HAZARDOUS SITES

TECHNICAL GUIDE

Ministry of Natural Resources

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HAZARDOUS SITES TECHNICAL GUIDE

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1.0 INTRODUCTION

Increasing pressure to live in hazard suceptible areas has resulted in extensive and increasing property damage, risks to public health and safety, detrimental impacts to local ecosystems, and immeasurable social disruption costs. For municipalities and governments, it has also meant increasing public liability, escalating public costs related to the installation, maintenance, replacement or upgrade of protection works and measures required to safeguard vulnerable developments, and mounting public costs to address unwise individual decisions.

Describing unstable soils (e.g., sensitive marine clays (leda), organic soils) and unstable bedrock (e.g., karst) as a "problem" or "natural hazard" is actually misleading. They are essentially naturally occurring physical and ecological processes that for thousands of years have shaped and re-shaped the landscape. These natural processes are only perceived to be a problem or a "hazard" when people and structures are located within the area(s) directly impacted by these natural processes.

The purpose of this Technical Guide is to provide technical support in identifying areas of *hazardous sites* inclduing:

- sensitive marine clays (unstable soils)
- organic soils (unstable soils)
- karst bedrock (unstable bedrock)

The approaches outlined in this Technical Guide reflect current engineering, geotechnical and scientific practices and principles that have been developed, tested and accepted within Ontario and internationally. For ease of use, the approaches have been structured in such a way that they can be applied directly to an area impacted by natural hazards, without the need of additional studies. For some types of natural hazards (e.g., organic soils, karst), the magnitude or severity of the hazard(s) may be localized and unique to the local area. Determining a single approach to be applied in all situations is more difficult. As such, studies may be essential to determine an appropriate approach to address the natural hazard within the local area while at the same time adhering to the provincial policy interest of minimizing risks to life and property.

To provide clarification and direction on how each of these natural hazards should be addressed, the document is divided into five sections as follows:

- **Section 1** introduces the three key types of *hazardous sites*;

- **Section 2** outlines the definition of *hazardous sites* and their relationship to the *Provincial Policy Statement* (May 1996);

- **Section 3** provides an indepth analysis of sensitive marine clays (first of the three types of *hazardous sites)* including the gelogical setting and soil chararcteristics, the type of hazard involved, how to identify the area of provincial interest through site and field investigation, and how to address the hazard in an environmentally sound manner. This section is then supported by a number of appendices;

- **Section 4** provides an indepth analysis of organic soils (second of the three types of *hazardous sites*) including the geological setting, the type of hazard involved, how to identify the area of provincial interest through site and field investigation, and how to address the hazard in an environmentally sound manner. This section is then supported by a number of appendices; and

- **Section 5** provides an indepth analysis of karst bedrock (third of the three types of *hazardous sites*) including the geological setting, the type of hazard involved, how to identify the area of provincial interest through site and field investigation, and how to address the hazard in an environmentally sound manner. This section is then supported by appendices.

2.0 DEFINITION OF HAZARDOUS SITES AND PROVINCIAL POLICY

The *Provincial Policy Statement* is based on three guiding principles:

Ontario's long-term economic prosperity, environmental health and social well-being depend on:

1. **managing change and promoting efficient, cost-effective devlopment and land use patterns which stimulate economic growth and protect the environment and public health;**
2. **protecting resources for their economic use and/or environmental benefits; and**
3. **reducing the potential for public cost or risk to Ontario's residents by directing development away from areas where there is a risk to public health or safety or property damage.**

In addition, Policy 1.1 states that to develop strong communities as part of efficient, cost-effective development and land use patterns:

(f) Development and land use patterns which cause environmental or public health and safety concerns will be avoided.

Development located within *hazardous sites* places the health and safety of area residents and their properties at risk. This has been demonstrated by property damages, land losses and increased risks to life where developments have been located within *hazardous sites* within the province.

In other words, it is more cost effective to avoid the hazards upfront and early in the planning process than it is to deal with the long-term, and often escalating costs of developing within *hazardous sites* (e.g., property damages, installing and maintaining protection works and measures, developing and implementing emergency preparedness plans and emergency evacuation plans).

For this reason, the primary premise of *Policy 3.1: Public Health and Safety* for *Natural Hazards* is that devlopment is to be directed to locations outside of *hazardous sites* to minimize the risks and costs posed by natural hazards (i.e., unstable soils, unstable bedrock).

In keeping with this overall intent, the policies governing *Public Health and Safety* have been structured in the following manner:

- Policy 3.1.1 defines the "areas of provincial interest" (i.e., *hazardous sites* and *hazardous lands*) and confirms the natural hazards to be considered;

- Policy 3.1.2 confirms that there are certain portions of the "areas of provincial interest", identified in Policy 3.1.1, where the risks and threats to life and property are unacceptable, and as such, *development* and *site alteration* should not be permitted (NOTE: applicable only to certain *hazardous lands* and not applicable to *hazardous sites*); and

- Policy 3.1.3 outlines the conditions whereby *development* and *site alteration* may be permitted within the "areas of provincial interest" (i.e., *hazardous sites*).

The "areas of provincial interest" outlined in Policy 3.1.1 include:

- *hazardous lands* adjacent to the shorelines of the *great Lakes - St. Lawrence River System* and *large inland lakes* which are impacted by *flooding, erosion* and/or*dynamic beach hazards;*

- *hazardous lands* adjacent to *river and stream systems* which are impacted by *flooding* and/or *erosion hazards;* and

- *hazardous sites* including those lands involving unstable soils (e.g., sensitive marine clays (leda), organic soils) and unstable bedrock (e.g., karst formations).

Policy 3.1.3 confirms the conditions that should be fulfilled where *development* and *site alteration* are being considered within areas of *hazardous sites* (and *hazardous lands)*. In specific, Policy 3.1.3 states that:

- **Except as provided in policy 3.1.2, *development* and *site alteration* may be permitted in *hazardous lands* and *hazardous sites*, provided that all of the following can be achieved:**

  a) the hazards can be safely addressed, and the *development* and *site alteration* is carried out in accordance with *established standards and procedures*;
  b) new hazards are not created and existing hazards are not aggravated;
  c) ne adverse environmental impacts will result;
  d) vehicles and people have a way of safely entering and exiting the area during times of flooding, erosion and other emergencies; and
  e) the development does not include *institutional uses* or *essential emergency services* or the disposal, manufacture, treatment or storage of *hazardous substances*.

The purpose of this Technical Guide is to provide additional technical information and to outline suggested approaches to identify and define the areas of provincial interest respecting *hazardous sites*, in accordance with Policy 3.1.1, and suggested approaches, practices and principles to aid in fulfilling the conditions of Policy 3.1.3 where *development* and/or *site alteration* may be considered within these defined areas of provincial interest.

As outlined in the Introduction section, the remainder of this Technical Guide is divided into three separate, comprehensive discussions on the individual natural hazards:

- sensitve marine clays (i.e., unstable soils) in Section 3 and supporting appendices;

- organic soils (i.e., unstable soils) in Section 4 and supporting appendices; and

- karst bedrock (i.e., unstable bedrock) in Section 5 and supporting appendices.

In each of these individual sections and their supporting appendices, guidance on how to apply and adhere to the intent of the Provincial Policy (i.e. Policies 3.1.1, and 3.1.3) is provided.

3.0 SENSITIVE MARINE CLAYS

Sensitive marine clays, commonly known as leda clay, formed as a result of sedimentation processes in the Champlain Sea during the last ice age. Over time, as the Champlain Sea receded, the clay sediments and salt minerals suspended in the Sea settled to the bottom and formed the current underlying and surficial soil structure that dominate significant portions of southeastern Ontario.

Leda clay is a highly unstable soil due to its highly variable internal structure. Consisting of non-uniformed size agglomerates (i.e., matrix of clay size particles) that are relatively porous, they are held together mainly through the electrostatic forces of the clay size particles and saline pore water (Bloom, 1978). Over time, as the soluble salts are removed from the clay structure through leaching, the relative stability of the soil structure becomes thixotropic or “sensitive” to disturbance.

When undisturbed, sensitive marine clays can appear as solid, stable soil structures. However, when these same soils are disturbed with excessive vibration, shock or become water saturated, the pore water moves, the weak framework of poorly arranged clay sediments collapses and the soil structure changes from a solid to a liquid. This change, depending on the level of disturbance, can occur within extremely short periods of time, sometimes in minutes.

Disturbance, in terms of vibrations or chocks, can be triggered by such things as earthquakes, thunder, heavy traffic, blasting or sping break-up of ice in a river. As many of these triggering vibrations or chocks are not predictable the potential instability or movement of an area can occur with little or no warning. Similarly, disturbances caused by water saturation can be triggered by climatic or seasonal conditions (e.g., rainfall, snowmelt, ice break-up, etc.), by excessive loading on a slope (e.g., placement of fill of building) or by toe erosion where the area is exposed to flowing water. Other contributing factors may include changes in slope configuration, changes in piezometric pressures (i.e., groundwater levels) within the slope, changes in drainage conditions and loss of vegetation.

The resultant **retrogressive failures** or **earthflows** are particularly dangerous as they can invoves hectares rather than square metres of land. For example, in Lemieux, Ontario a series of retrogressive failures resulted in approximately 30 hectares (i.e., 75 acres) of land sliding into the South Nation River in 1993. In Saint-Jean_vianney, Quebec a series of retrogressive and expanding slumps in 1971 engulfed numerous homes in a small village and resulted in the loss of 30 human lives.

3.1 Geological Setting and Soil Characteristics

Clay deposits formed in depressions left by the retreating Laurentian Ice Sheet, covered Canada about 14,000 years ago during the Pleistocene period. The different basins which formed at the various stages of ice retreat created conditions for different clay formations. Specifically, thin lacustrine sediments, typically varved clayey silts to clays, were deposited within the glacial Lake Algonquin. The retreat of waters of glacial Lake Agassiz located in north-western Ontario resulted in the deposition of red clay mainly within broad bedrock channels along the northern shores of Lake Superior. During further front recession, the basins of Lake Algonquin and Agassiz were inundated by the Tyrrell Sea. Sediments deposited within the Tyrrell Sea consisted predominantly of clays and silts that coarsen upward into beach deposits of sand and gravel. Lake Barlow-Ojibway then formed when the basins known as Lake Barlow and Lake Ojibway were combined due to glacial front recession. An extensive deposit of lacustrine clay was deposited within the area of Lake Barlow-Ojibway as the ice continued to withdraw into Hudson Bay.

There are known deposits of marine clay in the area surrounding Hudson Bay which were deposited within a saline environment. Up to 60m depth of glaciomarine sediments have been reported east of James Bay (Lee 1960) and deposits with a thickness of more than 7m have been found in the vicinity of the Pivibiska and Missinaibi Rivers as crustal rebound caused regression of the Tyrrell Sea to the current Hudson Bay basin. The possible extent of the Tyrrell Sea deposits, including marine clays, is shown on Figure 3.1.

Some 2,000 years later, the Atlantic Ocean inundated the Ottawa - St. Lawrence lowlands, resulting in the formation of the Champlain Sea. Clay was deposited in this marine and estuarine environment along the Ottawa - St. Lawrence valleys and tributaries. Several different theories have been proposed to explain the mechanisms of the Champlain Sea formation and marine sediment deposition (Geology of Ontario, OGS 1995). One of the mechanisms suggests (Anderson et al. 1985; Anderson 1988; and Rodriques 1988) that during the recession of the Laurentian Ice Sheet to the north-west, glacial lake waters in the Lake Ontario basin could expand into the Ottawa River valley as far upstream as Ottawa. Uncovering of the St. Lawrence River valley caused the water levels in Lake Ontario basin to drop. The isostatically depressed Ottawa, Champlain and upper St. Lawrence river valleys then became inundated by seawater. As a result, glaciolacustrine varves containing freshwater sediment lenses overlying glacial and glaciofluvial deposits occur beneath the glaciomarine sediments of the Champlain Sea.

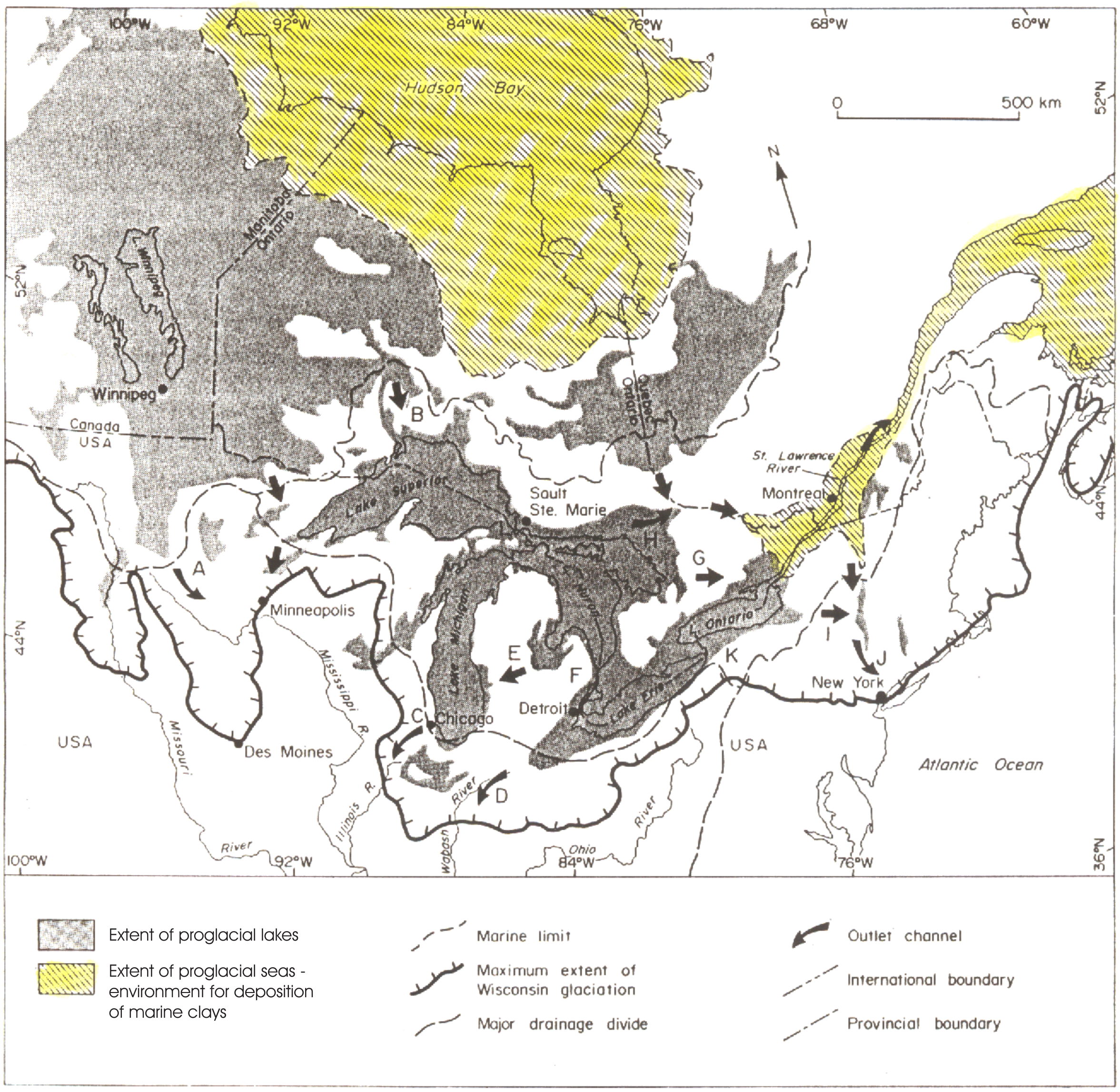
The following general sequence of deposition units of the Champlain Sea sediments can be distinguished (Geology of Ontario 1995):

- Depositional Unit 1 ice - marginal delta and subaqueous fan sand and gravel;
- Depositional Unit 2 laminated silt and clay containing trace fossils, present locally near Unit 1;
- Depositional Unit 3 massive to colour - banded clayey silt to clay containing marine fossils;
- Depositional Unit 4 laminated to varved silts and clays deposited as bottom sets of the large deltas formed during regression of the Champlain Sea; and
- Depositional Unit 5 fossiliferous near shore and beach sands and gravels.

The massive to colour banded clayey silt to clay deposits (Depositional Unit 3) are often referred to as "Leda" or "quick" clays due to their sensitivity to disturbance. These Leda Clays together with the laminated silts and clays of Depositional Unit 4 are very prone to landsliding. Figure 3.2 depicts the approximate aerial extent of marine clays in Ontario.

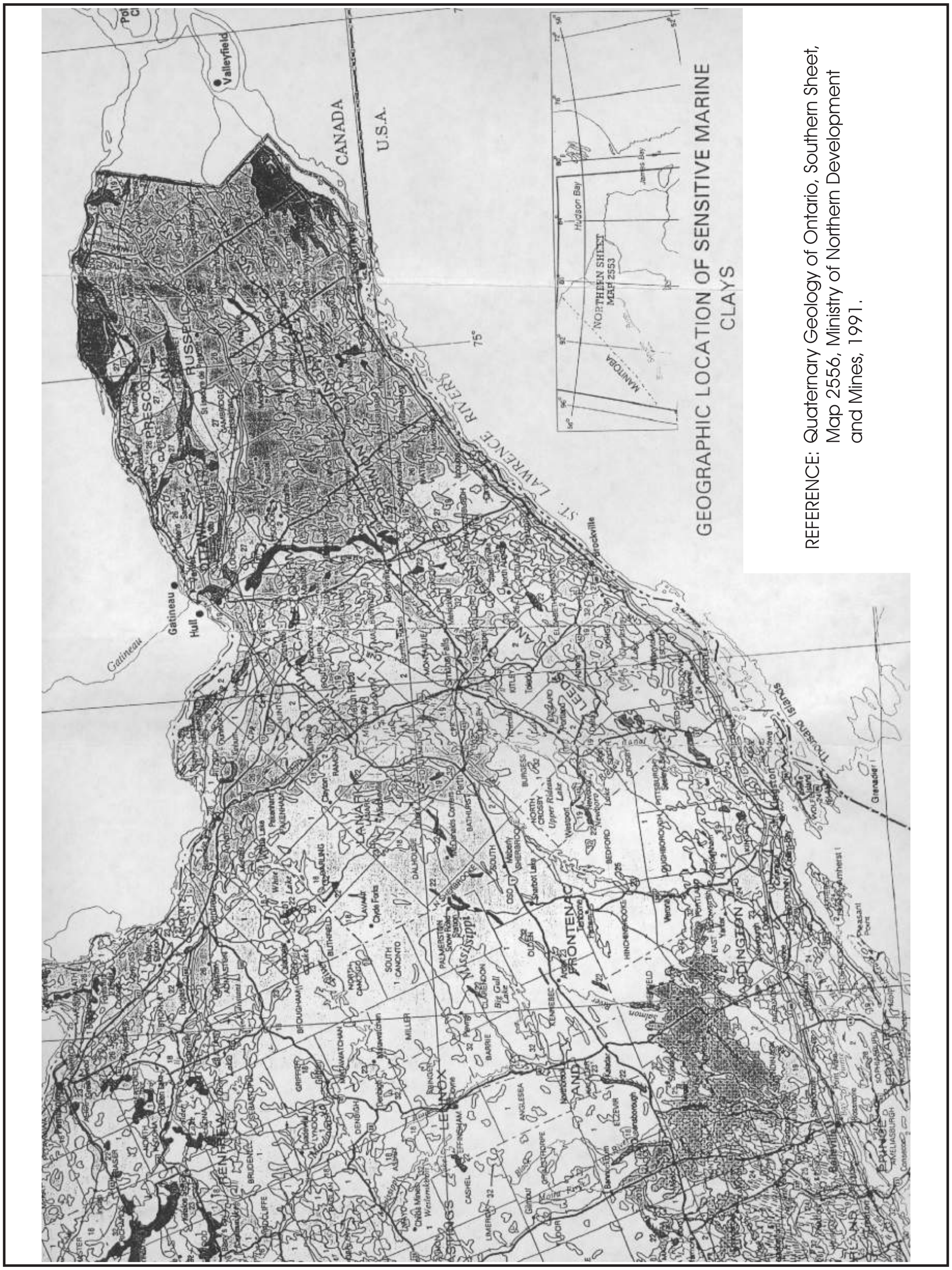
Marine clay deposits have proved to be unstable and prone to landsliding and are also extremely susceptible to settlement due to applied loadings. The marine clay deposits in the Hudson Bay Lowland area are also overlain by thick organic deposits. This thick blanket of organic deposits combined with the occurrence of marine deposits create a hazard for development.

Figure 3.1: Extent of Marine Environment Seas



REFERENCE: Geology of Ontario, Ontario Geological Survey, Special Volume 4, Ministry of Northern Development and Mines, 1995.

Figure 3.2: Geographic Location of Sensitive Marine Clays



3.1.1 Soil Characteristics

The term soil structure refers to the orientation and distribution of particles in a soil mass and the forces between adjacent soil particles. The force component of the soil structure refers primarily to those forces that are generated within the particles themselves (i.e., electrochemical forces). The two extreme types of soil structure are:

- flocculent structure, and
- dispersed structure.

Clay deposits developed from clay particles that have settled out of suspension in a fresh-water or salt-water environment tend to have a flocculent structure (Figure 3.3). In general, the contact and attraction between the clay particles depend on the net effect of the electrochemical forces; namely attractive and repulsive forces. If the net effect of the forces between the two particles is attractive, the two particles tend to move toward each other and become attached (i.e., flocculate). If the net influence is repulsive, they tend to move away (i.e., disperse). A tendency toward flocculation or dispersion varies with an alteration in the characteristics of the soil-water system (e.g., changes in the electrolyte concentration, temperature, pH, etc). The environment of deposition often has an influence on the way particles are arranged during deposition.

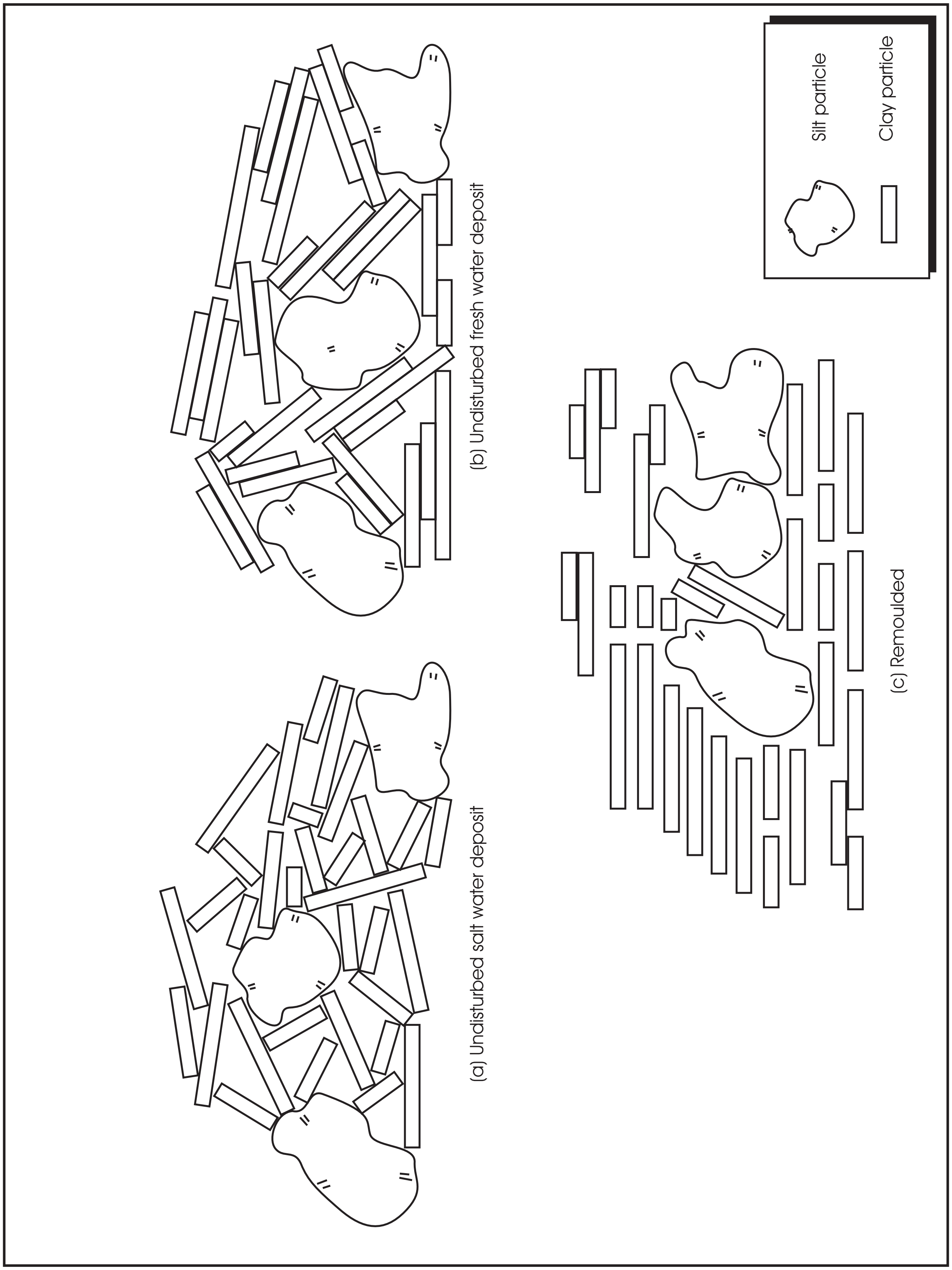
Salt was incorporated within the structure of the marine sediments with the mixing of the saline water during deposition of the clay particles. The salt content and the depositional history of the clays influence the behaviour and the structure of the clay particles. Clays settling out in a salt-water solution tend to have a structure more flocculent than clays settling out in a fresh-water solution (Figure 3.3). Salt-water solution acts as an electrolyte reducing the repulsion between clay particles creating the flocculent structure. Sedimentation in a weaker electrolyte, such as fresh water, produces a structure where some parallel orientation of settled particles occurs, but the overall structure is flocculent.

The engineering behaviour of a soil depends on the existing soil structure. Clay consisting of a flocculent structure has a typically high void ratio and high water content and is relatively strong and resistant to external forces as a result of the attraction between particles. When the clay is subjected to enough disturbance to break the bonds built up by years of confining stress, it loses all of its strength and may become a soil-water slurry with zero shear strength. In addition, gradual reduction of the electrolyte concentration around soil particles can reduce the net attraction forces between them and cause a reduction in shear strength.

The most dramatic reduction in shear strength is exhibited in the marine clays, which due to years of leaching, have most of the electrolyte from the pore water removed. Marine clays tend to be in the dispersed state and for the same water content, they would have very little strength as a result of the reduction in electrolyte. However, this change or loss in strength does not appear fully until the clay is subjected to enough disturbance to break the cementation bonds between particles.

Marine clay deposits encountered in Ontario typically have a stronger upper zone (i.e., surficial crust) formed due to weathering processes. The thickness of the crust typically varies between 1m and 6m. Desiccation and frost action result in fissuring of the upper strata of the clay and in a lowering of the water content. As the water content decreases, the undrained shear strength of the clay forming the crust increases, except where excessive fissuring occurs. The unweathered marine clays are predominantly grey, while the crust is often brown due to processes of oxidation. The brown crust is stiff and extremely brittle; fissures and slikensides can often be observed in this layer. The lowest shear strength of the deposit is typically found within the unweathered zone directly underlying the crust. The strength of the underlying unweathered clay and its sensitivity gradually increase with depth.

Figure 3.3: Structure of Natural Soil



3.2 Type Of Hazard

3.2.1 General

Marine clays are very sensitive to remoulding and exhibit considerable loss of strength whenever their natural structure has been damaged or destroyed. The loss of strength is related to their flocculent structure that collapses when disturbed (e.g., similar to a house of cards). The disturbance to the clay can be induced by:

- changes in loading conditions due to site development;
- construction activities;
- landscaping activities;
- changes in groundwater conditions; and
- slope instability.

3.2.2 Erosion and Slope Instability Processes

Erosion and slope instability are two different processes which are often associated with one another (Figure 3.4). The erosion process affects the soils at the particle level, by dislodging and removing the soil particles from the parent mass. Water movement is often the agent (Figure 3.5) commonly occurring in one of the following manners:

- water flow (e.g., banks or base of river or stream) and waves (e.g., shorelines of lakes, bays);
- internal seepage (e.g., springs) and piping; and
- surface runoff (e.g., sheet, rill or gully erosion).

Other processes such as wind and frost may assist in the weathering or dislodging and transport of soil particles.

Slope failures (i.e., instability) consist of the movement of a large mass of soil (Figure 3.6). Slope movement or instability can occur in many ways but is generally the result of:

- changes in slope configurations, such as steepness or inclination;
- increases in loading on a slope, such as structures or filling near the crest;
- changes in groundwater conditions or drainage of the soil which create higher water levels or water pressures, such as heavy rainfall, blocked drainage (e.g., the placement of fill), or broken watermains;
- loss of vegetation; and
- erosion of the toe of the slope.

a) Water Flow

Flowing water can cause erosion of the surface or exposed face of the bank or channel often resulting in slope instability. Erosion occurring at the toe of the slope acts to steepen the slope and support from the slope is removed by undercutting. The erosion can be a result of increased flow velocities from climatic events such as heavy rains or snowmelt. The magnitude or rate of erosion can be quite variable dependent on the volume, velocity, frequency and duration of the flows through the corridor. Locations of sensitive marine clays, as with many other soil structures, where there are changes in flow directions such as the outside bends of the rivers and channels are particularly susceptible to erosion.

Figure 3.4: Erosion Processes

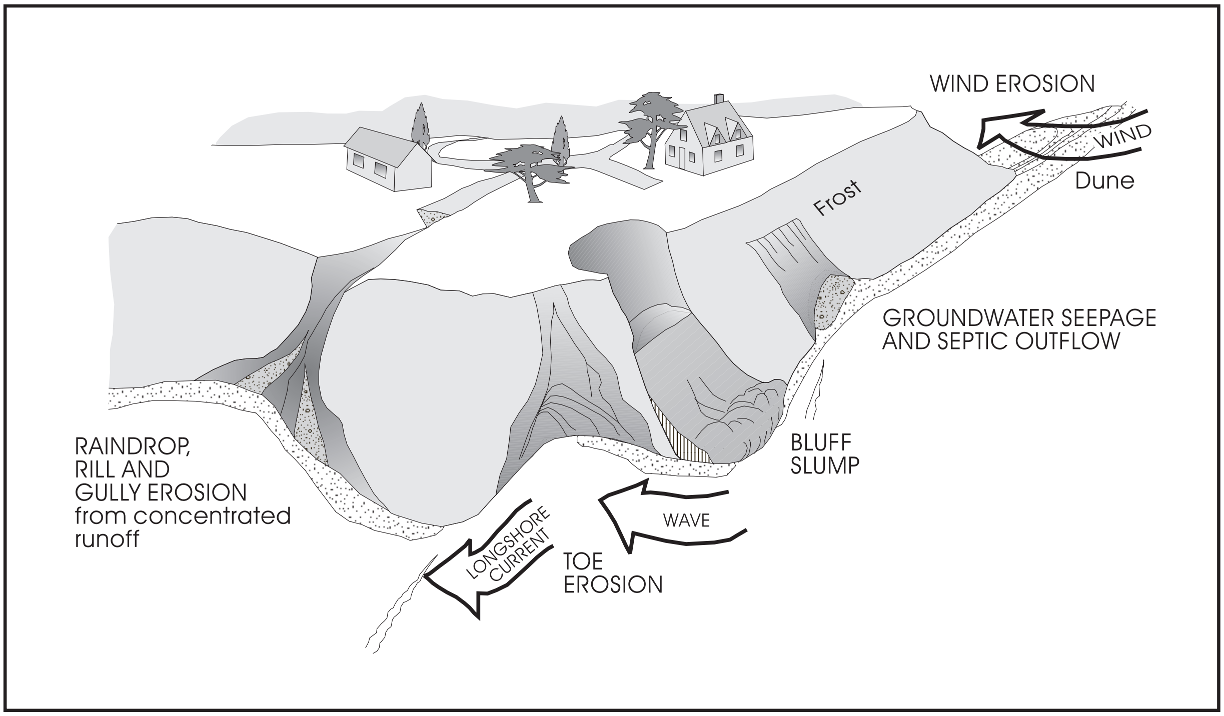


Figure 3.5: Water Action on Soil

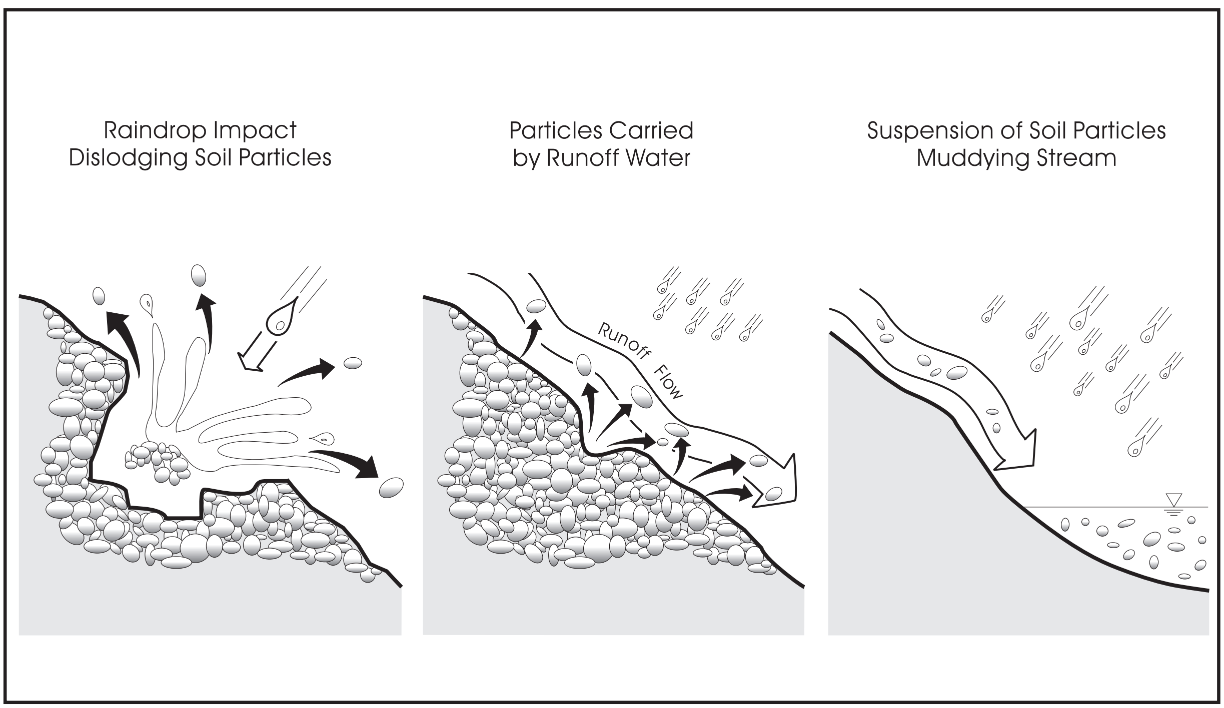
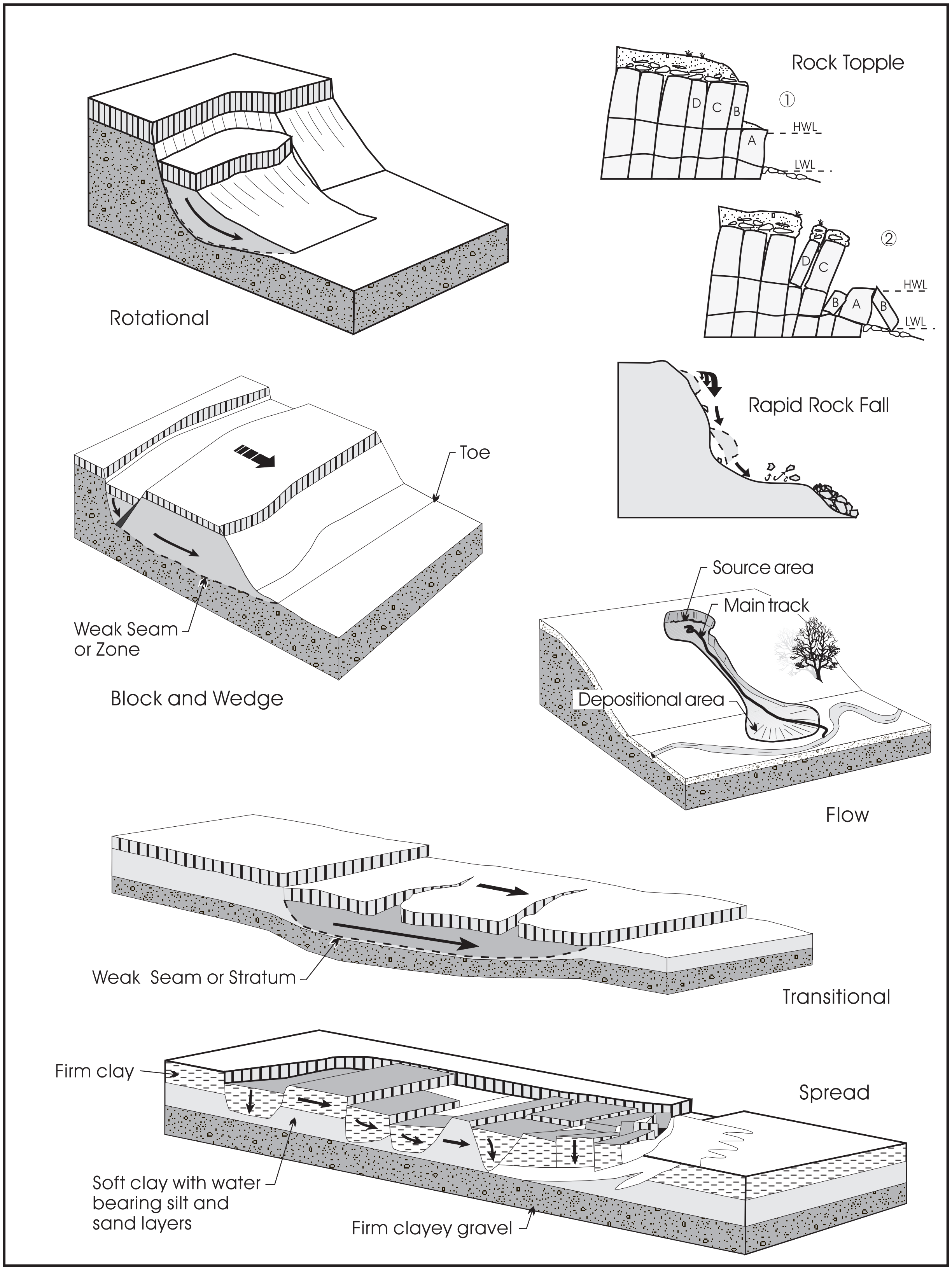


Figure 3.6: Slope Movements



b) Internal Seepage (Groundwater)

Water seepage or groundwater levels can also affect slope stability since they affect the soil strength. Piping on a slope face can be related to springs or seepage where soil erosion occurs in water bearing sands and slopes.

The interaction of groundwater and the soil layers is illustrated in Figure 3.7. Overall, water is generally regarded as the most significant cause or initiator of slope failures. As such, any hydrologic change to either surface or groundwater patterns can pose a direct impact on slope stability, particularly in areas of senistive marine clays. Beyond the lubricant influence of groundwater within a slope, the accumulation of groundwater combined with the natural action of gravity and the overall weight of the groundwater itself will lead to a reduction in slope stability.

c) Surface Runoff

Surface runoff from rainfall or snowmelt can cause soil particles to be broken up and dislodged. The dislodged soil particles can then be transported away by water flowing over the ground surface. Rill erosion can develop when shallow surface flow concentrates in low spots and cuts tiny channels, often only a few inches deep (see Figure 3.8). As flows increase, surface drainage becomes concentrated and erosion goes unchecked, gullies develop. Depending on the level of concentrated flows and erosion, extremely large gullies can develop. Where evidence of gullies exist in sensitive marine clays, the magnitude and size of the gullies, given the unstable nature of these soils, should be carefully monitored.

d) Vegetation

A vegetation cover on a slope is the primary defence against soil erosion and is very important to long-term erosion protection. However, in areas of sensitive marine clays, once the soil structure has become saturated or disturbed such that the soil structure becomes fluid, surface vegetation may provide very little, if any substantial protection or defence against the destructive forces of slope movements.

As indicated in Figure 3.9, vegetation protects against surface erosion and shallow translation slope slides by:

- by holding, binding, or reinforcing the soil with a root system,
- removing water from the soil by uptake and transpiration,
- reducing runoff flow velocity,
- by reducing frost penetration, and/or
- by the buttressing or reinforcing action of large tree roots.

By reducing surface erosion, the likelihood of shallow instability is also decreased.

Slope stability can also be decreased by the removal of stabilizing vegetation. This may be of particular importance where the removal of tree roots, especially the smaller and more numerous tree roots which provide a binding strength for any sedimentary layers they enter, may have been removed. By the cutting of trees in these areas, the naturally cohesive strength and anchoring force may be lost.

e) Human Activities

A number of human activities can aggravate or create slope instability. These include indiscriminate discharge or leakage of water from pools, septic systems, storm runoff control ponds and drains as well as agricultural tile drainage systems. Changes in the topography, by cut-and-fill earth moving or land grading, can also alter the strength of any sedimentary layer or add weight to the entire slope. The construction of buildings or protection works on or near these slopes can further weaken the slope which in turn may contribute to an increased instability of the feature.

Figure 3.7: Erosion by Waves and Water Flow

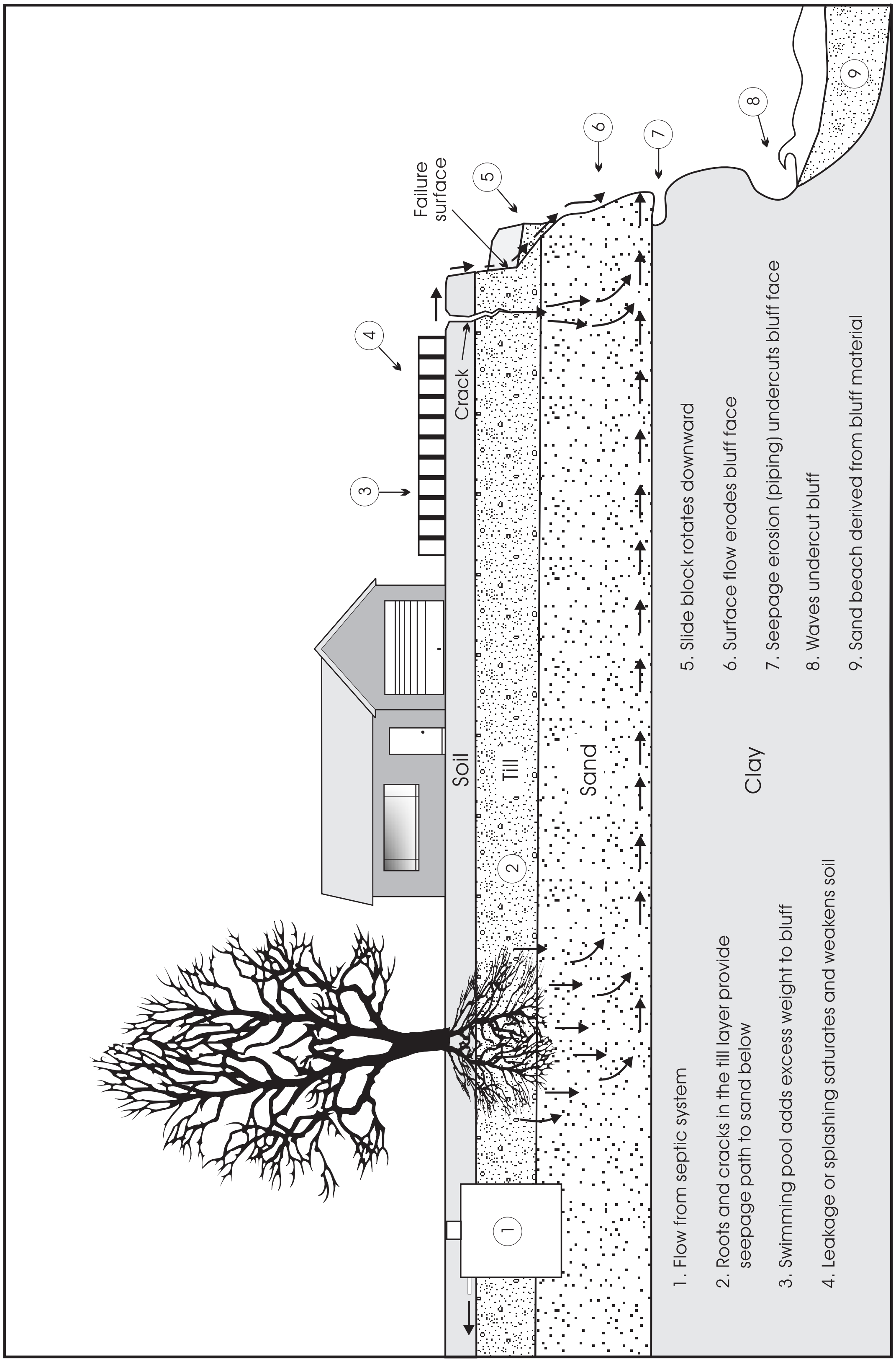
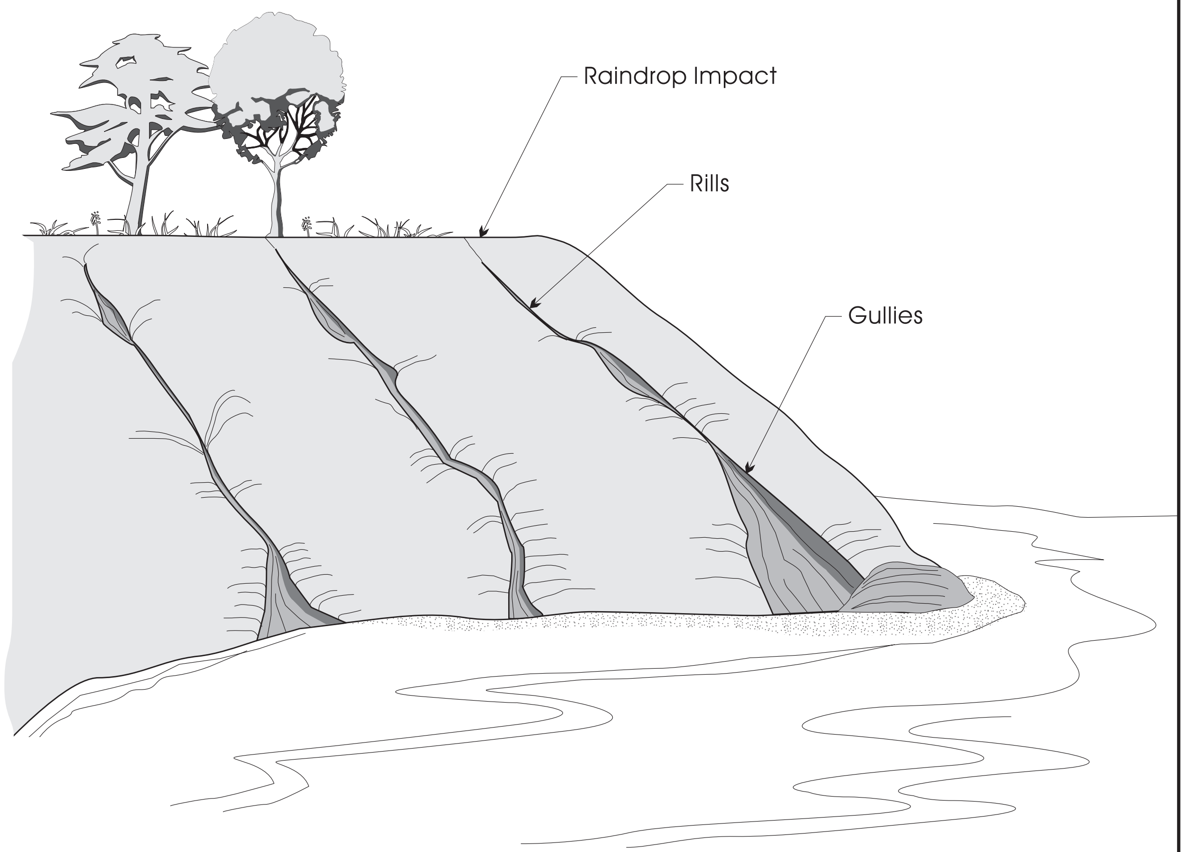


Figure 3.8: Gully Erosion



Surface Runoff

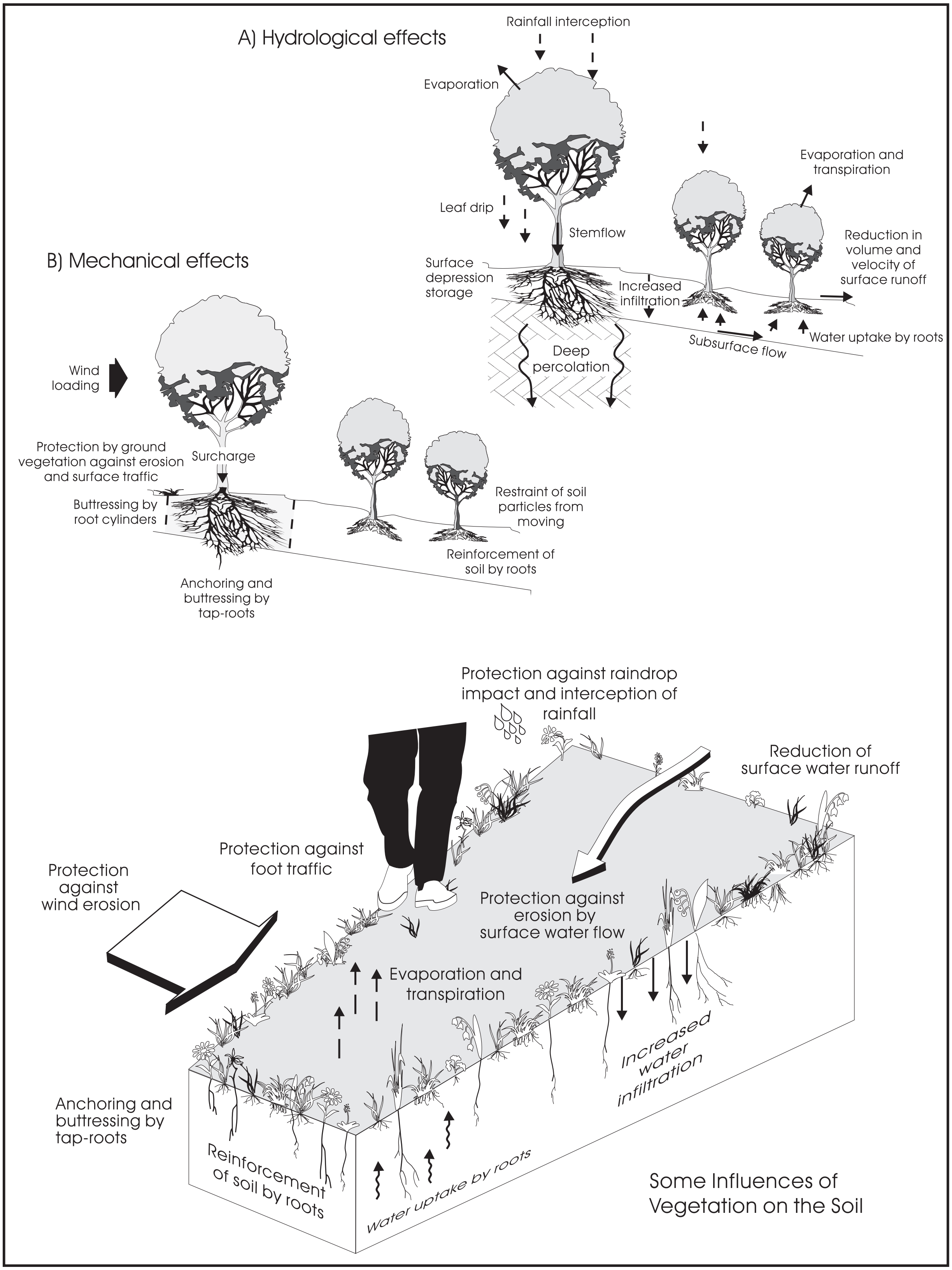
Raindrop Erosion

The splashing of soil from the direct impact of rain on soil.

Rill and Gully Erosion

Occurs when the concentration of runoff cuts grooves or rills into the soil with increased flow, gullies develop.

Figure 3.9: Importance of Vegetation Controlling Surface Erosion



With the activities of urbanization and land development, fill placement near slope crests and excavations into slopes or retaining walls may alter the stability. Filling is a common practice in most urban areas as people try to reclaim more usable flat tableland along existing slope crests. Fill placement often occurs in an uncontrolled manner, sometimes over an extended period of time, and may result in an unstable fill mass which eventually experiences movements. Slides within fill materials, placed randomly and not engineered, can be quite unpredictable and extensive. The resulting instability may occur through the fill materials only or, through both fill and underlying native soil (Figure 3.10).

Filling on slopes can be carried out in a safe and stable manner with suitable control and precautions, and preferably under the responsibility of a qualified geotechnical engineer.

f) Slope Failure or Instability

Slopes are by their very nature are subject to movement(s) and failure, whether it is large and a deep-seated failure or a shallow and local failure. Failures within both of these categories can occur rapidly under adverse conditions causing immediate and disastrous damage to structures within the failure area. It can also take decades to develop to such a magnitude that movements pose a danger to structures located within or near the zone of failure. Permitting development along or in close proximity to slopes must take into account the potential for slope failure.
Given the unpredictable nature of slope failure or instability associated with sensitive marine clays, it is suggested that development should be located outside the *hazardous sites* or defined area of provincial interest whenever possible.

Slopes comprised either in part or wholly of sensitive marine clay are prone to slope failure which in the case of flowslides and earthflows can involve many hectares of land. A large earthflow that occurred in sensitive marine clay on the east bank of the South Nation River at Lemieux, Ontario on June 20, 1993, involved an area of about 30 hectares and retrogressed about 680 metres into the land. Borehole information indicated that a zone of soft, sensitive marine clay was underlying a stiff cap consisting of laminated marine-estuarine sands and deltatic silts and sands. The landslide mechanism involved the fluidization of much of the landslide mass and subsidence, translation and rotation soils forming the cap. The flow most likely occurred as a result of extrusion of the soft sensitive clay layer due to increase in the water table.

3.2.3 Mechanism of Slope Failure

Slope movements are generally classified by a combination of the geometry and the nature of the failure. For details on all of the various slope slide classifications see "Geotechnical Principles for Stable Slopes" (Terraprobe, 1997). For the purpose of this document the four main classes of slope movement are as follows:

- translational or surficial sliding;
- rotational failures;
- retrogressive failures; and,
- flowslides or earthflows.

The mechanism of erosion at the toe of a slope (Figure 3.11A) can initiate any of these types of failure.

Translational failures (Figure 3.11):

- are generally associated with a planar or a non-circular failure surface;
- can occur in granular materials, (i.e., sands and silts), on relatively steep slopes where a thin layer of these soils at the surface of the slope slides over the underlying soils; and
- can occur on slopes composed of any type of soil if the failure surface is influenced by the presence of discontinuities within the slope materials such as bedding planes, faults, weak layers or fissures.

Figure 3.10: Filling on Slopes

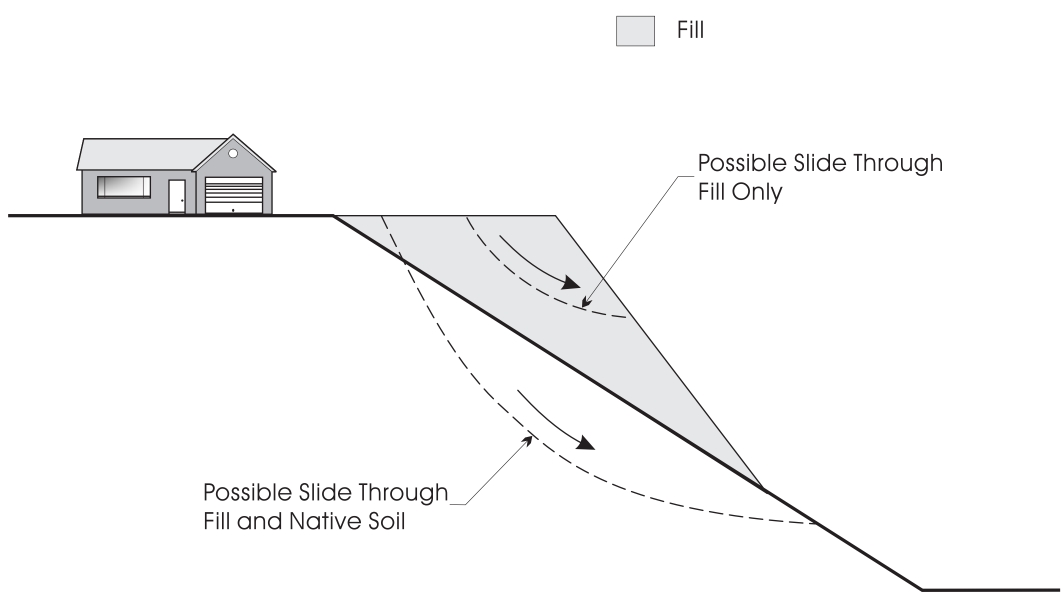
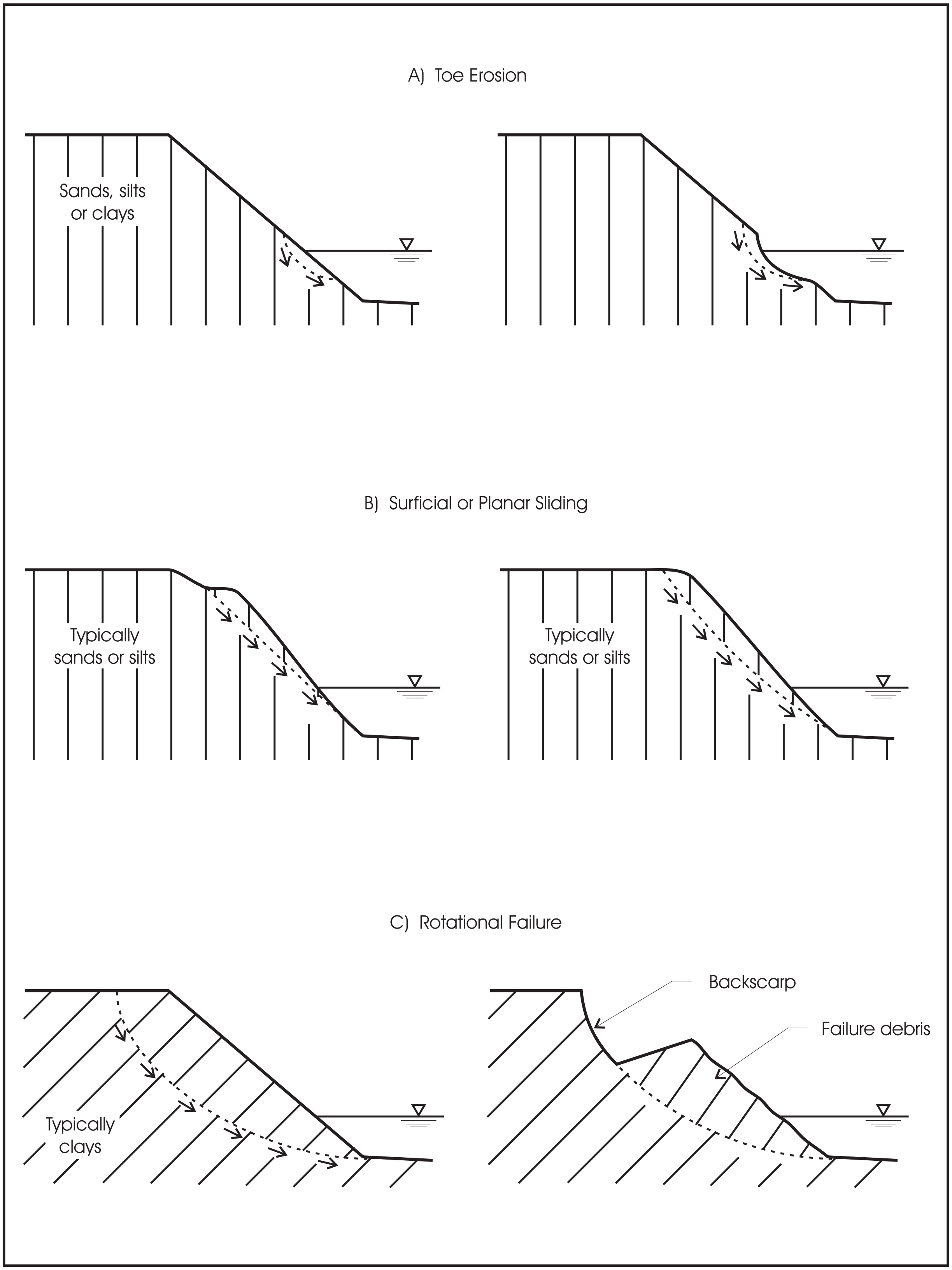


Figure 3.11: Failure Mechanisms



In the last case, the failure surface can be relatively deep depending on the location of the discontinuity. Where the discontinuity extends near the surface of the slope (e.g., the base of a weathered zone or loosened material, or where shallow sliding of the surface layers within granular deposits occurs) the failures are classified as surficial sliding .

Rotational failures (Figure 3.11C):

- generally are found in relatively uniform soil conditions;
- generally involve movement along a curved (i.e., concave upwards) failure plane; leaves an exposed backscarp and the shifted mass rises at the toe of the slope;
- may be shallow or deep depending on the subsurface conditions, soil strength and groundwater conditions; and
- are prevalent in fine grained soils occurring generally in clays and, to some degree, in silty soils.

Retrogressive failures (Figure 3.12):

- may be initiated by a single simple rotational or translational failure, but successive slippages continue to occur within the slope due to a combination of slope geometry and disturbance of the clay soils composing the slope;
- occur in slopes composed wholly or partially of clay;
- are characterized by the presence of remnants of the failed slices, a number of backscarps along the slope face and an apron of failure debris at the toe of the slope; and
- by successive rotational slippages can continue for large distances and the larger of these failures are known as retrogressive flowslides.

Earthflows (Figure 3.13):

- are typically initiated by a single slide at the toe of the slope; initially, they follow the general principal of retrogressive failures, but, movement of the soil mass continues, involving plastic flow of the soils within the slope and the resulting affected area is much larger.

Studies are still ongoing to investigate the principles of the movement involved in earthflows and to develop methods of determining both the environments within which earthflows may occur and the distance of retrogression which is possible for specific site conditions. Tavenas et al (1982) have proposed four criteria which may be used for evaluating the potential for the occurrence of an earthflow. The criteria are based on the geometry of the slope both prior to and during failure and on the soil properties (i.e., strength and ability to remould) as outlined below:

- the slope must be unstable; the slope height and soil strength must be such that an initial failure is possible;
- each successive failure backscarp must be unstable: again a function of slope height and soil strength;
- the slide debris must be able to flow; a combination of low remoulded strength (<1 kPa) and high liquidity index (>1.2); and
- the debris must continue to flow out of the failure scar without causing a blockage: there must be sufficient potential energy within the failing slope mass to cause remoulding and allow outflow of the slide debris. (Tavenas et al, have found this occurs if the liquid limit of the clay is 40 per cent).

Determination of a zone/extent of retrogressive earthflows in the marine clays is of considerable importance for planning development. An empirical method for determining the retrogression distance has been proposed by Mitchell, 1978. Further details on this can be found in Appendix A3.3.

Figure 3.12:   Retrogressive Failure Mechanism

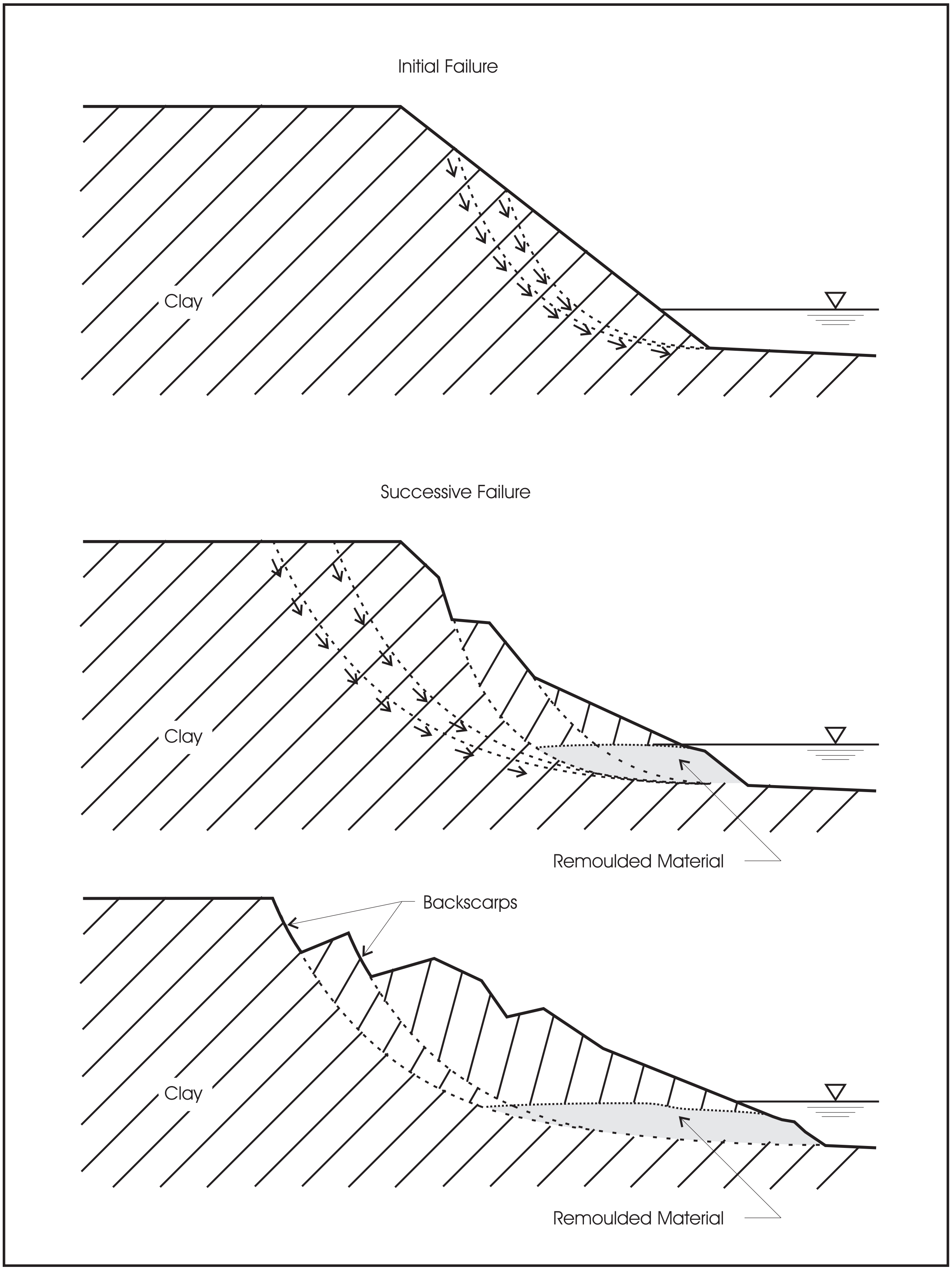
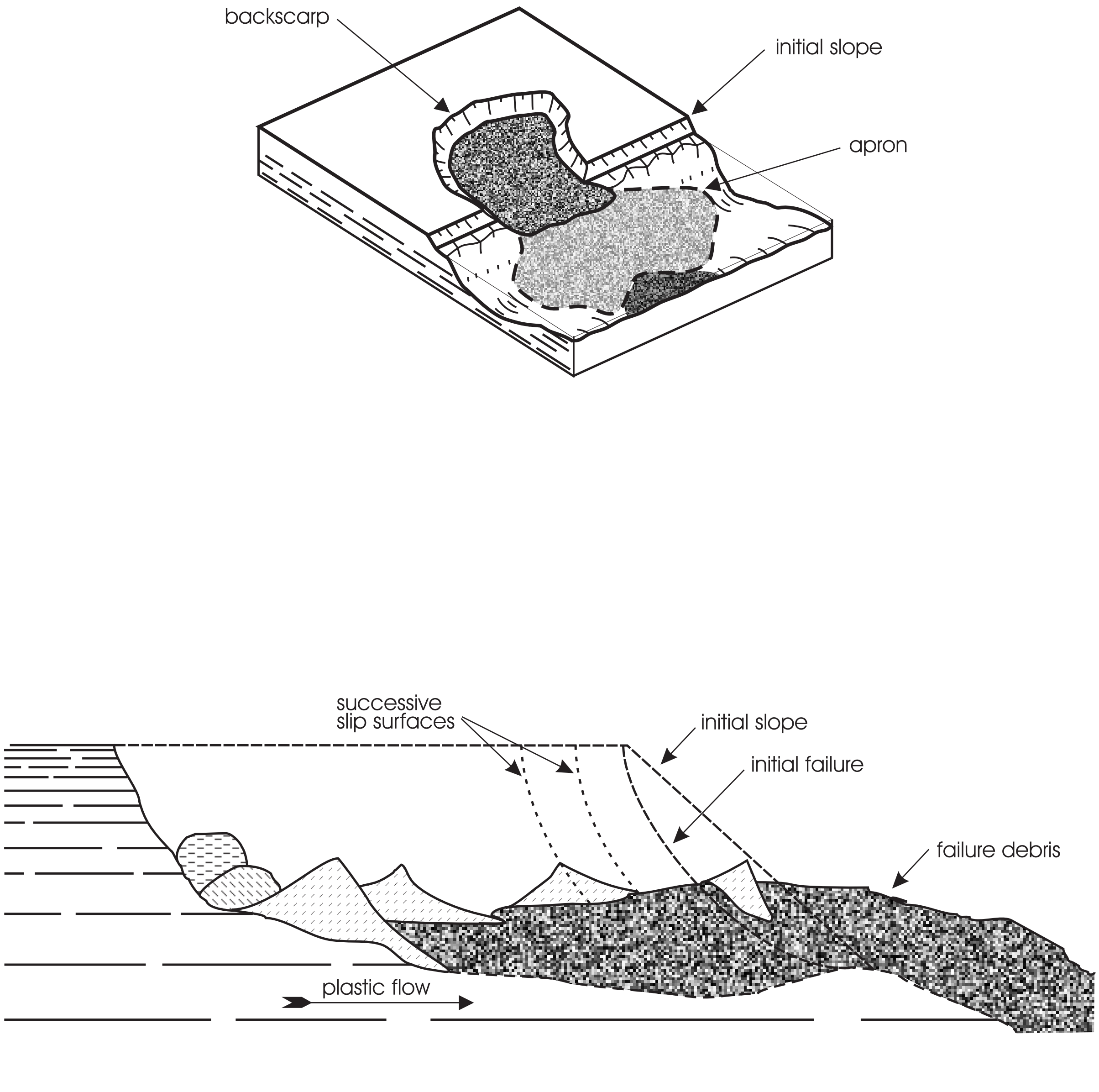


Figure 3.13: Earthflow



3.2.4 Erosion Rate Versus Cyclic Landslide Event

Williams et al (1979) carried out a study of the processes of erosion, landsliding and river bank recession for a portion of the south bank of the Ottawa River extending from Rockcliffe Point in Ottawa to Green Creek in Ottawa East. The study involved an examination of topographical maps and aerial photographs within the study area over the period of time for which the photos and maps were available. Slope profiles for 39 stations along the 6km length of the river bank were obtained by measurements taken from both the photos and maps, covering a combined time period of about 50 years.

The authors found that within the study area, it was possible to determine an average rate of erosion and an approximate rate of return for landslide events. For the portion of the Ottawa River studied, the average linear rate of toe erosion and bank recession was 0.4m per year and the frequency of landslide events was about 1 in every 30 to 70 years, depending on slope height, inclination and location. Within the study area the authors suggest that the average rate of return of a landslide event is 60 years.

3.3 Approaches to Address the Provincial Policy

Determination of the *hazardous sites* or area of provincial interest for a specific site should be based on the subsurface soil and groundwater conditions and slope geometry, and should take into account both slope stability and erosion considerations. Defining the *hazardous sites* limit is necessary to provide some measure of security for the safety of any development in areas subject to landslides.

Areas where instability of slopes is certain (i.e., in the unstable or high risk zones) require the highest assurance and should accordingly address the threat to life situation. Special consideration should be given to structures such as hospitals, schools and homes for the elderly where the general public may be endangered. Policy 3.1.3 states that development involving these types of structures or activities (i.e., institutional uses), as well as those inolving essential emergency services or hazardous substances should not be permited within the defined area of provincial interest or *hazardous sites*. Caution should be exercised in the siting and permitting of water reservoirs, private swimming pools, septic tanks, and even waste disposal sites, where there is potentiality or evidence of the likelihood for slope failure as these types of activities or structures may pose an additional risk to the health and safety of local residents.

The risk involved can best be quantified by a geotechnical stability analysis. This requires knowledge at each site of the slope geometry and soil stratigraphy as well as the shear strength profile through the clays and groundwater levels in the slope.

Defining the "area of provincial interest" or *hazardous sites* for sensitive marine clays is based on one of two conditions:

- **there has been a retrogressive failure(s) of the slope in the past; or**
- **there is no current evidence suggesting that a retrogressive failure(s) has occurred.**

Where there is evidence of a retrogressive failure(s), the "area of provincial interest" or *hazardous site* is defined by:

- **a horizontal allowance of 1.5 times the distance of the previous failure measured landward from the toe of the failure (Figure 3.14) OR as determined by a study using accepted scientific, geotechnical and engineering principles.**

Figure 3.14: Area of Provincial Interest (1.5x)

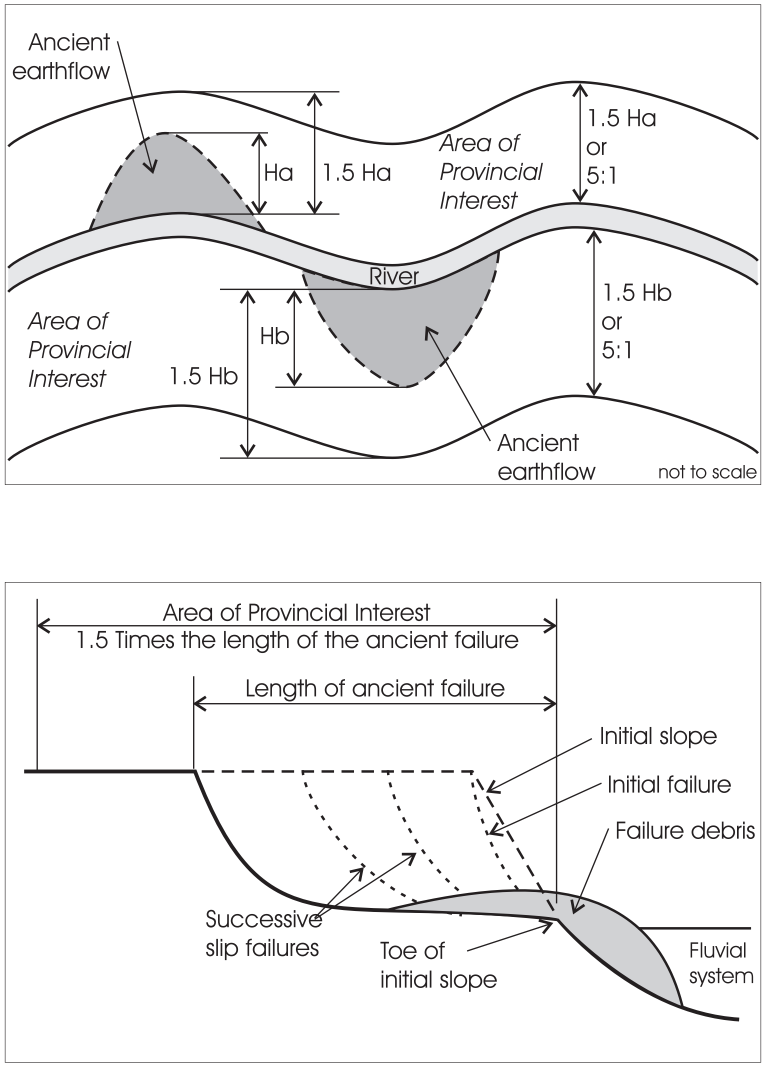
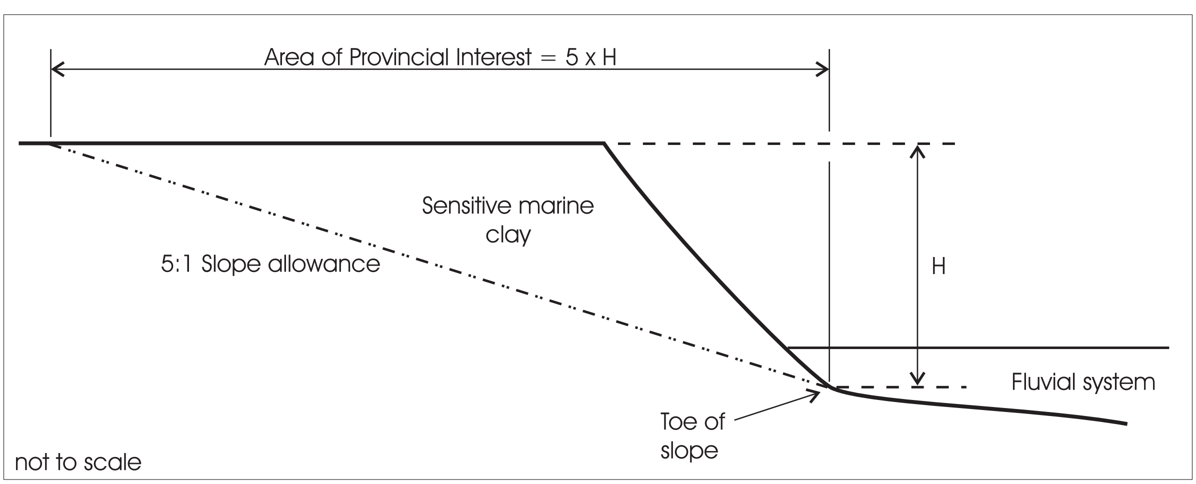


Figure 3.15: Area of Provincial Interest (5x)



Where there is no evidence of a retrogressive failure(s), the "area of provincial interest" or hazardous site is defined by:

- **a horizontal allowance of 5.0 times the height of the slope (Figure 3.15) OR as determined by a study using accepted scientific, geotechnical and engineering principles.**

It should be clearly noted that the application of either of these standards is intended to provide a level of protection against only the FIRST occurrence of slope failure.

Where a failure has occurred after either of these standards has been applied, the horizontal allowances must be re-calculated and the new or revised allowance(s) applied. In either case, a minimum factor of safety of 1.5 is recommended. Factor of safety is a measure of risk of failure or movement and is determined from an analysis of the ratio of the resistance of the movement of material (e.g., soil strength) as compared to the forces acting on the material (e.g., gravity, groundwater levels, slope inclination, soil weight) that may cause movement.

Consideration must be given to the nature of existing failures in a specific area. The horizontal allowance used to determine the limit of the *hazardous sites* should be adjusted if a study of aerial photographs indicates that retrogression has progressed beyond, or is encroaching on, the 5 : 1 standard.

The standard is time dependent (i.e., it is directly related to the position of the toe of the slope at any given point in time) and this factor must be taken into account when considering the location of a potentially unstable zone on a map.

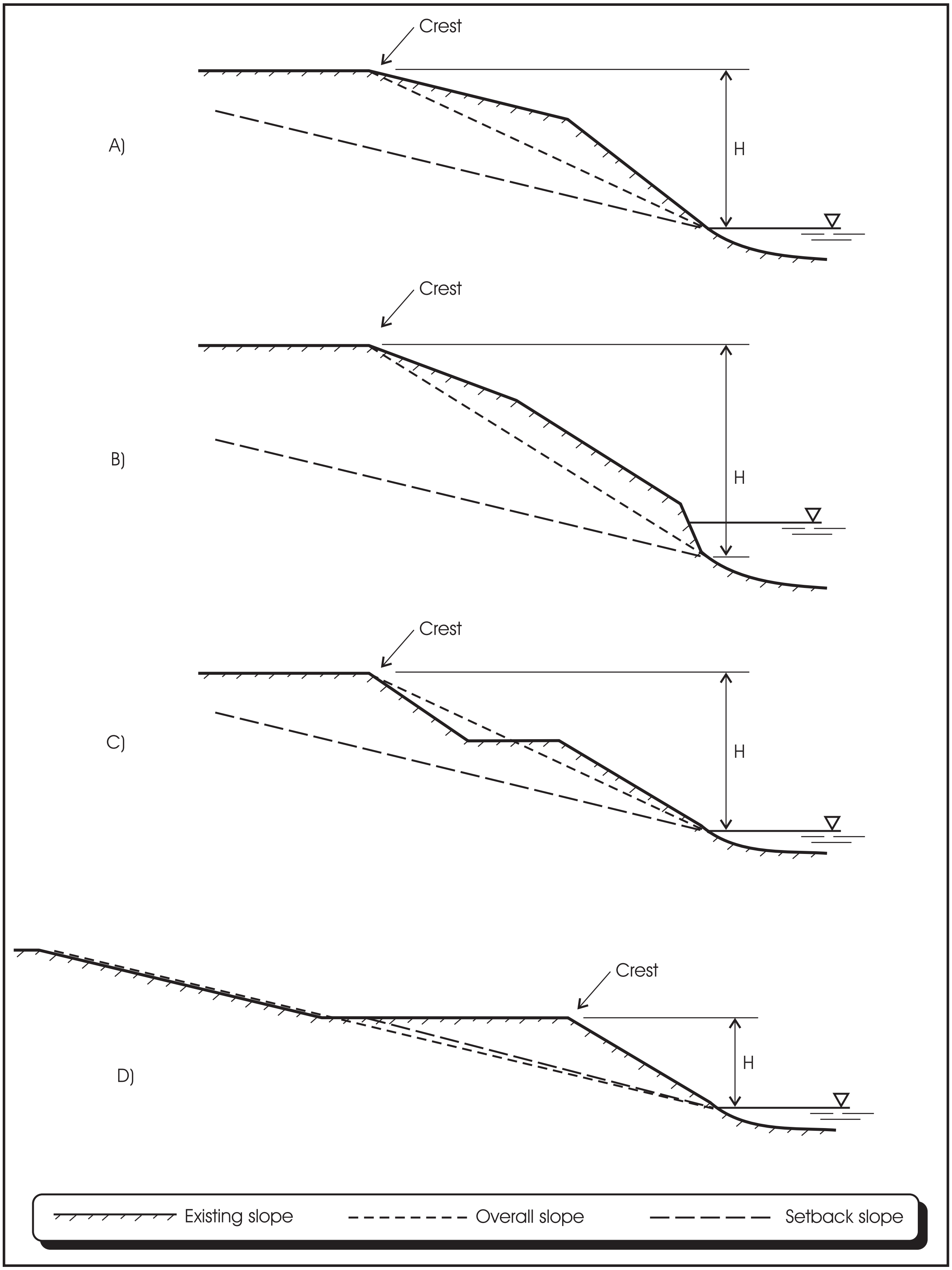
The slope height which should be used in determining the appropriate horizontal allowance will depend on the slope profile as shown on Figure 3.16. Where slopes consist of a single straight line segment, the relevant height is straight forward. Where the slope is benched or comprised of more than one segment, it is the overall slope angle and its relation to the slope angle which will determine the appropriate horizontal allowance to be used and as such, govern the limit of the *hazardous sites* or the area in which development should not be premitted.

As shown on Figure 3.16B, in situations where a river bed drops sharply (i.e., steeper than 5:1) and to depths substantially below the water level, the height should be measured from the base of the steep portion of the slope. For benched slopes, as shown on Figure 3.16C and 3.16D, where the overall slope is steeper than 5:1, the total slope height should be used. If the bench is wide enough and the upper portion of the slope shallower than 5:1, the *hazardous sites* limit is determined from the height of the lower slope. However, if the upper slope is substantially steeper than 5:1, an unstable situation would arise for reasons unrelated to the lower slope. Therefore separate geotechnical considerations must be given to these situations.

It should be noted that the horizontal allowance used in defining the *hazardous sites* limit, as calculated above, provide a margin of safety for the initial failure only. Should failure occur, the horizontal allowance should be adjusted accordingly or remedial measures undertaken, as required, to prevent subsequent failures from affecting existing structures. If stabilization works are undertaken, appropriate recommendations for design should be obtained from a geotechnical engineer and the works should be installed in accordance with the recommendations provided.

Where municipalities and planning boards determine that the identified horizontal allowance is excessive or not sufficient enough, mechanisms providing the flexibility to undertake a study using accepted scientific, geotechnical and engineering principles should be incorporated into the municipal planning process. This flexibility may not be warranted or desired where a more precise definition of the *hazardous site* (i.e., leda clay) horizontal allowance is not necessary, where there is sufficient area within the development lot to site any proposed development outside of the *hazardous site,* where development pressure is low and alternative development sites exist, or where the staff, administrative and financial resources within the municipality may preclude the ability of the municipality to support such studies.

Figure 3.16: Definition of Slope Height, "H", for Determining Setback Distance



For those situations where a study, based on minimum factor of safety of 1.5, is used to determine the *hazardous site* horizontal allowance, the study should be undertaken using "accepted scientific, geotechnical and engineering principles". Where studies using accepted scientific, geotechnical and engineering principles are approved by the municipality or planning boards, they should be applied only in the area studied.

Some studies of *hazardous sites* (i.e., leda clays) have already been undertaken by local agencies. These allowances are generally unique to specific locations or areas. Where local studies have been undertaken using accepted scientific, geotechnical and engineering principles they may be used to determine the *hazardous site* for the area studied.

3.4 Site and Field Investigation

Geotechnical site investigations should be carried out to assess the scale, nature and extent of the *hazardous sites.* Site investigation refers to the procedure of determining surface and subsurface condition in an area of proposed construction/study area. Information about the area is frequently available in the form of geological and topographic maps, and literature on general surface and subsurface conditions of the site. Aerial photographs can also provide considerable information on surface conditions and land forms that are not detectable from ground observations or not shown on the maps. For designing and planning purposes, detailed information on subsurface conditions, groundwater levels, soil and rock types and engineering properties should be obtained by means of on-site subsurface exploration. Typically, such information is obtained by drilling boreholes or excavation of test pits. The subsurface information can be supplemented by use of geophysical investigative methods. In-situ testing or laboratory testing on samples obtained during the investigation is used to determine soil and rock parameters.

3.4.1 Site Investigation

The investigation of a site potentially located within a sensitve marine clay area should involve a desk study and a site visit/field survey where the objectives are to:

- establish the geological and geographical extent of the hazardous site;
- review the type and extent of site hazard and the consequent risk to life, property and structure;
- review the techniques for investigating the site; and
- review remedial measures for mitigating the risks to life and property.

This is accomplished through the following:

- review and search of all available published and unpublished geological records;
- review of information on the existing landslides;
- review of aerial photographs; and
- site visit/field survey.

a) Review of Geological and Topographic Records

Information on the surficial geology of Ontario may be obtained from a number of potential sources including the Ministry of Northern Development and Mines, Geologic Survey of Canada, and the Ministry of Natural Resources. As well, mapping of the sensitive marine clay deposits within the Champlain Sea area have been completed by Fransham, Gadd and Carr. These maps, available for the majority of eastern Ontario, indicate those areas where the marine clays outcrop at ground surface and give an indication of the type of underlying soils where the clay deposits are relatively thin.

Topographic mapping for many urbanized areas prepared from aerial photographs interpretation is available from the municipal offices (e.g., Engineering or Public Works Department). The mapping should be at the scale sufficient to show details of the slope profile. Historical aerial photographs and other documents of slope instability, erosion, land filling and land development can be often obtained from the municipal level government offices.

b) Review of Aerial Photographs

Air photos provide detailed geological and topographic information over a relatively large area and are used to detect conditions that are difficult to observe/evaluate from a surface investigation (e.g., previous occurrence of landslides, meandering of rivers and potential future configuration of meandering, extent of glacially deposited landforms and land drainage pattern). The elements that are evaluated in the process of air photo interpretation include topographic features, drainage, erosion, soil tones and vegetation.

The density and pattern of drainage channels in a given area directly reflect the nature of the underlying soil and rock. Drainage patterns are better developed within the relatively impervious soils which promote surface run-off. For example, a closely spaced drainage system denotes relatively impervious materials; a tree-like drainage pattern develops in flat lying beds and relatively uniform materials; a parallel stream pattern indicates the presence of a regional slope. The basic patterns and their relationship to soils and bedrock are shown on Figure 3.17.

An important aerial photographic element is soil tone that can indicate moisture condition on the ground. A dark tone of soil generally indicates a high moisture condition on the ground (i.e., high groundwater level). The degree of sharpness of the tonal boundary between dark and light tone of soils aids in determination of soil type. Well-drained, coarse-textured soils show distinct tonal boundaries, while poorly drained, fine-textured soils show irregular, unclear tonal boundaries. Vegetative pattern is connected with soil moisture conditions and reflects local and regional climatic conditions. Typically, a small difference in soil moisture condition is detected by a corresponding change in vegetation. Examples of interpretation of moisture conditions on the ground, type of vegetation and land-use are presented on Figure 3.18.

The following features discernible on aerial photographs are typical of landslide or landslide-susceptible terrain (Rib, H.T. and Liang, T.):

- land masses undercut by streams;
- steep slopes having large masses of loose soils;
- sharp line of break at the scarp, or presence of tension crack, or both;
- hummocky surface of the sliding mass below the scarp;
- unnatural topography, such as spoon-shaped depression in the terrain;
- seepage zones;
- closely spaced drainage channels;
- accumulation of debris in drainage channels or valleys;
- distinctive change in vegetation and photograph tone indicative of changes in soil moisture; and
- inclined trees, displaced fences, distress to roadway, etc., due to creep.

Examples of the various forms of landsliding are shown on Figures 3.19 to 3.21.

Details of aerial photographic terrain evaluation are contained in Transportation Research Board Special Report 176, "Landslides, Analysis and Control", 1978.

c) Field Survey and Inspection

An important part of the investigation is a field survey to verify the date obtained from the map, literature and air photograph review. In the case of sensitive marine clay sites, the main concern with respect to hazard is the potential for slope instability and the consequent impact on the proposed works at the site. The site visit and/or field survey should therefore confirm the presence of sensitive marine clay and address those aspects which provide an indication of the stability of the slope.

Figure 3.17: Basic Drainage Patterns (Mitchell R.J., 1983)

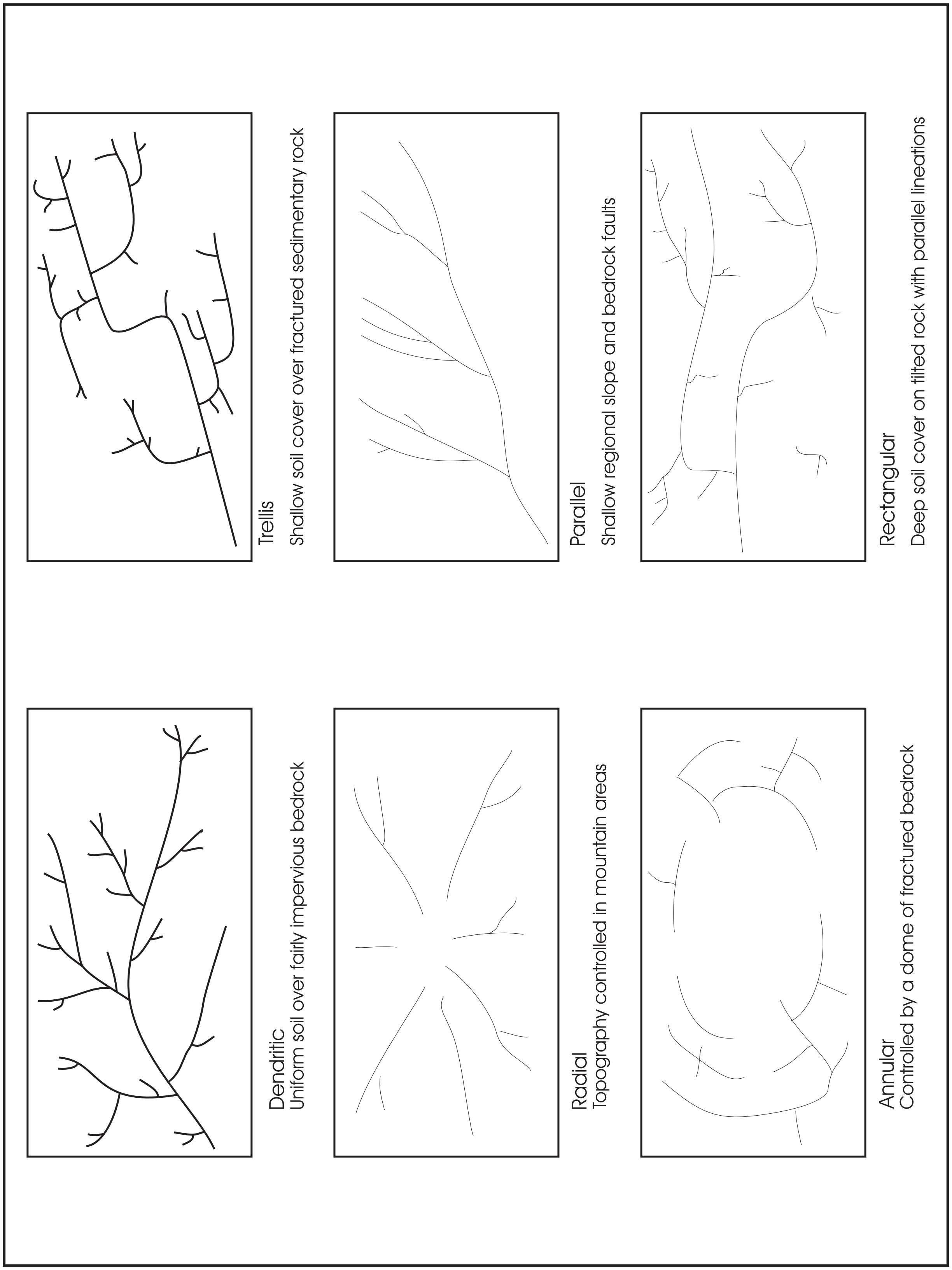
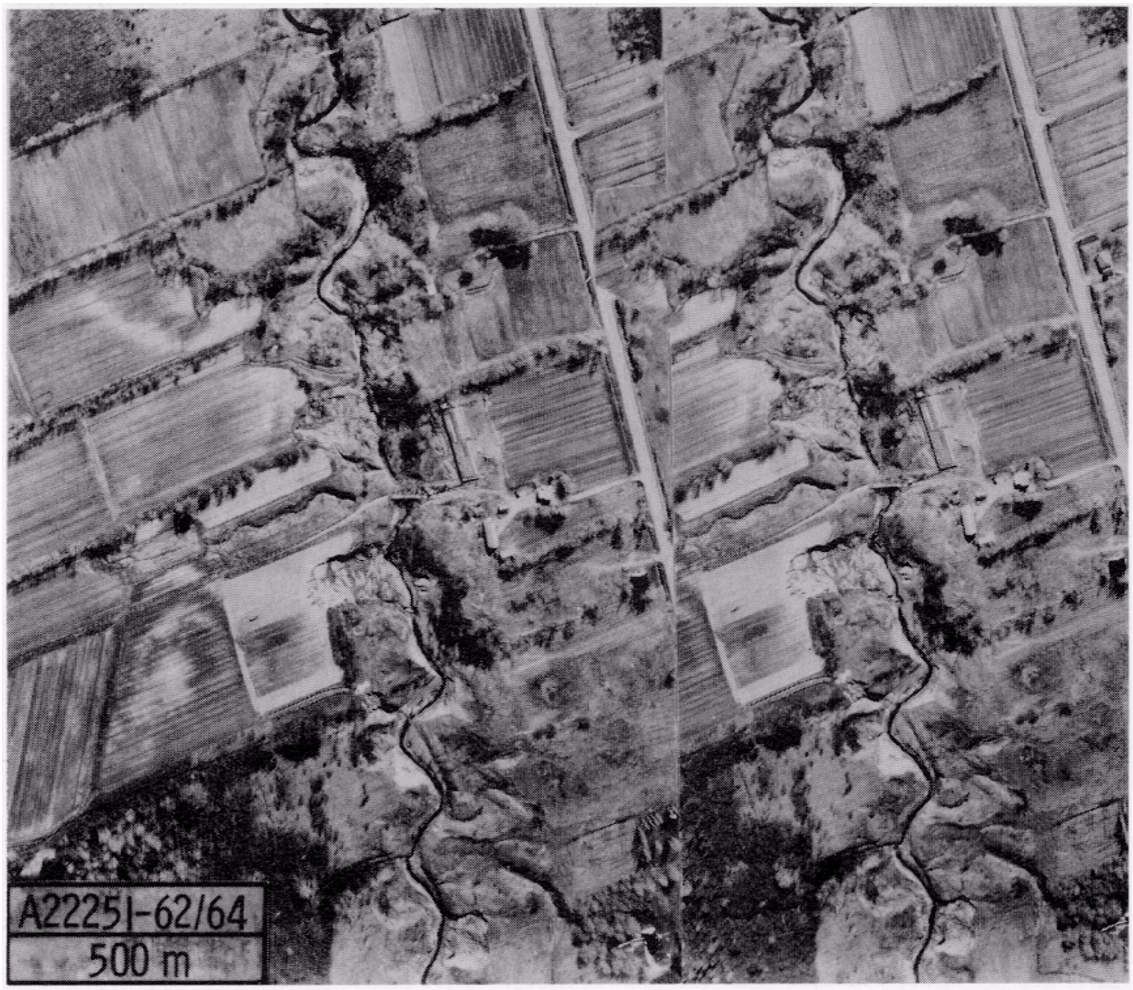


Figure 3.18: Moisture Condition, Vegetation and Land Use

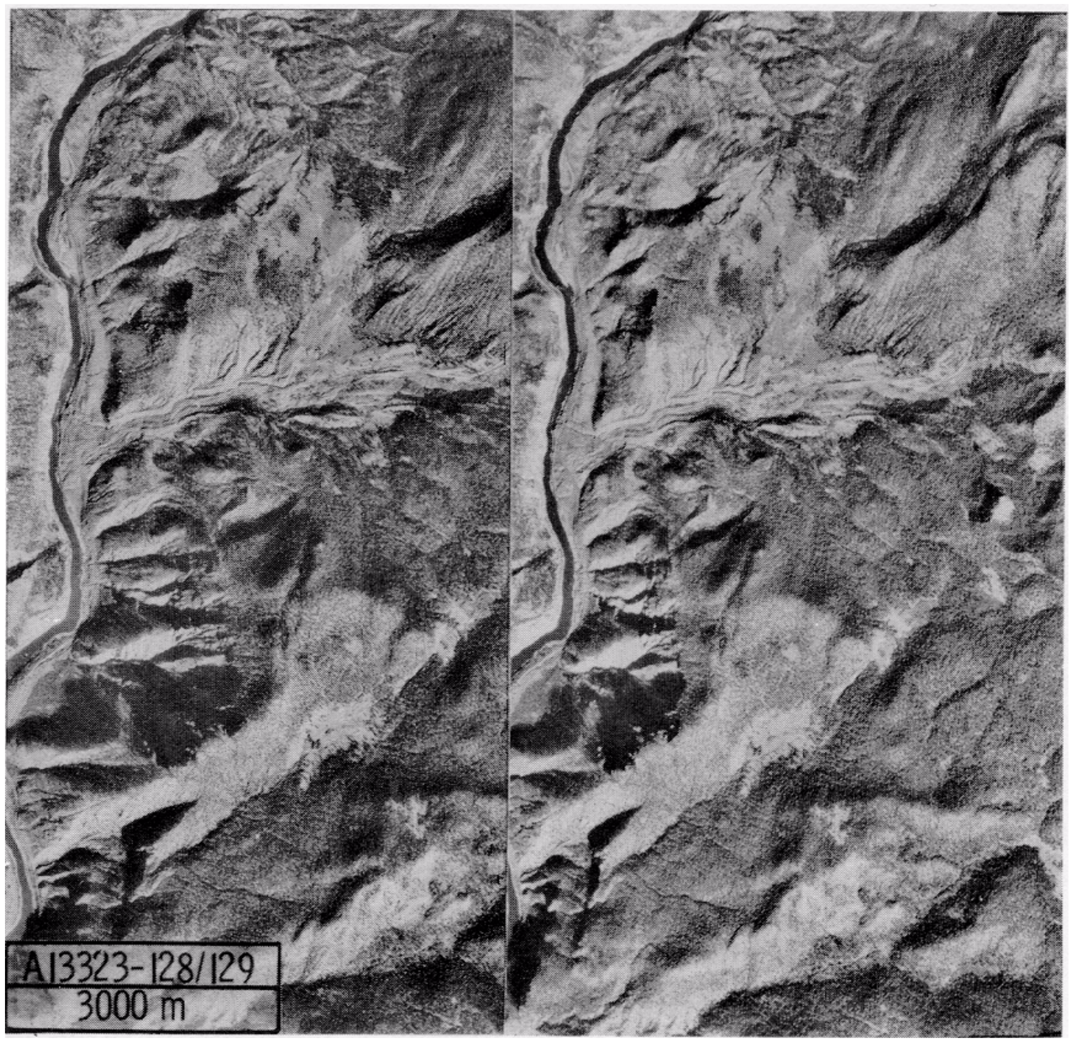
Contrast, vegetation and land use. (A) Silty clay ground moraine on the left has a high groundwater table indicated by the mottled tones. Deciduous trees predominate. The dark area on the right is peatlands. (B) Light tones of the well drained ancient beach sand contrasts with the darker tones of the bottom land. Contrast between photos is due to the difference in reflected light. (c) The second growth of trees in this area is mainly deciduous. Large areas have now been reforested to prevent the surficial sand from blowing. (Photos (a) and (b) courtesy of Energy, Mines and Resources, Canada. Photo (c) courtesy of the Ministry of Transportation and Communications, Ontario.) From Mitchell RJ., 1983.

Figure 3.19: Erosion and Landsliding



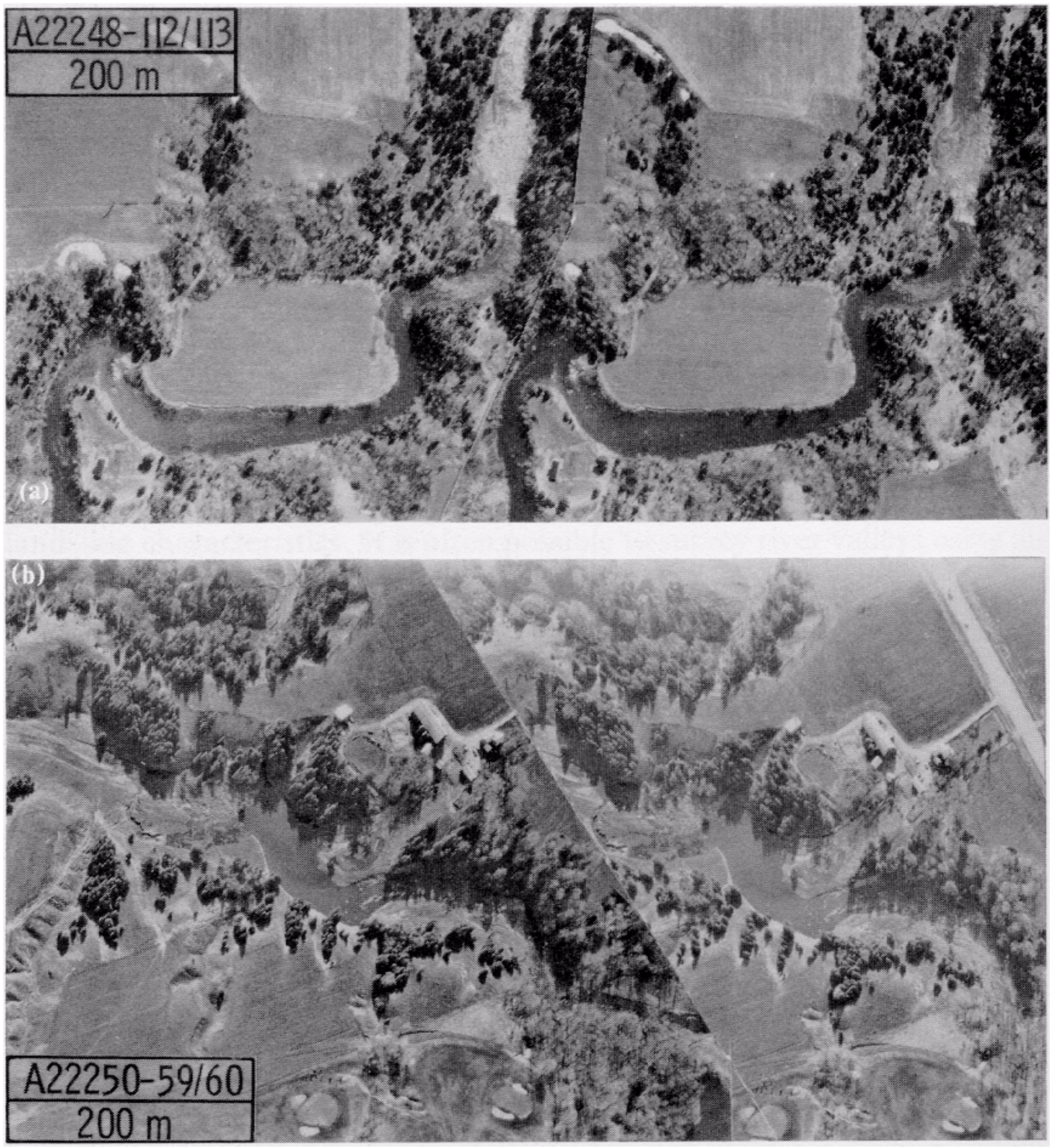
Erosion and landsliding. Toe erosion and high ground water promote valley widening by back-and-forth landslide activity (Photo courtesy of Energy, Mines and Resources, Canada.)
From Mitchell RJ., 1983.

Figure 3.20: Debris Flow or Mud Flow



Debris flow or mudflow. Active downslope movements at Drynoch landslide create maintenance for transportation routes. (Photo courtesy of Energy, Mines and Resources, Canada.)
From Mitchell R.J., 1983.

Figure 3.21: Common Landslide Types



Common landslide types. (a) Block slump in a lacustrine sediment (b) Rotational landslide. Cedar trees indicate abundant water. (Photo courtesy of Energy, Mines and Resources, Canada.)
From Mitchell R.J., 1983.

Typical data which should be recorded during the visual inspection of the site should include:

.	Date and time of inspection	including weather conditions, visibility and site accessibility
.	Site Location	describe site location with respect to major roads or regional features; provide sketch,
.	Watershed	record name of watershed site is located in.
.	Property Ownership	obtain name and address of property owner, and legal description for property; describe current land-use of site and adjacent properties.
.	Slope Data	record vertical height of slope from toe to crest; measure slope inclination (i.e., horizontal to vertical or angle from horizontal); also provide sketch and photographs.
.	Slope Drainage	describe locations and amounts of any seepage on the slope face or near the slope crest or toe; note location of any 'piping' if occurring, also provide sketch.
.	Slope Soil Stratigraphy	describe exposed soil stratigraphy (e.g., location, thickness, colour of soil layers) and soil types (e.g., sand, clay, rock) if possible; also provide sketch and photographs.
.	Water Course Features	indicate location and proximity of any nearby drainage features or water bodies (e.g., marshy ground, swale, channel, gully, springs, stream, creek, river pond, bay, lake); provide sketch.
.	Vegetation Cover	describe location, amount, and types of vegetation (e.g., grasses, weeds, shrubs, saplings, trees) on the slope (i.e., crest, face, toe) and on adjacent properties; provide sketches, and photographs.
.	Man-made Structures	describe location, types, and size of any man-made structures (e.g., buildings, retaining walls, fences, roads, stairs, decks, towers, bridges and utilities) on the slope face or near the slope crest or slope toe; provide sketches, and photographs.
.	Erosion Features	describe location, types and size of any erosion features (e.g., bare exposed soil, rills, gully, toe erosion, sour, undercutting, piping) on the slope face or near the slope crest or slope toe; provide sketches, and photographs.
.	Previous Landslide History	describe location, types, and size of any past slope movements (i.e., tension cracks, scarps, slumps, bulges, ridges, bent tree trunks or stands of dead trees) on the slope face or near the slope rest or slope toe; show on sketches, take photographs.
.	Comments	record any other general observations.
.	Plan View Sketch	show locations of slope crest, toe, structures, vegetation, stratigraphy, seepage, erosion, water course features.
.	Profile Sketch	show slope height, inclination, and shape.

The site inspection results should be documented on a Site Inspection/Record Sheet and Slope Stability Rating chart. An example of these sheets are presented in Appendix A3.4.

Where the consultant considers that the site conditions do not warrant further study, the report should:

- itemize all results from the full site inspection;
- provide an assessment of the stability of the slope;
- provide a clear statement of the justification for allowing the removal or altering of the setback limitations at the site (i.e., typically only where it is found that marine clays are not present at the site), and
- provide a clear statement of the effect of the proposed construction on the stability of slope.

It should be noted that the field measurements and observations are time dependent, and represent the slope conditions at the time of the survey.

3.4.2 Field Exploration

A field investigation consists of mapping, drilling and/or trenching possibly together with geophysical methods and laboratory testing.

The objectives of the field investigation are to:

- identify the type and nature of the soils/site stratigraphy;
- identify the marine clay/weaker soil formations that are likely to be involved in soil movement;
- identify the strong formations that provide significant resistance or that might limit the extent of the zone of movement or provide support for proposed development;
- locate aquifers, define groundwater levels and pressures, and determine water chemistry; and
- obtain data on the physical and mechanical properties of the formations for use in design of the development/analysis of stability.

The field investigation can be carried out in stages:

- preliminary exploration for economic and technical feasibility reports;
- project planning data should permit the selection of the locations, horizontal allowance (e.g., setback) requirements, types and dimensions of the development; and
- additional exploration for detailed design.

The borings, as a minimum, should extend to a depth sufficient to reveal the nature of all soils which could be affected by the development and which, by settlement and/or shear failure, could affect the integrity of the structure. The layout and spacing of the borings should permit to develop typical geological cross-sections. In a case of a river valley, the borings should be located on the lines across the valley up and down the slope and along the length of the slope. Where a slide has occurred, the borings should be located within the critical areas of the zone of movement, as well as an adjacent areas that have not yet failed.

Descriptions of boring techniques are provided in Appendix A3.5. The selection of the techniques will depend on the information required, the topography and ground trafficability and the information required.

Samples taken during field investigation should permit visual identification and classification of the materials encountered by means of laboratory testing (Appendix A3.5, Tables A3.3 and A3.4). However, evaluation of the significance of sensitive marine clays to a proposed development requires that the testing be carried out on both undisturbed and remoulded samples of the clay. Handling and testing of the samples of sensitive clays marine clays should be attempted only by competent personnel experienced in this type of work.

3.4.3 Report Requirements

Adherence to the provincial policy should result in proposed development being safely distanced from the slope crest with this distance being determined through a study of the subsurface soil and groundwater conditions and slope geometry. Where development, construction or changes to a slope are proposed within the defined *hazardous sites* limit, an appropriate site specific geotechnical investigation should be carried out prior to approval to commence the works. The purpose of the investigation at a specific site is to establish the subsurface conditions and reliable soil strength profile parameters and to carry out a stability analysis for the slopes of concern to determine:

- risk of failure;
- setback required to ensure the safety of the proposed works, and
- remedial measures, if any, which are required to maintain stability.

Numerous technical publications which deal with instability problems in sensitive marine clays including Champlain Sea clays and which provide techniques for analysis of slope failures are available (Tavenas et al

1978, 1982) and should be consulted.

The geotechnical consultant should provide comments on the existing stability and on the possible effects of the proposed works on the stability. The horizontal allowance used to define the *hazardous sites* limit provided by the consultant should contain an adequate factor of safety for a given design life expectancy for the structure. The calculated factor of safety required for a specific site is dependent on the level of confidence the consultant has for the assumptions or for parameters used in the analysis of the stability of the slope. With strength parameters based on reliable results of laboratory tests for the specific site, a factor of safety of 1.2 of 1.3 may be considered adequate depending on the proposed works. If strength parameters are assumed from regional data, a minimum factor of safety of 1.5 would generally be required.

To ensure that this safety factor is maintained, the consultant or proponent should ensure that the recommendations provided are complied with during construction. In addition, the effect of the development or remedial works on adjacent structures and/or slopes should also be addressed. The level of geotechnical investigation required, routine or detailed, will depend on the specific site conditions including the subsoil and groundwater conditions, the presence of existing instability and the type of construction which is proposed at the site. Any investigation should involve a routine site visit, with an initial visual inspection carried out by experienced geotechnical personnel to determine specific site conditions.

Depending on the results of the visual inspection, additional investigation may be required. If the site conditions can be shown to pose no threat to the proposed construction at the site, and vice versa, the initial inspection can be considered sufficient for analysis.

3.4.4 Overview of Review and Decision Process for Sensitive Marine Clays

A "checklist" of issues, questions or conditions to be ultimately addressed by a geotechnical consultant when involved with a project which includes slope stability concerns has been complied below. This list may not necessarily be applicable to all forms or types of development within the hazard limits at all sites; however, the reason for discounting an item should be stated where an individual item is considered to be unwarranted.

- **Step 1: Information Study**
  - refer to section 3.4.1, Parts a and b

- **Step 2: Initial Site Inspection**
  - refer to section 3.4.1, Part c

- **Step 3: Reporting of Visual Inspection**

  Where the consultant considers that the site conditions do not warrant further study, the report should:

  - itemize all results from Step 1 and Step 2 above;
  - provide an assessment of the stability of the slope;
  - provide a clear statement of the justification for allowing the removal or altering of the setback limitations at the site; and
  - provide a clear statement of the effect of the proposed construction on the stability of slope.

- **Step 4: Subsurface Investigation**

  Where a detailed geotechnical study is required, the consultant should:

  - put down boreholes at appropriate locations to provide a full stratigraphic section over the slope height and to well below the toe of the slope;
  - obtain samples at regular intervals of depth (e.g., typically 0.75m to 3m) and carry out field vane testing throughout the cohesive deposits to establish the undrained shear strength profile;
  - obtain undisturbed Shelby tube samples of the clay for laboratory testing;

- install piezometers in the boreholes to monitor the groundwater levels in the various strata encountered; the piezometers should be monitored over sufficient length of time to provide stabilized groundwater levels. Ideally, the seasonal fluctuations would also be monitored; and
- in addition to routine classification testing, carry out specific laboratory testing to determine the Atterberg Limits and effective shear strength parameters of the clay deposits encountered.

- **Step 5: Analyses and Reporting**

  In addition to summarizing and reporting the subsurface conditions encountered during the investigation, the consultant should:

  - with the subsurface information obtained and the laboratory test results, carry out appropriate stability analyses of the slope using effective strength parameters or undrained shear strength where applicable to establish the minimum factor of safety against failure of the slope;
  - provide the appropriate setback to ensure an adequate safety margin to the proposed construction;
  - assess the impact of construction on adjacent slopes;
  - provide recommendations for remedial works for repair of existing slope failures or for ensuring the stability of the slope following development, including an assessment of the impact of the remedial works on adjacent slopes;
  - provide a framework for inspection and monitoring during construction as well as long term monitoring, if necessary;
  - investigate the possibility of a retrogressive failure or earthflow occurring at the site. This can be investigated using the criteria proposed by Mitchell, 1978, in combination with that of Tavenas, 1984, utilizing the geotechnical data obtained as discussed above; and
  - carry out site inspections during construction to ensure compliance with the recommendations provided.

3.5 Addressing the Hazard

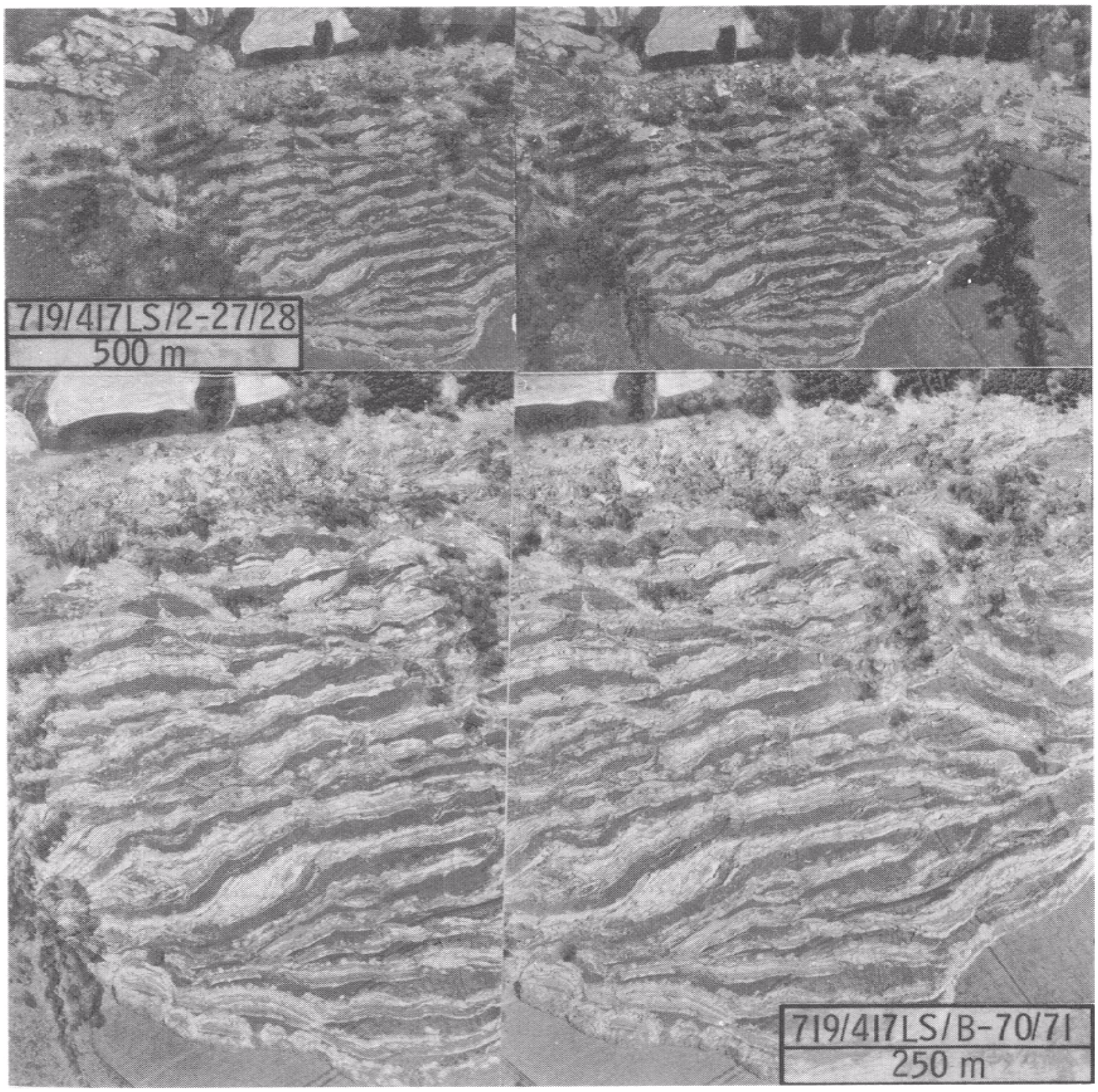
Geotechnical design of foundations on the sites where the sensitive marine clay deposit is present should be carried out in accordance with the requirements of the Canadian Foundation Engineering Manual and the National Building Code of Canada (i.e., the newest editions).

Where foundations are constructed on sensitive marine clays, the loading must be maintained low enough such that excessive settlement and/or catastrophic failure do not occur. Generally, this is achieved by ensuring that the preconsolidation pressure within the clay is not exceeded and by ensuring that the shear stresses produced are well below the shear strength of the clay. Construction and landscaping activities can have a great impact on the magnitude and depth of influence of ground movements. Large movements/instability can occur in deep excavations in sensitive marine clays which were not designed to accommodate the in-situ strength within the soil mass beneath and adjacent to the excavation. The introduction of deep-rooted vegetation in areas where it has not grown previously, or the removal of mature vegetation that has depleted subsoil moisture, can result in significant surface settlement or heaving (e.g., of an order of 300mm in magnitude), extending for great depths and horizontal distances (Figure 3.22). Heavy irrigation or change in ground surface covers can induce similar impacts on the movements at sensitive marine clay sites.

Proposed development, construction or alteration to the slope within the defined *hazardous sites* limit should have geotechnical input prior to authorization to proceed with the works. Site development such as swimming pools, parking lots and the like, should be included since their construction can have a more destabilizing influence on a slope than construction of a building. Site alteration involving small structures, such as garden sheds, in which there is no direct threat to life may not require regulation provided that the slope will not be altered and/or the stability will remain unaffected during or subsequent to construction.

In general, a geotechnical investigation should be undertaken for the design of foundations for structures regardless of slope stability considerations within areas of sensitive marine clays due to their unpredictable and inherent unstable nature. For structures located in close proximity to the slope, in addition to the routine type of foundation investigation, the effective strength parameters of the clay at the site and groundwater conditions in the slope should also be carefully examined for stability analyses purposes.

Figure 3.22 South Nation River Landslide



(From Mitchell R.J., 1983. Photos courtesy of the Ministry of Transportation and Communications, Ontario.)

3.6 References - Marine Clays

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APPENDIX A3.1

EXISTING GENERAL GUIDELINES

SENSITIVE MARINE CLAYS

1.0 Existing General Guidelines

Approaches to establishing horizontal allowances used to determine the *hazardous sites* limit are generally, or have previously been described in terms of “setback requirement”. This appendix provides brief descriptions of the existing guidelines or approaches used by the South Nation Conservation Authority and the Ministere de l'Energie et des Resources in southwestern Quebec. These are presented as examples of how different agencies have addressed the hazards associated with sensitive marine clays.

a) MNR Eastern Region/Southeastern Ontario Guideline

The South Nation Conservation Authority carried out a study which involved putting down a limited number of boreholes to obtain samples of the Leda clay for laboratory testing to determine appropriate shear strength parameters. These parameters were used in stability analyses carried out for sites for which a Factor of Safety less than 1.5 had been previously determined. The strength data obtained at each field testing location was assumed to be representative of a relatively long reach of the river system in the area of the borehole locations. The results of the analyses were used to establish a family of failure surfaces, with a Factor of Safety of less than or equal to 1.5, and the slip surface chosen which extended the furthest distance into the slope was found. The setback distance was then defined by adding a further 10m to the point of intersection of this failure surface with the tableland behind the slope crest (see Figure A3.1.1). The setback distance obtained in this manner was considered to be adequate safeguard to structures located behind this point in the event of an initial failure.

This guideline states that no construction within the setback distance should take place without a prior detailed geotechnical investigation and implementation of recommended remedial works. It is noted that the setback distance is a minimum value and is intended to prevent damage to structures located behind the crest of the slope from an initial failure. To prevent any subsequent failures from encroaching on lands beyond the setback distance, in the event of the initial failure occurring, some remedial work to the slope may be necessary. To this end, recommendations for remedial works should be provided by an experienced geotechnical engineer.

b) MER Southwestern Quebec

The Ministere de l'Energie et des Resources have adopted a similar approach to the landslide hazard problem. The areas where sensitive clay deposits comprise slopes have been classified into "risk zones" and further emphasis is placed on populated areas located within zones which have been classified as high risk. The zoning is carried out by field mapping to determine the slope geometry and by field borings to obtain subsurface information. A limited number of boreholes are put down at locations which are considered to be representative of a certain area. Undrained shear strength profiles are obtained through the clay deposits and site details such as signs of erosion and instability and presence of old earthflows in the area are recorded. A regional compilation of slope geometry and existing failures provides guidelines for determining the relative stability and/or risk of failure of various slope profiles over a certain area.

High risk zones include slopes with geometry indicative of near failure and which show signs of instability. Medium risk zones have similar slope profiles to the high risk zones, but those processes which affect the stability of the slope are not visibly occurring at the sites. Low risk zones include areas which are underlain by sensitive clays in which earthflows have occurred and which are adjacent to high risk zones. Hypothetical risk zones have the same features as low risk zones except that the geotechnical properties of the clay do not have the same earthflow potential.

Extensive studies have been carried out to obtain geotechnical data sufficient to allow comparisons between soil parameters, site conditions and failure mechanisms. The potential for flow/retrogression, as well as the distance of retrogression, has been found to be related to intrinsic soil properties such as sensitivity, remoulded strength and plasticity of the clay. This data has been utilized in establishing the zones of relative risk limits. These limits are not necessarily construction setbacks but rather they provide a guideline to delineate the potentially unstable zones.

Figure A3.1.1: Criteria Proposed by MNR South Nation Conservation Authority

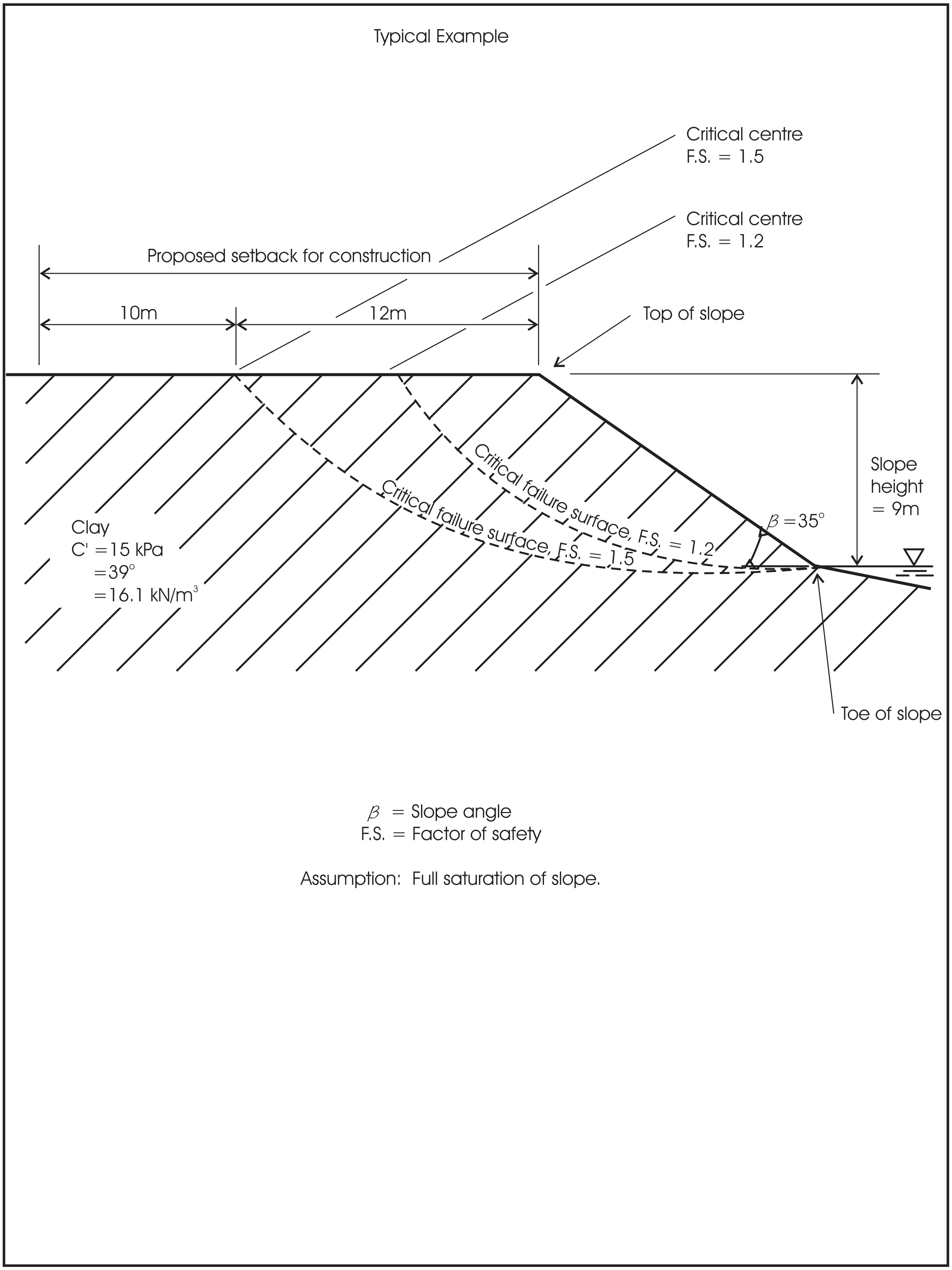
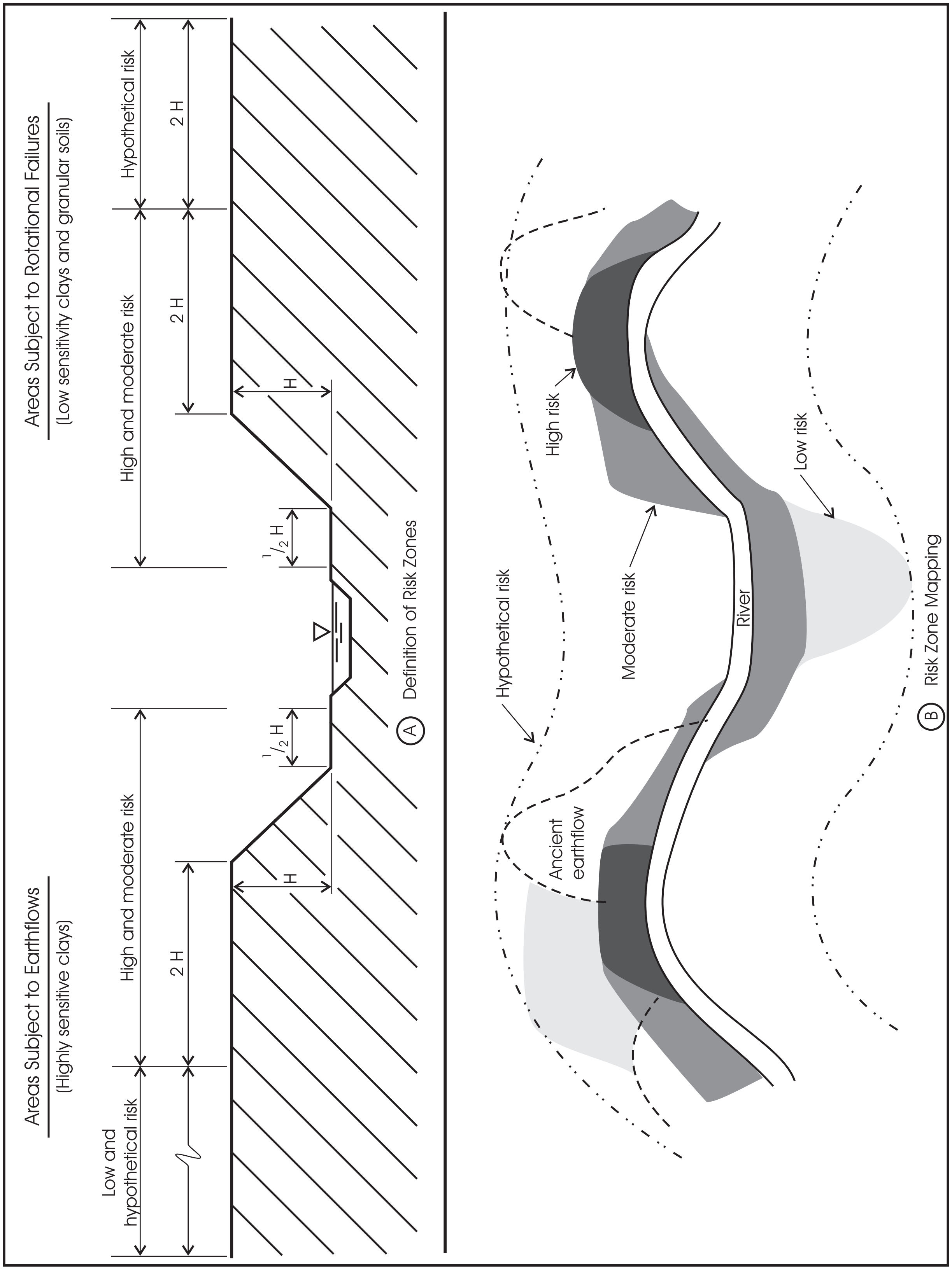


Figure A3.1.2: Schematics Showing Study Approach of Quebec Mer



The procedure used for outlining and mapping of the risk zones is shown on Figure A3.1.2. For both high and moderate risk zones, the limit is defined by a line located behind the slope crest a distance of twice the slope height. The zone also includes the area at the base of the slope (i.e., measured away from the toe) equal to 0.5 times the slope height.

The limit of low and hypothetical risk areas depends on the soil type. In highly sensitive clays, a comparative analysis of the nature of the clay within a general area is carried out to establish whether there is potential for retrogressive failure to occur. An estimate of the distance of retrogression is made from a regional compilation of existing failures and their retrogressive distances. For areas of low sensitivity clays and in granular soils, the hypothetical risk zone extends, behind the high risk zones, a further distance equal to 2 times the slope height. The maps which are produced from this work are at a scale of 1:20,000 and show the lateral extent of the zones along the river courses and on sloping ground.

Subsequent studies concentrate on those areas which are populated and which fall within zones of high risk. Detailed analyses are carried out to determine the stability of these slopes to assess the danger to existing structures and to establish setback distances for new construction. Effective strength parameters were determined for the soils at the site by laboratory testing and the groundwater conditions in the slope were monitored with piezometer installations. Recommendations for stabilization measures are then provided, if required.

It should be noted that the delineation of a potentially unstable zone by a distance of twice the slope height only, may not be conservative. In the case of the two Quebec studies reviewed, a large number of retrogressive failures and earthflows, which govern the extreme limits of the zones of risk (see Figure A3.1.2) have occurred in the past. Therefore, the superimposition of the extent of the low and hypothetical risk zones onto that of the high and moderate risk zones provides an additional security to the interpretation.

APPENDIX A3.2

TYPICAL PROPERTIES OF SENSITIVE MARINE CLAYS

1.0 TYPICAL PROPERTIES OF SENSITIVE MARINE CLAYS

The basic index, mineralogical and engineering properties of the marine clay are summarized below (Pegg et al 1982, Quigley et al 1985 and unpublished Golder Associates data):

- Property

Sensitivity of weathered crust	2-4
Sensitivity of unweathered crust	20-500
Natural moisture content, per cent	45-100
Liquid Limit, per cent	45-90
Plastic Limit, per cent	20-40
Liquidity Index	0.9-4.0
Density, kg/cu.cm	1.5-1.7
Permeability, cm/s	10^{-7} to 10^{-9}
Illite, per cent	8

- Mineralogy

Chlorite, per cent	16
Vermiculite, per cent	7
Quartz, feldspar, carbonates, per cent	39

- Strength

Undrained shear strength of weathered crust, kPa	0-200
Undrained shear strength of unweathered clay, kPa	20-150

Liquidity indices close to or in excess of 1 indicate that the clays are very sensitive to remoulding and liquefaction. High void ratios, as large as 2 to 4, and a considerable amount of free water trapped in the voids between the particles in addition to the adsorbed water mobilized by clay particles are typical to marine clays. When undisturbed, the particles are tightly bound through the electrostatic forces. During remoulding, the bonds between particles are destroyed and the water trapped in the voids is released to the adsorbed layers at the point of contact causing loss of strength and softening of the soil.

The ratio of the undisturbed to the remoulded strength of the soil is defined as sensitivity. Sensitive clays are defined as having sensitivity value between 4 and 16; those having sensitivity equal to or greater than 16 are called "quick clays". For the marine clays, the range of sensitivity values is extremely wide and for the unweathered clay deposits, it varies from about 5 to very large numbers; the sensitivity of the weathered crust is much lower and typically ranges between 2 and 4. The highest sensitivity values correlate to the lowest salinity measured in the clay (Quigley et al. 1983).

A broad description of marine clay (i.e., Leda Clay) properties is contained in the Ontario Geological Survey, Study 29, titled "Geology, Geotechnique, Mineralogy and Geochemistry of Leda Clay from Deep Boreholes, Hawkesbury Area, Prescott County", MNR 1985.

APPENDIX A3.3

CONSIDERATION FOR EARTHFLOW ZONES

1.0 Consideration for Earthflow Zones

Rapid earthflows are common in the marine clays. They involve usually retrogressive failure with lateral spreading and liquifaction of the subjacent materials and entire slide mass. Determination of a zone or extent of retrogressive earthflows in the marine clays is of considerable importance for planning development.

A study by Gorrell, 1984, has delineated an area along the South Nation River with a history of past earthflow events. In the above study, an attempt was made to isolate the stratigraphy which is prevalent in the earthflow zones and which may play a role in the development of this type of failure. Although not definitive, the study has compiled much relevant geotechnical information. Documentation has been made by Mitchell, 1978, of earthflows and retrogressive flowslides on Green Creek, east of Ottawa, and on Madawaska River, west of Ottawa, as well as on South Nation River.

An empirical method to determine the retrogression distance/extent of earthflow has been proposed by Mitchell (1978). The retrogression distance, R, measured from the crest of the slope towards the land is based on the stability number, N_s, defined as:

$$N_s = KH/c_u$$

where K- unit weight of soil;
H - height of bank;
c_u - minimum undrained shear strength of marine clay.

The retrogression distance, R, for the stability number N_s ranging between 5 and 12, is approximated by the following expression:

$$R = 100(N_s - 4)$$

One positive method of establishing general areas where earthflows are possible is to identify those areas where they have occurred in the past. The empirical method proposed by Mitchell when used to determine retrogressive distance for the recent Lemieux earthflow, confirmed almost exactly the existing earthflow extent.

The criteria proposed by Tavenas, 1984, as outlined briefly in section 3.2.3 of this report, can establish the risk of an earthflow occurring at a specific site. Although Tavenas outlines guidelines which can be used to estimate the distance of retrogression, the guidelines have been based on data obtained in the province of Quebec and they may not necessarily apply to conditions in south-eastern Ontario, although the clays have similar origins. Comparison studies at the Ontario earthflow locations should be carried out.

It is suggested, as has already been proposed and implemented by MNR, that the setback for construction within earthflow zones should be defined by a line drawn between existing failures at a distance from the river as per the observed retrogression. This procedure is also similar to that of the hypothetical risk zone defined by Quebec MER (see Appendix A3.1).

APPENDIX A3.4

SITE INSPECTION RECORD AND SLOPE STABILITY RATING CHART

Site Inspection Record

The Site Inspection Record has the following components to be recorded on site:

1. FILE NAME / NO

   - Inspection date (DD/MM/YY):
   - Weather and estimated air temperature:
   - Inspected by (name):

2. SITE LOCATION (describe main roads, features, provide sketch)

3. PROPERTY OWNERSHIP (name, address, phone):

4. LEGAL DESCRIPTION

   - Lot
   - Concession
   - Township
   - County

5. CURRENT LAND USE (circle and describe)

   - Vacant - field, bush, woods, forest, wilderness, tundra
   - Passive - recreational parks, golf courses, non-habitable structures, buried utilities, swimming pools
   - Active - habitable structures, residential, commercial, industrial, warehousing and storage
   - Infra-structure or public use - stadiums, hospitals, schools, bridges, high voltage power lines, waste management sites,

6. SLOPE DATA

   - Estimated height (m):
   - Inclination and shape

7. SLOPE DRAINAGE (describe)

   - Top
   - Face
   - Bottom

8. SLOPE SOIL STRATIGRAPHY (describe, positions, thickness, types)

   - Top
   - Face
   - Bottom

9. WATER COURSE FEATURES (circle and describe)

   - Swale, channel
   - Gully
   - Stream, creek, river
   - Pond, bay, lake
   - Springs
   - Marshy ground

10. VEGETATION COVER (grasses, weeds, shrubs, saplings, trees)

- Top
- Face
- Bottom

11. STRUCTURES (buildings, walls, fences, sewers, roads, stairs, decks, towers)

- Top
- Face
- Bottom

12. EROSION FEATURES (scour, undercutting, bare areas, piping, rills, gully)

- Top
- Face
- Bottom

13. SLOPE SLIDE FEATURES (tension cracks, scarps, slumps, bulges, grabens, ridges, bet trees)

- Top
- Face
- Bottom

14. SKETCH OF SLOPE IN PLAN

15. SKETCH OF SLOPE PROFILE

Table A3.4.1 Use, Capabilities and Limitation of Test Pits and Trenches (adopted from Canadian Foundation Engineering Manual, 3rd edition, 1992)

2.0 EXPLORATION METHOD GENERAL USE CAPABILITIES LIMITATIONS

Hand-Excavated Test Pits and Shafts Bulk sampling, in-situ testing, visual inspection. Provides data in inaccessible areas, less mechanical disturbance of surrounding ground. Expensive, time-consuming, limited to depth above groundwater levels.

Backhoe Excavated Test Pits and Trenches Bulk sampling, in-situ testing, visual inspection, excavation rates, depth of bedrock and groundwater. Fast, economical, generally less than 5m deep, can be up to 10m deep. Equipment access, generally limited to depths above groundwater level, limited undisturbed sampling.

Drilled Shafts Pre-excavation for piles and shafts, landslide investigations, drainage wells. Fast, more economical than hand excavation. Diameters typically range from 760mm to 2.0m. Equipment access can be difficult. Undisturbed samples and block samples can be obtained with some effort. Slotted casing limits visual inspection.

Dozer Cuts Bedrock characteristics, depth of bedrock and groundwater level, rippability, increase depth capability of backhoes, level area for other exploration equipment. Relatively low cost, exposures for geologic mapping. Exploration limited to depth above groundwater level.

Trenches for Fault Investigations Evaluation of presence and activity of faulting and sometimes landslide features. Definitive location of faulting, subsurface observation up to 10m. Costly, time-consuming, requires shoring, only useful where dateable materials are present, depth limited to zone above groundwater level.

APPENDIX A3.5

GEOTECHNICAL INVESTIGATION

1.0 GEOTECHNICAL INVESTIGATION

a) Objectives and Extent of Geotechnical Investigation

The objectives of the geotechnical investigation are:

- to establish nature and type of the soil;
- to determine the sequence, thickness and lateral extent of the soil strata;
- to obtain representative samples of the soils for identification, classification and laboratory testing of relevant soil parameters;
- to determine mechanical properties, such as strength and compressibility of the soils; and
- to identify the groundwater conditions.

The extent of the field investigation depends on the variability of the soil and groundwater conditions, type of project and extent of the existing information. In general, the number and location of the boreholes should enable the stratigraphy of the site to be determined and should provide data for an adequate and economical design of the project (i.e., slope assessment/foundation design).

Generally, more detailed information is required at the location of the important structures or where ground conditions are complicated, such as old landslide areas or suspected buried valleys. Spacing of boreholes depends on the variability of the site conditions, type of project, performance requirements and available existing information in the project area. More boreholes and closer spacing is typically recommended for sites located in less developed areas where previous experience is sparse. Where the general stratigraphy of the site is known from the previous borehole investigation and for simple project, a single borehole may be sufficient to confirm the stratigraphy. However, the results of one borehole investigation can be misleading and should be avoided.

For the foundation design purposes, the Canadian Foundation Engineering Manual suggest the following guidelines for the site investigation:

- a minimum of four boreholes for buildings between 250m^2 and 1000m^2 in plan, where the ground surface is level and the first two boreholes indicate regular stratification; five boreholes, four at building corners and one in the centre, are preferable; and

- a minimum three boreholes for buildings smaller than 200m^2 in plan.

The extent of the geotechnical investigation for assessment of stability of slopes comprising sensitive marine clays and the investigation for large and complex projects where usually larger number of boreholes are required, should be established by an experienced geotechnical engineer.

The depth of exploration should be such that the entire zone of soil affected by the changes caused by construction is investigated. Commonly, the boreholes drilled for the foundation design should be carried out not shallower that the depth where the net increase in soil stress due to the structure is less than 10 % of the applied loads, or less than 5% of the effective stress in the soil, whichever is less. Greater exploration depths are required for the slope stability assessment; the boreholes should penetrate all soils forming the slope and should extend to the stronger strata underlying the slope.

Since most landslides in the sensitive marine clays are influenced by the climatic changes, in planning the investigation for the slope stability assessment, climatic changes in the soil should be taken into consideration. The investigation should be undertaken during the season that is least favourable for stability. Typically, a minimum soil strength and maximum water pressures occur in the soils during periods of snowmelt or following heavy rain.

Types of test boring and test pits are summarized in Tables A3.5.1. In-situ and laboratory testing with references for standard test procedures are given in Tables A3.5.2 to A3.5.4. Brief description of common investigation techniques are provided in the following sections.

b) Boreholes

Reference is made to boreholes as means of site investigation, however, in some cases boreholes can be supplemented/replaced by test pits, soundings or probe holes. Boreholes will serve to provide a visual identification of soil strata based on the samples that are taken at the known depth and brought to the surface for examination and laboratory testing. The quality of the samples depends mainly on the boring method, the sampling equipment, and the procedure used in retrieving them. Various types of boring methods are summarized in Table A3.5.1 adopted from the Canadian Foundation Engineering Manual (1993).

The most common methods to advance boreholes are:

- hand or power operated auger boring;
- wash boring; and
- rotary auger (i.e., solid or hollow continuous flight augers) boring.

Hand auger holes are usually shallow and are carried out on steep slopes or in the areas where access is difficult. The information obtained from these holes are of limited values; the soil type can be detected by noting the soil materials carried up to the surface by augers. Rotary auger technique using continuous flight solid stem augers involves extraction of the augers at each depth interval to insert soil sampler and retrieve the soil sample or to carry out in-situ test. When borehole is advanced using continuous flight hollow stem-augers, the augers provide support to the borehole and sampling and testing can be carried out at the base of the borehole without augers extraction. Wash boring technique involves using pipe casing to support the hole and washing out the soils dislodged by drilling bit. The casing is advanced to the bottom of the hole, as required.

c) Test Pits

Subsurface conditions at shallow depth can be explored by excavating test pits. Shallow test pits can be excavated with hand tools. Mechanical equipment with front-end loaders, are required for deep pits or long trenches. The side of the excavation should be sampled, logged, and photographed to provide picture of the materials encountered. Care should be taken when excavating in soft clays, loose sand and close to the water table. General comments on test pit investigation are provided in Table A3.5.2.

d) In-situ Testing

- **Standard Penetration Test (SPT)**

Standard Penetration Test is the most commonly used in-situ test today. The test consists of driving a 51mm O.D. split-barrel sampler, driven into the soil with a 63.5kg weight having a free fall of 760mm. The blows required to drive the split-barrel sampler a distance of 300mm, after an initial penetration of 150mm, is referred to as the SPT N value. From the Standard Penetration Tests the soil sample for classification purposes are retrieved and the N values give a quantitative guide to the in-situ engineering properties and foundation design. Details of the split barrel sampler and procedure for the Standard Penetration Test are described in ASTM D1586. It should be pointed out that the results of the SPT are greatly affected by the sampling and drilling operations. The evaluation of the test results should be undertaken by an experienced geotechnical engineer.

- **Dynamic Cone Penetration Test (DCPT)**

The dynamic cone penetration test involves continuous driving the a cone and rod into the ground by a dropping weight. The number of blows for each 300mm is recorded. The most common equipment consist of a 44.4mm rods (i.e., same as used in Standard Penetration Test) and the drive weight and fall is the same as in the SPT. There is a wide range of cones that can be used in the test, with diameter of cones ranging form 50mm to 100mm. In test is used to delineate the boundary between the different soils and should not be used for quantitative of the soil parameters or densities. The main advantage of the dynamic cone penetration test is that it is fast, economical and a continuous resistance versus depth profile is obtained.

- **Cone Penetration Test (CPT)**

Many static cone penetrometers consisting of a slender metal rod equipped with a cone shaped tip are available. A cone point with a $10cm^2$ base area, apex angle of 60^0 has been specified in American Standards (ASTM D3441). A friction sleeve with an area of $150cm^2$ is located above the cone point. A pore-pressure gauge is located at the base of the cone. The cone tip is advanced by an electrical load-cell built-in within the penetrometer that record continuously the point pressure, q_c, and the local side shear, f_s. Pore pressure measurement during testing provides information on the stratification of the soil. The most important advantage of the electric cone penetration test is its repeatability and accuracy in determination of the soil profile. Extensive use is made of the friction ratio, defined as the ratio between the point pressure and the side shear, and excess of pore pressure measured during penetration as a mean of soil classification. Empirical correlations have been proposed for relating the results of the cone penetration test to the Standard Penetration Test, to soil parameters, such as shear strength, density index, compressibility and modulus.

- **Field Vane Test (FVT)**

The field vane test is used for the in-situ determination of the undrained shear strength of intact fully saturated soft to firm clays; the test is not suitable for cohesionless soils. Details of the test are given in ASTM D2573. The vane equipment consists of a vane blade, a set of rods, and a torque measuring apparatus. The vane blade may be rectangular or tapered and usually have a height to diameter ratio of 2; typical dimensions are 100mm by 50mm. The vane and rod are pushed into the clay below the bottom of the borehole to a depth of at least three times the depth of borehole diameter to avoid disturbance of the clay. Torque is applied gradually to the upper end of the rod by means of suitable equipment until the clay fails in shear due to rotation of the vane. The test interpretation is based on the simplified assumption of a cylindrical failure surface corresponding to the periphery of the vane blade. The undrained shear strength can be calculated from the measured torque. Assuming uniform shear distribution along the top and bottom of the failure cylinder and the vane height (H) to vane diameter (D) ratio of 2, the undrained shear strength for the rectangular vane can be calculated from the expression:
$C_u = T/3.66 D^2$

Vane test are usually carried out at intervals over the depth of interest. If, after the initial test, the vane is rotated rapidly through several revolutions the clay will become remoulded and the shear strength in this condition can be determined, if required.

- **Pressuremeter Test (PMT)**

Pressuremeters are used to measure the in-situ deformation and strength properties of a wide variety of soils. Two major types of pressuremeters have been developed:

- pre-bored pressuremeters or the Menard type, and
- self- boring pressuremeters.

Most devices use a cylindrical rubber tube that closely fits the inside of the borehole. The tube is inflated with a fluid under pressure, and the expansion of the hole is measured by the volume of the fluid that exceed that required to fill the original hole. Self-boring pressuremeters can be equipped with a pore-pressure transducer. A plot of fluid volume/hold diameter as a function of pressure is used to compute the in-place deformation characteristics of the soil.

All types of pressuremeter tests are sensitive to the method of probe installation and testing. The tests should be carried out by trained personnel.

e) Geophysical Survey Techniques

- **Microgravity Survey**

The microgravity technique relies on differences in the physical properties of the subsurface materials, specifically density, to alter the magnitude of the earth's gravitational field. Measurements of these relatively small changes in the gravitational field are used to map mass deficient areas, such as those found over cavities and voids. A microgravity survey involves taking readings with a gravimeter instrument at stations on a grid over the area of interest. The sensor in a gravimeter is either a zero-length spring mechanism, or a vibrating crystal. For detecting karst, the gravimeter used should have a sensitivity of 1 microgal or better, and the survey station interval should be no greater than 10 metres. The gravimeter is first levelled at the station, the reading taken and the exact time of the reading noted. The location and elevation of each station must be surveyed to an accuracy of +/-2 and +/-0.2 centimetres, respectively. Readings should be made hourly at a base station, so as to later correct for instrument drift and gravimetric variations due to earth tides.

The data at each station must then be corrected for latitude, elevation and drift. In areas where the topographic variation is significant, a terrain correction is also applied to the data. As part of the correction for elevation and terrain, the density of the soil at surface must be known or estimated. After these corrections, trends in the data due to regional changes rock type are removed. The resulting residual gravity potential field is then analyzed to identify gravity lows which could be indicative of karst. The density of the underlying rock must be known or estimated if modelling of these data to estimate depth and size of karst features is to be performed. A simpler approach to estimating depth to karst features is to assume a simple geometric shape of the feature, examine the half-width of the anomaly, and estimate the depth based on an analytical equation.

- **Seismic Reflection**

A high resolution seismic reflection survey consists of imparting a source of seismic energy at or near the surface, and recording, with a seismograph, the amplitude of seismic waves at or near the surface with velocity transducers (i.e., geophones) as a function of time.

The seismic source must be broad-band and its wavelet repeatable. Typical sources in ascending order of effectiveness are:

- a sledge hammer and steel plate;
- small explosive charges;
- accelerated weight drops;
- seismic shot guns; and
- seismic air guns and vibratory sources.

The geophones used should have a natural frequency of 40Hz or higher, and should be coupled to the earth with spikes that are at least 10 centimetres long to ensure that a broad-band of frequencies can be recorded. To detect karst features, a geophone interval of 2 to 5 metres should be used. Geophones can be arranged in a line (i.e., 2D survey) or on a grid (i.e., 3D survey).

For each given source location, the source excitation and the recording at geophones can be repeated or stacked to enhance the signal/noise ratio of the data. The seismograph is used to record seismograms for 12 to 240 geophones at once. During the course of a survey, the source is deployed at a spatial interval on the order of the geophones spacing used, and records made at each position.

The resulting data are then processed to produce a seismic reflection profile. Gain compensation for geometric spreading and attenuation is first applied. The source wavelet is then collapsed by deconvolution. The reflections of seismic energy from stratigraphic horizons and discrete features are then identified on the field records and the seismic velocity field as a function of travel time is then inferred from analysis of the reflections. The seismic traces are then grouped by common midpoint and a normal moveout correction is applied to produce a group of equivalent zero-offset traces. These groups of traces are then stacked to produce the final profile.

Diffraction patterns due localized features such as karsts are then identified on the final profile.

- **Seismic Refraction**

The equipment used in seismic refraction is similar to that used for seismic reflection, except that geophones with a lower natural frequency are used, typically between 4 and 40Hz. Seismic refraction is generally carried out on a line, producing a 2D model of the depth to underlying strata. The time of the first arrival of seismic energy as a function of geophone position is determined from the seismograms. For a given geophone array, seismograms are recorded for shot positions at the ends, centre and off the ends of the geophone array.

The premise of seismic refraction is that seismic velocity increases as a function of depth, and seismic refraction cannot be applied if this premise is not valid. For each seismogram obtained, first arrivals are picked and graphed as a function of geophone position. These graphs are analyzed and a layered earth model derived based on one of the three following methods in ascending order of accuracy/complexity:

- the dipping layer method;
- the delay time metho; and
- the generalized reciprocal method.

The latter two produce a depth to strata model at the spatial resolution equal to the geophone station spacing and both are suitable to delineate depressions in the bedrock surface created by the collapse of karst features.

- **Ground Penetrating Radar (GPR)**

A ground penetrating radar (GPR) survey consists of transmitting an electromagnetic pulse having a centre frequency between 10 and 1000MHz into the earth via an antenna and detecting reflected pulses via a second antenna as a function of time. Radar waves are reflected at the contact between materials which differ in dielectric permittivity; dielectric permittivity determines the velocity of propagation and attenuation of radar waves in a media.

The transmitter and receiver antennae are identical; they are dipoles, where the length of the dipole determines the centre frequency of the radar wave. The antennae are generally deployed at a fixed separation on the order of the dipole length, and are oriented with the axis of the dipole perpendicular to the survey line. The survey consists of recording a radargram trace at discrete positions along the line, generally at spatial intervals on the order of 1/10 to 1/2 the dipole length.

The radar traces are then processed to produce a final section. First, the traces are time shifted to align the first arrival (i.e., the air wave) to "zero time". Next, the traces are high pass filtered to remove system "wow". Finally the traces are gain compensated for geometric spreading and attenuation. The final radar section is then analyzed to identify diffraction patterns which may be due to reflections from karst features. It should be noted that lower frequency radar waves will penetrate deeper into the earth but their longer wavelength give them less vertical resolution. Electrically conductive soils will rapidly attenuate radar wave energy and in these conditions the limited penetration achieved by GPR may preclude its usefulness.

Table A3.5.1 - Types of Test Borings

BORING METHOD	PROCEDURE UTILIZED	APPLICABILITY
Auger Boring	Hand or power operated augering with periodic removal of material. In some cases continuous auger may be used requiring only one withdrawal. Changes indicated by examination of material removed. Casing generally not used.	Ordinarily used for shallow explorations above water table in partly saturated sands and silts, and soft to stiff cohesive soils. May be used to clean out hole between drive samples. Very fast when power- driven. Large diameter bucket auger permits examination of hole. Hole collapses in soft soils and sandy soils below groundwater table.
Hollow-Stem Flight Auger	Power operated. Hollow stem serves as a casing.	Access for sampling (disturbed or undisturbed) or coring through hollow stem. Should not be used with plug in granular soil. Not suitable for undisturbed sampling in sand and silt below groundwater table.
Wash-Type Boring	Chopping, twisting, and jetting action of a light bit as circulating drilling fluid removes cuttings from holes. Changes indicated by rate of progress, action of rods, and examination of cuttings in drilling fluid. Casing used as required to prevent caving.	Used in sands, sand and gravel without boulders, and soft to hard cohesive soils. Usually can be adapted for inaccessible locations, such as on water, in swamps, on slopes, or within buildings. Difficult to obtain undisturbed samples.
Rotary Drilling	Power rotation of drilling bit as circulating fluid removes cuttings from hole. Changes indicated by rate of progress, action of drilling tools, and examination of cuttings in drilling fluid. Casing usually not required expect near surface.	Applicable to all soils except those containing much large gravel, cobbles, and boulders in which case it may be combined with coring. Difficult to determine changes accurately in some soils. Not practical in inaccessible locations for heavy truck-mounted equipment, but track-mounted equipment is available. Soil samples and rock cores usually limited to 150mm diameter.
Percussion Drilling (Churn drilling)	Power chopping with limited amount of water at bottom of hole. Water becomes a slurry that is periodically removed with bailer or sand pump. Changes indicated by rate of progress, action of drilling tools, and composition of slurry removed. Casing required except in stable rock.	Not preferred for ordinary exploration or where undisturbed samples are required because of difficulty in determining strata changes, disturbance caused below chopping bit, difficulty of access, and usually higher cost. Sometimes used in combination with auger or wash borings for penetration of coarse gravel, boulders, and rock formations. Could be useful to probe cavities and weakness in rock by changes in drill rate.
Rock Core Drilling	Power rotation of a core barrel as circulating water removes ground-up material from hole. Water also acts as coolant for core barrel bit. Generally hole is cased to rock.	Used alone and in combination with other types of boring to drill weathered rocks, bedrock, and boulder formations.
Wire-Line Drilling	Rotary type drilling method where the coring device is an integral part of the drill rod string which also serves as a casing. Core samples obtained by removing inner barrel assembly from the core barrel portion of the drill rod. The inner barrel is released by a retriever lower by a wire-line through drilling rod.	Efficient for deep hole coring over 30 m on land and offshore coring and sampling.

Table A3.5.2 - Summary of Common In-Situ Tests ((adopted from Canadian Foundation Engineering Manual, 3rd edition, 1992)

TYPE OF TEST	BEST SUITED TO	NOT APPLICABLE TO	PROPERTIES THAT CAN BE DETERMINED	REFERENCES
Standard Penetration Test (SPT)	Sand	Soft to firm clays	Qualitative evaluation of compactness. Qualitative comparison of subsoil stratification.	ASTM D 1586-84 Peck et al. (1974) Tavenas (1971) Kovacs et al. (1981)
Dynamic Cone Penetration Test (DCPT)	Sand	Clay	Qualitative evaluation of compactness. Qualitative comparison of subsoil stratification.	ISSMFE (1977b, 1989) Ireland et al. (1970)
Cone Penetration Test (CPT)	Sand, silt, and clay	Gravels	Continuous evaluation of density and strength of sands. Continuous evaluation of undrained shear strength in clays.	Sanglerat (1972) Schmertmann (1970, 1978) ISOPT (1988) ISSMFE (1977b, 1989) ASTM D3441-79 Robertson and Campanella (1983a, b)
Field Vane Test (FVT)	Clay	Sands and Gravels	Undrained shear strength	ASTM D2573-72 Bjerrum (1972) Schmertmann (1975) Wroth and Hughes (1973) Wroth (1975)
Pressuremeter Test (PMT)	Soft rock, dense sand, gravel, and till	Soft sensitive clays loose silts and sands	Bearing capacity and compressibility	Menard (1965) Eisenstein and Morrison (1973) Baguelin et al. (1978) Ladanyi (1972)

Table A3.5.2 - Summary of Common In-Situ Tests (adopted from Canadian Foundation Engineering Manual, 3rd edition, 1992)

TYPE OF TEST	BEST SUITED TO	NOT APPLICABLE TO	PROPERTIES THAT CAN BE DETERMINED	REFERENCES
Flat Dilatometer Test (DMT)	Sand and clay	Gravel	Empirical correlation for soil type, K_0, over-consolidation ratio, undrained shear strength, and modulus	Marchetti (198) Campanella and Robertson (1982, 1991) Schmertmann (1986)
Plate Bearing Test and Screw Plate Test	Sand and clay		Deformation modulus. Modulus of subgrade reaction. Bearing capacity.	ASTM D 1194 -72
Permeability Test	Sand and gravel		Evaluation of coefficient of permeability	Hvorslev (1949) Sherard et al. (1963) Olson and Daniel (1981) Tavenas et al. (1983a, b)

Table A3.5.3 - Summary of Common Index Properties Tests and Standards (adopted from Canadian Foundation Engineering Manual, 3rd edition, 1992)

TEST	REFERENCE FOR STANDARD TEST PROCEDURE	VARIATIONS FROM STANDARD TEST PROCEDURES, SAMPLE REQUIREMENTS	SIZE OR WEIGHT OF SAMPLE FOR TEST [(1), (2)]
Moisture content of soil	ASTM D2216	None. (Test requires unaltered nature moisture content.)	As large as convenient
Moisture, ash, and organic matter of peat materials	ASTM D2976	None.	
Dry unit weight	None.	Determine total dry weight of a sample of measured total volume. (Requires undisturbed sample.)	As large as convenient.
Specific gravity: (relative density)			
Material smaller than No. 4 (4.75 mm) sieve size	ASTM D854	Volumetric flask preferable; vacuum preferable for de-airing.	25 gm to 50 gm for fine-grained soil; 150 gm for coarse-grained soils.
Material larger than No. 4 (4.75 mm) sieve size	ASTM C127	None.	500 gm.

Table A3.5.3 - Summary of Common Index Properties Tests and Standards (adopted from Canadian Foundation Engineering Manual, 3rd edition, 1992)

TEST	REFERENCE FOR STANDARD TEST PROCEDURE	VARIATIONS FROM STANDARD TEST PROCEDURES, SAMPLE REQUIREMENTS	SIZE OR WEIGHT OF SAMPLE FOR TEST (1), (2)
Atterberg Limits:		Use fraction passing No. 40 (0.425 mm) sieve; material should not be dried before testing.	
Liquid Limit	ASTM D423	None.	100 gm to 500 gm.
Plastic Limit	ASTM D424	Ground glass plate preferable for rolling.	15 gm to 20 gm.
Shrinkage Limit	OCE 1970	In some cases a trimmed specimen of undisturbed material may be used rather than a remoulded sample.	30 gm.
Gradation:			
Sieve analysis	ASTM D422	Selection of sieves to be utilized may vary for samples of different gradation.	500 gm for soil with grains to 9.5 mm; to 5,000 gm for soil with grains to 75 mm.
Hydrometer analysis	ASTM D422	Fraction of sample for hydrometer analysis may be that passing No. 200 (0.075 mm) sieve. For fine-grained soil entire sample may be used. All material must be smaller than No. 10 (2.0 mm) sieve.	65 gm for fine-grained soil; 115 gm for sandy soil.
Corrosivity:			
Sulphate content	ASTM 1965	Several alternative procedures.	Soil/water solution prepared, see reference.
Chloride content	ASTM 1965	Several alternative procedures.	Soil/water solution prepared, see reference.
pH	ASTM D1293	Reference is for pH of water. For mostly solid substances, solution made with distilled water and filtrate tested; standard not available.	
Resistivity (laboratory)	none	Written standard not available. Follow guidelines provided by manufacturers of testing apparatus.	
Resistivity (field)	NBS, Circular C450	In situ test procedures.	

1) Samples for tests may either be disturbed or undisturbed; all samples must be representative and non-segregated; exceptions noted.
2) Weights of samples for tests on air-dried basis.

Table A3.5.4 - Requirements for Structural Properties (adopted from Canadian Foundation Engineering Manual, 3rd edition, 1992)

TEST	REFERENCE FOR STANDARD TEST PROCEDURE	VARIATIONS FROM STANDARD TEST PROCEDURES	SIZE OR WEIGHT OF SAMPLE FOR TEST (UNDISTURBED, REMOULDED, OR COMPACTED
Permeability:			
Constant Head (moderately permeable soil)	Lambe 1951, OCE 1970		Sample size depends on maximum grain size, 40 mm diameter by 350 mm height for silt and fine sand.
Variable Head	Lambe 1951, OCE 1970	Generally applicable to fine-grained soils.	Similar to constant head sample.
Constant Head (coarse-grained soils)	ASTM D2434	Limited to soils containing less than 10% passing No. 200 (0.075 mm) sieve size.	Sample diameter should be ten times the size of the largest soil particle.
Capillary Head	Lambe 1951	Capillary head for certain fine-grained soils may have to be determined indirectly.	200 gm to 250 gm dry weight.
Consolidation:			
Consolidation	Lambe 1951	To investigate secondary compression, individual loads may be maintained for more than 24 hours.	Diameter preferably 63 mm or larger. Ratio of diameter to thickness of 3 to 4.
Swell	ASSHTO T258		
Collapse Potential	Jennings et al 1975		Two specimens for each test, with diameter 63 mm or larger. Diameter to height ratio 3 to 4.
Shear Strength:			
Direct Shear	ASTM D3080	Limited to tests on cohesionless soils or to consolidated shear tests on fine-grained soils.	Generally 12 mm thick, 75 mm by 75 mm or 100 mm by 100 mm in plan, or equivalent.
Unconfined Compression	ASTM D2166		Similar to triaxial test samples.
Triaxial Compression:			
Unconsolidated-undrained	ASTM D2850		Ratio of height to diameter should be less than 3 and greater than 2. Common sizes are:
Consolidated-undrained	Lambe 1951, Bishop et al 1962	Consolidated-undrained tests may run with or without pore pressure measurements	71 mm diameter, 165 mm high. Larger sizes are appropriate for gravelly materials to be used in earth embankments.
Vane Shear			Block of undisturbed soil at least three time dimensions of vane.

4.0 ORGANIC SOILS

Organic and peat soils are normally formed in an environment where the process of humification is occurring. Humification involves the process of transformation or decomposition of vegetative and/or organic materials into humus (i.e., dark brown to black organic matters similar to topsoil). The rate of humification is controlled by other conditions such as the level of saturation, the presence of inorganic matierlas, soil moisture content, temperature and the state of anaerobic or aerobic soil conditions (i.e., absence or presence of oxygen). The humification process of organic and peat soils also results in the release of various humic acids to the groundwater system and in the creation of methane gas which is highly explosive.

The determination of whether a soil is considered to be organic is based on the percentage loss by weight of the soil when heated. If 5 to 80 percent of the weight is lost when heated, the soil is considered to be organic (Johnson, 1995). By definition, there is a wide array of soil types that can be considered to be organic.

Peat materials are the most commonly encountered type of organic soils next to topsoil. Peat soils are defined as a complex composition of inorganic minerals and organic materials, derived from leaves, plant remains and wood fragments, in varying stages of decomposition (i.e., humification). If the peat soil is in a non-humified or slightly humified state, the soil is relatively fibrous. In a humified state, the peat soil is transformed into an amorphous (i.e., having no definite shape or form) mushy mass without any discernible structure (Johnson, 1995). In addition, peat soils are easily eroded by surface water and/or groundwater flow and are highly compressive (i.e., limited ability to support structures).

4.1 Geological Setting

Organic terrain covers most of the northern part of Ontario (i.e., Hudson Bay Lowland) and about 10 per cent of the southern portion of Ontario (i.e., Canadian Shield), (see Figure 4.1). The distribution of organic terrain coincides approximately with the boreal forest region, which is delineated by the range of black spruce (Redforth 1973). Typical of these deposits are swamp, fen, bog and tundra peat accumulations, developed primarily on flat, poorly drained glacial lake and marine sediments.

Most of the surface deposits of peat have accumulated since the last ice while some buried peats may have been developed during inter-glacial periods. During the process of deglaciation, there were several halts, re-advances and possible surges of the ice sheet margin which resulted in the development of numerous ice-dammed lakes. These glacial lakes often formed barriers to plant migration during the late glacial episode. When these lakes drained the exposed large areas of lacustrine sediments provided suitable substrates for peat/muskeg expansion.
Peats developed in post-glacial lakes and marshes are typically interbedded with silts and muds or they may be associated with salt marshes. Based on the muskeg studies (Redforth 1975), the large scale muskeg expansion in the northern Ontario was delayed until 6000 to 5000 years age, when the climate began to grow moister and cooler. In southern Ontario extensive muskeg developed in late-glacial time on lake sediment plains. However, as the climate warmed, the rates of peat deposition and decomposition became about equal over many areas, so that no further peat accumulation has occurred during the last 8000 years. Where the rate of decomposition of peat was greater than the deposition, the peat deposit disappeared. Figure 4.2 depicts muskeg development/peat accumulation.

The most commonly encountered organic deposit is peat. Peat is an accumulation of partially decomposed and disintegrated plant remains which have been fossilized under conditions of incomplete aeration and high water content. Physio-chemical and biochemical processes cause this organic material to remain in a state of preservation over a long period of time.

In the northern parts of Ontario, the thickness of the organic deposits is typically up to 5m, however thickness of as much as 10m have been found in some locations (e.g., Fort Frances area). Greater depths of organic deposits can be encountered in any part of Ontario where deep river valleys have been infilled since glacial times.

Many of the organic deposits within the Hudson Bay Lowland have high ice content both as interstitial ice and thin icy layers, and as major segregated forms (i.e., palsa and ice wedges). The thermal insulation provided by peat has preserved ground ice conditions in substrates which would not otherwise be within the permafrost zone.

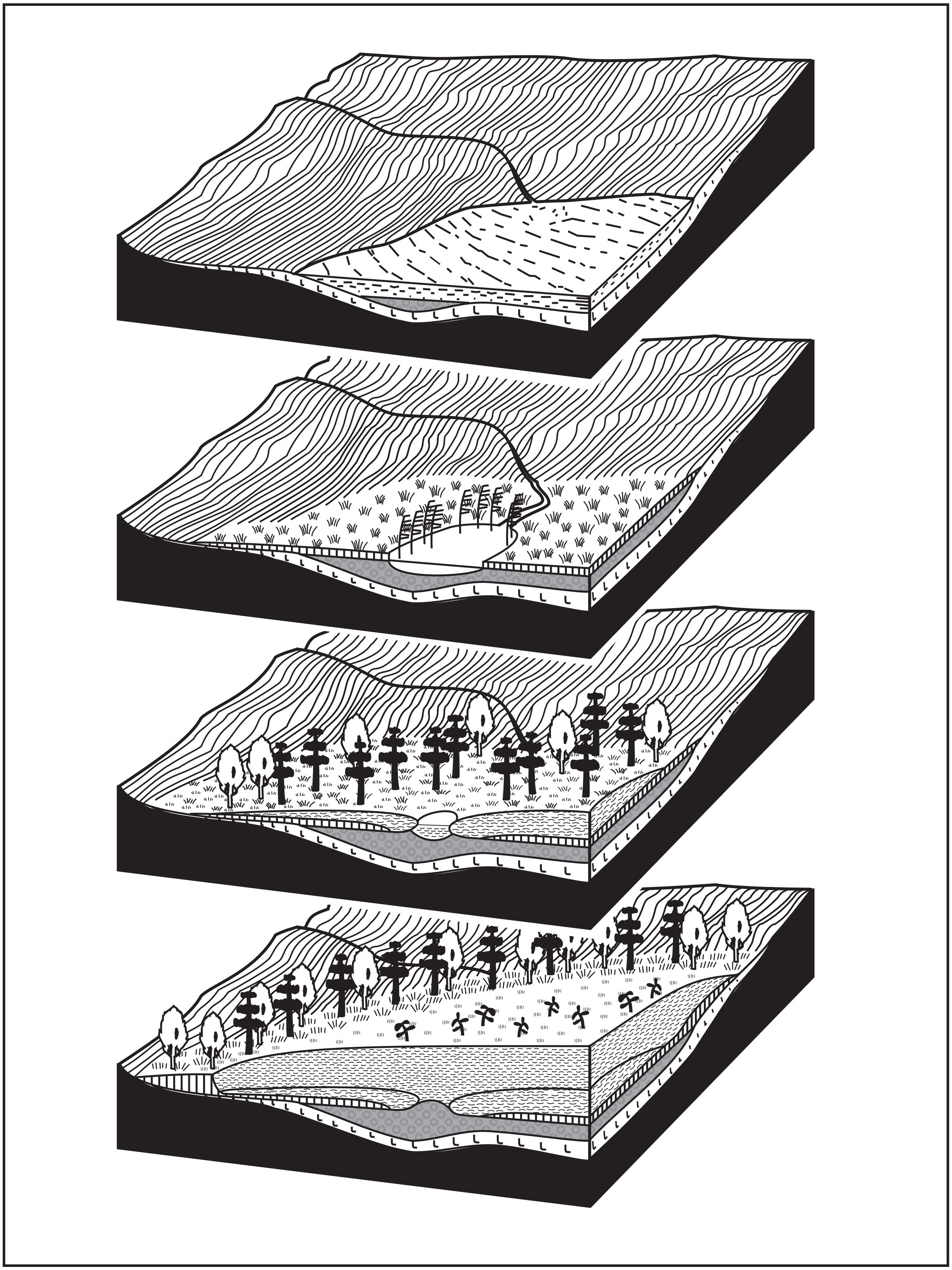
Figure 4.1: Geographic Location of Organic Deposits

LEGEND:

Organic deposits

Isolated areas of organic deposits present throughout.

Figure 4.2: Diagram of Muskeg development (after Kivinen, 1948)



4.1.1 Definition of Organic Soils

In the Canadian Engineering Manual (3rd edition, 1992, page 119), organic soils are defined as "soils containing significant amounts of organic materials, either as colloids or in fibrous form .....". Organic soils "are generally weak and will deform significantly under load. Such soils include peat and organic silts and clays and are typical of many estuarine, lacustrine, or fluvial environments." Generally, a soil is considered to be organic if it contains greater than 5 per cent organic matter determined by heating the soil to a specified temperature (e.g., typically 85^{o} C or 100^{o}C).

Organic soils are formed in an environment where transformation or decomposition of vegetative or organic materials into humus occurs. The decomposition process occurs within an organic terrain/muskeg. Muskeg is referred to as "a terrain composed of a living organic mat of mosses, sedges, grasses, with or without tree and shrub growth, underlain by a usually highly compressible mixture of partially decomposed and disintegrated organic material commonly known as "peat" or "muck" (MacFarland 1959). In engineering practice the term "muskeg deposit" includes any underlying peat, organic clay, soft marl, etc., which are likely to cause construction problems".

Peat deposits can be divided into two basic groups, namely, amorphous-granular and fibrous peat. The latter can be further subdivided into fine or coarse fibrous peat. In general, the amorphous peats have a high content of colloidal fraction, holding most of their water in an adsorbed rather than a free state; the adsorption occurring around the particles. The fibrous type of the peat is composed mainly of woody fibres with various proportions of colloidal matter. Most of the water contained in the fibrous peat is free water. Appendix A4.1 outlines the classification of peat based on the structural components in the peat matrix and Appendix A4.2 outlines the properties of organic soils.

4.2 Type Of Hazard

4.2.1 Bearing Capacity

Design and construction in organic terrain requires very careful attention to specific problems that arise when the underlying soils consist of organic deposits. The main concerns with construction on organic deposits are the low bearing capacity of the deposits and the potential for large settlement resulting from consolidation of the material due to increase in applied load.

Strength of organic deposits has been recognized as being influenced by factors such as structural disturbance, initial effective stress conditions, and anisotropy (Ladd and Lambe 1963; Sandroni 1983) and is highly variable and difficult to define. Deformation characteristics of the organic deposits are also highly variable. For this reason, construction works on organic deposits require detailed study in order to assess the impact of the works with respect to potential settlement and instability. Settlement characteristics can be determined either from laboratory consolidation testing carried out on site specific organic soil samples or from field settlement observations. Typically, when comparing laboratory and field data, the values of settlement observed in the field are much higher than those observed in laboratory testing (Lefebvre et al 1983). The discrepancy in the results is related to the variation in the boundary conditions, changes in water level, variation in the initial void ratio and compressibility.

4.2.2 Gas Occurrence

The organic matter which is present in the organic soils or peat undergoes a very slow decomposition accompanied by the production of methane (i.e., marsh gas) with lesser amounts of nitrogen and carbon dioxide. In deposits containing sulphur, hydrogen sulphur is a product of decomposition.

The presence of gas may create hazard for buildings or parking lots. The possibility of gas escaping through fissures, holes, borings and burning in contact with atmospheric oxygen has been reported.

4.3 Approaches to Address the Provincial Policy

By the very nature of the organic materials, defining the "areas of provincial interest" is a site specific process. To develop a single approach or standard for defining these areas of provincial interest is impractical and inappropriate given that the size, extent and severity of potential hazards and risks (e.g., structure collapse, release of methane gas) are governed by localized specific conditions.

These types of soils are most likely to be found in and associated with the delineation of wetlands (Policies 2.3) which may also preclude development of the site.

Numerous construction techniques are available when building in organic terrain or on a subgrade consisting of organic deposits. Depending on the specific site conditions information on some general techniques for construction have been provided in this document.

4.4 Site and Field Investigation

Geotechnical site investigation should be carried out to assess the scale, nature and extent of the organic terrain. Site investigation refers to the procedure of determining surface and subsurface condition in an area of proposed construction or study area.

Information about an area is frequently available in the form of geological and topographic maps, and literature on general surface and subsurface conditions of the site. Aerial photographs can also provide considerable information on surface conditions and land forms that are not detectable from ground observations or not shown on the maps.

For designing and planning purposes detailed information on subsurface conditions, groundwater levels, soil and rock types and engineering properties should be obtained by means of on-site subsurface exploration. Typically, such information is obtained by drilling boreholes or excavation of test pits. The subsurface information can be supplemented by use of geophysical investigative methods. In-situ testing or laboratory testing on samples obtained during the investigation is used to determine soil and rock parameters.

4.4.1 Site Investigation

The investigation for a site potentially located within an organic terrain should involve a desk study and a site visit/field survey where the objectives are to:

- establish the geological and geographical extent of the organic terrain;
- identify the type and extent of site hazard and the consequent risk to life property and structure;
- review the techniques for investigating the site; and
- review remedial measures for mitigating the risks to life and property.

This is accomplished through the following:

- review and search of all available published and unpublished geological records;
- review of aerial photographs; and
- site visit/field survey.

a) Review of Geological Records

Soft, highly compressible organic deposits are widely distributed across Ontario. Peat deposits cover most of Northern Ontario and part of Southern Ontario and are associated with flat, poorly drained lands. Information on the locations of organic deposits can be obtained from the Ministry of Northern Development and Mines, Geologic Survey of Canada, Ministry of Natural Resources or other sources.

b) Review of Aerial Photographs

Airphoto identification of organic terrain can be carried out by interpretation of the topography and variations of the characteristic patterns observed on the aerial photographs. Aerial photographs taken during different times of the year and at successive levels of altitude allow examination of the same areas under different climatic conditions and with different vegetal cover.

The following high-altitude patterns can be observed on the airphotos (Muskeg Engineering Handbook 1969):

- patch pattern resembling terrazzo (i.e., Terrazoid); typical to Northern Ontario where permafrost is present on flat terrains,(Figures 4.3 to 4.6);
- skin-like pattern (i.e., Dermatoid); featureless terrain where there is little outcropping of the underlying mineral terrain, typically found south of the permafrost zone;

- marbled pattern (i.e., Marbloid) with the components of the pattern making frequent contact; typical for the terrain in which permafrost exists, but may also occur where permafrost is discontinuous;
- featureless background superimposed with stippling in various arrangements (i.e., Stipploid); this pattern is often observed in the Southern Ontario; and
- pattern composed of a network or reticulum of terrain elements (i.e., Reticuloid); typically observed in Southern Ontario.

The use of low altitude aerial photographs enhance the identification of the organic terrain.

TABLE 4.1

DESCRIPTION OF LOW ALTITUDE (300 m - 1500 m) AIRFORM PATTERNS OVER ORGANIC TERRAIN

Planoid	An expanse lacking textural features; plane
Apiculoid	Fine-textured expanse; bearing projections
Vermiculoid	Striated, mostly coarse textured expanse; featured markings tortuous
Vermiculoid I	Striations webbed into a close net and usually joined
Vermiculoid II	Striations in close association, often foreshortened and rarely completely joined
Vermiculoid III	Striations webbed into an open net, usually joined and very tortuous
Cumuloid	Coarse-textured expanse with lobed or fingerlike "islands" prominent; components shaped like cumulus clouds
Polygoid	Coarse textured expanse cut by intersecting lines: bearing polygons
Intrusoid	Coarse-textured expanse caused by frequent interruptions of unrelated, widely separated mostly angular “islands”; interrupted

Examples of the above airform patterns are shown on Figures 4.7 to 4.14.

Figure 4.3: Terrazzoid Airform Pattern near Churchill area from 1000 ft. Altitude

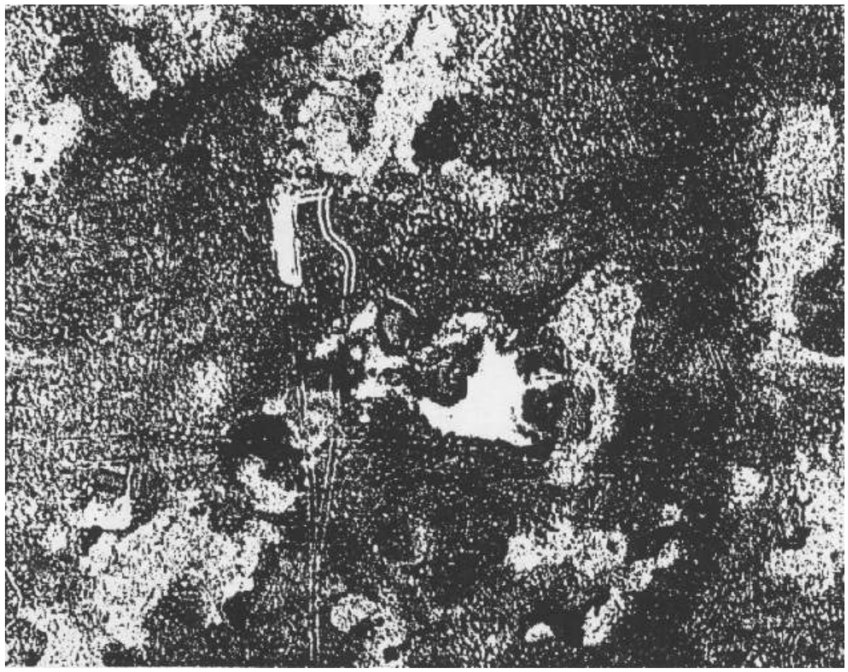


Figure 4.4: Terrazzoid Airform Pattern near Churchill area from 5000 ft. Altitude

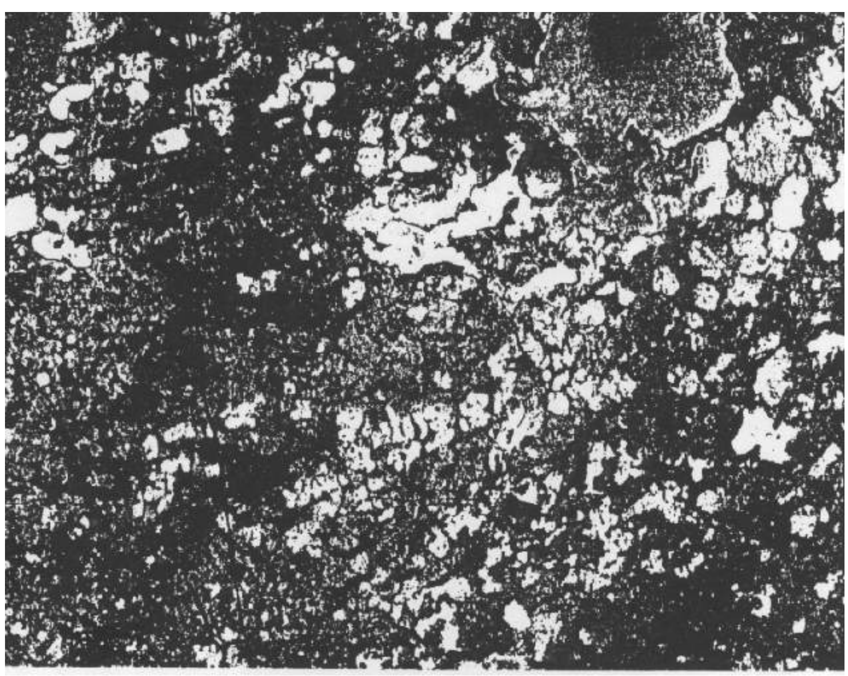


Figure 4.5: Terrazzoid Airform Pattern near Churchill area from 10,000 ft. Altitude

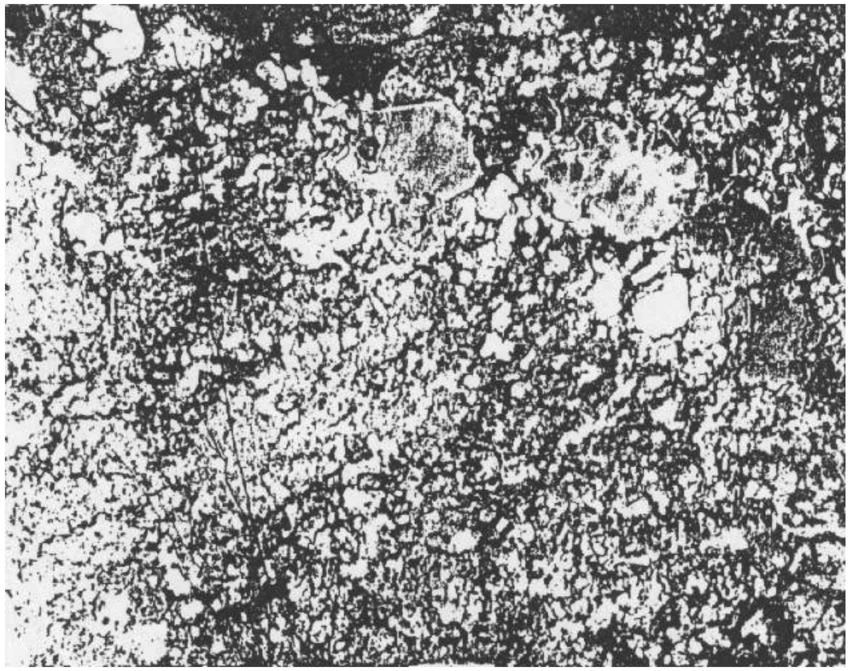


Figure 4.6: Terrazzoid Airform Pattern near Churchill area from 17,000 ft. Altitude

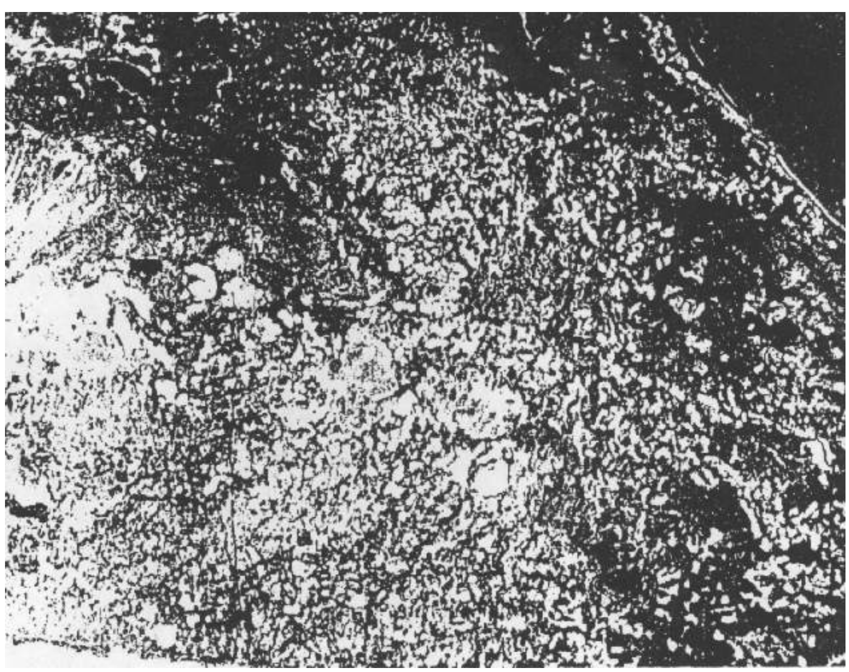


Figure 4.7: Planoid Airform Patterns of Organic Terrain from 1000 ft Altitude

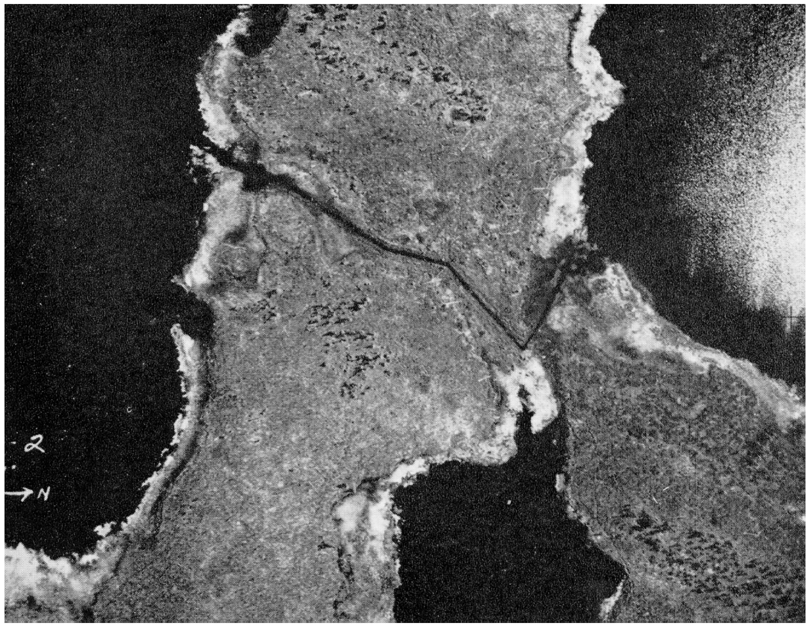


Figure 4.8: Polygoid Airform Patterns of Organic Terrain from 1000 ft Altitude

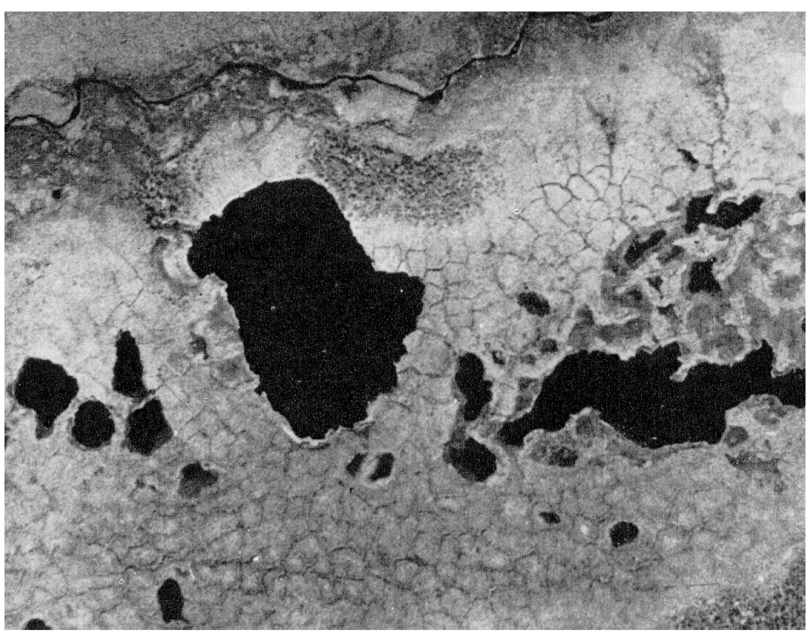


Figure 4.9: Vermiculoid III Airform Patterns of Organic Terrain from 1000 ft Altitude

Figure 4.10: Apiculoid Airform Patterns of Organic Terrain from 1000 ft Altitude

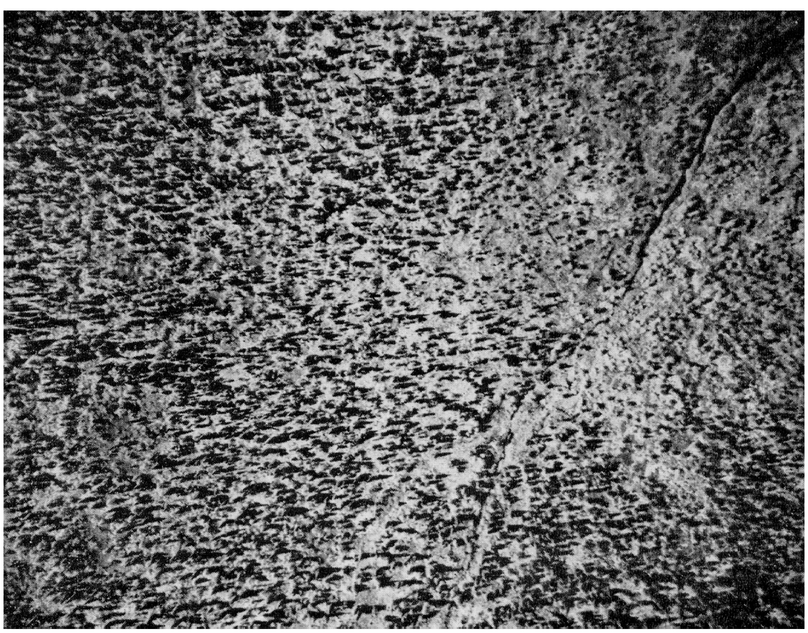


Figure 4.11: Vermiculoid I Airform Patterns of Organic Terrain from 5000 ft Altitude

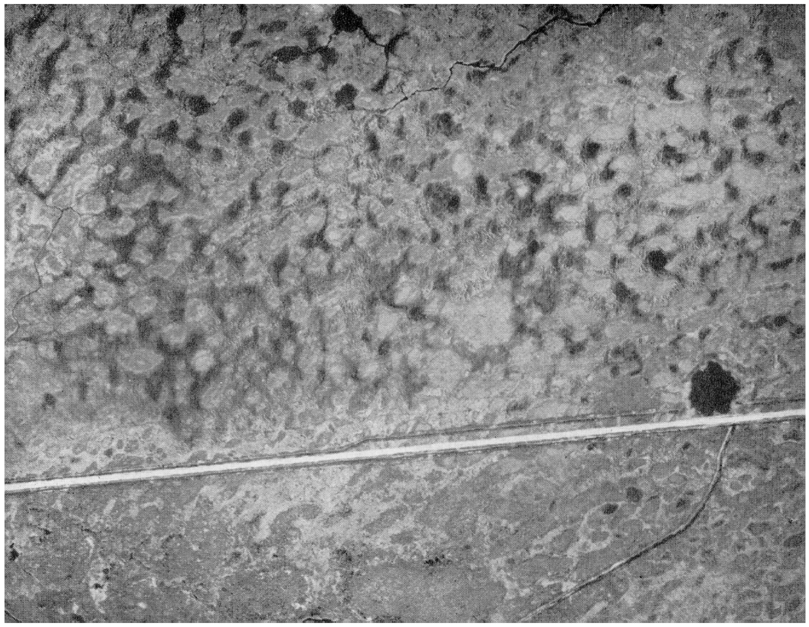


Figure 4.12: Vermiculoid II Airform Patterns of Organic Terrain from 1000 ft Altitude

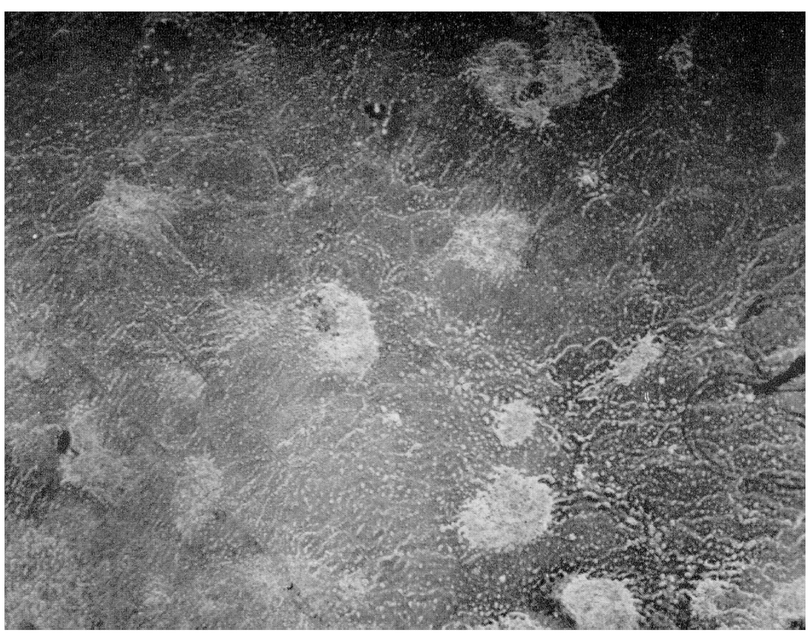
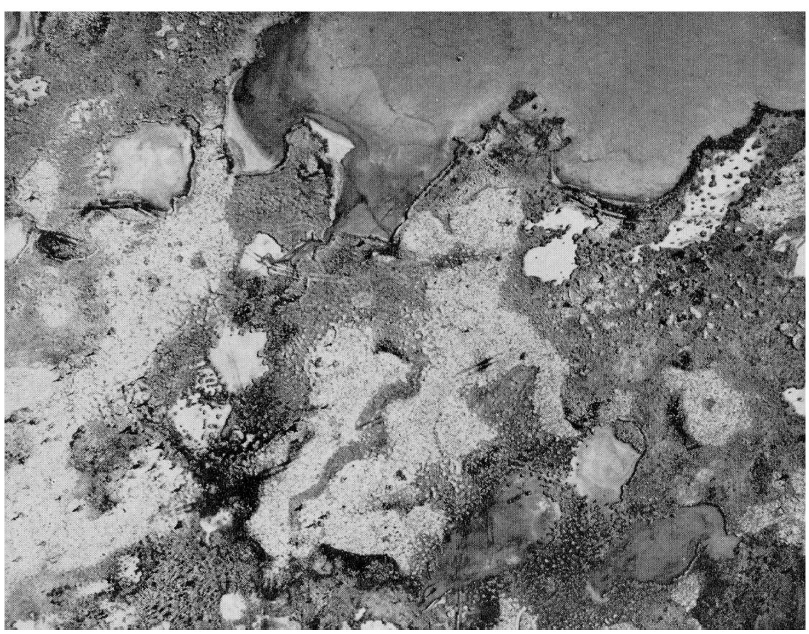


Figure 4.13: Cumuloid Airform Patterns of Organic Terrain from 5000 ft Altitude

Figure 4.14: Intrusoid Airform Patterns of Organic Terrain from 1000 ft Altitude



Review of aerial photographs review often allows to predict basic engineering problems for specific applications that could occur in the organic terrain. Prediction of the engineering problems can be made with respect to the feasibility of a structure, design requirements or construction techniques based on the interpretation of the following site features:

- geomorphology of the site;
- hydrological type of the terrain (e.g., fen, bog, marsh, water retention, flooding, etc.,); and
- topography of the site.

c) Site Visit

Subsequent to a review of available information (i.e., aerial photographs and geological records) a site visit should be conducted to confirm the surficial indicative of the presence of organic deposits. The information to be obtained includes site topography, type and thickness of vegetal cover and surface drainage conditions.

During the site visit, an initial assessment of the thickness of the organic deposit can be made using hand auger equipment pushed manually to refusal. These site observations and initial measurements can provide the basis for establishing the requirements for more detailed field exploration.

4.4.2 Field Investigation

A field investigation consists of mapping, drilling and/or trenching possibly together with geophysical methods and laboratory testing. The objective of the field investigation is to obtain information on the depth, thickness and aerial extent of organic deposits and underlying mineral soil strata. The soil stratigraphy can be determined using various exploratory techniques, including:

- test pit excavation;
- borehole drilling;
- electronic piezocone testing; and
- cone penetrometer testing (CPT).

Description of the various exploratory techniques are provided in Appendix A3.5. Comments on the applicability of the noted above techniques to organic deposits are provided below.

- **Test pit exploration** may be used where the thickness of organic material is limited; however, excavations in organic terrain are frequently hampered by the presence of relative groundwater levels limiting the effectiveness of this method. Borehole drilling usually provides the greatest amount of information and is used to determine the depth, strength and general nature of the organic deposit by obtaining samples of the soil as part of the Standard Penetration Test. In addition, undisturbed Shelby tube sampler may be obtained for specialized laboratory testing for determining consolidation properties.

- **Standard Penetration Test N-value**, often referred to as the blowcount, is frequently used to estimate relative density of non-organic soils, however, it has a limited use in the investigation of organic soils due to compressible nature of these soils. Field vane shear tests are typically used to estimate the in situ undrained shear strength of soft to firm soils, including organic soils.

- **Electronic piezocone testing** may be used in some cases to obtain subsurface data; however, the feasibility of this method is influenced by the fibrous nature of the organic deposits as the depth of penetration may be limited. The piezocone can provide continuous profile of the subsoils by measuring cone tip resistance, sleeve friction and porewater pressures from which the stratigraphy can be determined and the undrained shear strength can be assessed. Cone penetration testing may be used in organic soils to estimate both relative density and/or undrained strength through empirical correlations.

Information from the investigation should allow for detailed design and preparation of plans for construction. Soil sampling and in-situ testing should be performed at several locations across the site. The number of samples collected should be sufficient to obtain information on the shear strength, compressibility and index parameters of each organic deposit and underlying mineral soils.

To obtain a soil sample, the sampler is advanced by driving with a drop hammer or by pushing with a hydraulic piston or jack. The organic soils are very sensitive to disturbance and water content loss. Two types of peat sampler can be used. The choice of the sampler type relates to the properties tested and type of soil.

4.5 Addressing the Hazard

Numerous construction techniques are available when building in organic terrain or on a subgrade consisting of organic deposits. Selection of the method will depend on the anticipated standards of performance of the structure, thickness and mechanical behaviour of the organic deposit, location and drainage requirements.

The following general techniques may be considered for construction when dealing with organic deposits:

- direct construction on the organic deposit (i.e., "floating fill");
- displacement of the organic deposit by controlled loading;
- complete excavation of the deposit and backfilling; and
- use of deep foundations or raft foundation.

Direct construction over organic deposits is feasible when the works involve road embankment construction but is not generally considered to be appropriate when buildings are proposed.

Use of this technique may be considered where the organic deposit is very deep (i.e., where removal is uneconomical) or where there is a thick mat of vegetation at ground surface. The feasible height and configuration of embankments which can be constructed over organic deposits will be dependant on the strength of the soils and on the allowable settlement. Preloading and/or surcharging are techniques which improve the subgrade performance either by inducing consolidation of the organic deposit to reduce the magnitude of long term settlement after completion of the embankment. In some cases, installation of vertical sand drains may increase the rate of settlement and stability of embankments. In general, construction is best carried out in the winter months when the ground is frozen.

Complete excavation and backfilling may be feasible where the organic deposit is shallow and suitable fill materials are easily available. The advantage of this method is that settlement of the structure and/or embankment is minimized. This method is favourable at sites where low cost granular fill is available.

Displacement of organic deposits can be achieved by controlled loading or by blasting. The feasibility of displacement techniques depends on the thickness and strength of the organics. The major problem with these techniques is that there will be variable success in extent of displacement and there is potential for differential settlement.

Deep foundations are necessary where structures which cannot tolerate settlement are to be constructed over organic deposits. Piles should be driven and founded within suitable bearing strata below the organic deposit. Where steel or concrete piles are used, the potential for corrosion must be considered. The use of a raft foundation for support of structures may be considered where there is a relatively uniform thickness of organics and where some settlement can be tolerated. The objective of a raft foundation is to spread the loading over a large area (i.e., minimizing the applied load and the resulting settlement). The raft structure is reinforced to minimize differential settlement.

One of the methods of designing embankments over organic deposits include the provision of berms both to increase stability and to minimize settlement due to displacement during construction. The use of a reinforcing geotextile placed on the subgrade below the embankment fill can improve stability of the embankment with respect to a rotational type of failure, however, even a very strong geotextile will not prevent shear deformation of a subgrade when the embankment is placed on great depths of organics (Rowe et al 1983).

4.6 References: Organic Soils

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APPENDIX A4.1

CLASSIFICATION OF PEAT STRUCTURE

Table A4.1 (Muskeg Engineering Handbook, 1969) outlines the classification of peat based on the structural components in the peat matrix.

TABLE A4.1

CLASSIFICATION OF PEAT STRUCTURE
(Muskeg Engineering Handbook,
University Of Toronto Press. 1969)

PREDOMINANT CHARACTERISTICS	CATEGORY	NAME
Amorphous-granular	1	Amorphous-granular peat
	2	Non-woody, fine-fibrous peat
	3	Amorphous-granular peat containing non-woody fine fibres
	4	Amorphous-granular peat containing woody fine fibres
	5	Peat, predominantly amorphous-granular, containing non-woody fine fibres, held in a woody, fine-fibrous framework
	6	Peat, predominantly amorphous-granular, containing woody fine fibres, held in a woody, coarse-fibrous peat framework
	7	Alternate layering of non-woody, fine-fibrous peat and amorphous-granular peat containing non-woody fine fibres
Fine-fibrous	8	Non-woody, fine-fibrous peat containing a mound of coarse fibres
	9	Woody, fine-fibrous peat held in a woody, coarse-fibrous framework
	10	Woody particles held in non-woody, fine fibrous peat
	11	Woody and non-woody particles held in fine-fibrous peat
Coarse-fibrous	12	Woody, coarse-fibrous peat
	13	Coarse fibres criss-crossing fine-fibrous peat
	14	Non-woody and woody fine-fibrous peat held in a coarse fibrous framework
	15	Woody mesh of fibres and particles enclosing amorphous-granular peat containing fine fibres
	16	Woody, coarse-fibrous peat containing scattered woody chunks
	17	Mesh of closely spaced logs and roots enclosing woody coarse-fibrous peat with woody chunks

The peat categories described in Table A4.1 are illustrated in the Muskeg Engineering Handbook, 1969. Figures 2.1 to 2.17.

APPENDIX A4.2

PROPERTIES OF ORGANIC SOILS

1.0 Properties of Organic Soils

The most of the differences in the physical characteristics of organic soils are attributable to the amount of moisture present within the soil structure. The high water content of organic soils is associated with typically high void ratio. The water content depends on the amount of organic matter and varies depending on the type of organic soil; it may be as high as 600 per cent for amorphous peat. For organic silts and clays, the water content is usually much lower. Organic soils usually have high to extremely high liquid limits; Atterberg limits when plotted on plasticity charts. They are placed below a line named the A-line which defines the upper limits of organic soils. The void ratio of peat is typically less than 5 for dense amorphous peat and organic silt and clays, and may be up to 25 for fibrous peat.

Volumetric shrinkage of peat is usually associated with a drying process and is typically very high ranging between 10 and 75 per cent of the original volume of the peats. Furthermore, in a peat exposed to air, the process of decomposition of the organic matter may continue leading to further volumetric changes (i.e., shrinkage). Permeability of organic soils varies widely and depends on the amount and degree of decomposition of organic matter and degree of consolidation. In general, fibrous peats are significantly more permeable than amorphous peats. Peat deposits are acidic in character, the pH values typically vary between 5.5 and 6.5, although some fen peats are neutral or even alkaline.

Typically, dry densities of drained peat are within the range 6.5 to 12 kN/m^3. The dry density of the peat is influenced by the mineral content and higher values than those quoted can be obtained when peats possess high mineral residues. The relative density of peat has been found to range from as low as 1.1 up to about 1.8, again being influenced by the content of mineral matter.

The shear strength of peat deposits is influenced by moisture content, degree of humification and mineral content. The shear strength of peat increases with an increase in mineral content and degree of humification. Conversely, the higher the moisture content of peat the lower is its shear strength. The strength of peat can be increased by consolidation under applied load or drainage.

Various engineering properties of peat deposits have been correlated with water content and void ratio. Typical correlations of water content and void ratio with organic content, density, strength and compressibility parameters are illustrated in the Muskeg Engineering Handbook, 1969.

Definition of design parameters and prediction of behaviour of peat/organic deposits remain difficult due to heterogeneity of these deposits. Due to the high void ratio and organic content, organic deposits are susceptible to large settlements under applied loadings.

5.0 KARST BEDROCK

In Ontario, areas of unstable bedrock are usually associated wutg karst formations. By definition, karst is a geomorphological environment where water flowing over and through limestone and dolomite bedrock results in the creation of sinkholes, trenches and subsurface caverns.

Complex interactions between a number of variables combine to create karst formations. These include, but are not limited to:

- acidic concentrations of surface water, infiltrating water and groundwater in relation to calcium carbonate and/or magnesium carbonate of the bedrock (i.e., dissolution of the rock);
- proximity of the water table in relation to the bedrock (i.e., distance and depth that water will percolate through the bedrock from the surface to the groundwater table);
- degree and number of bedrock fractures, fissures, joints and faults allowing water movement; and
- presence of impermeable layers either above or below the limestone/dolomite that will influence the flow patterns of water.

Like any geomorphological feature, karst formations can be quite variable in their character and "maturity" depending on the interaction of the above variables over time. Karst formations, characterized by the presence of sinkholes, trenches and caverns of varying sizes and which may or may not be exposed at the surface, if undetected, pose concerns and difficultyfor those involved in planning or development due to their relative instability and unpredictable nature. Thi is particularly true when undetected caverns and trenches fail or collapse and sinkholes appear after contruction of a development. For example, the collpase and sudden appearance of sinkholes in a Florida karst formation resulted in the swallowing of cars, pavements and entire homes in 1981.

5.1 Geological Setting

Karst is a term applied to the topography of soluble bedrock (i.e., carbonate and non-carbonate rocks) exhibiting natural voids developed in response to many and varied processes including weathering cycles, climatic environments, tectonic changes and groundwater table level changes. Karst landforms appears as solution widened joints, bedding planes, fault planes, solution or collapse sinkholes, and subsurface caverns. A term "karren" is often used for a bedrock landform produced in an early stage of bedrock solutioning, where bedrock is exposed. The process of solutioning is very slow and may take thousands or millions of years before karst landforms are fully developed.

The term karst is derived from an area on the border of Slovenia (i.e., formerly part of Yugoslavia) and Italy, where solutioned enlargement in the limestone and dolostone bedrock were first documented. Rapid groundwater flow was noted in that area through the solutioned fractures in the bedrock.

Sedimentary rocks, prone to karst processes are encountered in Ontario mainly in the physiographic regions known as Hudson Bay Lowland, Great Lakes Lowland and Ottawa-St.Lawrence Lowland. In these regions relatively soft, sedimentary rocks (e.g., limestones, shales and sandstones) have, during their long history, been eroded and karstified. During the recession of the last glacier (i.e., Laurentian Ice Sheet) that covered Canada about 14,000 years ago and several retreats and advances of the ice, most of the surficial karst features were destroyed or scraped away or were covered by overburden with the deeper occurring features left intact.

In Ontario, some of the best known karst features are to be found along the base of the Niagara Escarpment in the Salina, Guelph and Lockport Formations and in areas of Kingston and Ottawa.

The Salina Formation consists of interbedded evaporitic carbonates (e.g., dolostones, anhydrite and gypsum) and shales. The karst features develop mainly within the dolostone interbeds. In addition to the Salina Formation, limestone and dolostone, which are prevalent throughout south Ontario and in the extreme north, are susceptible to solutioning processes. The Lockport Dolomite forms the caprock of the Niagara Escarpment and is divided into three members, namely: Gasport, Eramosa and Goat Island Members, of which the latter two members exhibit karst behaviour. The Guelph Formation overlies the Lockport Formation and consists of light cream, medium to fine bedded dolomite which is also prone to dissolutioning.

The following are generalized descriptions of soluble rock types present in Ontario which have some extent of susceptibility to formation of karst features:

- **Limestone,** composed mainly of the mineral calcite, is formed by chemical precipitation of calcium carbonate, by mechanical deposition of calcium carbonate shells or organisms, or by the mechanical accumulation of detrital limestone from a previously existing limestone or shell bank. Limestones are generally grey, light yellow or black in colour and effervesce vigorously in hydrochloric acid. In Ontario, many different soluble carbonate rocks are present, including hard crystalline low permeability limestones, soft high porosity limestones and metamorphosed limestones.

- **Dolomite** or **dolostone** is mainly formed due to alteration (i.e., dolomitization) of limestone or may be formed by chemical precipitation of calcium carbonate. Dolomite is a stratified sedimentary rock, light brown to grey and can be distinguished from limestone by the lack of or poor effervescence in hydrochloric acid. The Lockport Dolomite or Amabel Dolomite forms most of the crest of the Niagara Escarpment in Southern Ontario.

- **Salt beds** are most commonly chemical precipitates of sodium chloride laid down in a sea or basin in which there was excessive evaporation and no outgoing drainage. Salt beds are generally white, grey, yellowish or pink. The salt beds in the Salina Formation in southwestern Ontario form a thickness of nearly 200m; these salt beds are typically at significant depth below ground surface.

- **Gypsum** and **anhydrite** deposits are chemical precipitates of calcium sulphate, generally laid down in association with salt deposits. The colour ranges from white and grey to pink, yellow, brown or dark grey. Gypsum and anhydrite are present within the Salina Formation which outcrops in Southwestern Ontario.

Typically carbonate rocks are much more soluble than other sedimentary rocks. However, there is considerable variation among carbonate units with respect to how prone they are to solutioning. Factors that contribute to solubility are high porosity, high calcite content and low content of less soluble materials (e.g., dolomite, chert, etc.,). The rate of forming of dissolutional fractures depends also on the thickness of bedding. Commonly, thinly bedded limestone will promote large solutional conduits due to a higher frequency of horizontal fractures.

The formation of karst features appears to be prevalent where these rock types are at or close to the ground surface (i.e., minimal overburden cover). There are therefore only isolated locations in Ontario where karst features have been observed, the most notable being in the Niagara Escarpment area, Ottawa area and in the limestone present between Lake Simcoe and Kingston area. Figure 5.1 depicts the distribution of the soluble rocks with known karst features and with potential for karst development.

The karst features identified in Southern Ontario include the areas of disappearing streams extending for several kilometres from Greenville to Tews Falls and Webster Falls (Worthington 1994), springs in the Beaver Valley near Wodehouse and St. Edmund Cave at the tip of the Bruce Peninsula (Bruce Trail Guide 1992) and about 500m in diameter depression located south-west of Hayesland (Worthington 1994). There is some evidence of sinkhole development and disappearing streams to the west of Kingston being particularly notable at the hamlet of Burr Creek and at Collins Creek and in the Ottawa area (see Figure 5.2).

5.1.1 Development of Karst Topography

For karst topography to develop, the following conditions are necessary:

- rock of moderate solubility and high mechanical strength to allow concentrated water flow along the fractures causing enlargement of favourable water pathway;

- chemically aggressive groundwater to recharge the system; the concentration of carbon dioxide in the rain water increases when it passes through the soil and organic matter; and

- groundwater table well below the rock surface with continuous flow through the rock mass to a lower elevation.

The main process in the development of karst features is the widening of fractures (i.e., fissures, joints and faults) by solution of carbonic acid present in soil and groundwater (see Figure 5.3). As water continuously percolates downwards through the rock mass, it dissolves the solids before reaching a watercourse and moving downstream. The surface of the soluble rock (e.g., limestone, dolostone, salt, gypsum, etc.), whether exposed or buried under overburden, becomes conspicuously furrowed and eroded due to the irregular solvent action of the acidic waters (see Figures 5.4 and 5.5).

Figure 5.1: Geographic Location of Karst Bedrock

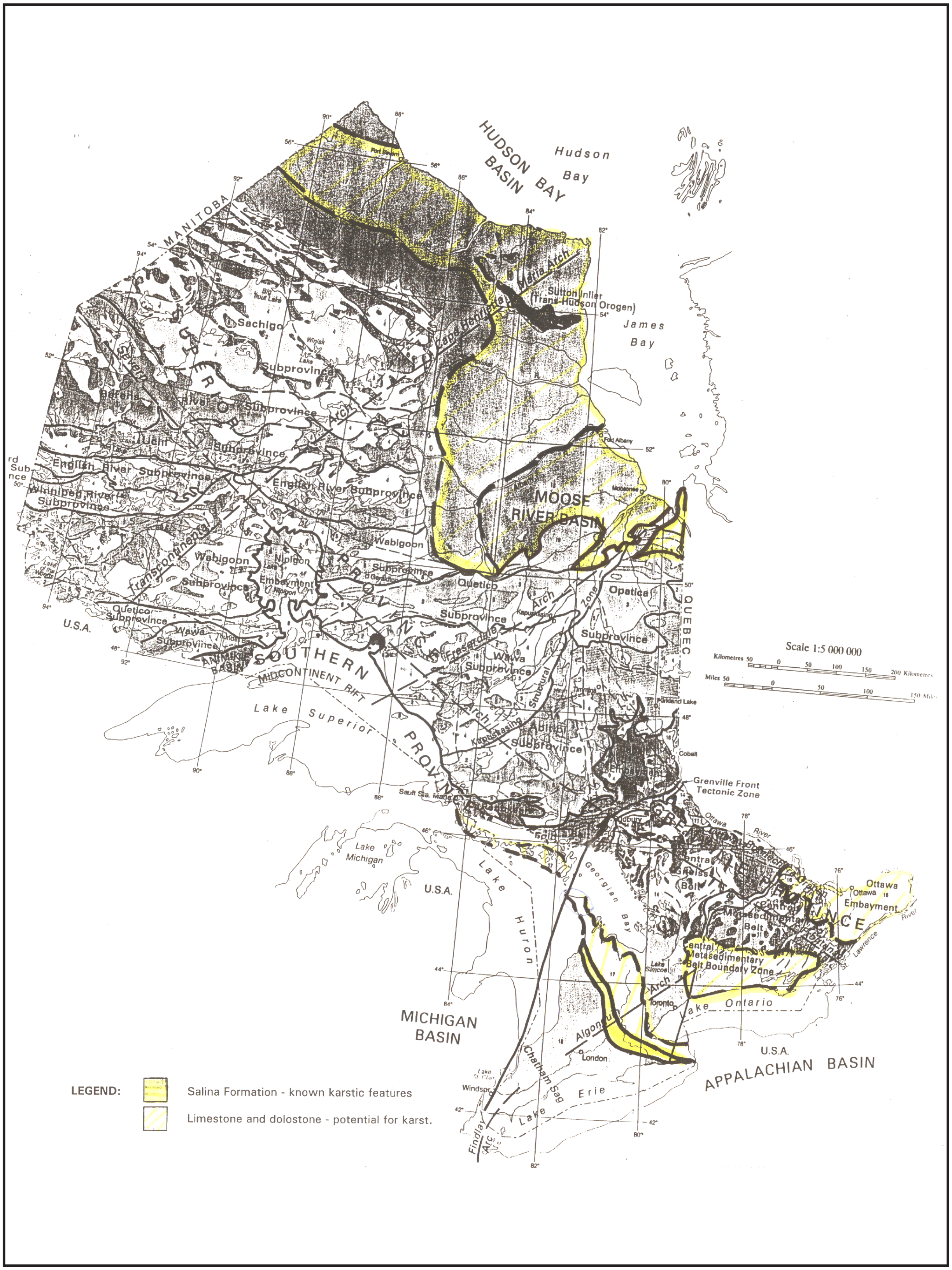


Figure 5.2: Disappearance and Emergence of Flows on Cardinal Creek Located East of Ottawa

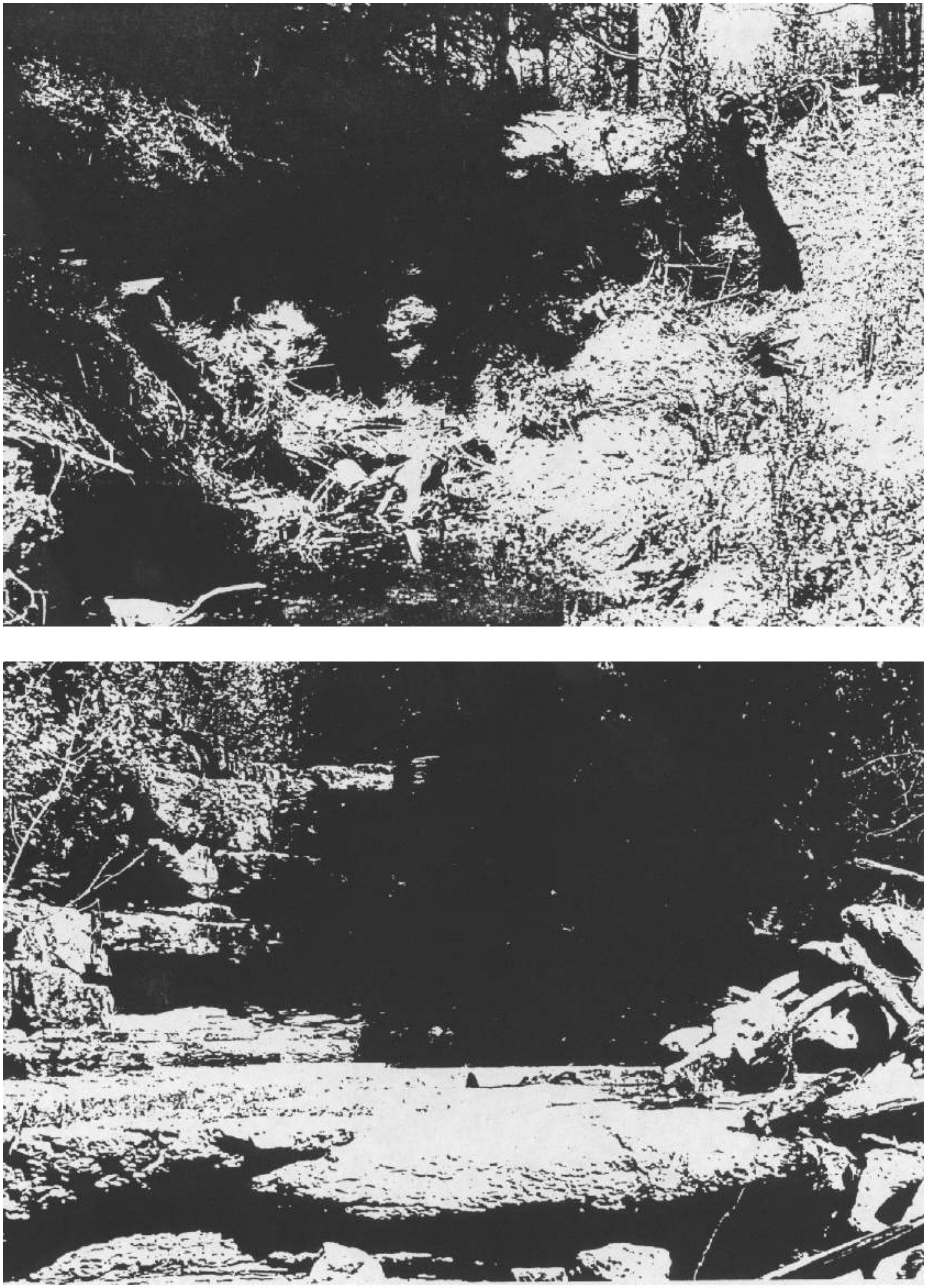


Figure 5.3: Surface Weathering Features (Plate 1)

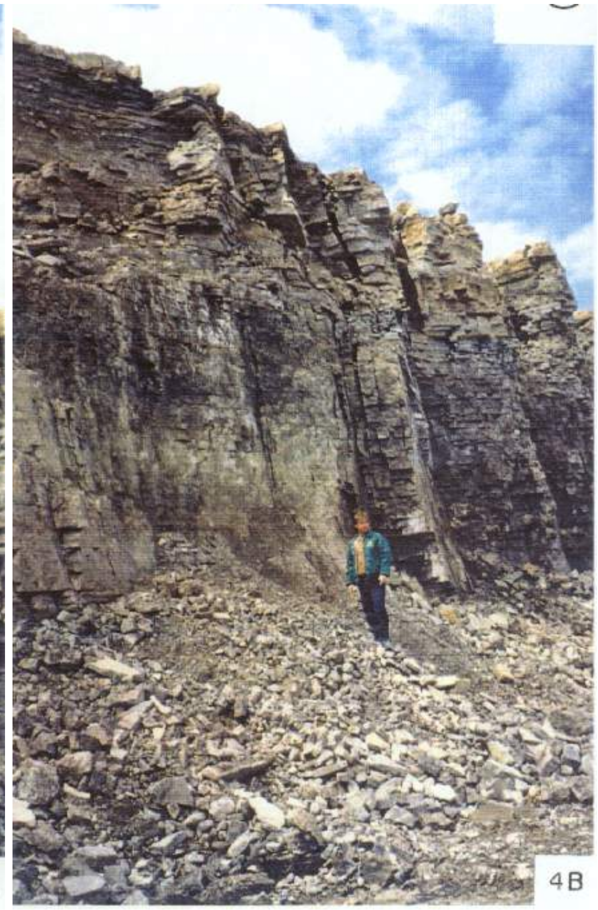
Weathered Vertical Fracture
and Bedding Plate

Weathered Vertical Fracture

Weathered Vertical Fracture

Figure 5.4: Bedding and Vertical Fractures - Flamboro Quarry (Plate 4)

Near Surface Weathered Zone



Bedding and Vertical Fractures

Detail of Open Vertical Fracture Exposed in Quarry Wall

Figure 5.6 illustrates an erosion/solution cycle in an idealized sequence. In the early stages of this idealized sequence, the upper impermeable layer would have been eroded by surface streams which then proceed to disappear underground through enlarged joints and fissures (Figure 5.6, diagram A). This stage involves general disintegration of the surface drainage pattern where streams flow down normal valleys only to disappear into solution holes at blind ends. Throughout this early stage, the underground drainage develops (Figure 5.6, diagram B). At this stage, a classic karst environment will have been created typified by an upper zone of downward movement, a lower zone of lateral movement, and a middle zone which, to the level of the water table, is alternatively one of lateral and downward movement (ref. also detailed flow structure diagrams on Figure 5.7). At the end of the cycle, all the roofs of the caverns or voids collapse and the drainage reappears at the surface on the exposed underlying impermeable bed (Figure 5.6, diagram D). At this stage, the limestone cover has been reduced to a few outliers and is often honeycombed with caves.

Where limestone units are thinner, and or occupy smaller outcrop areas, this clear cycle of solutioning cannot be so precisely categorized, as part of such areas would come under the influence of the normal cycle of erosion. Although the drainage may be locally underground in such areas, it is very unlikely that all of the streams will develop the classic features such as idealized on Figure 5.6. In these cases, it is usually possible to identify the potential karst zones based on topography. However, when karst is less mature (i.e., only characterized by solutioned interconnecting fractures), there is much more difficulty associated with identifying the location of solution zones which could cause problems for building foundation stability.

5.1.2 Karst Features

The following features are indicators of karst terrain:

- disappearing streams and pockmarked topography;
- solution sinkhole;
- collapse sinkhole;
- pinnacled surface;
- dome-like cavities in soil;
- cavities or caves in rock; and
- karren.

The presence of disappearing streams and pockmarked topography which has resulted from the collapse of sinkholes over solutioned voids are clear indicators of karst conditions. Since the capacity of the flow paths through the cave system can be quite appreciable, in times of low rainfall it is common that no surface streams exist; all of the flow occurs through the cave system (Figure 5.6, diagram A). In times of flood, however, the surface flows re-emerge. Such ephemeral streams typically show surface valleys which contain single or multiple points of ingress and egress between the surface flow system and the underground flow system (Figures 5.6 and 5.7). These types of features are often the most commonly identifiable feature of karst terrains.

Sinkholes are another typical feature of karst terrain, and in general, refer to areas of localized land surface subsidence or collapse due to karst processes (i.e., dissolution of the underlying rocks). Sinkholes may develop where joints intersect and these may lead to subterranean galleries and caverns; characteristic of thick, massiv limestones. Two basic types of sinkholes can be distinguished (Beck 1984):

- solution sinkhole;
- collapse sinkhole.

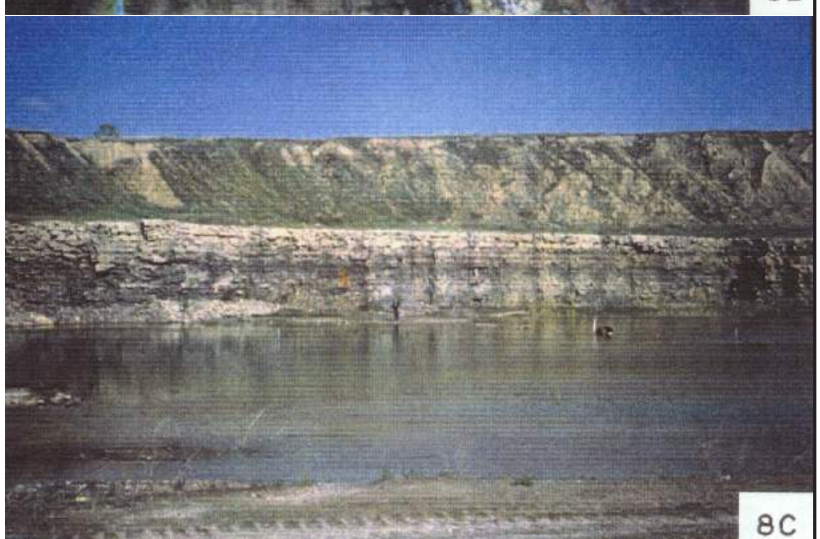
The solution sinkhole is a karst surface depression of conical shape (i.e., doline) caused purely by bedrock solution around joints or planes of weakness. The solutes and some insoluble residues are removed down solution-widened planes of weakness. Once the downward percolation is established, the doline will gather drainage causing its further development. This phenomena typically occurs slowly, with some small-scale collapse and subsidence caused by downward piping of residual materials and it rarely causes major collapse problems. This type of sinkholes is commonly associated with areas where highly soluble rocks are exposed at the ground surface.

The collapse sinkhole is a karst surface depression caused by collapse of the roof of bedrock cavern; the rapid subsidence produces a steep sided, rock walled hole, that widens into interconnected cave passages at depth. Sometimes solutioning produces a highly irregular, pinnacled surface on the limestone surface. The size, form, abundance and downward extent of the karst features depends upon the geological structure and the presence of interbedded impervious layers. Ravelling may occur when solution-enlarged openings extend upward to a rock surface overlain by soil. Initially, the soil arches over the openings, but as they are enlarged the soil can no longer support itself and collapses. Individual cavities may be open; they may be partially or completely filled with overburden soils or broken limestone/dolostone rubble in a soil matrix, or they may be waterfilled conduits.

Figure 5.5: Bedding Seepage - Flamboro Quarry (Plate 8)

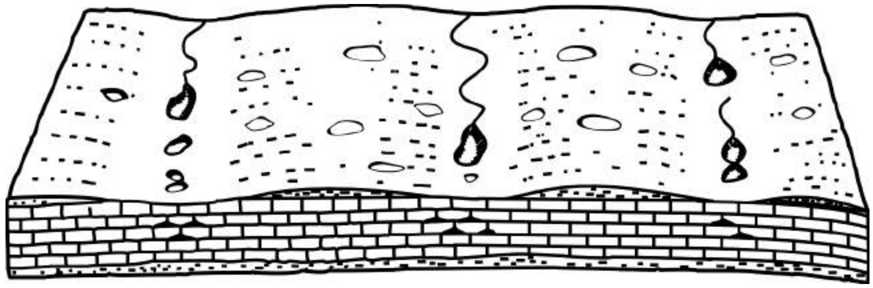
North Wall of Lower Bench Showing Bedding Controlled Groundwater Seep and Seepage From the Toe of Wall

Close Up of Bedding Seepage

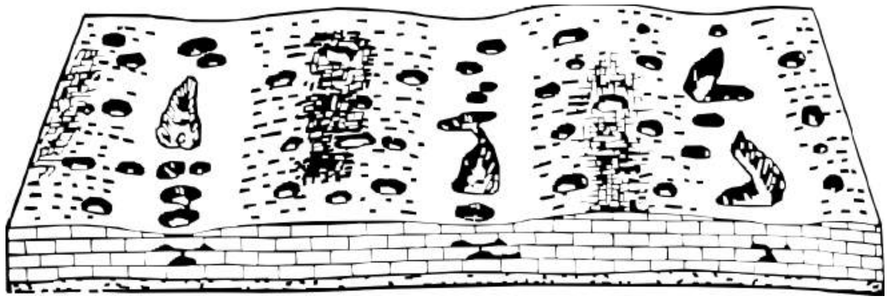


East Wall of Lower Bench Showing Guelph/Eramosa Contact

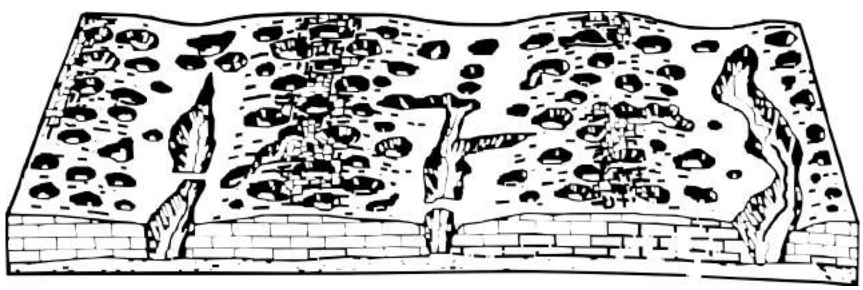
Figure 5.6: Classic Stages of Karst Landform Development



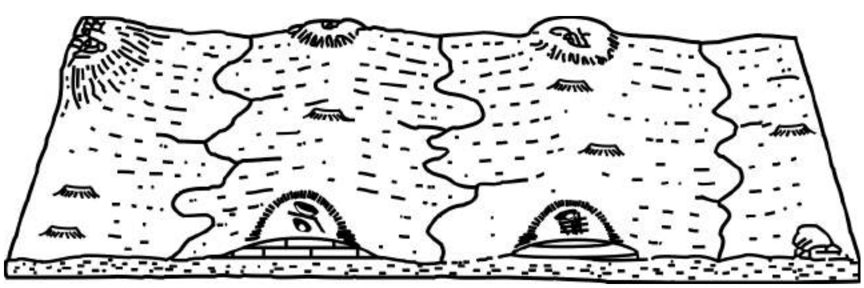
Pre-existing stream flow pattern showing classic development
of sinkholes and ephemeral (disappearing) streams



Mature Karst development, including
typical connected cave systems.



Fully developed Karst system but with major stream
channels cut down to underlying impermeable stratum.



Relict Karstic knolls and outliers remain in
re-established normal drainage system.

Simplified representation of the Karst cycle of erosion. The depth and size of the
solution holes is greatly exaggerated in relation to the thickness of the linestone.

The solution process may be accelerated by man-made changes in the groundwater conditions or by a change in the character of the surface water that drains into the rock. The primary effect of solution is to enlarge the pores/cavities which enhance water circulation, causing further solution. This produces an increase in stress and reduction in the strength of the remaining rock mass.

5.2 Type of Hazard

5.2.1 General

In kars
of:

- migration of soil from beneath footings into underlying cavities; and
- sudden collapse of a cavity roof or rock shear failure somewhere beneath the foundation.

In the first case, soil is carried through fissures or solution channels in the rock by downward percolating water. The water may result from surface infiltration or precipitation, or from a leaking piping system. In the latter case, increased stresses caused by imposed foundation loadings might be sufficient to cause the roof of an underlying cavity to collapse or shear the existing rock ledges undermining the support of the foundation.

Successful foundation design in these conditions requires:

- determination of the location of sinkholes so that the structure is located suitably;
- determination of appropriate bearing values; and
- determination of construction procedures.

An estimate of the relative risk of occurrence and probable magnitude of the settlement should be carried out based on the results of a geotechnical investigation at the site. If possible, economical and safe foundation solutions accommodating the expected movement may then be adopted. The design of safe foundations may be uneconomical where there is high risk of sinkholes and doline development, and such a site may have to be abandoned. It should be noted that areas between karst features can also be karst, even though there is no surface evidence of karst.

5.2.2 Settlement or Subsidence

The main problem with development within areas of karst formation is settlement. The following types of settlement can occur:

- normal settlement;
- sudden subsidence settlement; and
- gradual subsidence settlement (i.e., solution sinkhole/doline).

Normal settlement can occur as a combination of immediate elastic settlement and consolidation settlement. The magnitude of this settlement will depend on the load, thickness and properties of the residuum and transported soil cover. A structure founded partially on rock and partially on loose residuum may suffer from differential settlement.

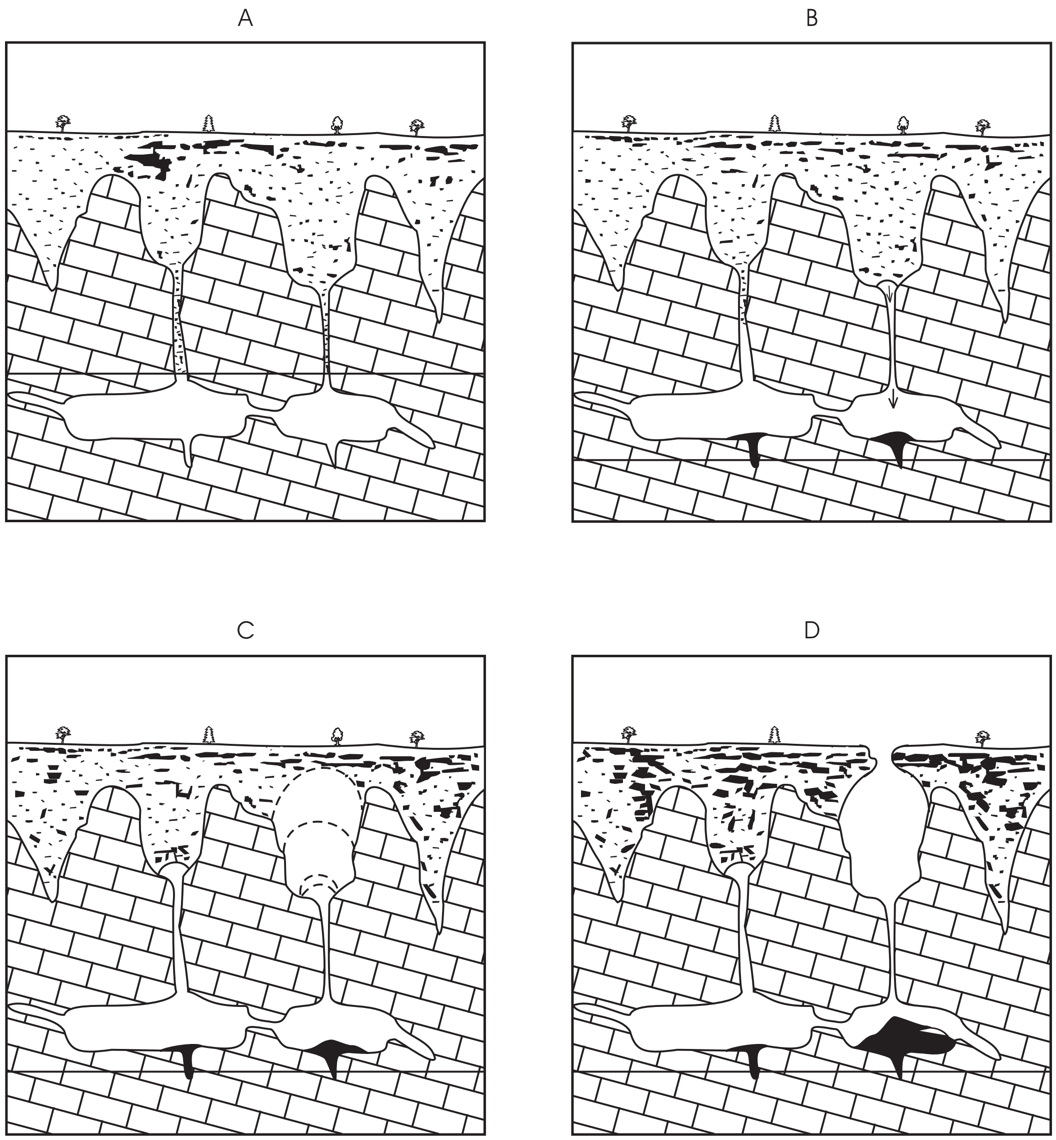
Sudden subsidence settlement can occur due to the appearance of the sinkhole caused by the collapse of an arch spanning over a cavity in the residuum. The size of the sinkhole formed will depend on the thickness of residuum and depth of water table. A large sinkhole cannot develop in areas with a thin soil cover or a high water table.

Gradual subsidence settlement (i.e., solution sinkhole/doline) is formed when the arching conditions required for the formation of the sinkhole do not develop. Large dolines can be formed when the water table is lowered in areas which contain thick layers of soils. The damage caused by the doline can be substantial but as it happens slowly it is not a threat to life.

5.2.3 Water Resources

Natural cavities will influence water resource development. Groundwater flow along the solution widened fissures can enhance transmissivity rates and volume of water for extraction, however, if contaminants are introduced into solution widened fissures, they will spread quickly and will be difficult to control. Adverse consequences can also occur during groundwater lowering as a result of water extraction due to removal of hydrostatic support to the cavities and increased seepage forces leading to degradation and failure of the cavity and subsidence of the ground surface.

Figure 5.7: Sinkhole Development Due to Lowering Water Table Conditions in Typical Karst Environment



Progressive development of sinkhole over a cavernous limestone due to ground water withdrawal in South Africa (after Brink, 1979). A. The equilibrium situation before the lowering of the water table. B. The lowering of the water table. There is active subsurface erosion (piping into the open cave) as the slot is flushed out by a process of headward erosion. C. The progressive collapse of the roof of the vault, possibly temporarily arrested by a ferruginised pebble marker. D. The collapse of the last arch to produce a sinkhole surrounded by concentric tension cracks.

5.3 Approaches to Address the Provincial Policy

Defining the "area of provincial interest" or *hazardous sites* associated with karst formations involves a site-specific process. To develop a single approach or standard applicable to all areas is impractical and inappropriate given that the size, extent and severity of the hazard and risk(s) are governed by localized specific conditions.

Depending on the magnitude of the karst feature it may be possible, economical and safe at some sites to provide foundation solutions which will accommodate the expected movement. In other locations the design of safe foundations may be uneconomical where there is a high risk of sinkholes and doline development, as a result these sites may have to be abandoned.

5.4 Site and Field Investigation

General information about the area is frequently available in the form of geological and topographic maps, and literature on general surface and subsurface conditions of the site. Considerable information on surface conditions and land forms that are not detectable from ground observations or not shown on the maps can be provided by aerial photographs. Detailed information on subsurface conditions for designing and planning purposes, should be obtained by means of on-site subsurface exploration.

5.4.1 Site Investigation

The investigation for a site potentially located within a karst terrain should involve:

- review and search of all available published and unpublished geological and topographic records, literature on general surface and subsurface conditions of the site;
- review of aerial photographs; and
- a site visit.

The objectives of this stage are:

- to establish the geological and geographical extent of the hazardous site;
- to identify type and extent of hazard specific to the site and the consequent risk to life, property and structure;
- to determine techniques for investigating the site; and
- to review remedial measures for mitigating the risks to life and property.

a) Review of Records

Commonly known karst features in the Southern Ontario are documented along the Niagara Escarpment and in the areas of Kingston and Ottawa. Information on the location of soluble carbonate and non-carbonate rocks in general is provided on geological base maps which may be obtained from the Ministry of Northern Development and Mines, Geologic Survey of Canada or the Ministry of Natural Resources.

Topographic maps for many urbanized areas based on aerial photographs interpretation are available from the municipal level government offices (e.g., Engineering or Public Works Department). The mapping should be at the scale sufficient to show details of the site. Historical aerial photographs and other documents of bedrock instability, erosion, filling cavities, installation of remedial works and land development can be often obtained from the local municipal offices (e.g., usually public works departments).

b) Review of Aerial Photographs

The use of aerial photographic interpretation may help to identify karst features in the study area by interpretation of the existing topography and "tonal" variations observed on the photographs. Aerial photographs taken during different times of the year and at several year intervals allow examination of similar areas under a different climatic conditions and at different stages of land development providing basis for verification of the karst topography. Low altitude photographs will usually provide a better picture of karst features.

Mottled areas are the most common surface feature observed in karst or carbonate terrain. They appear as circular to irregularly shaped depressions of different relief in a modified lattice or boxwork type pattern. These closed depressions retain moisture and can be identified by differences in colour tones. The depressions will range in tone from black to various shades of grey, contrasting with the surrounding terrain. When the depressions impound water (e.g., after prolonged rainfall or after winter thaw) they may appear light grey or white due to the reflection of the sky.

Solution channels coinciding with fracture traces or bedding usually appear on aerial photographs as straight linear features that are often connected with depressions or sinkhole. The linear features may appear darker than the surrounding soil.

Sinkholes, if large in size, can be easily recognized from the aerial photos. If the sinkholes are small in the size, they can generally be located using low altitude aerial photographs. Surficial features that can help in determining the presence of some sinkholes include isolated clumps of trees or other non-cultivated vegetation located in fields and small, forested areas adjacent to known karst areas. These areas should be located and checked during the field survey.

Surface Mines/Open Pit Mines appear on photographs similar to large sinkholes; however, there is usually evidence of roads and other mine workings or equipment (Figure 5.8).

c) Site Visit

The review of aerial photographs should be supplemented by a site visit to confirm the information derived from the aerial photographs. During a site walkover, the nature of the site and characteristic features should be recorded, measured and evaluated.

Changes with time can be expected with karst features due to weathering, land development or individual karst feature repair. Features identified on aerial photographs may not be visible or present in the field and new features may have developed.

In areas where karst features are prevalent, an interview with the property owners/public is carried out as part of the survey to request information on existing sinkhole/karst feature locations, physical characteristics, causative factors and repair methods. A factual database map of detected and surveyed natural and artificial/man made cavities and karst features, geology and hydrogeology is then produced.

5.4.2 Field Investigation

Following appraisal of the information obtained during the desk study stage of the investigation, the subsurface conditions at the site can be evaluated. The determination of subsurface conditions in karst terrain through the use of borings and test pits can be time consuming and expensive; a staged investigation program may prove to be more economical. Geotechnical investigation should be carried out in stages:

- First stage — geophysical methods and airtrack or percussion drilling (i.e., indirect methods); and
- Second stage — detailed drilling and sampling of soil and rock to determine design data (i.e., direct methods).

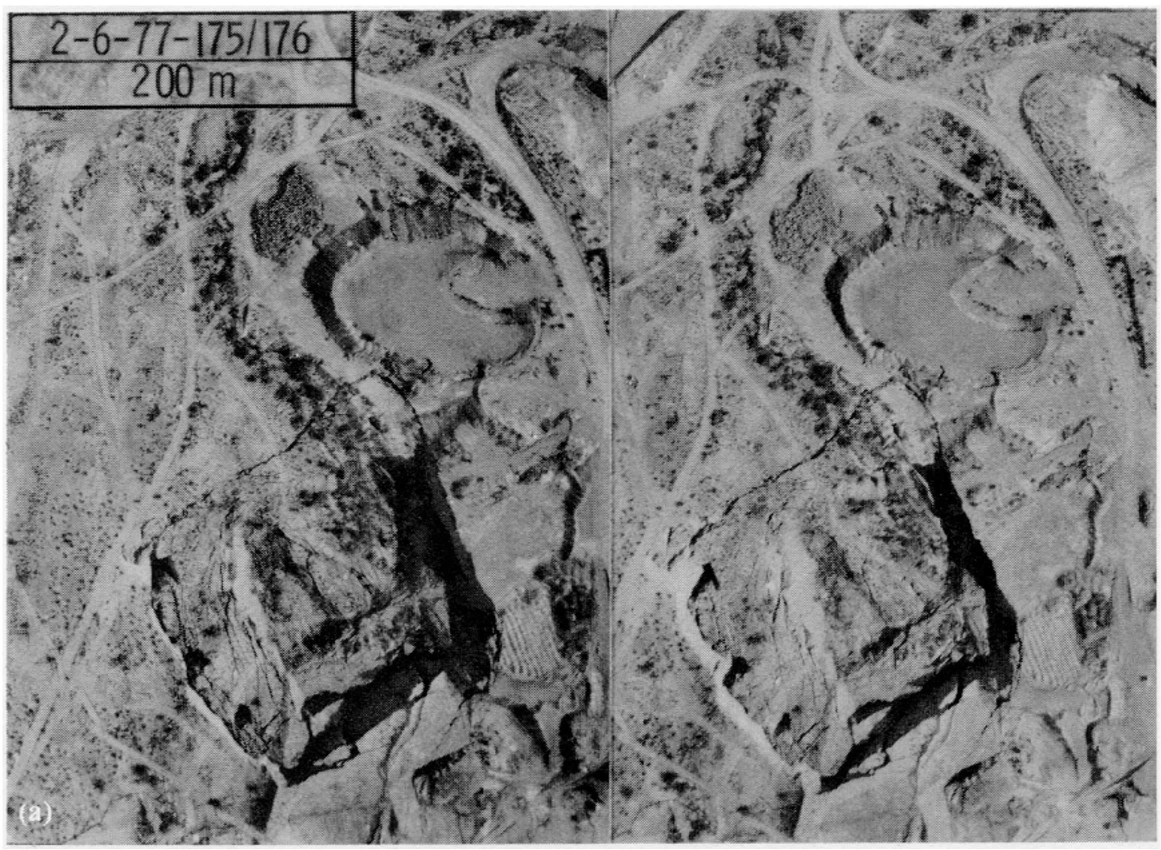
Indirect methods may be used to identify the presence of areas of possible problem foundation conditions, however, these methods are not suitable for all ground conditions and there are limitations to the information that can be obtained. Typically, it is possible to locate strata boundaries only if the physical properties of the adjacent materials are significantly different. It is always necessary to check the results of the geophysical methods against data obtained by airtrack drilling and/or percussion drilling techniques. If such correlations are established, geophysical methods can produce rapid and economic results, especially in estimating the depth to bedrock.

Since the geophysical methods provide interpretative information only, detailed geotechnical investigation involving drilling boreholes, electronic piezocone sounding, and/or test pit excavation can be used to explore and evaluate the subsurface conditions, to correlate the data obtained from the geophysical methods and provide data for project design. A description of geotechnical investigation techniques are provided in Appendix A3.5, and comments on the use of the methods in karst terrain investigation are provided in Appendix A5.2.

5.4.3 Overview of Review and Decision Process for Karst Terrain

In areas of major karst formations in the USA, there are several Ordinances/Zonings to follow during a study and design processes for development in the karst terrain. Two examples of phased investigation program for development in karst region that have been introduced in the USA are provided in Appendix A5.1 for reference, one in Clinton Township, New Jersey and the second in Lower Macungie Township, Pennsylvania.

Figure 5.8: Ground Subsidence due to Caving Into an Underground Mine



From Mitchell R.J., 1983.

A "checklist" of issues, questions or conditions to be addressed by a geotechnical consultant when involved with a project which involves development in karst terrain has been compiled in Table 5.1. This list is not necessarily applicable to development at all sites, however, the reason for discounting an item should be stated where an individual item is considered to be unwarranted.

A phased investigation program is suggested which also provides specific requirements or suggested methods of investigation for each phase. In this way, an applicant for a building permit has the opportunity to cancel a project at an early stage if it proves uneconomical.

5.5 Addressing the Hazard

Site improvement and mitigation of cavities typically involve implementation of grouting/filling methods to eliminate the cavities or to prevent their enlargement. Filling the cavities with grout or with soil or rock fills sometimes blocks further expansion of the void and prevents further rock solution or soil ravelling. The following techniques are frequently used to improve a site in karst terrain:

- compaction grouting;
- slurry grouting;
- dynamic compaction; and
- vibro-compaction.

Compaction grouting involves injecting cement based grout into the ground under high pressure. It can be used to partially seal and fill voids and to densify overburden soils. **Slurry grouting** involves the use of very fluid grout to fill fissures and broken seams.

Compaction grout would typically be used first to fill the cavities and to densify the loose soils, additional sealing of the formation would then be carried out using sl u
knowledge of the openings in the rock including major joint spacing. The spacing of the grout holes should be less than the major fracture spacing and the holes are typically angled since the primary fractures are likely to be orthogonal with the bedding (i.e., if the bedding is horizontal, the jointing will be vertical). It is these joints which will be solutioned, mainly at the interface with the horizontal bedding planes. Examples of implementation of these methods to improve founding strata in karst terrain are illustrated in Figures 5.9 to 5.16.

Generally, grouting for foundation improvement should extend at least 1.5 times the maximum dimension of the net pressure distribution (i.e., Boussinesq bulb) below each footing. Specific checks must be made of grout effectiveness, as grout will follow fractures irrespective of the care taken in grout hole layout. To be effective, the grouts should match the material that needs injection. Thin cement grouts will only penetrate material with grain sizes larger than medium sand, but will run for tens of metres in open fractures. Grouting is generally commenced with thick grout injected through primary holes to plug big voids and then the area is further grouted using secondary holes with successively thinner grouts.

The pressure to be used during grouting have to be assessed in order to ensure that excessive pressures which could cause heave are not used. Monitoring and testing of any grouting process in karst environments is critical as heave and grout consistency are the only controls for ensuring effective foundation grouting.

Another means of site improvement that has been successfully employed for sinkholes of limited depth involves the use of **dynamic compaction** or **vibro-compaction techniques**. These techniques are used to densify granular soils in a loosened state within or above the cavities formed in the rock.

Dynamic compaction involves dropping heavy weights on the ground surface from a site specific height such that energy waves are sent deep into the ground causing soil densification. **Vibro-compaction** involves use of vibrators which are extended to the required depth with the aid of high pressure water jetting and essentially densifies a column of soil by the vibration action. The advantage of vibro-compaction is that the surface cap above a cavity can be broken; however, the use of water can aggravate cavity formation (see Figure 5.17).

5.5.1 Drainage Measures

Drainage may reactivate soil movement in karst terrain. The effects of downward drainage can have a number of detrimental effects on ground stability. While increased seepage gradients can result in faster rates of solution with consequent enlargement of sinkholes, the most severe problems result from increased gradients which lead to dislodgement of loosely consolidated infilling material. Drainage should be diverted away from solution depressions and groundwater changes should be controlled where possible.

Table 5.1 Checklist for development in karst terrain

STAGE 1 - Information Study	The Consultant should:	· carry out an historical review of aerial photographs; a good appreciation can be gained of the trend of bedrock geology in the general area of a site, and · obtain geological, topographical mapping and previous known subsurface investigation data in the immediate area of the site; and · identify existing surface water bodies and location of any existing water production wells.
STAGE 2 - Initial Site Inspection	Typical data obtained from the visual inspection should include:	· confirmation of the information obtained during the information study (STAGE 1); · measurements and evaluation of topographic features of the site; · evidence of the karst features, such as sinkhole, conical depression, pinnacled surfaces, ravelling, etc., · evidence of the old karst feature repairs and the repair methods, if present; · site drainage, seepage, disappearance of surface drainage; · condition of vegetation; · general soil conditions determined by hand auger; and · human disturbance/man-made structures.
STAGE 3 - Reporting of Visual Inspection	A site inspection by the consultant, should be carried out to determine the presence or absence of karst surface features on the site, present the data on a prepared site plan. In particular, the following features should be noted, if present, on the site:	· closed depressions; · open sinkholes; · seasonal high water table indicators; · unploughed areas in ploughed fields; · surface drainage into ground; · disappearing lakes("ghost lakes") after rainfall; · lineaments and faults; · contacts between geologic formations; and · any other karst present on site. Where the consultant considers that the site conditions do not warrant further study, the report should: itemize all results from (STAGE 1, 2 and 3) above; · provide an assessment of the site; · provide a clear statement of the hazards revealed by the site inspection, and · provide a clear statement of the effect of the hazard on the proposed development.

STAGE 4 - Subsurface Investigation	The intent of the site investigation program is to define the nature and limits of possible design, construction and operating concerns that could result from the existence of karst underlying the proposed development site. The proposed investigation program should consist of investigation techniques, equipment and program objectives specific to site. Geophysical testing procedures and percussion or airtrack probes should be used as the primary means of identifying potential geological hazards. Detailed geotechnical investigation involving drilling boreholes, electronic piezocone sounding, and test pit excavation can be used to correlate the data obtained from the geophysical methods and provide data for project design.	In preparation for the subsurface investigation, the following techniques should be considered: 1. Indirect Testing Procedures a) Geophysical Testing · the method and equipment to be used, · location and number of tests to be performed. b) Percussion or Airtrack Probes · method and equipment to be used · depths anticipated · measuring techniques to be utilized (air, loss, drilling speed and rod drops). 2.Direct Testing Procedures a) Test Borings · location and number of borings, · depth anticipated and sampling intervals; · boring techniques to be utilized; · proposed borehole grouting techniques, b) Test Pits · number and depth of proposed test pits; minimum base area of pits to ensure suitable inspection of rock surface · method of backfill to be employed; typically, test pit backfill shall be composed of excavated material placed in layers and compacted to pre-excavation density; c) Piezometers and Groundwater Table Data · number, locations and types of groundwater recording to be used; these should be sufficient to identify depth to seasonably high water table and rate and direction of ground water flow.
STAGE 5 - Analyses and Reporting	In addition to summarizing and reporting the subsurface conditions encountered during the investigation, the consultant should:	· provide recommendations for remedial works and site improvements; · provide a framework for inspection and monitoring during construction as well as long term monitoring, if necessary; · carry out site inspections during construction to ensure compliance with the recommendations provided.

The following precautions should be considered to minimize further ingress of surface water into the rock formation where construction is proposed on sites where karst featu
karst occurrences:

- the site should be landscaped in such a way that concentrated ingress or ponding of water is avoided;
- trenches for services should be constructed such that the backfill is less permeable than the surrounding soils;
- utility pipes carrying water must be designed to accommodate some movement and should be checked for leaks periodically; in critical areas (i.e., where there is potential for large movements), the services should be maintained at or above ground surface;
- stormwater channels should be lined and discharged away from the development; and
- water retaining structures should be provided with an impermeable liner to prevent infiltration into the subgrade soils.

The unpredictability of collapse and its catastrophic mode of occurrence add a risk factor to all aspects of and use in karst terrain. Provision should be made to minimize the threat of karst subsidence to life and property.

5.5.2 Foundation Design

The design of foundations in karst formations is one of the most challenging tasks in rock foundation engineering and there are unfortunately many instances of failures related to solution of limestones and the formation of sinkholes (Sowers 1975; Costopolous 1987 etc.). These failures are often the result of the location of structures on undetected sinkholes which may be reactivated by subsequent water movement or possibly, may even develop after construction. In order to reduce or prevent such threats it is necessary to conduct geotechnical investigations in known or expected areas of karst topography prior to permitting new development.

Engineering design to control the hazards in karst terrain requires detailed geotechnical data on subsurface conditions to determine:

- depth, thickness and engineering properties of the soil and rock strata;
- groundwater levels, their changes and groundwater movement;
- location of karst features in the rock and the reflection of these features on the overlying soil; and
- nature and the extent of the voids or defects and the possible changes due to construction and natural changes within the rock and soils.

There are several approaches to design works in the karst terrain and all should include measures to correct or mitigate the existing hazardous conditions:

- optimalization of the location of the structure on site;
- use a shallow foundations, modified to overcome the cavities;
- use a deep foundations, modified to overcome the cavities; and
- precaution measures to minimize future activation of the cavities.

5.5.3 Optimalization

The location of the structure should be established based on the analysis of the patterns of karst features present on site. This can be achieved by direct observations, physical investigation and remote sensing. The structure should be positioned in the area of fewest defects to minimize the exposure to the existing cavities, as illustrated in Figure 5.18. It is difficult to identify all voids or defects and new defects can develop from changes in the environment, however, the greatest probability of new activity is in the area of previous defects in the underlying karst bedrock.

5.5.4 Consideration of Shallow Foundations

Shallow foundations supported directly on the soil or rock surface will often require remedial measures to overcome the karst hazard. Various methods exist for dealing with karst foundation conditions. Conventional bearing pressures can be reduced, coverings to bridge or cap identified cavities can be designed and compaction and consolidation grouting measures can be implemented to permit support of structures on strip or spread footing. A number of different designs for shallow foundations, include:

- use of concrete plug;

- partial replacement of collapse material with concrete;
- bridging;
- implementation of screw jacks for settlement adjustment;
- eccentricity of loadings;
- inclined piers; and
- mat foundation.

In most cases the footings are supported on the peripheral rock surface. Other remedial measures consist of filling the sinkholes and placing the footing on the fill. This includes either filling of voids and cavities using various types of infill or improvement of soils loosened as a result of the formation of sinkholes. Shallow, cone-shaped sink holes can be cleaned as deep as possible and plugged with lean concrete or grouted to form a plug.

For small sinkholes where there is sound rock around the periphery, it is often possible to enlarge and reinforce the foundation to bridge over the sinkhole and design footings using a conservative bearing capacity. If the arrangement of the holes or footings results in some eccentricity of the foundation it may be necessary to combine one or more footings to form a strap or mat foundation.

In addition, the nature of the bedrock surface in shallow karst formation is such that there is potential for differential settlement if some of the footings are supported on soils present in the bedrock depressions and other footings are founded on the rock.

If a certain thickness of the soil cover is removed and replaced under controlled conditions, a mattress of known thickness and with known strength parameters will be formed. The functions and advantages of the mat foundation would be:

- to control the total and differential settlement;
- to reduce the foundation stress to an acceptable level at the underside of the mattress; and
- to reduce the risk of formation of small sinkholes and dolines.

An advantage of the mat foundation method is that it enables an in-situ evaluation of the subsurface conditions after subexcavation for mat construction. If conditions are found to differ from those predicted, it is possible to excavate deeper or to revert to an alternative founding method. The thickness of the mattress will depend on a number of factors, the most important of those being the properties and thickness of soil cover and the sensitivity of the proposed structure to settlement. If the mattress has to be installed to span over cavities of given dimensions, it can contain metal or fibre strips at the bottom of the mattress. The strips will act as a tensile member of the composite section with the soil above being the compressive member.

5.5.5 Consideration of Deep Foundations

Deep foundations such as piles or caissons founded on rock can be used in karst terrain. Several examples of deep foundation applications are illustrated in Figures 5.17, 5.19 to 5.22. Structures may be supported on driven or drilled piles which are extended through the overburden and loosened zones within the cavity to be founded on competent bedrock. Care has to be exercised to ensure that adequate bearing capacity is achieved. Longer piles may enter pits or open joints resulting in their bending and/or twisting and may have a lower bearing capacity for a specified deflection than shorter piles. Piles will also have reduced bearing capacity where they are end bearing on a hard seam above a compressible seam or where they punch into porous, weak rock.

Figure 5.9: Concrete Plug Bonded to Rock

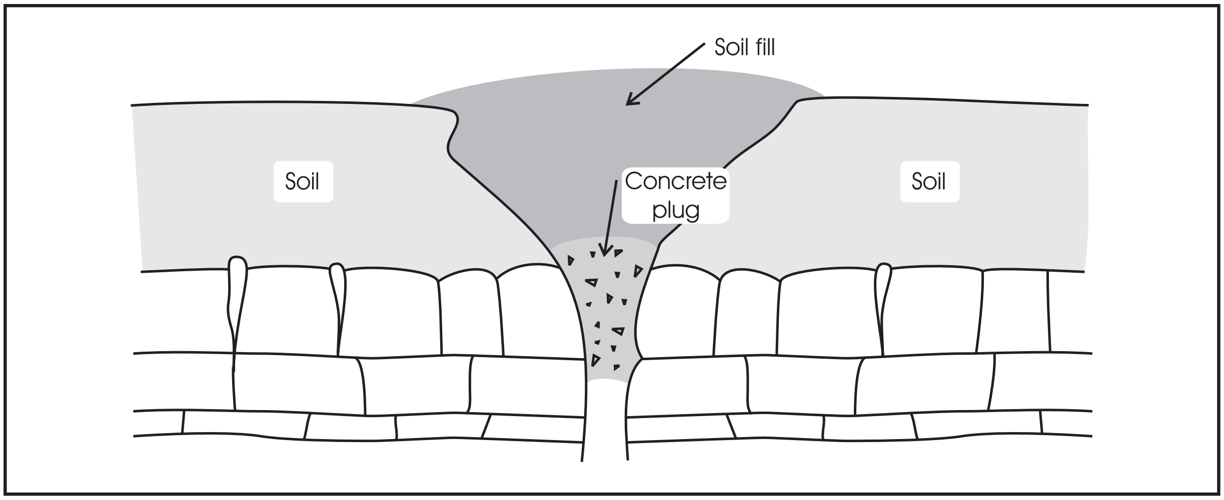


Figure 5.10: Mining Out Clay-Filled Cavity; Concreting

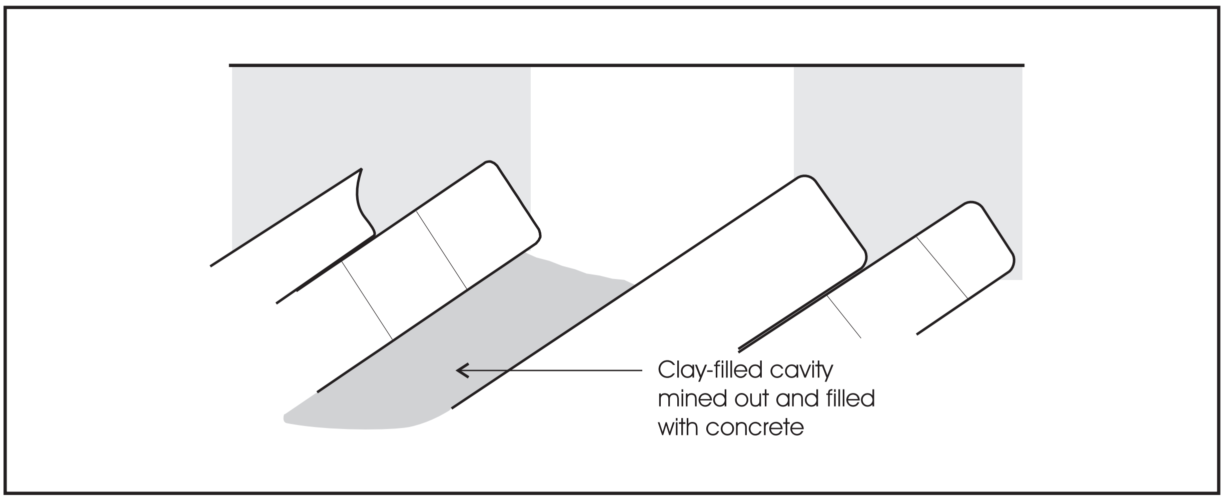


Figure 5.11: Grout in Open and Clay Filled Rock Cavities

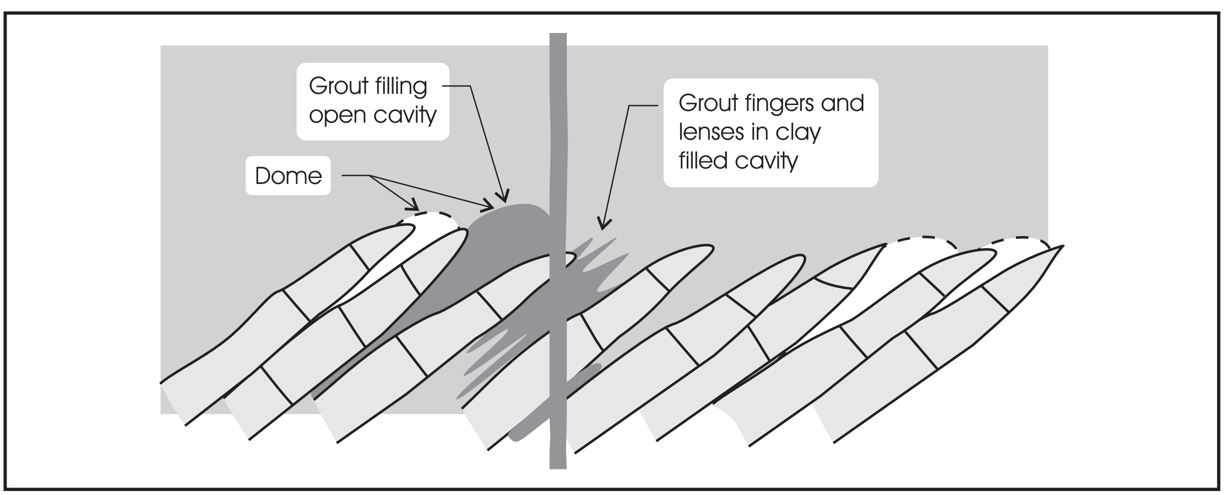


Figure 5.12: Grouting Cavity or Dome in Soil

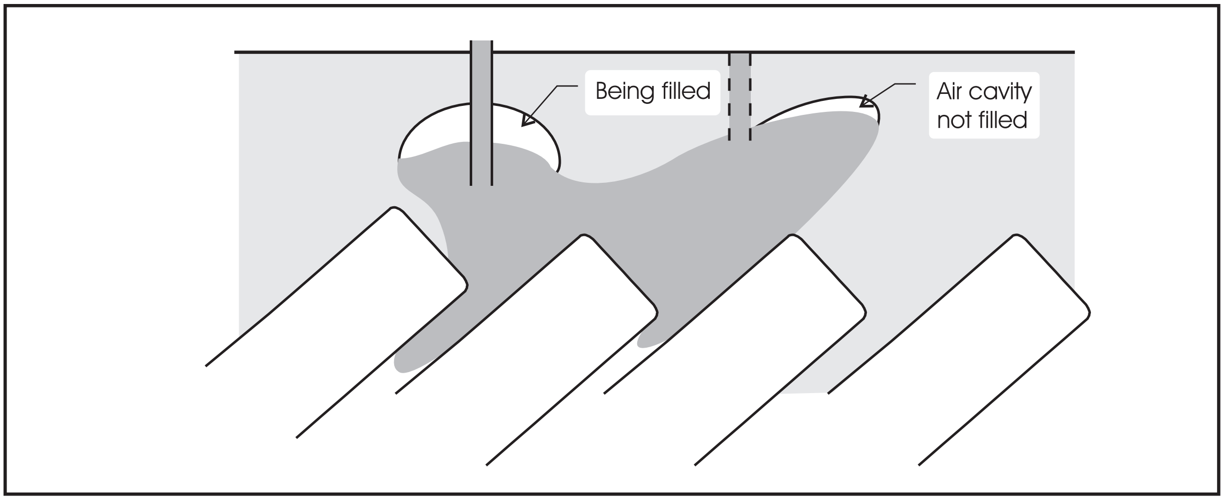


Figure 5.13: Dental Filling, Z = 1.5 to 2B

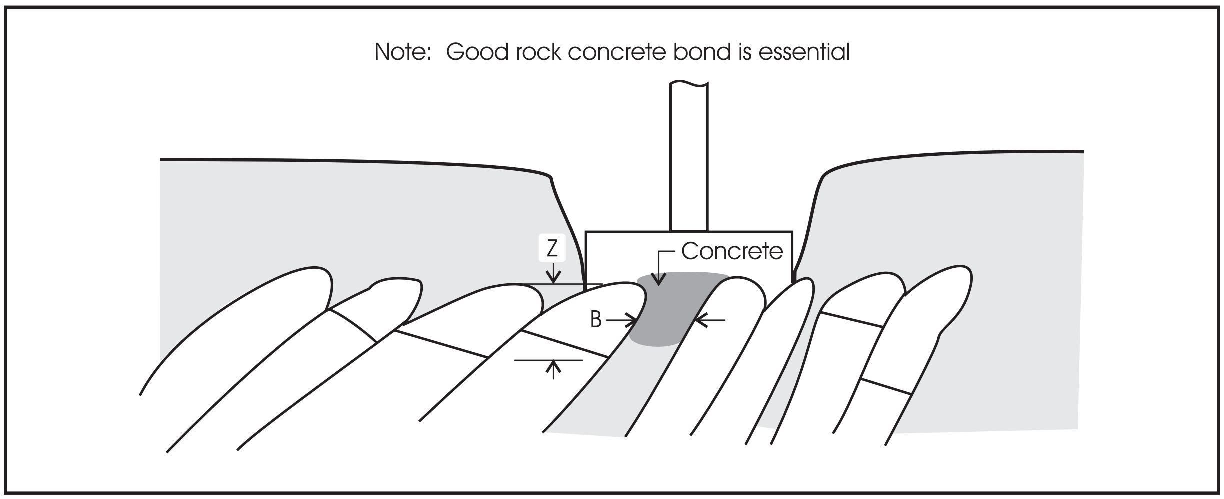


Figure 5.14: Filling Sinkhole with Graded Fill

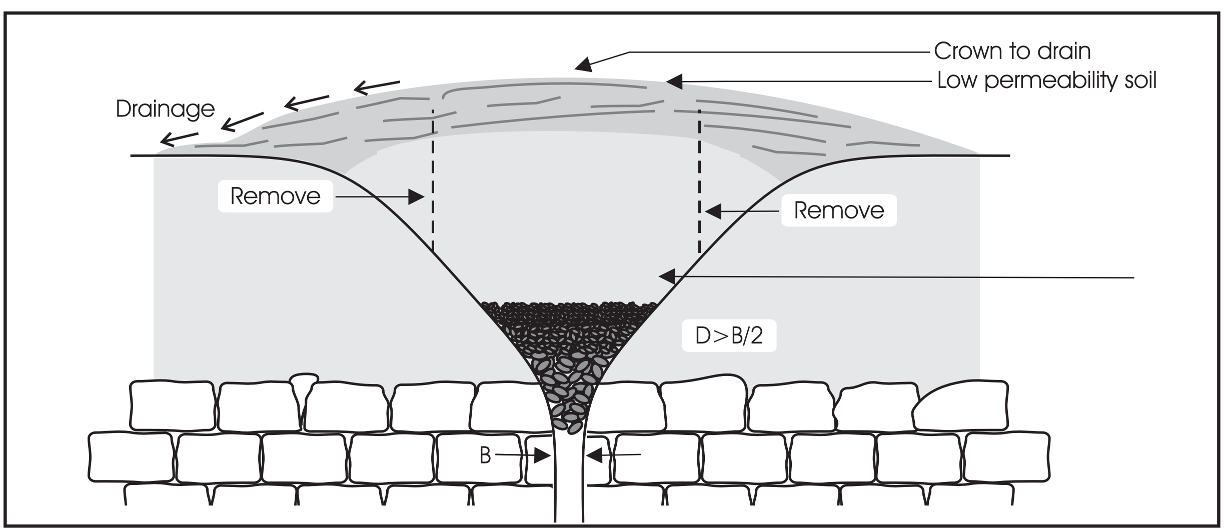


Figure 5.15: Filling Sink with Stone Plug, Geotextile and Sand

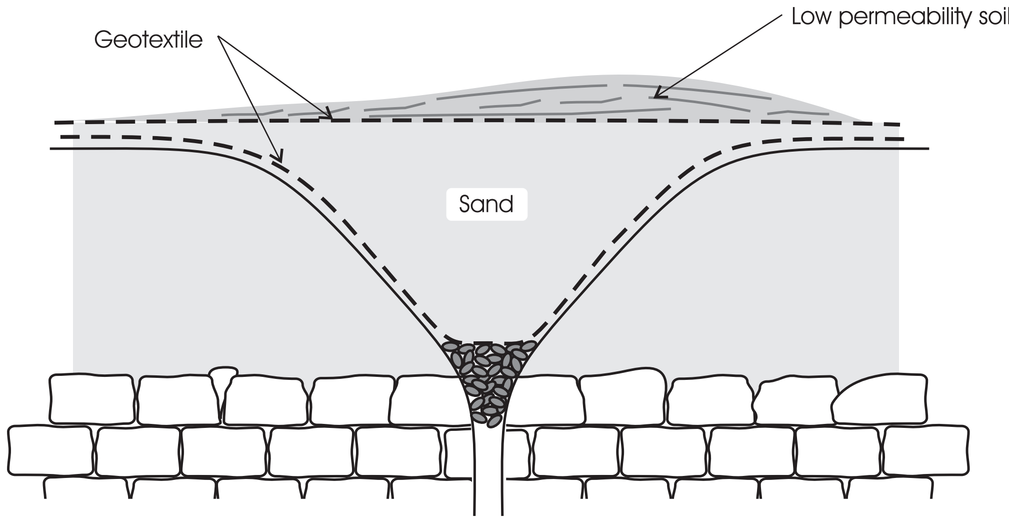


Figure 5.16: Preload Compressible Sink Hole Filling

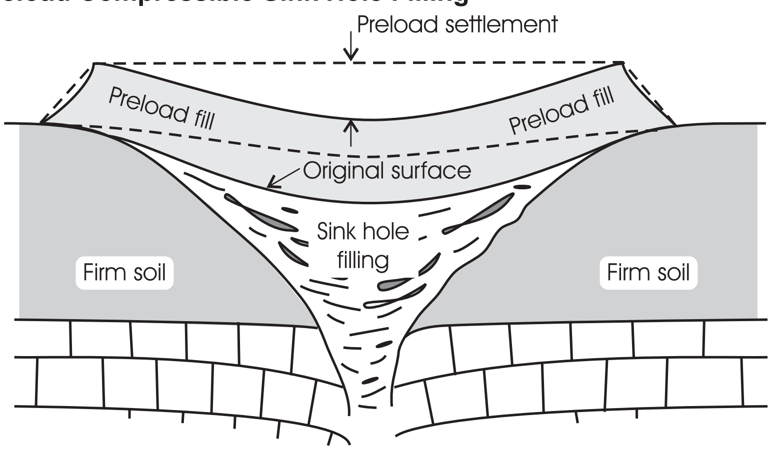


Figure 5.17: Straddle Cavity with Batter Piles

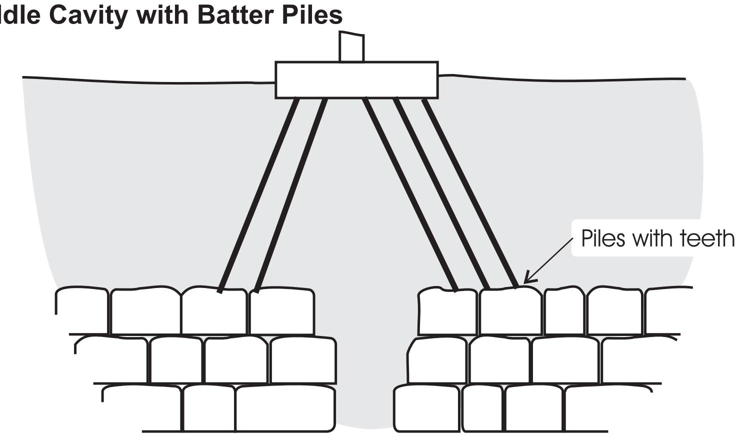


Figure 5.18: Optimize Location - Orient to Fit Site

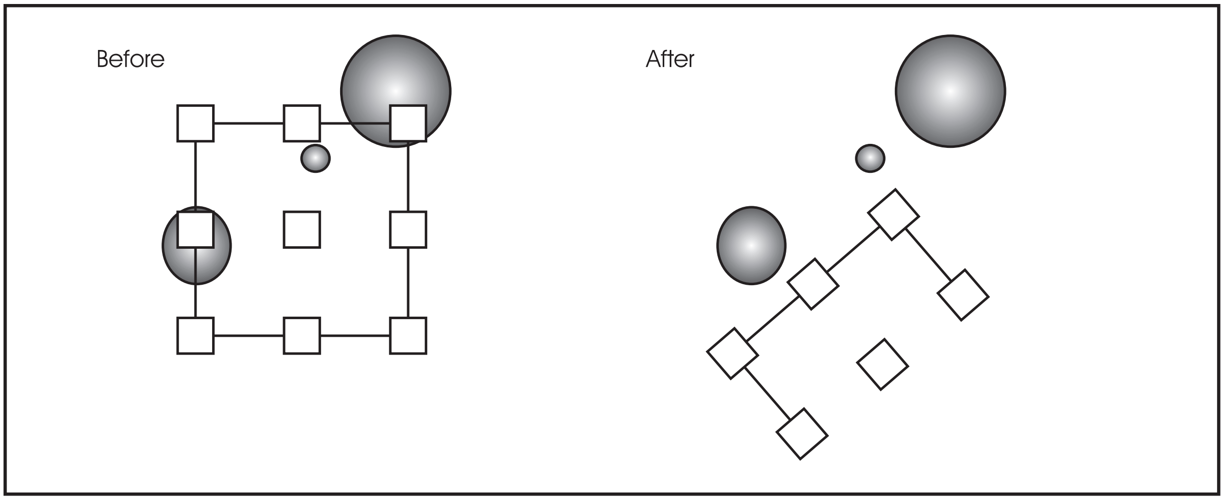


Figure 5.19: Increase Depth of Pier Foundation

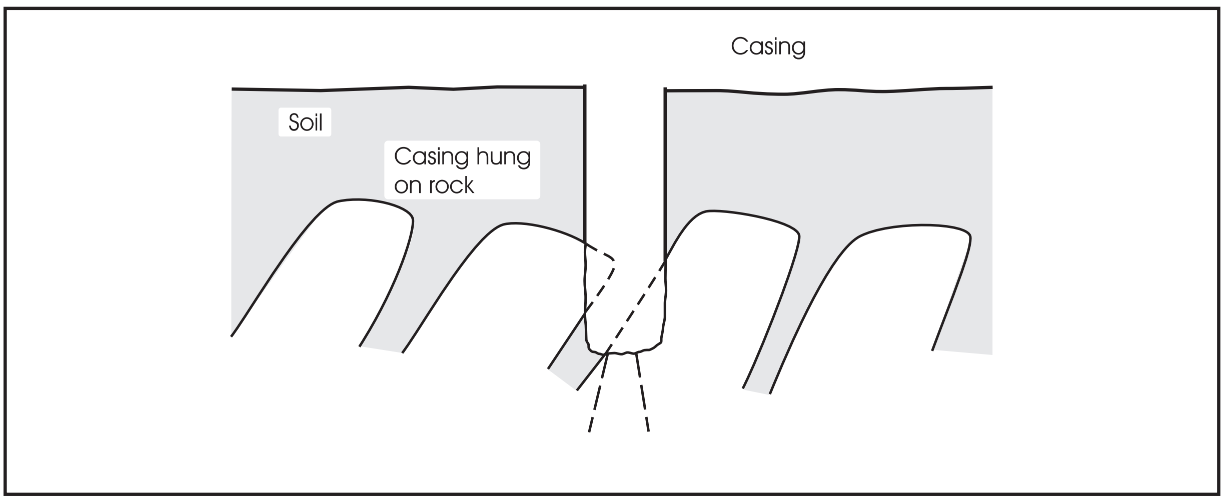


Figure 5.20: Caisson

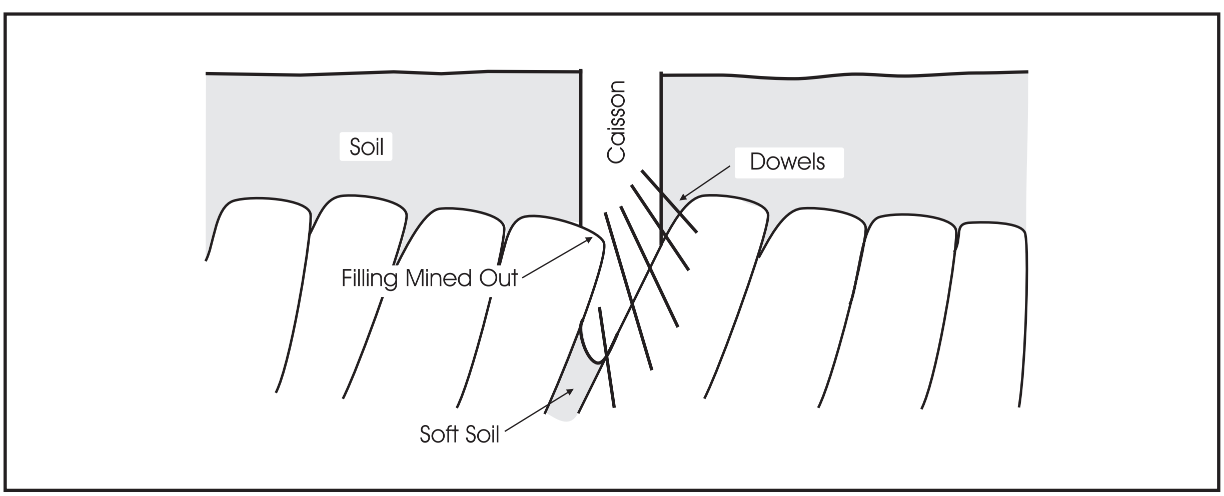


Figure 5.21: H-Pile in Caisson Bottom

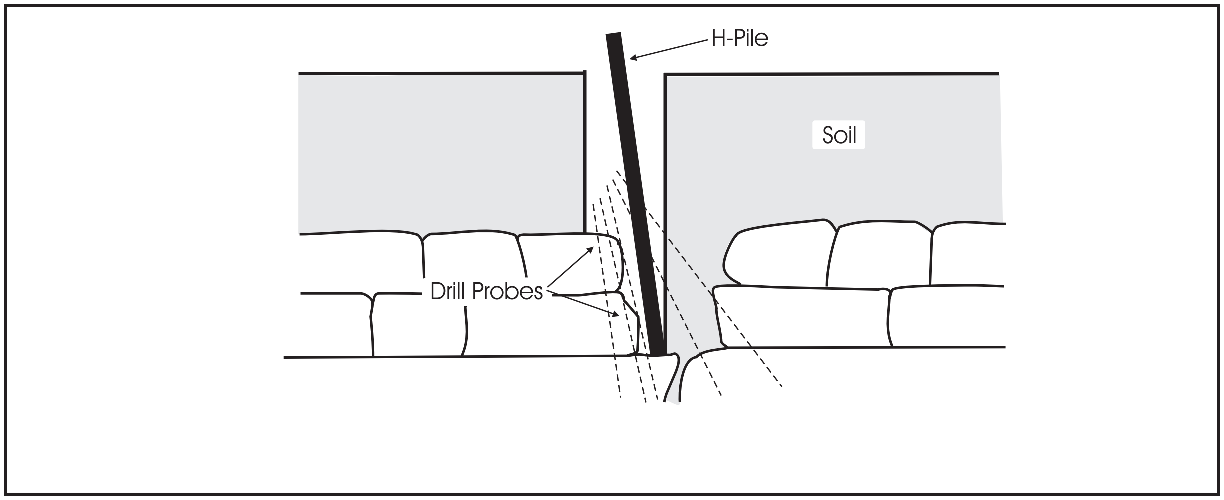
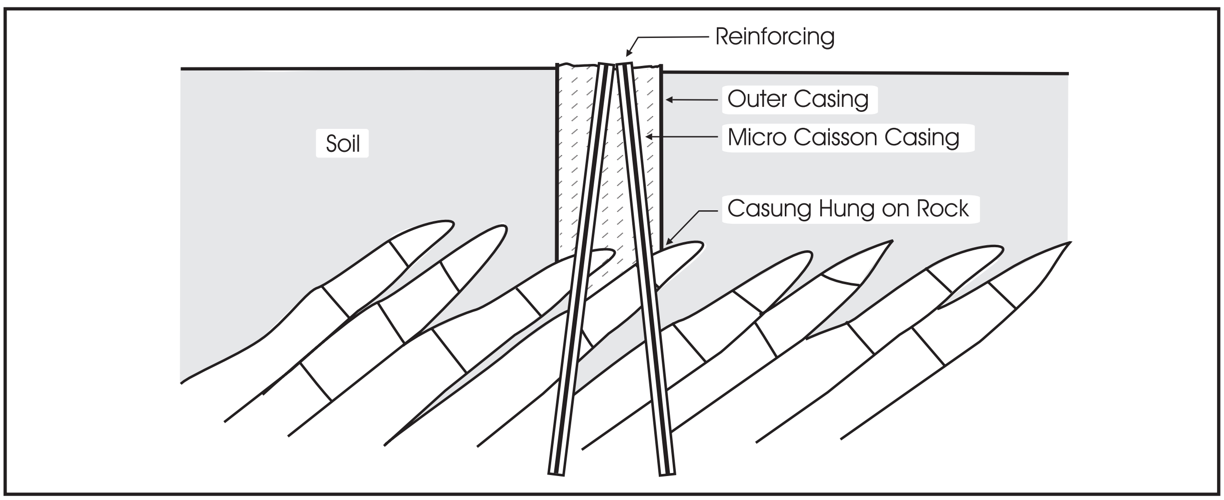


Figure 5.22: Micro Caissons Inside Large Caisson Hung on Rock Seam



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APPENDIX A5.1

EXAMPLES OF PROCEDURES FOR DEVELOPMENT IN KARST TERRAIN

1.0 EXAMPLES OF PROCEDURES FOR DEVELOPMENT IN KARST TERRAIN

In areas of major karst formations in the USA, there are several Ordinances/Zonings to follow during a study and design processes for development in the karst terrain. Two examples of phased investigation program for development in karst region that have been introduced and approved; one in Clinton Township, New Jersey and the second in Lower Macungie Township, Pennsylvania, are provided below for reference.

a) Example 1: Clinton Township, New Jersey

- **Checklist 1- Preliminary Investigation Requirements.**

  - Review of Critical Geological Formation Zone (C.G.F. Z.) map of Clinton Township
  - Review of New Jersey geological survey maps
  - Review of USDA publications and maps on Hunterdon County, Natural Resource Inventory prepared by the south branch Watershed Association (Vol. 1).
  - Review of Special Report #24, Geology and Groundwater Resources of Hunterdon County, New Jersey, Division of Water Policy and Supply, Department of Conservation and Economic Development, N.J. State Geologic Survey, 1966.
  - Submit with this checklist a site plan map at a scale of 1:24,000 identifying proposed development site and boundaries of site that are within the C.G.F.A. and/or C.F. W. P.A. as designated on the C.G.F.Z. Map.
  - Review of aerial photographs for the proposed site and surrounding area (at a minimum scale of 1"-1,000", obtained during periods of little or no foliage cover.).
  - Submit a summary of all known water production well logs and previous known subsurface investigations in the immediate area.
  - submit a site map at a scale of 1" - 100' with a contour interval of two feet identifying existing surface water bodies, topography of the site, location of any existing water production wells.
  - submit any other published geologic information available to applicant which applicant deems pertinent.
  - submit the following information on the proposed development:

    Owner's name and address:
    Developer's name and address:
    Location of proposed development site:
    Tax Block, Tax Lot(s)
    Type of proposed development:
        Residential
        Commercial
        Industrial
        Single-family
        Multi-family
    Proposed density (units per acre or lot coverage)
    Any other data that applicant wishes the Planning Board to consider:

- **Checklist II - Detailed Investigation Program**

  Checklist II is a detailed outline of the proposed investigation program, including reference to site specific investigation techniques, equipment and program objectives. Proposed investigation program to be conducted in C.G.F.Z. in Clinton Township.

  A) General Requirements:

  Test borings and test pits are to be used as the primary means of identifying potential geological hazards.

Percussion probes geophysical techniques (e.g. seismic refraction and reflection, ground penetration radar, magnetic, gravity and conductivity) can be used to provide data between test borings and pits.

Proposed exploration techniques which are not outlined in the checklist may be submitted for review and possible inclusion in the approved investigation program. Alteration to the planned investigation program can be made during the progress of the field investigation by request to the Township Geotechnical Consultant (GTC) if so required by the nature of the encountered subsurface conditions.

The intention of the site investigation program is to define the nature and limits of possible design, construction and operating concerns that could result from the existence of carbonate soil and/or rock formations underlying the proposed development site.

B) Direct Testing Procedures:

. Test Borings
  a) numbered proposed
  b) depth anticipated
  (Note: If rock encountered is within 40' of ground surface, a minimum of 10' of rock is to be cored. Rock cores shall be a minimum of 2" in diameter, to be obtained by double tube, split barrel coring device).
  c) boring techniques to be utilized:
  (Note: unless written approval is authorized, all test borings will be drilled using rotary wash boring procedures without use of drilling muds. Water losses in borings are to be monitored as to depth and quantities.
  d) proposed borehole grouting techniques
  e) soil and rock sampling to be performed in accordance with ASTM Standards D420, D1586, D1587, and D2113.
  f) logging of all test borings or test pits in accordance with the Unified Soil Classification System and in relation to the geologic origin of the constitutents of the encourntered materials, e.g., light yellow brown silty clay (CH), with occasional angular dolomite fragments, moderately stiff, residual soils, some stained paleo jointing.

. Test Pits
  a) number and depth of proposed pits
  (Note: to be acceptable, minimum bottom area of pits shall be 10 square feet and shall encounter rock surface over 50% of the pit area).
  b) method of backfill to be employed
  (Note: Test pit backfill shall be composed of excavated material, placed in layers and compacted to pre-excavation density, unless authorized otherwise by GTC).

. Piezometers, Lysimeters And Water Table Data
  a) number, locations and types to be used
  b) other methods to be used
  Noted: These shall be installed and monitored in sufficient locations to identify depth to seasonably high water table and rate and direction of ground water flow.

C) Indirect Testing Procedures:

. Percussion Probes
  a) number proposed
  b) depths anticipated
  c) measuring techniques to be utilized (air, loss, drilling speed and rod drops must also be monitored)

. Geophysical Studies
  a) seismic refraction and reflection: location and number of runs anticipated; equipment to be used
  b) ground penetrating radar specify procedures ad location of traverses
  c) magnetic, gravity or conductivity techniques

. Geologic Reconnaissance
Factors to be examined - soil types, rock types, vegetative changes, observable seeps or ground water discharge, circular depressions, swales."

b) Example 2: Lower Macungie Township Karst Ordinance 1205: Karst Hazard (Overlay District)

1. Purpose

The purpose of this district is to recognize the potential for
or death, and the disruption of vital public services which may arise by the potential for sinkholes and/or subsidence within potential for such sinkhole and/or subsidence occurrence and to protect the ground water resource.

2. Application

The Karst Hazard District operates as an overlay district to the districts otherwise found in the Township Zoning Ordinance. Should the regulations of this overlay district and other applicable regulations conflict, the most stringent regulations shall apply.

3. Disclamation of Liability

Whereas the exact occurrence of sinkholes and/or subsidence is not predictable, the administration of these regulations shall create no liability on behalf of the Township, the Township engineer, Township employees, or township agencies as to damages which may be associated with the formation of sinkholes of subsidence. That is, compliance with these regulations represents no warranty, finding, guarantee, or assurance that a sinkhole and/or subsidence will not occur on an approved property. The municipality, its agencies, consultants and employees assume no liability for any financial or other damages which may result from sinkhole activity.

4. Delineation of Area Affected

The Karst Hazard Overlay District is portrayed on the Karst Hazard Indicator Map which can be found at the end of this Ordinance. The areas affected by the Karst Hazard Overlay District are the "carbonate" areas shown on the map. The sinkholes, solution pans, kettle holes, quarries and limonite excavations delineated on the Karst Hazard Indicator Map were taken from two sources: (1) Sinkhole Occurrence and Geologic Maps prepared by VFC Engineering & Construction Services as part of the Lehigh-Northampton Sinkhole Study; (2) the Little Lehigh Creek Basin Carbonate Prototype area Closed Depression Map prepared by R.E. Wright Associates, Inc. Should dispute arise as to the boundary of the district or the location of any of the karst features shown on the map, the applicant may present information to the Zoning Officer in support of his or her position. This information shall be prepared by a recognized professional with competence in the field. The Zoning information presented, with the assistance of any technical review deemed appropriate.

5. Procedure

Whenever an applicant for a building permit, conditional use or special exception is made, the Zoning Officer shall determine from the Karst Hazard Indicator Map whether or not karst features are likely to be present and shall so notify the applicant. The applicant must provide the Township with a map at a scale of 1”=100’ that shows the karst features listed in 5.2 of this section.

5.1 Whenever notified by the Zoning Officer that karst features are likely to be present, or when the applicant knows these features are present, the applicant shall engage a qualified engineer to review the existing aerial photos, soils, geological and related data available to him as it may pertain to the subject property and to make a site inspection of the property.

5.2 A site inspection by the applicant’s engineer, using all available data and with such assistance as is needed, shall determine the presence or absence of karst surface features on the site, and locate the same if present on a site plan at a scale no smaller than 1”=100’, In particular, the following features shall be located, if present, on the site:

- 5.2.1 Closed depressions
- 5.2.2 Open sinkholes
- 5.2.3 Seasonal high water table indicators
- 5.2.4 Unploughed areas in ploughed fields

5.2.5 Surface drainage into ground
5.2.6 “Ghost lakes” after rainfall
5.2.7 Lineaments and faults
5.2.8 Limonite excavations and quarries
5.2.9 Contacts between geologic formations
5.2.10 Any karst feature shown on the Karst Features Indicator Map.

5.3 Based upon the site inspection, the applicant’s engineer shall determine what further testing should be done by the applicant to ensure compliance with the performance standards set forth in Section 6. Testing methodology
u shall be reasonable under the circumstances, incl
hazards revealed by examination of available data and site inspection.

5.4 The applicant shall cause the additional testing if any, to be effected and shall submit test results to the Township Engineer.

5.5 The Township Engineer shall report to the Zoning Officer and Planning Commission, with a copy to the applicant, his opinion concerning the adequacy of the report submitted based upon the scale of the development and the hazards revealed by the report, and shall make recommendations to the Planning Commission based upon the report submitted concerning the layout of utility lines, and building location. the Township Engineer may require the applicant to perform such additional testing as may be appropriate.″

6 Performance Standard

6.1 All applicants for building permits, subdivision, land developments, conditional uses and special exception uses shall comply with the requirements of the Karst Hazard Overlay District.

6.2 No stormwater detention facility shall be placed within one hundred (100) feet of the features listed in Section 1205.5.2.

6.3 No stormwater swale in excess of ten(10) cubic feet per second for the ten (10) year flood may be constructed within on hundred (100) feet of the features listed in Section 1205.5.2.

6.4 No storm sewer pipe shall be constructed within one hundred (100) feet of the features listed in Section 1205.5.2 unless it is concrete pipe utilizing 0-ring joints.

6.5 No principal or accessory building, no structure, and no impervious surface shall be located closer than fifty (50) feet from the edge of the features listed in Section 1205.2 unless a detailed geotechnical solution to the subsidence, pollution, and safety problems of the karst feature has been presented by a competent professional in carbonate terrain.

6.6 No septic system or tile field, no swimming pool no solid waste disposal area, transfer area or facility, no oil, gasoline, salt or chemical storage area, and no blasting for quarrying or well enhancement activities shall occur within one hundred (100) feet of the features listed in Section 1205.5.2 unless a detailed geotechnical solution to the subsidence, pollution, and safety problems of the karst feature has been presented by a competent professional in carbonate terrain.

6.7 Soil conservation plans filed with the County Soil Conservation Service shall detail safeguards to protect karst features from runoff changes.

6.8 All underground utility lines located in the Karst Hazard Overlay District shall be so constructed as to not permit the flow of water along the utility line trench, and shall be imperviously dyked at thirty (30) foot intervals.

APPENDIX A5.2

GEOTECHNICAL INVESTIGATION TECHNIQUES FOR KARST FEATURES

A5.2 Geotechnical Investigation Techniques

5.2.1 Indirect Methods

a) Geophysical Detection of Karst Features

Karst features present a contrast in physical properties with respect to the surrounding rock mass that often make them detectable by surface geophysical methods. The most relevant surface geophysical methods to be applied are: microgravity, seismic reflection, seismic refraction, ground penetrating radar (GPR), resistivity imaging and electromagnetics (EM).

- **Microgravity**

  Because karsting can produce a subsurface cavity which will be either water or air filled, the difference in mass with respect to the surrounding rock mass can produce a drop in the microgravity potential field measured at surface. Microgravity methods therefore are commonly used to investigate for karst cavities and the technique can be quite successful provided that the cavity is at a depth on the order of
  of microgravity survey application is presented on Figure A5.2.1.

- **Seismic Reflection**

  A cavity produced by karsting also presents a contrast in compressional wave velocity with respect to the surrounding rock mass. If the cavity is relatively on the order of 1 to 5 times the wavelength of compressional seismic waves, then the seismic reflection method can be used to detect such features. The cavity acts essentially as a point source, and a diffraction pattern observed on the seismogram indicates the position and depth of the cavity.

- **Seismic Refraction**

  If a cavity due to karsting has caused a collapse feature which has been subsequently overlain by unconsolidated sediments, then this depression in the upper bedrock surface can be detected with the seismic refraction method. Very thick overburden (> 40 metres) may preclude the usefulness of this technique, particularly if the water table is at a depth greater than 10 metres.

- **Ground Penetrating Radar**

  Particularly if a cavity is water filled, it presents a significant change in dielectric conductivity with respect to the surrounding rock mass, and can sometimes be detected by the GPR method. Like the seismic reflection method, the cavity will produce a diffraction pattern on the radargram indicating its position and depth. If overlying sediments are clay rich or overlying rock is shale, the penetration of this method will be limited. The best detection will be achieved when the cavity is a size on the order of 1 to 5 times the GPR wavelength. An example of GPR technique application is presented in Figure A5.2.2.

- **Resistivity Imaging and Electromagnetics**

  If the cavity is water filled, it will have a contrasting electrical resistivity, or conductivity, with respect to the surrounding rock mass. This makes the cavity a potential target for resistivity imaging or EM. Resistivity imaging is preferred over EM because electrical properties versus depth can be determined by resistivity imaging in addition to spatial variations in electrical properties. This makes resistivity imaging able to detect a wider range of cavity sizes at a wider variety of depths. For successful detection, the cavity should be of a size as large as one-quarter of its depth below surface. Very dry conditions at surface may limit the depth of investigation of resistivity imaging.

Figure A5.2.1: Microgravity Survey Application

A microgravity suvey was undertaken in Ireland. Although there was no surface expression of sink holes, it was known that they existed in the underlying dolomitic limestone. The microgravity survey revealed a mass deficit in the central portion of the survey area. Drilling confirmed that this feature was a sink hole. In the absence of sink holes, the average depth to bedrock was less than 30 metres. The drill hole at the centre of the gravity target was abandoned at 90 metres without encountering bedrock.

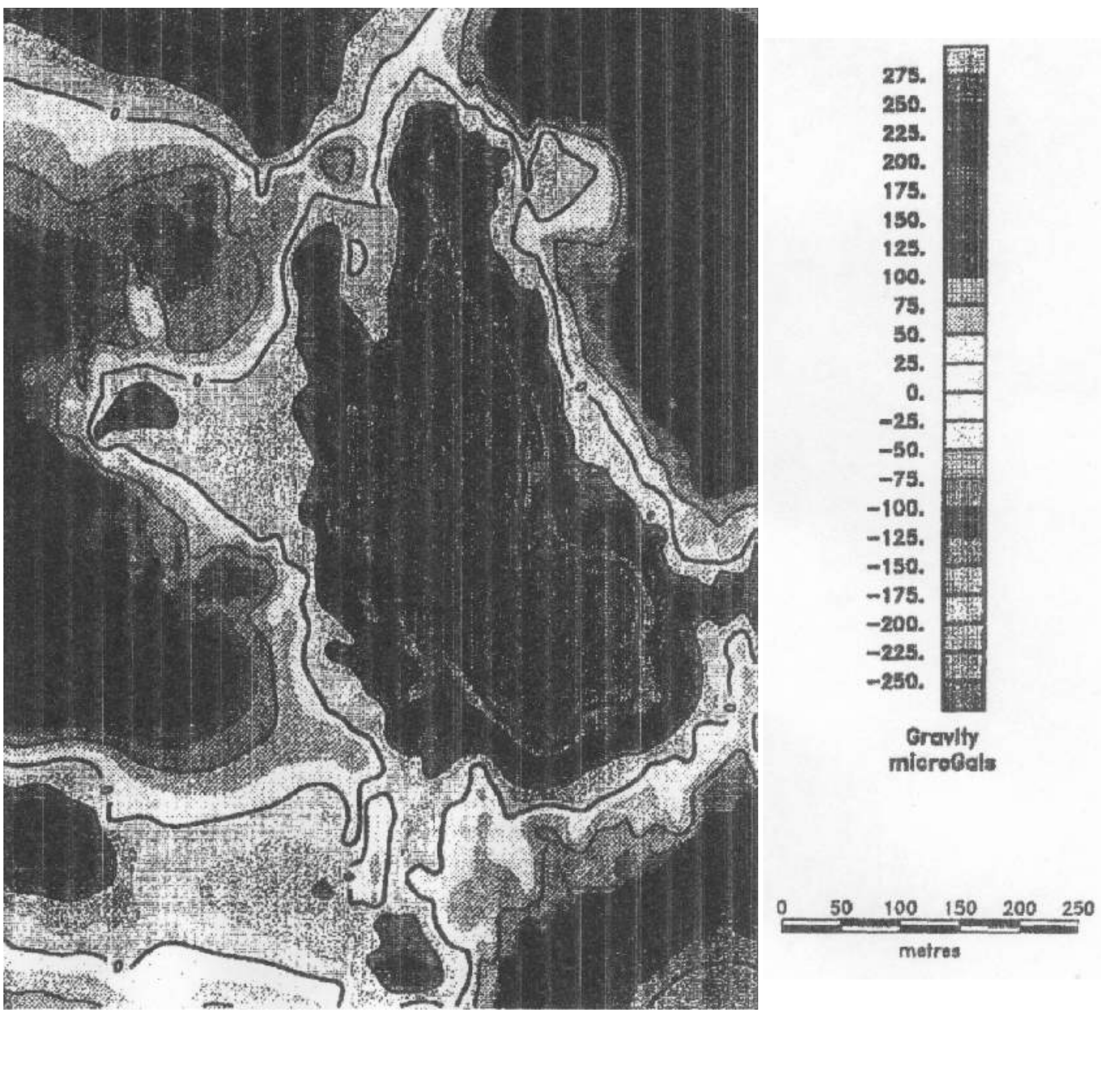
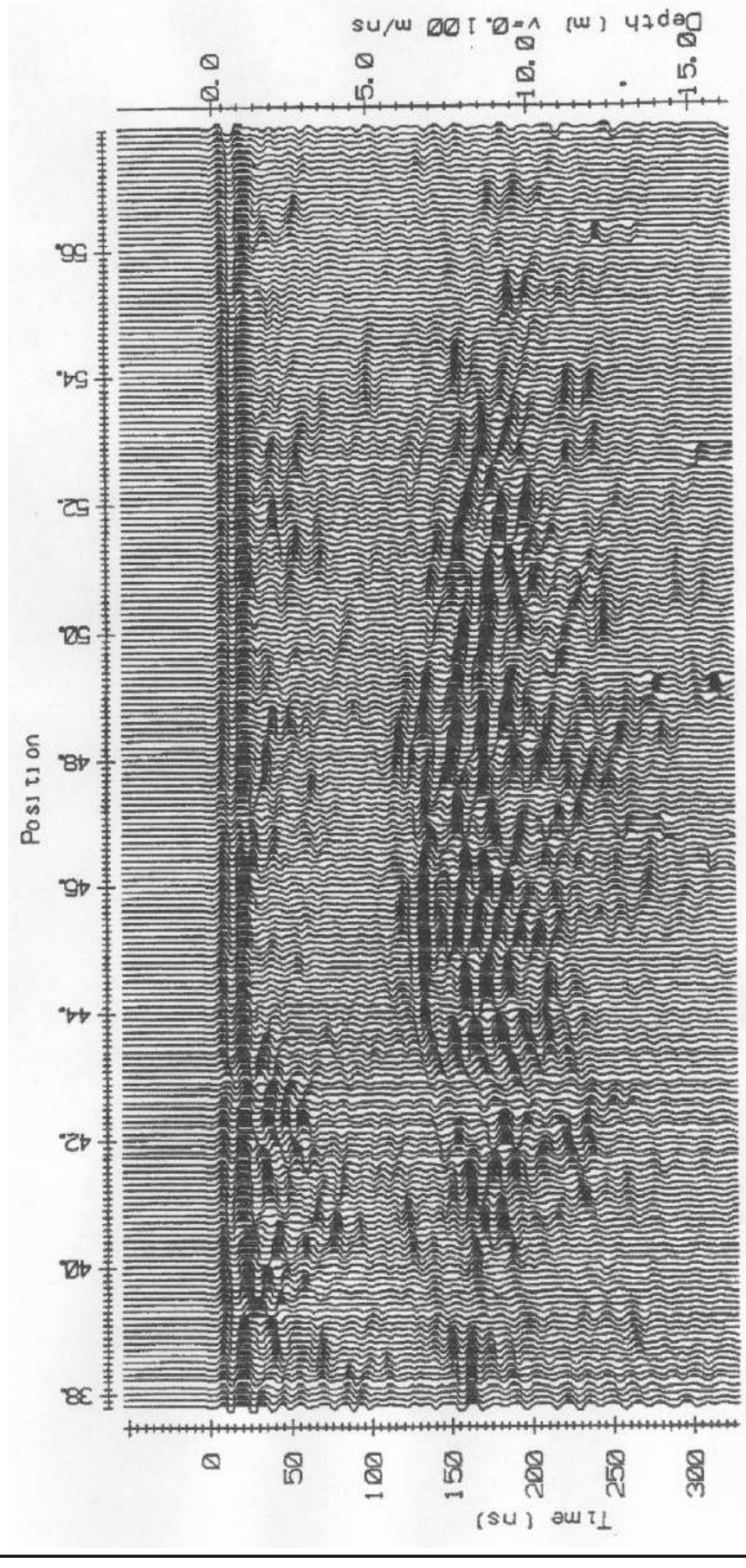


Figure A5.2.2: Ground Penetrating Radar Technique Application



Ground Penetrating Radar (GPR) was used to investigate for subsurface cavities at a site near Rockwood, Ontario. The site consists of limestone and dolostone overlain by a very thin (<30 cm) veneer of clay soil. The diffraction anomaly centered at line position 47 metres on the radargram below was identified and interpreted to be consistent with the presence of an air-filled cavity at a depth of about 6 metres, underlain by a rubble filled zone to a depth of about 10 metres. This anomaly was subsequently drilled and confirmed to be an air-filled cavity. This example is courtesyof the Geophysics Group at the Waterloo Centre for Groundwater Research, Waterloo, Canada.

b) Airtrack and Percussion Grid Pattern Drilling

Pneumatic probes put down on site in a grid pattern can provide information on the depth of the bedrock and an extent of karst features on site. This technique can be valuable when used in conjunction with test boring. Although, no core can be extracted, depths to the bed
where a loss in return air is experienced or when the air loss in one hole returns from another aids in the interpretation of the drilling results. With an experienced operator,
the overburden soils. The airtrack rig mobility, the speed of drilling and the relative economy of use make this investigation method very useful in defining zones of problem.

Alternatively, percussion drilling can be carried out to probe cavities and weaknesses in rock. The percussion drilling involves power chopping of soils or rock with a limited amount of water at the bottom of the hole. Changes in subsurface conditions are indicated by rate of drilling progress, action of drilling tools, and composition of slurry removed.

5.2.2 Direct Methods

- **Electronic Piezocone Sounding**

  The piezocone test consists of pushing a cone penetrometer into the soil and electronically measuring the corresponding cone tip resistance, sleeve friction resistance and dynamic and equilibrium pore water pressures. These measured properties can be used to classify soils, estimate soil shear strength and analyze the hydrogeologic characteristics of the explored area. Piezocone data are obtained continuously in the sounding as the cone is pushed from the ground surface to a depth at which refusal is encountered. Loosened zones (i.e., ravelling conditions) associated with karst features can be identified as well as measurements indicative of groundwater flow into an active sinkhole.

- **Test Pit Excavation**

  Test pit excavations can be used to identify soil fabric disturbance in the karst bedrock area. The face of the excavations are examined for the presence of locally dipping strata, mixed lithologies, shearing, faulting, dissolution residues or precipitates. The test pit excavations can provide useful info
  overburden thickness are of limited vertical extent.

- **Borehole Drilling**

  Borehole drilling and obtaining soil samples as part of the Standard Penetration Test (SPT) is the most common subsurface exploration method. The soil samples obtained are collected for examination and laboratory classification testing. Loosened zones which might be associated with karst formations can be detected based on the results of the Standard Penetration Test. The thickness of the loosened zone can be determined by extending the borehole to the depth at which "practical" refusal to further penetration is encountered, typically corresponding to SPT values of greater than 100 blows per 0.3m penetration.

- **Rock Core Drilling**

  Rock core drilling refers to the procedures in which rock underlying the area is investigated by coring to obtain intact samples of the rock. Rock coring is commonly performed to determine the quality of rock and to check for possible detrimental properties such as fractures, weathering and other deterioration that could affect the strength of the rock formation. Core runs are made to drill the hole in segments, usually 1.5m in length. At the completion of a core run, the barrel and the rock sample are brought to the surface; the rock sample is removed for examination and the barrel is reinserted for additional drilling.

In case of deep hole coring (e.g., over 30m) and sampling, wire-line drilling method is more efficient. The method involves rotary drilling where the coring device is an integral part of the drill rod string and it also serves as a casing. Core samples are obtained by removing the inner barrel assembly from the core portion of the rod.

6.0 Glossary

Adsorbed Water Water bound to soil particles because of the attraction between electrical charges existing on soil particle surfaces and (dipole) water molecules.

Airform Pattern An arrangement of shapes, apparent at a particular altitude, which is characteristic for significant terrain entities and their spatial relationship and thus useful in the application of aerial interpretation.

A-Line In a plasticity chart, the A-line represents the empirical boundary between typical inorganic clays which are generally above this line, and the plastic soils containing organic colloids which are below it.

Amorphous-Granular A descriptive term applied to one of the primary macroscopic elements of peat which is granular in nature but to which no particular shape can be ascribed.

Angle of Internal Friction The maximum angle of obliquity between the normal and the resultant stress acting on a surface within a soil.

Angle of Repose The angle with a horizontal plane at which loose material will stand on a horizontal base without sliding.

Arching The transfer of stress from a yielding part of a soil mass to an adjacent, less yielding or restrained part of the mass.

Ash Content The ash or mineral residue remaining after a peat sample has been ignited, expressed as a percentage of the dry weight; also known as ignition loss. Expressed as ratio of dry weight, it is known as ignition loss ratio.

Atterberg Limits The liquid limit, plastic limit, and shrinkage limit for soil. The water content where the soil behaviour changes from the liquid to the plastic state is the liquid limit; from the plastic to the semisolid state is the plastic limit; and from the semisolid to the solid state is the shrinkage limit.

Backfill Soil material placed back into an area that has been excavated, such as against structures and in pipe trenches.

Bearing Capacity The pressure that can be imposed by a foundation onto the soil or rock supporting the foundation.

Bentonite A naturally occurring clay material, with the ability to swell significantly with the addition of water.

Blanket Bog Also blanket mire. See Bog, blanket.

Bog An area of confined organic terrain, the limits of which are imposed by the physiography of the local mineral terrain. Differentiated from “muskeg” mainly in terms of area but often because variations in coverage, peat structure, and topography occur more frequently than in extensive areas of organic terrain.

Bog, Blanket Equivalent of blanket mire, also “Terrainbedeckendmoore”. Peat information initiated in basins, drainage, axes, and on all water partings where the drainage slope is not too great, the peat forming a blanket over all gently undulating terrain. In Ireland it includes a variety of peatland types, both ombrogenous (water supplied by precipitation) and soligenous (water supplied by high water table), occurring between 200 and 700 m. Variation in surface vegetation is related to climate (and perhaps supply to atmospheric nutrients), geology, topography, state of erosion, land use, etc.

Bog Mire Confined organic terrain; equipment valent of bog.

Bog Raised Equivalent of raised mire and "Hochmoore". Peat development initiated in basins, peat growth producing a dome or cupola rising above the mineral groundwater table. The classic case is the Baltic raised bog with ring lagg.

Boring The method of investigating subsurface conditions by drilling into the earth. Frequently, soil or rock samples are also extracted from the boring for classification and testing.

Borrow Soil or rock material obtained from an off-site source for use as fill on construction projects.

Caisson Large structural chamber used to keep soil and water from entering into a deep excavation or construction area. Caissons may he installed by being sunk in place or by systematically excavating below the bottom of the unit to the desired depth.

Calcareous An adjective applied to rocks containing calcium carbonate.

Capillarity The movement of water, due to effects other than gravity, through very small void spaces that exist in a soil mass. Water movement occurs in very small channels such as capillary-sized openings because of the affinity between soil and water, which acts to increase the boundary of contact between the two materials, and the surface tension property developed by water in contact with air. Capillary flow can occur in a direction opposite to that of the pull of gravity.

Category, Peat A descriptive term, one of a series of 17, applied to combinations of primary elements of peat structure. See amorphous-granular, fine-fibrous, and coarse-fibrous.

Chemical Weathering The process of weathering whereby chemical reactions such as hydration, solution, oxidation, and ion exchange break down and possibly change rock and soil materials.

Clays (Clay Materials) Very small soil particles having a crystalline (layered) structure, created as the result of the chemical alteration of primary rock minerals. Most clay particles, because of their mineralogical composition, are flat or platelike in shape, with a large surface area to mass ratio. Clay particle dimensions are often smaller than 21l.

Coarse-Fibrous A descriptive term applied to one of the primary macroscopic elements of peat which may be woody or non-woody and has a diameter greater than 1 mm.

Coarse-Grained Soils Those soil types having particles large enough to be seen without visual assistance. The coarse-grained materials include the sand and gravel (or larger) soil particles.

Coefficient of Compressibility A stress strain ratio of a soil. Numerically, it is the slope of the void ratio pressure curve from a consolidation test.

Coefficient of Consolidation Obtained by plotting degree of consolidation against square root of time for any particular consolidation test.

Coefficient of Earth Pressure at Rest Ratio of the horizontal effective stress to the vertical effective stress for a condition of zero lateral strain during consolidation.

Coefficient of Permeability k The rate of discharge of water under laminar flow condition through a unit cross-sectional area of a porous medium under a unit hydraulic gradient and standard temperature (20°C).

Coefficient of Secondary Compression Is expressed as the shape of the settlement log time plot divided by the thickness of the peat sample at the beginning of the long-term or straight line stage; is also expressed in terms of change in void ratio over one cycle of the logarithmic scale on a plot of void ratio vs. log time.

Cohesion The bonding or attraction between particles of fine-grained soil that creates shear strength.

Cohesion, C The portion of the shear strength(s) indicated by the term c in Coulomb's equation: $s = c + \Phi \tan \Phi$, where Φ is the angle of internal friction. It has the nature of an intergranular binding force. Also see shear strength.

Compaction The process of increasing the density or unit weight of a soil (frequently fill soil) by rolling, tamping, vibrating, or other mechanical means.

Compression index The slope of the void ratio log pressure curve from a consolidation test. The larger the compression index, the greater the compressibility of a soil.

Compressibility The change, or tendency for change, that occurs in the thickness of a soil mass when it is subjected to compressive loading.

Compression The reduction in volume of a soil mass caused by the application of external forces.

Conduit Pipe that is buried in a soil mass or passing through a soil embankment, and carries water or other fluid materials, electrical cables, and the like.

Consolidation The process by which compression of a newly stressed clay soil occurs simultaneously with the expulsion of water present in the soil void spaces. Initially, the newly imposed stress acting on the clay is imparted onto the water in the soil voids (pore water), and not onto the soil particles. Because of the increased pressure, the water is gradually forced out of the soil. As the pore water pressure is reduced, the magnitude of stress being imposed onto the soil particles is correspondingly increased. Compression of the clay layer occurs only as rapidly as pore water can drain from the soil, and this is related to the permeability of the soil layer.

Creep An increase in plastic strain with time is usually called creep. Also associated with the long term, gradual movement of a slope or embankment.

Crest The top of a slope or embankment.

Critical Surface (Critical Failure Surface) The sliding surface for which the factor of safety is at a minimum in an analysis of a soil slope.

Degree of Saturation, S The ratio of the volume of water in the soil voids to the total volume of voids.

Density (Mass Density) The mass of a material in relation to a unit volume. See also Relative Density.

Density The mass per unit of volume. In reference to soil, the term often also indicates weight per unit volume and is synonymous with unit weight.

Dewatering The procedure utilized to remove water from an area, such as pumping from an excavation or location where water covers the planned working surface, the procedure used to lower the groundwater table in order to obtain a "dry" area in the vicinity of an excavation which would otherwise extend below water.

Dispersive Clays Clay soils that deflocculate in still water and erode when exposed to a low-velocity flow of water. A clay-pore water system that has a high concentration of sodium ions tends to have high dispersivity.

Drawdown The lowering of the level of the groundwater table that occurs in the vicinity of a water well (on dewatering equipment) when it is pumped.

Dynamic Compaction Procedure Whereby surface and near-surface zones of soil or fill are compacted by dropping a heavy weight (commonly 5 to 15 tons) from a relatively great height (drops of 30 to 100 ft are typical). Multiple poundings are provided at each drop location, and closely spaced drop locations are utilized to improve a construction site.

Earth Pressure Normally used in reference to the lateral pressure or force imposed by a soil mass against an earth-supporting structure such as a retaining wall or basement wall, or on a fictitious vertical plane located within a soil mass. The coefficient of earth pressure refers to the ratio of lateral pressure to vertical pressure existing at a point in a soil mass.

Effective Stress The average normal force per unit area transmitted from grain to grain in a granular mass. It is the stress that is effective in mobilizing internal friction.

Effective Stress The actual particle-to-particle contact stress (or pressure) existing between soil grains. This stress compensates for the possible buoyancy influence of water pressure. Effective stress relates directly to the shear strength possessed by a soil.

Factor of Safety The ratio of available shear strength to shear stress on the critical failure surface.

Fen Muskeg (Mire/Peatland) Consisting of organic terrain supplied with eater previously in contact with mineral soil. If the area or areas are large this minerotrophic condition arises through contact with mineral subsoil rather than from surrounding mineral soil or rock slopes.

Fill Earth placed in an excavation or other area to raise the surface elevation. Also referred to as earth fill or soil fill. Structural earth fill refers to material which is placed and compacted in layers in order to achieve a uniform and dense soil mass which is capable of supporting structural loading.

Filter Natural soil or artificial material with pore sizes such that it will permit the free passage of water, yet sufficiently small to prevent the passage of fine soil panicles from the protected soil.

Fine-Fibrous A descriptive term applied to one of the primary macroscopic elements of peat which may be woody or non-woody and has a diameter less than 1 mm.

Fine-Grained or Fines Refers to silt and clay-sized particles that exist in a soil mixture.

Finger Drains A drainage system which consists of strips of pervious drainage materials.

Flow Line The path of travel traced by moving water as it flows through a soil mass.

Flow Net A pictorial method used to study the flow of water through a soil. Used to indicate the paths of travel followed by moving water and the subsurface pressures resulting from the presence of the water.

Footing Type of foundation typically installed at a shallow depth and constructed to provide a relatively large area of bearing onto the supporting soil.

Friction, Internal The particle (solid to solid) friction developed by cohesionless soils, and the property responsible for most of the shear strength which this type of soil can develop.

Gabions Stone-filled steel wire baskets which can be assembled or stacked like building blocks to act as retaining walls or provide slope and erosion protection.

Groundwater Table The surface of the underground supply of water. Also referred to as the phreatic surface. The level below which the pores of the subsoil, down to indefinite depth, are full of water.

Growth Habit A contributory distinguishing property of vegetal coverage used in conjunction with stature and woodiness vs. non-woodiness to determine cover class. A description of plant form and arrangement.

Head Shortened form of the phrase pressure head, referring to the pressure resulting from a column of water or elevated supply of water. Pressure would be computed from $K_w h$, where K_w is the unit weight of water and h is the height or elevation of the water supply. The h term is the pressure head.

Hydraulic Gradient Mathematical term indicating the difference in pressure head existing between two locations divided by the distance between these same locations. Given the designation i.

In-Situ Refers to soil when it is at its natural location in the earth and in its natural condition.

Isotropic Pertaining to a soil whose properties are the same in all directions.

Landslide The relatively rapid lateral and downhill movement of a generally well-defined earth mass or land form due to gravitational forces.

Limit Equilibrium A method of analysis used to evaluate the stability of soil mass (such as in a slope or foundation support) that could be involved in movement associated with failure. The method involves determining the soil shear strength on an assumed failure surface as required to maintain equilibrium or stability, and compares this value with the actual shear strength of the soil; this comparison indicates if equilibrium will exist or if the limits of equilibrium will be exceeded.

Liquefaction Loss of strength occurring in saturated cohesionless soil exposed to shock or vibrations when the soil particles momentarily lose contact. The material then behaves as a fluid.

Liquid Limit, W_L The water content at which a soil exhibits low shearing strength is taken to be the boundary between the soil's liquid and plastic behaviour.

Marsh Low-lying tract of land with a high water table, usually covered with grass or sedge-like plants growing directly on mineral terrain.

Mechanical Weathering The process of weathering whereby physical forces, such as frost action and temperature changes, break down or reduce rock to smaller fragments without involving chemical changes.

Method of Slices A general procedure used for slope stability analysis in soil. A trial surface is chosen and the potential sliding mass is divided into a number of vertical slices. Each slice is acted on by its own weight which produces shearing and normal forces on its vertical boundaries and along its base.

Mineral A naturally formed chemical element or compound having a definite chemical composition and usually a characteristic crystal form.

Mound A microtopographical feature with a rounded top, often elliptic or crescent shaped in plane view.

Muskeg The term designating organic terrain, the physical condition of which is governed by the structure of peat it contains, and its related mineral sublayer, considered in relation to topographic features and the surface vegetation with which the peat co-exists.

Organic Content The weight of the organic material present in a sample of soil, expressed as a percentage of the dry weight.

Organic Terrain A tract of country comprising a surficial layer of living vegetation and a sublayer of peat or fossilised plant detritus of any depth, existing in association with various hydrological conditions and underlying mineral formations.

Peak Shear Strength At a certain level of shear stress the shear strength of the surface is exceeded and funkier displacement will take place without any further increase in shear stress. This limiting value defines the peak shear strength at that particular normal stress.

Peat A component of organic terrain consisting of more or less fragmented remains of vegetable matter sequentially deposited and fossilised.

Penetration Test Term generally applied to subsurface investigative methods for determining a strength-related property of a soil by measuring the resistance to advancement of penetration or boring equipment.

Permafrost The permanently frozen ground located in the northern regions of the earth.

Permeability The ability of water (or other fluid) to flow through a soil by travelling through the void spaces. A high permeability indicates flow occurs rapidly, and vice versa.

pH The measure of soil acidity. Defined as the negative logarithm of the hydrogen ion concentration in an aqueous suspension of the soil.

Phreatic Surface The surface along which the pressure in the fluid equals atmospheric pressure.

Pier Category applied to column like concrete foundations, similar to piles. The pier is generally considered the type of deep foundation that is constructed by placing concrete in a deep excavation large enough to permit manual inspection. Pier is also used frequently to indicate heavy masonry column units which are used for basement-level and substructural support.

Piezometer A device for measuring the hydrostatic pressure at a point in the ground. Simple piezometers are open holes or standpipes for measuring the groundwater table.

Pile The relatively long, slender, column like type of foundation that obtains supporting capacity from the soil or rock some distance below the ground surface.

Piping Erosion by subsurface water moving through a soil zone, which results in the formation of continuous tunnels or "pipes" through which water then travels rapidly. Progressive erosion or cave-in of th
permeable dam foundations.

Plane Strain A state of strain in which all displacements that arise from deformation are parallel to one particular plane.

Plastic Limit, W_p A boundary region of water content representing a change in characteristics of the soil from those of a plastic to those of brittle material. This water content is called the plastic limit.

Plasticity Term applied to fine-grained soils (particularly clays) to indicate the soils (plus included waters) ability to flow or be remoulded without ravelling or breaking apart.

Plasticity Index The numerical difference between the liquid and plastic limits; indicates the degree of plasticity of a soil.

Plate Tectonics The concept that the earth's outer zone consists of a small number (10 to 25) of large thick plates that "float" on a viscous underlayer and can move more or less independently. The continents are carried on the plates and move with them; oceans are similarly carried on the plates and expand or shrink as the distances between continents change.

Pore Pressure Water pressure developed in the voids of a soil mass. Excess pore pressure refers to pressure greater than the normal hydrostatic pressure expected as a result of position below the water table.

Preconsolidation
A construction technique whereby a load in excess of that which will be finally carried by the soft stratum is placed and allowed to settle until the ultimate settlement that would occur under the final load has been reached. The excess load or surcharge is then removed and the construction is completed; also called preloading.

Pressuremeter An instrument used to determine the in-situ strength of a soil zone through measurement of the pressure related lateral expansion of a flexible cylinder which is at a known depth in a borehole.

Principal Stressors Stresses acting normal to three mutually perpendicular planes, intersecting at a point in a body, which is at equilibrium.

Relative Density Term applied to sand deposits to indicate a relative state of compaction compared to the loosest and most dense conditions possible.

Residual Shear Strength After a surface of sliding forms and extensive slip occurs, the particles along the slide surface assume an orientation favourable to a low resistance to shear, and shear displacement takes place at a constant shear stress level. This shear stress is called residual shear strength.

Rock Flour A general term for fine-grained, non-plastic material corresponding in size to silt, comprised of more or less equidimensional grains of unweathered mineral; particles.

Rotational Shear Slide
A slide resulting from the yielding and redistribution of shear stresses in a soil so that a more or circular surface of failure envelope develops before the cohesion breaks down and permits a comprehensive, circular sector of the slope to fail by rotating.

Sand The category of coarse-grained soil whose particle sizes range between about 0.07 and 5 mm in diameter.

Seepage Generally refers to the quantity of water flowing through a soil deposit or soil structure such as an earth dam. Also may refer to the quantity of subsurface water leakage into a building's underground (basement) area.

Seismic Exploration The method of determining subsurface soil and rock conditions (without excavation) by inducing a shock wave into the earth and measuring the velocity of the wave's travel through the earth material. This seismic velocity indicates the type of earth material.

Seismic Pertaining to an earthquake or earth vibration, including one that is artificially induced.

Sensitivity A term designating the sensitivity of a cohesive soil to remoulding. It is the ratio of the undisturbed shear strength to the remoulded shear strength.

Settlement The downward vertical movement experienced by a structure or a soil surface as the underlying supporting earth compresses or consolidates.

Shear Strength The ability of a soil to resist shearing stresses developed within a soil mass as a result of loading imposed onto the soil.

Shear Strength The internal resistance offered to shear stress. It is measured by the maximum shear stress, based on original area of cross section, that can be sustained without failure.

Shear Stress (Shearing Stress) The stress component tangential to a given plane.

Silt The category of fine-grained soil particles (individual soil grains whose particle size is smaller than 0.07 mm or too small to be seen without visual aid) whose mineralogical composition remains similar to the rock they were derived from.

Soil Resistivity The electrical resistance of a soil. It is usually measured as the number of ohms resistance across 1 cubic centimetre of soil (ohm cm).

Soil Sampler The equipment used to extract soil samples from borings or test pits made in a subsurface investigation.

Soil Stabilization Treatment of soil to improve its properties; includes the mixing of additives and other means of alterations such as compaction or drainage.

Specific Gravity The ratio of the weight in air of a given volume of soil particles to the weight in air of an equal volume of distilled water at a temperature of 4°C.

Spruce Bog A term in common use loosely applied to confined areas of organic terrain where coniferous trees (often not spruce) are a prominent feature of the vegetable coverage.

Stratum A layer that is discernible along bedding planes from layers above and below it. The separation arises from a break in deposition or a change in the character of the material deposited.

Stress An intensity of force. The ratio of forces per unit area, acting within a body.

Sump Small excavation or pit provided in the floor of a structure, or in the earth, to serve as a collection basin for surface water and near-surface underground water.

Swamp Similar to marsh but usually with higher water table and interruptions in the vegetal mat.

Talus A term used for the description of the accumulation of mixed fragments and particles fallen at or near the base of cliffs or slopes.

Thermal Conductivity An empirical coefficient defined as the number of calories per second flowing through a plate 1 cm square and 1 cm thick with a temperature difference of 1◦C between the two surfaces of the plate.

Thermokarst Subsidence of the ground surface in permafrost regions, producing uneven undulations and hollows caused by the melting of ground ice.

Till Description given to glacially transported soil formations consisting of a heterogeneous mixture of fine-grained and coarse-grained material.

Total Stress The total force per unit area acting within a mass. It is the sum of neutral and effective stresses.

Unit Weight The weight per unit volume of a material such as soil, water, concrete, etc. Typically expressed as kilonewtons per cubic meter (kN/m^3) per cubic foot (lb/ft^3) on grams per cubic centimetre (g/cm^3).

Void Ratio The total volume occupied by a soil mass includes the soil particles plus void spaces (which in nature always exist between the particles because of their irregular shape). The void ratio is the ratio of the void space volume to the volume of soil solids.

Water Content The amount of water contained in the voids of a soil. In standard soil mechanics terminology it represents the loss in weight expressed as a percentage of the oven-dry material when the soil is dried to a constant weight at 105 to 110◦C.