SLOPE HAZARD MANAGEMENT WITH THE CAUTIONARY ZONE APPROACH: A CASE HISTORY

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ABSTRACT
This paper documents 20 years of economical and successful slope hazard management along a 30 m high riverbank in Thunder Bay near Lake Superior using the Cautionary Zone Approach (CZA). The CZA to slope hazard management eliminates the need for stabilization works until the slope encroaches within a strip of land (the 'cautionary zone') adjacent the property to be protected. At this site, stability analyses using primarily the principles of saturated soil mechanics indicated unacceptable safety factors and could not justify the conventional design approach. However, a theoretical basis for the CZA was established using the principles of unsaturated soil mechanics. This confirmed that the CZA (using a 6 m zone for streets, 10 m zone for residences) provided a suitable degree of safety. The stabilization option which integrated best with the CZA was found to be a proprietary soil nailed biotechnical solution.

RÉSUMÉ
Cet article examine 20 ans de gestion économique menée avec succès d'une pente le long d'une berge de 30 m de hauteur à Thunder Bay près du lac Supérieur en utilisant l'approche de zone de sécurité (AZS). L'utilisation de l'AZS dans la gestion de risque des pentes élimine le besoin de travaux de stabilisation jusqu'à-ce que la pente empiète sur une zone (la "zone de sécurité") adjacente à la propriété à protéger. Sur ce site, l'analyse conventionelle utilisant principalement les principes de la mécanique des sols saturés indique un facteur de sécurité inacceptable et ne justifie pas la conception conventionelle. Cependant, une base théorique pour l'AZS a été établie en utilisant des principes de mécanique des sols non-saturés. Ceci confirme que l'AZS (en utilisant une zone de 6 m pour les rues, 10 m pour les résidences) procurait un niveau de sécurité acceptable. L'option de stabilisation qui s'intégrait le mieux avec l'AZS était une solution privée de clouage biotechnique.

1. INTRODUCTION

Twenty years ago an innovative approach to managing landslide hazards was developed in Thunder Bay, Ontario. This practical observational approach was successfully applied to stabilizing more than a kilometer of 30 m high riverbank that was regressing annually at a rate of 0.5 m per year (Fabius and Suke, 1990). Conventional slope stability analyses using the principles of saturated soil mechanics indicated unsafe conditions for streets and residences (i.e., factor of safety values less than unity). However, it was noted that there was a long history of observations indicating that slides were always shallow. This ensured substantial advance warning as long as the crest of the slope did not encroach within a 'cautionary zone' established adjacent the properties to be protected.

A Cautionary Zone Approach (CZA) was subsequently developed whereby the slope was allowed to regress, and stabilization measures were applied only when and where the crest of the slope reached the cautionary zone. Cautionary zone widths varying with respect to the degree of acceptable risk were established: 6 m for existing streets and 10 m for residences (Figure 1).

The CZA provides a new and economical approach to landslide hazard management, allowing land development within traditional 'hazard' zones and delaying by decades the need for remedial work (Fabius et al. 2004). Furthermore, it provides a rational basis for stabilizing only localized areas currently at risk. This approach has significant practical value with respect to reducing hazard management costs.

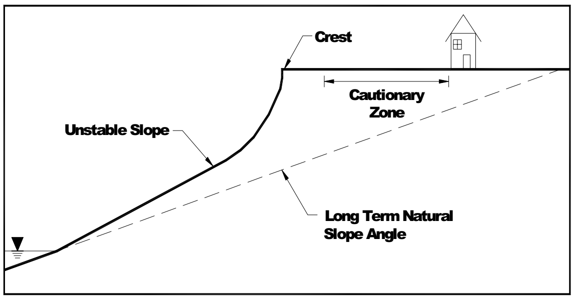

Figure 1. Illustration of the cautionary zone.

The CZA is fundamentally and significantly different from conventional approaches to hazard management in three ways:

i. the CZA does not rely on conventional stability analyses which have traditionally used saturated soil mechanics (often on the basis that unsaturated soil effects cannot be predicted, are temporary, and should not be relied upon in the long term),

ii. 'safe' zones (areas which are deemed safe in the long term) and their associated 'hazard' zones do not need to be established, and

iii. the zone to be managed for hazards is referenced to the property to be protected rather than to the hazard itself (the river or the slope).

The CZA can only be considered where there is confidence in predicting the maximum possible extent of a slide at any one time. This is the key limitation to the approach. Due to this reason, there is a need for providing a theoretical basis to the CZA in order to apply it successfully to other sites. This paper provides a case study of the Kaministiquia River slope and uses it to develop a theoretical basis for the CZA by applying the principles of unsaturated soil mechanics.

Another key limitation to the CZA is the difficulty in economically stabilizing small localized areas on an as needed basis when they encroach within the cautionary zone. Stabilization methods at this site were therefore reviewed with respect to their ability to economically integrate with the CZA.

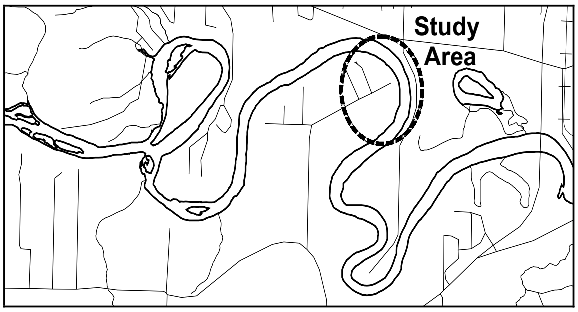

Figure 2. The Kaministiquia River.

2. BACKGROUND

The Kaministiquia River meanders through deep glacio-fluvial and glacio-lacustrine deposits on the way to its delta on Lake Superior in Canada. The study area comprises a 2 km stretch of steep 30 m high riverbank section in Thunder Bay (Figure 2). The slope is partially covered with mature trees. Where past failures have occurred, gullies have formed, devoid of vegetation. Perched groundwater in some areas seeps from sand layers near the top of the slope. Slope angles vary from 27° to 45° to the horizontal, the steeper sections being generally between gullies or at the scarps of gullies. Slope failures observed in the past have always been shallow, and occur after rainfall or during spring thawing.

Since 1972, the Lakehead Region Conservation Authority (LRCA) has had a concern for several houses, a street and a water main at the crest of the bank (Fabius and Suke, 1990). In 1977, Thunder Bay experienced a major storm and flood. Erosion caused some residents to loose as much as 10 m of soil from their yards (in multiple shallow slides) during that event which lasted several days. In 1982, a study concluded that the riverbank was regressing at a rate averaging 0.5 m/year. This natural process involved erosion at the toe resulting in an over-steepened slope that periodically failed and slid into the river.

A 1,200 m length of rockfill protection was placed along the toe to arrest further river erosion (Figure 3). It was predicted that the slope would naturally and gradually flatten to 24°, based on conventional slope stability analyses as well as observations of a nearby slope at an oxbow lake cut off from the river. This meant that eventually the street and several houses would be lost. Six houses that were within 10 m of the crest were purchased and demolished by the LRCA. Furthermore, a 350 m length of street where the crest encroached within 6 m (a cautionary zone) was stabilized with a retaining wall along the top of the slope (Figure 3) as described by Fabius and Suke, 1990.

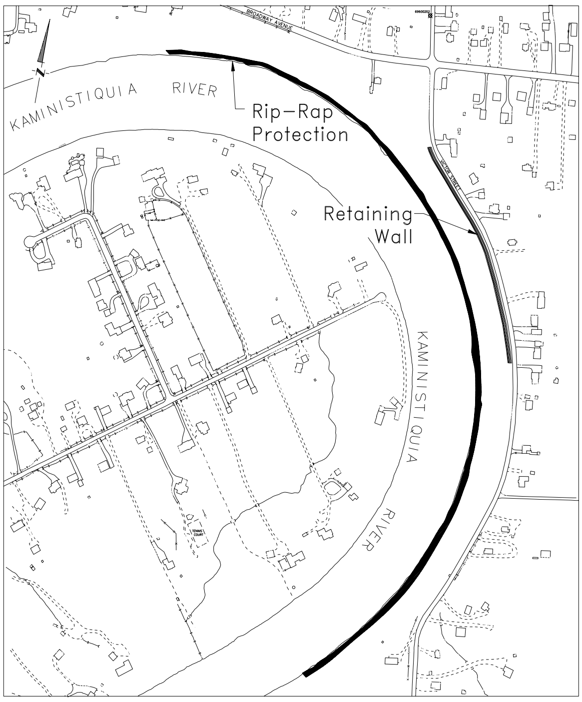

Figure 3. The study area.

The 6 m and 10 m unsafe zones established for the street and houses respectively were subsequently applied as a cautionary zone approach (CZA) for future landslide hazard management as the slope regressed. The CZA was found to be a practical and economical way to manage the hazards and risks in a prioritised, diligent, and acceptably safe way. Stabilization measures are only applied to localized areas where the slope crest encroaches on the cautionary zone, meaning that stabilization costs can be deferred for decades.

In summary, a cautionary zone consists of a strip of land along a property to be protected. Remedial action is required when the crest encroaches on the cautionary zone (Figure 1). The width of the cautionary zone can be varied with the desired degree of risk to be applied. At this site, the cautionary zone is 6 m for the street and 10 m for a house. These values are based on experience and on long term observations that slope failures at the site have always been shallow. The facilities to be protected are unlikely to be affected by a slope failure until the crest regresses to the cautionary zone.

3. SUBSURFACE CONDITIONS

The soil conditions at the site comprise (a) an upper 3 m of sand, over (b) a medium plastic firm to stiff clay layer varying in thickness from nil to 2 m along the slope, and (c) a deep deposit of non plastic silt, faintly varved, extending to more than 25 m below river level. The silt is compact with some dense zones.

Some of the slope has perched water tables above and within the upper clay layer. Monitoring has found that the main water table has a maximum level at 19 m below the crest at elevation 187.4 m as measured over the course of 1982. A higher level at 189 m was selected for design, based solely on engineering judgment. A minimum water table at 185 m was selected, 2 m above nearby Lake Superior where the river discharges.

4. UNSATURATED SOIL BEHAVIOUR

Conventional slope stability analysis is based on saturated shear strength parameters assuming the soil to be either dry or completely saturated. Such an approach conservatively models the worst case scenario of the slope behavior. In many practical cases, however, the soil failure surface can be unsaturated. The shear strength of a soil in an unsaturated condition is higher in comparison to soil that is in a state of saturated or dry condition due to the contribution of matric suction. For this reason, the cost of stabilizing a slope will be more economical if the analyses are based on the principles of unsaturated soil mechanics.

Surficial soils have high suctions due to the removal of water during evapotranspiration in dry weather. The matric suction (i.e., the negative pore water pressure with respect to atmospheric pressure) contributes towards the increase in soil strength at this time. However, soil strength decreases during wet seasons due to loss of suction. Specific case studies from Hong Kong, Singapore, Malaysia and more recently Canada have shown examples where instability was related to a loss of suction caused by climatic conditions (Ferreira et al. 2001).

The contribution of matric suction towards an increase in shear strength may be taken into account in the slope stability analysis if the ground water table is deep and the slip planes are shallow. The factor of safety of slopes will be higher under such conditions in unsaturated soil slopes. The shear strength contribution due to matric suction, ϕ^b, is typically higher closer to the surface of the soil and decreases to zero at the water table (Leong et al. 1998).

The typical cost of determining the engineering properties of unsaturated soils is ten times the cost of saturated soil properties. For this reason, simple techniques are used to apply unsaturated soil mechanics to engineering practice (Fredlund, 2000). In recent years, the soil-water characteristic curve (SWCC) has been used as a valuable tool to this end. The SWCC defines the relationship between water content (gravimetric or volumetric) or degree of saturation and suction. The variation of the shear strength of unsaturated soils with respect to matric suction can be predicted using the SWCC and the saturated shear strength parameters.

The SWCC can be obtained from direct laboratory measurement or indirect methods. To measure the SWCC in the lab, typically the suction in a saturated soil sample is increased and the water content change for each corresponding matric suction value is measured. A Tempe cell or pressure plate apparatus is used in the measurement of SWCC. Indirectly, SWCC can also be estimated from the grain size distribution curve.

The shear strength equation for unsaturated soils is given below.

$$\tau = c' + (\sigma_n - u_a)\tan\phi' + (u_a - u_w)\tan\phi^b \qquad [1]$$

where

τ = the shear strength;
c' = the effective cohesion;
σ_n = the total normal stress;
u_a = the pore-air pressure;
u_w = the pore-water pressure;
ϕ^b = the angle defining the rate of increase in shear strength with respect to soil suction;
ϕ' = the effective angle of internal friction;
$(\sigma_n - u_a)$ = the effective net normal stress on the failure plane; and
$(u_a - u_w)$ = the matric suction.

The above equation illustrates that matric suction in an unsaturated soil increases the shear strength.

5. OTHER CASE STUDIES

5.1 Slope Failure Due to Rainfall

Lin and Kung (2000) studies in Hong Kong showed that most of the rainfall-induced landslides in residual soils consist of relatively shallow slip surfaces above the groundwater table. The failed zone is usually less than 3 meters in thickness and is usually unsaturated. During the dry season the slope remains stable due to the contribution of matric suction, which increases the strength of the soil. Percolating water from heavy rainfall, however, saturates the soil and decreases the matric suction resulting in a lower safety factor and an increased possibility of slope failure.

A study by Nishigaki & Tohari (2000) includes measurements of pore-water pressure at failure initiation on a series of soil slope models due to rainfall. Experimental studies showed that non-circular shallow sliding was the most prevailing mode of failure, with the shearing plane cutting through the toe of the slope. The increase of failure potential mainly occurred in the near-surface region.

Gasmo et al. (2000) used numerical models to study the effect of infiltration on the stability of unsaturated slopes. The case study performed was on an unsaturated residual slope in Singapore. The numerical study found that infiltration was highest at the crest of the slope. The reason for such a behavior is associated with the movements of water vertically downwards. A large amount of infiltration at the crest increases the pore-water pressures in the soil and decreases the stability of the slope at that location.

In a study by Tsparas et al. 2003, two unsaturated slopes in Singapore were instrumented to monitor the pore water pressure changes during infiltration. It was found that total rainfall was one of the two controlling parameters for changes to the pore-water pressure, the other being initial pore-water pressures.

This background research on rainfall and its effects on slope stability were useful to understand the Kaministiquia River slope behavior. Observations over a 25 year period indicate that the depth of slides have always been shallow, circular and less than 3 m (Fabius & Suke 1990). Non flood related failures usually occur during spring runoff from snowmelt or during periods of heavy rainfall. With the toe protected from further river erosion, continuing failures are attributed to loss of soil suction as a result of water infiltration from precipitation.

5.2 Outer Slope Soil Layer

Rahardjo et al. (1998) showed that the pore-water pressure at the ground surface of a slope during the beginning of rainfall can increase to 0 kPa as the ground surface saturates. However, even though the ground surface is saturated, below the ground surface significant negative pore-water pressures still exist. As rainfall continues, the matric suction decreases as water infiltrates into the soil. When rainfall stops and evaporation begins, the matric suction will recover.

A study by Deutsher et al. (2000) on a Singapore residual soil showed that significant negative pore-water pressures developed in the upper three meters of slopes. However, during a moderate wet period, or even because of perched water tables, these suctions dissipated.

This information proved important when the unsaturated Kaministiquia River slope model was developed.

6. THE SLOPE MODEL

A model of the Kaministiquia River bank in 2003 was selected based on stratigraphy from several boreholes and logging various surface sections (Figure 4). The section includes a 2 m thick surface layer over the slope to represent soil susceptible to desaturation from infiltration. Laboratory testing included direct shear tests, grain size analyses, Atterberg limit tests, and moisture content tests.

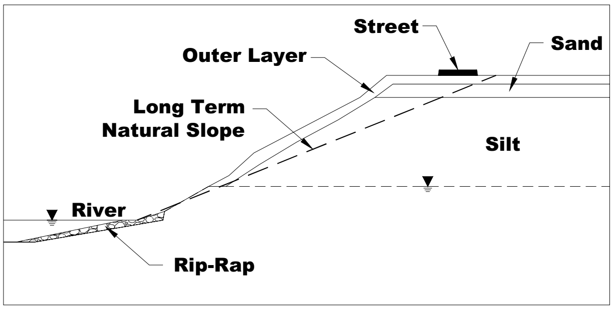

Figure 4. 2003 Kaministiquia River slope model.

The stability of the section was analyzed with a limit equilibrium analysis using SLOPE/W. Pore pressure contours were applied both above and below the water table, parallel to the soil layers. The following saturated parameters were applied: for the sand, $\phi'=30°$, $c'=0$, $\gamma=18.5$ kN/m^3; for the silt, $\phi'=30°$, $c'=0$, $\gamma=20$ kN/m^3.

The measured negative pore-water pressure and ϕ^b values were required for the two soil types (sand and silt) to undertake a rigorous slope stability analysis. However, negative pore-water pressure or matric suction data are unavailable for the Kaministiquia River slope. Due to this reason, an indirect approach was undertaken for the analysis through use of estimated SWCC and ϕ^b values.

The soil-water characteristic curves for sand and silt layers were estimated by SoilVision software based on grain size (Figure 5), specific gravity and, void ratio data. The SWCC for the predominant silt layer is shown in Figure 6. The soil matric suction (negative pore-water pressure) for the sand was estimated to vary between 1 and 100 kPa, and 5 and 500 kPa for the silt layer.

Because no value of the parameter ϕ^b for the soil layers was available, it had to be estimated. The parameter was varied from a maximum value of ϕ' (i.e., 30°) to a minimum of $0.5\phi'$ (i.e., 15°) based on published literature.

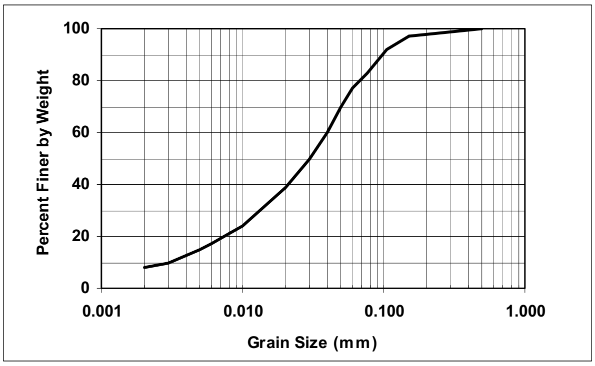

Figure 5. Grainsize curve of Kaministiquia River silt.

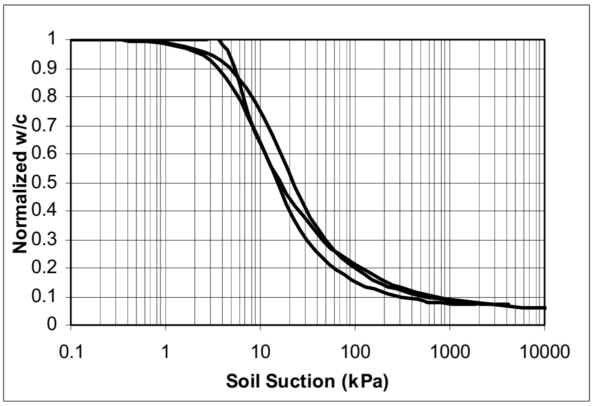

Figure 6. Estimated SWCC for Kaministiquia River silt.

7. ANALYSIS OF THE SLOPE

7.1 Back Analysis

A back analysis was carried out to check the slope model for reasonable factors of safety and failure depths. If necessary, the model would be adjusted to conform more closely to the actual conditions of the site. Two variables, matric suction and ϕ^b, were considered in the slope stability analysis.

7.1.1 Changes in Matric Suction

Matric suction was the first variable considered in the analysis. Results are summarized in Table 1 for seven different matric suction values using the minimum ϕ^b value, which is equal to 15°. Figure 7 illustrates the analysis results. For each case it was observed whether the critical slip surface was deep or shallow.

Case 1 illustrates the conventional analysis without soil suction (i.e., assuming fully saturated conditions). The failure mode is very shallow, basically a translational failure normally seen in a dry non-cohesive soil. This analysis provides the lowest factor safety of 0.82. This is too low for the Kaministiquia River slope since it is not actively failing except during severe rainfall events or as a result of further steepening from toe erosion.

Table 1. Effect of Matric Suction on Unsaturated Layers

	Suction Values at Top of Soil Layer (kPa)						
Case	1	2	3	4	5	6	7
Outer Layer	0	500	300	100	50	5	0
Sand	0	470	280	90	45	100	100
Silt	0	420	240	75	38	-80	80
Min. F of S	0.82	1.39	1.27	1.12	1.08	1.08	0.82
Mode	shallow	deep	deep	deep	deep	deep	shallow

Cases 2 to 7 model the 2003 unsaturated slope shown in Figure 4. From Case 2 to Case 5, the factor of safety increases with soil suction. The failure mode at the minimum factor of safety is deep, which does not model the observed slope failures.

Cases 6 and 7 simulate the outer 2 m layer of soil becoming saturated from rain, losing much of its strength contribution due to matric suction. Case 6 has 5 kPa of suction remaining in the outer layer, and 100 kPa in the layer below. The result is a deep seated failure illustrating that even a very small suction will hold a slope against shallow instability. With the matric suction at zero in the outer soil layer (Case 7), a shallow failure occurs within the outer layer.

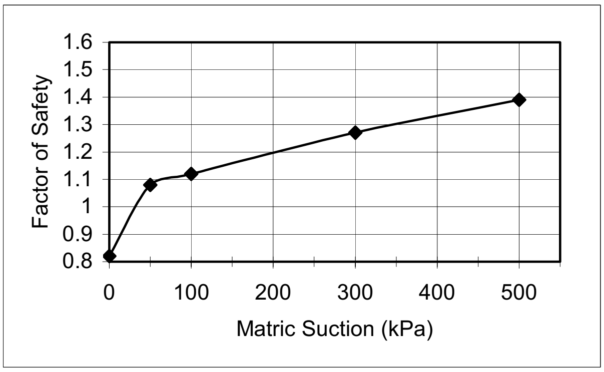

Figure 7. Variation in stability with matric suction in the outer layer.

Case 7 best represents actual site observations of failures and corresponds to published case studies with deep groundwater tables.

7.1.2 Changes in the Parameter ϕ^b

Case 4 from the previous section was used to compare how different values of the angle ϕ^b affected the factor of safety. Case 4 was chosen because it included a low suction in the outer layer, whereas Case 7 had the critical failure going through the saturated outer layer. From these studies it can be observed that changing ϕ^b in Case 7 would not cause any difference in the minimum factor of safety.

The parameter ϕ^b was modelled from a conservative value of 15° up to 30° in 5° increments. Figure 8 shows the variation of factor of safety with respect to the friction angle ϕ^b. The factor of safety increases as ϕ^b increases. However, the change in factor of safety is not significant showing that changing ϕ^b does not affect this particular slope significantly. However, to be conservative, ϕ^b=15° was selected to model the actual case.

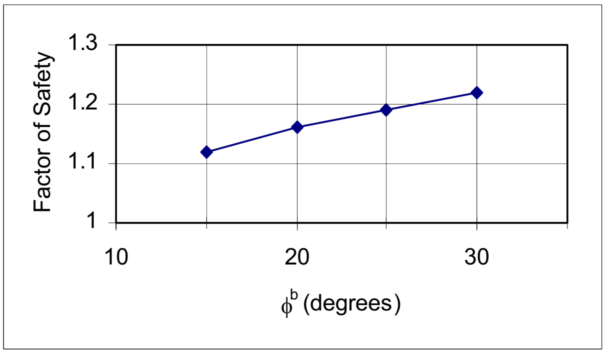

Figure 8. Effect of ϕ^b on stability.

7.1.3 Conclusions from Back Analysis

Case 7 with no suction in the outer 2m thick layer appears to most reasonably match the observation of shallow failures. The ground water table of 189 m elevation and friction angle due to suction, ϕ^b =15° will be used to analyze the CZA. Since there are no recorded deep failures, and there are no signs of deep soil movements such as cracks in the street at the crest, the safety factor against deep failures (through unsaturated soil below the outer layer) must be at least 1.1 if not higher. To ensure a factor of safety of 1.1 the soil suction must be at least 100 kPa (Table 1, Case 4). However, the matric suction could be up to 500 kPa (from the soil water characteristic curve for silt, Figure 6)

7.2 Cautionary Zone Analysis

The cautionary zone approach (CZA) has been successfully applied to the Kaministiquia River slope for the last 15 years, managing the risks without applying a factor of safety. Action is taken when the crest of the slope regresses to within 6 m of a road or 10 m of a house.

The factor of safety for the cautionary zones were determined by setting the slip surface radii limits such that slips were not allowed to go shallower than 6 or 10 m from the crest. A factor of safety range for both these cautionary zones was calculated. The matric suction range of 100 kPa to 500 kPa was considered in this analysis.

Table 2. Cautionary Zone Factor of Safety for ϕ^b=0.5ϕ'

	Factor of Safety	
Top suction contour (kPa)	100	500
For 6 m zone (road)	1.1	1.32
For 10 m zone (houses)	1.14	1.35

The analysis results for the cautionary zones along the Kaministiquia River bank are shown in Table 2 for ϕ^b=15°. The 6 m cautionary zone has a factor of safety range of about 1.1 to 1.3. The 10 m cautionary zone has a factor of safety ranging from about 1.2 to 1.4. The results would be slightly higher for ϕ^b=30°. Actual soil suction measurements are needed to calculate a more accurate factor of safety within this range. Note that as the slope regresses and flattens, the safety factors will increase slightly.

These factors of safety can be compared to target values used in conventional approaches to managing slope stability risks. Where conditions are well defined, target factors of safety usually range from 1.2 to 1.3 for embankments and slopes that support roads. Therefore, the CZA for managing risk to the street on the Kaministiquia riverbank in fact provides a factor of safety of at least between 1.1 and 1.3 (depending on the actual value of soil suction) and appears reasonable.

A typical target factor of safety value for a higher consequence failure (i.e., a house) is 1.5. Therefore, the CZA for managing risk to residences in fact provides a factor of safety of at least between 1.15 and 1.35. This is slightly lower than the conventional approach.

The relative degree of safety can also be assessed with respect to the width of the cautionary zone. The wider the zone, or the further the feature to be protected is from the crest, the higher its degree of safety. The CZA provides an increase in stability relative to that of a zero cautionary zone (a slip intersecting the crest) of at least 34% for the 6 m zone, and at least 46% for the 10 m zone. This in itself also provides a rational design basis.

8. STABILIZATION OPTIONS

The ‘do nothing’ option is not advisable once the slope crest has reached the cautionary zone because the next slide may take out part of the property to be protected. When this failure will occur is unpredictable, but would most likely be during the spring thaw, during extreme flooding or during/after prolonged/severe rainfall. It is not certain that accurately monitoring surface or underground slope movements would provide sufficient warning here.

Flattening the slope by placing fill at the toe can be utilized to improve stability. However, to preserve the property to be protected at the crest, this would require the placement of substantial fills into the river to achieve adequate stability. This was not deemed suitable due to significant negative impact on the river hydraulics and environment.

A logical extension of the CZA concept is to simply rebuild the crest of the slope for a small distance outside the cautionary zone using high strength soil such as rockfill. Then, the slope could be managed once more with the CZA (i.e. delaying further stabilization until the fill slides down the slope and the crest once again encroaches within the cautionary zone). This could potentially be a simple and low cost approach. The option was discarded after assessing the site preparation requirements, the impacts of the continuing slides on the slope and river, and the unpredictable rate of further slides of the fill.

Slope drainage by lowering or eliminating the upper perched water tables was also investigated. It was found that this had an insufficient effect on the overall slope stability.

A 12 m deep soldier pile and lagging retaining wall located near the crest of the slope was originally used to stabilize 350 m of street along the crest. This system has performed adequately to date. To the south, two decades later, a few new areas have encroached on the 6 m cautionary zone. The application of such a substantial wall to short (< 15 m long) sections is very costly, and is not considered suitable for stabilizing localized sections on an as required basis. Another option considered for the top of the slope was mechanically stabilized earth walls. However, these involve excavating and backfilling large portions of the property to be protected (including street and a water main), and are therefore neither economical nor practical for use with the CZA.

The construction of a retaining wall near the toe of the slope was also considered. Several types were investigated. It was found that such a wall would be more costly than a wall at the top of the slope given the larger forces, the longer wall lengths involved for the same area to be protected and the cost of access.

Lightweight fill can be used to replace the upper soil, but even 4 to 8 m of sub-excavation at the top of the slope was found to have limited effect given the height of the slope. Deeper excavations would be impractical and the costs of the lightweight fill over the length of roadway would be very excessive.

An in situ method of stabilization is soil nailing. This is a retaining system where soil nails (long steel rods) are typically grouted into a grid of holes drilled into the slope face, and then faced with wire mesh and shotcrete. Soil nailing was originally rejected due to the practical and economic issues with drilling equipment supported on the steep slope. Furthermore, the facing and associated drainage layer would not stand up to frost action.

Recently, an alternative (proprietary) soil nailing system has been developed specifically for over-steepened slopes (Tozer and Fabius, 2000). This system eliminates the need for drilling/grouting, and replaces the shotcrete facing with a biotechnical facing. The soil nails in this case provide shear resistance along potential slip surfaces. As such, the soil nails must be installed (in this case by driving, vibrating or rotation) to below potential slip surfaces that have an inadequate factor of safety. For potential slip surfaces near the surface of the slope, additional resistance is provided through vegetation (roots) and/or the installation of special heads near the top of the soil nail.

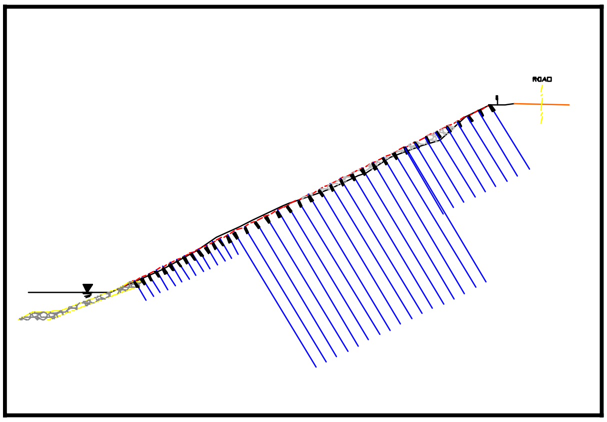

Figure 9. Typical soil nail design layout.

The limiting factor for economical soil nails is the ability to install the nails to sufficient depths without drilling. Based on numerical analyses, soil nails driven to a depth of 21 m along the upper half of the slope increase the factor of safety to about 1.3, equivalent to that of the soldier pile retaining wall already in place. Subsequent field trials confirmed that soil nails could be economically installed on the slope to the required depth without adverse impacts to the existing vegetation. A typical design layout for nails is illustrated on Figure 9.

A major attraction of this method is that it can be applied to specific areas where the slope has regressed to within the cautionary zone without treating an entire length of slope as in the case of a retaining wall. This is a flexible system that can be improved or modified as required, based on performance. Costs were less than half of other systems.

9. CONCLUSIONS

This paper has provided a case study of slope hazard management of a 30 m high riverbank in an urban area, using the Cautionary Zone Approach. Unsaturated soil mechanics provides a rational basis for the CZA. For this approach, stabilization measures are not required until the crest encroaches on the cautionary zone, defined as a strip of land adjacent the property to be protected. The width of this land is selected based on the maximum possible slide depth and the degree of acceptable risk, and allows for sufficient warning before action needs to be taken. The CZA has been successfully applied along the Kaministiquia River for 20 years, in spite of conventional approaches indicating unsafe conditions. This has delayed by decades the implementation of remedial work, providing the ability to stabilize only localized areas currently at risk.

For the CZA to be confirmed with analytical tools, the analysis must involve the principles of unsaturated soil mechanics and needs to address the changing slope geometry with time as the crest approaches the cautionary zone. By analyzing the unsaturated soil conditions at the site in question, factors of safety were calculated to compare the CZA to the more conventional approach, (which assumes fully saturated parameters, indicating unstable conditions). The 6 m cautionary zone for the street provides it with a factor of safety value of at least 1.1 to 1.3, similar to a conventional design value, and has a 34% improvement compared to a street with no setback from the crest. The 10 m cautionary zone for houses provides a factor of safety of at least 1.15 to 1.35, slightly lower than a conventional design value, and has a 46% improvement over no setback. Actual in situ measurements of matric suction would reduce the range of these values.

For successful application of the CZA, it was found that there must be confidence in the following: (a) that no deep failures will occur as a result of excessive toe erosion (for example, by waves or currents), (b) that the water table is always deep, ensuring adequate safety against deep failures, (c) that the depth of matric suction loss from infiltration is shallow, and (d) that the deeper soil will always retain adequate matric suction for the necessary factor of safety against deep failures.

All the above can readily be addressed with current tools, investigation methods and published experience available to the geotechnical engineer.

For this case study, it was found that an in situ stabilization system using driven soil nails with a biotechnical facing was by far the most economical system. Furthermore, it integrates well with the CZA permitting stabilization of localized areas on an as needed basis.

10. REFERENCES

Deutscher, M.S., Gasmo, J.M., Rahardjo, H., Leong, E.C., & Tang, S.K. 2000. Field measurements of pore-water pressure profiles in residual soil slopes of the Bukit Timah Granite Formation, Singapore. Proceedings of the Asian conference on unsaturated soils, Singapore, Asia, pp 777-782.

Fabius, K.A., Vanapalli, S.K., Garga, V.K. & Fabius, M. 2004. Managing landslide hazards with the cautionary zone approach. IX International Symposium on Landslides, Rio de Janiero, Brazil.

Fabius, M., & Suke, S.M. 1990. The investigation and stabilization of a high river bank in Thunder Bay. 43rd Canadian Geotechnical Conference, Canada, Quebec, pp 69-75.

Ferreira, N.J., Blatz, J.A., Graham, J., & Kenyon, R.M. 2001. Examination of instability in an unsaturated highway cut. The International Association of Hydrogeologists, The Canadian Geotechnical Society, pp 894-901.

Fredlund, D.G. 2000. The 1999 R.M. Hardy lecture: The implementation of unsaturated soil mechanics into geotechnical engineering. Canadian Geotechnical Journal, 37: 963-986.

Fredlund, D.G., and Rahardjo, H. 1993. Soil mechanics for unsaturated soils. John Wiley & Sons, New York.

Gasmo, J.M, Rahardjo, H., Deutscher, M.S., & Leong, E.C. 2000. Preliminary assessment of slope stability with respect to rainfall induced slope failures. Proceedings of the Asian conference on unsaturated soils, Singapore, Asia, pp 783-788.

Gasmo, J.M., Rahardjo, H., Leong, E.C. 1999. Infiltration effects on stability of a residual soil slope. Computer and Geotechnics, 26: 145-165.

Leong, E.C., Rahardjo, H., Chiam, S.L., and Gan, C.C. 1998. Suction profiles of a residual soil slope as affected by climatic conditions. Proceedings of the second international conference on unsaturated soils, Beijing, China, Vol. 1, pp. 231-236.

Lin, H.D., & Kung, J.H.S. 2000. Rainfall induced slope failures in Taiwan. Proceedings of the Asian conference on unsaturated soils, Singapore, Asia, pp 801-806.

Low, T.H., Faisal, H.A., & Saravanan, M. 2000. Suction and infiltration measurement on cut slope in highly heterogeneous residual soil. In Proceedings of the Asian conference on unsaturated soils, Singapore, Asia, pp 807-811.

Nishigaki, M., & Tohari, A. 2000. Laboratory experiments on initiation of rainfall-induced slope failure. Proceedings of the Asian conference on unsaturated soils, Singapore, Asia, pp 819-824.

Nishimura, T., & Fredlund, D.G. 2000. Relationship between shear strength and matric suction in an unsaturated silty soil. Proceedings of the Asian conference on unsaturated soils, Singapore, Asia, pp 563-568.

Rahardjo, H., Leong, E.C., Gasmo, J.M., & Tang, S.K. 1998. Assessment of rainfall effects on stability of residual soil slopes. Proceedings of the second international conference on unsaturated soils, Beijing, China, Vol. 1, pp. 280-285.

Tozer, S.E. and Fabius, M. 2000. Steep nailed embankment technology: 2 case studies. 51st Annual Highway Geology Symposium, Seattle, USA.

Tsaparas, I., Rahardjo, H., Toll, D.G., Leong, E. 2003. Infiltration characteristics of two instrumented residual soil slopes. Canadian Geotechnical Journal, 40: 1012-1032.

11. ACKNOWLEDGEMENTS

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