Slope Stability Guidelines for Development Applications

[PDF 157 KB]

1.0 Introduction

The purpose of this document is to:

1. Provide some fundamental background information needed for City staff to understand and interpret engineering reports on the stability of natural slopes;
2. Provide a general guideline on the local (minimum) state-of-practice for completing those assessments of the stability of natural slopes.

These reports, generally prepared by engineering consultants working for property developers or individual home owners, are submitted to the City as part of applications for site plan approval, land subdivisions, or building permits. As discussed further below, these reports may also be required for Official Plan Amendments and re-zoning applications.

The general intent of these studies is to assess whether or not a slope is considered to be stable and, if not, to provide a “safe” set-back distance from the slope crest behind which structures and other facilities can be constructed. Unstable slopes and the land located between the unstable slope and the set-back line are called “Hazard Lands.” In a broad sense, Hazard Lands are those at risk of being adversely impacted by any natural geologic processes. In the case of unstable slopes, Hazard Lands are those that have a factor of safety of less than 1.5 against being affected by a slope failure. In some circumstances, Hazard Lands may, in accordance with Ontario’s Planning Act, be transferred to the municipality or other agency (such as a Conservation Authority). The development of Hazard Lands is restricted. They should not be developed with permanent structures, parking or roadway areas, amenity areas (i.e., the landscaped area around a house, typically in the rear yard, that could be developed with pools or decks), septic beds, or any other valuable infrastructure. Buildable lots in a residential subdivision should not include Hazard Lands. In some circumstances, it may be permissible for properties to contain Hazard Lands, provided a separate “buildable lot” exists, that is large enough to include an amenity area, and that on its own conforms to all applicable laws (e.g., zoning, building code, etc.), and provided that the Hazard Lands are entirely contained within a separate parcel of land. In this case, restrictive covenants would be placed on title that would limit or restrict certain uses within that separate parcel (e.g., no permanent structures). In a rural setting, with large lots, a separate parcel may not be required, provided there is large separation between the structure and active use areas from the Limit of Hazard Lands.

The location of the Limit of Hazard Lands must consider both the distance behind the slope needed to provide a factor of safety of 1.5 (or 1.1 for seismic loading conditions) and, if appropriate, allowances for future erosion and, in some cases, an additional allowance to allow for access to future slope failures.

The assessment of the stability of a slope and the determination of the Limit of Hazard Lands should consider the most conservative condition that the slope could reasonably be expected to experience (i.e., the “worst case” condition), such as might relate to groundwater levels, earthquakes, etc.

It should also be noted that some Hazard Lands may already be identified in Schedules of the City’s Official Plan, and through zoning by-laws. The Planning Act also requires that the local Conservation Authority (CA) be included in the circulation of development applications for properties along waterways within their watersheds; the CAs have been delegated by the Province the responsibility with regards to the management and prevention of erosion and slope stability hazards. Hazard Lands may be identified by CAs as part of that circulation and review. There are three Conservation Authorities whose watersheds include lands within the City of Ottawa, namely the Rideau Valley Conservation Authority, the Mississippi Valley Conservation Authority, and the South Nation Conservation Authority. In relation to the identification of Hazard Lands, an existing Memorandum of Agreement between these three CAs and the City of Ottawa directs that CA staff should be requested by the City to review slope stability assessment reports for development applications along their waterways.

Where the potential for Hazard Lands exists on a site, the City requires that slope stability assessment reports be provided before approval can be given for:

- Site Plans
- Subdivisions
- Severances
- Condominiums
- Building permits

However, depending upon the order of magnitude of concern with respect to slope stability, the City may also require that a slope stability assessment be carried out earlier in the development process, such as in advance of:

- An Official Plan Amendment (OPA)
- A re-zoning application, or establishment of a zoning
- Draft site plan or draft subdivision approval

Otherwise, the development plans could be found at a later date to not be feasible, subsequent to the slope stability assessment, new plans developed, and the application re-circulated to City staff and, if appropriate, to the local CA.

The focus of this document is on the local state-of-practice, which has developed in part due to Ottawa's geology. The state of practice may be different in other jurisdictions with different geologic conditions.

It should be emphasised that the background information and guidelines provided in this document are not intended to train the reader to make an independent interpretation or assessment regarding the stability of a slope. Nor does this document provide a comprehensive discussion of all possible factors involved in the assessment of the stability of slopes within the Ottawa area.

It should also be noted that there are many variations of local or personal practice amongst geotechnical engineers; those different practices may all be acceptable from an engineering perspective given that there are no specific codes which govern slope stability assessments. However some guidance on the issues that must be considered is provided in a series of documents produced by the Ministry of Natural Resources (MNR), as follows:

- Understanding Natural Hazards. Great Lakes – St. Lawrence river system and large inland lakes, river and stream systems and hazardous sites;
- Adaptive Management of Stream Corridors in Ontario;
- Technical Guide - River and Stream Systems: Erosion Hazard Limit;
- Geotechnical Principles for Stable Slopes; and
- Hazardous Sites Technical Guide.

Further, it should be noted that this manual addresses only the stability of natural slopes, and not embankments (slopes that are constructed, above the natural ground surface), which require different analyses. This document also only addresses the *geotechnical* engineering issues associated with slope stability assessments. Other related issues that may need to be considered by City staff include the preservation of the ecological and recreational functions of river valley lands and the general desire to maintain them within public ownership. As well, where slope modifications are proposed as part of creek/river bank erosion protection or for slope stabilization, those works must conform to regulations regarding aquatic and terrestrial habitat. These other issues, though important, are not addressed in this document.

2.0 Ottawa's Geology

The methods by which stability assessments are carried out for slopes in the Ottawa area are dictated in part by Ottawa's geology which, amongst other North American cities, is rather complex and challenging. A basic understanding of Ottawa's geology is therefore helpful to the understanding of slope stability assessment reports.

Ottawa's geology consists of some, but not necessarily all, of the (simplified) list of geologic materials given below, in approximate sequence from youngest to oldest (i.e., from top to bottom, in the soil profile):

1. Fill materials (either random fill or engineered fill)
2. Organic soils (e.g., topsoil and peat)
3. Sandy soils (e.g., sand, silt, silty sand, sandy silt, sand and gravel)
4. Sensitive silty clay
5. Glacial till
6. Bedrock

These materials are generally composed of mineral constituents. In simple terms, minerals are solid materials with specific chemical compositions and structures. Quartz and mica are minerals that are familiar to most people, however there are hundreds of other kinds of minerals. For bedrock, those minerals are bound together in a solid mass (in which you may be able to see the individual grains of different minerals, or the grains may be so small that the rock appears homogeneous, or there may be only one kind of mineral present), while soils are formed of particles made from those minerals. Those soil particles can be of different sizes which, in increasing size, are termed clay, silt, sand, gravel, cobbles, and boulders. The size limits for these particle types are generally considered to be as follows:

Table 1

Particle Type	Size Range[1]
Boulders	> 200 mm
Cobbles	75 – 200 mm
Gravel	4.75 – 75 mm
Sand	0.075 – 4.75 mm
Silt	0.002 – 0.075 mm
Clay	< 0.002 mm

Note: [1] – Different organizations and countries may use slightly different size ranges for each particle type. Those given in the table are those most commonly used by geotechnical engineers in the Ottawa area.

The geologic materials listed above are described below in further detail, in order of their formation (i.e., from oldest to youngest, or bottom to top):

2.1 Bedrock

In simple terms, there are two kinds of bedrock present in the Ottawa area, the granite bedrock of the Canadian shield and the sedimentary rocks present across the rest of southern Ontario, such as limestone, dolomite, sandstone, and shale.

Fundamentally, the earth's crust beneath the continent of North America consists of granite which, in simplified terms, consists of magma ("lava") from within the earth that cooled to form a solid crust over the earth's surface about 4.5 billion years ago. Granite is the bedrock type that forms the Gatineau Hills, well known to most Ottawa residents, but is exposed within the City of Ottawa only over parts of the Kanata Lakes community and along the ridge line that extends to the north, past the community of Carp.

However granite underlies all of the younger sedimentary bedrock types that exist with the remainder of the Ottawa area (often at up to 200 to 300 metres depth).

Those sedimentary bedrocks were formed from soils that were deposited as sediments on the floors of oceans that covered the Ottawa area several hundred million years ago. Those sediments solidified into rock, forming limestone, dolomite, shale, and sandstone, depending upon the type of soil from which they were formed.

2.2 Glacial Till

The last Ice Age started about 40,000 years ago. As the Earth's climate cooled, vast ice sheets and glaciers formed across much of North America. These glaciers were often several kilometres thick and would move (at a very slow rate) across the landscape under the pressures from topography and climate. All soils which existed at the time were scraped away by the glaciers. The underlying bedrock was also scraped by the tremendous pressure from the ice, carrying up large quantities of broken and crushed rock into the glaciers. When the glaciers melted, about 12,000 years ago, those materials were deposited, where the glaciers melted, as a sheet of glacial till. The glacial till therefore consists of all particle sizes: boulders, cobbles, gravel, sand, silt, and clay. These materials were deposited on the surface of the bedrock, which had been scraped clean by the glaciers. Though glacial till is not necessarily present at every location in the Ottawa area, where it is present, the glacial till always overlies the bedrock surface.

2.3 Sensitive Silty Clay

After the glaciers melted from this area (with the ice limit retreating to the north) eastern Ontario was flooded by the Atlantic Ocean, forming the Champlain Sea, the approximate limits of which are shown on Figure 1. The Champlain Sea covered this area for about 2,000 years, during which time the melting glaciers to the north were releasing massive amounts of melt water containing large quantities of sediment that had been present in the glaciers (scraped up from the underlying bedrock). Once the melt water flowed into the Champlain Sea, those clay and silt particles were deposited as a thick layer of silty clay on the floor of the sea. Up to about 60 metres of silty clay were deposited during that period. The Champlain Sea dried-up about 10,000 years ago, leaving the thick deposit of silty clay that covers eastern Ontario. That deposit is known as Champlain Sea Clay, or more commonly as Leda Clay.

Because the groundwater level is generally within about 2 to 5 metres of ground surface, the silty clay to that depth has been exposed to air, oxidized, and dried to form a brown or grey brown coloured crust that is relatively dry and stiff. The silty clay below the crust has never been exposed to air and is therefore grey in colour, wet, compressible, and relatively weak.

2.4 Sand Deposits

The sand deposits present in the Ottawa area originate from two main sources. First, in a broad delta extending across the Ottawa airport site and the lands to the south, formed by rivers flowing off the melting glaciers at the end of the last ice age. Secondly, along beaches that formed along the shores of old channels of the Ottawa river, such as in the Mer Bleu area east of the City, before it established its current course. However sand deposits also exist in many other areas within the City.

These sand deposits may vary greatly in composition, from clean "beach like" sand, to silt, silty sand, sandy silt, as well as sand and gravel, but their origin and engineering behaviour are all similar.

2.5 Organic Soils

The decomposition of vegetation, in relatively recent times, forms soils that may consist partly or almost entirely of organic matter (rather than only mineral particles). Common types include topsoil, peat, and alluvium. Topsoil is generally a mineral soil (such as sand or clay) with up to about 10 percent organic matter; it is the darker upper-most soil (usually less than about 0.3 metres thick) found in most undeveloped areas. Peat is generally found in “swampy” or “marshy” conditions and is formed by the accumulation and decomposition of organic matter to form a material that is almost entirely organic (i.e., no mineral soil). Alluvium is the name of soils (generally silt and sand) deposited as a dark silty “mud” along river and creek banks. A fourth type is marl, which, though not typically highly organic in composition itself, is commonly overlain by peat and is typically biologic in origin (containing numerous shells). Marl is white or grey in colour, feels much like a soft wet clay, but is weaker and often behaves “jello-like” when shaken.

2.6 Fill Materials

Fill materials, the youngest and therefore top-most layer, consist of soils excavated from their natural location and then re-placed at another location by the activities of people. As an example, fill materials are often placed as “landscaping fill” around the houses within a subdivision to regrade the site to a higher level. There are two broad categories of fill materials: random fills and engineered fills. Random fills are those fill materials that are typically uncontrolled (and thus variable) in their composition; they can contain any portion of clay, silt, sand, gravel, cobbles and boulders, and can vary in composition over short distances. They often contain organic matter and refuse (garbage). They are placed without compaction. In contrast, engineered fills are premium materials (such as crushed bedrock, clean sand, or clean sand and gravel), placed under controlled compaction conditions, and generally for the purpose of supporting other structures (such as a house or a pavement).

The above description provides a greatly simplified description of Ottawa’s geologic history and the major geologic materials that are present. It should be noted that all of these deposits are not generally present at every location. For example, the bedrock surface outcrops under much of the downtown area and there is very little natural soil cover, the Westboro area is underlain by glacial till over bedrock, and the Sandy Hill community is underlain by a thin sand cap over a thick clay deposit over glacial till over bedrock. For all three communities, though only some of the soils are present, the sequence of soil types (from youngest to oldest), is consistent with the above description.

The actual geology at a specific site can only be determined by drilling boreholes and taking samples of these materials. However there exists published geologic mapping by the Geologic Survey of Canada that provides a general overview of the natural geology (i.e., not including alteration by humans related to fill placement within different parts of the City). A somewhat simplified version of that mapping is provided on the attached Figures 2A and 2B.

3.0 Slope Stability

3.1 The Formation of Slopes in Ottawa’s Geology

As described above, much of the Ottawa area (though not the entire Ottawa area) is underlain by thin surficial layers of organic soil and possibly sand, then by a thick deposit of silty clay, then glacial till, and then bedrock. Most of the significant natural slopes within the Ottawa area are located along river and creek courses, such as the Rideau River, the Ottawa River, Green’s Creek, Bilberry Creek, and Cardinal Creek, amongst many others.

In simple terms, natural slopes (i.e., those not constructed by excavation or filling by people) are generally formed by the erosive action of flowing water, such as rivers, stream and creeks. Erosion and the formation of slopes is a natural part of the evolution of the topography. In simple terms, flowing water “picks-up” particles of soil as it flows along. By this manner, rivers become wider and deeper. At some point, the slopes along a river become sufficiently high and steep that the slope “fails” and a new, flatter slope remains. This process continues until a deep, broad river valley is formed. It would, in theory, continue indefinitely until the river bed level is at the same elevation as its eventual outlet. For example, rivers in the Ottawa area would continue to deepen, forming broader valleys, until the river bed level was at the level of the Ottawa river (the eventual outlet for essentially all of the surface water flows in the Ottawa area). However geology often places a natural limit on the process. In the context of Ottawa’s geology, with most of the river valleys and creek ravines cut into the thick sensitive clay deposit, that limit often occurs once the river bed has reached the level of the underlying glacial till (which is relatively resistant to erosion) or the bedrock surface. That natural evolution of a river/creek valley within the geologic conditions of Ottawa is shown on Figure 3, where erosion along a river gradually deepens and widens the valley, first by shallow failures of the river bank and then by deeper failures of the subsequent higher valley slopes, until the surface of the glacial till is encountered. In concept, the ultimate river valley would also, initially, have some unstable slopes. However eventually conditions such as full saturation of the slopes during particularly wet springs or (very infrequent) seismic events would trigger failure of the unstable slopes until the valley reached a stable slope geometry and a "mature" condition was created. It should be noted that wave action on the shores of lakes can also erode soil and form slopes, in a similar process.

It should also be noted that, particularly in the clay soils that underlie much of eastern Ontario, rivers do not flow along a straight path. They tend to meander back and forth along a winding route. The most active erosion, and therefore the locations of the most active instability, exist along the outsides of bends in the watercourse. Failures at those locations tend to create flatter slopes, sometimes increasing the amount of meandering, and may also re-direct the water flow at another section of river bank, subjecting it to higher erosion and ultimately creating another meander. This process is shown schematically on Figure 4.

3.2 Mechanisms of Slope Instability

Slope failures (i.e., landslides) occur when the forces generated by the weight of the soil in a slope exceed the shear resistance (strength) of the soil, as shown schematically on Figure 5. Figure 6 provides some of the terminology used in describing a slope failure. These failures can take the form of an essentially planar surface (more typical of failures in sandy soils) or more deep-seated failures with the movement apparently involving a curved failure surface and rotation of the soil mass around a point in space above the slope (more typical of failures in clay slopes). However erosion at the toe of a clay slope will often initially oversteepen the slope toe generating a series of shallow, planar sloughs, until ultimately a deep-seated rotational shear failure occurs which cuts further back up the slope and may involve the table land area (see Figure 7). In the extreme case, the soil displaced by the slope failure in a clay slope may flow out of the failure area, leaving a steep un-supported scarp which then also fails and, through repetition of that same mechanism, the failure can rapidly retrogress a significant distance into the table land. Failures of this type are called earth flow slides (or earthflows or flowslides), as shown on Figure 7.

So, in summary, although planar slides predominate in sandy slopes, failures in clay slopes can consist of three basic types, as summarized on Figure 7:

1. Sloughs, which are generally the result of erosion at the slope toe, generally involve a shallow and planar failure surface, typically affect only the slope toe area, and may be a precursor to a larger slope movement
2. Deep-seated rotational failures
3. Earth flow slides which, following the initial failure, may rapidly retrogress a significant distance into the table land.

Failures of natural clay slopes are usually triggered by either erosion at the slope toe or elevated groundwater levels in the slopes, such as those that more often occur during wet seasons of the year such as the spring thaw. However, other triggers do exist, such as seismic events or rapid drawdown of the water level in a watercourse at the toe of the slope. Construction activities, such as cutting at the toe of a slope or filling the land above (or on) a slope and thus loading or steepening the slope, can also trigger slope movements. In extreme conditions, the vibrations caused by blasting might also trigger slope movements. Possibly the most common trigger for slope instability is toe erosion resulting from increased water flows and velocities within creeks and rivers that result from increased stormwater discharge to the water bodies from upstream land development.

3.3 Slope Instability Hazards

Instability of slopes can create several different hazards to people. Foremost, a slope could destroy facilities (houses, buildings, parking areas, etc.) constructed on the table land above a slope and injure or kill the people within them at that time. The debris from a slope failure (i.e., the “apron” in Figure 6) could also destroy or cover facilities below a slope, also injuring or killing people. Thirdly, the debris from a slope failure can dam a river/creek, flooding upstream lands. For large water courses, eventual overtopping of the dam could result in rapid erosion and failure of the debris, leading to the rapid down stream discharge of large quantities of water (i.e., flash flooding) which might cause more flooding, destroy buildings and bridges and injure or kill people.

3.4 Methods of Slope Stability Analysis

In the Ottawa area, any land which is sloped or inclined more steeply than about 11 degrees from horizontal (5 horizontal to 1 vertical) and has a grade difference of more than 2 metres across it has the potential for instability. This is a relatively conservative criteria but, in the absence of specific knowledge of the subsurface conditions at a site, is an effective “screening” criteria for requiring a slope stability assessment report.

Where the stability of a slope is in question and development within Hazard Lands needs to be avoided to protect people and valuable infrastructure from the hazards described above, geotechnical engineers are called upon to assess the stability of a slope.

As described previously, slope failures occur when the “forces” generated by the weight of the soil in and on a slope exceed the shear strength of the soil. For a sliding-type failure (such as generally occurs in sand or for the sloughing at the toe of a clay slope due to erosion), the mechanism is akin to sliding down a ramp. However for rotational failures the mass of soil encompassed by the failure rotates around a point in space above the slope; the tendency for the weight of the slope to rotate the soil mass must be resisted by the soil strength along the possible failure surface, as indicated previously on the lower drawing on Figure 5. However the concept is the same for both cases.

There are many different equations that may be used by engineers for evaluating the factor of safety against instability of a slope, however the concept of all of the equations is generally consistent, in that:

Factor of Safety equals Magnitude of Forces from Soil Strength Resisting Failure divided by Magnitude of Forces Tending to Cause Failure

Or Factor of Safety equals Magnitude of Torque from Soil Strength Resisting Failure divided by Magnitude of Torque Tending to Cause Rotational Failure

The various equations generally differ only in the manner by which are calculated the forces/torques which cause failure and the soil strength which resists the failure. However the sequence of the analysis is essentially the same for all methods.

Although there are simplified charts (i.e., graphs) by which simplistic assessments can be carried out, most slope stability analyses are carried out using computer software, as follows:

1. The slope geometry, soil layers, soil densities, soil strength properties, and groundwater flow conditions are determined and input into the program, as shown on Figure 8. Note: the meaning and importance of these parameters are described in a subsequent section.
2. Typically several hundred (or thousand) possible failure surfaces (called slip circles) are defined. For analysis of possible rotational failure (such as of a clay slope) the possible slip circles are defined by specifying multiple possible centres of rotation (called slip circle centres) and, for each centre, several different possible slip circle radii (i.e., sizes of slip circles), as shown on Figure 9.

3. The factor of safety against instability of each possible slip circle is calculated using one of several possible different equations. No equation is universally considered to be the most accurate or appropriate, however two common equations are the Bishop equation and the Morgenstern-Price equation. All of the equations divide the mass within each possible slip circle into “slices”. The force which drives the failure is the weight of the soil within the slip circle which is calculated as the sum of the weights of each slice, which are determined as the volume of each slice times the density of the soil (one of the five inputs from item 1). The shear strength (i.e., the strength which resists sliding of the soil) along the bottom of each slice is also calculated and totalled for the length of the failure surface. The factor of safety is, in concept, the ratio between the two. The results of a typical analysis are shown on Figure 10.
4. The slip circle with the lowest factor of safety is considered to be the critical failure surface (i.e., the most likely to fail), and the slope is described as having the factor of safety of that critical slip circle. Contours of factor of safety for the different possible slip circle centres can the drawn, with the centre of rotation for the critical slip circle being within the lowest contour.
5. Where structures or other infrastructure will be located on the table land adjacent to an unstable slope, the set-back from that slope (generally given as a distance from the slope crest) is assessed by determining the greatest distance behind the slope crest that any of the analyzed slip circles having a chosen factor of safety or less (typically 1.5), extend. In other words, all of the slip circles which extend further back into the table land area, beyond the set-back distance, have a calculated factor of safety greater than the chosen value. A schematic drawing showing the critical factor of safety and the calculated set-back for a typical slope is provided on Figure 11.

Theoretically, a slope with a factor of safety of less than 1.0 will fail and one with a factor of safety of 1.0 or greater will not fail. However, because the modelling is not exact and natural variations exist for all of the parameters affecting slope stability, a factor of safety of 1.5 is generally used to define a stable slope, or alternatively to define the “safe” set-back distance for permanent structures or valuable infrastructure from an unstable slope. That set-back distance, with possibly additional distance to account for future erosion of the slope and/or access along the slope crest to future slope failures, is called the “Limit of Slope Hazard Lands” or more commonly just the “Limit of Hazard Lands” with the understanding that it is a slope hazard being considered. It should be noted however that other Hazard Lands exist, such as related to flood hazards or karst terrain. It should also be noted that the limit has sometimes been previously called the “Geotechnical Limit of Development.”

In some specific cases, “passive” land uses, such as possibly parkland or unpaved pathways, a lower factor of safety can sometimes be considered acceptable, e.g., 1.3, which is marginally stable. This might be appropriate where valuable structures or other infrastructure would not be constructed close to the slope, such that the consequence of a slope failure would be less severe.

It is important to understand, however, that these factors of safety are more truly representations of different levels of risk associated with a landslide hazard, which has a certain probability of occurrence. That risk is a measure of:

- the probability of a landslide occurring over a given time period,
- the probable size of the potential landslide (e.g., length of slope or volume of soil affected), and
- the potential loss, both in terms of human safety and damage to property.

Although some jurisdictions, notably Australia, require that slope stability analyses determine the actual probability of failure, conventional practice for North America is to provide the required safety by means of a required minimum factor of safety against slope instability, noting that, although the two are related, that relation is not generally known for most projects.

3.5 Existing Slope Stability Assessments for Ottawa

In 1976, the Ministry of Natural Resources (MNR), in co-operation with the then Regional Municipality of Ottawa-Carleton (RMOC) carried out a broad assessment of the stability of essentially all of the significant river and creek valley slopes in the Ottawa area. That information was required for use in the long term planning of the development of the Ottawa area. In particular, the RMOC wanted to identify, in advance of development, those sites where slope stability issues were a concern and thus detailed slope stability assessments would be required prior to development of the adjacent lands.

The MNR study produced mapping of the Ottawa area which classified the slopes into sections with similar factors of safety against instability. It should be emphasized, however, that although the mapping is useful for the early identification of slope stability concerns at a site, the mapping is very broad and not sufficiently detailed to provide a thorough assessment of the slope stability constraints on a specific development. Further, no set-back distances are provided from unstable slopes. Therefore this mapping must not be used as the basis for assessing the Limit of Hazard Lands in relation to a specific site or development application.

3.6 Existing Guidelines for Slope Stability Assessments

There are no specific federal, provincial, or municipal documents (i.e., codes, regulations, or guidelines) which govern the methods by which slope stability assessments are carried out. The analysis methods are generally left to the discretion of the geotechnical engineer, which allows engineers to use judgement and experience as well as methods of analysis that have been shown to be appropriate for the geology of the area.

However, notwithstanding the above, the Provincial Policy Statement of the Planning Act of Ontario contains specific “Natural Hazard Policies” which state that slope stability issues must be considered (as part of general natural hazards) at the municipal planning level. To assist municipalities in identifying hazard lands and avoiding development within them, the MNR produced a set of documents, as follows:

- Understanding Natural Hazards. Great Lakes – St. Lawrence river system and large inland lakes, river and stream systems and hazardous sites;
- Adaptive Management of Stream Corridors in Ontario;
- Technical Guide - River and Stream Systems: Erosion Hazard Limit;
- Geotechnical Principles for Stable Slopes; and
- Hazardous Sites Technical Guide.

The MNR guidelines state that the “Limit of Slope Hazard Lands” should consider the potential for continued erosion at the slope toe for a period of 100 years (i.e., toe erosion allowance), plus an allowance for slope instability (i.e., stable slope allowance), plus an allowance for equipment to access the site of a slope failure via the table land (i.e., erosion access allowance, typically 6 metres). The values of each of these components depend on several factors, but for a typical slope in Champlain Sea clay, the set-back would be determined as indicated on Figure 12. The “stable slope allowance” can be simplistically and generally conservatively calculated based on a projection up from the slope toe level at 5 horizontal to 1 vertical for Champlain Sea clay or 3 horizontal to 1 vertical for other soils. However the more detailed engineering analysis procedures discussed in the following sections can also be used for determining the “stable slope allowance” provided the appropriate “toe erosion allowance” and “erosion access allowance” values are added. The difference is illustrated on Figure 13. Reference should be made to the MNR documents for specific values of the “toe erosion allowance” which depends on the soil types and river width; only typical values are shown on Figures 12 and 13.

4.0 Inputs to Slope Stability Analyses

The preceding sections described the methods by which the factor of safety against instability of a slope are calculated. The methods and equations are relatively uniformly used by geotechnical engineers and relatively little engineering interpretation is required for the actual calculation of the factor of safety. There is however significant discretion regarding the five fundamental inputs to those analyses, as listed previously, which are:

1. the geometry of the slope;
2. the geology of the slope (i.e., the composition of the various soil layers within the slope and their depth, thickness, and orientation) ;
3. the groundwater conditions (the groundwater levels and the hydraulic gradient/flow conditions);
4. the strength parameters for the soils; and
5. the unit weights (i.e., densities) of the soils within the slope.

For large or complex projects, each of the above five parameters can be determined or measured in detail. However, for routine and small projects, some of the last four parameters are often estimated, based on experience. In that context, the experience and competence of the engineer is important. These are also the situations where there may be the greatest need for the City to be able to evaluate whether the assumptions have been appropriate and consistent with good engineering practice. For this reason, reasonable ranges for these parameters are given in the follow sections.

The following sections discuss the significance of each of those parameters and reasonable values for them.

4.1 Slope Geometry

The factor of safety is, understandably, affected by the slope geometry. High steep slopes have lower factors of safety than low, flatter slopes.

The factor of safety against instability can only be evaluated at locations where the slope geometry has been measured. For example, if the slope geometry is surveyed at 10 locations along the slope at a particular site, then the factor of safety can only be evaluated for a maximum of those 10 locations. The cross sections must therefore be representative of the overall length of slope (which could be quite variable) and should include any critical locations. Although there are no rules regarding the maximum distance between slope cross section measurements, 50 to 75 metres would be a reasonable maximum spacing for the topography of a typical slope along a creek valley in Ottawa, though a much closer spacing could be required. For example, closer spacing between cross sections might be required where the slope crest/water course is meandering. Where the water course meanders, slope cross sections should generally be surveyed at the outsides of all bends (i.e., at the location of most active erosion and instability), as shown on Figure 14. Intermediate cross sections, (i.e., along the promontories of land between the bends) are also beneficial for delineating a higher amount of useable land, as shown on Figure 15. Slope cross sections should also generally be measured at points closest to important/valuable infrastructure and areas which people would occupy (i.e., at locations key to the protection of the public safety and property). For example, a slope cross section should be surveyed at the closest point of approach between a building, road, or parking lot to the crest of the slope.

The factor of safety can, in some situations, be rather sensitive to small differences in the slope geometry used in the analyses. The slope geometry therefore needs to be determined accurately.

Topographic contours generated by surveyors can, depending upon the intensity of the survey points used to generate the contours, significantly misrepresent the actual slope geometry. It is generally good practice to measure the slope inclination with a hand clinometer (a hand-held optical device); the overall slope height can also be measured by stepping up the slope in increments of known height (usually foot to eye height), using the clinometer to define the level of the next step, however topographic survey data is more accurate in that regard.

Slope geometries generated solely from topographic survey data should therefore be treated with caution. At a minimum, the slope geometry from topographic survey data should generally be field-checked at a few (critical) locations.

Slope stability analyses should also not be undertaken without a thorough field reconnaissance of the slope (even if clinometer measurements are not carried out), to identify areas of active slope movement or erosion, as well as previous slope instability areas.

The slope geometry used in the analyses also needs to be determined at sufficient frequency along the length of the site that critical locations are identified and variability of the slope geometry is identified.

The slope geometry should also reflect possible changes to the slope by construction. Proposed steepening or flattening of the slope should be included in the analyses. Most importantly, proposed filling along the table land above the slope (such as the general grade-raise fill placed in most residential subdivisions) needs to be considered in the analyses.

In the same context, the loads from buildings or other structures that will be constructed on the table land also need to be considered. Conversely, the removal of soil from the excavation for the basement of a structure can have a beneficial “unloading” effect that could, under some circumstances, be considered.

In summary, a review of a slope stability report should identify that:

- the geotechnical engineer (or their representative) saw the slope, and that areas of active slope movement or erosion as well as areas of previous slope instability have been noted.
- the slope geometry has been measured sufficiently accurately
- the distances between surveyed slope cross sections are not excessive
- the slope geometry has been measured at critical locations, i.e., at the outsides of bends in the water course and adjacent to proposed buildings, etc.
- possible modifications to the slope, including the loads from buildings constructed on/beside the slope, have been considered.

4.2 Slope Geology

The geotechnical engineer must develop a geologic model for the conditions within the slope at each cross section. In general, that geology can be inferred from one of three ways:

- Assumed based on general knowledge/experience of the geology within the area of the site Note: This method is only appropriate for very small and routine projects, such as slope stability assessments for pools and decks within existing subdivisions where a broad knowledge of the subsurface conditions already exists.
- Inferred from observation of the soils exposed on the surface of the slope. For example if clay is observed along the entire face of the slope, from crest to toe, then the slope could be assumed to be composed entirely of clay. However the soils may be obscured by vegetation. If multiple soil layers are present, they may not extend into the slope horizontally as would be assumed. However, for routine projects and where the geology of the area is already familiar, this is an acceptable method by which the slope geology can be inferred.
- Determined by drilling boreholes through the slope. At a minimum, boreholes should be drilled from the table land above the slope and down to the level of the slope toe (i.e., borehole depth = slope height). Where the geology is known or expected to be complex, the drilling of a borehole at the slope toe and/or on the slope face can be considered, though boreholes on the slope face can be difficult to access and expensive to drill. Direct investigation of the slope by means of boreholes may be necessary for large or complex projects, locations of particular instability, or where there is little existing knowledge of the geology.

The latter two options are more appropriate for large or complex projects or for projects within undeveloped areas where there may be little existing knowledge of the subsurface conditions. Sometimes a combination of any of the above three approaches may be used.

All three methods generally involve some extrapolation. Even where boreholes are drilled above and below a slope, some assumptions need to be made of conditions between boreholes. In that regard, it must be recognized that the depth of weathering of a clay deposit will often follow the slope face, since it develops as a result of weathering, rather than from the soil’s formation/deposition.

In summary, a review of a slope stability report should identify that:

- the geology within the slope has been inferred from one of the three methods described above (which should be clearly stated in the report), and
- there should be a rational basis for the method selected, which should be appropriate for the magnitude and complexity of the project

4.3 Groundwater Levels

The stability analyses need to consider the highest groundwater levels which can reasonably be expected to exist within the slopes. This is the conservative condition since, as described subsequently, the strength of some soils decreases as the water pressure in the pores between the soil grains increases; a higher groundwater level results in high water pressures in the soil at depth. In concept, that higher water pressure pushes the soil grain apart (i.e., reduces the pressure in the ground from the weight of the overlying soil that is holding the soil together) reducing the resistance to the soil being able to shear along a slip surface.

It must be noted that the calculated factor of safety from a slope stability analysis is extremely sensitive to the groundwater conditions (i.e., levels) used in the analyses. However the ground water conditions are often one of the greatest unknowns, and most open to interpretation by the engineer. There are three typical groundwater conditions that are used in slope stability analyses of slopes with simple geologies:

1. Full hydrostatic saturation. This condition assumes that the groundwater level is at ground surface (i.e., all of the soils are saturated) and that the groundwater flows horizontally towards the slope. This condition is somewhat unrealistic because it means that water would be flowing out of and along the slope face, from crest to toe, which is not generally observed. This condition is, however, conservative (i.e., yields a lower factor of safety).
2. Full saturation, with groundwater flow parallel to the slope face. This condition also assumes that the groundwater level is at ground surface (i.e., all of the soils are saturated) but that the groundwater flows towards (and discharges at) the slope toe. Where the slope is adjacent to a river/creek, this is a reasonable assumption since the groundwater would typically flow out into the river/creek.
3. Groundwater level at some depth in the slope. This condition assumes that the upper part of the soil profile is not and never will be saturated, and the groundwater level is usually assumed to slope down towards the slope toe.

Ideally, the actual groundwater flow conditions would be measured using piezometers installed within boreholes. However, even for that case, the measured groundwater levels are unlikely to be the highest that could occur.

For slopes in sand or glacial till, which due to their relatively granular and permeable nature may never be fully saturated, Case 3 from the above list may be realistic, but some judgement will need to be made as to how high the groundwater levels could become. The analyses should use the highest groundwater level that could reasonably be expected to occur (i.e., the “worst case” condition). Different groundwater levels may also be assigned to different soil layers within the slope.

The groundwater levels used in the analyses should also consider the possible influences of flooding of the slope, possible saturation of the slope due to discharge from adjacent septic beds, the effects of significant irrigation, and the effects of adjacent stormwater management ponds.

For slopes in silty clay, numerous previous published studies of the natural slopes in the Champlain Sea clays of eastern Ontario and west-central Quebec have shown that these slopes can reasonably be expected to become fully saturated (i.e., groundwater level at ground surface) during periods of high and extended rainfall/groundwater infiltration, with the spring commonly being the critical time period. This condition could conservatively be modelled by assuming hydrostatic conditions (Case 1). However the previous engineering research has also shown that the critical groundwater condition can reasonably be assumed to consist of groundwater flow being parallel to the slope face (Case 2).

The slope stability analysis methods and software allow the Case 1 and Case 2 groundwater flow regimes to be considered by specifying the value of the pore pressure parameter, r_u, used in the analyses such that the pore water pressure at any point in the slope is determined as a function of the weight of overlying soil by:

$u(z) = r_u \cdot \gamma \cdot z$

Where: $u(z)$ = pore water pressure at depth 'z', kilopascals
r_u = pore pressure parameter
γ = unit weight of overlying soil, kilonewtons per cubic metre
z = depth below ground surface, metres

These terms are shown schematically on Figure 16.

The value of r_u used in the analyses can account for different groundwater level and flow conditions. For Case 1, with horizontal flow, $r_u = \gamma_w/\gamma$, (i.e., the slope angle is neglected) which usually equates to an r_u value of about 0.6. For Case 2, with groundwater flow parallel to the slope face, the value of r_u can be can be calculated as:

$$r_u = \gamma_w/\gamma \cdot \cos^2(\alpha)$$

Where: γ_w = unit weight of water, 9.81 kilonewtons per cubic metre
α = average angle of inclination of the slope face, degrees

The terms in this equation are shown schematically on Figure 17.

Although the "r_u" method of calculating the pore water pressures in a slope treats the groundwater flow in a rather idealized manner and therefore the calculated pressures are not necessarily as accurate as those determined from more sophisticated analyses, the method provides a practical and convenient method of assessing the stability of multiple slopes, is consistent with the level of accuracy of the other slope stability parameters, and is one of the most common methods in engineering practice for approximating for the pore pressure distribution in natural slopes for routine projects.

This method does not however account for the groundwater flow conditions that may develop in a more complex geologic setting, such as where more permeable strata underlie and drain the clay, increasing the downward drainage component of the flow. Such conditions can only be assessed by more complex analyses of the groundwater flow. In such a situation, it may be appropriate to measure groundwater levels at different elevations in a slope and determine an r_u value from that data. For more complex geologies, different r_u values may be assigned to each soil layer.

In summary, a review of a slope stability report should identify that:

- the groundwater levels have been established or assumed based on a rational method, and
- for slopes in clay, it should generally assumed that the slope will be fully saturated, though the effect of groundwater flow/drainage may be considered.

4.4 Shear Strength

Unlike many other materials, soils generally fail by shearing (i.e., a surface develops upon which the soil particles slide) rather than by tension (pulling part) or compression (squeezing). The shear strength of a soil is conventionally described in terms of "cohesive" and "frictional" components. The magnitude of the "frictional" component depends on the stress acting vertically above the potential failure surface while the cohesive component does not. The shear strength of a soil to resist sliding failure is described by:

$\tau = c' + \sigma' \cdot \tan \phi'$

Where: τ = shear strength of the soil
c' = effective cohesion of the soil
σ' = effective normal stress (i.e., compressive stress acting on the shear surface, squeezing it)
ϕ' = effective internal friction angle of the soil

These parameters are shown schematically on Figure 18.

In general, only clayey soils have a cohesive strength component; granular soils (gravel, sand, and silt) do not.

The frictional component is affected by the stress (load) acting on the shear surface, which is reduced, as described previously, by increased (pore) water pressures. Therefore higher groundwater levels reduce the overall shear strength of a soil. For that reason, the analyses should use the highest groundwater level that could reasonably be expected to occur.

By analogy, this model can be envisioned in relation to a heavy block of wood, glued to a platform, that is pulled horizontally. The block will not move until the glue breaks; that is the “cohesive” component. But there will also be friction between the block and the platform that must be overcome. The value of the friction depends on how heavy the block is. The heavier the block, the more the force that acts on the sliding surface (i.e., the “normal stress”), and the more friction that must be overcome to make the block slide.

It is quite complex and expensive to directly measure the shear strength of soils. Undisturbed samples need to be retrieved and subjected to complex laboratory testing. For sands, it is almost impossible to retrieve a truly undisturbed sample. Therefore the shear strength parameters are generally either estimated from either typical values for the soil types or inferred from more simple tests carried out in the boreholes (i.e., “in situ” tests such as the Standard Penetration Test in sand or vane testing in clay). There are many different possible correlations that may be used and there is therefore an extremely large range in the values of these parameters used by different engineers. However, some typical ranges would be as follows:

Table 2

Soil Type	Bulk Unit Weight (kN/m^3)	Effective Cohesion (kPa)	Effective Internal Friction Angle (kPa)
Sand	17 – 21	0	28 – 40
Weathered Silty Clay Crust	15.5 – 19	0 – 12	25 – 38
Grey Silty Clay	15.5 – 19	0 - 12	25 – 38
Glacial Till	19 - 22	0 - 3	30 – 35

These represent very large ranges and the actual values are fully up to the discretion of the engineer; however the values used in the analyses should be based on sound engineering judgement and principles.

Values outside of these typical ranges may be appropriate in certain circumstances. For example, sand may be assumed to have some cohesion (theoretically it has none) in order to model conditions above the groundwater level. Fill materials are very difficult to assign values to and can be assumed to have no strength or a reduced strength from its original value (prior to excavation from its original location) in order to model its disturbed condition.

Sometimes, when there is an existing failure on the slope, the failed area may be assumed to have had a factor of safety of 1.0 and the soil parameters “back-calculated”. Some judgement is required, though, since the results of these “back analyses” are not always reasonable.

Where the slope is composed of clay overlying glacial till, the glacial till may often be assumed to be impenetrable; many documented slope failures have shown that the much higher strength of the glacial till versus the silty clay results in the failure surface not penetrating the glacial till but rather extending along the interface between the two, as shown on Figure 19.

It should also be noted that these parameters apply to what geotechnical engineers term "drained" failure. That is, a failure condition that occurs from slow loading of the slope and is generally the realistic condition for a natural slope. "Undrained" conditions apply for sudden or short term loading (such as an earthquake, as described subsequently) and different soil strength parameters need to be used for the clayey soils; the sand and glacial till parameters generally remain unchanged.

In summary, a review of a slope stability report should identify that:

- the soil strength parameters have been established or assumed based on a rational method, which should be described, and
- the soil strength parameter values lie within or close to the limits in the above table

4.5 Unit Weights (Density)

Engineering analyses are generally carried out using soil unit weights, rather than densities. The two are related, but have different units. A soil's unit weight is equal to its density multiplied by the acceleration due to gravity (9.81 m/s^2).

The unit weights of the soils determine the overall weight of the slope, which is the force that causes it to fail. The unit weights of the soils used in the analyses therefore need to be relatively accurate.

Typical ranges for the unit weights of some common soils were provided in the preceding Table 2, although values outside of these ranges may be appropriate. The unit weights of most soils are very difficult to measure directly. For clay soils, the unit weight can, in contrast, be relatively easily measured from undisturbed samples or can alternatively be calculated from the measured water content (an inexpensive test).

In summary, a review of a slope stability report should identify that:

- the soil unit weights have been established or assumed based on a rational method, which should be described, and
- soil unit weight values lie within or close to the limits in the above table.

5.0 Other Factors

The preceding discussion has listed and described the fundamental parameters involved in the analysis of the stability of a slope. However there are other issues and conditions that need to be considered, as described herein.

5.1 Erosion, Triggers, Progressive Failure, and the Limit of Hazard Lands

The preceding discussions have focussed in the determination of the factor of safety against instability of a slope and, for potentially unstable slopes, on the determination of the "stable slope allowance." However, as described briefly in Section 3.6, the MNR guidelines suggest that the Limit of Hazard Lands should be determined based on two additional factors, as follows:

Limit of Hazard Lands = Stable Slope Allowance + Toe Erosion Allowance + Erosion Access Allowance

The application of these additional set-back "allowances" is shown schematically on Figure 13.

The magnitude of the toe erosion allowance depends upon the soil types, the state of erosion along the creek/river bank, and upon the width of the channel; reference should be made to the MNR's *Technical Guide - River and Stream Systems: Erosion Hazard Limit* for the magnitude of the allowance. However, as an example, the Toe Erosion Allowance for clay soils with indications of active erosion is suggested at between 5 and 8 metres.

Slope stability assessment reports must therefore include an assessment of the current state of bank erosion. Where active erosion is occurring, it is often planned to install erosion protection along the bank (and/or slope toe) in order to eliminate the need for including a Toe Erosion Allowance and thereby gain more useable land.

An important concept that should be understood is that the installation of erosion protection does not generally modify the slope geometry significantly and does not therefore actually change the mathematical computation of the factor of safety. The installation of erosion protection does not therefore achieve stabilization of the slope.

In addition, it should be understood that although the MNR procedures permit either the installation of erosion protection or alternatively the consideration of a Toe Erosion Allowance, there may be specific circumstances where the installation of erosion protection should be the required choice. Continued erosion at the slope toe is probably the most common trigger of failure of slopes along water courses. In the absence of slope toe erosion, the failure of a slope which is "mathematically" unstable will often only be triggered by high groundwater levels, such as those corresponding to full saturation of the slope. Other triggers exist, such as seismic events, but these are much less frequent.

Therefore the installation of erosion protection, by removing one of the two most likely triggers of a slope failure, is an effective method of reducing the probability of a slope movement occurring during a given year, and thus reducing the general frequency of slope failures. As such the installation of erosion protection not only permits the development of more usable table land, since a Toe Erosion Allowance may not need to be included, but in fact reduces the probability of a slope failure actually occurring.

Where areas of active bank erosion are not protected, slope instability can often be expected in these areas in the near future. Although these slope failures would be (statistically) unlikely to impact on table land located behind the Limit of Hazard Lands, the failures might alarm the public and the failure area itself might be unstable and unsafe and therefore require repair. Further, if the slope failure leaves a steep scarp, the set-back line from the new (post failure) slope could be located further back than the original Limit i.e., there could be a potential for a "progressive" failure to develop which would slowly cut-back into the table land, beyond the Limit of Hazard Lands, as illustrated on Figure 20. In other words, conventional practice is to delineate a Limit of Hazard Lands corresponding to the "first failure". That is, the Limit of Hazard Lands is based on the first failure of the slope that is expected to occur, corresponding to a factor of safety of 1.5, not considering that the resulting new slope, after the failure, may also be unstable and subsequent failures of the slope may occur in a relatively short period of time. In general, there is no practical means of predicting the possible Limit of Hazard Lands corresponding to subsequent potential failures of a "first failure" area.

Therefore, where areas of current active erosion are left un-protected and instead a Toe Erosion Allowance is included in the identification of the Limit of Hazard Lands, the proponent is deciding to accept that a slope failure is more likely to occur. This may be acceptable in circumstances where there is sufficient table land on the site that the additional Toe Erosion Allowance is not problematic but is must be recognized that, for the reasons stated above, the owner (and possibly the City) may be compelled to repair future slope failures.

Erosion protection can therefore be viewed as a method of reducing (though not eliminating) the probability of needing to undertake slope repairs in the near future, which presents a financial liability to the owner, and possibly to the City. Those potential future costs should be balanced against the lesser amount of table land available for development (as a result of the application of a Toe Erosion Allowance) and the costs of constructing the erosion protection. It should also be noted that the construction of erosion protection can also be undesirable with regards to disturbance of the natural environment and a detraction in the natural aesthetic appeal of the valley/ravine. There can also be restrictions on the nature of the erosion protection as a result of regulations on the protection of aquatic habitat.

The above discussion is intended to explain both the MNR procedures for including a Toe Erosion Allowance in the determination of the Limit of Hazard Lands as well as the relative benefits and disadvantages of the installation of erosion protection for slopes along waterways in regards to limiting the potential future financial liabilities. As a further point, it should be noted that those liabilities can only be eliminated, in a practical sense, by stabilizing a slope, as discussed further in Section 5.3 of this document.

The third item included in the MNR procedures for determining the Limit of Hazard Lands is the Erosion Access Allowance which is intended to provide of corridor of sufficient width across the table land that equipment could access the site of a future slope failure to undertake a repair. The width of the Erosion Access Allowance is typically 6 metres. The MNR documents do not provide guidance on those situations where the Erosion Access Allowance need, or need not, be applied. However, as a general guideline, the Erosion Access Allowance should be included wherever the development plans would preclude equipment access to the slope. For example, it should be included where the rear lot lines of residential lots will be constructed right up to the Limit of Hazard Lands. But it probably need not be included in the Limit of Hazard Lands associated with the construction of a parking lot on the table land area, since equipment could cross the parking lot. Judgement needs to be used in its application.

In summary, a review of a slope stability report should identify that:

- areas of active erosion have been identified, for slopes along water courses
- areas requiring erosion protection are specified, along with some detail on the nature of the erosion protection (type, limits, and height of application).
- a toe erosion allowance and/or erosion access allowance have been included in the determination of the Limit of Hazard Lands, if appropriate.

5.2 Staking the Limit of Hazard Lands

For many developments, the developed part of the site (rear lot line) will extend right up to the Limit of Hazard Lands. For residential subdivisions in particular, there is a desire to consume all of the available table land into the development. It is therefore important that the Limit of Hazard be established accurately.

Scaling back the set-back distance from an inferred crest-of-slope, established from a topographic survey, to then draw the Limit of Hazard Lands onto a plan should not generally be acceptable in situations where the development will extend right to the Limit of Hazard Lands. The crest-of-slope can be quite inaccurately inferred from topographic surveys. The demarcation of the Limit of Hazard Lands should also not be allowed to the discretion of others, based solely on calculated set-back distances; on sites with complex topography and meandering valleys, there is too much opportunity for misinterpretation of the direction in which the set-back distance should be measured. It is therefore generally good practice for the Limit of Hazard Lands to be established by one of two methods:

1. the Limit of Hazard Lands could be staked in the field by the geotechnical engineer and tied-in by a surveyor, or
2. the surveyor could tie-in top-of-slope (or other reference) pickets installed by the geotechnical engineer who would then, based on the results of the survey, graphically produce a plan showing the Limit of Hazard Lands.

That level of detail may not be required where the development is located a significant distance from the Limit of Hazard Lands or for smaller sites and simpler slope geometries where there is less opportunity for misinterpretation.

The Limit of Hazard Lands that is either staked in the field or drawn by the geotechnical engineer may also need to include a “toe erosion allowance” and an “erosion access allowance” as suggested previously in Section 5.1 of this report. Current best practice does not allow the property line (lot line) to extend beyond the Limit of Hazard Lands.

In summary, a review of a slope stability report should identify that:

- where a development will extend right to the Limit of Hazard Lands, that it has been located accurately and that the geotechnical engineer has been involved in its demarcation.
- that, where necessary, the set-back includes the “toe erosion allowance” and “erosion access allowance” as suggested by MNR.

5.3 Stabilization Measures

As discussed above, the installation of erosion protection is the most commonly suggested modification to a slope. However the installation of erosion protection does not generally alter the factor of safety or change the calculated set-back distance, except in that an additional “toe erosion allowance” may no longer need to be included.

True “stabilization” of the slope to achieve a higher factor of safety and/or to reduce the set-back requires a change in one of the five inputs to the analysis: geometry, groundwater conditions, geology, soil strength, and soil unit weights. In a practical sense and for routine projects, it is usually only feasible to alter the first two. That is, a slope can be flattened (by cutting, filling, or adding berms at the toe) or possibly drained. Other options exist, such as reinforcing the slope, which could involve excavating and rebuilding the slope in compacted lifts reinforced with plastic geogrid reinforcing. However a discussion of such more complex stabilization options is beyond the intended scope of this manual.

5.4 Rapid Draw Down

Rapid draw down is the situation that occurs when surface water that was present against the toe of a slope (i.e., contained within the river valley or a storm water detention pond) is suddenly drained away, removing its supporting effect but leaving high water pressures in the ground within the slope toe. Those water pressures had previously been in balance with the water pressure outside the slope but, once that water is removed, the relatively higher water pressures in the ground may persist, maintaining a lower shear strength as described previously, and a more critical condition develops.

This situation can develop within any slope where the surface water could be sustained at high levels for an extended period of time adjacent to the slope, the slope is formed of moderate to lower permeability soils (such as clays, silts, or possibly silty sand) in which water pressures in the slope would take time to reduce, and the water level within the area at the slope toe can be lowered rapidly. Depending on the permeability of the soil, that would required draining of the water within a few hours (silty soils) to a few days (clayey soils).

Evaluation of the factor of safety under rapid draw down conditions requires engineering judgement on the rate of water pressure dissipation and more detailed engineering analyses. For the purposes of this manual, no simple guidelines can be provided on how those analyses should be undertaken.

In summary, a review of a slope stability report should identify that:

- if there is the potential for sustained water levels adjacent to the slope to be rapidly draw down, that the geotechnical engineer has considered that possibility and made a rational assessment of its impact on the stability of the slope.

5.5 Earth Flow Slides

As described previously, the analysis procedures described in the preceding sections determine only the factor of safety for the first failure of a slope. These procedures do not assess the potential for the "special case" of earth flow sliding to develop, whereby the debris from a failure flows away from the site, leaving an unsupported scarp which fails again, possibly repeatedly, such that the failure rapidly retrogresses into the table land and an earth flow slide develops. A schematic of this phenomenon is included on Figure 7. This is essentially an extreme case of the "progressive" failure discussed previously (see *Erosion, Triggers, Progressive Failure, and the Limit of Hazard Lands and Figure 20*).

There are several methods by which the potential for earth flow sliding may be evaluated. None of these methods is considered to be very accurate and, though not necessarily complex, the details of those methods are beyond the intended scope of this document. One of the best methods for assessing whether there is a realistic potential for earth flow sliding to develop is to identify whether there is any evidence of this form of failure in the area. The potential for earth flow sliding is also greater for higher slopes and softer clays. Based on the general properties of the silty clays within the Ottawa area and the documented history of earth flow sliding, the potential for earth flow sliding to develop should at least be considered in clay slopes higher than about 8 metres.

In summary, a review of a slope stability report should identify that:

- the potential for earth flow sliding has been considered wherever the slopes are composed of clay and are higher than about 8 metres or there is a history of significant instability in the area

5.6 Seismic Conditions and Seismic Liquefaction

The ground shaking that develops during an earthquake can generate both horizontal and vertical forces within a slope and reductions in soil strength which may lead to instability. It should be noted that the ground vibrations travelling up to the Earth's surface from depth travel in waves such that, except for low slopes, the ground motion would not all be in the same direction simultaneously. In other words, although part of the slope would be pushed outwards (encouraging failure), other parts would be pushed inwards (resisting failure), as shown on Figure 21. Vertical ground motions also occur, but are generally of lesser magnitude and impact and are often not taken into consideration unless analyses indicate that the slopes are marginally stable when the horizontal earthquake forces alone are taken into consideration.

It must be recognized that a realistic assessment of the magnitude of the seismic forces and their impacts on the slope is complex and thus not practical for most projects. Simplified methods exist and are commonly used in North America, but vary from jurisdiction to jurisdiction. No specific methods or minimum criteria are mandated for use on projects in the Ottawa area at this time.

One common method is to carry out a simplified "seismic" slope stability analysis (often referred to as a pseudo-static analysis) for which a horizontal force is included, the value of which is related to a seismic coefficient, typically identified as k_h, as shown on Figure 22. The magnitude of k_h to be used in the analyses is debatable, however a value of half the peak (horizontal ground acceleration (PGA) specified in the National Building Code of Canada (NBCC), as referenced by the current edition of the Ontario Building Code (OBC), is typically used. For example, for a PGA of 0.42 (as specified in the 2005 NBCC and referenced by the 2006 OBC), a kh of 0.21 would be used.

However, the design earthquake accelerations described in the NBCC are typically considered to be the "firm ground" values, representing shaking forces at the surface of the bedrock or dense soil. Loose or soft soils overlying the “firm ground” may, and often do, result in a significant increase (possibly 10 to 30 percent) in the earthquake accelerations within these soils, which should be considered in selecting the design value of k_h.

The analysis for seismic conditions would be carried out separately from the “static” analyses. A minimum factor of safety of 1.1 is generally required (for example, by the Los Angeles County government), although other jurisdictions require 1.0 (for example, suggested by the Southern California Earthquake Centre). The use of these seemingly low required factors of safety (relative to the 1.5 factor of safety required for “static” conditions), is based on a number of factors, namely the limited accuracy of the method, the relative infrequency and short duration of earthquakes, and that seismic slopes failures often result only in movement of the slope, rather than a more dramatic “landslide” type failure.

If the factor of safety is determined as being less than 1.0 or 1.1, the permanent displacement (i.e., movement) of the slope is calculated; a maximum allowable movement of either 50 millimetres or 150 millimetres is often specified, depending on the jurisdiction and the nature of the project. However the methods for evaluating these displacements, though commonly used in many seismically active areas of western North America, should only be used with caution for sites in the Ottawa area since the methods were generally developed for soils with very different properties and for earthquakes with different characteristics. The Champlain Sea clay is a very “sensitive” soil meaning that its strength reduces drastically once it’s sheared, so the displacements from a slope in the Champlain Sea clay may be much larger than would be calculated by those methods, unless that possible strength loss is considered in the calculation.

Based on the above, a minimum factor of safety of 1.1 is suggested for seismic slope stability analysis. Where the factor of safety is less than 1.1, a Limit of Hazard Lands may be determined corresponding to a factor of safety of 1.1, in the same manner as used for the ‘static’ loading cases. However more sophisticated analyses may also be justified, which might yield a more accurate assessment of the stability of the slope. Caution is urged in these circumstances. It should also be noted that a Toe Erosion Allowance need not be included in the determination of the Limit of Hazard Lands under seismic conditions, since erosion is not the trigger of the slope movement. As well, as a general guideline, an Erosion Access Allowance need also not be included since this Limit of Hazard Lands for seismic design corresponds to a post-disaster condition. The general philosophy for seismic design (due to their low frequency of occurrence) is to avoid immediate collapse and loss of life; structures are not expected to remain serviceable.

The resulting Limit of Hazard Lands determined for seismic loading conditions is then compared to the limit corresponding to ‘static’ loading conditions and the more conservative value is used.

It is important to note that the soil strength parameters that apply to the silty clay for the assessment of the factor of safety under “seismic” loading conditions are not those that apply to the more conventional “static” analyses. “Undrained “ soil strength parameters need to be used, to reflect the rapid nature of the loading, whereby the clay soil has a much higher cohesion than described previously for “static” analyses (typically between 25 and 150 kilopascals) and a friction angle of 0 degrees. However, as indicated above, some clays, in particular sensitive clays, may have a reduction in strength due to “softening” when subject to earthquake shaking.

Another very important consideration for these assessments is the potential for seismic “liquefaction” of loose sandy soils. Seismic liquefaction is a phenomenon whereby the ground vibrations from an earthquake generate high (pore) water pressures (i.e., the water pressure in the pores between the soil particles is increased above its natural level). These excess pore water pressures reduce the stresses between the soil grains and thus reduce the frictional resistance to shearing of the soil. This phenomenon leads to a temporary but dramatic reduction in the shear strength of the soil (potentially changing it to a liquid-like state) and thus, in the period immediately following an earthquake, could lead to failure of an otherwise stable slope. Even very flat slopes may experience large, permanent movements if the soils underlying these slopes liquefy.

The details for assessing the potential for seismic liquefaction are beyond the scope of this document. However, the potential should be considered wherever the slope contains “looser” sand, sand and gravel, or silt located below the highest projected groundwater level in the slope. In this context, “looser” means a Standard Penetration Test blow count less than about 18, as measured in a borehole. Many routine projects do not include borehole drilling and so that information would often be unavailable. Caution is urged on such occasions.

In summary, a review of a slope stability report should identify that:

- the factor of safety against slope instability under seismic conditions is at least 1.1;
- a seismic coefficient of half of the PGA specified in the current version of the OBC (and applicable referenced version of the NBCC), or greater, was used for the analyses, and that the potential for increase in the seismic coefficient from the “firm ground” value has been taken into consideration;
- undrained soil strength parameters were used for clay in the analyses, with consideration of the potential for decreased strength due to “softening” during the earthquake;
- where sand or silt deposits are present, the potential for seismic liquefaction is addressed.

5.7 Stable Slopes, Unconfined Systems, and Large Inland Lakes

The general guidelines and procedures described in this document pertain to the typical slope conditions encountered along the river and creek valleys in the Ottawa area. These slopes are typically located along narrow valleys and ravines, with marginally unstable to unstable slopes, and where the bank erosion results from flow in the water way. However other cases which can be encountered include:

1. Stable slopes (rather than unstable slopes) but with active bank erosion that could result in future slope movements
2. Water courses that are not confined within a distinct valley
3. Slopes adjacent to large bodies of water where the bank/toe erosion results from wave action, not water flow.

Different procedures can be required when considering sites such as these. For the first case, it may be appropriate to assign a Toe Erosion Allowance only, as the distance between the bank and the Limit of Hazard Lands, but not to include a Stable Slope Allowance.

Where the water course is not confined within a distinct valley (the second case from the above list), the MNR guidelines would classify the water course as an *unconfined* system and different procedures need to be applied which consider the potential for a meandering water course’s channel to evolve over time. Unconfined systems are not common in eastern Ontario, where most water courses flow along valleys incised into the table land. However, where unconfined systems are encountered, reference should be made the MNR’s *Technical Guide - River and Stream Systems: Erosion Hazard Limit* for the appropriate procedures for determining the Limit of Hazard Lands.

Slopes adjacent to large inland lakes where wave action is the primary agent of bank/toe erosion (the third case from the above list) also require assessment using procedures different from those presented in this manual. In those cases, reference should be made to the MNR’s *Technical Guide for Large Inland Lakes.*

5.8 Retaining Walls

Retaining walls are structural features that support a grade difference across a site at a steeper inclination than could be supported solely by a slope. Retaining walls are often constructed of reinforced concrete, timber cribbing, or precast concrete masonry units (typically tied-back with geogrids), although many other types also exist. The design of these walls must consider several possible failure modes, including toppling of the wall (overturning), forward sliding of the wall, structural failure of the wall, bearing capacity failure of the foundation soil, and failure of the tie-backs, if present. These failure modes are typically addressed at the detailed design stage and can generally be addressed through suitable engineering design. However another important failure mode is *global instability*, as indicated on Figure 23, which involves failure of the soil above and beneath the wall, in essentially the same manner as a slope failure. The resistance against this type of potential failure is often in large part independent of the retaining wall system being used and the details of the design. Rather, the factor of safety against global instability of a retaining wall is largely controlled by the site geology and the wall height/geometry. The factor of safety against global instability can therefore be assessed before the detailed design of the wall itself, based only on the details of the Grading Plan / Site Plan. From the perspective of evaluating the feasibility of a planned development and the potential hazards / restrictions that the geologic conditions on a site may present and, in the context of an application to the City for development approvals, the potential for *global instability* of proposed retaining walls for a site should be addressed in the same manner as would potentially unstable slopes. The only significant difference is that retaining walls, being expensive constructed features, *must* have a factor of safety of at least 1.5 against global instability; unlike natural slopes, it is *not* acceptable to define un-developable Hazard Lands above an unstable retaining wall. Therefore, development applications for sites that involve a retaining wall greater than 1 metre high should include an engineering report, prepared by a qualified geotechnical engineer licensed in the province of Ontario, which indicates that the proposed retaining walls will have a factor of safety of at least 1.5 against *global instability,* and provide sufficient details to support that conclusion.

Minimum Requirements for Slope Stability Assessment Reports

A report addressing the stability of slopes, prepared by a qualified geotechnical engineer licensed in the Province of Ontario, should be provided wherever a site has slopes (existing or proposed) steeper than 5 horizontal to 1 vertical (i.e., 11 degree inclination from horizontal) and/or more than 2 metres in height. A report is also required for sites having retaining walls greater than one metre high, that addresses the global stability of the proposed retaining walls. The requirements for the assessment of the stability of slopes listed in this document pertain also to retaining walls.

The study should assess whether or not the slopes are considered to be stable and, if not, provide a setback distance from the slope behind which structures, amenity areas, and other facilities can be constructed. Unstable slopes and the land located between the unstable slope and the setback line are considered to be "Hazard Lands." Hazard Lands are, in general, those at risk of being adversely impacted by any natural geologic process. In the case of unstable slopes, Hazard Lands are those that have an inadequate factor of safety (less than 1.5 for 'static' loading or less than 1.1 for seismic loading) against being affected by a slope failure. In some circumstances, Hazard Lands may, in accordance with Ontario's Planning Act, be transferred to the municipality or other agency. The development of Hazard Lands is restricted. They should not be developed with permanent structures, parking or roadway areas, amenity areas, septic beds, or any other valuable infrastructure. Buildable lots in residential subdivisions should not include Hazard Lands.

The location of the Limit of Hazard Lands (also sometimes called the Geotechnical Limit of Development) must consider both the distance back from the slope needed to provide an adequate factor of safety (greater than 1.5 for 'static' loading and greater than 1.1 for 'seismic' loading) and, if appropriate, allowances for future erosion and, in some cases, an additional allowance for access to future slope failures, in accordance with the Ministry of Natural Resources' Technical Guide - River and Stream Systems: Erosion Hazard Limit.

The assessment of the stability of a slope and the determination of the Limit of Hazard Lands should consider the most conservative condition that the slope could reasonably be expected to experience (i.e., the "worst case" condition).

The slope stability assessment report should:

- Provide a scaled plan showing the location of the slope, the legal limits of the property (e.g., proposed rear lot lines of subdivisions), and the significant features of the planned development (e.g., structures, roadways, and paved areas).

- Indicate on the plan the locations at which cross-sections of the slope geometry have been established and at which the stability of the slope has been assessed.
- Describe how the geometry of the existing slopes was determined (e.g., field measurement by the geotechnical engineer, interpreted from topographic survey data, etc.).
- Provide the cross sections showing the measured slope geometry and the geometry of the slope used for the stability analyses, including any filling of table land or changes in the slope geometry (e.g., cutting or filling of the slope).
- Indicate that the geotechnical engineer (or their representative) examined the slope and noted areas of active or previous instability; for slopes along water courses, identify where active erosion of the bank / slope toe exists.
- Indicate the geologic conditions within the slope that were used in the analyses and how those conditions were determined (e.g., based on the results of boreholes, inferred from observations of the soils exposed on the slope face, inferred from knowledge of the area's geology, etc.).
- Indicate the groundwater conditions within the slope that were used in the analyses and how they were determined. Note: For slopes in Champlain Sea (Leda) clay, the clay should generally be assumed to be saturated (i.e., located below the groundwater level) for the purposes of 'static' stability analyses, unless justified based on site specific data and conditions. However the effects of groundwater flow (i.e., conditions other than hydrostatic) may also be considered.
- Indicate the soil strength parameters that were used in the stability analyses (i.e., cohesion and friction angle) and how they were established (e.g., based on experience, inferred from testing in boreholes, or measured by laboratory testing).
- Indicate the soil densities (i.e., unit weights) used in the analyses and how they were determined (e.g., based on experience, correlations, laboratory test results, etc.).
- Indicate the factors of safety against instability of the analyzed slopes. Where the factor of safety against instability of the slope is less than 1.5, the Limit of Hazard Lands corresponding to a factor of safety of 1.5 should be provided and indicated on the plan. Note: the Limit of Hazard Lands can be assessed by suitable engineering analyses or by the simplified methods described in the Ministry of Natural Resources' Technical Guide - River and Stream Systems: Erosion Hazard Limit. The assessment should consider the most conservative condition that the slope could be expected to experience (i.e., the "worst case" condition). Loads from structures should be considered in the analyses. All development must be located outside of the Hazard Lands that are defined by that Limit of Hazard Lands (either above or below a slope). Note: Since retaining walls are constructed features, a factor of safety of less than 1.5 against global failure can not be accepted. That is, Hazard Lands should not be permitted to exist above retaining walls
- Indicate whether the Limit of Hazard Lands considers, where appropriate, the toe erosion allowances and erosion access allowances described in the Ministry of Natural Resources' Technical Guide - River and Stream Systems: Erosion Hazard Limit.
- For slopes along water courses, identify where erosion protection is required or planned. Details of the erosion protection, where required, should also be provided (e.g., type, limits and height of application, etc.).
- Show on a plan the location of the Limit of Hazard Lands determined by the analyses and indicate how the limit was defined (e.g., marked at the site by the engineer and surveyed, or drawn on the plan).
- Provide details on stabilization measures, if any are required, to increase the factor of safety and provide the Limit of Hazard Lands that is reported.
- Where the potential for rapid drawdown of the water level against the slope exists, indicate how the potential impact of that condition on the stability of the slope and the position of the Limit of Hazard Lands has been assessed.
- For slopes in clay greater than eight metres in height (or for any height of slope, where justified by the subsurface conditions or history of previous instability), indicate that the potential for retrogressive earth flow sliding has been considered and evaluated and, if such a potential exists, indicate how that potential has been considered in the establishment of the Limit of Hazard Lands.
- Indicate the factor of safety against instability under seismic conditions, including the potential for seismic liquefaction. Note: The minimum required factor of safety would generally be 1.1, or a suitable setback should be provided to a Limit of Hazard Lands. The horizontal seismic coefficient,

based on the current National Building Code, should generally be no less than 0.1, but should also consider the potential increase in the seismic coefficient due to the presence of loose or soft soils. Undrained loading conditions should be considered in these analyses.

- Be signed and sealed by a geotechnical engineer licensed in the Province of Ontario.

Notes:

1. For individual assessments of small structures proposed for the rear yards of individual residences (such as swimming pools and decks), lesser detail may be required. In those cases, a report should be provided which describes the slope geometry, describes the distance between the slope and the structure, and states whether or not there is a factor of safety of at least 1.5 against slope instability affecting that structure. At the discretion of the City, additional details may be required.
2. For sites with slopes exceeding the aforementioned 5 horizontal to 1 vertical and/or 2 metre high criteria for providing a slope stability report, but which are not steeper than 4 horizontal to 1 vertical and are no higher than 3 metres, and which in the opinion of the geotechnical engineer are not unstable or warrant detailed analyses, lesser detail may be required. In those cases, a report should be provided which describes the slope geometry, describes the distance between the slope and the structure/development, describes the expected subsurface conditions and states whether or not there is a factor of safety of at least 1.5 against slope instability affecting that structure/development. That information could be included as a section in a geotechnical/foundation design report. At the discretion of the City, additional details may be required.
3. Retaining walls should have a factor of safety of at least 1.5 against global instability; that is, against a failure of the soil behind and beneath the wall. However the City may also require that additional details be provided on the internal stability and/or structural design of a wall.
4. At the discretion of the City, the geotechnical engineer may be required to review and accept the grading plan (by means of a letter signed and sealed by a qualified geotechnical engineer, licensed in the Province of Ontario).
5. At the discretion of the City, the geotechnical engineer may be required to submit the detailed results of the analyses, for possible peer review.

Appendix A

Minimum Requirements for Slope Stability Assessment Reports

Appendix B

City of Ottawa Planning and Environment Committee and Council Approval