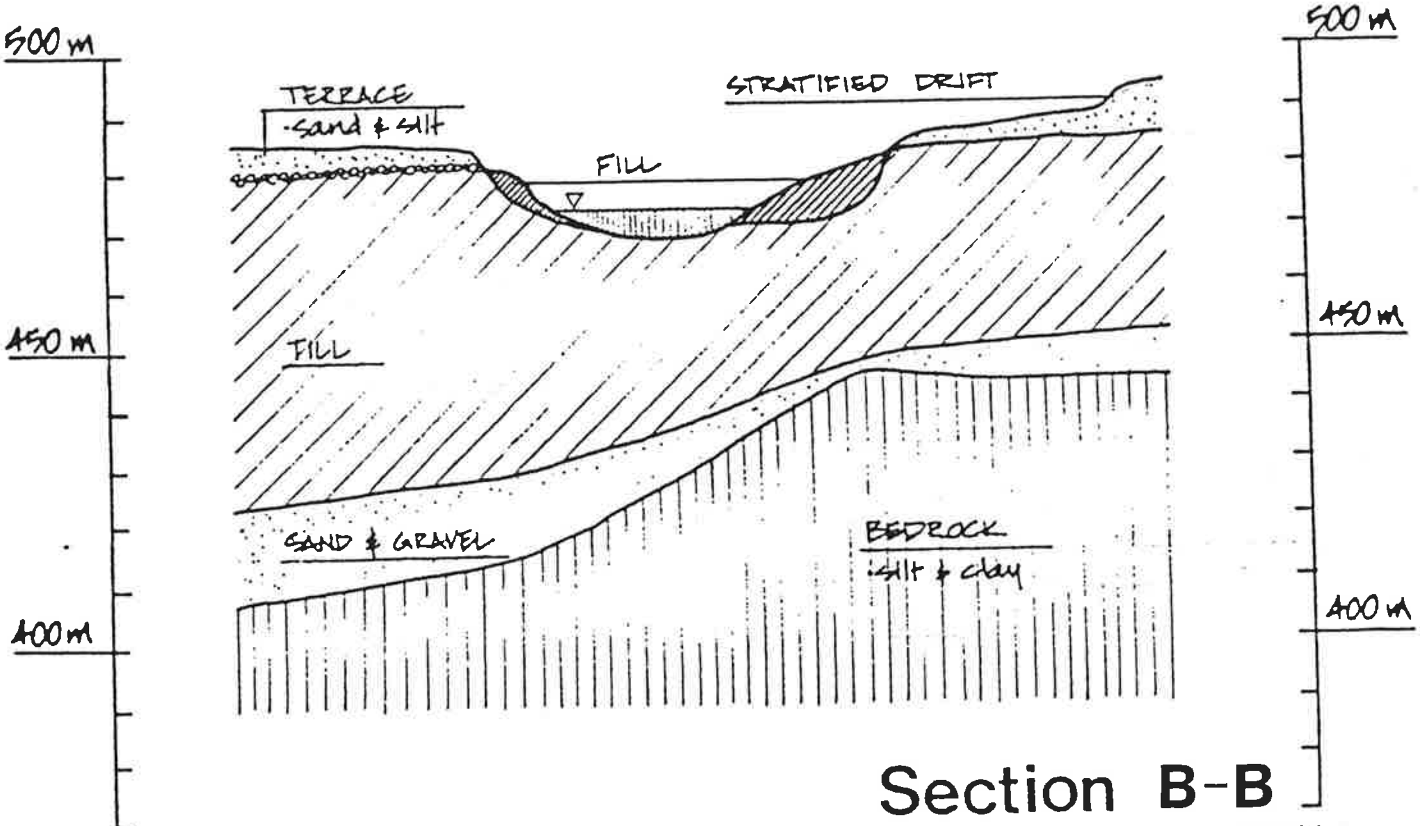
The variation in geologic conditions along the valley is shown by schematic cross sections of the river valley, Figures 11 to 15 inclusive.



SCALE - 1:6000 HORIZONTAL / 1:1250 VERTICAL _ ELEVATIONS - GEODETIC DATUM

CROSS SECTION AT DIEFENBAKER PARK FIGURE 11

(from CAL 1984f)



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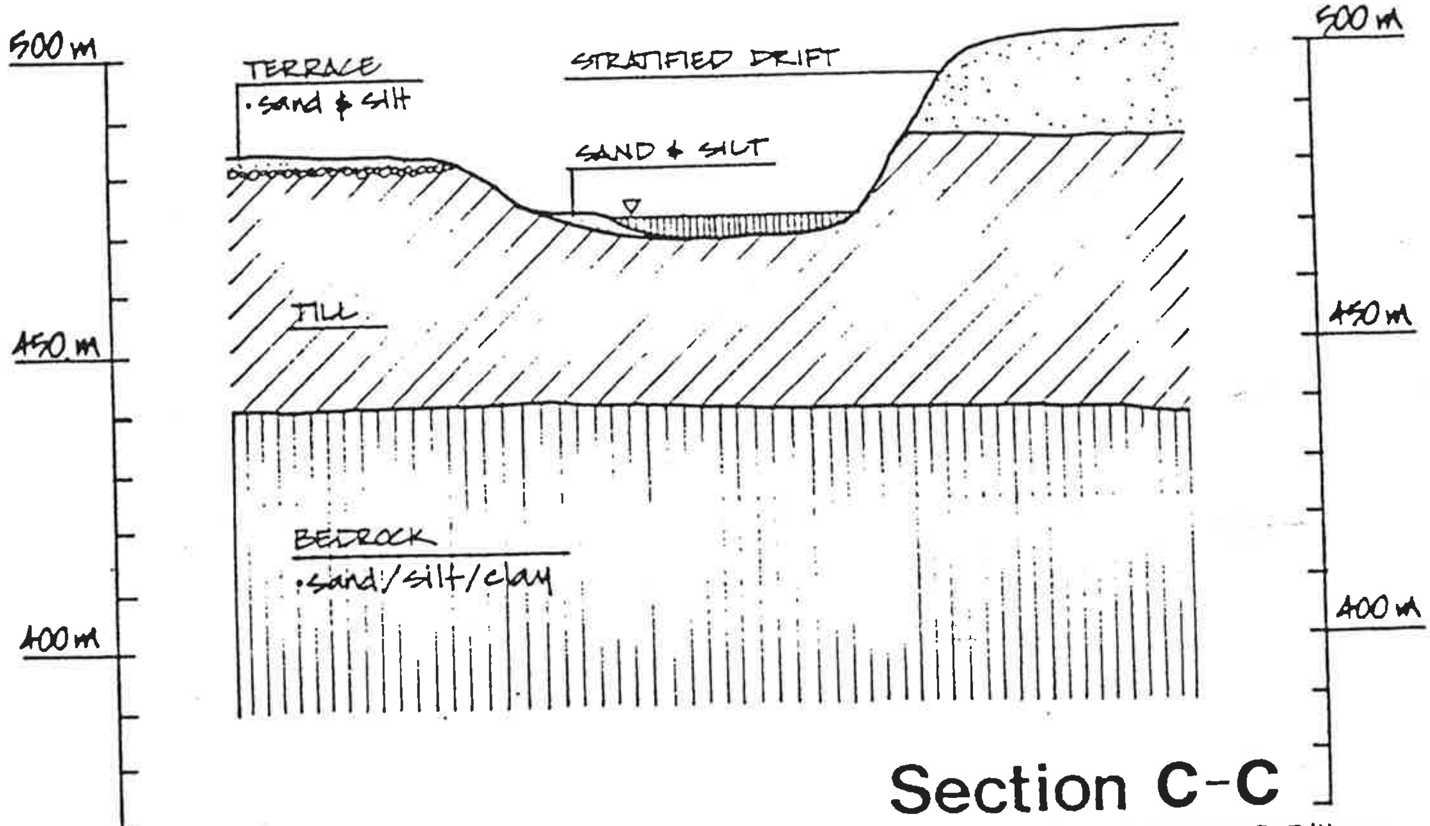
CROSS SECTION AT ROTARY PARK FIGURE 12

(from CAL 1984f)



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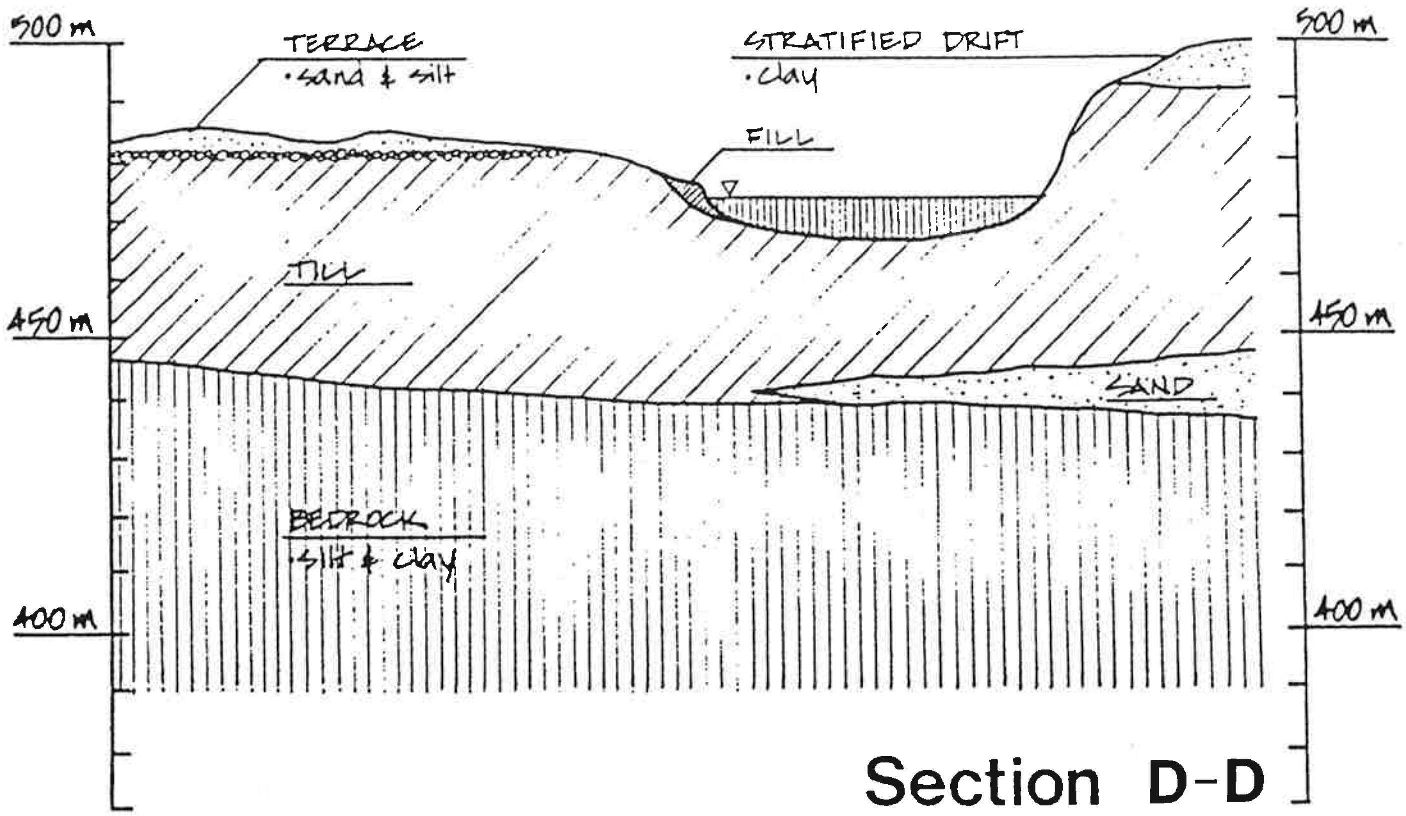
CROSS SECTION AT SASKATCHEWAN CRES. FIGURE 13
(from CAL 1984 f)



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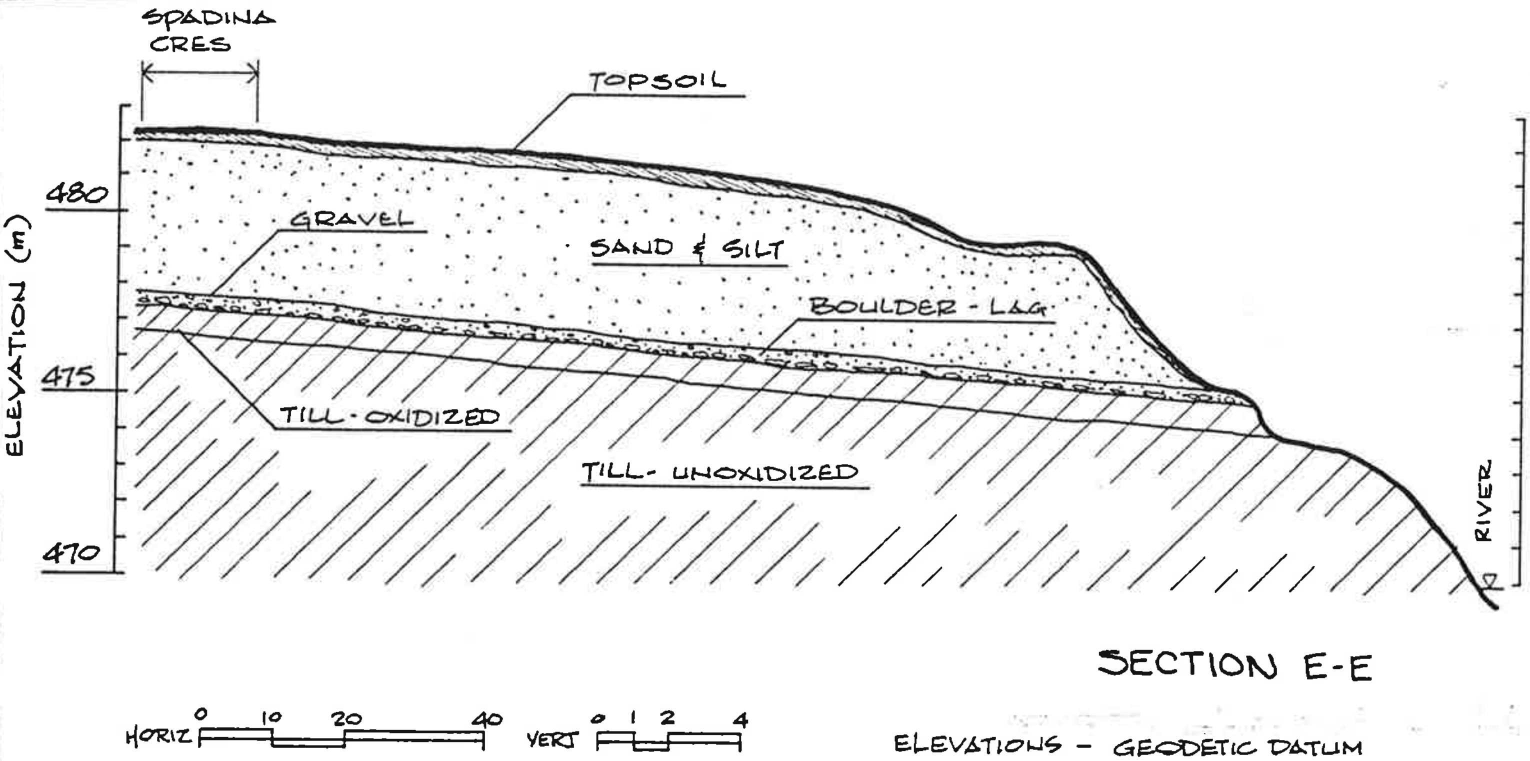
CROSS SECTION AT UNIVERSITY OF SASK FIGURE 14
(from CAL 1984 f)



STRATIGRAPHIC SECTION OF WEST BANK
AT MEEWASIN PARK
(from C.A.L. FILES)

FIGURE 15

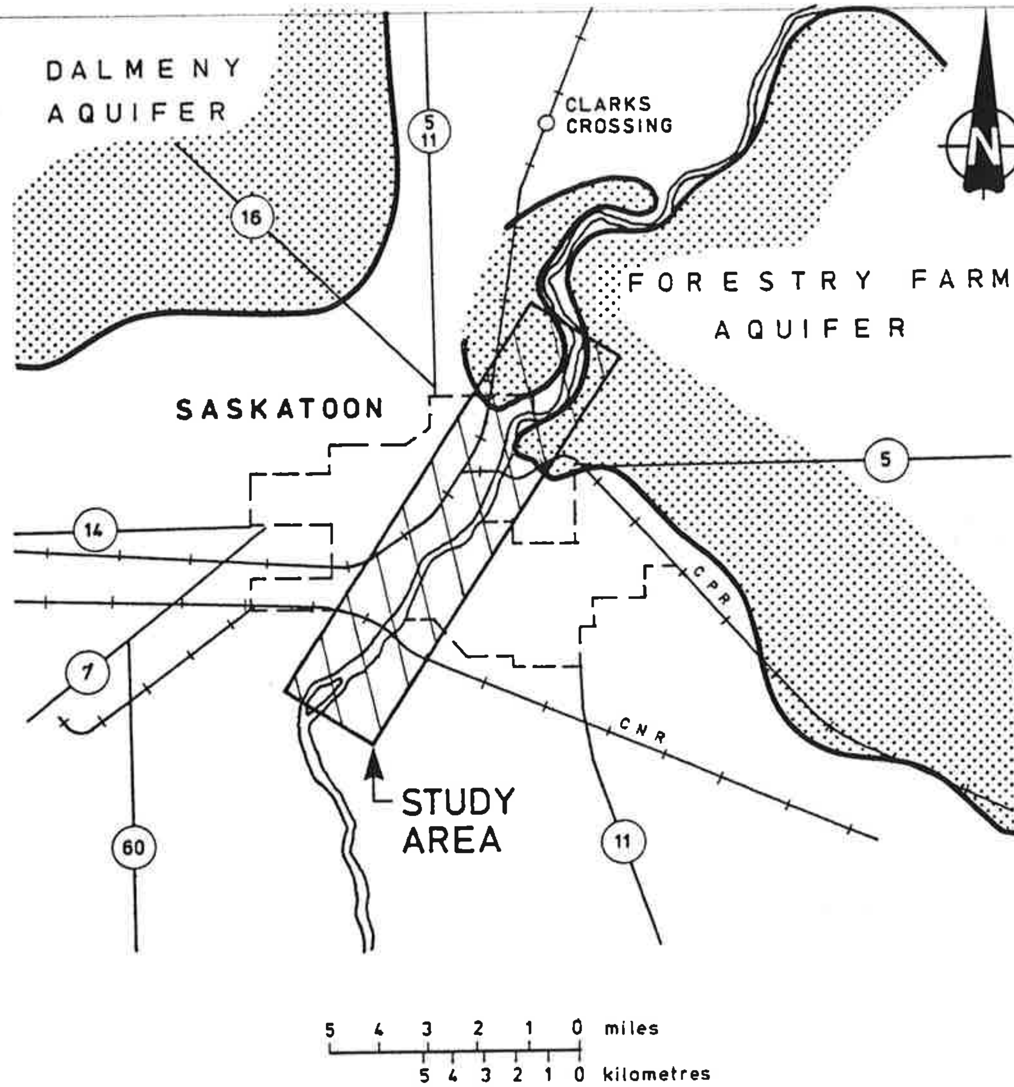
3.0 HYDROGEOLOGY

The South Saskatchewan River valley and the tributary ravines fuction as a major drain for groundwater. Thus, the groundwater regime on the valley walls is one of groundwater discharge. Infiltration from the surrounding uplands migrates by gravity to outcrop as springs at numerous points along the valley. These springs originate from two major types of aquifers: surficial aquifers open to the atmosphere; and intertill and intratill aquifers which are layers of sand and gravel enclosed top and bottom by glacial till. Other aquifers, such as the sub-glacial Empress Group sand and gravel, and the sand of the Judith River Formation, exist at greater depths under the study area and exhibit artesian heads which cause a slow percolation of water upwards to river level. However, there is no indication of that these deeper aquifers have any influence on slope instability in the valley.

Intertill and Intratill Aquifers

The approximate extent of the intertill and intratill aquifers are shown in Figure 16. The major confined aquifers include the Dalmeny and Forestry Farm aquifers. These are intratill sand and gravel deposits that are recharged in the surrounding uplands. The Forestry Farm aquifer outcrops on the east river bank as a series of highly mineralized springs at Petursson's Ravine and near the Penitentiary Hospital, and as a nearly drained sand unit below the Chemical Plant on the west bank.

An intertill sand and gravel unit between the Floral and Battleford Formations is exposed at several locations



EXTENT OF INTERTILL AND INTRATILL AQUIFERS IN SASKATOON AREA
(AFTER CHRISTIANSEN, 1979c)

FIGURE 16

(Ski Jump, Devil's Dip, University Bridge) on the east bank. It has subartesian heads on the Nutana side of the river and is a well known source of construction problems at the University of Saskatchewan. It is also the probable source of perennial flow from the storm drain outfalls at Devil's Dip on the University grounds.

Surficial Aquifers

The presence of surficial aquifers is an important consideration with respect to river bank stability. The geologic units which form surficial aquifers in the study area include:

- River terrace sand and gravel
- Surficial fluvial-lacustrine sand and silt.

The sand and gravel on the river terraces is generally very pervious. Surface infiltration and lateral drainage to the river is not restricted. This results in a line of seepage near the base of the gravel at a few locations along the valley but the piezometric gradients are low and the head in the perched aquifer minimal. This, combined with very dense, strong Floral Formation till under the terrace sediments results in very low potential for instability due to piping or slumping.

The lacustrine stratum on the east valley wall is also an aquifer. The groundwater level in these sediments is controlled primarily by climatic events and local surface infiltration. The response and the yield of the aquifers is a function of texture and available infiltration. Significant seepage occurs from lacustrine silt and sand in

Diefenbaker Park where these sediments are thick and are efficiently recharged by surface ponding. Conversely, there is little discharge from this stratum north of the University Bridge where the stratum is thin and comprised primarily of silt and clay, and the surface is well drained. Even in this setting, the lacustrine sediments are saturated with the piezometric surface sloping towards the South Saskatchewan River at a gradient of about one foot in a hundred (Fredlund, 1970).

Infiltration to the water table varies seasonally. It increases significantly during spring thaw and the piezometric levels reach a maximum during the summer. Piezometric observations on the University of Saskatchewan campus have shown that the water table begins to rise in the spring and peaks in July and August after which it slowly recedes to the pre-breakup level (Hamilton et al, 1977).

4.0 GEOMORPHOLOGY

The progressive downcutting and migration of the river to the east has resulted in an over-steepened easterly valley wall, and bevelling or flattening of the valley wall on the west side (and on the east side in the vicinity of Rotary Park) through development of slip off slopes and depositional terraces. The over steepening typically occurs at the outside of bends where erosional undercutting is most severe, as illustrated in Figure 17. Slumping has subsequently occurred in many of these over-steepened areas.

Erosion has also reduced the thickness of lacustrine deposits on the upland. The thickest lacustrine deposits occur along the east river bank south of the University Bridge (Drawing S134-1). The frequency of landslides along the river bank can be directly correlated to the thickness of the lacustrine sediments. Few landslides occur north of the University Bridge where erosion has removed most of the these materials close to the valley wall. Slumping is quite active between Broadway and University Bridges and south of Labatt's Park where lacustrine sediments have substantial thickness near the top of the valley wall.

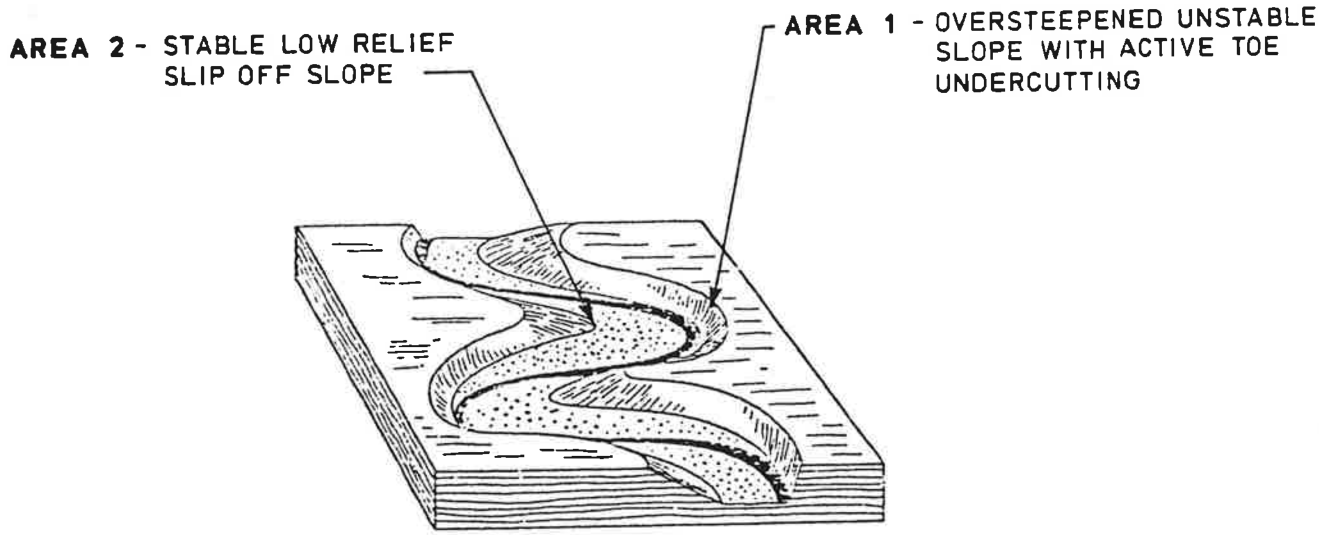


DIAGRAM TO SHOW THE WIDENING AND DEEPENING OF A VALLEY FLOOR BY OBLIQUE EROSION ON THE OUTSIDE OF RIVER BENDS, FORMING SLIP OFF SLOPES ON THE INSIDE OF BENDS. DOWNTOWN SASKATOON AND IDYLWYLD CRESCENT ARE ON THE SLIP OFF SLOPES; THE REMAINDER OF NUTANA MAINLY OVERLOOKS OVER STEEPENED ERODED SLOPES.

DIAGRAM TO SHOW THE WIDENING AND DEEPENING OF A VALLEY FLOOR BY OBLIQUE EROSION (FROM HOLMES, 1965) FIGURE 17

5.0 FACTORS AFFECTING SLUMPING

River bank instability within the study area results from shear failure within the soil mass (landsliding) or from removal of soil from the slopes by seepage (piping). The degree to which these mechanisms prevail, and therefore the extent of slope instability experienced, is a function of stratigraphy, geologic materials, slope geometry, groundwater conditions and time.

Stratigraphy and Geologic Materials

Few slope failures, with the exception of shallow seated minor slumping, occur in the till strata. Significant slope failures within the study area are almost entirely based within the lacustrine sediments. Piping failures occur in pervious strata, generally sand or silty sand, that cannot withstand seepage forces generated by gradients within the aquifers. Piping is presently contributing to degeneration of the slopes in the vicinity of Diefenbaker Park and has been the principle mechanism in the development of ravines in Petursson's Ravine, the Penitentiary hospital and Devil's Dip.

Slope Geometry

The thickness of the lacustrine sediments and the slope angle, particularly the slope on the lacustrine materials, are major factors in the occurrence of landslides in the study area. No landslides are noted where the surficial clay is less than 3 m thick and, with the exception of the President's Residence where the clay was approximately 6 m thick (Drawing S134-1) all of the major slides occur where the base of the clay is more than 8 m below existing ground surface. The most severe slides (McCraney, Nutana, Queen's House and Diefenbaker Park) occur where the till surface is 13 to 16 m below existing ground level.

All of the major slides have occurred on the east bank where the natural valley walls have been oversteepened by erosion. Most of the active slide areas have occurred on the outside of river bends (Figure 17) where both the till and lacustrine series are oversteepened. Instability is greatest where over-steepening has occurred in thick lacustrine material. By comparison, most of the west bank, with the exception of the most northerly section, has much flatter slopes, bevelled by water erosion. Only local, shallow seated sloughing has been noted on these slopes.

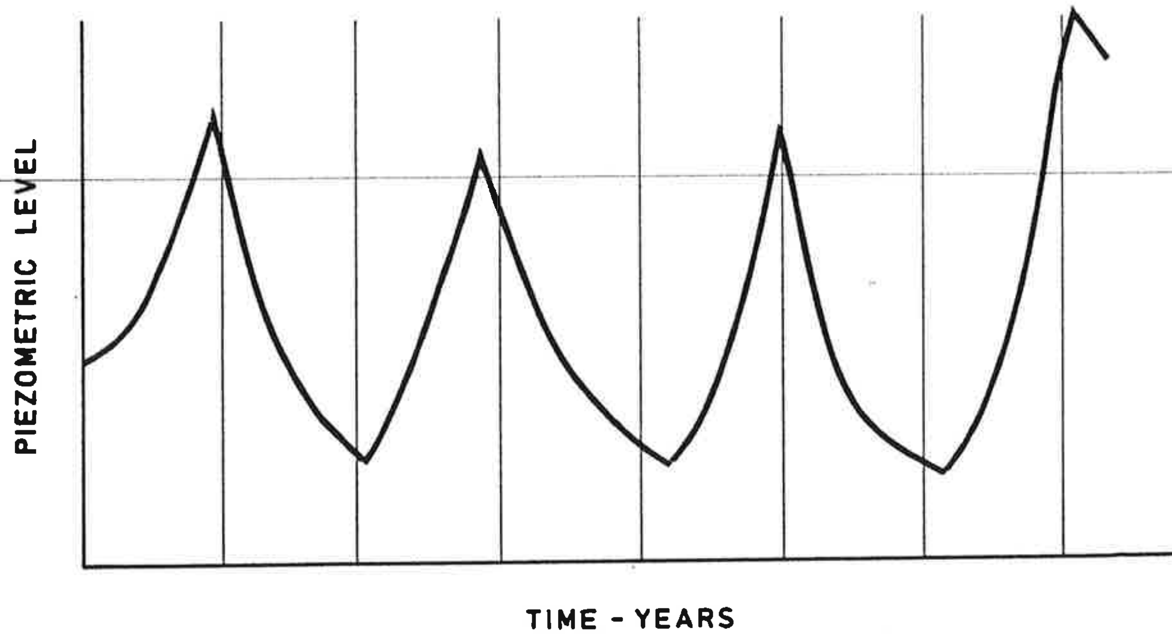
Groundwater Level

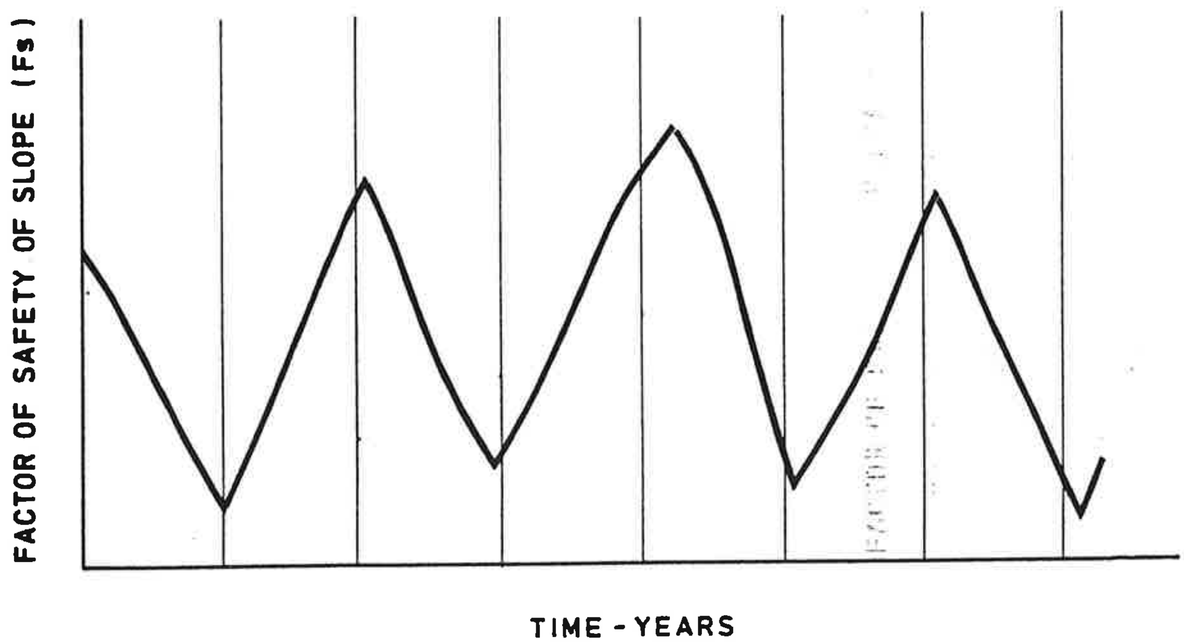
Piezometric level is a very important factor in the occurrence of landslides. In this geologic setting, water levels in the surficial clay and the intertill sand and gravel strata have the most influence on stability. The clay has sufficient strength to be stable at relatively steep slope angles if it is well drained, that is if the piezometric pressures within the clay are low. However, an old landslide slope in the clay may become unstable if the water table rises only a metre.

Variations in the elevation of the groundwater table (also called the phreatic or piezometric surface) cause similar variations in the factor of safety of the slope with respect to stability. As will be discussed in Section 10.0 following, variations in the piezometric level of up to 3 m have been observed. Figure 18 schematically illustrates that fluctuations in the piezometric level in the surficial clay have an inverse effect on the factor of safety of the slope with respect to stability; that is, a high water table gives a low factor of safety and vice versa. As is shown later in Figure 41, the high water table condition can be expected immediately following spring thaw, following intensive irrigation, or after prolonged precipitation. The minimum water table condition is reached during winter when there is no recharge. This is the major reason why most slope instability occurs immediately following spring thaw, or after periods of prolonged precipitation.

While water levels in the surficial clay govern shear failure, the piezometric level and gradients in silt, sand and gravel strata govern the rate at which piping occurs. When sufficiently high seepage gradients occur, soil particles are carried from the slope by discharging groundwater, leaving the bowl-like features characteristic of piping. The ravines developing north of the President's Residence result from discharge from sand and gravel units within the till. The best documented of these is Petursson's Ravine (Figure 19) which has developed because of seepage discharge from the Forestry Farm aquifer.

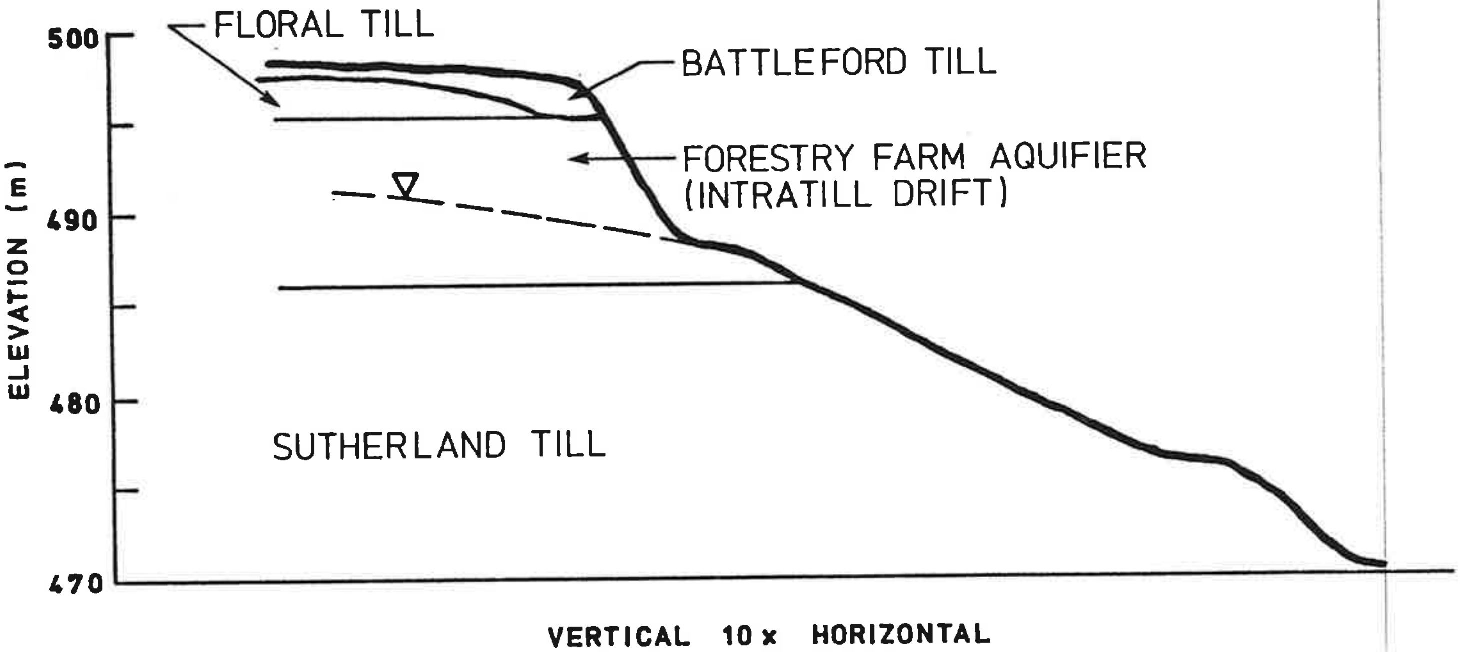
The impact of groundwater level on stability of the slope can be related to the angle of internal friction (Ø') of the soil. A dry soil will stand at an angle of repose equivalent to the angle of internal friction (Ø'). However, a saturated soil with no seepage forces and with groundwater levels at the soil surface will stand at an





SCHEMATIC REPRESENTATION OF VARIATION IN (a) PIEZOMETRIC LEVELS, (b) FACTOR OF SAFETY WITH TIME

FIGURE 18



PROFILE SHOWING STRATIGRAPHY AT PETURSSON'S RAVINE
(AFTER CLIFTON et al 1981)

FIGURE 19

angle of repose of only one-half of the internal friction angle (Ø'/2). Thus, considering the clay soil with Ø' = 18, a dry clay slope would stand at an angle of 18° (3:1) while a similar slope with a water table at surface would stand at an angle of only 9° (6:1).

Time

The stability of slopes varies with time, both in response to changes in groundwater conditions as discussed above, and in response to very slow changes in soil strength with time. The changes in piezometric levels and factor of safety are cyclical (Figure 18). The change in soil strength is less well defined. It is well documented that many overconsolidated clay soils slowly lose strength with time. This is called the softening effect. As the soil softens, the factor of safety of the slope slowly drops until, reaching unity, a landslide occurs. For this reason, slopes which appear to be stable for years may start to undergo strain, as evidenced by bulging slopes or sagging pavements, and then suddenly fail. This phenomenon must be carefully considered in planning developments on slopes, or in assessing the results of slope monitoring. Warping taking place in roadways and curves along St. Henry Avenue and Saskatchewan Crescent may be a manifestation of this phenomenon.

6.0 MECHANISMS OF SLUMPING

Types of the Landslides

Six distinct landslide types occur within the study area. These include:

- rotational;
- translational;
- composite;
- graben;
- debris flows;
- piping.

The above landslide types are illustrated in Figures 20 to 22 inclusive.

Rotational and Composite Landslides

The most common types of landslides in Saskatoon fall into the rotational and composite categories (Figure 20). These slope failures usually involve a sliding mass of soil which moves downward at the crest with backward rotation, and moves laterally outward at the toe. The movement generally occurs along a well defined shear zone above which the soil moves and below which the soil remains undisturbed.

In slopes which have relatively homogeneous material properties, the zone of shear forms a circular arc extending from near the toe to the headscarp which is a point beyond the crest of the slope. This type of circular slip is shown in Figure 20a.

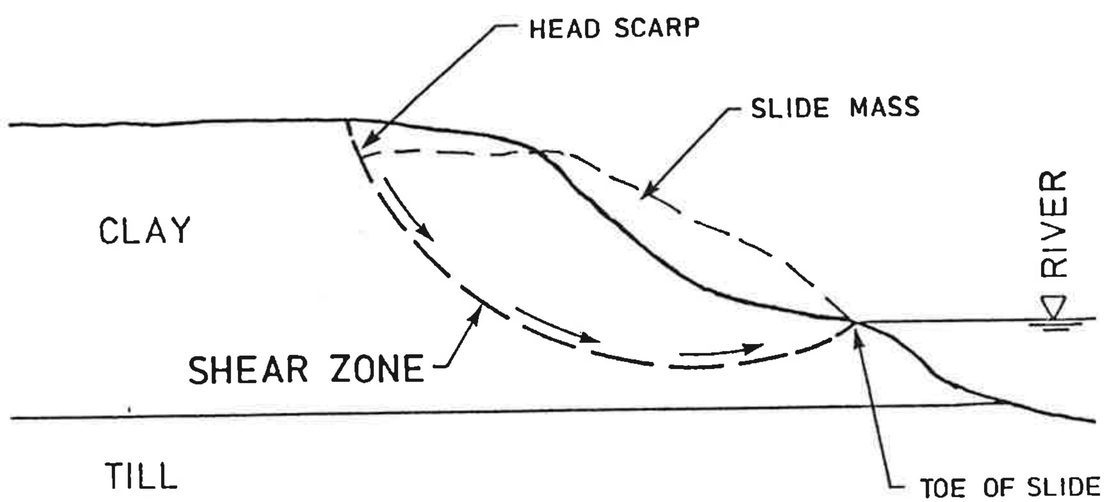
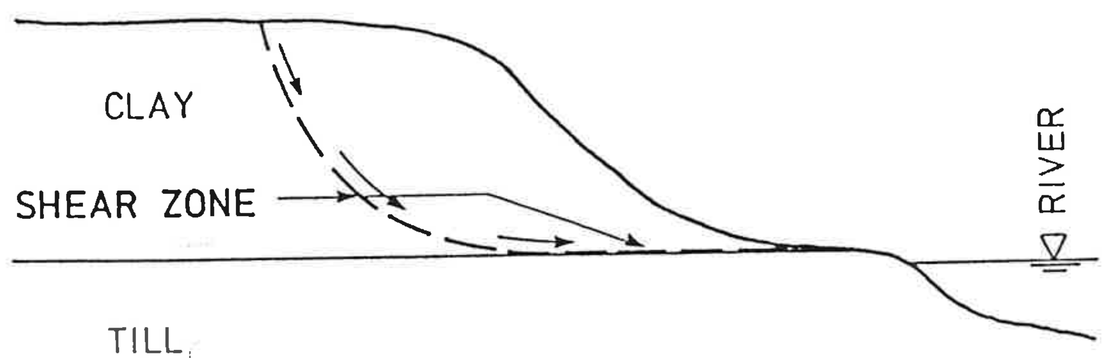


FIGURE: 22a CIRCULAR ARC SHEAR ZONE IN HOMOGENEOUS MATERIAL



NONCIRCULAR SHEAR ZONE IN NONHOMOGENEOUS MATERIALS

FIGURE 20

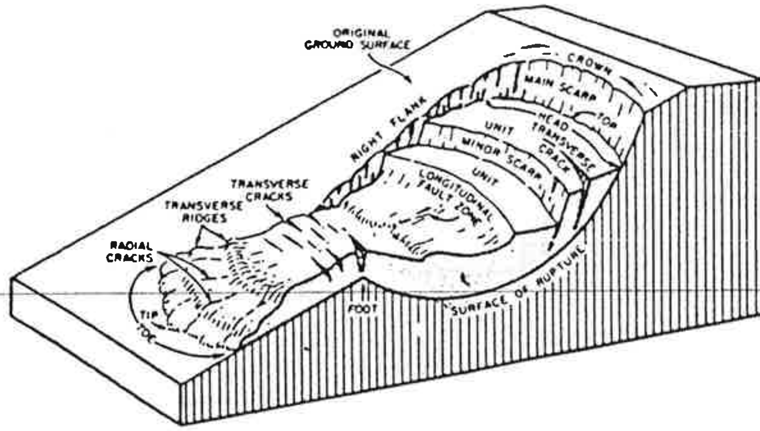
Lateral and vertical variations in shear strength alter the shape of the circular arc. In cases where a variation in stratigraphy occurs, the shear zone will follow the path of least resistance; that is, it will travel through the weakest soil. These irregular slip surfaces are called composite or noncircular slip surfaces. Most of the landslides in Saskatoon have noncircular slip surfaces. This type is illustrated in Figure 20b. Typically, the shear zone is defined by a portion of a circular arc in the lacustrine sediments and runs horizontally in the highly plastic clay just above the till-contact.

Translational Landslides

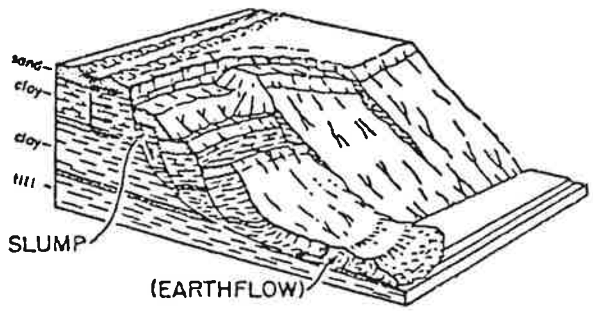
Translational landslides, illustrated in Figure 22, develop where a very weak stratum overlies a much stronger one. Movement is largely horizontal along the upper boundary of the strong layer. This type of movement is rare in Saskatoon, although the movements along the bench below the headland at Diefenbaker Park appears to have been largely translational after the initial composite failure had formed in the lacustrine clay.

Debris Slides

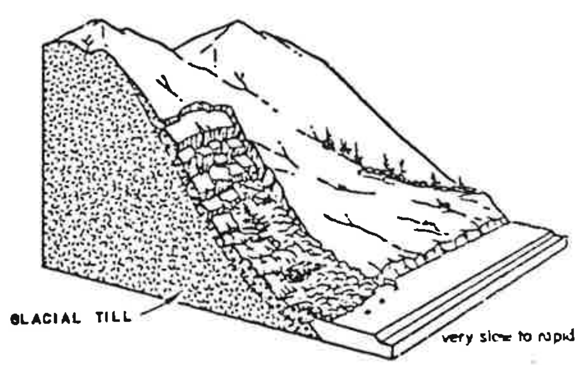
Debris slides are shallow seated failures that form in the very strong glacial till. The shear surface is seldom more than 1 m below the natural ground surface. These slides typically occur immediately following break up when the near surface soils have been softened by freezing and thawing and infiltrating water. This type of slide is illustrated in Figures 21 and 23 following.



ROTATIONAL



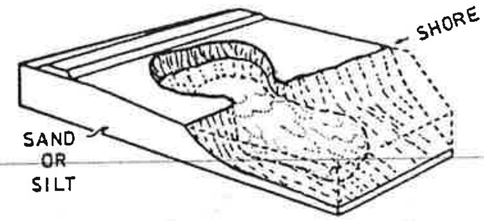
COMPOSITE



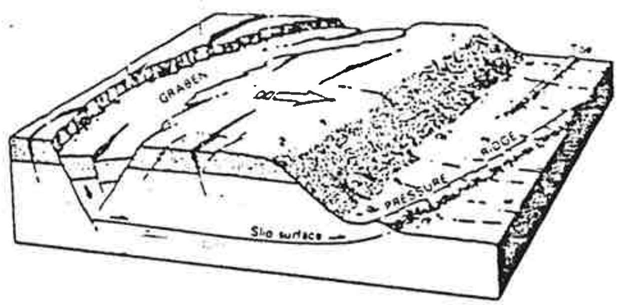
DEBRIS SLIDE

DIAGRAM SHOWING TYPICAL ROTATIONAL COMPOSITE AND DEBRIS SLIDES (AFTER CLASSIFICATION OF LANDSLIDES, HIGHWAY RESEARCH BOARD LANDSLIDE COMMITTEE)

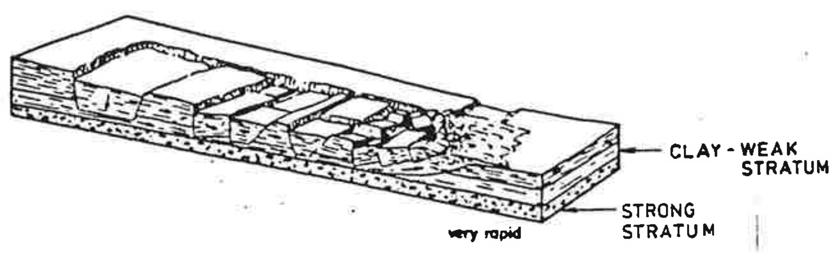
FIGURE 21



PIPING



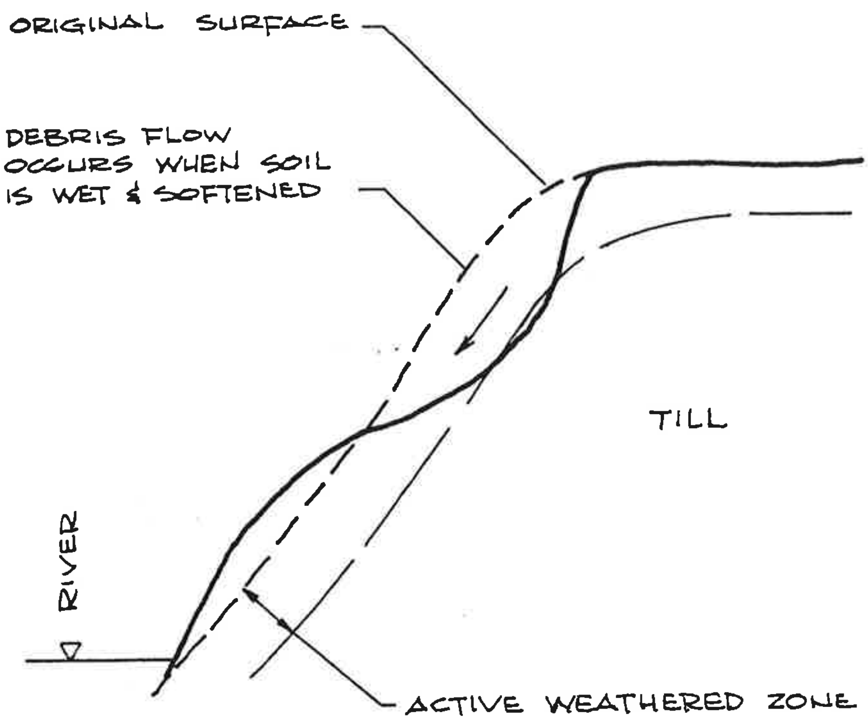
GRABEN BLOCK



TRANSLATIONAL

DIAGRAM SHOWING TYPICAL PIPING, GRABEN BLOCK AND TRANSLATIONAL SLIDES
(AFTER CLASSIFICATION OF LANDSLIDES, HIGHWAY RESEARCH BOARD LANDSLIDE COMMITTEE)

FIGURE 22



DEBRIS SLIDES IN WEATHERED ZONE OF TILL

FIGURE 23

Debris slides are common in the steep fill slopes or on over-steepened till slopes. They are particularly common on the east bank till slopes between the President's Residence and the Ski Jump.

Piping

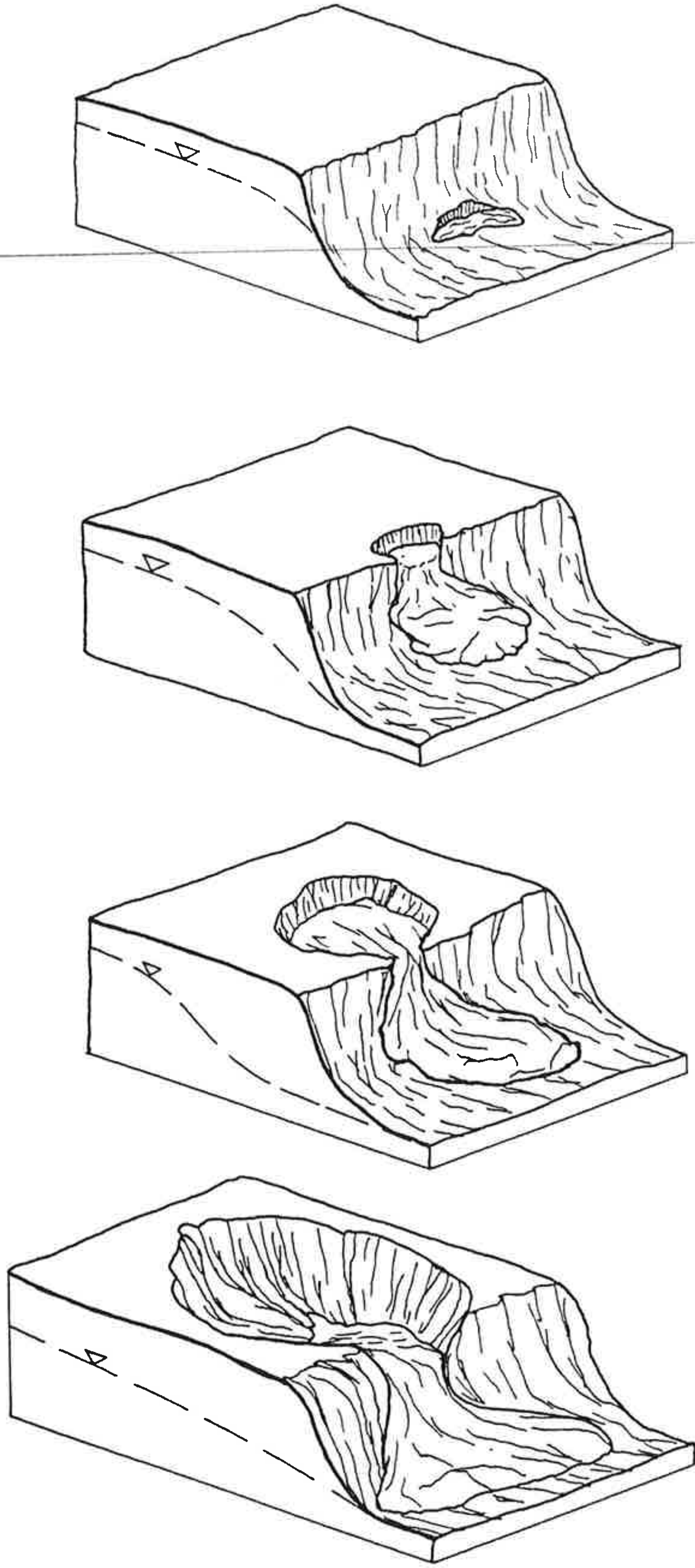
Piping features have been discussed previously. They develop where water table and gradients are high in cohesionless materials. The typical amphitheatre shape and features are illustrated in Figure 22. The progression of piping to form these features is illustrated in Figure 24.

Graben Block Failures

When a landslide mass moves largely horizontally on a deep seated weak layer, the main slump block tends to move horizontally and remain almost intact. A block of material moves almost vertically at the headscarp forming a graben block. This type of failure, illustrated in Figure 22, can be observed in the vicinity of Beaver Creek. A tongue of material moves horizontally, driven by the wedge of soil at the rear. The "wedge" remains intact and drops vertically as the slide moves riverward.

Ravine Erosion

Instability has been observed along ravines in the vicinity of Diefenbaker Park and at Devil's Dip on the University of Saskatchewan Campus where surface water has been concentrated in a pre-existing ravine. Cracking of the



STAGE - 1

SEEPAGE ON FACE
SMALL SEEPAGE SCAR

STAGE - 2

FIRST PIPING FEATURE
(OVERHANGING BANK &
TUNNELS IN SOIL)

STAGE - 3

MATURE AMPHITHEATRE

STAGE - 4

ANCIENT PIPES
DEVELOPED INTO RAVINES

STAGES IN DEVELOPMENT OF A RAVINE FORMED BY PIPING

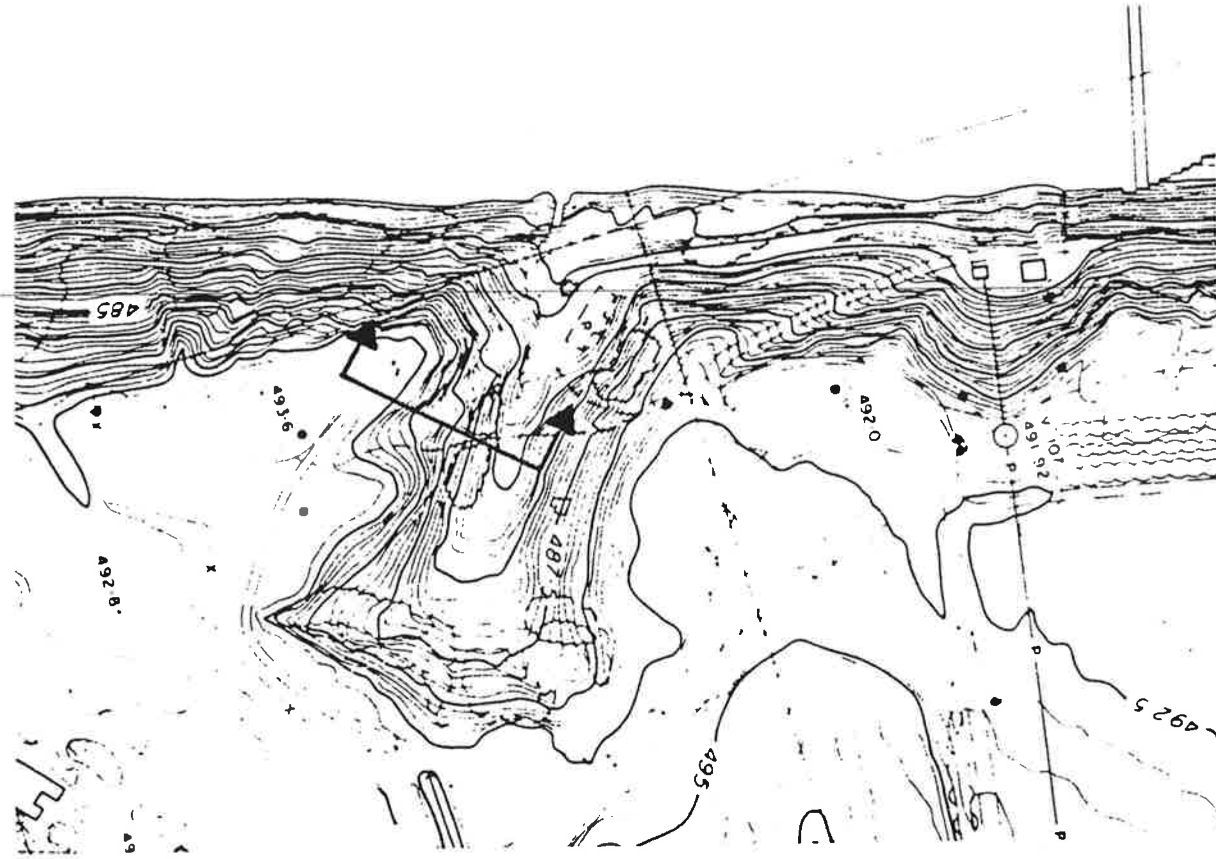
FIGURE 24

ground parallel to and a short distance back from the crest of the eroded gullies has taken place. Slumping of the ravine slide slopes will eventually occur because of the oversteepening of these slopes by erosion and because of the existence of high seepage gradients within the slopes leading to some piping at the toe. A sketch plan and section of a typical gully is shown in Figure 25. The shallow seated "debris slide" type failure that takes place on adjacent oversteepened till slopes is illustrated in Figure 23.

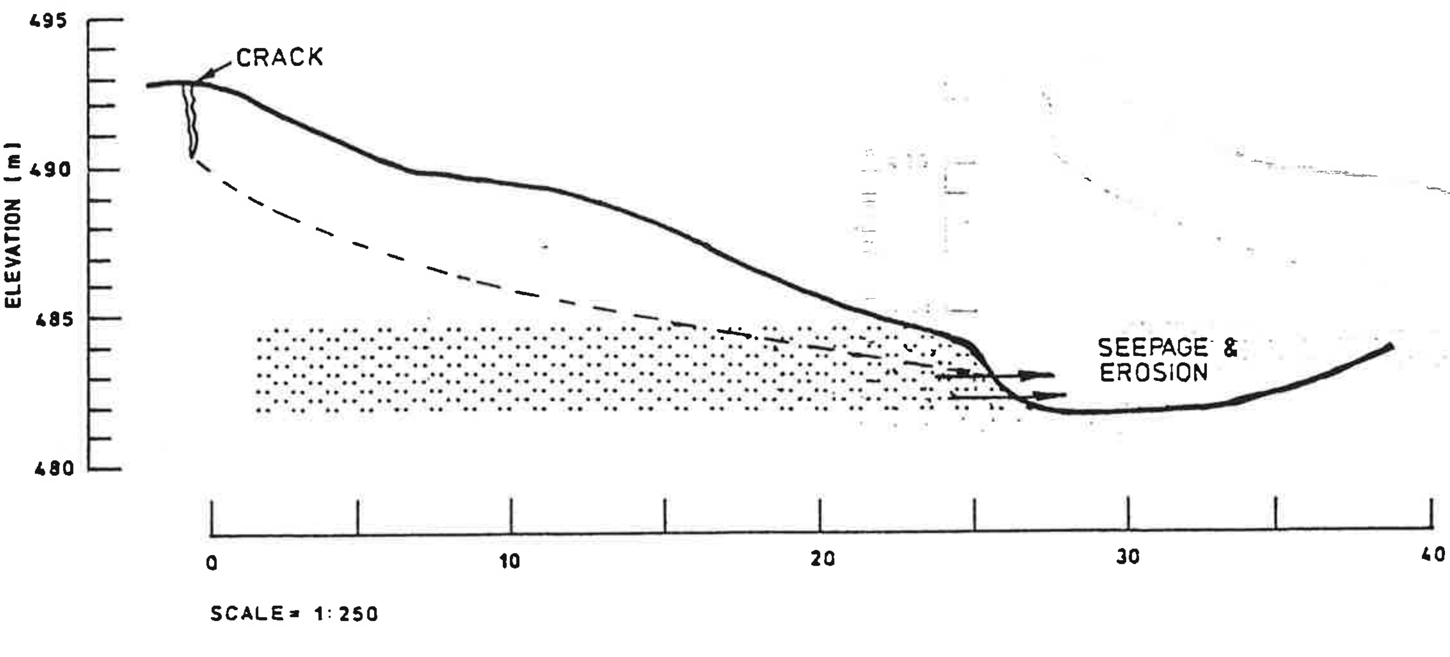
6.1 Indicators of Slope Movement

Large slope movements can usually be detected by visual inspection. Individual slump blocks, deformed slopes, tension cracks and large displacements of the ground surface are readily identifiable, particularly when compared with the features of a stable, mature slope as illustrated in Figure 26.

A stable natural slope goes through a progression of changes in response to erosion, depositional and biological action. An immature slope, has the angular features illustrated in Figure 26a. With time, erosion takes place at the crest and deposition at the toe leaving the classic sigmoidal slope illustrated in Figure 26b. Both of these slopes have a planar sloping section which is concave downwards at the crest and concave upwards at the toe. By comparison, the unstable slope illustrated in Figure 26c is hummocky rather than planar; is poorly drained with



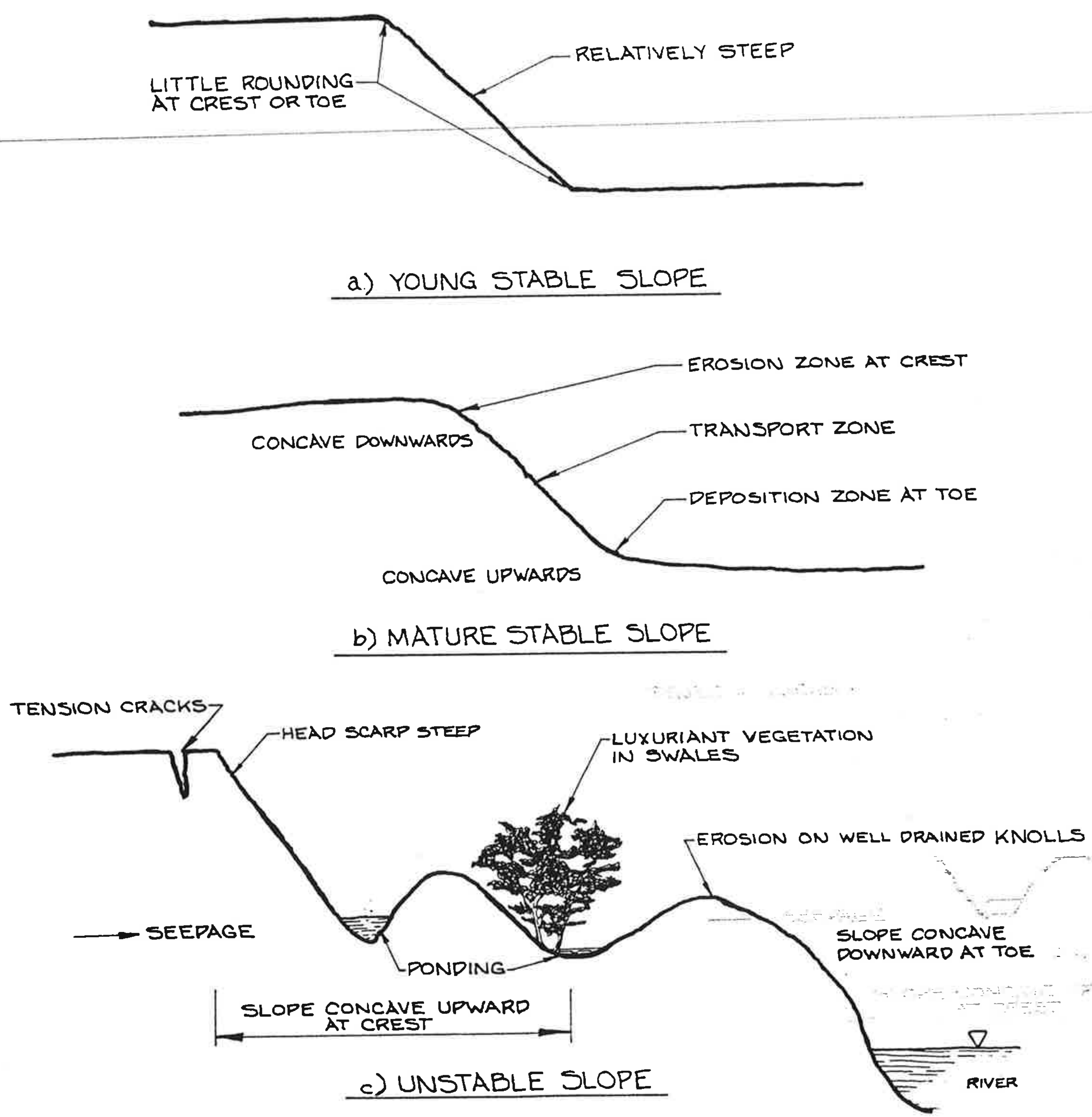
SCALE = 1:2000



SCALE = 1:250

INSTABILITY ALONG RAVINE SLOPES, SKI JUMP AREA

FIGURE 25



TYPICAL PROFILES OF SLOPES

FIGURE 26

numerous hummocks on the slope; has a steeply sloping headscarp that is concave upward; and a bulging toe portion which is concave downward.

Similar features can be seen on many slopes within Saskatoon. Young slopes are found along ravines and young headscarps. Mature, well rounded slopes are found in the strong tills north of the University. The classic unstable slope profile can be seen in many stages of development, from immature at Diefenbaker Park and Queen's House of Retreat, to a more mature stage along Saskatchewan Crescent.

Slow deformation of a slope, especially in the early stages of landslide movement, are difficult to detect without reliable instrumentation. Subtle indicators may be gentle bulging of the slope, trees with bent or curved trunks, and the presence of poorly defined tension cracks marked only by a slight linear depression near the crest of the slope.

Evidence of movement will also appear in any structure or facilities constructed on unstable slopes. Cracks usually first appear in rigid structures. Linear structures such as sidewalks and pavements that cover a large part of a slope usually crack first. Thus, typical indicators of slope movements in urban areas include:

° Vertical and lateral displacement of sidewalks, pavements, railway tracks, fences and other linear structures. Typical examples of these would be cracking patterns in pavements along St. Henry Avenue and Saskatchewan Crescent.

- Misalignment of curbs or guard rails such as along Saskatchewan Crescent and St. Henry Avenue.

- Breakage of water mains and sewer services.

- Tightening of telephone or power lines, or tilting of poles or towers.

- Rapid infiltration of surface water which should normally pond. The rapid infiltration is into cracks in of the soil which may not be obvious to visual inspection.

Slope instrumentation provides the most reliable indicator of slope movement. Slope movement indicators such as those recently installed along the crest of the valley, alignment hubs, accurate survey monuments and other forms of instrumentation which may detect movement in the order of fractions of a millimetre, are much more reliable than visual inspection for detecting shear strain in slopes. Experience in similar geologic settings indicates that instrumentation may sense movement of a shear plane several years in advance of it becoming apparent on the surface.

6.2 Rates of Movement

The rate of movement of landslides varies widely. Displacements may occur at the rate of several metres within hours, or may be as little as 1 or 2 mm per year, the lower limit of reliable detection. Slower movements are more common in stiff soils.

The slides at the Queen's House of Retreat and Diefenbaker Park occurred very rapidly. By comparison, sliding occurring along Cherry Lane is very slow; inspection of structures indicates that the rate must be much less than 10 mm per year.

Only a few reliable long-term measurements of landslide movements have been made within Saskatoon. One of these was the east abutment of the Broadway Bridge where a slope movement indicator was installed by PFRA. Soil at the abutment has been moving riverward at least since the early 1960's and was instrumented in 1963. During the period of 1963 to 1968, approximately 75 mm of displacement occurred, an average of 15 mm per year, but 50 mm was measured in a single year (See Section 8.0).

The rate of movement depends on the ratio of driving to resisting forces. This is influenced by water levels, soil property, movement type and slope history. Slope failures on new slip surfaces in the lacustrine clay should be more rapid than movements re-initiated on old shear zones. The reason for this is the sudden reduction in shear strength from peak to softened or residual values after initial failure occurs (see discussion on shear strength in Section 7.0). The re-initiation of movement on the old landslides does not cause a similar reduction in shear strength.

The geometry of the slopes at Diefenbaker Park suggest that shear movement in this area was relatively rapid. Investigation of the Diefenbaker Park landslide (Clifton Associates Ltd., 1984d) indicated that, once movement was initiated, the slide mass redistributed itself into a thin tongue of debris which formed a bench in the slope at the elevation of the top of the till. This unique slide mass shape suggests that considerable energy was expended and the soil must have had very low shear strength, once disturbed, to be distributed in this fashion. The soils at this location, with saturated loose sand in the lacustrine sediments, are conducive to liquifaction taking place once the initial shear strain occurs. In this situation, movement would be very rapid, perhaps in the order of 10 m or more in less than a minute.

7.0 SLOPE STABILITY ANALYSIS

A stable slope is one that is in equilibrium with its physical environment. All forces acting on the slope are such that the forces which want to disrupt the equilibrium condition (driving forces) are less than the forces that want to preserve the equilibrium condition (resisting forces). When changes occur in the physical environment, changes in the driving and resisting forces acting on the slope result. These forces consist of three components including:

- horizontal forces;
- vertical forces;
- moments.

When the driving forces become greater than the resisting forces, the slope can no longer remain in equilibrium and slumping occurs. The slope can only become stable again by changes in the physical environment which make the resisting forces greater than the driving forces.

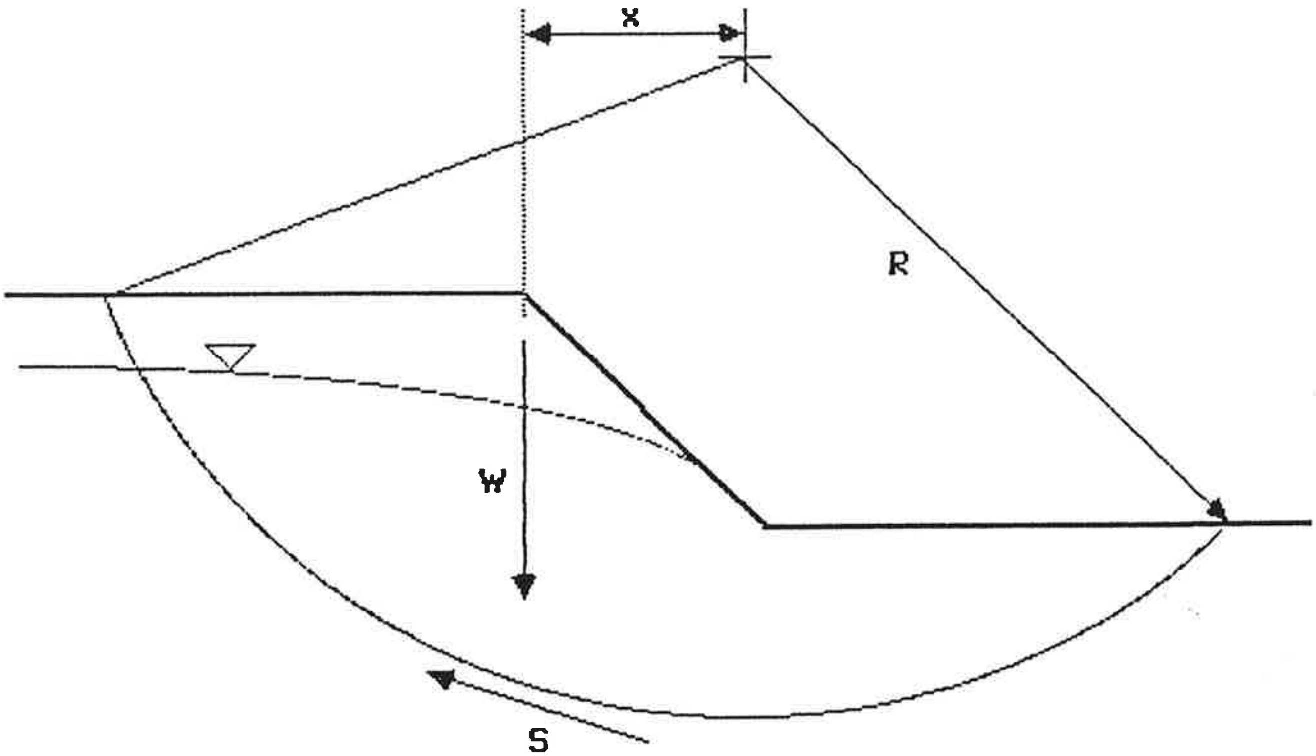
The degree of stability of the slope is called the factor of safety and is the ratio of the resisting forces to the driving forces. When the slope is stable and at rest, it has a factor of safety greater than 1. When the slope is unstable and slumping occurs, the driving forces are equal to or greater than the resisting forces and the factor of safety is 1 or less than 1.

The driving forces acting on a slope are due to gravity, such as the weight of soil, the weight of manmade structures such as buildings built on the slope, and other applied forces such as transient forces due to traffic. The latter are usually small in relation to other gravitational forces. The primary resisting forces are the available soil resistance (soil shear strength) and the weight of soil or manmade structures which counteract the driving forces. The concept of the system of forces acting on a slope is shown in Figure 27.

The factor of safety of a slope can be calculated numerically by calculating or estimating all the forces acting on the slope. These calculations require knowledge of:

- the geometry of the slope, usually obtained by survey;
- the piezometric pressures within the slope, usually obtained through installation of piezometers;
- any external loads applied to the slope, usually estimated in engineering calculations;
- the unit weight of the soil, usually obtained from laboratory testing;
- the shear strength of the soil, either calculated from analysis of existing landslides or obtained from laboratory shear strength tests.

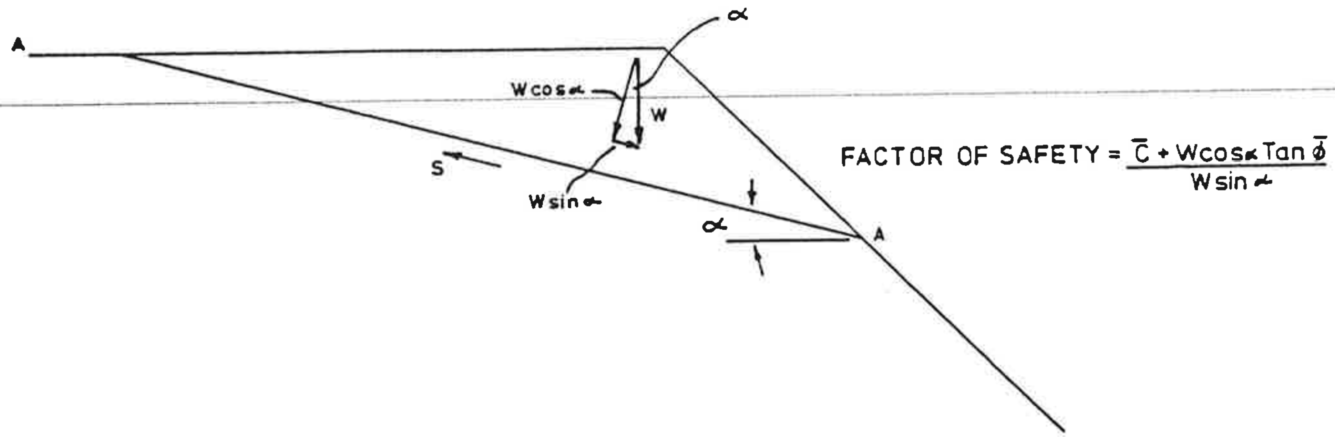
A very simple analysis is illustrated in Figure 28. The driving forces illustrated are the total weight of the soil in the sliding wedge which causes the wedge to move down the shear plane. The resisting forces are due to the



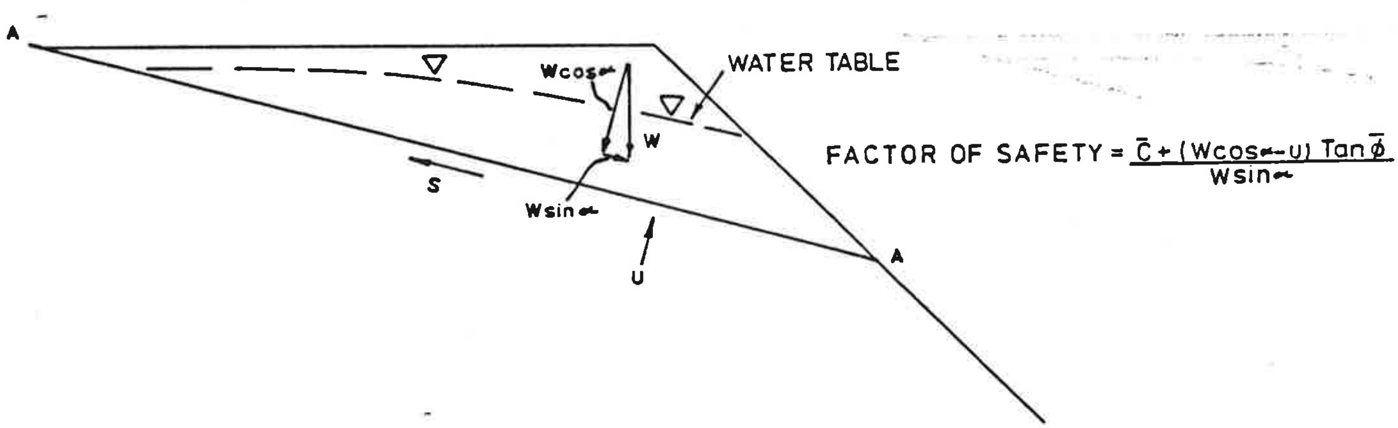
FORCES ACTING ON A SLOPE

FIGURE 27.

ANALYSIS OF SIMPLE TRANSLATIONAL FAILURE



(a)



(b)

W = weight of soil mass above plane A–A

S = shear strength along plane A–A = $\bar{C}$ + WCosα Tan $\bar{\phi}$
or $\bar{C}$ + (WCosα−u) Tan $\bar{\phi}$

u = force due to water pressure

$\bar{C}$ = effective cohesion

$\bar{\phi}$ = effective angle of internal friction

ANALYSIS OF A SIMPLE TRANSLATIONAL FAILURE

FIGURE 28

strength of the soil discussed below. The factor of safety is calculated as:

$$\text{factor of safety (F)} = \frac{\text{resisting forces}}{\text{driving forces}};$$

$$= \frac{c' + W \cos\alpha \tan \phi'}{W \sin}; \text{ where;}$$

c' = effective cohesion;
ϕ' = effective angle of internal friction;
W = mass of soil; and
α = angle of the sliding plane to the horizontal.

If a water table is considered as shown in Figure 28b, the factor of safety is calculated as:

$$F = \frac{c' + (W \cos \sigma - u) \tan \phi'}{W \sin\alpha}$$

This simple example illustrates the interrelationship of the soil mass (W), soil strength (c' and ϕ'), pore water pressure (u) (determined from piezometer levels), and the slope geometry which controls the angle α .

This method of analysis is called a limit equilibrium analysis, which means that the slope will remain stable as long as resisting forces are greater than disturbing forces, ie. the factor of safety is greater than 1.0. However, a computed factor of safety of 1.0 means the available shear strength of the soil has all been mobilized and a condition of limiting equilibrium exists and failure is impending. Alternatively, a computed factor of safety of greater than 1.0 means that the available shear strength of the soil along the sliding plain is more than is required for a condition of limiting equilibrium. Under such a condition, no landsliding could take place.

A method of slices analyzing rotational or composite failure surfaces is more appropriate to conditions along the river valley in Saskatoon. Appendix B introduces the mathematical equations that must be solved for the simplified Bishop's method, one of the more reliable analytical techniques for slopes in this geologic setting.

Shear Strength

The resisting forces along the shear plane are mostly due to the strength of the soil (τ) and is defined as follows:

$\tau = c' + \sigma' \tan \phi'$, where:

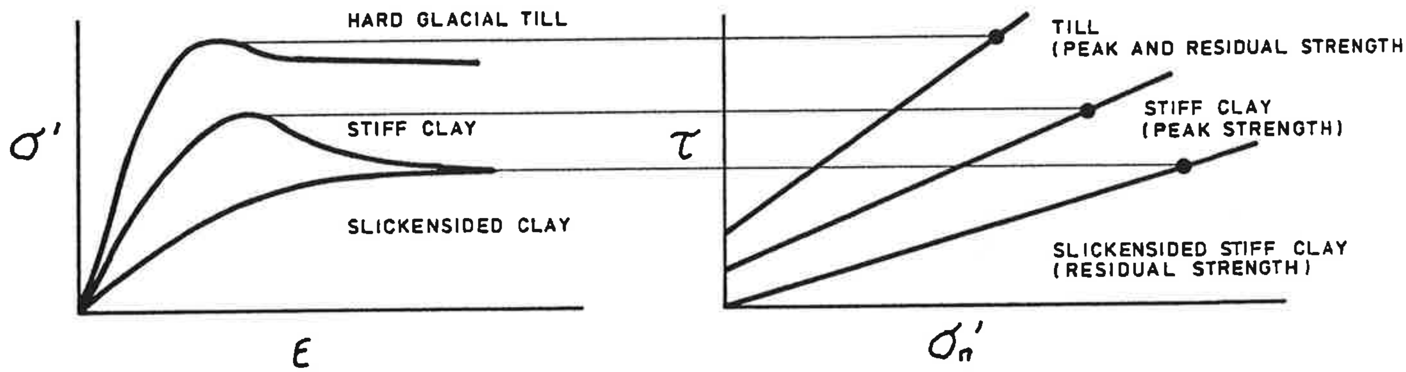
τ = effective soil shear strength;

σ' = effective normal stress acting on the shear plane;

c' = effective cohesion; and,

ϕ' = effective angle of internal friction.

This relationship is known as the Mohr-Coulomb failure criteria and may be represented graphically in the manner illustrated in Figure 29. In order to develop shear resistance, a soil must undergo some shear strain. The amount of strain needed to develop a particular soil strength is an intrinsic soil characteristic and varies according to soil type as illustrated in Figure 29. Some soils, particularly highly plastic, montmorillonitic clays



STRESS-STRAIN CURVES AND STRENGTH ENVELOPES
FOR TILL AND CLAY

FIGURE 29

exhibit a substantial reduction in strength after they reach their peak strength as illustrated in Figure 29. Such a soil exhibits several effective strength parameters: namely the peak strength; the fully softened strength where enough strain has been experienced to remove the cohesion intercept; and, the residual strength which is the strength remaining after the shear plane has experienced large strains. Residual strength behaviour can be expected in the highly plastic lacustrine sediments, while the dense, well graded tills exhibit a peak and softened strength but show little reduction in the effective angle of internal friction at large strains.

The water pressure in the soil (pore water pressure) controls the effective strength of the soil. This influence comes through the reduction of the effective stress (σ') between soil particles. Thus:

$\sigma' = (\sigma - u)$, where:

σ' = effective stress perpendicular to the shear plane;

σ = total stress; and,

u = the pore pressure acting on the shear plane.

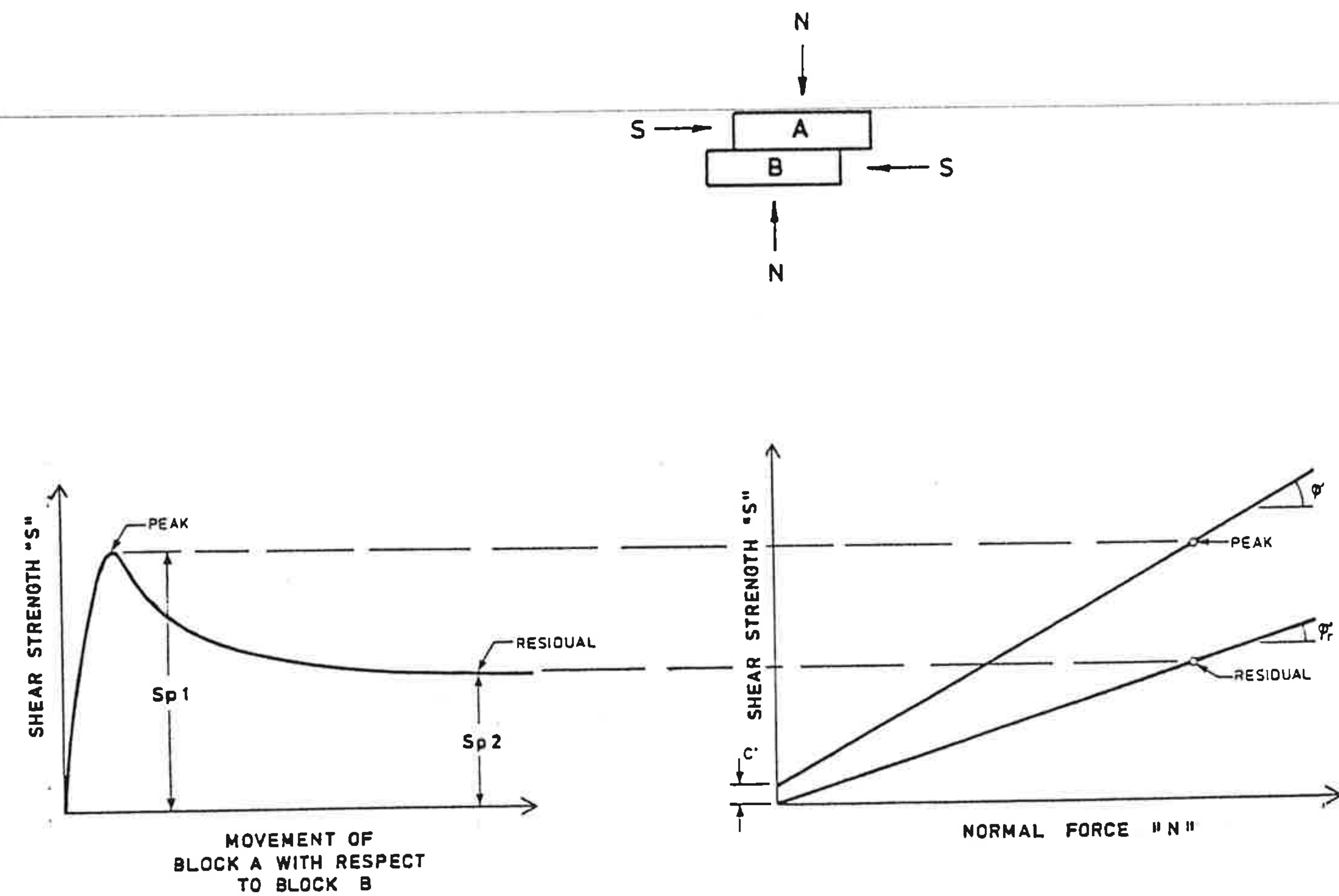
Considering pore water pressure, the effective shear strength of the soil then becomes:

$$\tau = c' + (\sigma - u) \tan \phi'.$$

Determination of Shear Strength

The soil strength properties are obtained by means of a laboratory shear testing program. This is commonly accomplished with the direct shear test or the triaxial test. These tests are normally conducted only on cohesive soils since sand, such as that found in the surficial stratified drift, does not present a problem as far as selection of shear strength parameters is concerned. The effective cohesion of sand is normally presumed to be 0 and the angle of friction (Ø') can be reasonably estimated indirectly from standard field and laboratory tests such as standard penetration tests in the field and grain size determinations in the laboratory. The direct shear test or the triaxial test is commonly used to estimate the effective soil shear strength parameters c' and Ø' in the laboratory. Both require undisturbed samples.

In the direct shear test, the upper half of a small sample of soil is sheared with respect to the lower portion as shown in Figure 30. For each value of stress normal to the shear plane, a relationship between stress and strain can be obtained. As indicated in Figure 30, this shear stress rises to a maximum or peak value after which it then decreases to a residual value as shown. If another test is run using a higher normal stress, a proportionally higher shear stress is required to shear the specimen. A graphical plot of shear strength versus normal force defines the failure envelope from which the effective cohesion (c') and the effective angle of internal friction (Ø') can be interpreted.



$\bar{\phi}$, $\bar{C}$ = Peak shear strength parameters, effective angle of internal friction and effective cohesion

$\bar{\phi}_r$ = Residual shear strength parameters, effective angle of internal friction

DETERMINING SHEAR STRENGTH FROM DIRECT SHEAR TEST

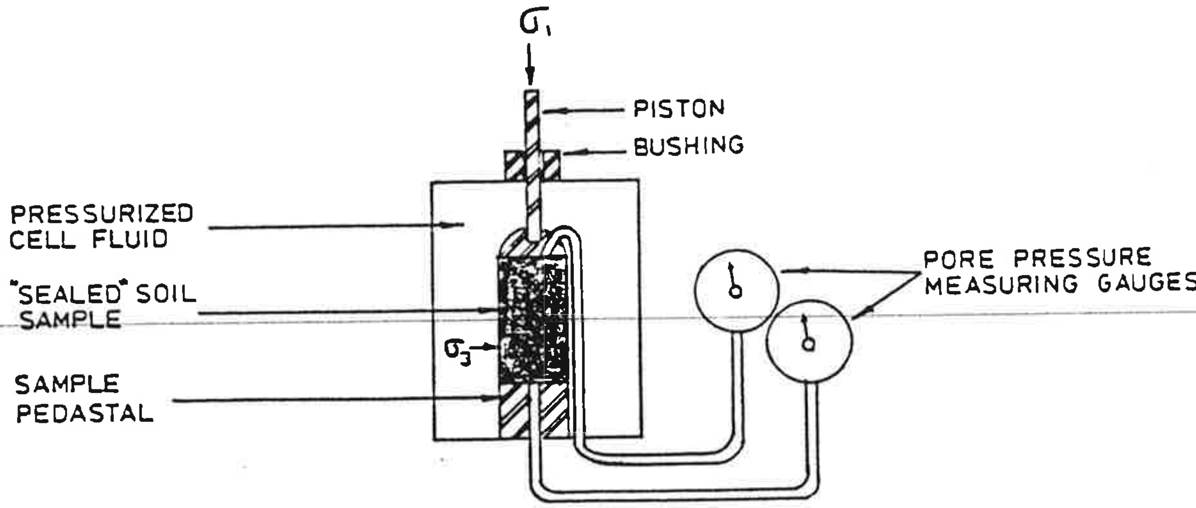
FIGURE 30

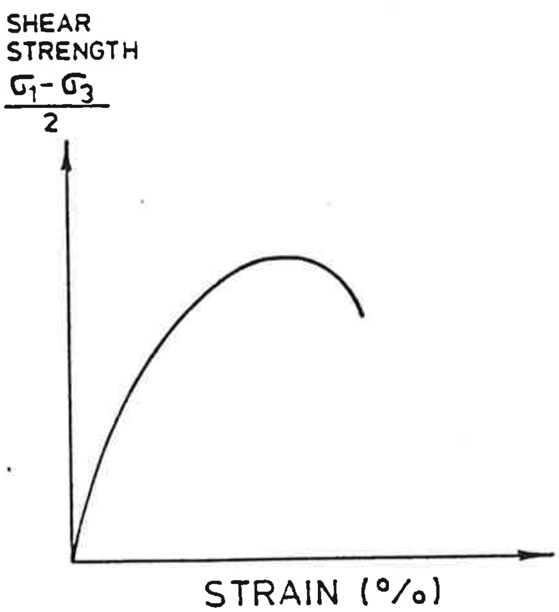
The triaxial test is a more complicated method of testing that allows more accurate simulation of the soil and pore water stresses that exist in nature. In this method, a sample is wrapped in a waterproof membrane and placed in a waterproof cell which can be pressurized to simulate the soil and water pressures experienced by the soil in the slope. Such an apparatus is shown schematically in Figure 31. This apparatus measures not only the shear strength and resulting strain but also measures the pore pressure (u) and allows the strength envelope (c', Ø) of the soil to be defined.

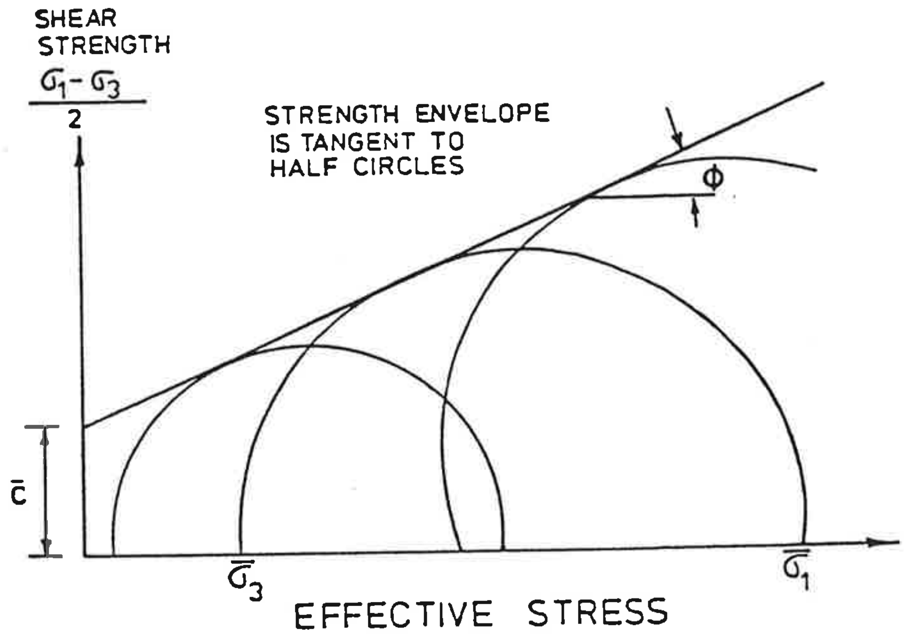
Soil strength must be defined in terms of effective stress to analyze the long term stability of the slopes. Only by this method can the effects of variations of pore water pressures and external stresses be properly evaluated. Methods such as the unconfined compression test or vane shear test, which evaluate strength only in terms of total stresses, are not suitable for use in the analysis of the long term stability of the slopes.

Selection of Shear Strength Parameters

The soil shear strength parameters must be obtained from field investigations which include drilling and sampling of the soils, followed by laboratory testing. Normally, some degree of engineering judgement is required in determining the soil strength parameters that are appropriate for a particular slope stability analysis. The typical process of selecting shear strength parameters is illustrated in Figure 32.

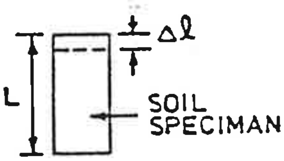




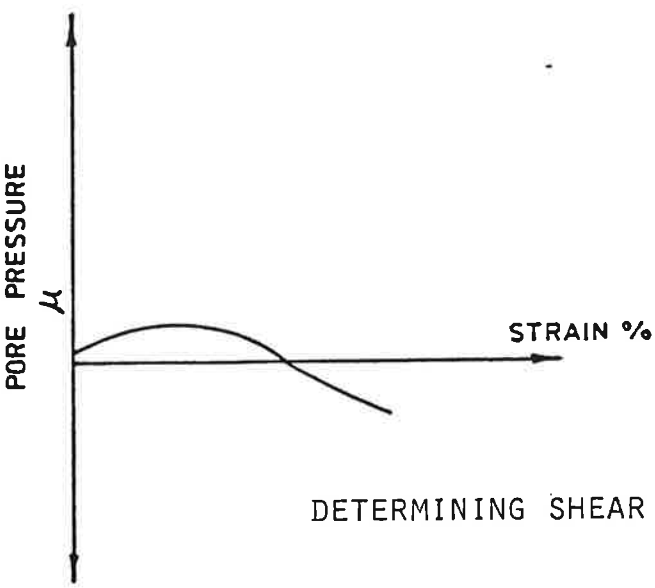


$\overline{\sigma_1} = \sigma_1 - \mu$ AT SPECIFIC STRAIN

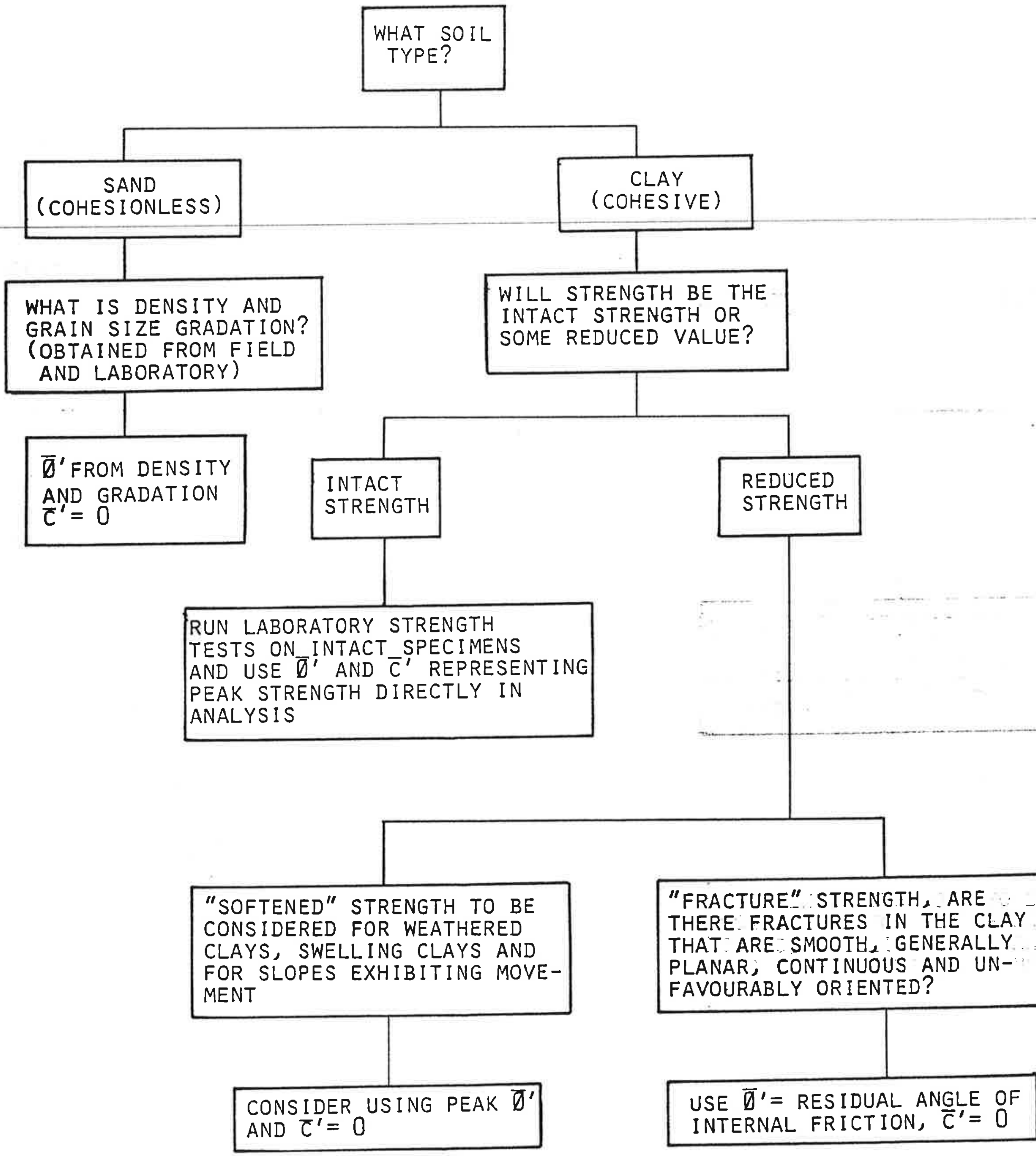
$\overline{\sigma_3} = \sigma_3 - \mu$ AT SPECIFIC STRAIN



$\text{STRAIN} = \dfrac{\Delta l \times 100\%}{L}$



DETERMINING SHEAR STRENGTH FROM THE TRIAXIAL TEST FIGURE 31



PROCESS OF DETERMINING
SHEAR STRENGTH PARAMETERS

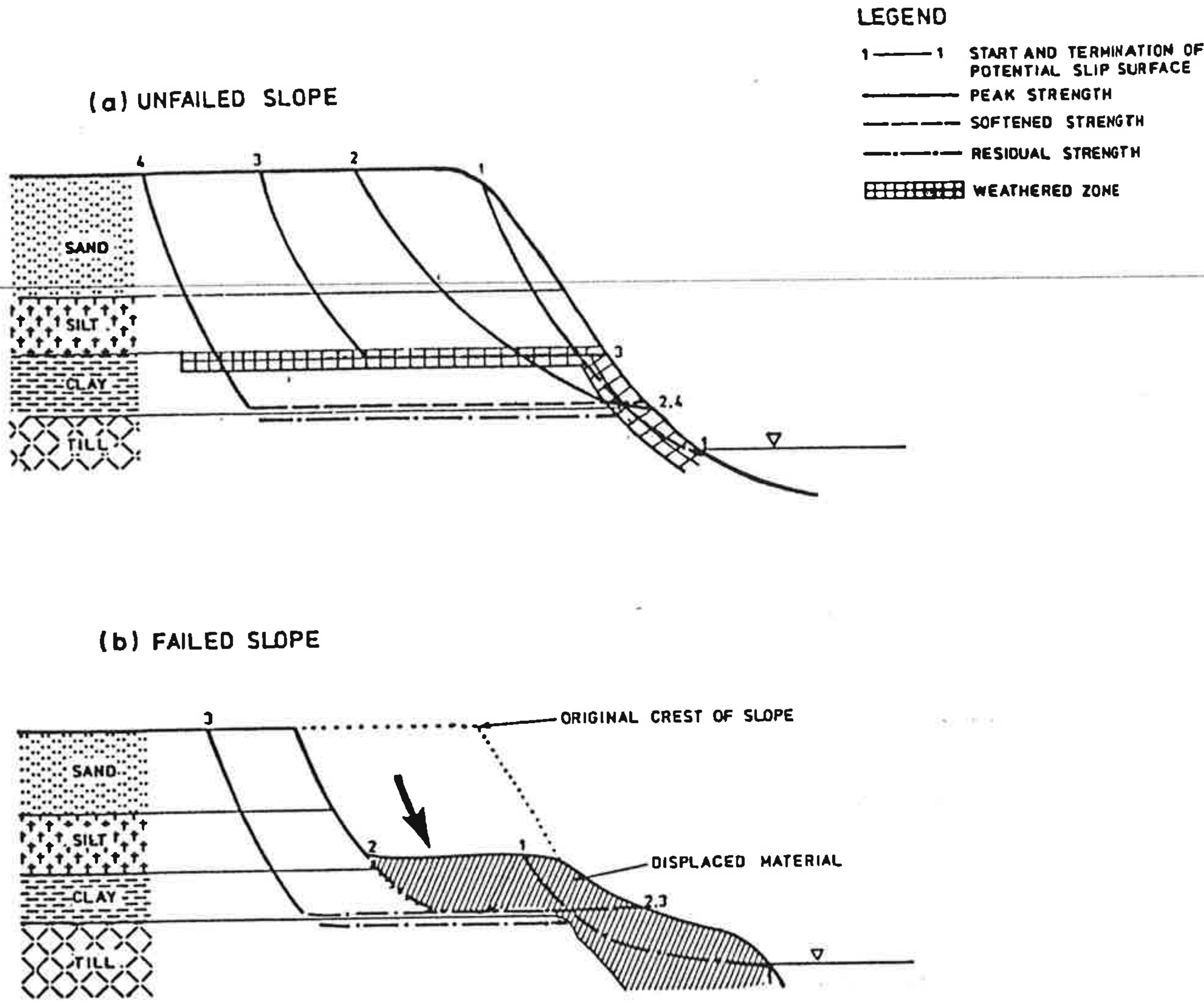
FIGURE 32

The effective shear strength parameters for sand and gravel are commonly estimated from other tests as described above. However, considerable judgement is required in selection of shear strength parameters for clays. A decision must be made on the use of intact peak strength or some value of reduced strength for each problem. In general, the peak strength can be utilized below the depth of weathering and where the soil mass has not been subjected to previous shearing. Such a location would be at some distance behind the headscarp in intact soil which had not been subjected to yielding.

The fully softened strength value is recommended for soils which have been subjected to yielding or have been weathered, thereby reducing the effective cohesion. This strength value is appropriate where discontinuities such as slickensides or polished fracture surfaces are not continuous throughout the soil mass. Examples of this, illustrated in Figure 33, would be surficial weathered zones in till or at the base of lacustrine clay zones immediately behind the headscarp of existing slides.

The residual shear strength value should be used in the analysis of all landslides where movement may be re-initiated along an existing shear plane. This is the shear strength that can be expected to be mobilized on surfaces that have been previously subjected to extensive movement.

Typical applications of these various parameters is illustrated in Figure 33.



USE OF PEAK, SOFTENED AND RESIDUAL SHEAR STRENGTHS — FIGURE 33

Selection of Porewater Conditions

Porewater pressures active in each soil stratum must be accurately determined to enable proper analysis of the factor of safety. For existing natural slopes, porewater pressures should be determined by installation of piezometers. At a minimum, piezometers should be installed to monitor porewater pressures at the base of the surficial clay and in the intertill sand and gravel between the Battleford Formation and Floral Formation tills. Sufficient piezometers (at least two installations in each) should be installed to determine the piezometric pressure profile in each stratum.

Conditions other than static porewater pressures may have to be considered in some analyses. Hamilton et al (1977), monitored head variations of approximately 3 m in the surficial stratified drift. If piezometric levels are measured in the winter, the analysis should consider a rise in piezometric levels in the surficial stratified drift, while if measured in the summer, the impact of an additional rise in piezometric levels should be evaluated. Judgement as to where water levels are in the hydrologic cycle can be made by comparison with long term monitors such as the closest SRC recording well or the U of S piezometers at the President's Residence and elsewhere on the campus. Under certain conditions, such as evaluation of natural slopes or fills subject to temporary flooding, or in the case of cut slopes which intercept the groundwater table, the rapid drawdown condition should be considered.

Method of Analysis

Several methods of slope stability analysis have been identified. These were explored in detail by Fredlund (1983). The most commonly used method is the Simplified Bishop Method which has been used successfully in the in the analysis of slopes in Saskatoon. It is a method that is easy to use and readily accomodates consideration of non-circular or composite failure surfaces.

Simple slope stability calculations such as the sliding wedge analysis illustrated in Figure 28, can be done by hand. However, calculations for more complex problems are onerous and best done with one of several computer programs which are available for performing the numerical calculations. The most comprehensive programs, Slope-II and PC Slope, were developed by GEOSLOPE Programming Ltd. and have been thoroughly tested in the marketplace. They have gained international acceptance as reliable software capable of handling extremely complex slope conditions. It is recommended that all slope stability analyses for MVA use the Slope-II or PC Slope programs marketed by GEOSLOPE Programming Ltd., and that the Simplified Bishop Method of Slices be specified as the analytical standard. All slopes much be checked for the both the circular and non-circular mode of failure. Those slopes for which the non-circular mode gives a lower factor of safety should be checked utilizing the Morgenstern-Price method of analysis to determine the minimum factor of safety for the slope.

Factor of Safety

The factor of safety required in the analysis will depend on many factors including:

- the assumptions necessary to complete the analysis;
- the reliability of the input data, particularly shear strength and porewater pressure conditions; and,
- the consequences of failure.

Very often, it is very difficult to predict the factor of safety prevailing in a slope, primarily because of the difficulty in accurately assessing the shear strength parameters, and in accurately predicting the exact porewater pressures that will be present on critical potential failure zones. As a result, it is often much more valuable to compare the factor of safety after a development has taken place with the one that prevailed prior to development. Using this procedure, it can be determined whether the factor of safety will be increased or decreased because of a development. If a slope is presently stable and the proposed development increases the factor of safety, the slope can be expected to remain stable if other environmental factors do not change. However, if the predicted factor of safety after development is lower than before development, slope instability may result and a more rigorous analysis is required, including a suitable level of care in defining piezometric conditions and soil strength parameters.

Opinions vary widely on what constitutes an acceptable factor safety. A major factor is the consequences of failure. For example, a lower factor of safety would be accepted on a natural slope where movement would produce little property damage or pose little hazard to public safety. However, a higher factor of safety would be essential where economic loss or public risk were involved.

As a general rule, lower factors of safety invite higher rates of strain in the slope. Experience has shown that when the absolute factor of safety is less than about 1.25, slow "creep" like movements can be expected. The lower the factor of safety, the larger these movements will be. If such movement is unacceptable then the minimum factor of safety should be at least 1.25.

Some pertinent observations can be made with respect to factors of safety. It is widely accepted for instance that the factor of safety for a recently failed slope is approximately 1.0. It has also been found that an increase in the factor of safety of at least 10 percent is required to show a significant stabilizing effect on the slope. Even with this increase, small downslope movements can continue. Conversely, a similar decrease in the factor of safety of a metastable slope may be sufficient to precipitate a landslide.

In summary, the following guidelines with respect to factors of safety are recommended:

° The factors of safety should be determined using reliable input data, accurately determined by field surveys, drilling and sampling programs, piezometer installations and laboratory testing of undisturbed representative samples of the pertinent soil horizons.

- The factor of safety should be determined using the Slope-II or PC Slope software systems and the simplified Bishop's and Morgenstern-Price methods of slope stability analysis as recommended above.

- The stability analysis must consider changes in conditions that would occur in the service life of the slope. This would include factors such as erosion or cutting of the slope, changes in piezometric levels, or additional structural loads or fills. Such analysis should estimate the minimum factor of safety that would be expected to prevail during the service life or design life of the slope.

- The minimum design period for natural slopes should be 10 years and for improved slopes supporting a structural load should be 100 years. Changes in slope geometry through erosion or landscape and in groundwater level through flooding or natural water level fluctuations should be considered.

- The minimum recommended factors of safety are summarized in Table 7.1.

- Where monitoring is specified, it should consist of slope movement indicators and other means for the earliest possible identification of movements in the critical soil strata.

TABLE 7.1

RECOMMENDED FACTORS OF SAFETY

APPLICATION	DESIREABLE FACTOR OF SAFETY	MINIMUM FACTOR OF SAFETY
1. Natural Slope - Little or no economic risk and no safety hazard	1.3	1.1
2. Natural Slope with no Improvement - Failure would produce some economic loss and some risk to public safety	1.5	1.2 (with monitoring)
3. Existing Landslide, Stabilizing for Aesthetic Purposes Only - Low economic or public safety risk	2.0	1.1
4. Improved Slope - Substantial risk of economic loss and some public safety considerations	1.5	1.3 (with monitoring)

° Where a development on an existing stable slope can demonstrate an average degree of improvement of 15 percent in the factor of safety of the slope, and under no condition during the design life of the structure does the factor of safety drop to less than that of the existing slope, further rigorous analysis of the slope would not be required.

8.0 LANDSLIDE HISTORY

Slope instability along the banks of the South Saskatchewan River in Saskatoon is an ongoing problem. Many landslides have occurred in the past resulting in damage to roadways, bridges, buildings and services. The first recorded landslide that the City of Saskatoon, Engineering Department was called on to remedy occurred on Long Hill in the spring of 1913 (City of Saskatoon). Numerous other slides have occurred along the riverbank, primarily on the east side, since that time. The location of these landslides is shown in Figure C1 in Appendix C, a vertical aerial photograph of the river valley through Saskatoon, and also on Drawing S134-1.

A summary of the landslide events which have been documented is presented in Appendix C. Copies of the original reports and drawings have been obtained from the City of Saskatoon Engineering Department who performed most of the work. Each documented landslide is discussed below beginning at the south (upstream) limit of the study area.

Diefenbaker Park and Queen's House of Retreat Slides

The Diefenbaker Park slide (Figure 34) is located on the east riverbank at the south limit of the city. Recent movement of the ancient slide was relatively rapid, and continues to be active. A large amount of material has dropped and moved laterally, leaving a steep, sharp headscarp and a lobe of debris extending into the river. The slide has removed a portion of a historic cemetery, is interfering with park development and is considered a hazard to park users.

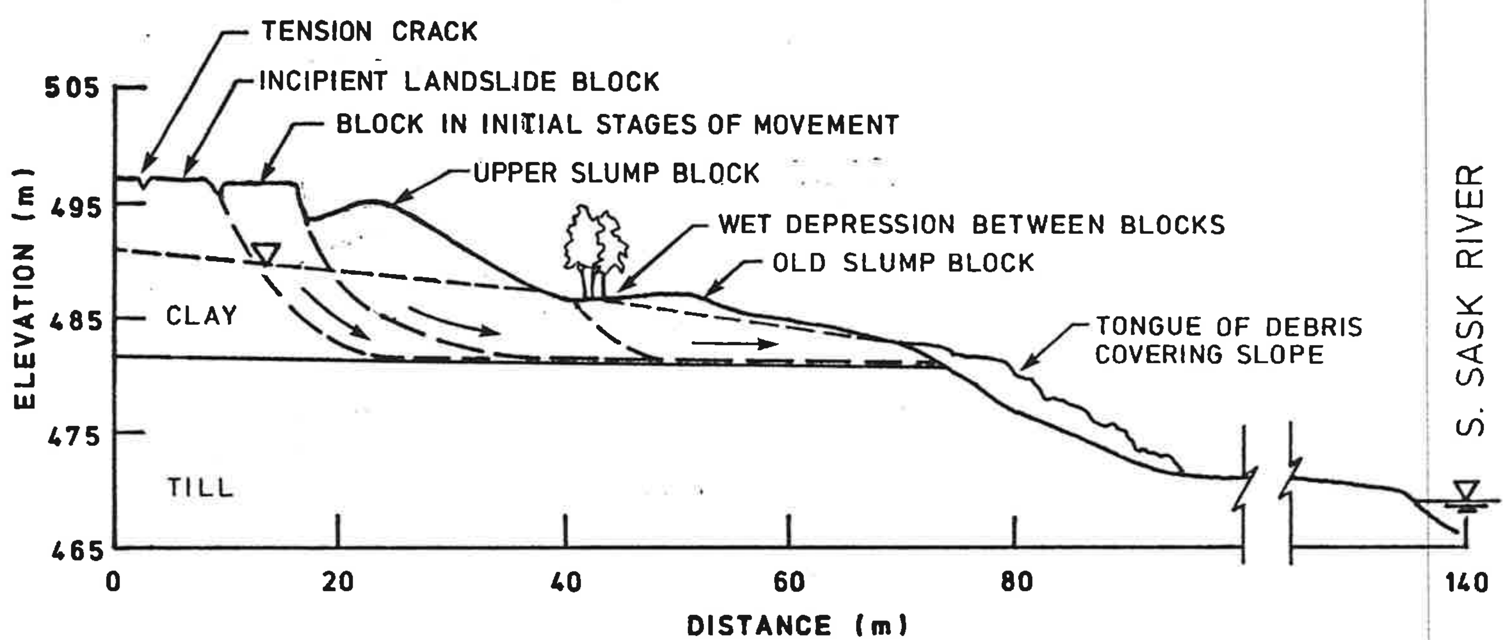


DIAGRAM SHOWING PROBABLE MECHANISIM OF PROGRESSIVE DEVELOPMENT OF LANDSLIDES IN THICK LACUSTRINE SEDIMENTS AT DIEFENBAKER PARK AND SASKATCHEWAN CRESCENT AT 18TH STREET

FIGURE 34

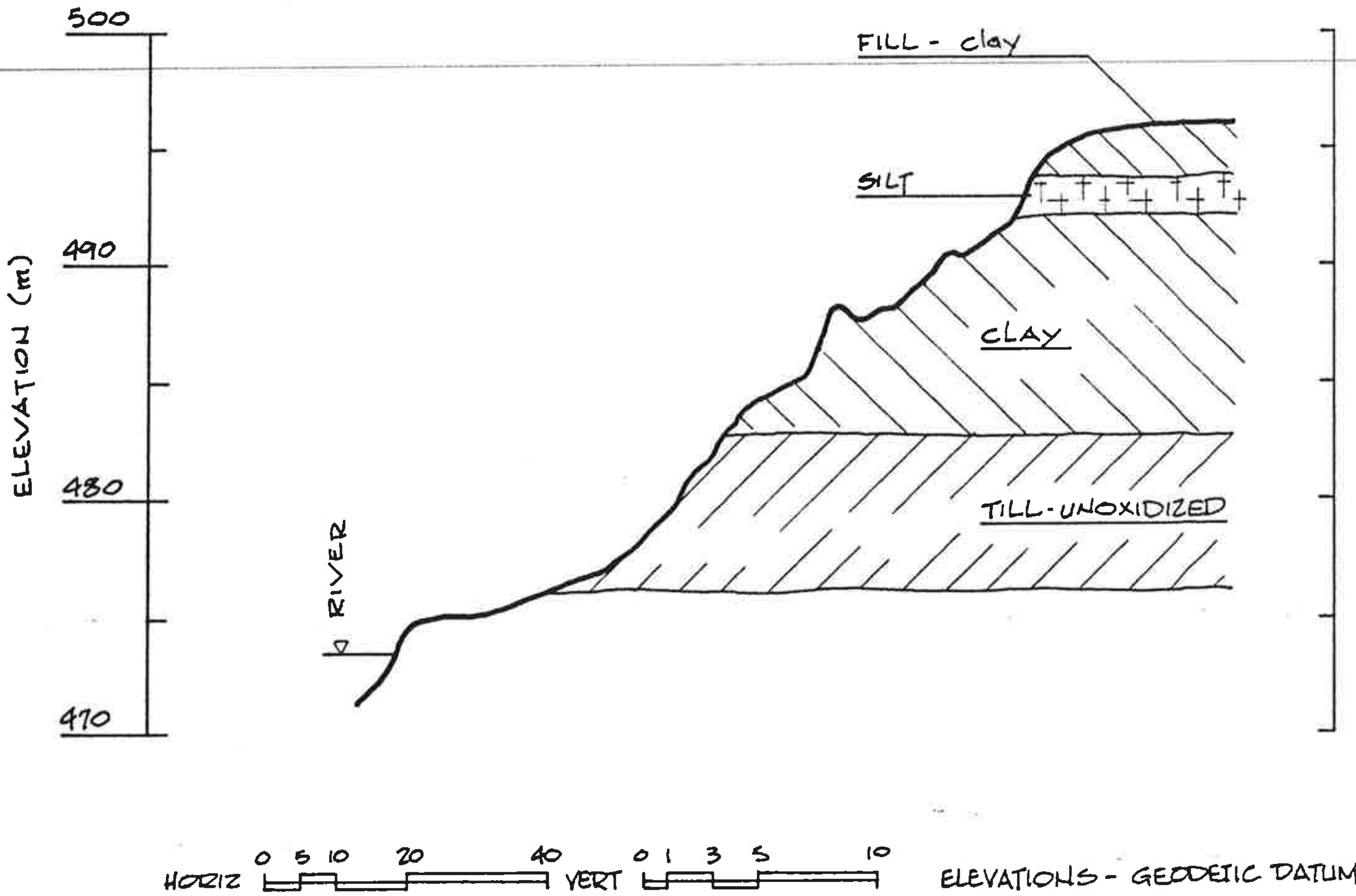
The Queen's House of Retreat slide (Figure 35) is similar to the Diefenbaker Park slide although recent movement may have been initiated by irrigation on the grounds. This slide is also active. The headscarp extends onto private land. In an attempt to stabilize the slope, fill material has been placed at the toe of the slide. However, the base of sliding is above this landfill and the stability has not been improved by filling.

Nutana Collegiate Slide

In the summer of 1960 a slide occurred just below the Nutana Collegiate. This slide (Figure 36) encroached upon the tennis courts on the uplands. The failure plane intercepted the slope near the elevation of Saskatchewan Drive. Slope geometry modification was undertaken which included excavation at the crest and rock fill along the toe. Surface drainage improvements were also made. This slide has recently been active again. Minor movement occurred in July, 1983, causing dislocation of curbs.

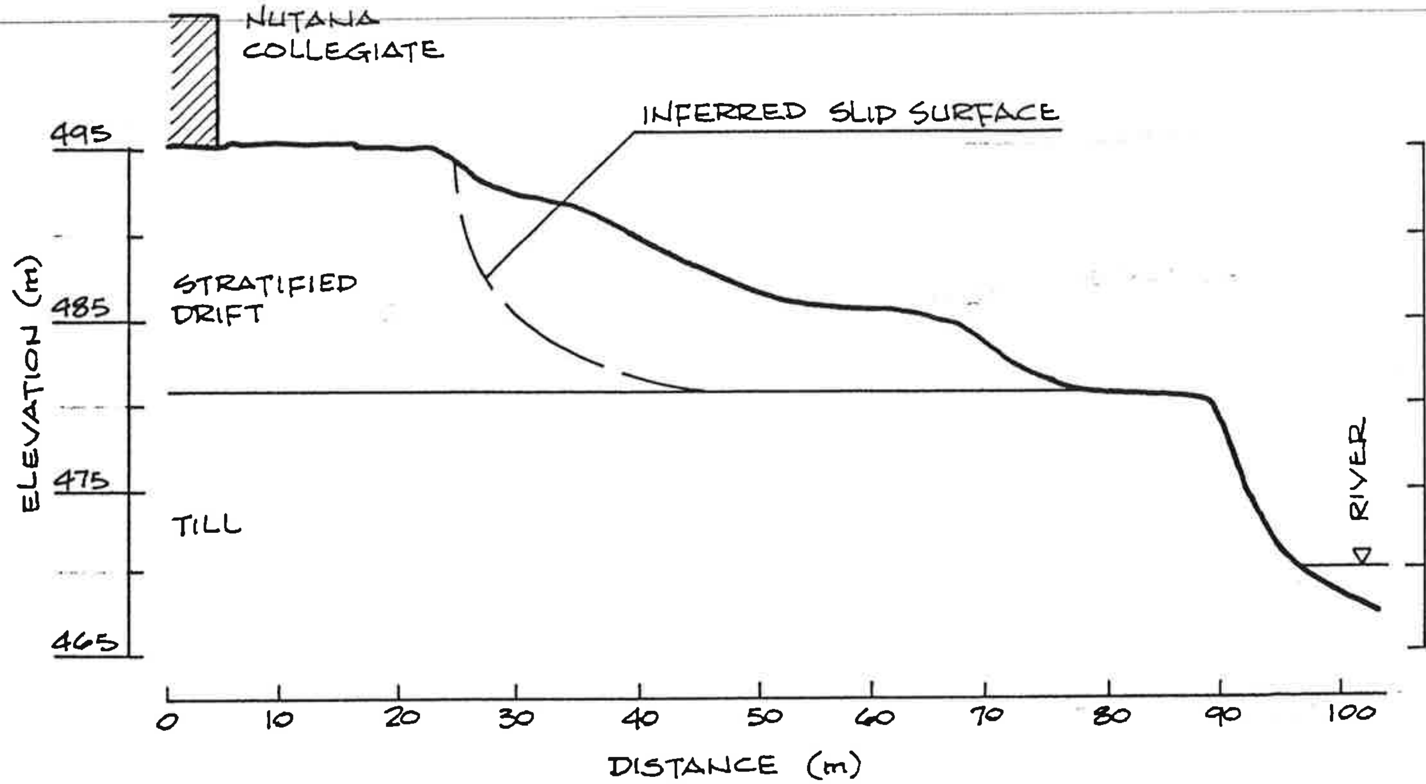
Long Hill Slide

The Long Hill Slide was the first in the series of slides that occurred between the existing Victoria and University Bridges. In 1913, a section of the grade (Long Hill) leading down to the Victoria Bridge suddenly dropped. The existing street car rails, sidewalks and curbs were damaged sufficiently to interrupt service. A reinforced concrete and brick drainage system was installed in 1914 to reduce water levels and increase the stability of the slope.



STRATIGRAPHIC SECTION ON EAST BANK AT QUEEN'S HOUSE OF RETREAT (FROM CAL, 1984d)

FIGURE 35



CROSS SECTION AT NUTANA COLLEGIATE
(COMPILED FROM P.F.R.A. DATA)

FIGURE 36

Broadway Bridge

Slopes at the south abutment (Figure 37) of the Broadway Bridge have experienced movement. A geotechnical investigation in August, 1962, indicated a potential for sliding. A short time later, a slide occurred in the 600 block of Saskatchewan Crescent just north of the Broadway Bridge. Structural distress was subsequently noted at the southeast abutment of the bridge in the latter part of the year.

Two slope movement indicators were installed on each side on the southeast abutment in April of 1963. Approximately 13 mm of movement was observed in the slope movement indicator on the north side of the bridge by the end of May, 1983. A total of 50 mm of movement occurred in this slope movement indicator, between April 1963 and April 1964 and it was found to be sheared off in April of 1965. A third slope movement indicator was subsequently installed to continue monitoring displacements. The major portion of the displacement occurred during the spring seasons, with a total of 75 mm of movement was recorded between April, 1963 and April, 1968. No movement was observed in the instrument installed on the slope south of the bridge.

Saskatchewan Crescent - 13th Street to 18th Street

Several landslides have occurred along Saskatchewan Crescent between 13th and 18th Street. The first slide occurred at 16th Street in May, 1929. This slide is sometimes referred to as the McCraney slide (Figure 38). Subsequently, slides occurred near 15th and 17th Street

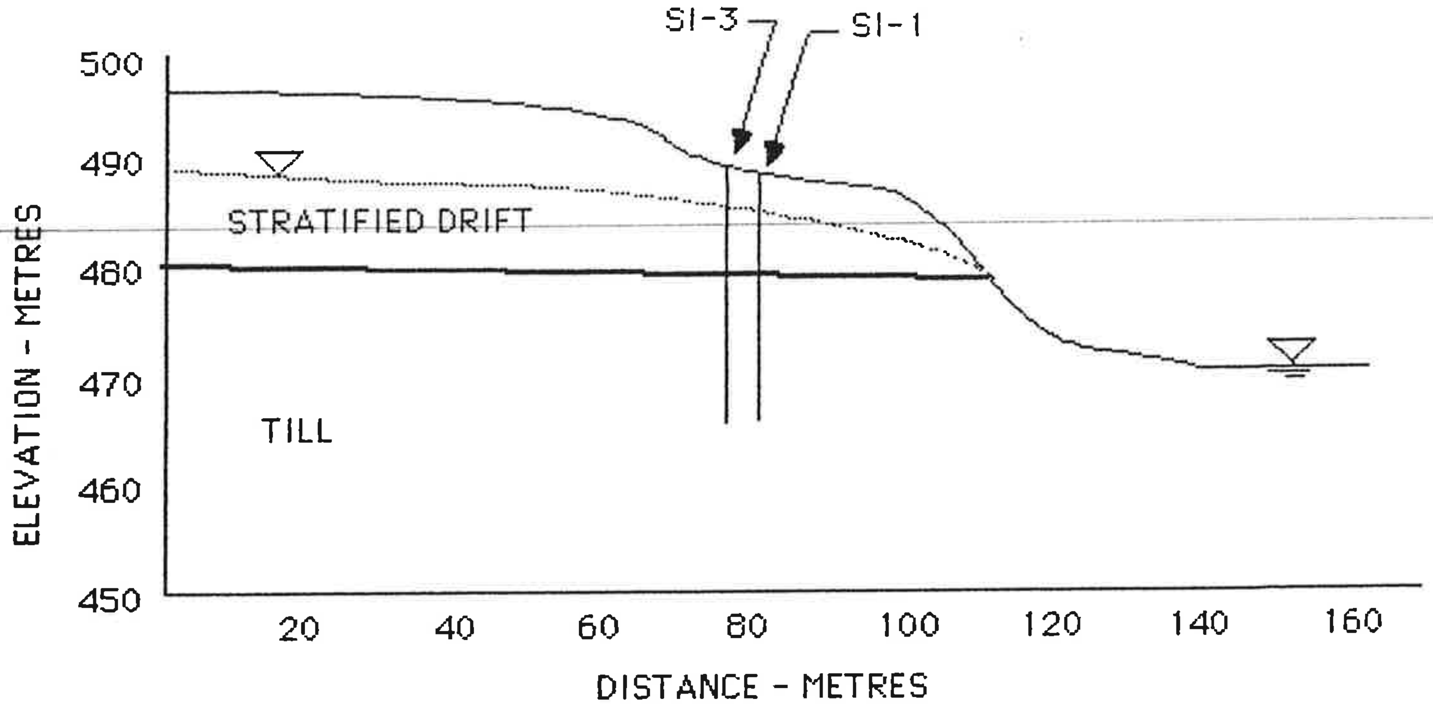
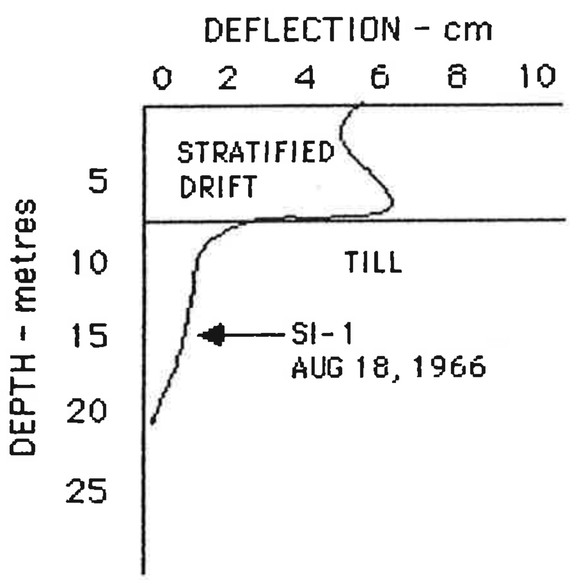
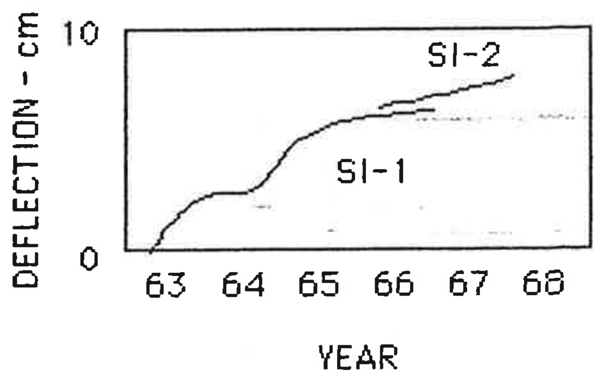


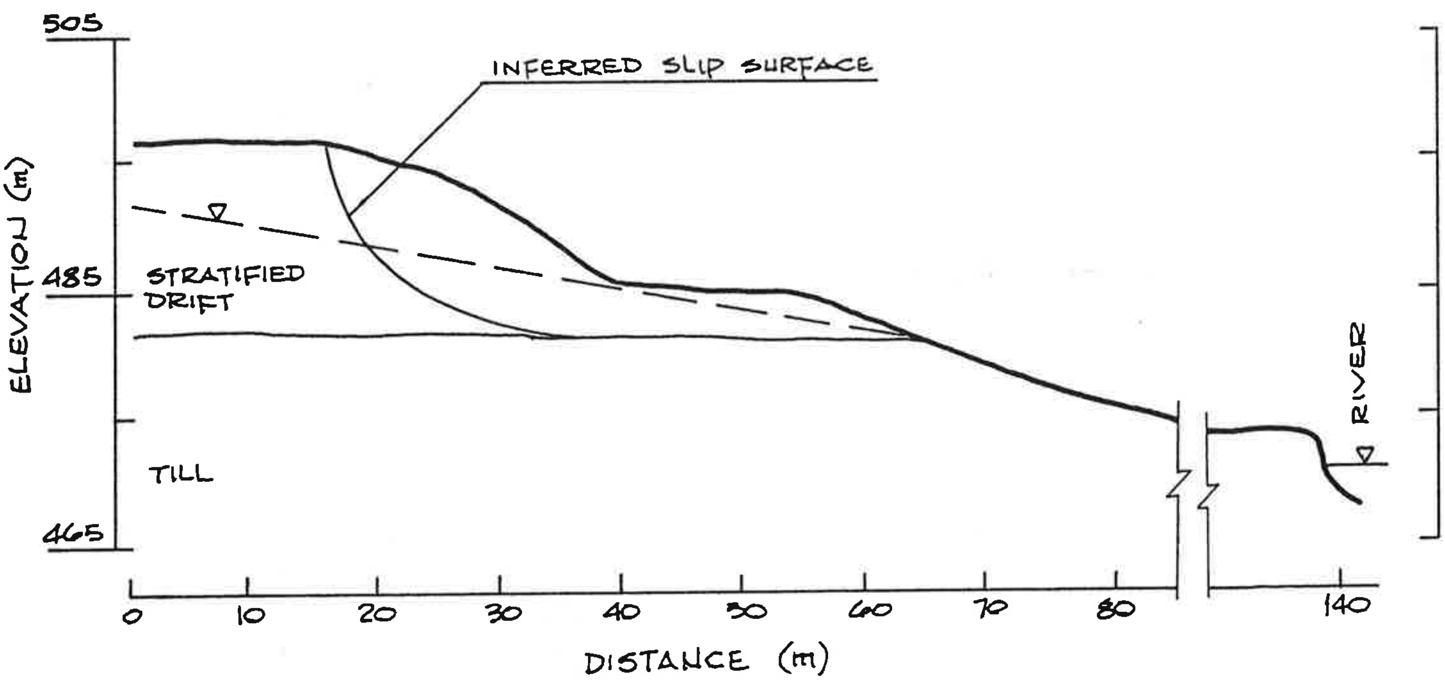
Figure 35a : The 13th Street Cross Section Just North Of The Broadway Bridge





SLOPE INDICATOR MOVEMENTS NORTH SIDE OF EAST ABUTMENT OF THE BROADWAY BRIDGE (COMPILED FROM P.F.R.A. DATA)

FIGURE 37



CROSS SECTION OF EAST RIVER BANK AT THE 18TH STREET LANDSLIDE (COMPILED FROM CITY OF SASKATOON DATA)

FIGURE 38

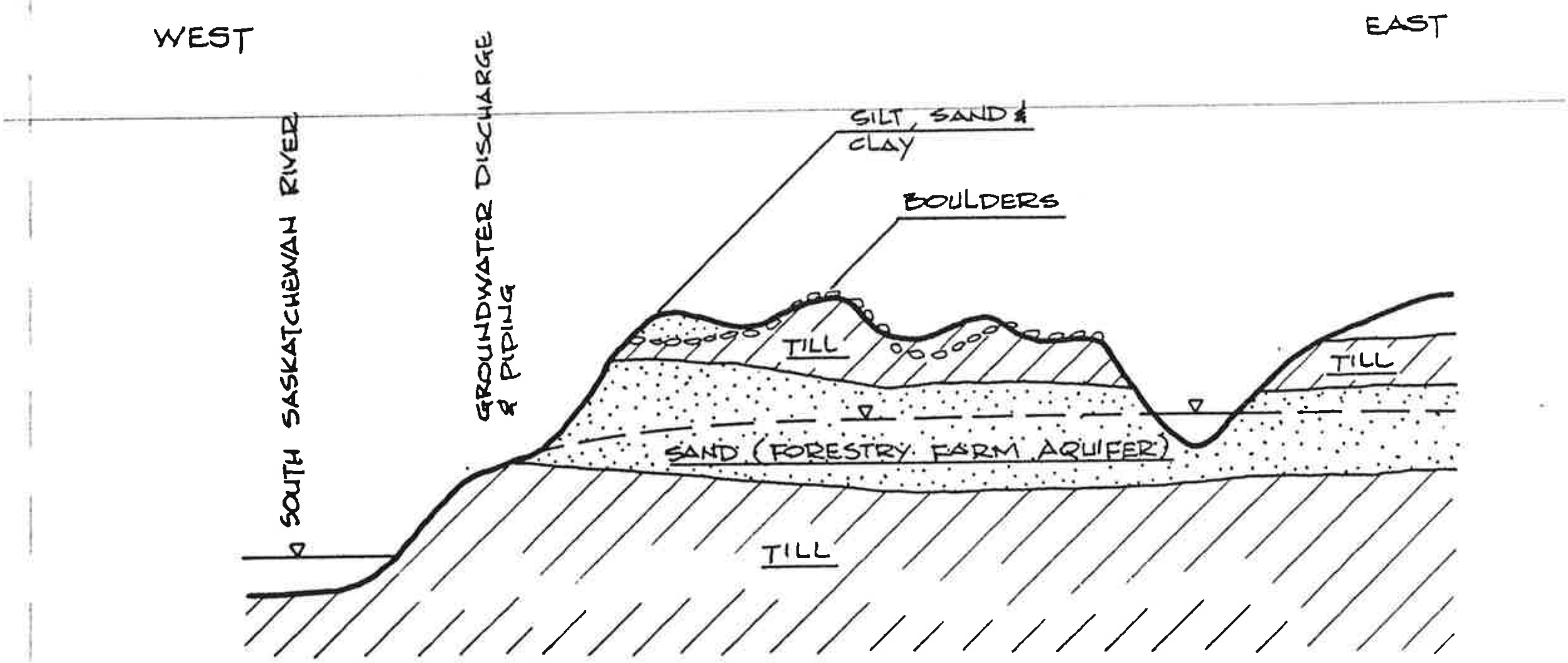
during the spring of 1950. The slide at 15th Street reoccurred in May of 1954. Two more slides occurred at 13th and 18th Street in the early 1960's.

The slides which occurred between 15th and 18th Street caused significant damage to the roadway along Saskatchewan Crescent, particularly at the site of the 18th Street slide. Geotechnical investigations were carried out at each site. The remedial measures adopted were principally the installation of subdrainage systems and berming. Major movement along this portion of Saskatchewan Crescent has not been recorded in recent times, although distortion of the street and curbs indicate that shear movement is continuing at a slow rate.

North of University Bridge

Slope failures have been observed north of the University Bridge. The east abutment of the University Bridge has not shown signs of distress. However, sliding has occurred at the President's Residence to the north side of the bridge. This instability was largely attributed to watering of the grounds.

Slope failures have occurred on the east river bank north of the University, at the Penitentiary and Petursson's Ravine (Figure 39). These failures are piping failures and are not landslides. The discharge of groundwater at these locations is undermining the slope.



SCHEMATIC CROSS SECTION AT PETURSSON'S RAVINE

FIGURE 39

9.0 PRESENT SLOPE CONDITIONS

Inventory of Slopes

Slope conditions along the river valley were examined by a walking inspection along the east bank and portions of the west bank. Detailed observations during this inspection are included in Appendix D and the pertinent points are shown on Drawing S134-D1. The riverbank can be separated into three main categories. These are:

- Sections of essentially virgin, undisturbed riverbank where the headscarp is gradually encroaching onto the upland at a very slow, but perceptible, rate. These sections are from the southern study limits to Labatt's Brewery, and from the 25th Street Bridge to just north of the President's Residence. Along most of these sections, the riverbank is still in a relatively natural state, although minor earthwork has been done at some locations such as at bridge abutments or in the vicinity of the Queen Elizabeth Spur where fill material was pushed over the valley wall for dyke construction. Recent landslides, such as those at Diefenbaker Park and the Queen's House of Retreat, are simply expressions of the ongoing geologic processes which naturally tend to widen the valley. Interference by man may temporarily delay this process, but unless something is done to change the groundwater regime or the geometry of the slopes, it will not be stopped.

- Virgin riverbank which is showing little degradation, except in the vicinity of groundwater outcrops. These are slopes composed primarily of till and are essentially stable, except for areas like Devil's Dip or the piping failures near the Penitentiary where groundwater outcrops are slowly eroding the slopes. However, slumping is virtually absent, except for shallow seated sloughing on locally oversteepened slopes. All of the area from Devil's Dip north falls into this category.

- Valley walls that have been regraded or recontoured to accommodate urban construction. The principle area involved extends from approximately Eastlake Avenue to Labatt's Brewery where a section of the valley wall was graded and intensively developed for housing. Based on present knowledge of the geology and from observations on several structures in the area, it is more probable that these are ancient landslides.

- Valley walls that have been improved by various remedial actions. The principle sector where this applies is from Eastlake Avenue to immediately north of the 25th Street Bridge. In this vicinity, intensive development has taken place near the top of the slopes. Landslides have occurred. Remedial measures, consisting of regrading the valley walls and installation of drainage and erosion protection, have been installed to reduce, but not eliminate, slope movements. Some improvements, such as subsurfrace drainage, may lose their effectiveness with time as drains may plug. Consequently, ongoing maintenance and monitoring of the slopes will be required and future capital investments will likely be needed to maintain their stability.

Existing Improvements Potentially Threatened

Drawing S134-1 identifies areas where existing landslides potentially threaten structures or improvements placed on or near the top of the slopes. These areas are shown in more detail on Drawings S134-2 to S134-12. The principle areas are:

° Diefenbaker Park, where surface drainage is causing erosion leading to landsliding. This sliding, combined with natural instability, is threatening some of the park development and at least two towers supporting power lines.

° St. Henry Avenue, where it encroaches on the crest of the slope. Deformation of the pavement and guardrails indicate that shear movement is occurring.

° Queen's House of Retreat, where continuing instability has removed part of the landscaped area and the irrigation system.

° Cherry Lane, where the effects of movement can been seen on several parcels of private property and on several structures over a length of about 2 blocks. Part of this area has been developed over old landslide scarps.

° Nutana Collegiate, where the grounds have experienced damage due to landslide movement in the past.

° Long Hill and Broadway Bridge.

° Saskatchewan Crescent from Broadway Bridge to University Bridge, where pavements and curbs still exhibit evidence of damage.

- President's Residence, which was threatened by landsliding.

- Devil's Dip and ski jump ravine, where the effects of surface water are creating some instability.

- Ravine Drive, where minor damage was experienced in a roadway cut.

It is emphasized that these areas are designated as high risk areas on the basis of geology and landforms only.

New Improvements Inadvisable

An examination of slope conditions indicates there are areas where it would be inadvisable to locate new developments without undertaking detailed analysis of slope stability. Such analysis may reveal that special measures may be required to strengthen or otherwise improve the stability of the slopes. The highest risks are in areas of known instability where slides are either active now or are known to have occurred in the past and the efficacy of measures taken to stabilize them is not known. The second area includes areas where the slopes are not presently unstable, but are sufficiently steep or include a geologic condition which increases the potential for instability. The areas are shown on Drawings S134-2 to S134-12 and include areas that:

- Are active landslides;

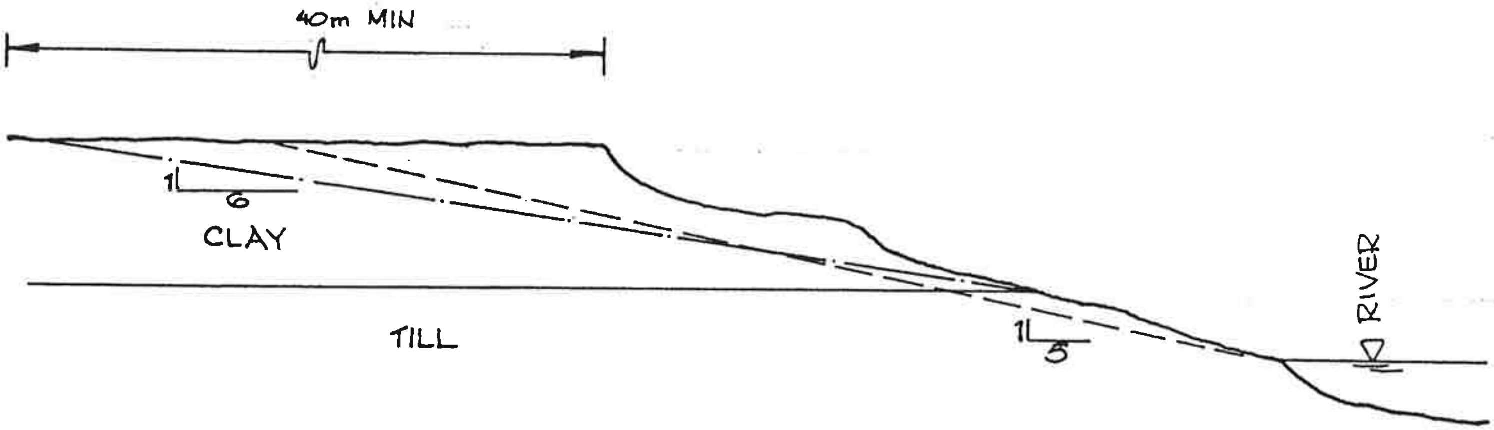
- Were active landslides during the recorded history of Saskatoon;

- Lie on a landform, such as the old headscarps landward from Cherry Lane, where shear strain, however slow, can be expected;

- Are within 40 m of the top of any slope;

- Are not closer to the top of the slope than 6 times the depth of the clay, or a 5:1 slope from rivers edge as shown in Figure 40.

Designation of these risk zones should not be construed as meaning that there is no geotechnical risk involved in developments located landward from the limits of the zone outlined in Drawing S134-2 to S134-12 inclusive. Normal geotechnical investigations by a qualified geotechnical engineer, including checking of the stability of the site, are recommended for all development sites in proximity to the slopes, even beyond the designated risk zone. The level of effort may be proportionally reduced as the distance from the risk zone increases.



LANDWARD LIMIT OF AREA WHERE FUTURE IMPROVEMENTS ARE INADVISABLE IS THE GREATER OF :

- 5:1 SLOPE FROM RIVER EDGE — — — — —
- 6:1 SLOPE FROM OUTCROP OF TILL ON SLOPE —·—— ·——
- 40m FROM EXISTING HEADSCARP

DEFINITION OF ZONE WHEREIN
FUTURE IMPROVEMENTS ARE INADVISABLE

FIGURE 40

10.0 ACTIVITIES WHICH CONTRIBUTE TO SLUMPING

Earlier discussions (Section 7.0) indicate that the factor of safety with respect to stability of the slope is in general terms dependent upon:

- Slope height, in that the higher the slope the greater the stress is in the slope;

- Slope angle, in that the steeper the slope angle the greater are the stresses in the slope; and,

- Shear strength of the soils.

A review of the circumstances and activities that may reduce the stability of a slope indicate that these can be divided into categories, namely:

- Factors which increase the disturbing forces. This would include steepening of slope through filling or cutting or erosion, or application of external loads such as buildings.

- Factors which reduce the vertical effective stress, (σ'). This would include factors such as raising the groundwater level through leaking services or irrigation; by concentrating drainage; reducing evapotranspiration by measures such as covering slopes with impermeable membranes; or, interfering with natural seepage from the slopes such as by construction of fills.

Activities that contribute to slumping are summarized in Table 10.1.

ACTIVITIES THAT CONTRIBUTE TO SLUMPING

TABLE 10.1

ADD WEIGHT TO SLOPE

1. Buildings unless foundations transfer loads to depth that does not load slope or are set-back from crest out of danger zone.
2. Fills on slope.
3. Snow dumps.

EROSION

1. Change in river regime from new bridge piers, fills, etc. that lead to erosional attack on toe of slope.
2. Concentration of surface runoff over slope by regrading.

INCREASE ELEVATION OF WATER TABLE

1. Lawn watering.
2. Leaking watermains.
3. Leaking swimming pools, reservoirs, etc.
4. Surface runoff directed toward slope from paved parking lots or streets, or collected on slope in excavations, sloughs or swales.
5. Blockage of zones of seepage by fills.
6. Reduction of evapotranspirtation through removal of vegetation or covering slope with a membrane or gravel.
7. Increasing infiltration by ponding, snow storage or increasing snow pack.

VIBRATIONS

1. Pile driving.
2. Explosives.

EXCAVATION ON SLOPE FACE

1. Road Cuts.
2. Basement excavations.

REMOVING NATURAL VEGETATION

1. Mature trees tend to stabilize slope.

Factors Which Increase Disturbing Forces

The stresses in a slope rapidly increase when the slope ratio becomes steeper than about 1 vertical in 3 horizontal. Thus, erosion or cutting at the toe of the slope or filling at the crest which steepens the slope can drastically reduce the overall stability of a natural slope. On the slopes within Saskatoon, particularly those formed in lacustrine sediments, such activities should not be undertaken without careful analysis.

The additional load applied by a conventional single storey residential building is approximately equivalent to the load applied by about 400 mm of earth fill. However, whereas the earth fill is a uniformly distributed load, the building transfers stresses to the soil in a much more concentrated fashion. The local effects of a building foundation may therefore be much greater than an equivalent amount of soil.

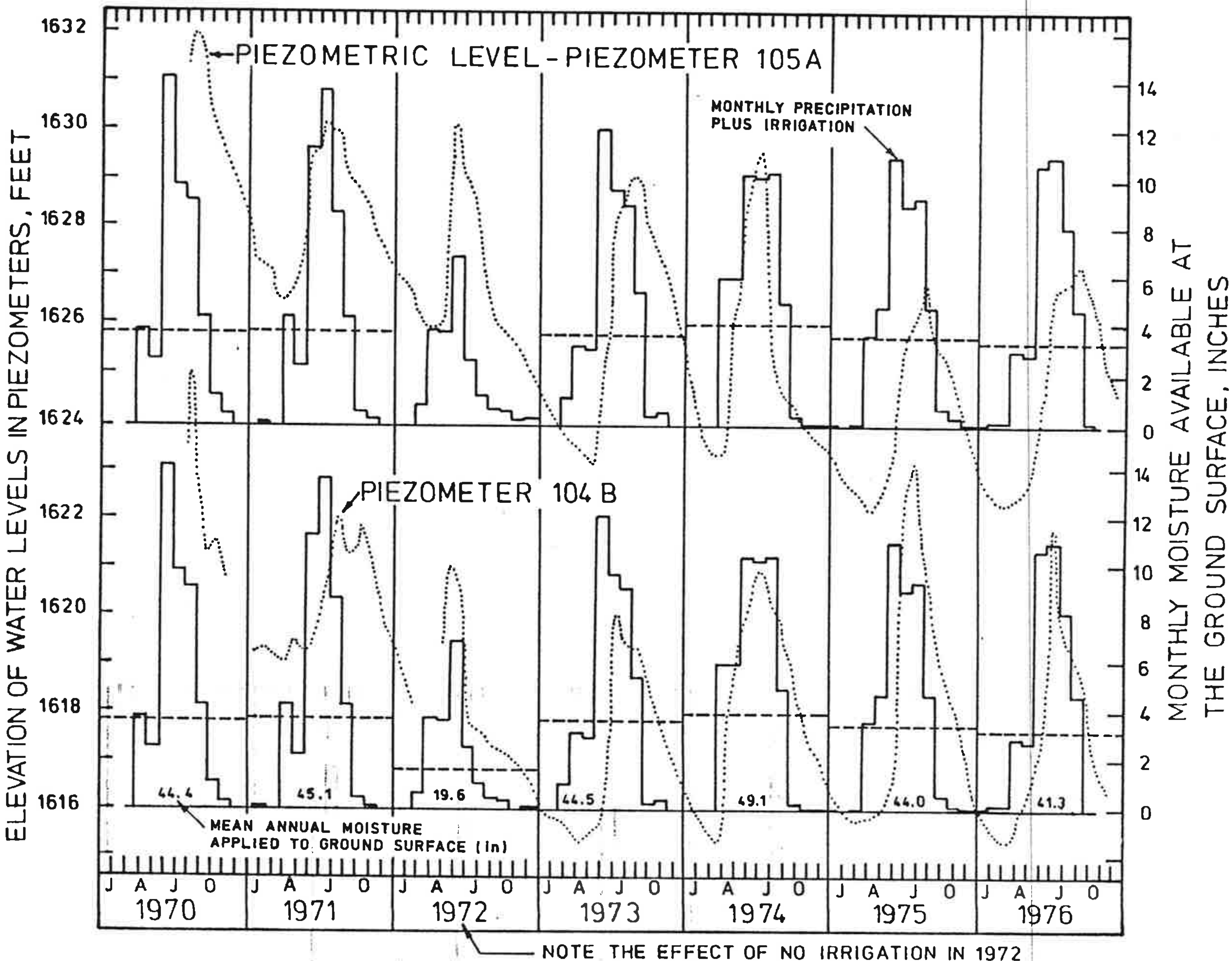
The effect of the building foundations on slope stability will depend on how the stresses are transferred to the slope. A foundation reaction transferred in a shallow footing at the top of the slope could have a potentially large impact. However, in the case of most of the slopes in Saskatoon, the impact of a building on slope stability can be greatly reduced with properly designed deep foundations which transfer the structural loads deep into the Floral Formation till.

Cut slopes require special consideration. As indicated above, steepening of natural slopes can drastically reduce stability, so cut slopes parallel to the river valley must be very carefully engineered. Care is also required in developing cut slopes that run transverse to the natural slopes, but the consequences of failure are usually substantially less in that the length of valley wall involved is usually small. Well accepted geotechnical practices exist for engineering of such slopes.

Factors Which Reduce Available Shear Strength

For the purposes of this discussion, these factors will be limited only to those which increase the pore water pressure within the soil. Further, no attempt is made to differentiate between the effects of water in reducing shear strength and in increasing the driving force due to hydrostatic pressure.

Any activity which raises the piezometric level, and therefore the pore water pressures, within the slope will decrease the stability of the slope. The natural slopes in an undeveloped area reach equilibrium with the natural groundwater regime. Development of the area, or of an adjacent area, can have a dramatic impact on piezometric levels, either from leaking services, from irrigation or from changes in surface drainage. Hamilton et al (1977) monitored changes in the groundwater level on the slopes near President's Residence. Figure 41 illustrates the fluctuations observed during a 7 year period. During this period of observation, a landslide occurred at the President's Residence and lawn watering was restricted until other mitigative measures could be put into place. The piezometric record for 1972 illustrates that not watering the lawn during that period reduced the piezometric rise by



VARIATION IN PIEZOMETRIC ELEVATIONS AND MOISTURE APPLIED TO GROUND SURFACE, WITH TIME

FIGURE 41

more than 2 feet when compared to 1971 or 1973. This reduction was significant in improving the stability of that slope.

Urban development usually means rerouting surface drainage. In natural drainage systems, runoff is concentrated in natural ravines, or uniformly distributed over a large area of the slope. When drainage patterns are changed, infiltration can be significantly increased, raising local water levels and causing local landslides. Such events are presently occurring at Diefenbaker Park and at Devil's Dip.

The melting snow pack is an efficient source of recharge. In the natural setting, the slope adjusts to natural variations in infiltration. However, enhanced infiltration from snow dumps can also lead to an unnaturally elevated water table and local landsliding.

Evaporation and transpiration from plant cover removes a large amount of water from the soil. A healthy vegetation cover will use more water than naturally infiltrates to the slope, thereby providing natural control of piezometric levels. This is especially helpful since this control occurs at the time (summer) when natural recharge is the most efficient. Removal of the vegetation cover reduces this control, and covering of the slope with a membrane such as asphalt or concrete, or even sand or gravel, reduces or removes the evaporation component. Either of these activities will cause a rise in the piezometric surface and a loss of soil strength, potentially leading to slope instability.

Groundwater flows naturally through the soils in a slope, discharging in a manner that the water level will reach equilibrium, self-regulating the amount of flow. Since the amount of flow is dependent both on head (water level) and the length of the flow path, anything that interferes with either the quantity of seepage that can occur, or changes the length of flow path, will change the piezometric levels in the slope. For this reason, fills placed on slopes must be carefully designed so that they do not interfere with the natural drainage. If natural seepage paths are restricted or lengthened, the piezometric levels in the slope will rise, reducing the available shear strength. For this reason, fills placed on or at the toe of the valley slopes must incorporate drainage to preserve the stability of the slope.

11.0 PROCEDURES TO STABILIZE SLOPE

If analysis shows the factor of safety with respect to stability of a slope is unacceptably low, measures must be considered to raise the factor of safety to a minimum acceptable level. Numerous procedures and construction techniques are available for improving stability. These may be categorized into three main classifications which are:

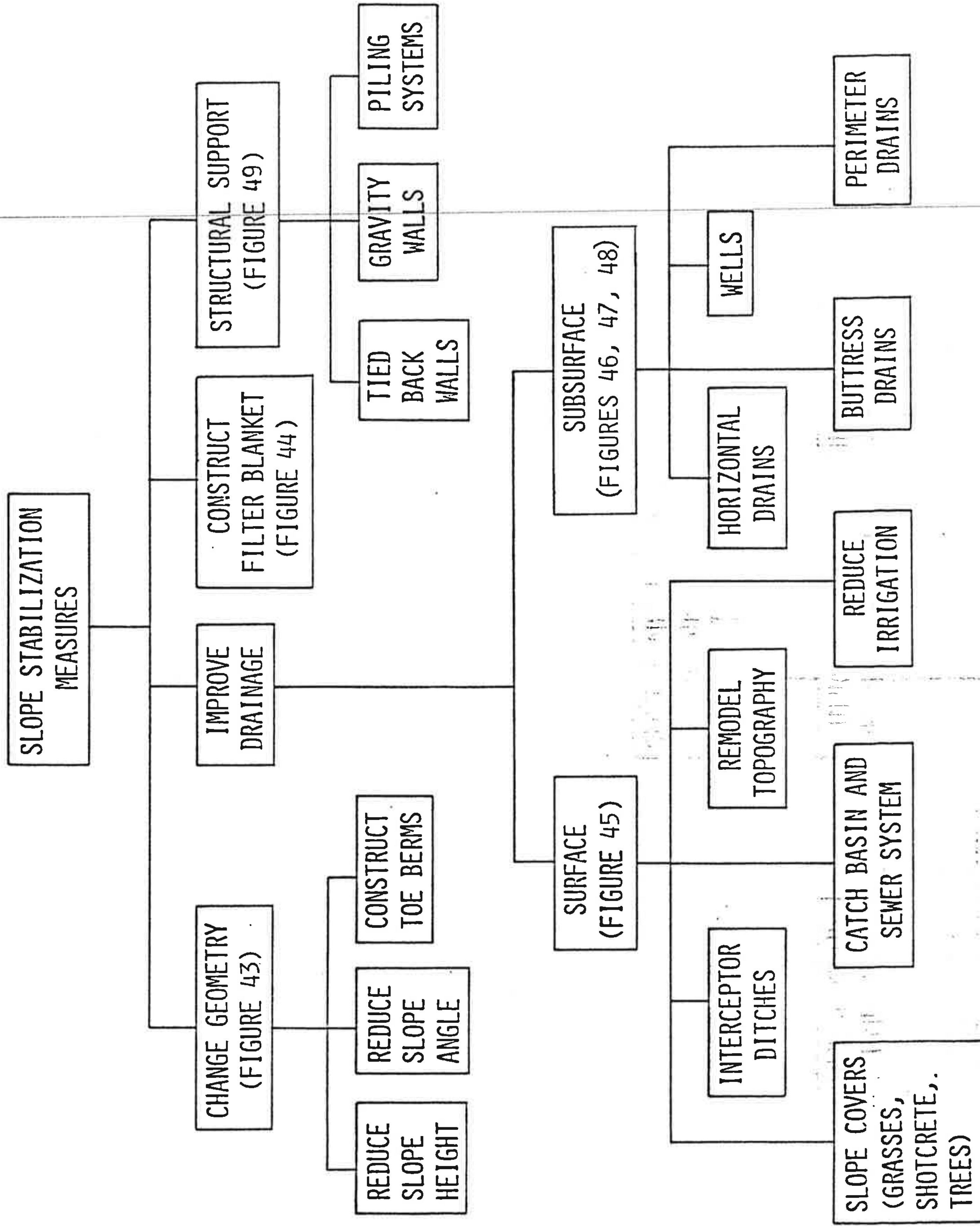
- Slope geometry modifications;
- Control of groundwater level;
- Structural restraint.

Some of the principle stabilizing measures are outlined in chart form in Figure 42. Some examples of measures that have been used to stabilize slopes are illustrated in Figures 43 to 49 inclusive.

Slope Geometry Modifications

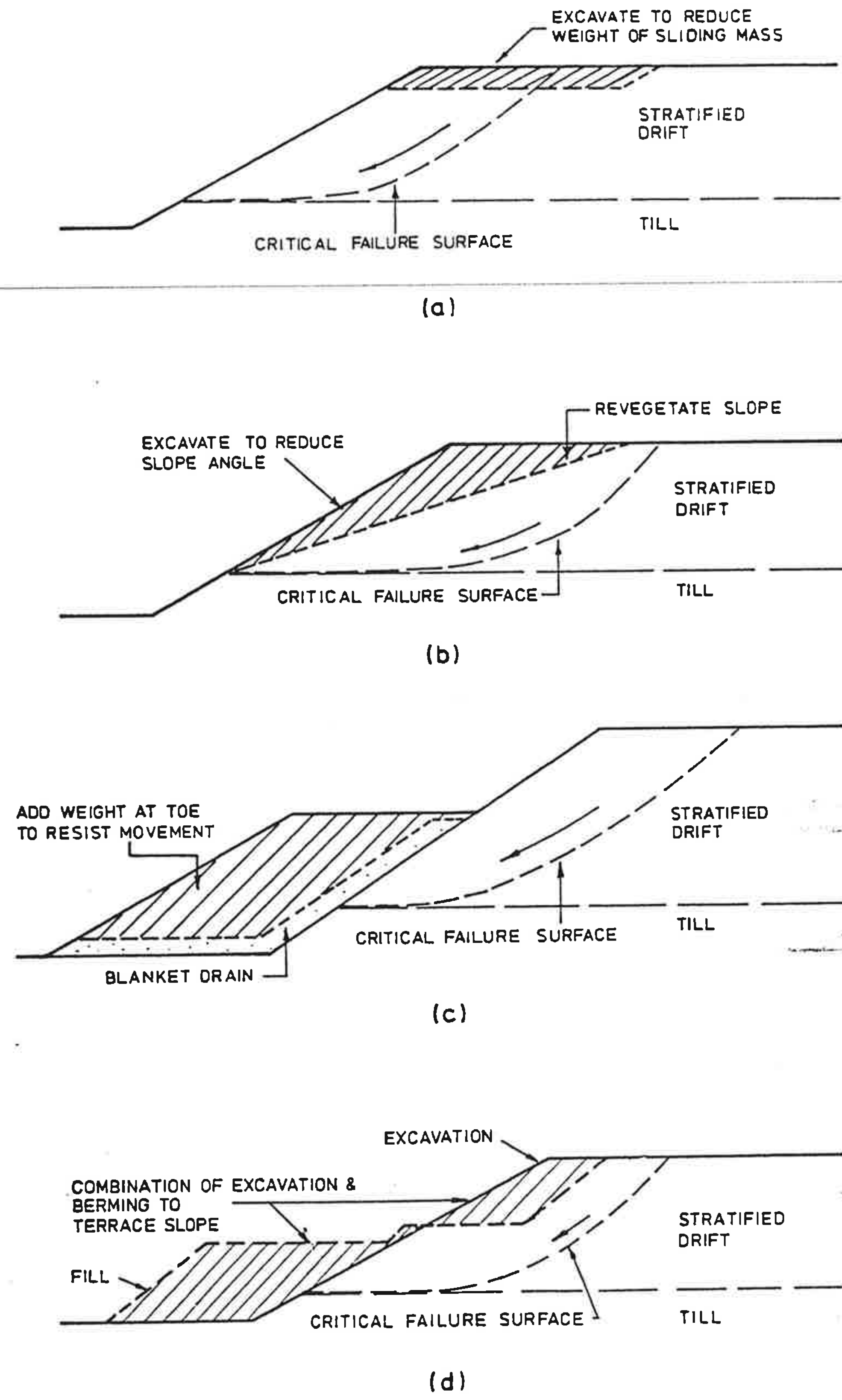
Regrading slopes is normally the most economical and most permanent means of stabilizing landslides. The methods illustrated in Figure 43 include:

- Excavating material from the crest of the slide (Figure 43a);
- Flattening the slope (Figure 43b);
- Placing fill material at the toe to create a berm (Figure 43c); or,
- Using a combination of cut from the crest and fill at the toe to create a terraced effect (Figure 43d).



SLOPE STABILIZATION OPTION

FIGURE 42



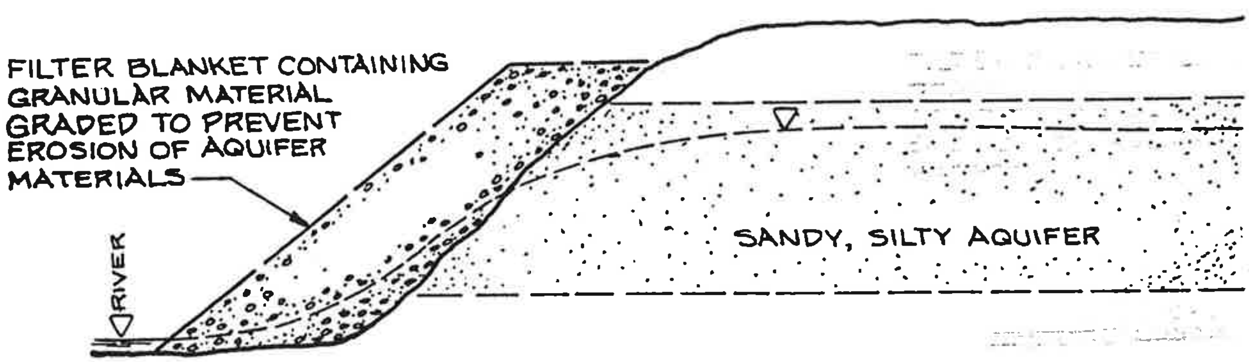
USE OF SLOPE GEOMETRY MODIFICATION TO STABILIZE LANDSLIDES FIGURE 43

Control of Groundwater Levels

Groundwater levels are controlled to remove the threat of piping, or to reduce heads to improve soil strength. Control of piping usually involves construction of a filter blanket as illustrated in Figure 44. This, perhaps combined with installation of weeping tile, trench drains or other subdrainage systems, provides effective control in sand and gravel.

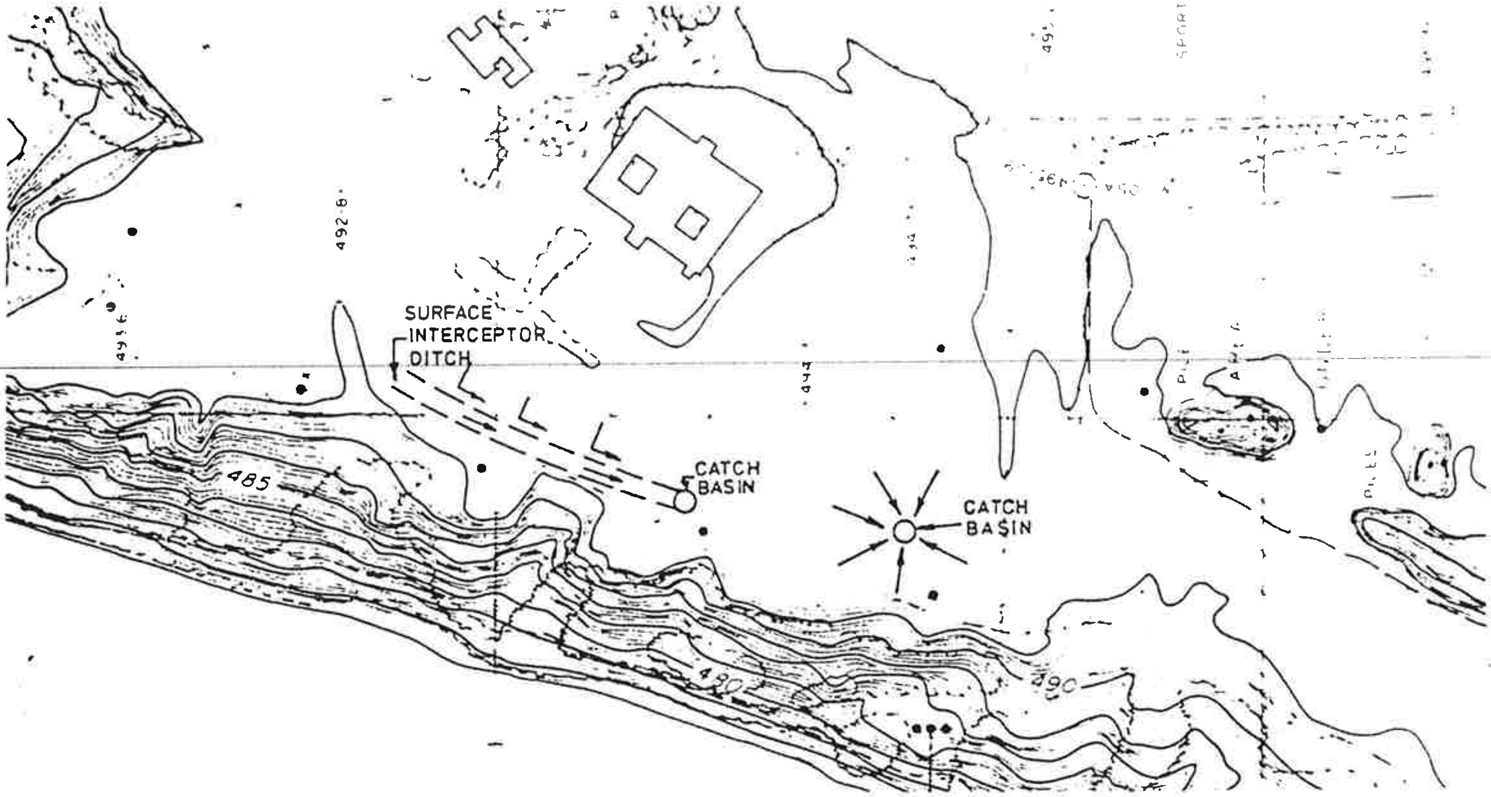
Drainage of fine grained material is more difficult. The most effective method of controlling water levels is to prevent infiltration through proper development of surface drainage. A typical manner in which this can be accomplished in illustrated in Figure 45. The methods illustrated seek to avoid free drainage of water onto the slopes through construction of catch basins and pipes to conduct water to river level, or to a storm sewer, and also avoid concentration the flow on the surface of the slopes thereby preventing erosion. Filling of tension cracks on the headland is also effective in reducing infiltration and improving stability.

A variety of techniques are available to provide subsurface drainage in the slopes. These include buttress drains and horizontal drains (Figure 46), wells and perimeter drains (Figure 47). Also illustrated in Figure 47 is a subdrainage system for a swimming pool or other water retaining structure to prevent development of a groundwater mound under these facilities. Water from these drainage systems would be discharged, preferably by gravity, either to the existing storm sewer system or to a properly constructed conveyance which would conduct the water down the slope with a minimum of infiltration.

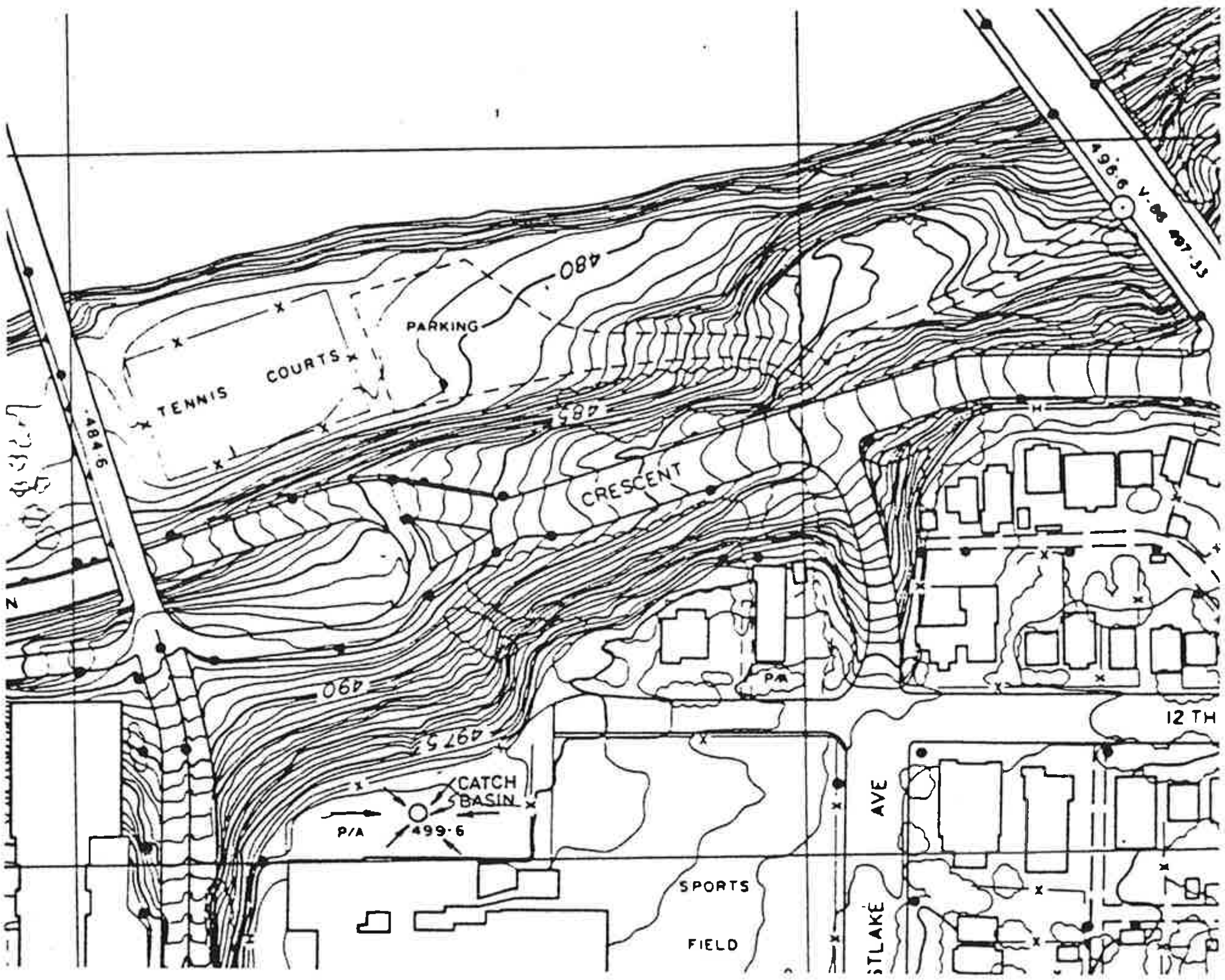


USE OF FILTER BLANKETS TO PREVENT PIPING EROSION OF SLOPES

FIGURE 44



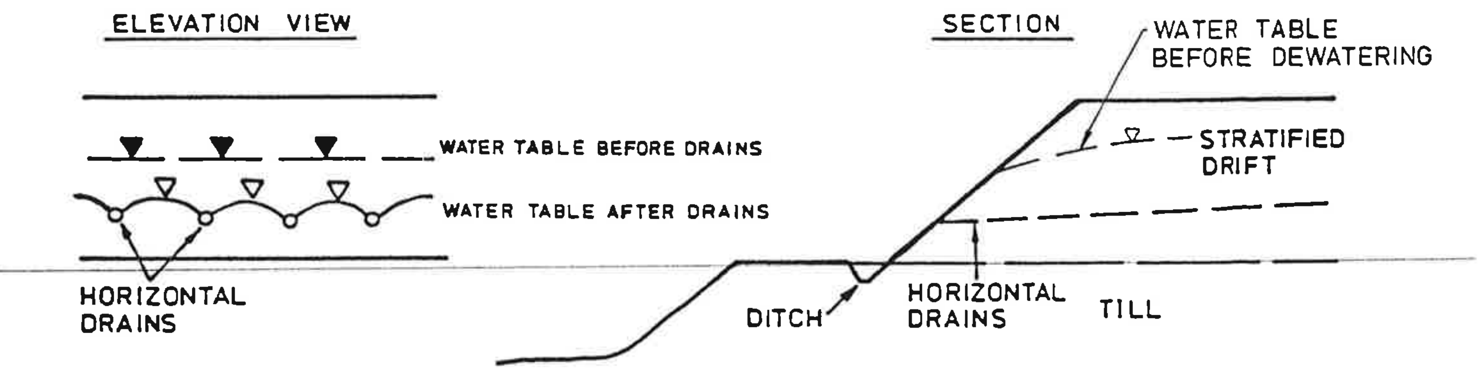
(a) SURFACE INTERCEPTOR (b) SURFACE GRADE TO CATCH BASIN



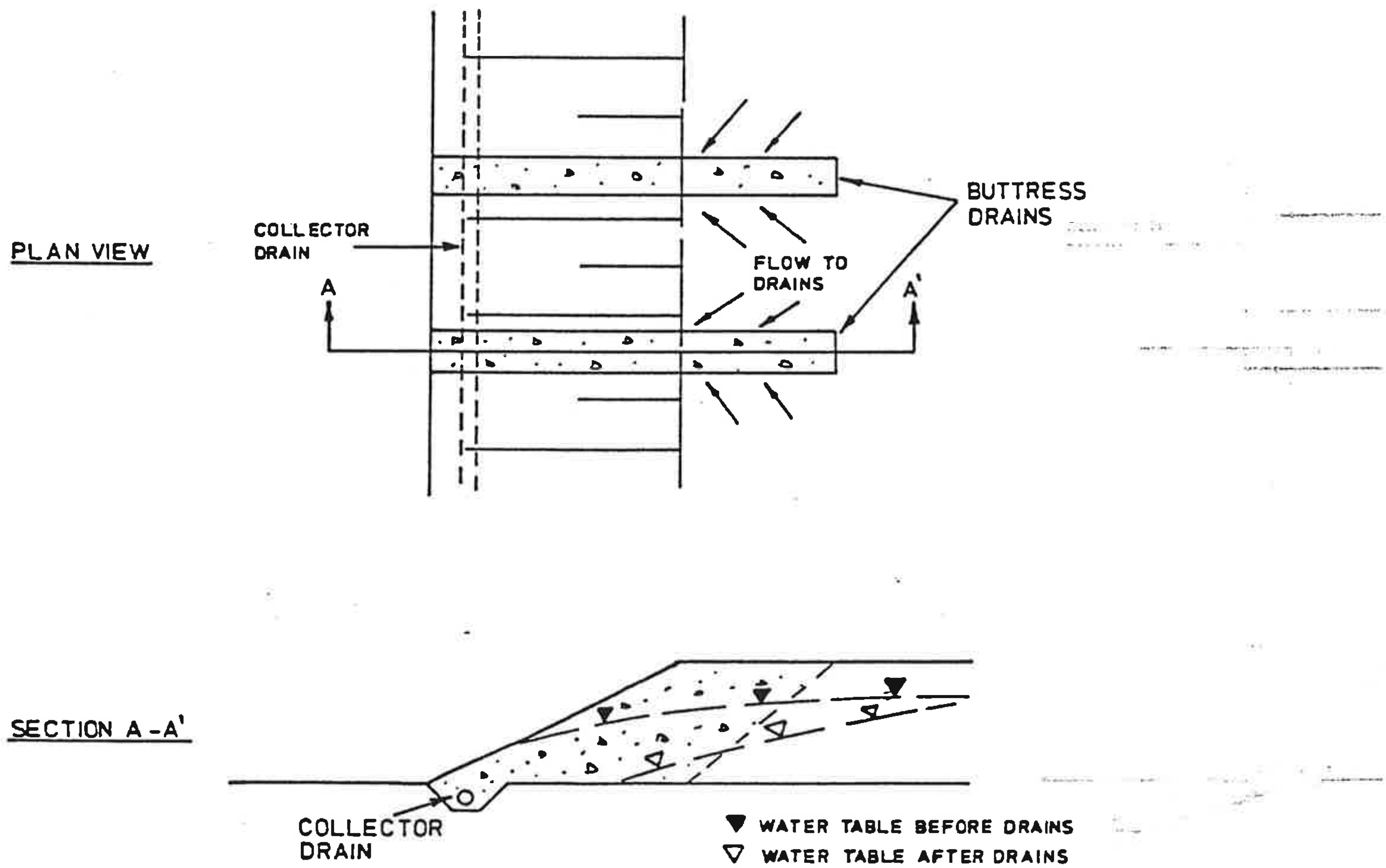
(c) INTERNAL PARKING LOT DRAINAGE

EXAMPLES OF METHODS OF SURFACE WATER MANAGEMENT

FIGURE 45

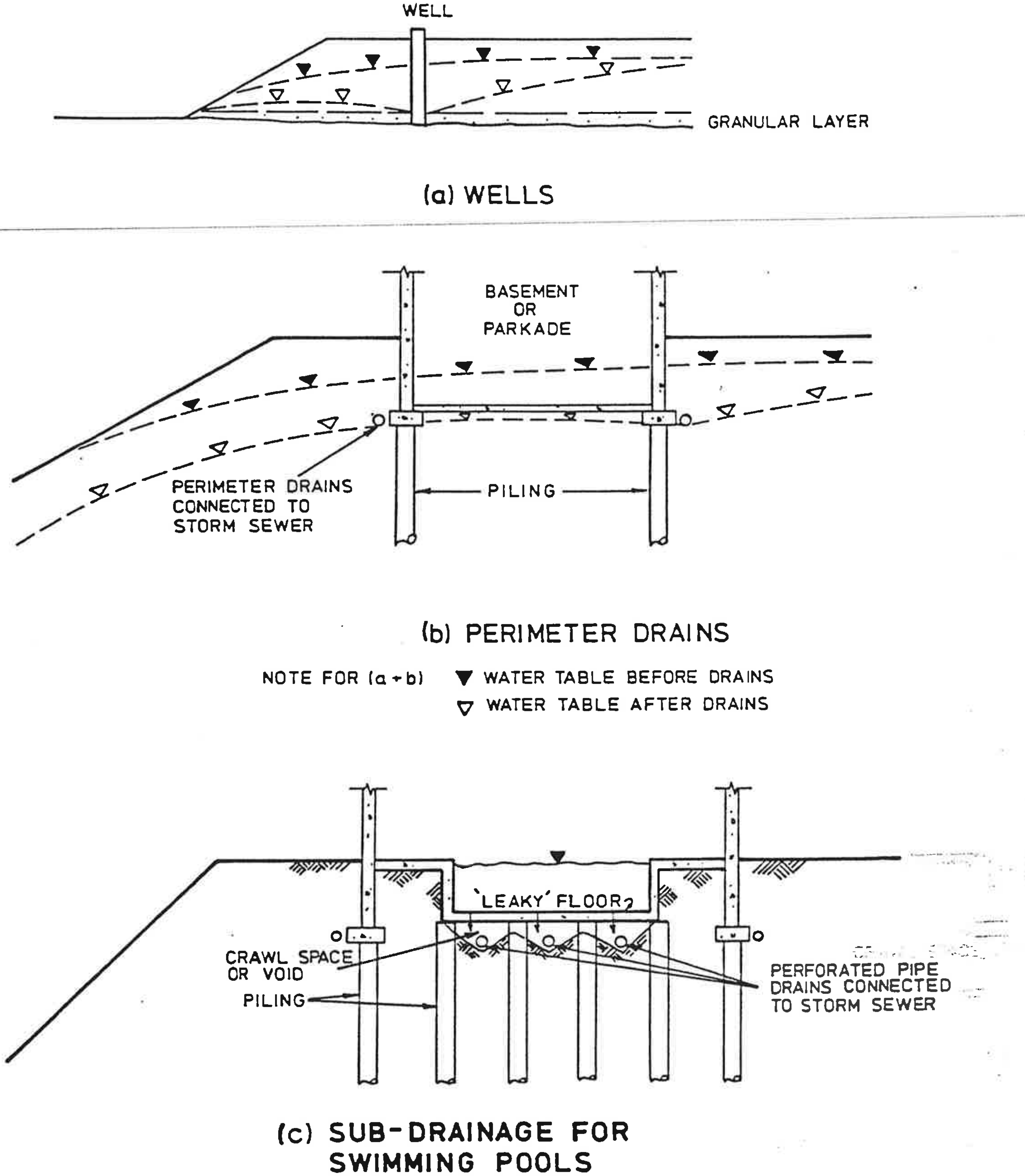


(a) HORIZONTAL DRAINS



(b) BUTTRESS DRAINS

EXAMPLES OF SUBSURFACE DRAINAGE MEASURES FIGURE 46



EXAMPLES OF SUBSURFACE DRAINAGE MEASURES

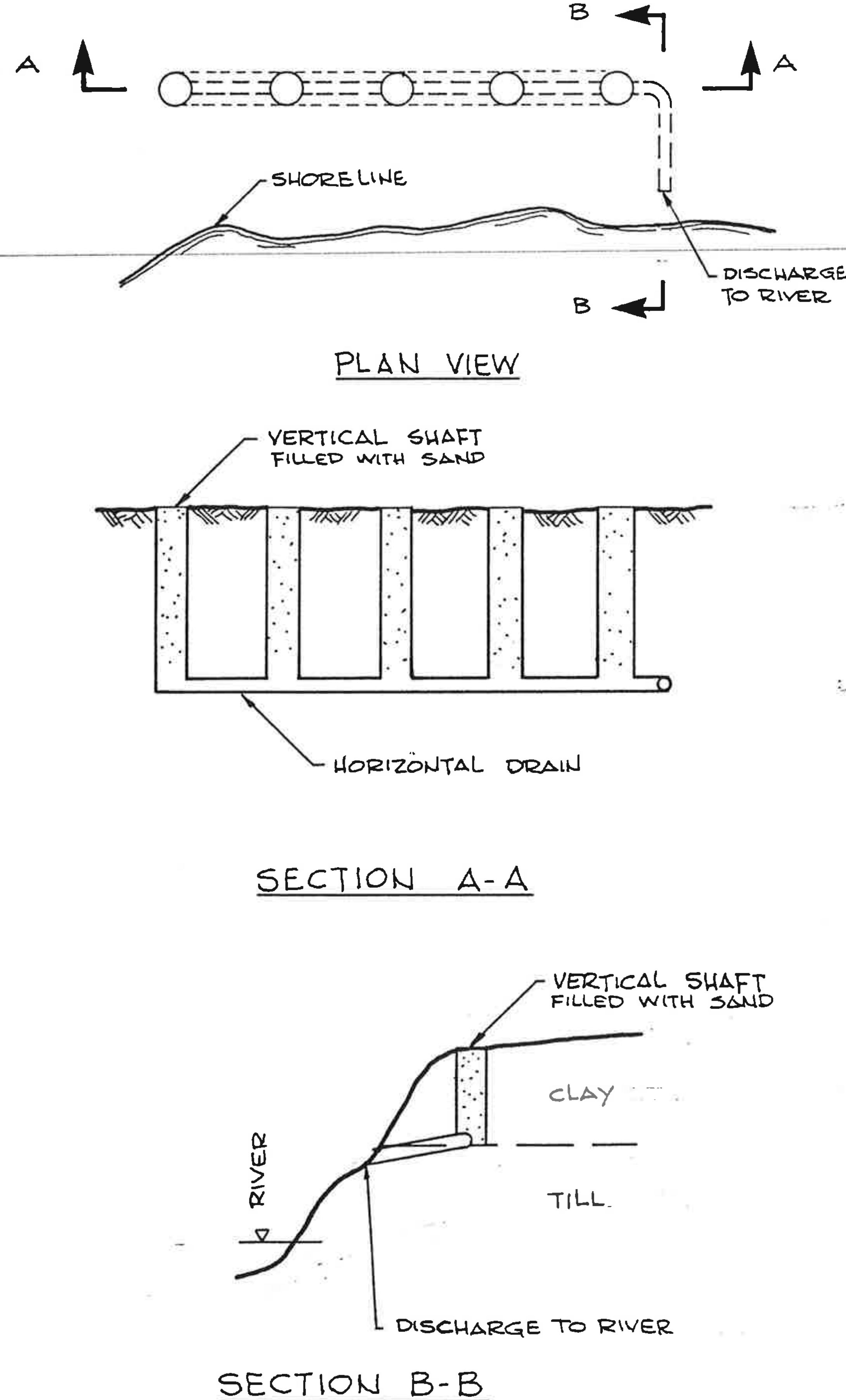
FIGURE 47

Two unique subdrainage systems have been utilized to provide groundwater level control along the east bank. The oldest of these is a reinforced concrete and brick drainage system installed in 1914 to stabilize the Long Hill landslide. This measure was apparently extremely effective since no further landsliding was reported and the slope appears stable today. It is assumed that this drain is still functioning and the surface of it can be seen at the intersection of East Lake Avenue and Saskatchewan Crescent.

An elaborate subdrainage system was installed in the 1960's to stabilize landslides on Saskatchewan Crescent between University Bridge and Broadway Bridge. The extent of subdrainage installation cannot be confirmed, nor was it possible to locate details of the construction. However, it is understood that the construction consisted of large diameter shafts drilled at close spacings into the dense till. The intervening soil between the base of the shafts was excavated and a drainage pipe installed to form a continuous drain that discharges to the river. The shafts were backfilled with sand to form a vertical drainage curtain. An illustration of this concept is given in Figure 48.

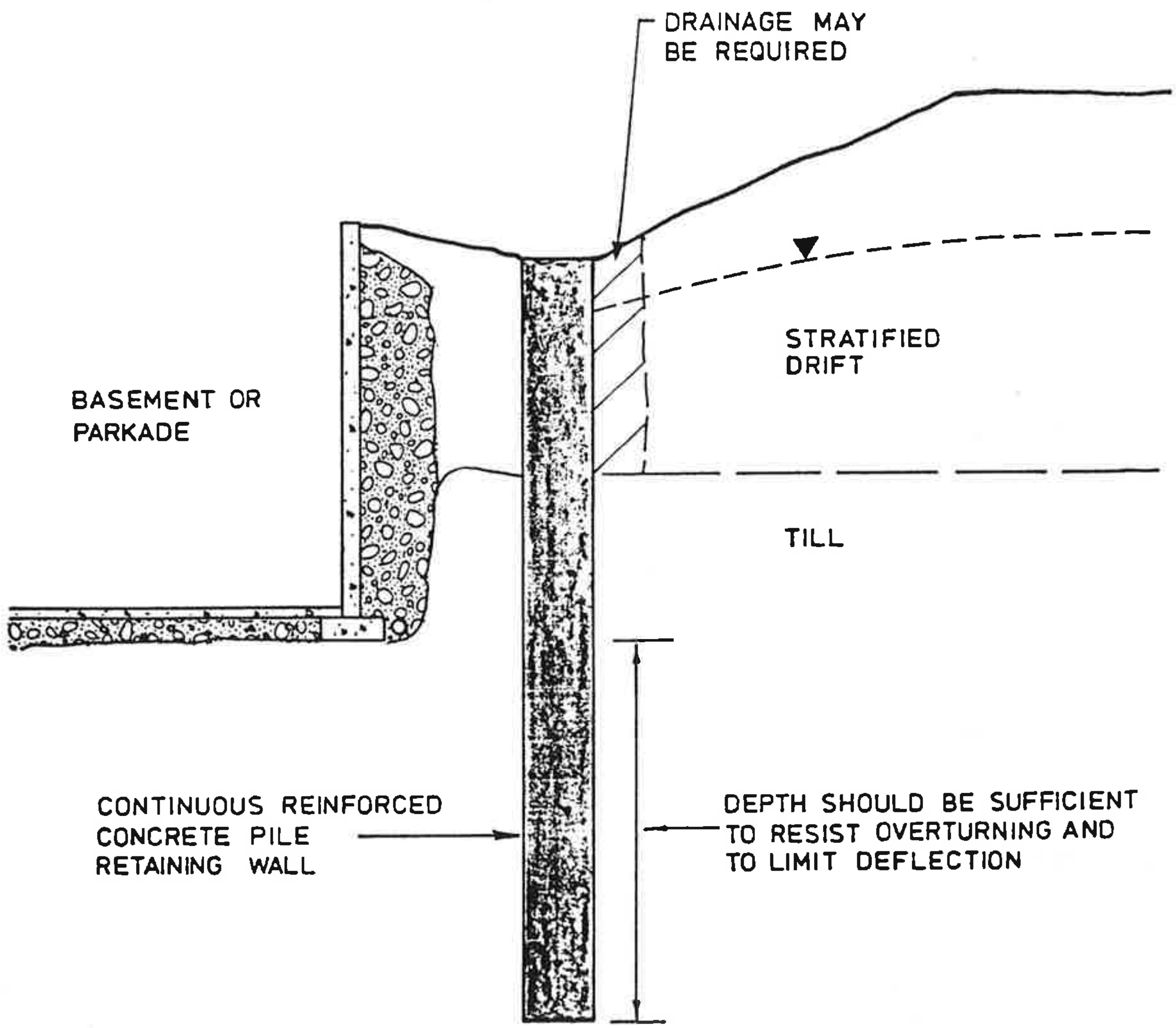
Structural Restraint

Structural methods such as retaining walls, tie backs, piping systems or implementation of deep foundations are a relatively expensive means of stabilizing slopes. One method utilized for a project in Saskatoon is illustrated in Figure 49.



SKETCH OF SLOPE DRAIN ALONG SASKATCHEWAN CRESCENT NORTH OF BROADWAY BRIDGE

FIGURE 48



AN EXAMPLE OF STRUCTURAL SUPPORT OF A SLOPE

FIGURE 49

Vegetation

Development of species that are heavy water users can provide effective assistance in control of groundwater levels. Some easily cultivated grasses and legumes such as alfalfa are well known for their stabilizing effects on slopes. Any species of vegetation will assist in removing water from the soil and slopes should be kept planted. Where slope stability is a consideration, species should be chosen both for their aesthetic quality as well as for their ability to utilize available groundwater.

12.0 GEOTECHNICAL INVESTIGATION REQUIREMENTS

12.1 General

Section 3.1 of the Development Review Policy requires that: "When there is reasonable belief that a proposed improve-ement may affect slope stability on or adjacent to a site . . ., the authority shall require the submission of a geotechnical report". The purpose of this section is to delineate the requirements for a geotechnical investigation at a particular site. The specific objectives are to:

- Specify appropriate procedures for field reconnaissance and exploration.
- Identify the types, and roles for, slope instrumentation that can be installed.
- Outline the requirements of a suitable laboratory testing program.
- Describe suitable numerical methods for slope stability analysis.
- Identify appropriate factors of safety.
- Examine the influence of operating and maintenance standards on slope stability.
- Review requirements for and methods of implementation of monitoring.

° Propose a role for MVA in developing and enforcing the required standards of geotechnical investigations.

° Examine the aspects of liability.

Background Information

A considerable amount of background information is available to the geotechnical analyst studying slope stability in Saskatoon. These data are listed in Section 13.0.

Types of Development

The level of field investigation and analysis required for a specific site must depend on the type of structure or facility, and its proximity to the slope. In general, the level of investigation must be proportional to the risk involved and consequences of failure. Installations which passively interact with the slope, i.e. have very little direct impact, such as pathways or landscaping features which do not change the stress system and which have relatively low capital value and minimal consequences if a failure occurs, cannot be expected to undergo the same level of analysis as a higher cost, high risk installation that will significantly change the stress regime within a slope. (An example of a high impact development would be a water reservoir, a high-rise structure or a significant earth fill on or in proximity with the slopes.) On the lower end of the investment and risk scale, the appropriate level of investigation may be a geotechnical reconnaissance; while at the upper end, intensive investigation would be required. The pertinent question must be whether or not a proposed installation directly or indirectly changes the stresses or affects the physical environment of the slope.

12.2 Field Investigation

The objective of a field investigation for a slope stability analysis is to characterize the physical environment and engineering parameters of the slope being studied. At a minimum, the field investigation should:

- Define the geometry of the ground surface.
- Define the subsurface stratigraphy.
- Define the groundwater regime in all soil strata of interest.
- Provide sufficient samples to determine the engineering properties of materials in each pertinent strata within the slope.

The field investigation can usually be subdivided into two separate phases; site reconnaissance and site investigation.

Site Reconnaissance

A comprehensive site reconnaissance can be extremely valuable to learn about the site and design the site investigation and instrumentation programs. The site reconnaissance should include:

- A review of all available topographic and subsurface data, and interpretation of aerial photographs of all available dates and scales. This will allow definition of regional and site geology, and may allow reconstruction of the past history of slope performance.

- A thorough site inspection of field conditions including:

  - Current site use and conditions.
  - Topography and vegetation.
  - Evidence of seepage.
  - Damage or potential danger to nearby structures.
  - Surface drainage, particularly areas of runoff concentration or erosion.
  - Signs of movement.
  - Outcrops from which natural materials can be identified.
  - Activities, such as erosion, fill construction or surface water infiltration, which may aggravate instability.

The detailed requirements of the site investigation and instrumentation program can be better defined following a site reconnaissance.

Site Investigation

The site investigation program provides all the data and samples necessary for laboratory testing and engineering analysis. The program should include:

- A topographic survey (referenced to geodetic datum) if adequate maps are not available.

- Drilling of sufficient test holes for definition of stratigraphy and retrieval of disturbed and undisturbed soil samples from each stratum relevant to the investigation.

- Installation of a piezometer in each hydrostratigraphic unit, or at other positions as required to determine porewater pressures at the elevation of potential slip surfaces.

- Installation of slope movement indicators or survey reference points to measure slope movements and define the location of shear zones.

- Collection of initial data from all installed instrumentation.

Topographic Survey

All topographic surveys and mapping should be referenced to geodetic datum and UTM coordinates, and conducted at a scale suitable for slope stability analysis. A plan scale of 1:2000 with a contour interval of 0.5 m would be suitable for most slope stability analyses, although greater resolution may be required on small scale problems or where the slope geometry is complex. In these cases, a detailed field survey of the critical cross section may be required. It is recommended that all subsurface information be referenced to geodetic datum and horizontal tie-ins be to the UTM coordinate system to first order accuracy so that these data can be easily referenced to topographic information.

Test Holes

The number of test holes required depends on the site geology and geometry of the slope under investigation. Generally, a minimum of three test holes is required to define a simple stratigraphic sequence. One of the test holes should be on undisturbed upland, one near the toe and one on the slope. Additional test holes may be required in complex stratigraphic situations or in areas of active sliding.

All test holes should extend to the base of exploration which is governed by the geologic setting at the site. In general, there is no evidence of sliding seated deeper than the top of the Floral Formation, except those types of instability related to seepage from sand units within the Floral Formation. Therefore, the base of investigation for slope stability studies along the river valley should normally be about 5 m below the upper surface of the Floral Formation. An exception to the above would be where the Forestry Farm aquifer or similar intratill stratified drift units exist, in which case, at least one test hole or nearby geologic reference should extend to the base of that sand unit.

As indicated above, the till underlying the surficial lacustrine sediments is generally below the base of sliding. Shear zones can be expected to develop in the finer, weaker, highly plastic clay sediments immediately above the till contact. All test holes should be advanced to at least the undisturbed till of the Floral Formation and every effort should be made to obtain undisturbed samples of the contact zone between the lacustrine clay and till. Even where the clay-till contact is above river level, it is desirable to have one test hole or nearby geological reference extend to a depth equivalent to 5 m below river level.

Representative soil samples should be obtained from all stratigraphic units. High quality, undisturbed samples at a maximum spacing of 1.5 m intervals should be obtained from the clay strata in at least the three reference test holes (i.e. on the upland, on the slope, and at the toe). Where failure zones are present, continuous sampling may be required. The remaining test holes should be accurately logged to delineate the stratigraphy and piezometers installed. Geophysical (electrical) logs (i.e, self potential and resistivity logs similar to those illustrated in Figures 2 and 3) are an extremely valuable aid in stratigraphic interpretation.

All samples must be collected in accordance with good engineering practice, properly labelled and preserved to ensure high quality samples suitable for detailed strength testing are received in the laboratory. All samples should be protected against disturbance or desiccation until the laboratory testing program is complete.

12.3 Laboratory Testing

A credible slope stability analysis must be supported by sufficient laboratory data to allow an experienced analyst to form judgements as to the character and strength of the natural materials in the slope. The minimum recommended laboratory testing program is given below.

Index Properties

Standard index properties should be performed on at least two representative samples from each stratigraphic unit encountered in the investigation. Further tests should be conducted on those strata which are deemed to be the weakest and where the critical shear zones are likely to form. Standard tests should include natural moisture content, plasticity index, Unified Soils Classification, and bulk density.

Visual Description

All samples must be carefully examined and described by a knowledgeable practitioner. Description should include primary structure, e.g. degree and orientation of bedding; secondary structure including jointing, fissuring, fracturing, slickensiding, and distortion of beds; lithology are texture; color; geologic origin; and a judgement as to the condition of the sample, particularly whether or not it has been disturbed by sampling and handling.

Shear Strength Properties

The accurate determination of suitable shear strength parameters is one of the most difficult tasks in slope stability analysis. While it was earlier stated (Section 7.0) that methods that measure total stress are of little value to the slope analyst, they do give an indication of soil consistency and allow identification of softened zones. For this reason, the pocket penetrometer or vane shear resistance of each sample should be recorded and the sample carefully observed for presence of slickensided or softened planes. This must be done with great care since these planes may be only a few millimetres thick.

High quality representative samples should be selected for determination of the effective strength parameters utilizing either the direct shear or triaxial test methods.

The triaxial test and its application to the determination of soil strength parameters has been well documented by Bishop (1962). This method is best suited to determination of peak strength or the fully softened strength of the soils.

The direct shear test is suited for determination of either peak, softened or residual strength but is influenced to a greater degree by soil heterogeneity. Good success has been achieved utilizing a multistage test on a single sample (as described in Appendix E) to determine the fully softened and residual strength of the soil. The direct shear method is most commonly used for determination of residual soil strength either on a natural shear plane or on a precut sample. Selection of the applicable shear strength parameters was previously dealt with in Section 7.0.

12.4 Monitoring

Monitoring consists of installing and taking observations from various types of instrumentation installed for various purposes, including:

- Determination of the exact position of a failure plane in a landslide mass.

- Providing the earliest possible information on movement of a landslide mass to preserve property and warn of a potential safety hazard.

- Warning when movements are of a magnitude that structural damage may occur to buildings and services.

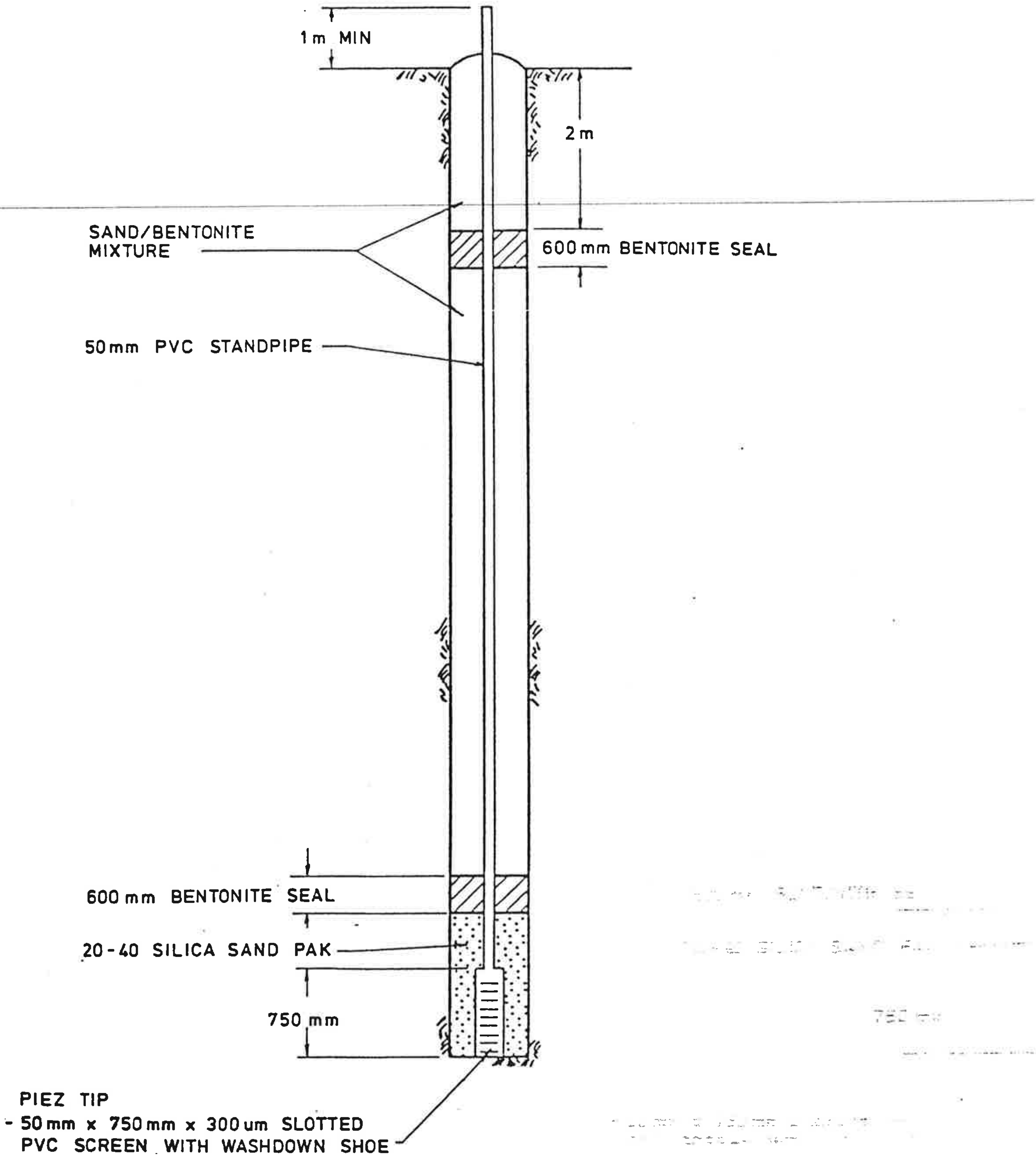
- Determining static groundwater levels and their fluctuations.

- Confirming that slopes or structures are performing in accordance with assumptions or design criteria.

Instrumentation that may be used for long-term observations on the east bank would include piezometers, slope movement indicators and survey monuments.

A detailed specification for piezometer installations in given in Appendix F. A conventional standpipe piezometer installation is illustrated in Figure 50. Each piezometer should be carefully sealed in a particular stratum. A piezometer installed so that it allows communication of seepage between 2 or more stratigraphic units may provide dangerously misleading information.

Piezometers should be situated to define the vertical and horizontal piezometric gradients within the slope. A minimum of three piezometers are required to reasonably estimate the gradient on any piezometric surface. Thus, at least three piezometers should be installed in any stratigraphic unit. Therefore, at a minimum, each site



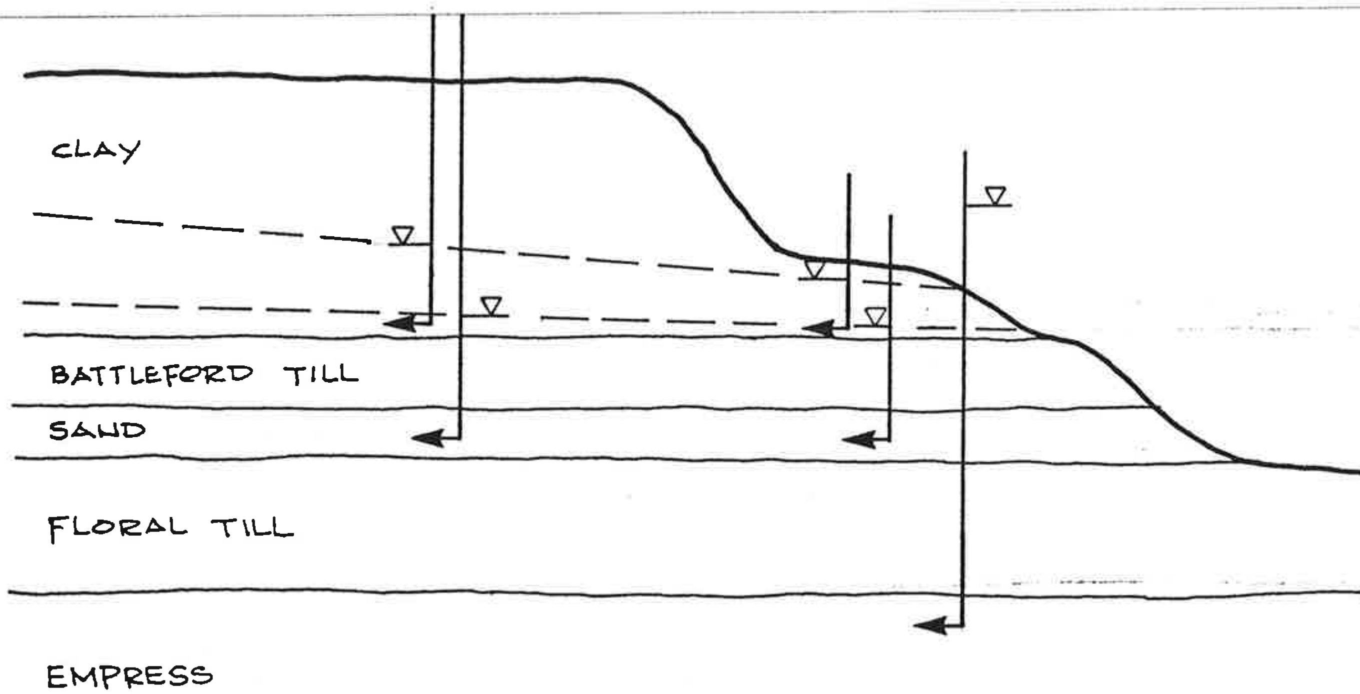
TYPICAL PIEZOMETER CONSTRUCTION DETAIL

FIGURE 50

investigation should require three piezometers installed near the base of the stratified drift. One of these would normally be on the uplands near the headscarp of the slide with a second installed midway down the slope and a third near the toe (Figure 51). The piezometers should be installed as close as possible to the elevation of the anticipated shear zone. All piezometers should be read immediately after installation and at successive intervals until the water level in them has stabilized. Piezometers installed to monitor the long-term fluctuations should be fitted with a permanent recording device. If this is not possible, sounding should be done on a semi-monthly basis during the summer and monthly during the winter.

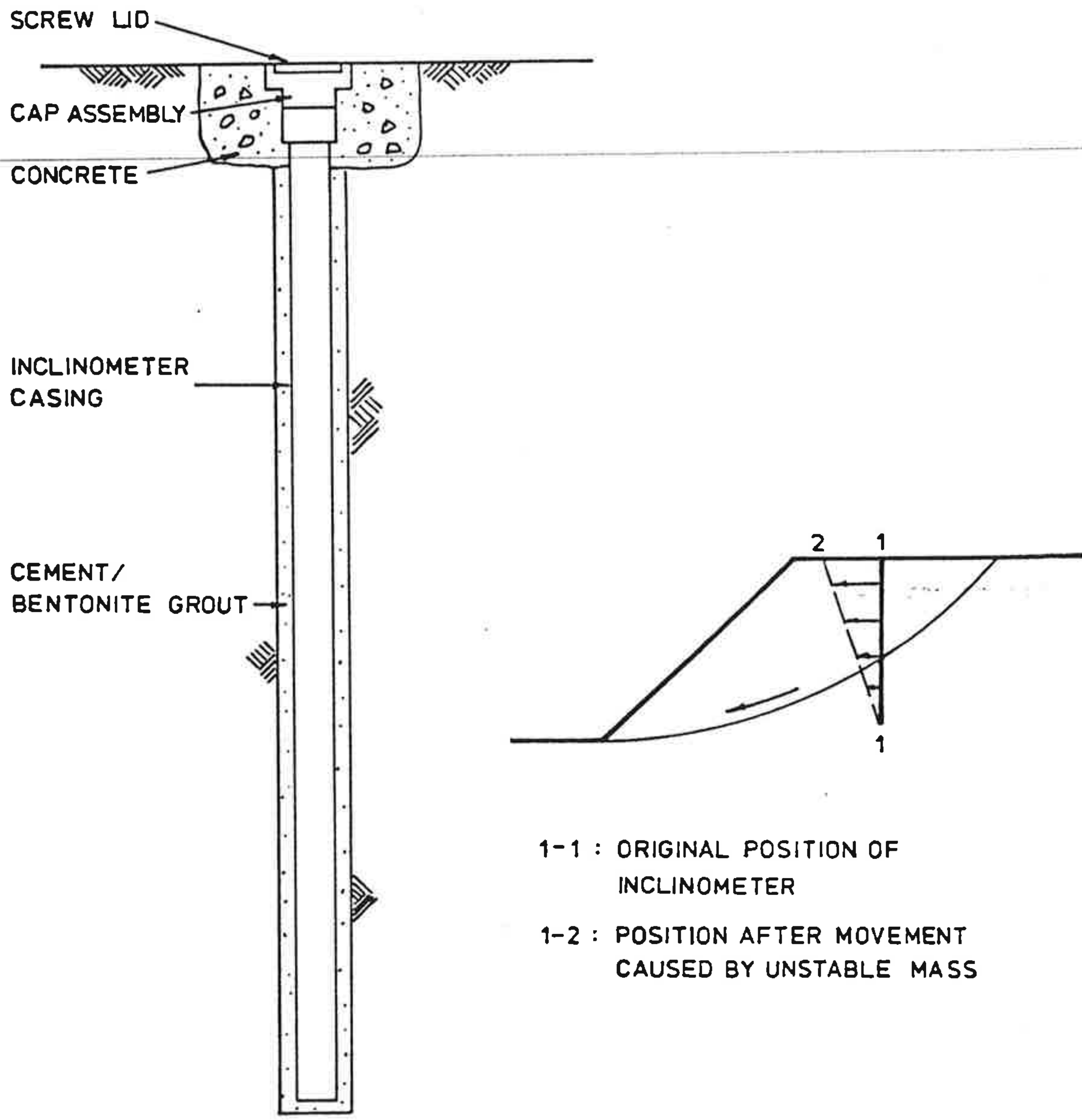
Slope movement indicators record the approximate amount of movement at its locations within the slope. Survey monuments will accurately monitor movement of the ground surface while instruments such as SINCO slope indicators installed in a vertical bore hole will give a profile of movement within the slide mass, including the surface movement.

Slope movement indicators installed in a bore hole are the most commonly used instrumentation. This instrument consists of a small diameter casing with tracking grooves at the four quadrants. The casing is installed in an open bore hole and extends to a depth below the zone of movement (Figure 52). An electronic sensor, which measures the orientation of the grooves, is guided down the tracks. Numerical integration of the inclinometer readings allows calculation of the lateral displacement. In this fashion, the orientation and amount of movement across the shear zone can be accurately recorded.



VARIATION IN PIEZOMETRIC LEVELS IN TYPICAL RIVER BANK SECTION

FIGURE 51



TYPICAL SLOPE MOVEMENT INDICATOR INSTALLATION

FIGURE 52

Careful installation techniques are required for long-term reliable performance of slope movement indicators. Improperly installed casings or improperly maintained reading instruments will give confusing or misleading displacement readings. Detailed recommendations for the construction of slope movement indicators are given in Appendix F.

Monitoring Responsibility

Monitoring is undertaken to protect property and enhance public safety. At first glance, it would seem that there is therefore shared responsibility between property owners and governments or public bodies. A private property owner benefits if instrumentation warns of pending damage and allows early remedial measures. A public body such as MVA or the City of Saskatoon can benefit in a similar manner in that public facilities (streets, roads, parks and other public funded facilities) are similarly protected. Expenditure of public funds can be minimized and rationally budgeted. Mitigation of a natural hazard such as sliding of the riverbanks, has normally been deemed a public responsibility.

In light of this shared responsibility, monitoring should also be shared. It is recommended that the proponent of a project be required to monitor any development constructed within the limits established on Drawings S134-2 to S134-12. The proponent should be responsible for installing monitoring devices to the specifications provided by MVA, and be responsible for monitoring these devices as least quarterly during the first year. All data should be provided to MVA on the condition that it can be made public and included in their data base. Further provision should be made that such instrumentation form a part of the permanent part of the permanent monitoring system established by MVA and that MVA or its designated agents have ongoing access to read the monitors installed on private property.

Monitoring on public property is a different responsibility. As indicated above, this monitoring is undertaken for reasons of safety and protection of public property. Much of the municipal infrastructure in these areas is not the property of MVA. However, MVA wishes to preserve the landscape and therefore the infrastructure will also be protected. For this reason, it would seem that monitoring should be a shared responsibility. MVA is in the best position to coordinate installation of the instrumentation and collection and interpretation of the data. However, since protection will be provided not only for MVA, but also for the University of Saskatchewan and City of Saskatoon property, it would seem a strong case could be made for cost sharing of this function. It should be recognized that initial installation of instrumentation of any type is only the first cost. Instrumentation is useless unless it is properly maintained, regularly read, and the data interpreted. Analyses must be undertaken, action limits set, and a contingent action plan developed for implementation should it be required.

12.5 Enforcement

Building permits for developments within MVA are issued by the City of Saskatoon. It has not been the practice of the

City to require submission of geotechnical reports with application for building permits. However, this is a requirement in some other urban jurisdictions which have special geotechnical concerns. These concerns are accomodated in the National Building Code which makes provisions for the local jurisdiction to set such foundation engineering provisions as deemed applicable. It would therefore seem that the most expedient method of enforcement is for MVA to set a policy for geotechnical investigation on land within their jurisdiction, and seek to have the elements of that policy enacted into the appropriate City of Saskatoon bylaw. Building permits would not be issued until the Authority's requirements had been fulfilled. Provision could also be made for inspection of foundations or geotechnical elements during construction. Occupancy permits would be issued only when the geotechnical requirements for the project had been properly certified.

Enforcement of ongoing instrumentation would be more difficult. Legal opinion should be sought on this issue, but it appears that authority for this provision could be obtained under provisions for public health and safety similar to those presently exercised by Civic Building Inspectors staff.

13.0 EXISTING GEOTECHNICAL INFORMATION

Numerous geotechnical investigations have been conducted along the riverbanks that fall within the jurisdiction of MVA. The first documentation of a slide was published in 1913 for the Long Hill Slide which occurred approximately at the present location of Saskatchewan Crescent and Eastlake Street. Since then, many investigators and agencies have carried out a large number of studies, both on specific landslides and on other sites in the valley. In addition, research has been done on the natural soils in the vicinity. There is therefore a large amount of data available; but it is in widely disseminated repositories. MVA wishes to develop a data management system which will allow collation and cataloguing of all reports and data in one central source. The objectives of this section are to:

- Identify sources of existing information;
- Propose a system of indexing and cataloguing available information;
- Recommend a data management system for public and private usage;
- Identify possible difficulties associated with viability and responsibility.

Sources of Data

A large amount of background information is available on geology, hydrogeology, slope conditions and material properties within the City of Saskatoon. These data include:

- Background reports prepared for MVA.
- Engineering records and project reports from City of Saskatoon files.
- Geologic data from Saskatchewan Research Council.
- Geologic data published in the Physical Environment of Saskatoon (Christiansen 1970).
- Other published geologic maps and cross sections such as those from Federal or Provincial agencies.
- University of Saskatchewan site investigation records and the U of S Guide to Foundation Design (Fredlund 1970).
- University of Saskatchewan research material including departmental reports, research papers, and post-graduate theses.
- Detailed manuals outlining background theory and use of the SLOPE II analytical package.
- Selected texts on geology, hydrogeology, and geotechnology, particularly as they relate to analysis of the stability of slopes.

A partial compendium of available references is tabulated in Appendix G. The available data covers a wide range of disciplines which may be generally categorized as follows:

- Topographic data which describes the geometry of the ground surface.
- Geomorphological data which describes the makeup of the earths surface i.e., landforms, their distribution and materials.
- Geologic data which describes the stratigraphy and structure of geologic strata making up the valley, their origin, lithology, and texture.
- Hydrogeologic data which describes various hydro-stratigraphic units and the groundwater regime i.e.,

groundwater levels, gradient and flow directions, applying to each.

- Geotechnical data which deals with the physical and engineering properties and behavior of soils; engineering analysis; design; and monitoring.
- Hydrologic data which deals with the flow of surface water.
- Historical data which describes the engineering and service history of natural slopes and improvements along the valley.

A system is needed to catalogue all existing data and future information which may be collected. The system must allow easy access to both the skilled and unskilled user, and must easily accommodate future expansion.

13.1 Recommended Cataloguing System

The proposed cataloguing system should be based on three reference items which are:

- Geographic location.
- Date of report.
- Type of data or information.

The coding system which describes the above items and allows an ordered sequence is required.

Location

Several alternative location systems are available. These include the UTM Coordinate System, Legal Land Description, and Civic Address. The UTM Coordinate System is the most precise and logical and is best suited for computer application. In addition, it is the system that future land survey systems will probably adopt in Canada. However, the use of this system without computer assistance would be difficult for most members of the public who relate primarily to Civic Address and perhaps Legal Land Description. Until a general data base is available which readily converts civic and legal property descriptions to UTM coordinates, it is recommended that data be filed and cross referenced according to all three location systems. A detailed location map, preferably computer generated should be compiled referencing the physical location of each data point. This analogue, easily updated, would allow members of the general public to readily assess the areal distribution of the data and select those which may be pertinent to their interests.

Date

The date of a reported document is extremely useful in that it gives an indication of the quality and reliability of the information and techniques that may have been used in its preparation. In addition, it is useful in sorting the information into a logical sequence. It is recommended that data be catalogued only according to the year and month of publication.

Type of Data

The type of data in a particular reference should be described according to standard categories. A suggested method of classification is given in Table 13.1.

File S134

TABLE 13.1

GENERAL DESCRIPTION OF DATA TYPES

SYMBOL	DESCRIPTION OF DATA TYPES
D (Drainage)	Hydrology
G	Geology
H	History
L (Landforms)	Geomorphology
S (Soils)	Geotechnology (General)
f	Foundation Engineering
s	Slope Stability Studies
m	Monitoring
g	Geotechnical Design
o	Others
T	Topography
W (Water)	Hydrogeology

Selected Cataloguing System

It is recommended that the reports be catalogued chronologically according to discipline and filed according to a single code as follows:

Report Number - Date of Report - Information Type

Thus, a document having the catalogue code 112-62.4-Ss would indicate Report No. 112 which is a geotechnical report on slope stability (Ss) published in April, 1962. A separate cross referencing system would be required between report number and geographic location.

13.2 Borehole Indexing

A large number of test holes have been advanced along and near the riverbanks within Saskatoon. These test holes number in the hundreds and have been drilled by many agencies and consulting firms over the last 70 years. This type of information is of extreme value because it is a technical record based on physical observation rather than technical recommendations which rely on knowledge of the publisher. The records of the test holes are scattered among all the geotechnical reports which have been written. This information in its current form is useful; however, its very difficult to use. The user cannot quickly find a group of test holes drilled near or at a particular site of interest. He must search through a series of reports published at different times by different authors.

It is recommended that copies of all test holes be extracted from all the various reports, and compiled into one record.

The following index system is recommended.

1. The position of each test hole, both horizontal and vertical should be referenced to the UTM grid and geodetic datum respectively. A master site plan should be prepared showing the test hole locations. An appropriate numbering system should be applied once all logs are compiled. A UTM sector designation and number would likely be simplest.

2. The test hole log title should include a reference to:
   - UTM coordinate
   - Date drilled
   - The civic address
   - Legal land description
   - Source report
   - Method of drilling
   - Test hole diameter
   - Test hole elevation

3. The detailed test hole log information should include the following:
   - Visual description of stratigraphic units
   - Moisture Contents
   - Atterberg limits
   - Unified Soils Classification
   - Shear strength measurements
   - Dry density of soil
   - General observations such as water levels and sloughing conditions

4. Provisions should be made to digitize geophysical logs and store the analog along with the descriptive text.

13.3 Data Management System

One of the objectives of this project is to eventually establish a data base which is stored and accessed on a micro-computing system. This undertaking involves obtaining the necessary computer hardware and software. However, the primary mandate is to first compile and catalogue all available reports and documents; and to establish a properly indexed bank of test hole logs.

Hardware

A wide variety of suitable micro-computers are available on the market. Some of the major manufacturers include IBM, Apple, Compaq, Digital, AT and T and Radio Shack. The final selection of an appropriate computer can only be made by the Authority based on its overall computing needs, preferences and cost. However, the major choice is between IBM and IBM compatible machines and the Apple systems. While the latter offers great ease of use even for the unskilled user, the IBM systems must be strongly considered because of possible future compatability problems.

Some of the criteria which must be satisfied for a geotechnical data base are listed below:

1. The operating system must be compatible with software programs such as Lotus, dBase, Supercalc, VisiFile, Omnis or Overview.
2. The microcomputer must have two disc drives and at least 20 megabytes of hard disc.
3. The microcomputer should have at least 250 K of random access memory (i.e., 250 K RAM).

4. The hardware system must have a printer with graphics capability.
5. The machine should be compatible with a suitable selection of software programs.

Software

Several data management programs are available. These include:

1. Symphony (IBM) or JAZZ (Apple) (Lotus Development Corporation)
2. dBase III (Ashton Tate)
3. SuperCalc (Computer Associates Ltd.)
4. VisiFile (Creative Computer Applications)
5. Multiplan (Microsoft Corporation)
6. Overview (Proview Development Corporation)
7. Filevision 512 (Telos Software Products)
8. Geoed (International Geosystems Corporation)

Any of the above programs would be suitable for catalogue and index listing. These programs can sort and store abbreviated test hole log or report information.

Test hole information could be summarized as single lines of information which include: number, location, date, depth, soil units, water levels and lab testing data. Catalogued geotechnical reports could also be summarized in a similar fashion. The database programs would store this information as single packages of data called records. The user would be able to list any desired records or use the computer to search, select and list any records with specific information or near a chosen site.

There is a considerable difference in the capability, operating features and cost of the database programs (1 to 6 above). This can be illustrated by comparing OMNIS 3 with the Overview software.

OMNIS 3 is a disk based program which requires a hard drive for efficient operation. It is a fully relational data-base with capability to handle text but cannot produce graphics. It is very powerful in its data handling and data manipulation capabilities. It can be operated either with a mouse or be menu driven. User definable entry screens and menus can be established. It is programmable, allowing operations to be executed utilizing various files within the database. It offers nine levels of security for data protection and is capable of accomodating multi users over a network. It is limited in data size to 65 megabytes. It could be effectively used utilizing a base map which specifies general locations through UTM coordinates and do a prompted search interactively. The software cost is approximately $600.00 and the system development cost and programming would probably be about $1,500.00. It requires a 512 k micro with a printer and a 10/20 meg hard drive. The total hardware cost is probably about $11,000.00 for a total installation cost of approximately $13,000.00.

The Overview software system is RAM based and therefore extremely fast to use. The data limitation would be established by the size of the available Internal Random Access Memory. The system does not set file protection but some protection can be established by disabling the save commmand for data (disk) protection. The database is relational but not fully and the software offers some programmable function through a macro capability. It outputs in text only with no graphics and is best suited for recall, display/print type operations. The system may be somewhat overwhelming to casual users since all options are always displayed in the menus. The software cost is approximately $400.00 but would require approximately $1,000.00 for system development. The hardware cost would be approximately $10,000.00 for a total system cost of approximately $12,000.00.

The GEOED data base offers a superior system for data manipulation. It is a fully relational disk based, menu driven database with full text and minor graphic capabilities. It has a keyboard entry system with operations similar to OMNIS 3. Data can be stored in text form and logs generated either on the screen or in a printout. It will accept alpha-numeric data and will store digitized electric logs. While this is a better system for an experienced user since it allows complex data manipulations, it may be somewhat overwhelming for the casual user. However, it offers a great deal of flexibility because of its definable menu and entry screens and the fully relational nature of the stored tables. The software cost is estimated to be $4,000.00 with an additional $4,000.00 of programming and developement required. It would require a hardware system worth approximately $6,000.00 for a total installed cost of approximately $14,000.00.

Filevision 512 is a disk based fully relational graphical or object oriented database which requires a hard drive for efficient operation. It is menu driven by a mouse and accepts conventional text entry or graphics entry by a graphics digitizer. It is a display type database well suited for displaying location maps showing where test holes are located relative to other geographic or legal subdivision features. This software is not as powerful as the others in its calculating ability or search and sort capability. It is however, due to its graphical nature, extremely easy to use even for an inexperienced user. The software cost is estimated to be $400.00 with an additional $1,000.00 of software development required. It is estimated that approximately $8,400.00 of hardware would be required giving a total installed system cost of approximately $10,000.00.

The software listed above presents three general levels of capability. The Filevision 512 software offers ease of use for the inexperienced user but only elementary data management functions. Most of the other database programs (items 1 to 6 above) offer varying levels of data management and manipulation and graphics. However, the test hole logs could not be stored in their entirety on any of these systems. Only the GEOED system marketed by International Geosytems Corp. can store entire test hole logs including geophysical logs, plot data and print test hole logs or cross sections. It can also perform a variety of sort and statistical functions on the stored data.

A firm recommendation cannot be given on a data management system without further information on the intended purpose, scope of operations and potential users. Further study is required.

Resources

Implementation of a data management system will require substantial human and financial resources. Appropriations will be needed both for the hardware and the software, and for the human resources to implement the system. Several options are available to house such a system including MVA office, University of Saskatchewan and the City of Saskatoon. However, considering that policy and regulation initiatives would be primarily an MVA concern, and given its role of public information and ease of access, it seems logical that such an installation be housed at the MVA office with an individual designated to manage and support the hardware and software system to ensure maintenance and updating of the database.

Several options should be explored which may lead to economic implementation of a data management system. One of these may be through use of summer students familiar with the computer sciences and data management on a micro computer system. Another avenue to explore is incorporation of this topic into a graduate research program in an appropriate faculty at the University of Saskatchewan and funding of the program through a research grant. However it is implemented, the program will require $12,000 to $15,000 in direct costs plus labour costs to establish the initial data base and train the system operator. It is estimated that a minimum of six months would be required for this task giving a total cost to establish the database of between $30,000 to $40,000.

14.0 ROLES AND LIABILITY

The role of MVA is a complex one, dealing with preservation and interpretation of the environment, education and regulation of activities. Communication and education of the public as to the nature and degree of hazard, and the need for special care in dealing with the valley slopes, and the development of policies are required. Enforcement will become easier if the public is aware of the hazard and participates in policy development.

Public education is a first step. The Meewasin Valley is a rich geological resource that can be interpreted at many locations to explain the landslide phenomena. Beaver Creek was a laudible first effort. However, it is somewhat remote and similar opportunities exist to interpret slope instability at Diefenbaker Park, along Saskatchewan Crescent, at the President's Residence, the Ski Jump and Petursson's Ravine among others. Excellent geologic exposures exist at the University Bridge, the Ski Jump and Petursson's Ravine. These form a natural base from which to explain the phenomena of the valley slopes.

Policy development is a second step. Some of the policies suggested herein will impact on individual landowners and will not be accepted without public discussion and input. However, policies which provide for betterment of the individual landowner as well as protection of the public and preservation of the landscape have a better chance of success.

Regulation and enforcement are the final step. As discussed above, it is recommended that this be done in concert with the City of Saskatoon to avoid administrative duplication.

Liability

The question of liability and ethical responsibility require serious consideration. It is common practice for many public agencies to disavow liability through a disclaimer that "The Authority accepts no responsibility for the accuracy or liability of data provided", although such disclaimers have not faired well in the courts.

It would seem that the Authority can take one of two positions. It can establish firm guidelines for approval and establish review procedures and approve projects which meet public regulations. Alternatively, it can establish guidelines for practice but clearly leave the responsibility for the analysis and implementation to the professional engineer under whose seal such analyses and designs are issued. This is not a simple matter and considerable liability may be involved considering the usual high cost and risk to safety associated with landslides. The aspect of liability should be referred to the Authority's solicitor for advice. Should advise be received that some liability may accrue to MVA and its staff, consideration should be given to acquiring suitable liability insurance to cover such exposure.

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CONSULTING GEOTECHNICAL ENGINEERS

REGINA SASKATOON

SYMBOLS & TERMS

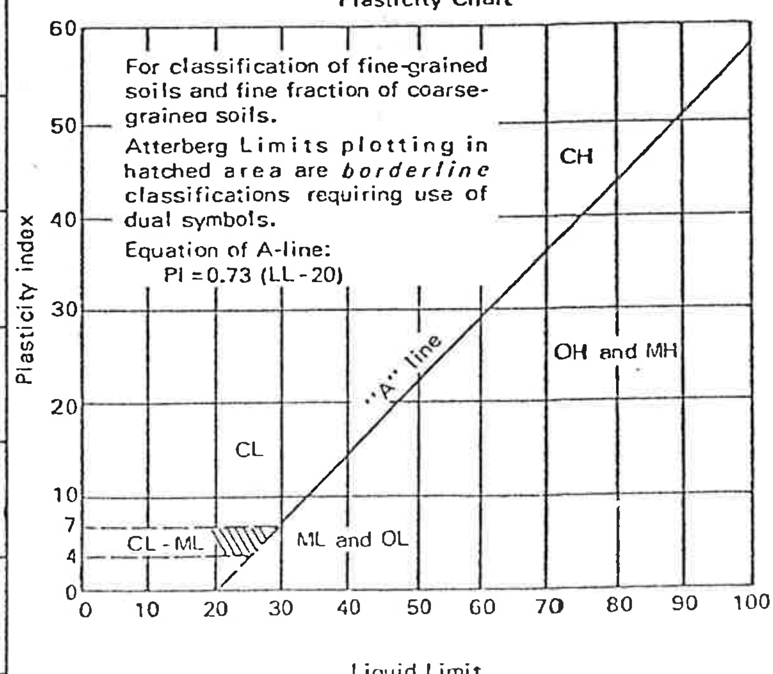
CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

ASTM Designation: D 2487 – 69 AND D 2488 – 69

(Unified Soil Classification System)

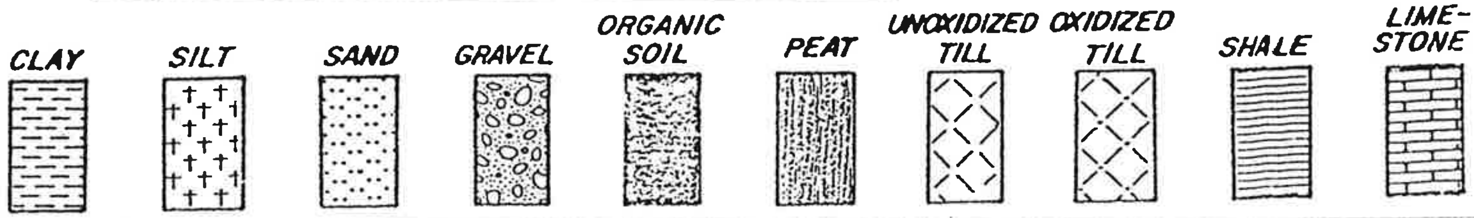
Major divisions			Group symbols	Typical names	Classification criteria		
Coarse-grained soils More than 50% retained on No. 200 sieve*	Gravels 50% or more of coarse fraction retained on No. 4 sieve	Clean gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines	Classification on basis of percentage of fines Less than 5% pass No. 200 sieve . . . . . . GW, GP, SW, SP More than 12% pass No. 200 sieve . . . . . . GM, GC, SM, SC 5 to 12% pass No. 200 sieve . . . . . . *Borderline* classifications requiring use of dual symbols	$C_u = \dfrac{D_{60}}{D_{10}}$ greater than 4; $C_z = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines		Not meeting both criteria for GW	
		Gravels with fines	GM	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Atterberg limits plotting in hatched area are *borderline* classifications requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixtures		Atterberg limits above "A" line with P.I. greater than 7	
	Sands More than 50% of coarse fraction passes No. 4 sieve	Clean sands	SW	Well-graded sands and gravelly sands, little or no fines		$C_u = \dfrac{D_{60}}{D_{10}}$ greater than 6; $C_z = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP	Poorly graded sands and gravelly sands, little or no fines		Not meeting both criteria for SW	
		Sands with fines	SM	Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Atterberg limits plotting in hatched area are *borderline* classifications requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures		Atterberg limits above "A" line with P.I. greater than 7	
Fine-grained soils 50% or more passes No. 200 sieve*	Silts and clays Liquid limit 50% or less		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands			
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL	Organic silts and organic silty clays of low plasticity			
	Silts and clays Liquid limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts			
			CH	Inorganic clays of high plasticity, fat clays			
			OH	Organic clays of medium to high plasticity			
Highly organic soils			Pt	Peat, muck and other highly organic soils			

Plasticity Chart



* Based on the material passing the 3 in. (76 mm) sieve.

SYMBOLS & TERMS USED IN THE REPORT



The Symbols May Be Combined to Denote Various Soil Combinations, The Predominant Soil Being Heavier.

RELATIVE PROPORTIONS

TERM	RANGE
Trace	0 - 5%
A Little	5 - 15%
Some	15 - 30%
With	30 - 50%

CLASSIFICATION BY PARTICLE SIZE

Boulder		Over 200 mm
Cobble		75 mm - 200 mm
Gravel		
	Coarse	20 mm - 75 mm
	Fine	#4 - 20 mm
Sand		
	Coarse	#4 - #10
	Medium	#10 - #40
	Fine	#40 - #200
Silt		#200 - 0.002 mm
Clay		Finer Than 0.002 mm

NOTE:
Sieve Sizes Shown Are Canadian Standard

DENSITY OF SANDS AND GRAVELS

DESCRIPTIVE TERM	RELATIVE DENSITY	STANDARD PENETRATION TEST
Very Loose	0 - 15%	0 - 4 Blows Per 300 mm
Loose	15 - 35%	4 - 10 Blows Per 300 mm
Medium Dense	35 - 65%	10 - 30 Blows Per 300 mm
Dense	65 - 85%	30 - 50 Blows Per 300 mm
Very Dense	85 - 100%	Over 50 Blows Per 300 mm

CONSISTENCY OF CLAYS AND SILTS

DESCRIPTIVE TERM	UNCONFINED COMPRESSIVE STRENGTH - kPa	N VALUE STANDARD PENETRATION TEST	REMARKS
Very Soft	Less Than 25	Less Than 2	Can Penetrate with Fist
Soft	25 - 50	2 - 4	Can Indent with Fist
Firm	50 - 100	4 - 8	Can Penetrate with Thumb
Stiff	100 - 200	8 - 15	Can Indent with Thumb
Very Stiff	200 - 400	15 - 30	Can Indent with Thumb-Nail
Hard	400 and Greater	Greater Than 30	Cannot Indent with Thumb-Nail

NOTES: 1. Relative Density Determined by Standard Laboratory Tests.
2. N Value - Blows/300 mm of a 64 kg Hammer Falling 760 mm on a 50 mm O.D. Split Spoon

SYMBOLS & TERMS USED IN THE REPORT (continued)

DRILLING AND SAMPLING TERMS

SYMBOL	DEFINITION
C.S.	Continuous Sampling
Sy	3 Inch Thin Wall Tube Sample
Sy(2)	2 Inch Thin Wall Tube Sample
SS	2 Inch O.D. Split Spoon Sample
$\frac{\text{Blows}}{\text{Ft.}}$	"N" Value - Standard Penetration Test
Bag	Disturbed Bag Sample
No.	Sample Identification Number
→	Piezometer Tip
S.I.	Slope Indicator
SPG →	Observed Seepage

LABORATORY TEST SYMBOLS

SYMBOL	DEFINITION
O	Moisture Content - Percent of Dry Weight
⊢→	Plastic and Liquid Limit Determined in Accordance with ASTM D-423 and D-424
▲	Dry Density - Pounds Per Cubic Foot
■	Shear Strength - As Determined by Unconfined Compression Test
⊠	Shear Strength - As Determined by Field Vane
□	Shear Strength - As Determined by Pocket Penetrometer Test
% SO_4	Water Soluble Sulphates - Percent of Dry Weight
M.A.	Grain Size Analysis

GROUNDWATER

▽ - Water level measured in the borings at the time and under the conditions indicated. In sand, the indicated levels can be considered reliable groundwater levels. In clay soil, it is not possible to determine the groundwater level within the normal scope of a test boring investigation, except where lenses or layers of more pervious waterbearing soil are present and then a long period of time may be necessary to reach equilibrium. Therefore, the position of the water level symbol for cohesive or mixed texture soils may not indicate the true level of the groundwater table. The available water level information is given at the bottom of the log sheet.

▼ - Water level determined by piezometer installation - In all soils the levels can be considered reliable groundwater levels.

DESCRIPTIVE SOIL TERMS

Well Graded	-	having wide range of grain sizes and substantial amounts of all intermediate sizes.
Poorly Graded	-	Predominantly of one grain size.
Slickensides	-	refers to a clay that has planes that are slick and glossy in appearance; slickensides are caused by shear movements.
Sensitive	-	Exhibiting loss of strength on remolding.
Fissured	-	Containing cracks, usually attributable to shrinkage. Fissured clays are sometimes described as having a nuggetty structure.
Stratified	-	Containing layers of different soil types.
Organic	-	Containing organic matter; may be decomposed or fibrous.
Peat	-	A fibrous mass of organic matter in various stages of decomposition. Generally dark brown to black in color and of spongy consistency.
Bedrock	-	Preglacial Material.
Drift	-	Material deposited directly by glaciers or glacial melt-water.
Alluvial	-	Soils that have been deposited from suspension from moving water.
Lacustrine	-	Soils that have been deposited from suspension in fresh water lakes.

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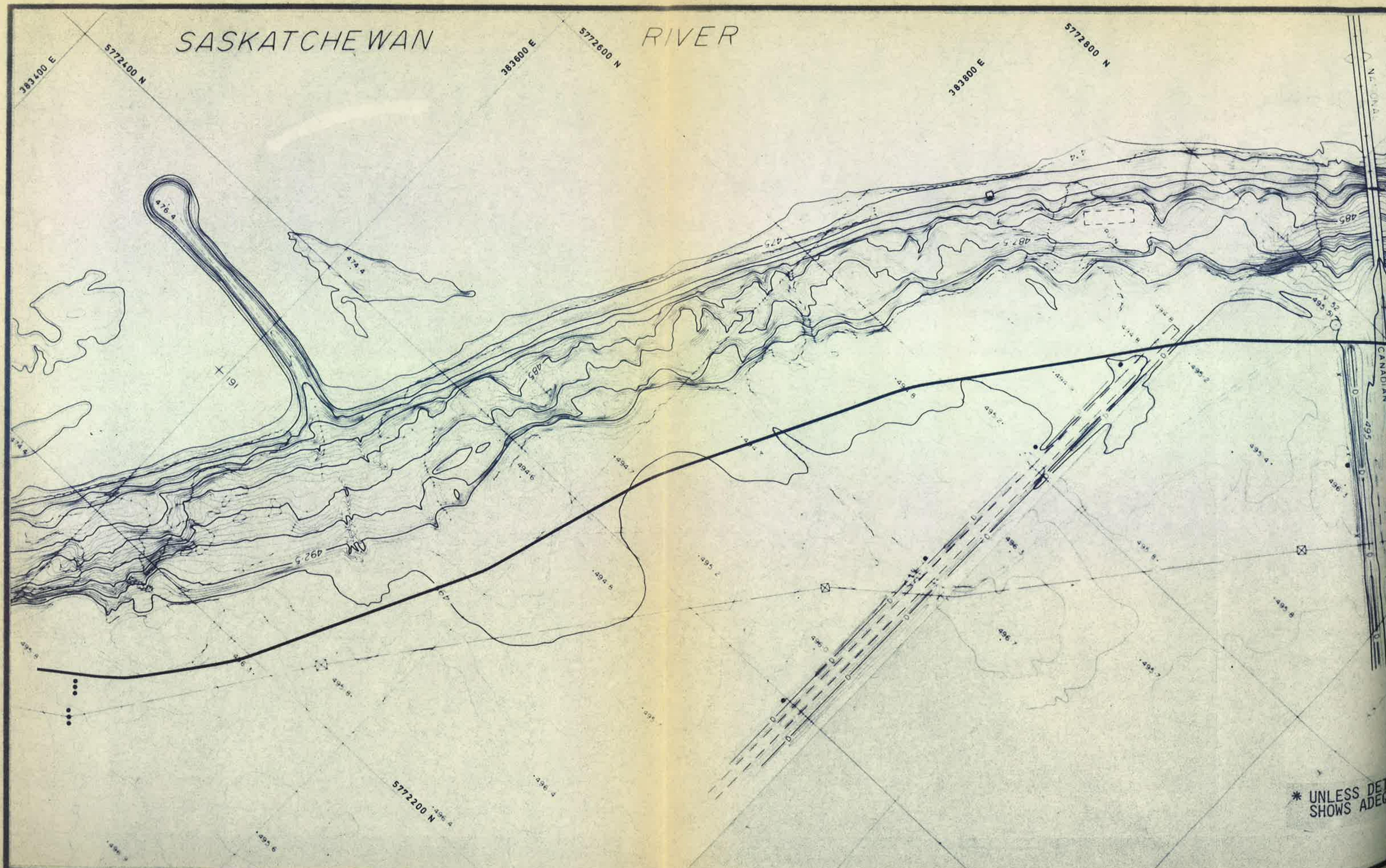
CONSULTING GEOTECHNICAL ENGINEERS

REGINA SASKATOON

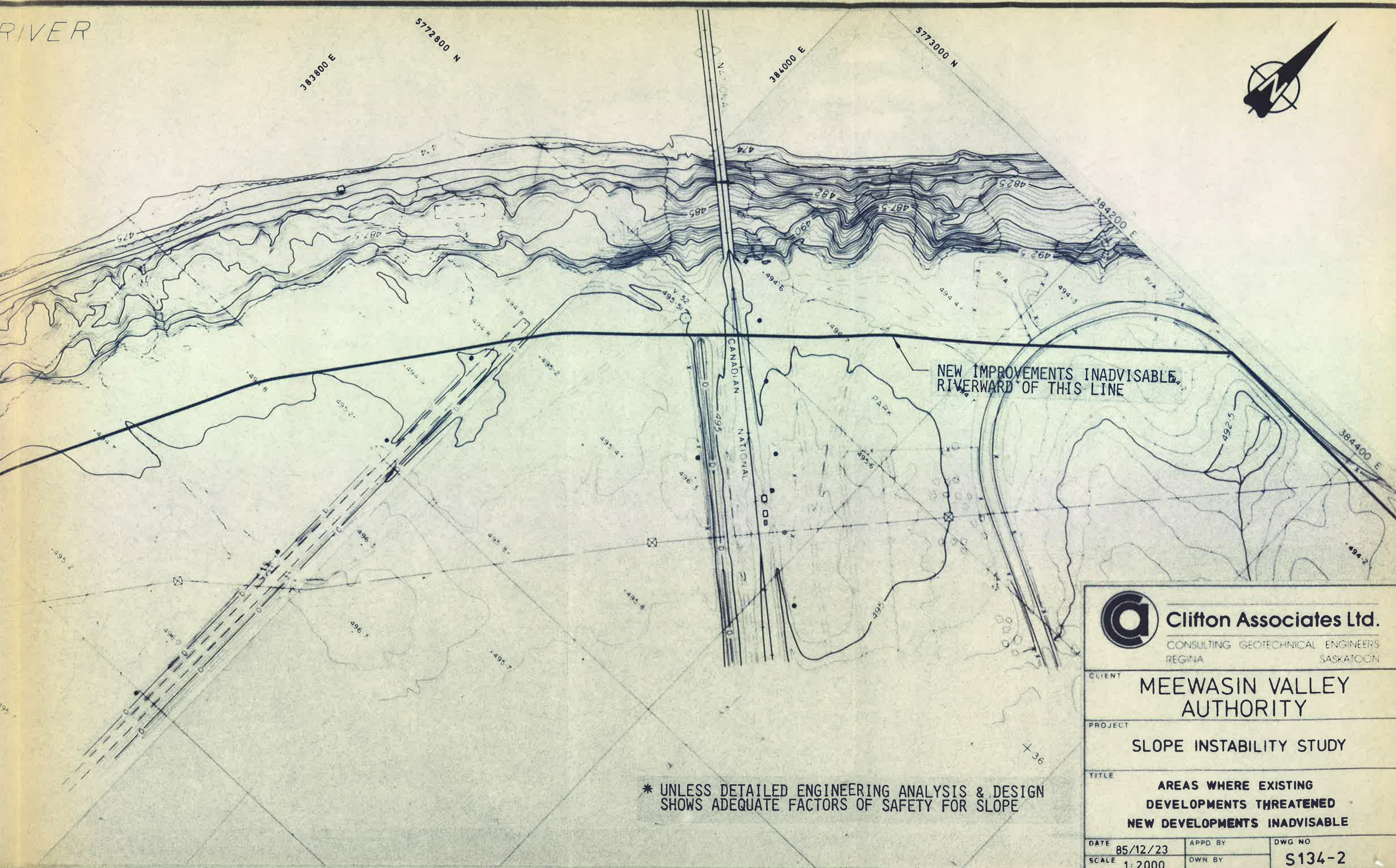
DRAWINGS


LACUSTRINE PLAIN
SASKATOON TERRACE
- sand & gravel
& cobbles over till
TERRACE
SOUTH
LACUSTRINE PLAIN
- silt, clay & fine sand
LANDSLIDE LOCATIONS
1 DIEFENBAKER PARK
2 QUEEN'S HOUSE OF RETREAT
3 CHERRY LANE
4 NUTANA COLLEGIATE
5 LONG HILL
6 BROADWAY BRIDGE
7 13TH STREET
8 15TH STREET
9 McCRANEY (16TH STREET)
10 17TH STREET
11 18TH STREET
12 PRESIDENT'S RESIDENCE
13 DEVIL'S DIP
14 SKI JUMP
15 PETURSSON'S RAVINE
16 PENITENTIARY PIPING
17 RAVINE DRIVE


LEGEND:
EXISTING IMPROVEMENTS POTENTIALLY THREATENED
DEPTH OF SURFICIAL STRATIFIED DRIFT (M)
ISOPACH OF LACUSTRINE SEDIMENTS (M)
LACUSTRINE PLAIN
SASKATOON TERRACE
-sand & gravel & cobbles over till
SASKATCHEWAN
RIVER
ERODED TILL PLAIN
- boulders over till
TERRACE
-silt & sand over till
ERODED LACUSTRINE / TILL PLAIN
ERODED TILL PLAIN
TH STREET)
RESIDENCE
RAVINE
PIPING
Clifton Associates Ltd.
CONSULTING GEOTECHNICAL ENGINEERS
REGINA
SASKATOON
CLIENT
MEEWASIN VALLEY AUTHORITY
PROJECT
SLOPE INSTABILITY STUDY
TITLE
GENERAL PLAN SHOWING SLIDE LOCATIONS, SECTION LOCATIONS & AREAS WHERE IMPROVEMENTS THREATENED
DATE 85/12/23
APPD BY A.W.C.
DWG NO S134-1
SCALE 1:20,000
DWN BY J.M.B.



SASKATCHEWAN RIVER
383400 E
5772400 N
383600 E
5772600 N
383800 E
5772800 N
5772200 N
CANADIAN NATIONAL
* UNLESS DETA
SHOWS ADEQU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Clifton Associates Ltd.
CONSULTING GEOTECHNICAL ENGINEERS
REGINA SASKATOON
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PROJECT
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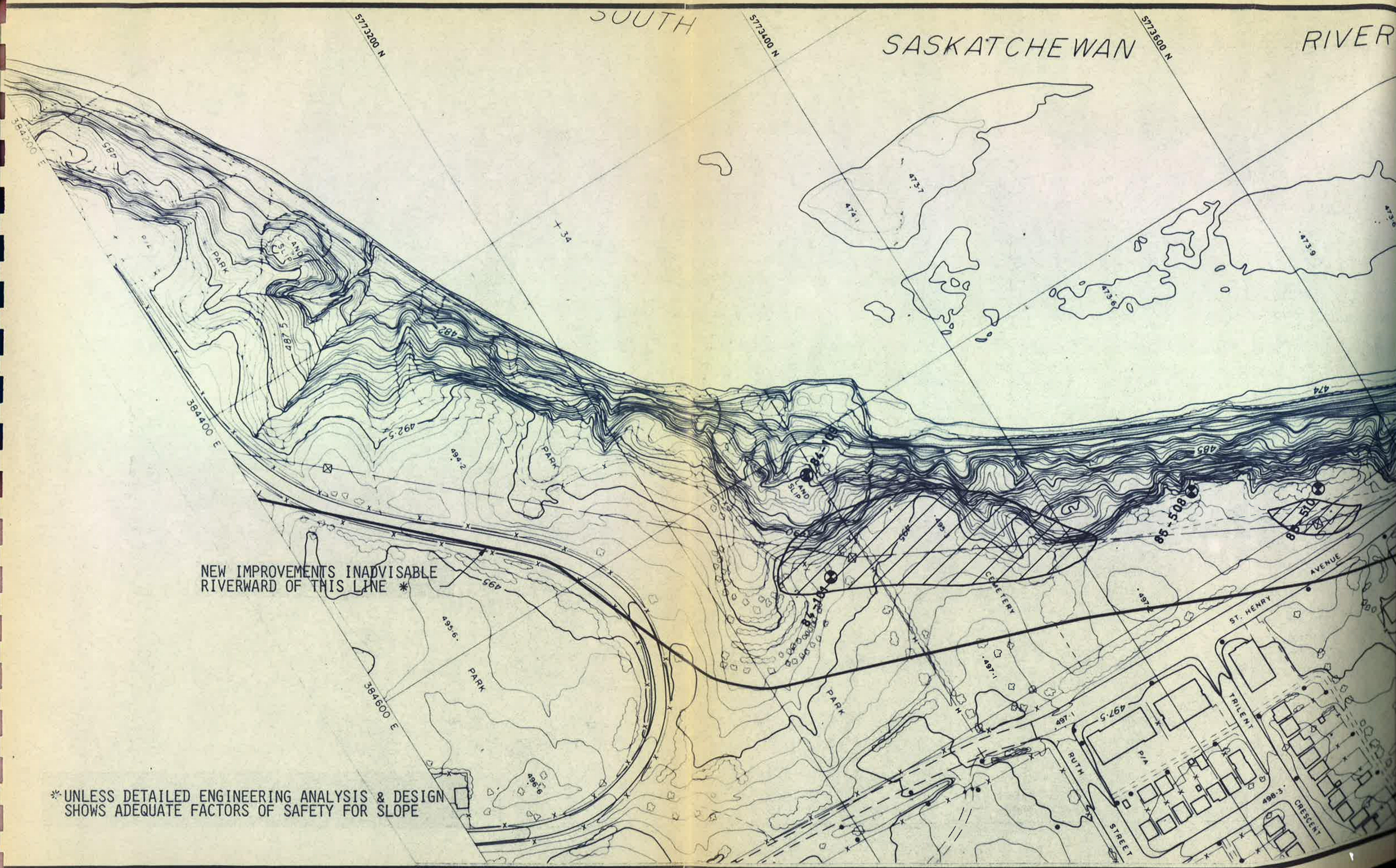
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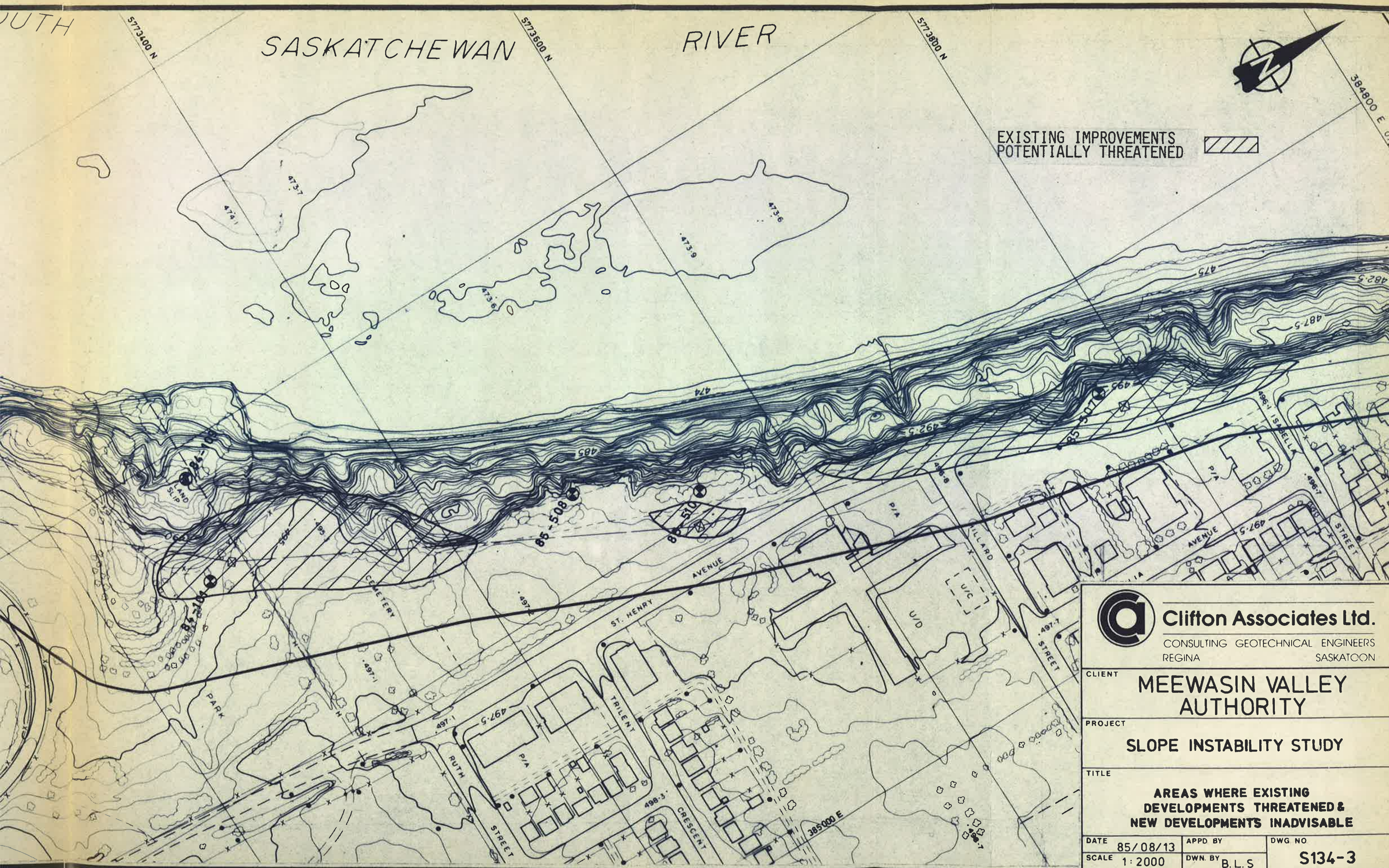

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Clifton Associates Ltd.
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REGINA SASKATOON
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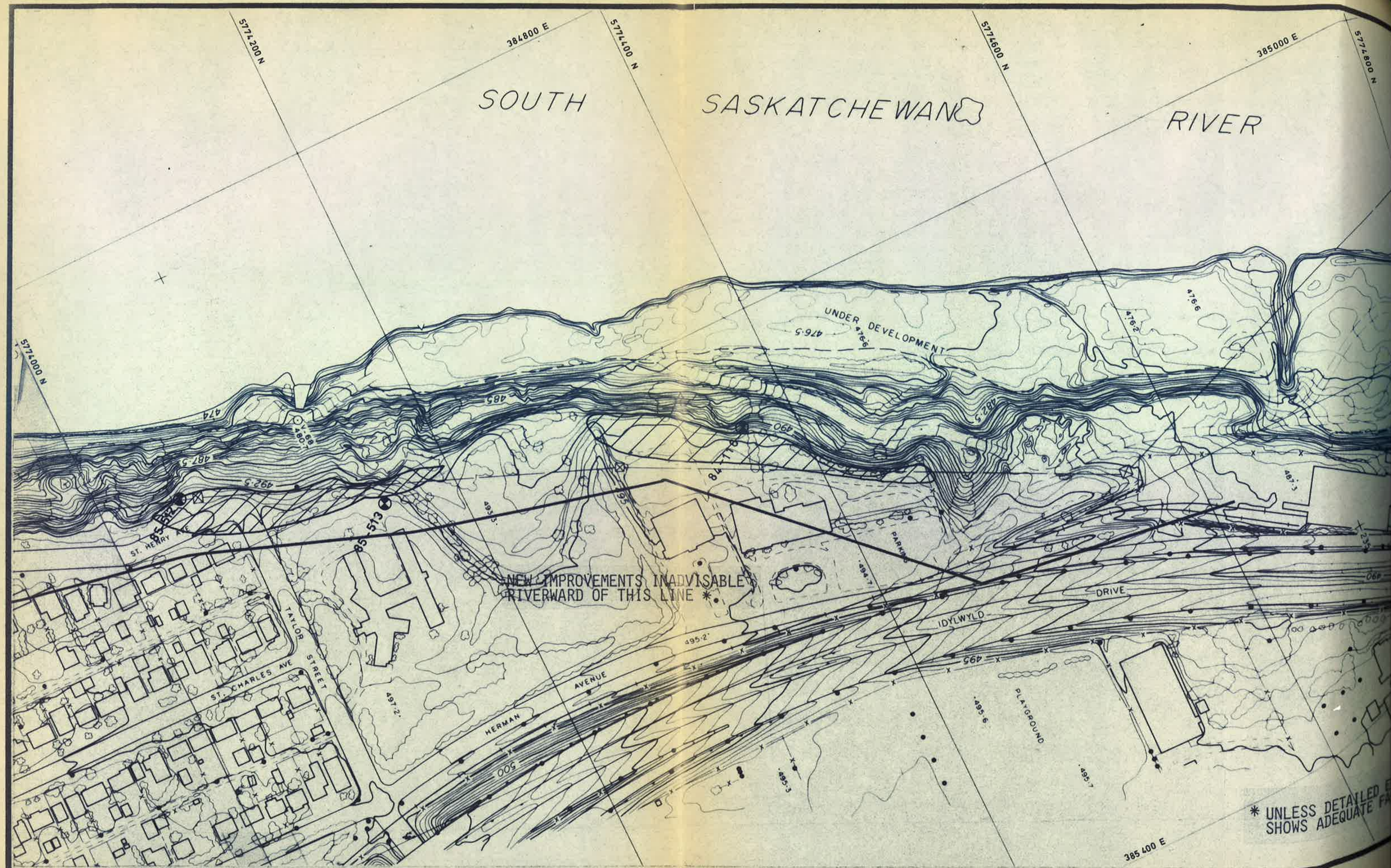

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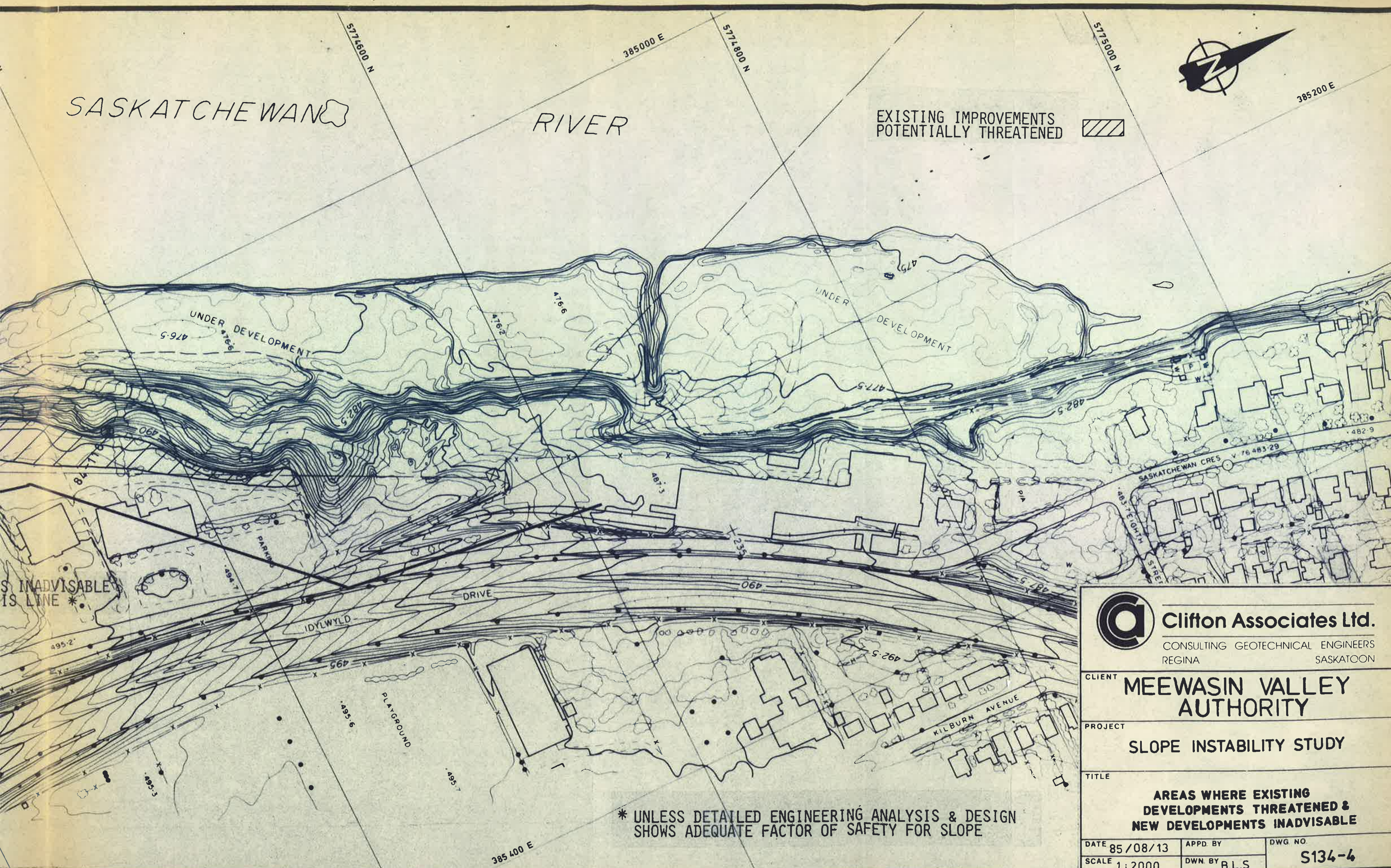
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Clifton Associates Ltd.
CONSULTING GEOTECHNICAL ENGINEERS
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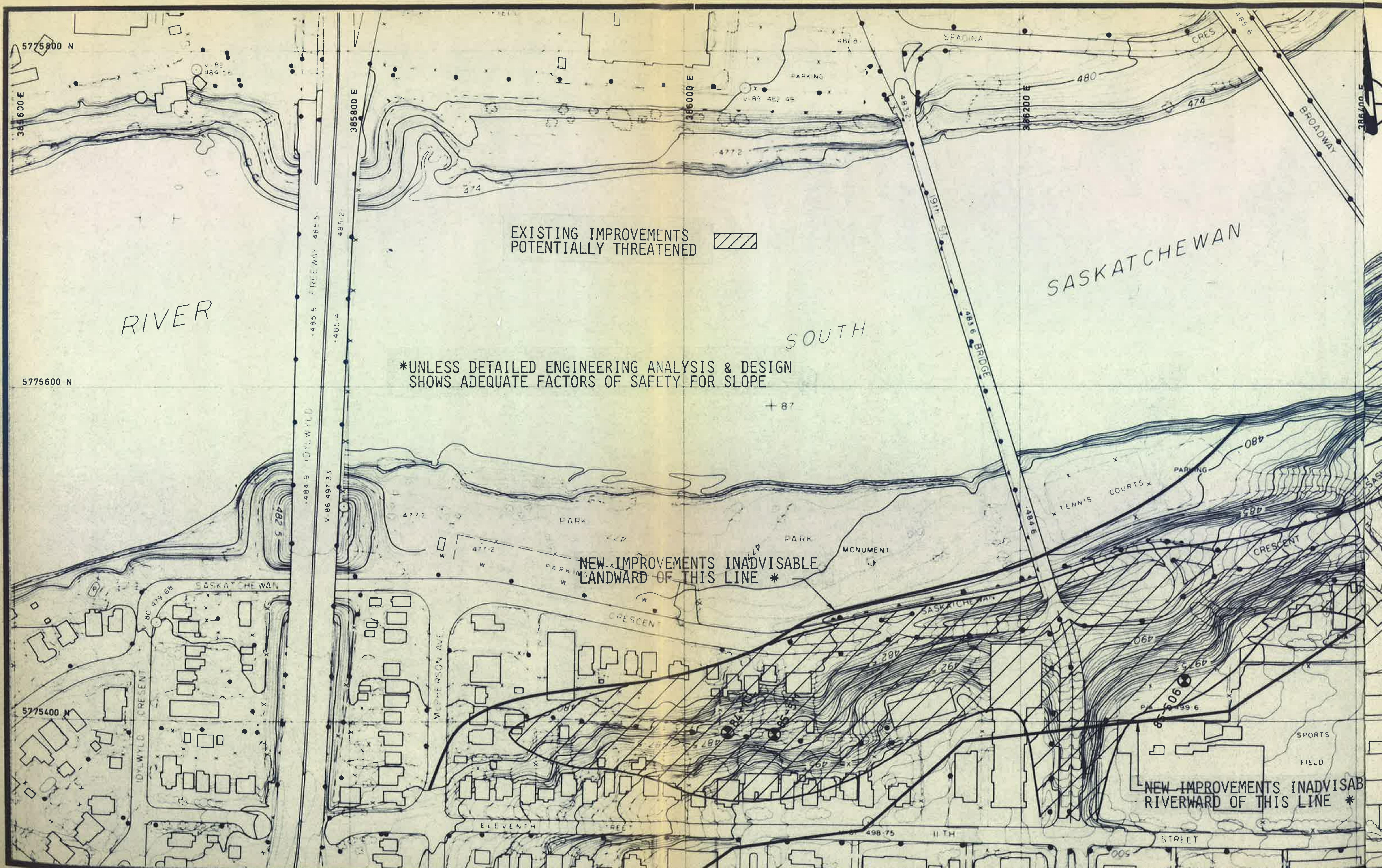

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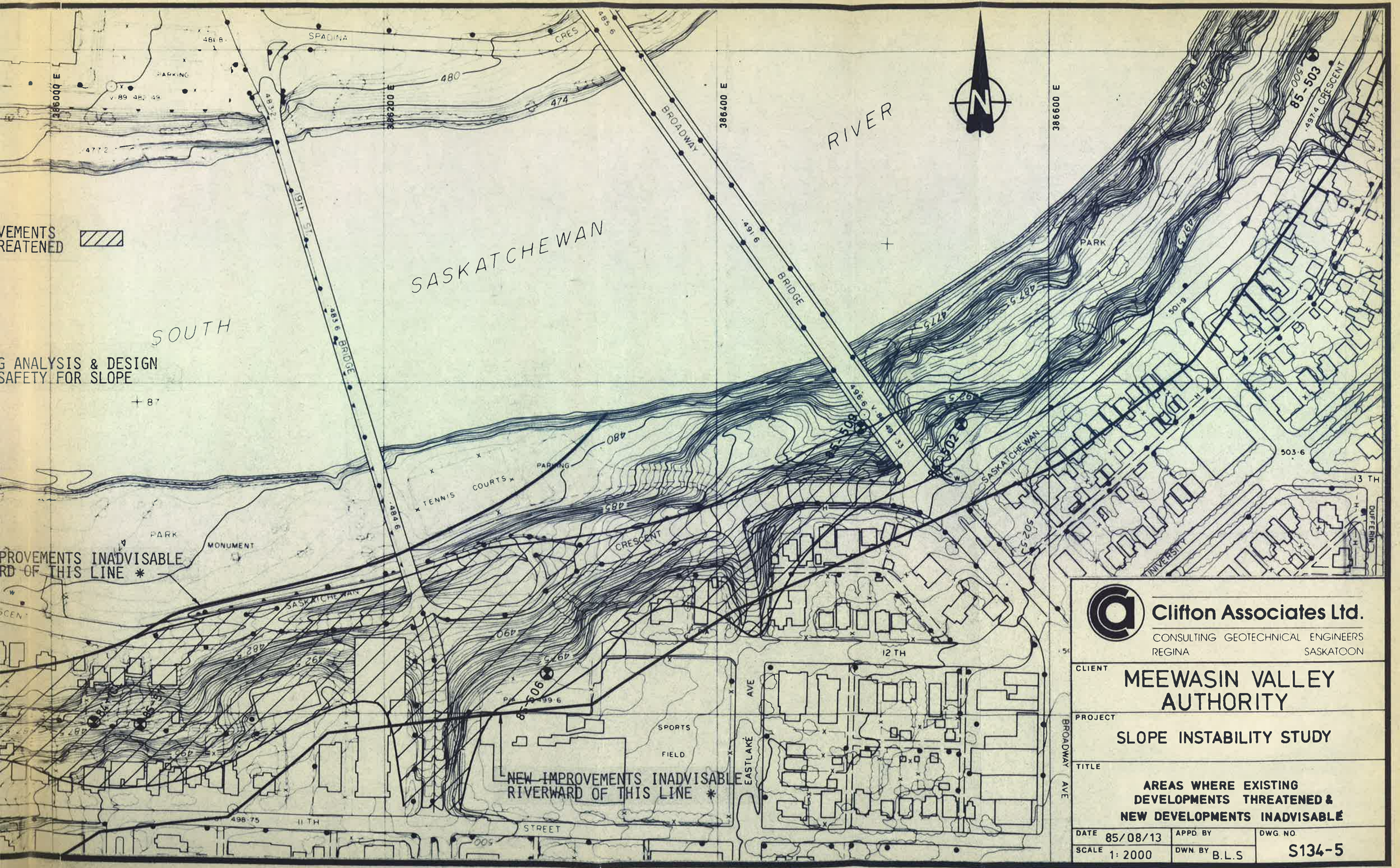
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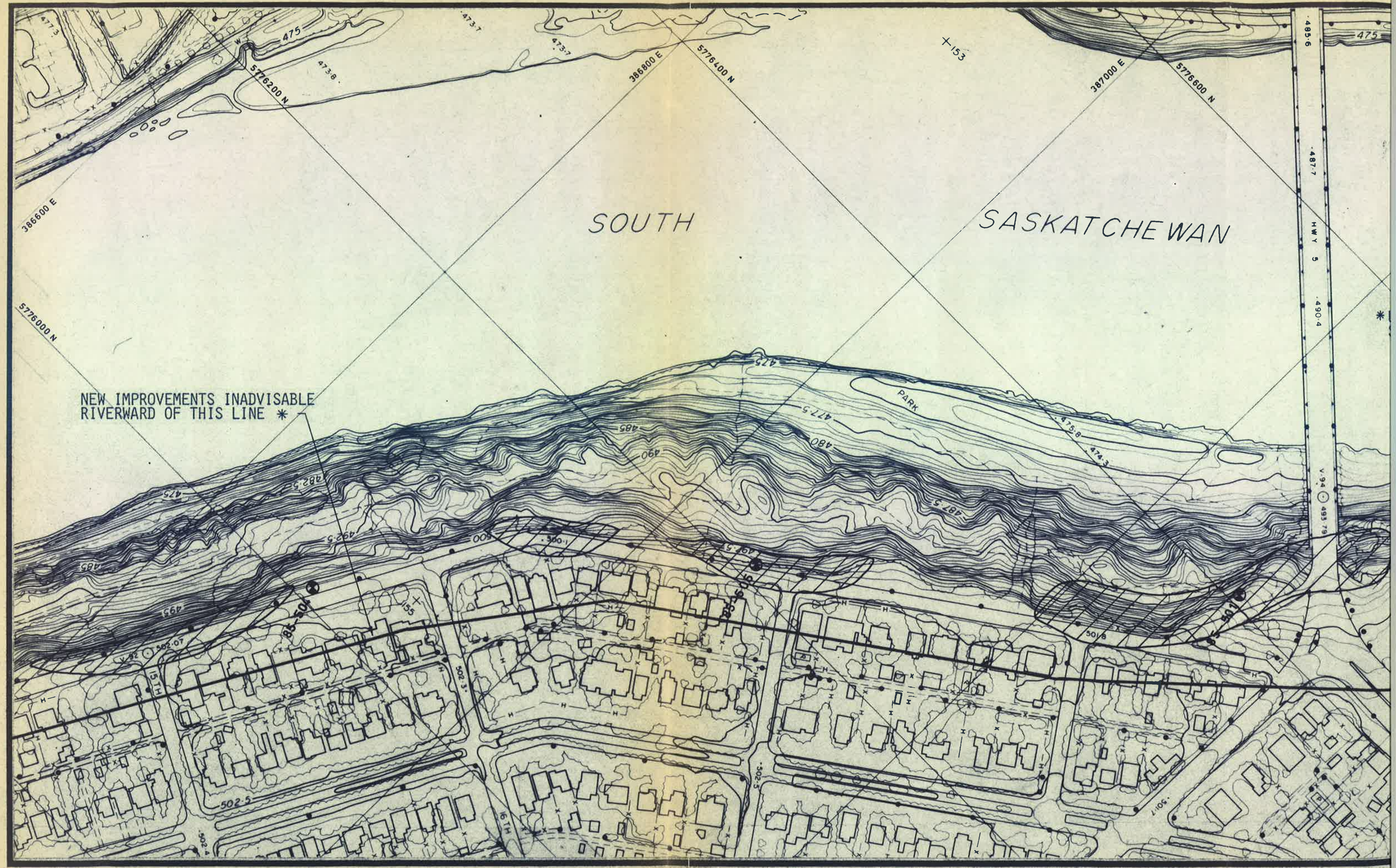

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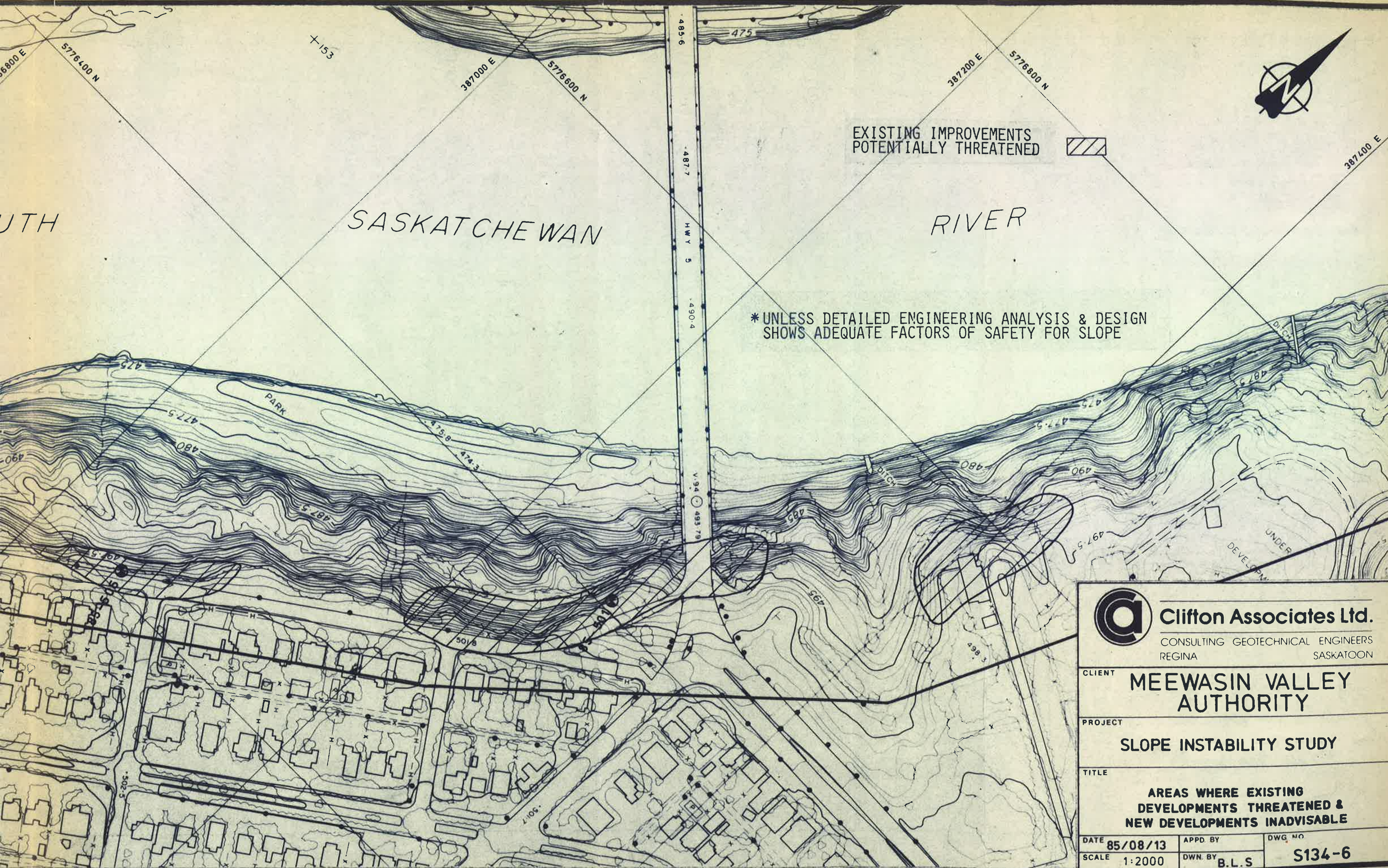
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Clifton Associates Ltd.
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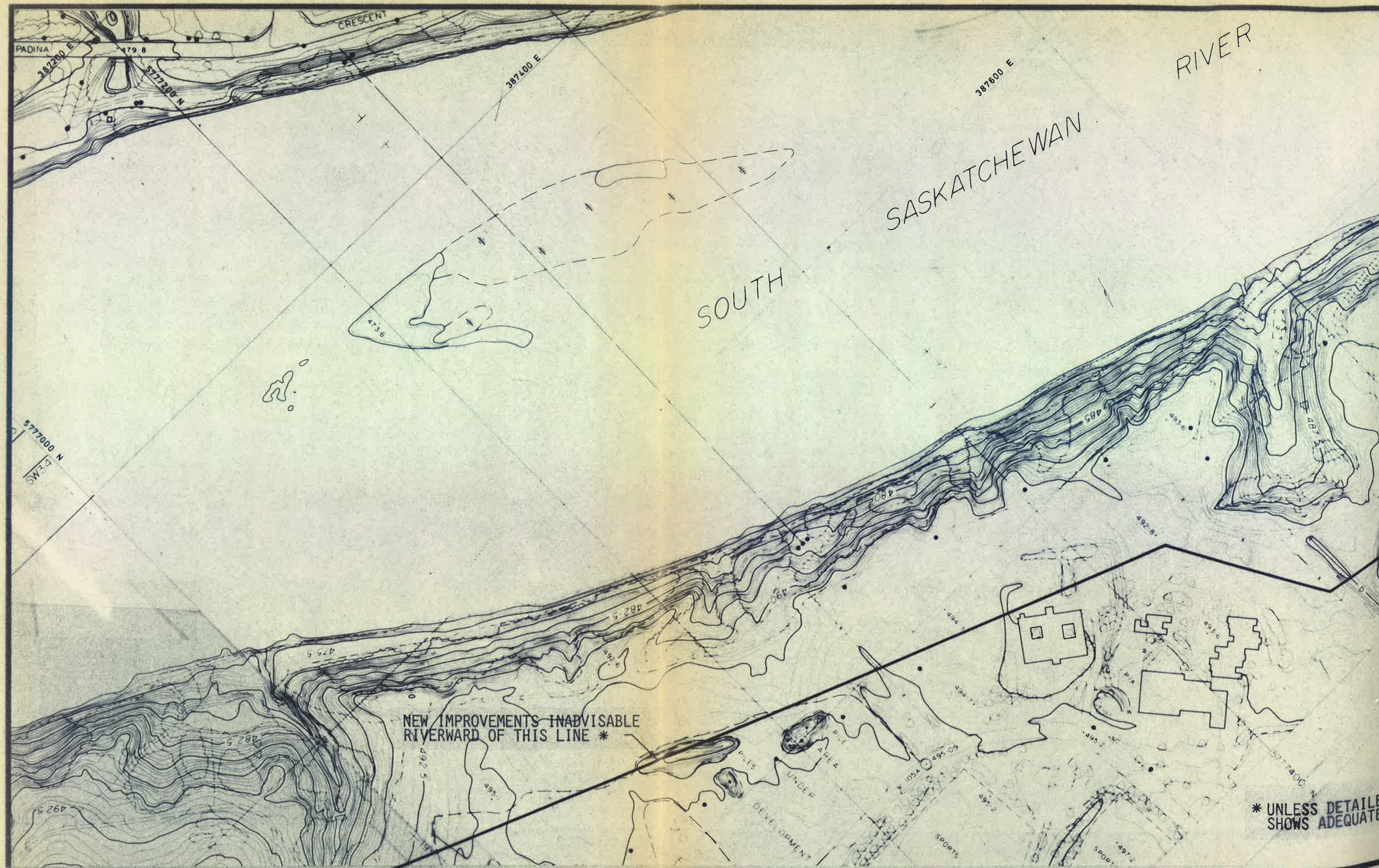

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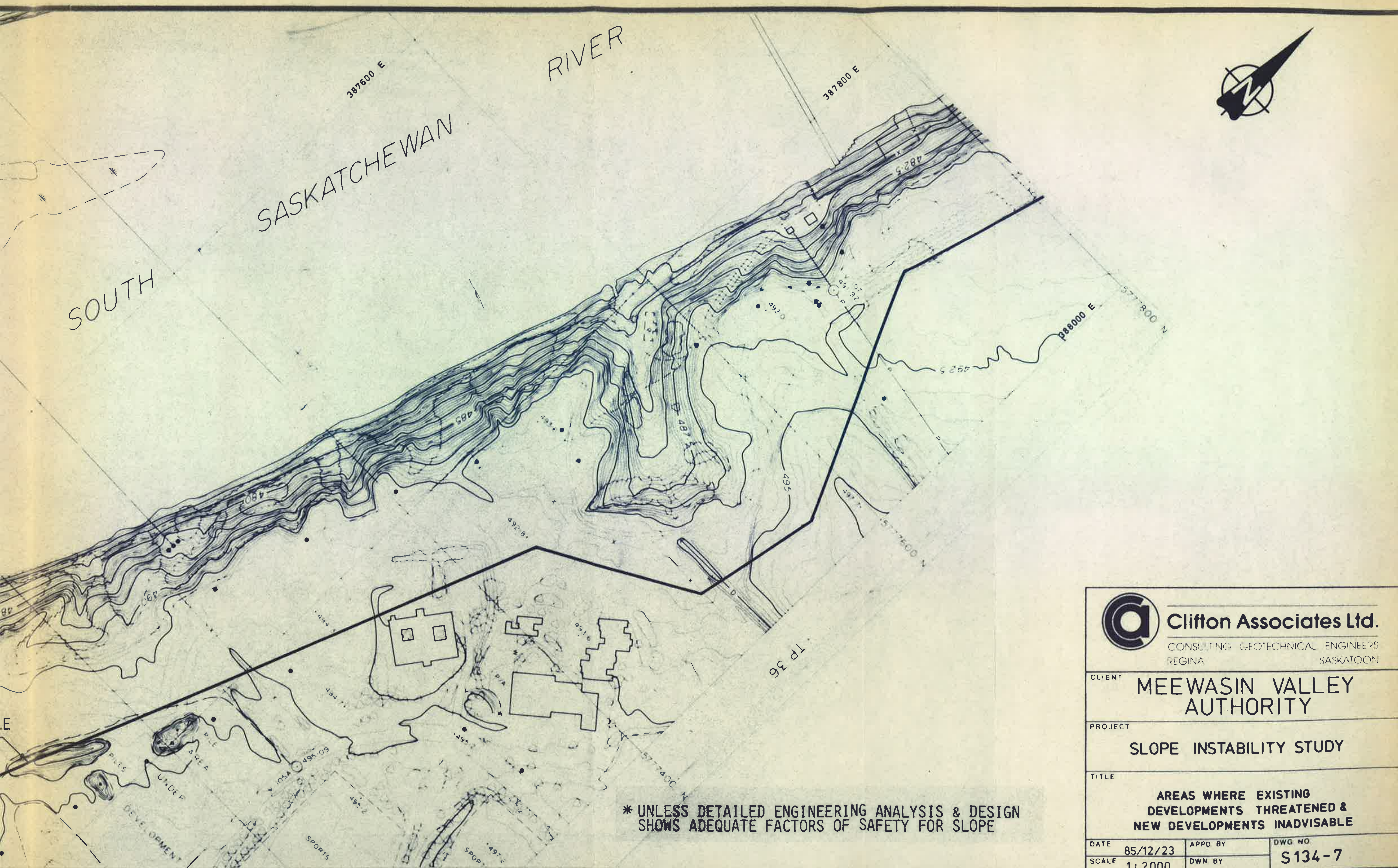

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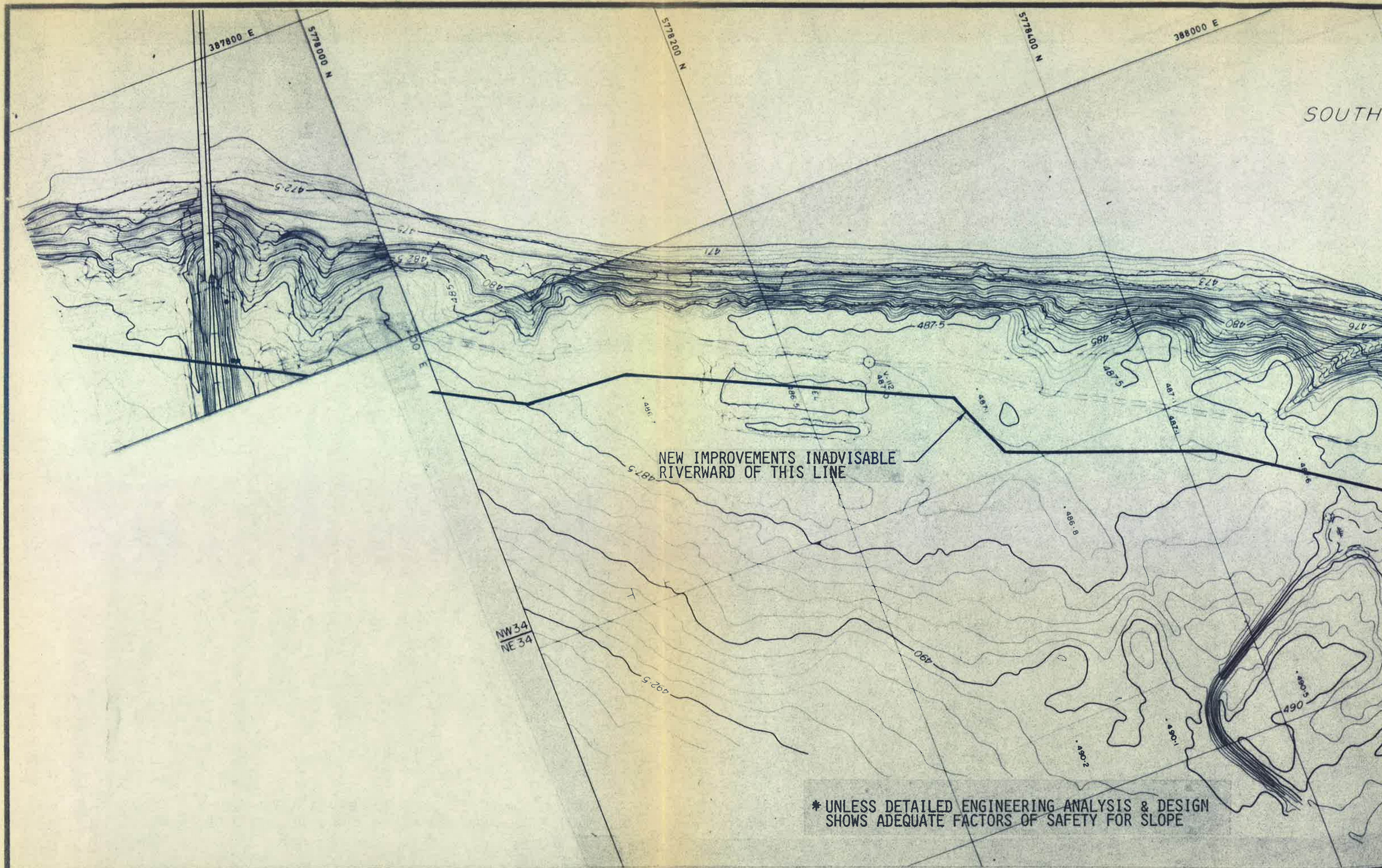

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Clifton Associates Ltd.
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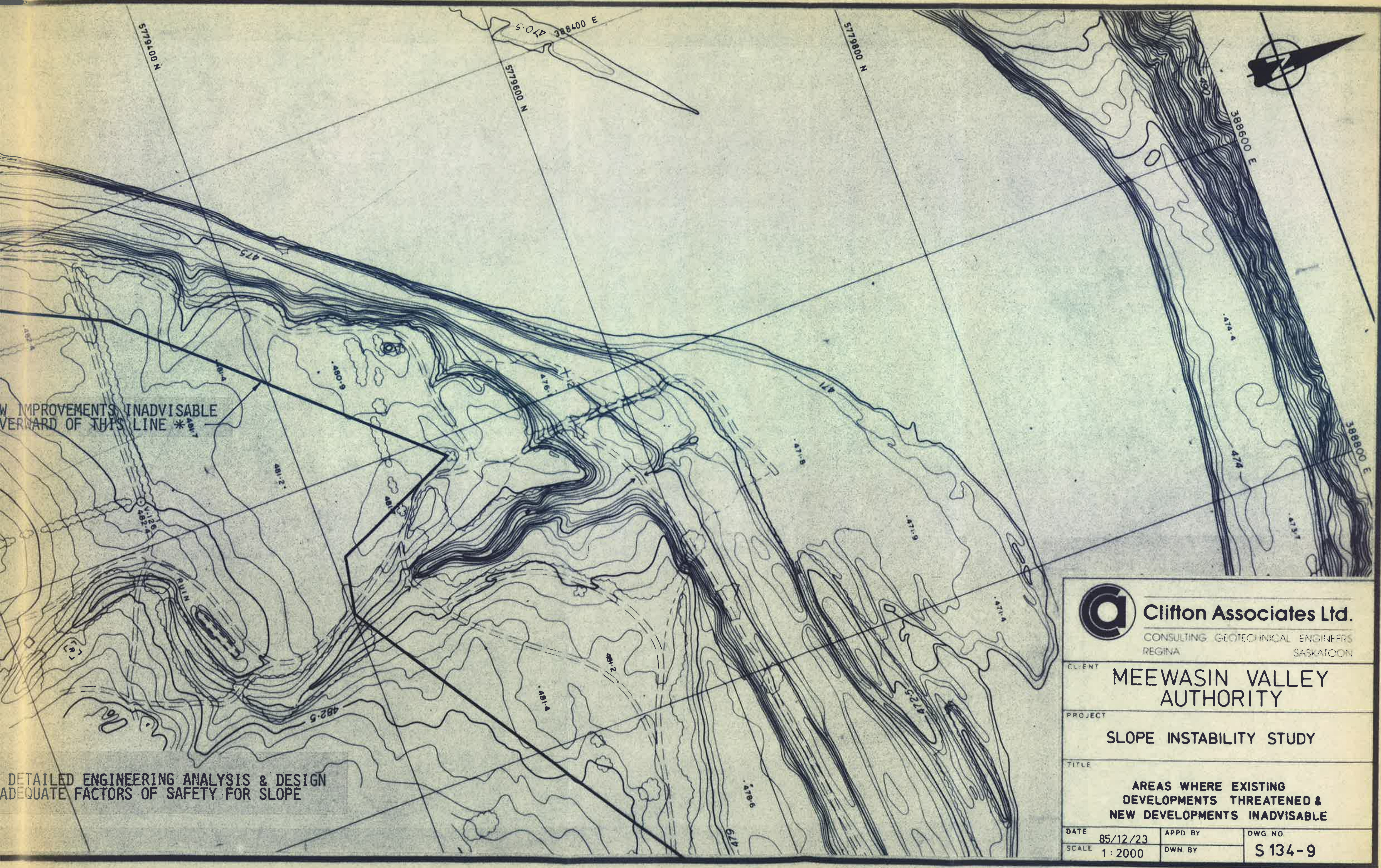

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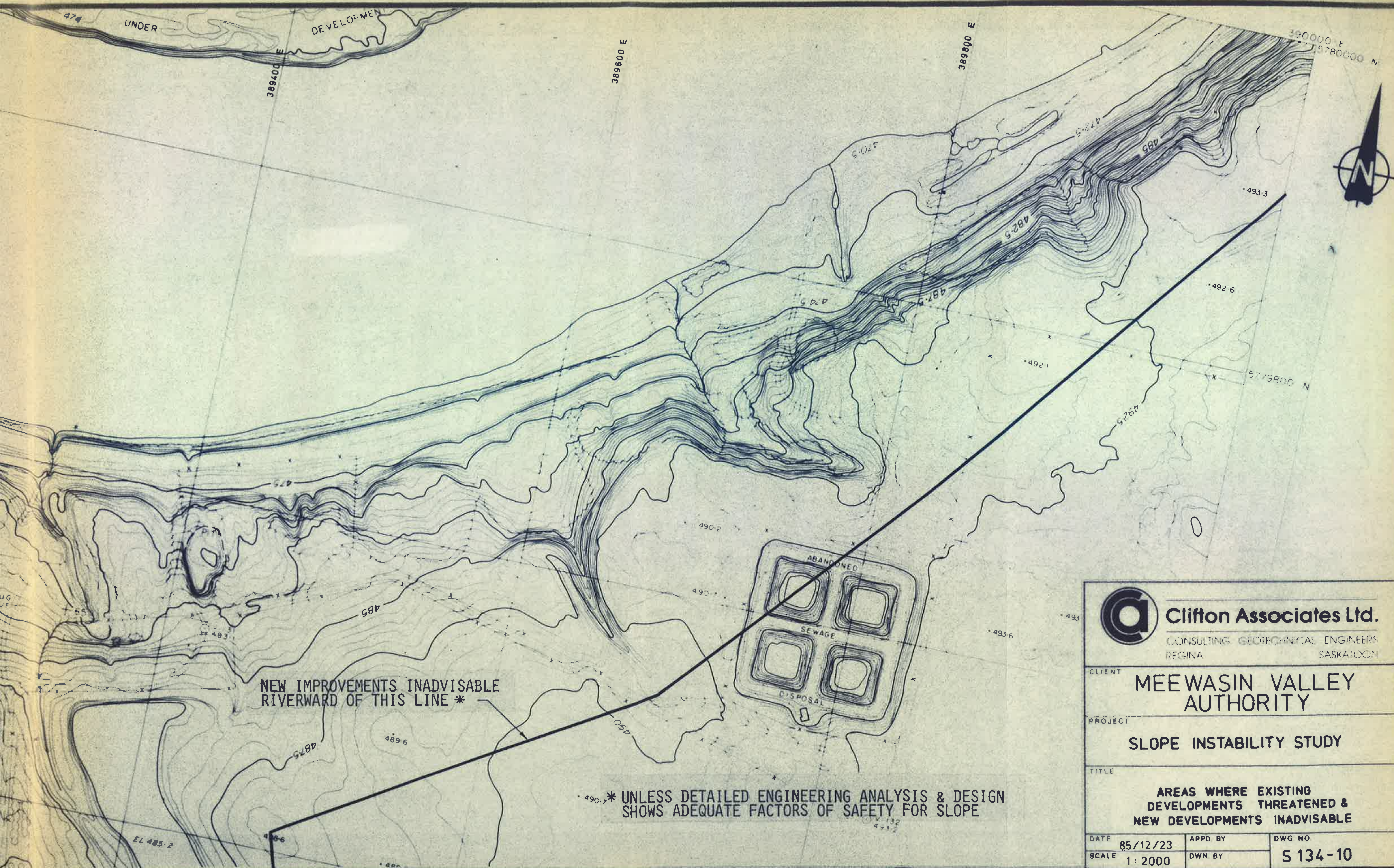

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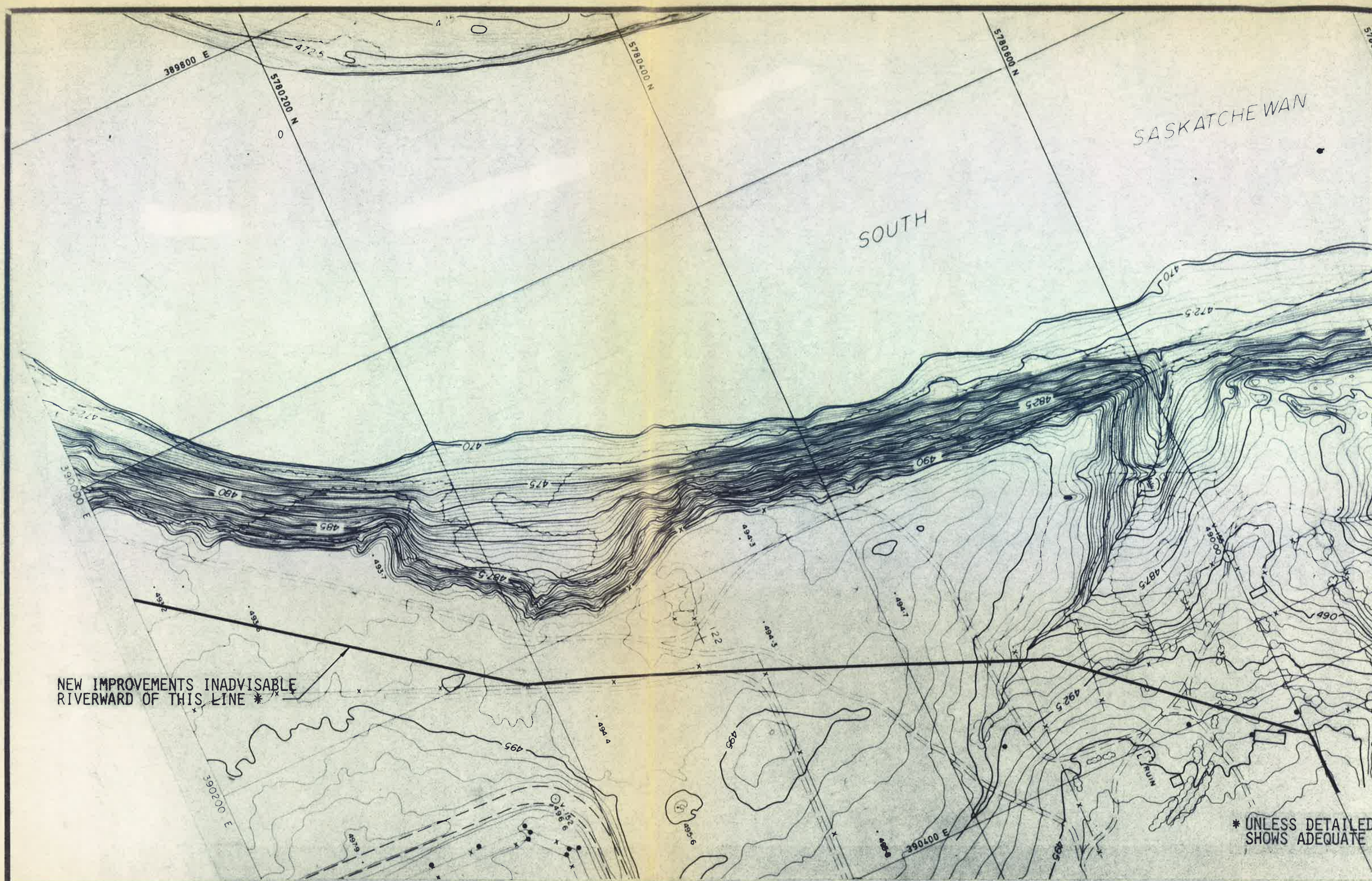
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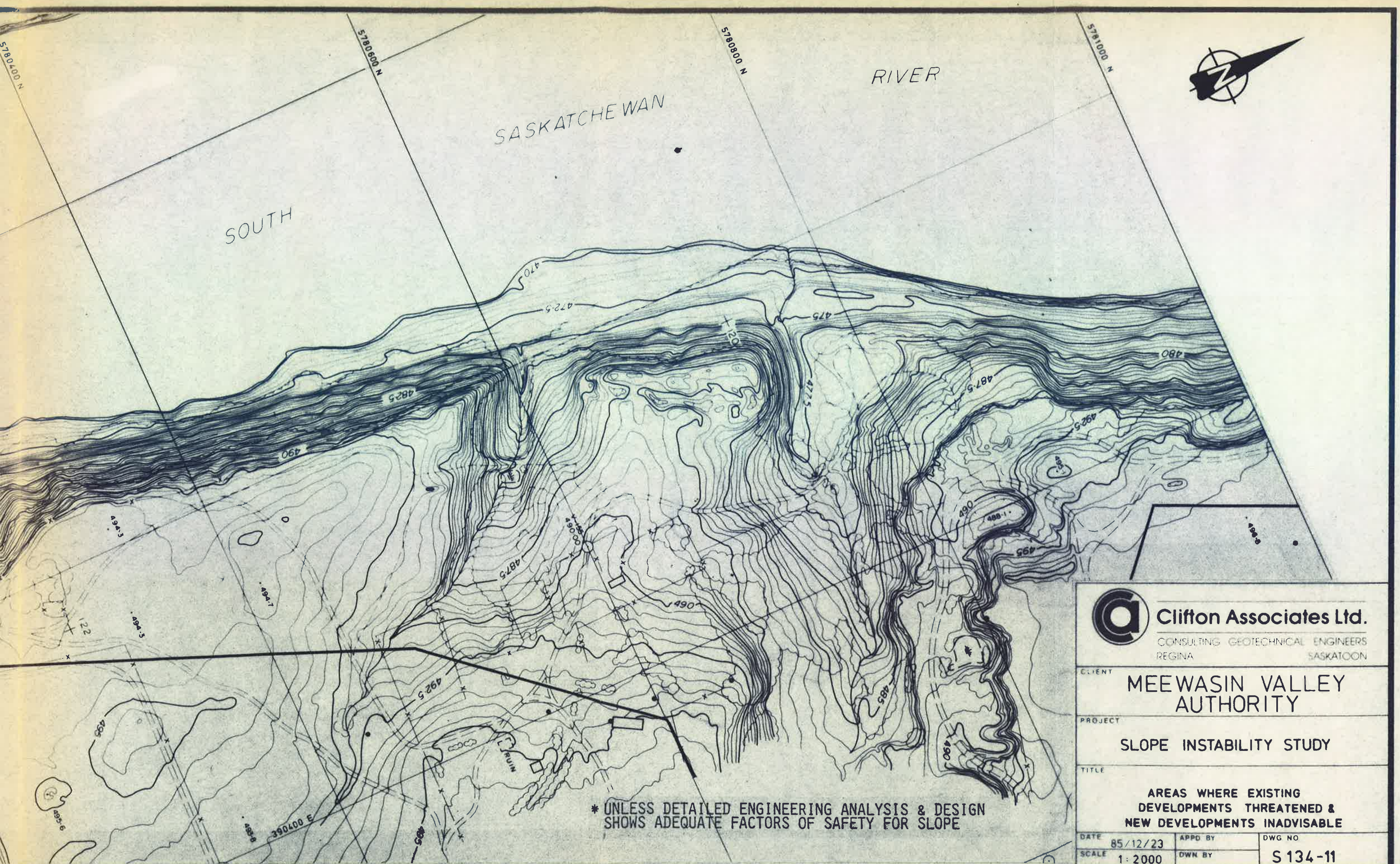
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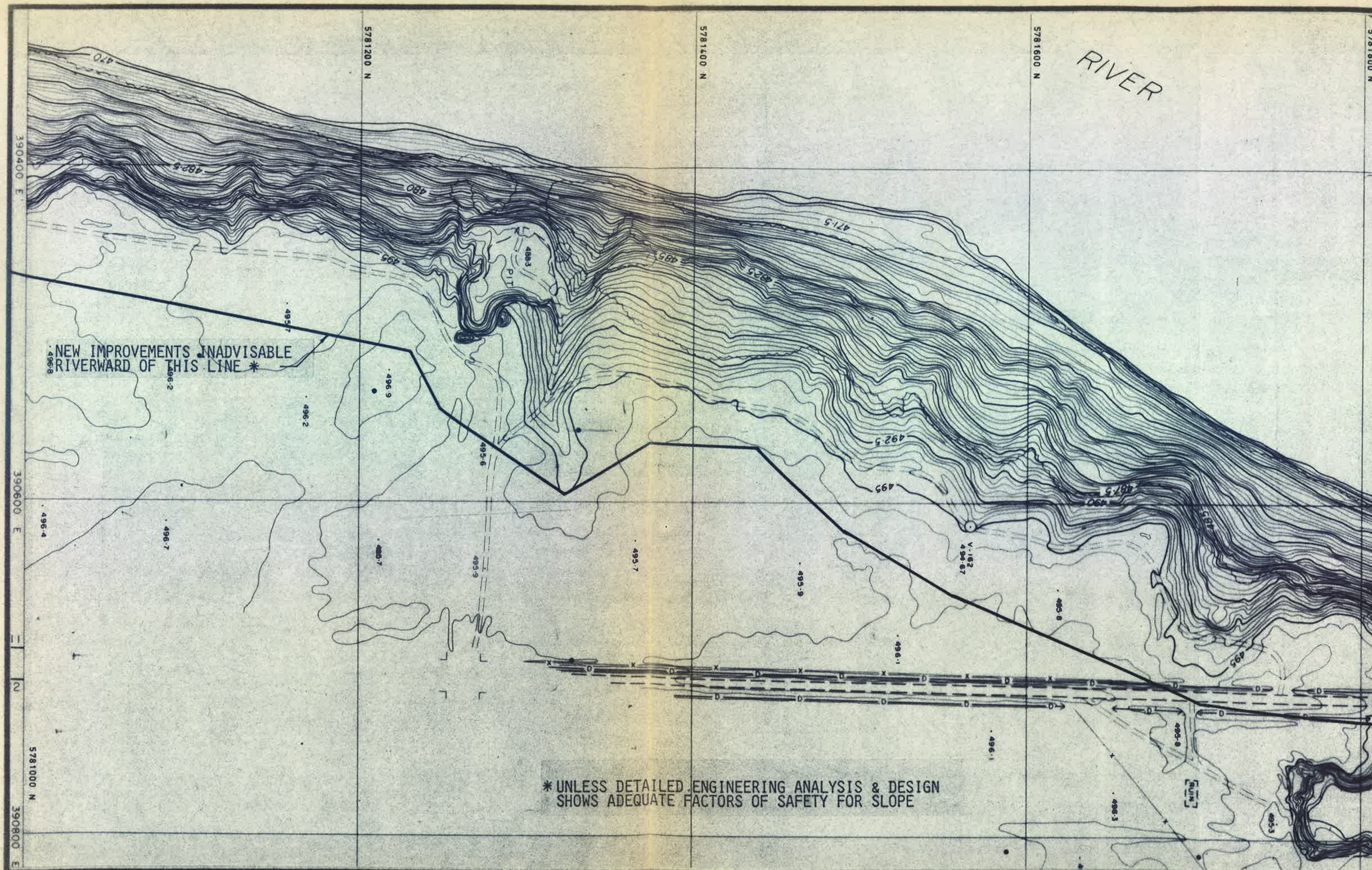
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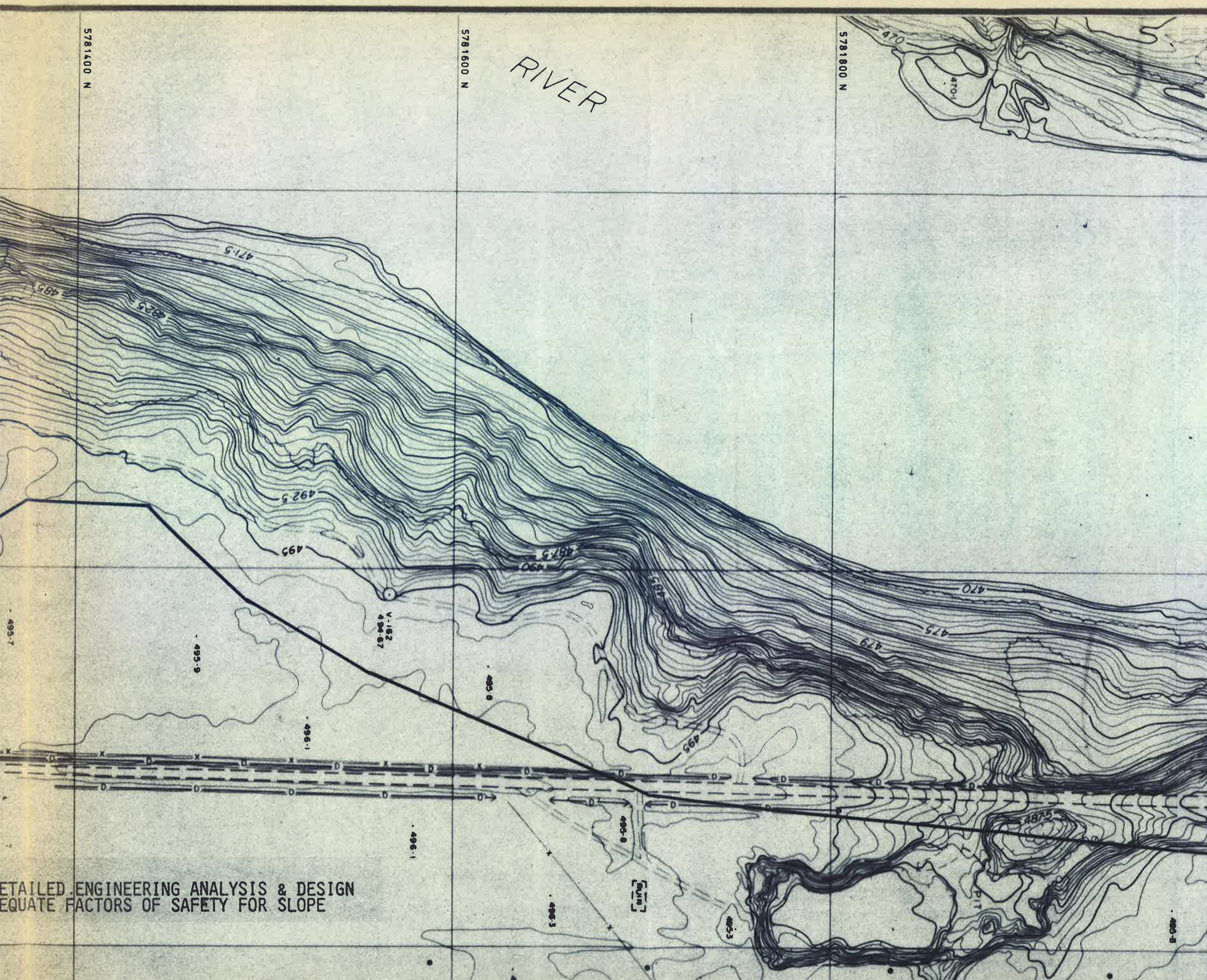

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REGINA SASKATOON

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MEEWASIN VALLEY AUTHORITY

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APPENDIX 'A'

RIVER FILL CONSTRUCTION STUDY - TERMS OF REFERENCE

A. Introduction

Outlined below are terms of reference for a three-phase study of the potentials and problems of using fill to extend the bank into the channel of the South Saskatchewan River.

Phases I and II as outlined below can occur concurrently. Phase III is an application of the results of Phases I and II to specific projects. Phase III is contingent upon the findings of Phases I and II.

B. Subject of the Study

The study shall examine the subject of fill construction in the South Saskatchewan River in the following manner.

Phase I: A brief, general assessment of the history of past river fill construction and a detailed evaluation of the ability of the river and the river channel to accept fill construction without incurring detrimental effects.

Phase II: The comparative effects and advantages of various fill methods and fill site design options and their relative impacts and advantages on channel characteristics and site use.

Phase III: The preferred fill methods and design, potential impacts, and projected costs of fill projects listed in Appendix I.

C. Study Area

The study area shall include the channel and shores of the South Saskatchewan River within the City of Saskatoon, and any areas which may be affected by the river as a result of fill construction. This area generally is shown on the attached map.

D. Study Period

The study shall begin no later than March 31, 1983, and shall be completed within a four-month period.

E. Study Requirements

As parts of a comprehensive evaluation of the potentials and problems of fill construction in the South Saskatchewan River in Saskatoon, the consultant shall undertake:

Phase I: an evaluation of the hydrology and morphology of the river channel and its banks with the following objectives:

a) a brief documentation and assessment of the history of fill construction in the river;

b) to identify and describe the impacts of channel alteration by fill construction in terms of:

i) flood damage potential;
ii) erosion and deposition patterns;
iii) channel and shore habitat;
iv) operation of existing utilities; and
v) other impacts identified by the consultant;

c) to deliniate those portions of the river channel and banks within the study area where, in consideration of the potential impacts identified in (a), above, fill construction would:

i) be beneficial;
ii) be of little or no consequence;
iii) cause impacts which are easily mitigated; and
iv) cause impacts which are difficult or impossible to mitigate.

For each of the results described above, the benefits, impacts and mitigating measures shall be specified.

Phase II: an assessment of the design options, materials, and methods of river channel fill construction with the following objectives:

a) to recommend a number of alternatives which provide safe, stable, and aesthetically pleasing fill sites.

b) to identify and describe the relative impacts of each of the options identified in (a), above, with the objective of minimizing the potential impacts of fill as detailed in phase (1) of the study;

c) to identify the advantages and constraints to improvement and use of each of the fill options identified in (a), above; and

d) determine the relative cost of each of the fill options identified in (a), above.

Phase III: an assessment of two hypothetical fill projects (as detailed in Appendix I) with the following objectives:

a) to recommend the most suitable design, matherials and methods for each project;

b) to identify and describe the probable impacts of each project, and of any combination of the two projects on present bank conditions and riverbank plans of the Authority;

c) to recommend measures required to mitigate detrimental impacts and identification of those impacts for which there is no solution; and

d) to outline the general cost implications of the two fill projects, including the cost of mitigating probable impacts.

The Consultant Shall

While undertaking the required study, the consultant shall:

1. Provide bi-weekly updates of the progress of the study and receive guidance from Meewasin staff as required.

2. Provide reports of work completed including invoices or other materials necessary to determine interim payment for work each month.

3. Supply adequate drawings, documents and other materials required to illustrate the contents of the study.

4. Attend Meewasin committee or Authority meetings, or any other such meetings as may be required, in order to present and discuss the results of the study once the work is completed.

5. Complete all required work within the study period determined at the outset of the work.

6. Provide MVA with 40 copies of the study report including one copy which is unbound and suitable for reproduction.

7. Adequately reference the content of the report and provide one copy of documents, or pertinent portions of documents referenced in the report.

8. In consultation with Meewasin staff, include an executive summary within the written report.

Information Supplied

To assist the work the Meewasin will lend the following information:

1. Air photo mosaic mylars (1978) at a scale of 1:12,000.

2. Air photo prints (1978) black and white in 9" x 9" format.

3. Contour maps (1 metre interval) of the study showing all improvements, etc.

4. The following Meewasin study reports which contain a variety of information which may be pertinent to the study:

a) storm sewers outfall study;
b) report on the ecology and biology of the South Saskatchewan River;
c) South Saskatchewan River Use Study; and
d) The Meewasin Valley River Resources Baseline Data Study.
e) all existing plans and concepts for development of the river and riverbanks in the Meewasin Valley.

APPENDIX I

Specified fill construction projects for consideration in Phase III of the study are shown on the attached map and are labelled A and B.

Project A

This fill construction is an extension of the existing "Labatt's" fill site. The purpose of this site is to allow pedestrian and bicycle movement from the existing fill area to Diefenbaker Park. The width of fill needn't be extensive and the area will be planted in such a way as to provide an extension of the natural valley growth. No structures are required other than trail facilities, benches and the like.

Project B

Project B involves the use of fill to improve the sand bar area near the Bessborough Hotel. The area would be an extension of the passive park use nearby but also would have to accommodate some form of beach use during low flow periods in the summer--that is, people would have to be able to get safely to the water's edge for wading and other similar activities. No major facilities would be required other than suitable pedestrian linkage.

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APPENDIX 'B'

APPENDIX B

THEORY OF SLOPE STABILITY ANALYSIS

The most widely used method of determination of the factor of safety against slope movement considers the case of limiting equilibrium for an assumed circular or composite slip surface. The factor of safety against movement is defined as the amount by which the strength parameters are reduced in order to bring the soil mass enclosed within the slip surface to a state of limiting equilibrium. The factor of safety is determined by a method of slices, with the enclosed soil mass divided into a series of vertical slices. The slices above the slip surfaces are perceived as individual free bodies while the soil beneath is considered perfectly rigid.

The analysis assumes that:

1. Soil behaves as a Mohr/Coulomb material, such that the effective shear strength is defined as:

$$s = c' + (\sigma_n - u) \tan \phi' \quad ..1$$

where:
s = shear strength,
c' = effective cohesion,
ϕ' = effective angle of internal friction,
σ_n = total normal stress, and
u = pore water pressure.

2. The factor of safety for the cohesive component and the frictional component of strength are equal for all soils involved, and

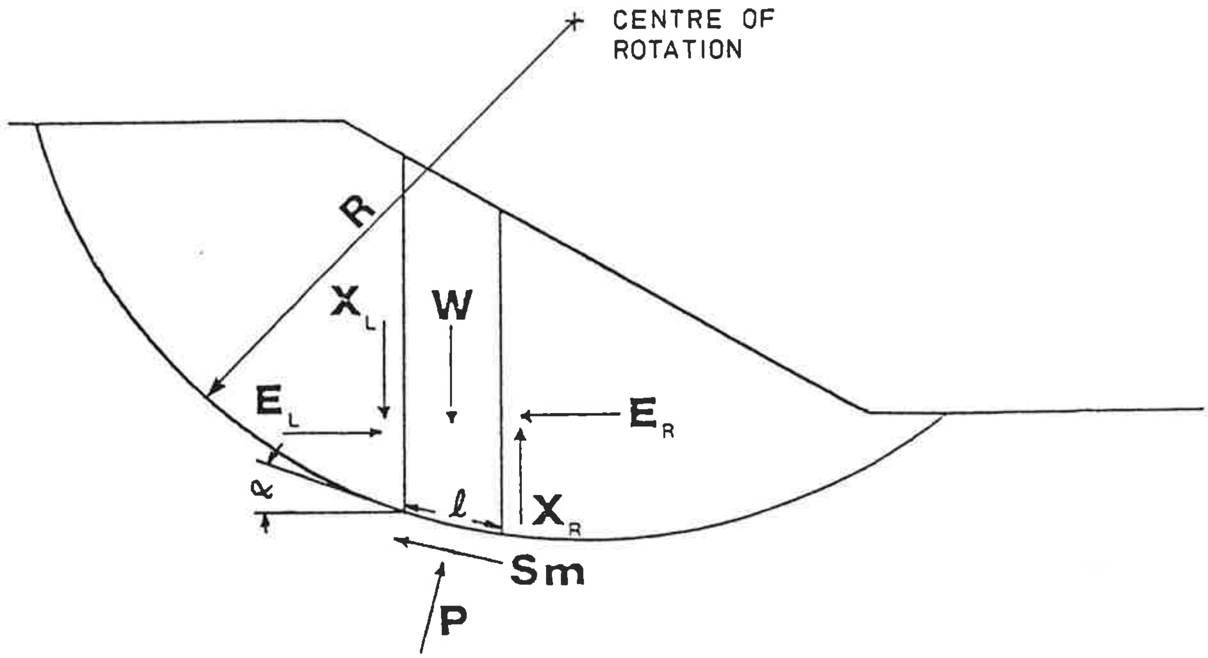
3. the factor of safety is the same for all slices.

Figure B1 illustrates and defines the forces acting on each slice of each circular slip surface. The magnitude of the shear force mobilize, S_m, required to satisfy the conditions of limiting equilibrium is:

$$S_m = \frac{s\,b}{F} \qquad ..2$$

where: s = shear strength,
b = length of the base of the slice, and
F = factor of safety.

The factor of safety is computed for moment or force equilibrium or both, by considering the variables shown on Figure B1. For a composite slip surface, eccentricities are introduced for the weight component, W and the normal force, P. A summary of the parameters that are known and unknown for the solution of the factor of safety are summarized on Table B1. Because the numbers of unknowns are greater than the number of equations available to solve the problem, the problem is indeterminate. Therefore, assumptions must be made regarding the directions, magnitude or point of application of some of the forces in order to render the analysis determinate. The names of some of the methods used to solve the factor of safety and the assumptions for their derivation are summarized on Table B2. Most methods first assume that the point of application of the normal force at the base of the slice acts through the centerline of this slice. Secondly, an



W = Total Weight of Slice

P = Total Normal Force Acting at The Base of Slice

Sm = Shear Force Mobilized at The Base of Slice

E_L, E_R = Total Interslice Normal Forces

X_L, X_R = Total Interslice Shear Forces

FIGURE B1 FORCES ACTING ON A SLICE

TABLE B1

ASSUMPTIONS USED IN VARIOUS LIMIT EQUILIBRIUM METHODS

(from Fredlund 1985)

METHOD	ASSUMPTION
Ordinary or Fellenius	Interslice forces are neglected
Bishop's Simplified	Resultant interslice forces are horizontal (i.e., there are no inter-slice shear forces).
Janbu's Simplified	Resultant interslice forces are horizontal. An empirical correction factor, f_o, is used to account for interslice shear forces.
Janbu's Generalized	Location of the interslice normal force is defined by an assumed line of thrust.
Spencer	Resultant interslice forces are of constant slope throughout the sliding mass.
Morgenstern-Price	Direction of the resultant interslice forces is determined using an arbitrary function. The percentage of the function, λ, required to satisfy moment and force equilibrium is computed.
GLE	Direction of the resultant interslice forces is defined using an arbitrary function. The percentage of the function, λ, required to satisfy moment and force equilibrium, is computed.
Corps of Engineers	Direction of the resultant interslice force is i) equal to the average slope from the beginning to the end of the slip surface or ii) parallel to the ground surface.
Lowe - Karafiath	Direction of the resultant interslice force is equal to the average of the ground surface and the slope at the base of each slice.

TABLE B2

SUMMARY OF KNOWNS AND UNKNOWNS IN SOLVING FOR THE FACTOR OF SAFETY

(from Fredlund 1985)

NUMBER OF KNOWNS	DESCRIPTION
n	Summation of forces in the horizontal direction
n	Summation of forces in the vertical direction
n	Summation of moments
n	Mohr–Coulomb Failure Criterion
4n	Total number of equations

NUMBER OF UNKNOWNS	DESCRIPTION
n	Magnitude of the normal force at the base of a slice, N
n	Point of application of the normal force at the base of each slice
n − 1	Magnitude of the normal force at the interface between slices, E
n − 1	Point of application of the normal force at the interface between slices,
n − 1	Magnitude of the shear force at the interface between slices, X
n	Shear force on the base of each slice, S_m
1	Factor of safety, F
1	Value of Lambda, λ
6n − 1	Total number of unknowns

assumption is made concerning the magnitude, direction or point of application of the interslice forces.

A commonly used method of analysis is the Bishop Simplified method. This method assumes that the resultant interslice forces act horizontally. The normal force at the base of each slice is determined by summing the forces vertically. For equilibrium of the overall slope, the analysis considers the summation of moments about a point, usually the center of rotation.

Consideration of the forces in a vertical direction yields an equation for the normal force on the base of each slice:

$$W - P\cos\alpha - S_m \sin\alpha = 0 \qquad ..3$$

The summation of moments about the center of rotation yields:

$$\Sigma Wx - \Sigma S_m R - \Sigma Pf = 0 \qquad ..4$$

where: x = moment arm of W, and
f = moment arm of P for a composite slip surface.

For a composite slip surface, the total normal force acting at the base of the slice, P, has a moment arm, f=0. Thus, the term involving P drops out of equation 4.

Equation 3 may be solved for the normal force, P by substituting equation 1 and 2 into equation 3. This yields the following equation:

$$P = \frac{W - (c' b \sin\alpha)/F + (u\, b \sin\alpha \tan\emptyset')/F}{m_\alpha} \qquad ..5$$

where: $m_\alpha = \cos\alpha + \sin\alpha \tan\emptyset' / F$

Substituting equation 5 into equation 4 yields an expression for the factor of safety:

$$F = \frac{\Sigma(c' b R \cos\alpha + (W - u\, b \cos\alpha) R \tan\emptyset') / m_\alpha}{\Sigma W x - \Sigma P f} \qquad ..6$$

The factor of safety equation for the Bishop Simplified method is nonlinear because the factor of safety term appears on both the left and right hand sides of the equation. Thus, iteration is required to determine the factor of safety in the numerical solution.

The General Limit Equilibrium method (GLE) is numerically a more rigorous solution. The analysis is rendered determinate by assuming a function to define the direction of the interslice forces:

$$\frac{X}{E} = \lambda f(x) \qquad ..7$$

where: λ = a constant indicating the portion of the function utilized, and
$f(x)$ = function of the horizontal coordinate.

The function f(x), is referred to as the side force function. The factor of safety equations are derived on the basis of the summation of forces in a horizontal direction as well as the summation of moments. Lambda, can be varied until a value is found where the factors of safety for force and moment are equal. Methods such as the Newton-Raphson numerical technique are available to solve for force and moment equilibrium.

The Morgenstern-Price method is similar to the GLE method in terms of the assumptions and statics used in the derivation. The manner in which the equations are formulated and solved varies slightly. Solutions for the GLE method and the Morgenstern-Price methods vary insignificantly.

Reference:

Fredlund, D.G. 1985. PC-SLOPE User's Manual S-30. GEO-SLOPE Programming Ltd.

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APPENDIX 'C'

APPENDIX C
SUMMARY OF LANDSLIDE EVENTS
SOUTH SASKATCHEWAN RIVER VALLEY
SASKATOON

The landslides listed below are shown on Figure C1.

DIEFENBAKER PARK	Multiple landslides since late 1960's Currently active
QUEEN'S RETREAT	Large Landslide during mid 1970's Currently active

NUTANA COLLEGIATE SLIDE

1957	Report by PFRA regarding soil conditions made with reference to settlements experienced by tennis court, north of Nutana Collegiate
June 1960	Occurrence of slide
July 1960	Further movement noted
August 1960	BBT site investigation regarding approach fill stability at Victoria Bridge.
October 1960	BBT supplemental report recommending slope flattening
July 1961	BBT report recommending earth fill at toe
LONGHILL SLIDE	South of Broadway Bridge (opposite Eastlake Avenue)
Spring 1913	Initial slide movement of curbs and sidewalk
1913	Curbs and sidewalks repaired More movement occurred
November 1913	Plans for reconstruction drawings Construction of sheet piling and installation of pipe drains complete by November 13
February 1914	Plans drawn up for alternate method of reconstruction of slope failure involving removal of installed pipe

	drains and installation of reinforced concrete and brick drainage system
October 1914	Construction of revised drainage system complete
1932	Drainage system extended at time of construction of Broadway Bridge
November 1962	Flow of 2160 gal/day recorded outfall of drainage system

BROADWAY BRIDGE

August 9, 1982	BBT site investigation report - reporting stratigraphy at southeast abutment
September 1962	Occurrence of slide, noted as superficial in nature
December 13, 1962	BBT site investigation report outlining stratigraphy and recommending slope drainage
December 14, 1962	Letter - BBT - supplemental to report recommending slope indicators
January 16, 1963	Correspondence with C.J. McKenzie outlining structural distress at southeast abutment of Broadway Bridge
April 3, 1963	Installation of SI south of bridge
April 3, 1963	Installation of SI north of bridge
May 29, 1963	First indication of movement (13 mm)
October 1964	50 mm movement to date
April 1965	S.I. tubing sheared off
April 29, 1965	Install S.I. #3 next to sheared SI
November 18, 1965	Last correspondence on record from PFRA w.r.t. Broadway Bridge
April 1963 - April 1968	3" movement
April 23, 1968	S.I. #3 sheared off

13th STREET SLIDE

August 1962	Occurrence of slide
January 1963	Work began on installation of drainage system

15th STREET SLIDE

Spring 1950	Occurrence of slide
May 1954	Second occurrence of slide
May 1954 - 1955	Drainage system installed to stabilize slide Rip rap placed to protect toe of slope

McCRANEY SLIDE	16th Street
May 1929	Occurrence of slide
1929	Installation of drainage system
1954 - 1955	Installation of galvinated pipe drainage system
1955	Rip rap protection of toe of slope during stabilization of 15th Street slide

17th STREET SLIDE

Spring 1950	Occurrence of slide

18th STREET SLIDE

May 12, 1961	Occurrence of slide
June 1961	Initial site investigation BBT - June 5 - preliminary - June 18 - final
July 1961	Letter from BBT, regarding slope drainage system
September 11, 1961	Report BBT, formal report
1961 - 1962	Installation of drainage system and road construction
November 1962	Flow of 1000 gal/day recorded at outfall of drainage system

PRESIDENT'S RESIDENCE SLIDE

1970 - 1972	Slope movements threatened a portion of the President's Residence Remedial work included reduction of watering and underpinning the structure
RAVINE DRIVE	Shallow slope movements following construction of a road cut

SLOPE INSTABILITY LOCATIONS IN SASKATOON

FIGURE C1

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APPENDIX 'D'

APPENDIX D
FIELD RECONNAISSANCE OF EAST RIVERBANK
03 MAY 1984

The locations of the points numbered below are shown on Drawing S134-D1.

Point 1

Is immediately above the SSE water supply intake directly opposite Queen E Power Plant. An access road has been constructed down to the pump house and has precipitated the landslides in the lacustrine sediments.

Point 2

A shallow seated landslide precipitated by excavation for the access road to the pumphouse. This steep excavation has precipitated shallow seated slides in lacustrine sediments.

Point 3

A three pole SPC structure anchored on a landslide. There is a risk of the poles being toppled if slope movements occur.

Point 4

Steep regressing headscarp in thick lacustrine sediments. The till outcrop appears to be about 3 m above river level and the scarp is slowly regressing by thin shallow side blocks. There is a point where the scarp is overhanging and held in place by the vegetation root system.

Point 5

An area where the lacustrine slopes have been stabilized by flattening. Borrow material has pushed into the river to construct the Queen Elizabeth spur dyke. The till contact rises gradually from the spur dyke downstream towards the CNR bridge and the lacustrine sediment are slowly bulging over the toe of the lower till. Although the slope is


FLUVIAL LACUSTRINE PLAIN
FLOOD PLAIN
RIVER TERRACE
FLUVIAL LACUSTRINE PLAIN
FLUVIAL LACUSTRINE PLAIN
RIVER TERRACE
LACUSTRINE PLAIN
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38

LEGEND:

OBSERVATION POINT SEE TEXT (APPENDIX D) FOR DESCRIPTION 3

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CLIENT
MEEWASIN VALLEY AUTHORITY

PROJECT
SLOPE INSTABILITY STUDY

TITLE
AIR PHOTO MOSAIC, LANDSLIDE LOCATIONS

DATE	84/04/02	APPD BY	G.W.W.	DWG NO	S134 - D1
SCALE	1 : 20,000	DWN BY	J.M.B.		

relatively flat there is some active movement on the lower slopes. All of the slope movement from here to beyond Point 6 is on top of the Floral till.

Point 6

The current returns to undercut the south bank at this location. Further west, the current is deflected away from the south bank by the Queen Elizabeth Spur and the SSE water intake.

Point 7

A small landslide at a point where an access road had been constructed down to an old foundation located at about the top of the till immediately upstream from the CNR bridge. The old concrete foundation has been broken up but there is not much evidence landslide movement. However, small shallow seated movements occur on the slope above it.

Point 8

The south abutment of the CNR bridge has been stabilized by flattening the upper slopes in lacustrine clay. This has been relatively successful since the slopes have revegetated and are showing no further sign of movement. The lacustrine slopes immediately downstream of the bridge are very hummocky and extremely susceptible to erosion.

Point 9

The upper lacustrine slopes in Diefenbaker Park have slid out on a bench formed by the underlying till. The park benches and picnic tables have been constructed on this bench giving a relatively scenic setting. The lower slopes are being undercut by the river, on the outside of the bend, but are very resistant to erosion.

Point 10

Culvert on the corner of the road to Diefenbaker Park is discharging into a small ravine. Where the ravine has downcut below the water table, seepage is outcropping through the lake sediments and a glacier has formed in the bottom of the ravine. The clay sediments are slumping into the ravine and a separate very active landslide has developed immediately river ward of the head of the ravine. All of this area, on the outside point of the bend, is exremely active and lake sediments are slowly overthrusting at the toe. There is continuous seepage along the toe of the slope as well. This area would benefit from drainage which would probably stabilize the landslides.

Point 11

Heavy seepage along the toe. The slope is very active with a prominent overthrust at the shoreline.

Point 12

Very steep clay scarp extending down to a pathway which is on a bench of exposed till. This bank is extremely steep and is probably prone to a major landslide movement.

Point 13

This site is a very active wet landslide that appears to have turned into a flow slide once movement initiated. It has the typical appearance of all of the area along Diefenbaker Park with a steep rear scarp and a bench at midslope height. The water table is very high with cattails and willows growing in the rear scarp. There is major SPC tower immediately behind the rear scarp of the slide.

Point 14

Site #14 is immediately north of the old cemetery along St. Henry Avenue. This is a series of large landslide in deep clay sediments. The rear scarps create a scalloped effect the clay is very thick the rear scarp is very steep and is slowly regressing in a series of small blocks a few metres wide. The lower part of the slope from the end of the present river fill to the cemetery is extremely active probably at about the point where the groundwater table outcrops.

Point 15

Located at the outside of the bend opposite the intersection of Hilliard Street and St. Henry Avenue. The curb and guard rail at this location have required constant shimming and have gone out of line. The slope has been flattened below this point but further work is needed to stabilize it. Active tension cracks are visible behind the guard rail.

Point 16

Immediately below the power line tower across the street from the Valley View and Sundown Appartments. The groundwater is outcropping halfway down the slope. Plush growths of phreatophytic vegetation, reed grass and chokecherries mark the seepage outcrop. This point is approximately 100 feet south along St. Henry Avenue from the intersection of St. Henry and Isabella Street.

Point 17

Located at the intersection of Taylor Street and St. Henry. An SPC tower sitting on point of land at an intersection where the outside of the curve, curb and guard rail are experiencing a lot of distress. Slope indicators should be installed immediately downslope of the tower.

Point 18

Located to the rear of Queen's House of Retreats. Movement continues to be very active. The slope appears to be somewhat oversteepened over the sewer outfall between Point

17 and 18 and stability would be improved if the slope were flattened and drainage installed.

Point 19

Is on a bench at midslope height immediately at the northerly edge of the Queen's House of Retreats property. This is of interest because sliding has started at the riverward edge of the bench. Thus, the slide has not been stabilized by placement of the river fill and a further look is needed to locate where the base of the clay is. There are two piezometers on the Queens House property one on the lower bench and one on the upper lawn plus a third one at the northerly edge of the property on that middle bench all should be read and water levels recorded.

Point 20

Located at the building immediately south of Elks' Hall. Fill, probably from development of the lot, has been pushed over the edge and the rive ward bank is very steep. The parking barricade at the rear of the lot has settled and gone out of line. It can be expected to slide in the future and would be an ideal spot for instrumentation.

Point 21

To the rear of the Elks' Hall active landsliding is occuring along that bank. Active tension cracks extend well onto the Elks' Club property. This is a good candidate site for a slope indicator since it is accessible to a drill.

Point 22

This is the fill and slope bordering the Labatt's property. A considerable amount of fill has been pushed over the edge of the scarp over the years but little deformation appears to be taking place. The clay scarp is very steep (probably about 1 1/2:1) and needs to be flattened for long-term stability. Immediately north of the brewery, the water table appears to be higher with free water evident at the top of the river fill at the toe of the slope. At this location, the slope is very undulating with evidence of shallow seated slumping on the wet slope. The vegetation in this area is very bent and distorted, apparently being affected by very active movements on the slope. To preserve this slope, the ponds in the gardens of the Labatt's property should be checked for leakage.

Point 23

Is near the intersection of 10th Street East and McPherson Avenue. The scarp of on ancient landslide runs diagonally from near the Labatt's brewery and crosses McPherson avenue diagonally from about 714 to about 707 McPherson then heads diagonally across 11th Street crossing at about 215 11th Street. Once across 11th Avenue it raps around behind the houses on the north side of 11th Street and runs parallel to Cherry Lane. Cherry Lane exits on to 11th Street about 1/4 block west of Victoria. The first house west of the lane (315) is probably not affected however the next houses stretching from about 313, 309, 307, 305, 303, and 241 are sitting over the scarp. 237 is severely affected and has severe cracking near the rear of the house. The affected houses probably end about 231 which is just west of the intersection with Melrose Avenue. The house at 237 and 241 have pushed a lot of fill over their backyard and the yards are extremely steep. This and the next two houses to the east are extremely susceptible to sliding.

Point 25

Nutana Collegiate. A small slide developed last year developed on the lower part of the slope. It is apparently quite shallow seated and fed by runoff which has since been diverted.

Point 26

South abutment Broadway Bridge. Very active sliding on the upstream side above the pathway scarp appears to cross the road towards a break in the high retaining wall near the intersection with Eastlake Avenue. This is probably the site of the old Long Hill slide which was drained by a masonary drain. There are paving stones showing through the pavement at the intersection of East Lake and Saskatchewan Crescent.

Point 27

Is halfway between 14th and 15th Street in front of 720 Saskatchewan Crescent. The curb has deflected riverward indicating slope movements where the scarp comes very close to the edge.

Point 28

Is at the intersection of 15th and Saskatchewan Crescent. This is the site of the old 15th Street slide which has since been stabilized. However, evidence of slope movement can be seen in the pavement and the trace of the scarp can be identified on the slope. Slight bulging of the toe of the slope can be seen.

Point 29

Point 29 is directly opposite 16th Street where cracking has developed in the asphalt at the location of the old scarp. This is typical of the problems that can be expected when rigid structures are placed across the scarp interface.

Point 30

Located at the curb of Saskatchewan Crescent in front of 848 Saskatchewan Crescent. Fill with very steep slopes has been placed over the scarp and warping of the curb and deflection of the outer edge of the roadway has occurred. Some slope flattening will be required if this is to remain stable.

Point 31

Is 17th Street and Saskatchewan Crescent. Pronounced deflection of the roadway and curb has taken place in an area of steepened fill. The trace of the false scarp is evident in the patching on the roadway. The roadway has warped downward considerably and it appears that water is piping under the curb and eroding the bank on the downslope side.

Point 32

At the top end of the 25th Street Bridge where the asphalt walk has been placed over very steep fill which is unstable. The edge of the walk has dropped away near its intersection with Saskatchewan Crescent and is cracking badly indicating imminent movement east of that intersection.

Point 33

Is under the 25th Street Bridge. Fill placed for the walkway is starting to slide on oversteepened clay fills. The contact between the lacustrine sediments, Battleford till, the intertill stratified drift and the underlying Floral till is exposed in the cut immediately east of the abutment.

Point 34

Structure across the slide at the rear of the President's residence. Till comes very close to surface immediately east of that location.

Point 35

Devil's Dip. Relatively severe erosion is taking place due to groundwater being discharged from storm sewers entering the gully. Downcutting has left overhanging banks. It appears that most of the water is groundwater probably initiating in the stratified drift unit between the Floral and Battleford tills. A boulder lag exposed in the bottom of the ravine is likely boulder lag at the base of the Battleford till. North of Devil's Dip lacustrine sediments thin and the riverbank steepened being comprised mainly of Floral till.

Point 36

Typical slopes in the Floral till. Some of the slopes are relatively well grassed and apparently stable at a steep angle in other areas shallow seated sloughing occurs.

Point 37

Immediately riverward of St. Pius Seminary where a gully in the till has been filled with loose fill and debris. This is not a desirable way to fill these gullies since movement will probably reinitiate in the unconsolidated material.

Point 38

Gully immediately north of the ski jump. Seepage from stratified drift causing a piping failure in the slope below the road. It is probably aggravated by drainage from irrigation or drainage ditches above it. It is causing fairly severe erosion in the bottom of the gully near the river's edge.

Point 39

Section of riverbank between Weir and 42nd Street bridge. Minor shallow seated sloughing is occurring in till.

Point 40

Section of riverbank immediately north of storm sewer outfall. Minor shallow seated sloughing in Floral till. Some groundwater discharge along the valley wall.

Point 41

Penitentiary piping failure. Slow active piping still occurring. Ground surface at the base of dish is moist to wet.

Point 42

Minor Sloughing and shallow seated sliding in glacial till. A boulder lag exists at surface. Some groundwater discharge along the valley wall.

Point 43 and 44

This is the area of Petursson's Ravine. Heavy groundwater discharge is still occurring at Point 44 and the ground is very wet with free water. A second smaller piping failure is occurring in the gully at Point 43.

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APPENDIX 'E'

APPENDIX E
MULTISTAGE DIRECT SHEAR TESTING

The direct shear test is used to evaluate peak and residual shear strength for a soil. While the test is practical and easy to perform, there has been some reluctance to fully adopt it as readily as the triaxial test since the stress conditions throughout the sample are not known. The soil specimen is forced to shear along a planar surface corresponding to the separation in the direct shear box. Thus, the stress conditions along the failure surface are not known.

The usual procedure for conducting a direct shear test involves the selection of three soil specimens to be failed at three separate normal forces. Peak and residual shear stresses are measured for each sample. When the shear stress values corresponding to the normal stress are plotted, Mohr-Coulomb failure envelopes are constructed. Typical shear strength versus displacement and Mohr-Coulomb envelopes are shown on Figure E1. The shape of the shear strength versus displacement is characteristic of an over-consolidated clay which commonly exists along the failure surface of landslides studied in Southern Saskatchewan.

Some economy may be realized if multistage direct shear testing is undertaken. Multistage testing involves loading a single sample to at least three different normal stresses and measuring the peak or residual shear strength or both at that normal. Advantages of this type of testing are economy and the fact that all testing is done on same sample of soil instead of three different soil specimens that may differ in texture or origin. However, some

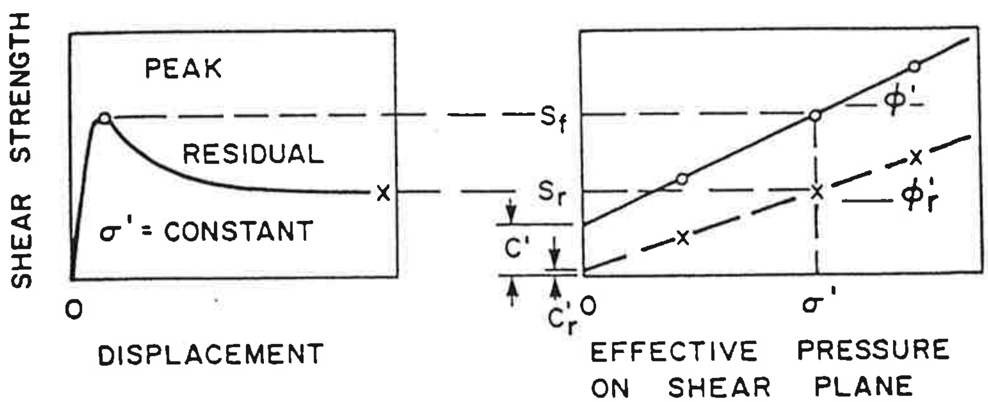

FIGURE E1 SHEAR CHARACTERISTICS OF AN OVER-CONSOLIDATED CLAY (AFTER SKEMPTON, 1964).

comparison should be available with direct shear test results conducted in the traditional matter as confirmation of results. There is some merit in conducting several multistage direct shear tests rather than a single direct shear test using three samples.

Multistage testing to determine peak soil strength parameters using triaxial tests has been documented by Ho (1981) with reference to Kenney and Watson (1961) and Lumb (1964). Although extensive research has not been conducted to determine the accuracy of results, similar techniques may be applied to direct shear testing. After the sample is fully consolidated to the desired normal stress, it is sheared at constant rate of strain. The shear stress versus displacement behavior is monitored as the test progresses in order to determine the maximum peak strength. At this point the shear stress is reduced to zero. It is important to terminate the test before any strain softening occurs which indicates an alteration of the soil fabric and structure at failure. The sample is then consolidated under the next normal stress and the entire process is repeated.

The same soil specimen may be used to evaluate the residual shear strength after the determination of peak shear strength. Under the selected normal stress, the sample is sheared repeatedly until two consecutive cycles of shearing do not vary significantly. Because the residual shear strength is independent of the stress history of the sample, the sample can be loaded or unloaded to another normal stress and tested again. Cullen and Donald (1971) and Chowdhury and Bertoldi (1977) have examined various methods to determine residual shear strength parameters.

If it is not the intent of the testing program to evaluate the peak shear strength parameters and only the residual shear strength parameters, the use of a pre-cut failure plane reduces the testing time required by facilitating the development of a failure plane. Some time is required to consolidate the sample after the failure plane is cut in order to allow the complete mobilization of the shear strength. Because there is no direct evidence to prove the residual strength obtained in this manner are true residual strengths, it is suggested that the residual shear strength values be compared with those obtained by more traditional techniques. In general it has been found that residual shear testing conducted on specimens that have not been pre-cut produce residual shear strength parameters that are greater than those obtained when the specimen is pre-cut. It is speculated that the increased shear strength for specimens not pre-cut is the result of dilation of the sample caused by non-planar failure surfaces.

Because the residual shear strength parameters are dependent only on soil texture and are not dependent on stress history or moisture content, it is possible to utilize remolded samples for the determination of shear strength parameters.

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APPENDIX 'F'

APPENDIX F

PIEZOMETER AND SLOPE INDICATOR CONSTRUCTION DETAILS

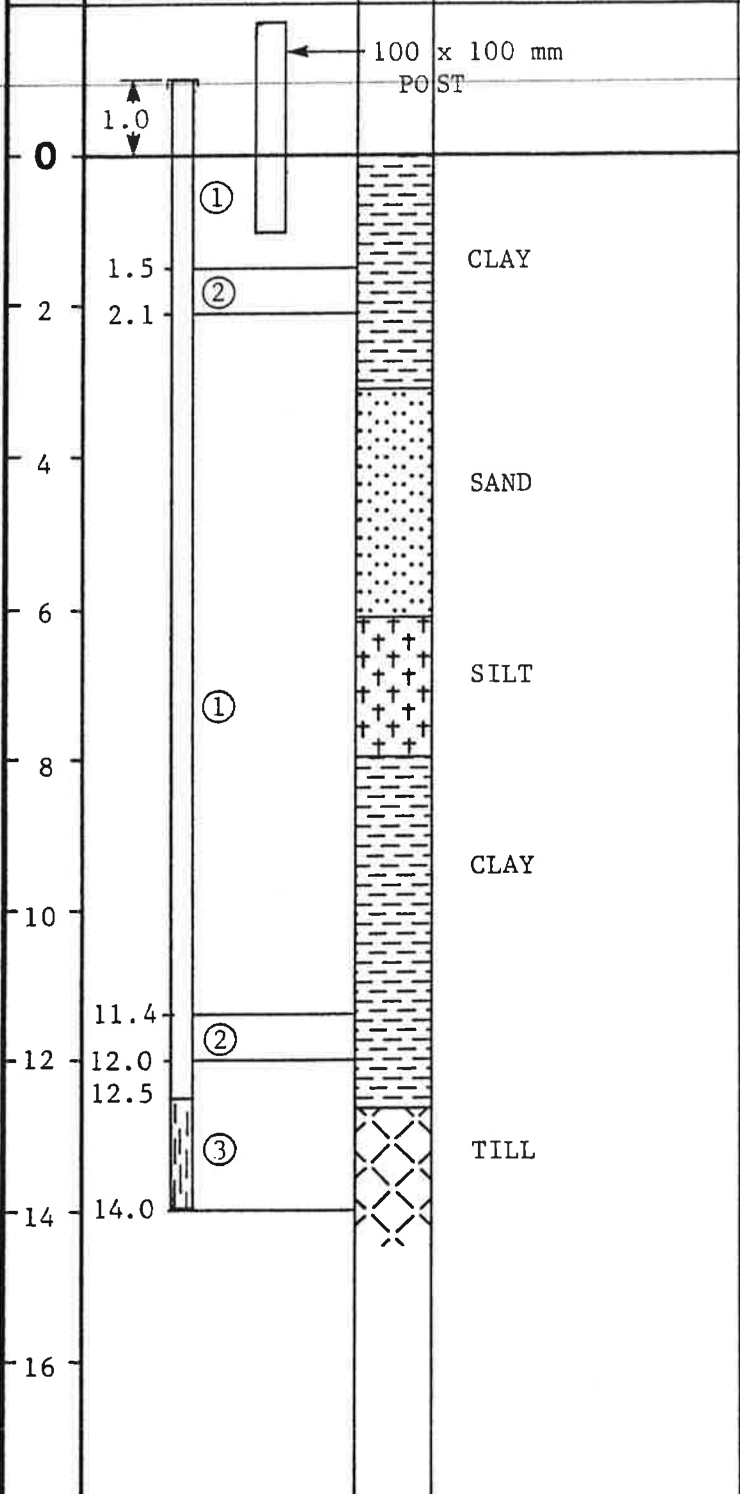
Piezometer Construction

A typical piezometer construction detail is shown in Figure F1. The requirements and procedure for constructing this installation are given below:

1. Piezometers should be constructed using rigid 50 mm diameter PVC pipe. This material is relatively inexpensive and easily handled in the field.
2. The bottom 1.5 m of the standpipe should have 4 rows of staggered vertical slots. The slotted section of pipe must by wrapped in filter cloth to prevent silting and clogging. This forms the piezometer tip. The piezometer tip must have a bottom cap and the standpipe above must be sealed and secure.
3. The piezometer tip should be backfilled with clean frac sand after placement at the correct elevation. If severe sloughing conditions occur within the test hole, hollow stem augers must be used. The sand pack should extend approximately 0.5 m above the Top of the slotted piezometer tip.
4. A bentonite seal should be placed on top of the sand pack. This seal should be at least 0.6 m thick. Solid bentonite pellets are the most suitable construction material.
5. The remainder of the hole may be backfilled with auger cuttings. However, a second bentonite seal 0.6 m thick should be placed 1.5 m below natural ground surface. The piezometer standpipe extending above ground should

PIEZOMETER CONSTRUCTION DETAILS

DEPTH m	PIEZOMETER DETAIL	SYMBOL	SOIL DESCRIPTION	ELEV. m



PIEZOMETER NO. ____
TEST HOLE NO. ____

LOCATION Site Description

TOP PIPE ELEV. Geodetic Datum
GROUND ELEV. Geodetic Datum
BASE SCREEN ELEV. Geodetic Datum
PIPE TYPE 50 mm P.V.C.

SCREEN 50 mm P.V.C. vertical slotted with filter sock
TEST HOLE DIA. 150 mm
INST. DATE 84/05/15
SUPERVISOR ____
CONTRACTOR ____
DRILL Brat 22

WATER LEVELS

DATE	TIME	DEPTH-m	ELEV.-m

REMARKS

Backfill Material:

1. Bore hole cuttings
2. Bentonite seal
3. 10-20 silica sand

DRAWN BY J.M.B. APPROVED BY

Clifton Associates Ltd.
CONSULTING GEOTECHNICAL ENGINEERS
REGINA SASKATOON

CLIENT Meewasin Valley Authority
PROJECT Slope Stability Investigation
LOCATION SASKATOON, Saskatchewan
DATE 84/05/15 PROJECT NO. S134

FIGURE F1

be protected with a fence post embedded 1.0 m below ground surface. The post should be painted bright colors (such as orange and white) and labelled using standard house numbers.

Slope Inclinometer Installations

Two methods of drilling may be utilized for slope indicator installations; dry hole augering or wet hole rotary. The procedure for the installation of slope indicators by either method is given below.

Dry Hole Auger

1. A minimum borehole diameter of 150 mm is required for the installation of a 100 mm slope indicator casing. The placement of backfill will be impossible if the borehole is less than 150 mm in diameter.
2. The borehole and slope indicator casing should extend well into undisturbed material. This will usually be accomplished by advancing the casing 5 m into the fluvial till. All slope indicators should extend at least 5 m below river level.
3. Fine silica sand (20-40 frac) should be used as backfill in the annulus between the casing and borehole. This material must be placed with extreme care to avoid bridging and the formation of voids. Water should be used as an aid for uniform flow and density.
4. Where severe sloughing conditions occur, hollow stem flight augers are required. The casing should be placed inside the hollow stem auger. Fine silica sand

may be poured in the annulus of the casing and auger as the auger is carefully extracted. The sand should be poured at a rate which prevents sloughing around casing.

5. The slope indicator should be protected with a metal culvert and manhole cover. A 400 mm corrugated metal casing which extends 1.0 m below ground is recommended. In addition the metal culvert and cover should be flushed with the natural ground.
6. Initial readings should be obtained 24 hours and 1 week after installation.

Wet Hole Rotary

The wet hole rotary installation is generally the same as the dry hole auger method. However, this method of installation allows a more suitable means of backfill.

1. A cement bentonite grout should be used to grout the casing in place. A water/cement ratio of 3:1 and a bentonite/cement ratio of 1:5 are recommended.
2. The grout should be placed with a tremy placed at the base of the casing. The tremy pipe is gradually withdrawn as the voids fill with grout.
3. The grout should be allowed 24 hours to settle and cure. Additional grout should be added if the grout surface drops below ground surface.
4. Initially readings should be obtained 48 hours and 1 week after installation.

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REGINA SASKATOON

APPENDIX 'G'

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