On the mechanics of seepage induced cohesionless soil slope instability as applied to foreshore engineering

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ABSTRACT Bank instability is a common feature along rivers and coastal areas, owing to erosion by moving water. Further instability is likely to occur due to groundwater seepage exiting the slope at the face of the bank, either due to surface water infiltration, or changes in water elevation along the toe of the bank, leading to "draining conditions" (i.e. seepage of water towards the face as the tide recedes).

Observations of seepage induced soil slope instability have been made by numerous hydrologists, geotechnical engineering practitioners, and researchers. However, the mechanics of the mode of failure have not been quantified, or likely even fully understood.

A review of the available literature provides case histories for seepage induced bank and slope failures, and the factors affecting a slope's ability to resist the seepage forces. Although there is a significant amount of literature devoted to this topic, very little slope stability related literature discusses the failure mechanisms. Researchers studying the stability of zoned earth fill dams have published numerous papers, which discuss the stability of soil subjected to seepage. Applying this work to slopes provides insight into the mechanics of seepage induced slope instability.

The author provides further discussion regarding the susceptibility of natural soils to seepage induced slope instability by comparing natural soils to laboratory testing. A possible solution to seepage induced slope instability utilizing Critical State Soil Mechanics is also discussed.

Introduction

Many published papers have provided a good, qualitative analysis of seepage-induced slope instability (Williams 1966; Eisbacher & Clague 1981; Hungr & Smith 1985; Hagerty 1991a; Crosta & di Prisco 1999; Evans & Savigny 1994; Cavers 2003). However, none touch upon the quantitative analysis required to sufficiently describe the failure mechanism(s). Terzaghi & Peck (1948) touched on the subject in regards to piping failure in dams, which they described as follows:

> [Piping] may be due to scour or subsurface erosion that starts at springs near the downstream toe and proceeds upstream along the base of the structure or some bedding plane. Failure occurs as soon as the upstream or intake end of the eroded hole approaches the bottom of the reservoir. The mechanics of this type of piping defy theoretical approach.

Although Terzaghi and Peck did not consider natural slopes when discussing the mechanics of seepage-induced erosion, the mechanics of seepage-induced instability in a slope and dam are one and the same (Li & Fannin 2013), with the latter being a man-made structure with a known groundwater source.

The intent of this literature review was to bring together concepts from several areas – soil mechanics (soil properties, strength criteria, and filter design), groundwater hydrology, geomorphology, and others – and determine a framework for modelling seepage-induced slope instability.

Many researchers and practitioners have considered rainfall-induced slope instability; however, rainfall intensities are a trigger, not a failure mechanism. Many studies have been conducted to correlate rainfall and slope failures (Yoshida et al. 1991; Au 1998; Collins & Znidarcic 2004; Zhang et al. 2005; Zhang et al. 2014; and many others), but due to the complex nature of the geology and hydrological properties of the system (Au 1998), correlations fail to capture all slope failures. Most often, studies focused on rainfall induced slope failure are concerned with shallow, translational slides. The inability to find a suitable correlation between rainfall and slope failure is likely due to the lack of knowledge regarding the actual failure mechanism present that is being triggered by the rainfall. This review is meant to form a basis for providing a framework to clear up any misconceptions regarding rainfall induced slope failures.

Case Histories

Hagerty (1991a), Hagerty (1991b), and Fox et al. (2007) provide descriptions of seepage induced erosion of shorelines. While Hagerty does not implicity provide a quantification of the mechanisms, Fox et al postulate what the mechanisms of the erosion could be.

Although not presenting examples of shoreline instability, many case studies have been presented in engineering literature over the years. The following case histories, as described by Harrison (2014), provide examples of seepage induced soil slope instability in cohesionless soils:

- 1979 rainstorm in Vancouver, BC (Eisbacher & Clague 1981)
- Gilley Brothers Gravel Pit in Port Moody, BC (Armstrong 1984, Hungr & Smith 1985, and Evans & Savigny 1994)
- Gropello Slope Failure, Po Plain, Italy (Crosta & di Prisco 1999)
- Groundwater blowoff failures (Cavers 2003)

Influence of Soil Properties

Muir Wood (2007) observes that granular soils are affected by a particle-continuum duality, in that granular soils are modeled as a continuum, when in fact they are a collection of disconnected particles interacting through physical contact. Engineers attribute granular soil strength parameters to a soil continuum, although it is the interaction between the particles resulting in cohesion (c) and friction (□) for a soil.

The mechanical properties of the granular soil forming a slope will determine how the applied stresses will affect the stability of the slope. In this case, the failure mechanism under consideration is seepage-induced stresses. The effect of water on the soil cannot be ignored. Furthermore, the in-situ stress state and slope geometry must be considered to determine what affect they may have on the soil.

The soil properties typically considered for slope stability are the Mohr Coulomb failure criteria, and unit density/weight. However, literature indicates that the grain-size distribution (Muir Wood 2007), saturation (Yoshida et al. 1991; Orense et al. 2004), and subsequently, the hydraulic conductivity (K), need to be considered.

Geomorphological Nature of Slopes

Homogeneous Slopes

Laboratory experiments are typically conducted on homogenous soils, manufactured to provide researchers with homogeneous and isotropic materials, although the manner in which the samples are reconstituted will affect the results (Oda 1972). In nature, no soil slope is truly homogeneous, as the gradation of the soil will differ from one sampling point to the next. The sampling volume will determine the extent of the homogeneity of the soil.

For soil slopes in thick alluvial deposits, common in southwest British Columbia, the entire slope, taken as one sampling volume, can be considered as homogeneous. However, due to the depositional environment of river systems, interbedded layers of finer and coarser materials will be present (Bear 2007).

For the case of modelling seepage-induced slope instability in slopes, it is not a valid assumption to assume the slope consists of homogeneous, isotropic soil. Examples of slope failures in southwest BC (Hicock & Armstrong 1985) indicate that slopes never consist of one soil that can be considered as homogeneous and isotropic.

Layered Slopes

The typical geology of southwest BC consists of a complex layered system of alluvial, colluvial, and glacial sediments (Armstrong 1957) deposited over the course of several epochs. The most recent glaciation ended some 10,000 years ago. Since then, mainly alluvial and colluvial deposits have accumulated over dense glacial sediments and bedrock (Armstrong 1957).

After the last glaciation, and as the glaciers retreated, the underlying geology underwent uplift (Eisbacher & Clague 1981) and likely was not uniform across the entire area of glaciation due to the differing thicknesses of soil and bedrock. This has led to soils, which were once flat shortly after deposition, becoming tilted. Furthermore, soils deposited over bedrock will conform to the inclined surface of the bedrock.

Throughout the layers of soil, preferential flowpaths will form in the layers of higher hydraulic conductivity (K) (Bear 2007). Furthermore, the contrast in K between an overlying alluvial or colluvial soil layer (aquifer) and either a basal till deposit or bedrock (aquitard), will lead to a ponding of water and increased flow in the alluvial or colluvial soil layer (Bear 2007). It is these layers of preferential flow paths that are susceptible to seepage-induced instability and when the layers daylight in a slope face, surficial erosion can lead to slope instability.

Experiments have shown, categorically, that natural soils have fabric resulting from the method of deposition (Oda 1972). Experiments as early as 1972 and as recent as 2011 have shown that fabric has a direct impact of the behaviour of soils. Li & Dafalias (2011) have shown that upwards of 120 percent strain is required to destroy the fabric, after which it could be hypothesized that a soil is homogeneous and isotropic; however further study of this phenomenon would be required to show that the soil is truly isotropic in all cases (e.g. $K_x = K_y = K_z$).

Modelling of Seepage Induced Erosion

Current slope stability techniques are usually limited to limit equilibrium (LE) analysis (Crosta & di Prisco 1999; Zhang et al. 2005). Any number of commercially available software programs can be used for this task. However, they only consider groundwater with respect to the buoyancy effect of a soil layer(s), while completely ignoring the effect of seepage forces due to gradients present in the slope. This is due to the fact that transient conditions are not considered. Groundwater levels fluctuate, especially as the influence of tidal affects impact a slope along a lake, river, or coastline. The seepage forces acting on the slices are ignored.

Goal

The goal of any numerical model (LE, finite element (FE), finite difference (FD), or other modelling method) is to accurately predict the effects of a perceived failure mechanism (i.e. circular slip failure, compound slide, etc…). The predictive nature of a model is limited by the user's understanding of the problem.

With respect to seepage-induced slope instability, a model would need to quantify the gradients throughout a slope and determine if the soil in the slope can achieve resisting forces greater than the seepage forces, in addition to the driving forces traditionally considered in slope stability modelling.

Use of Available Numerical Models

At the time of writing, no numerical method was available that combined seepage analysis and the movement of soil due to seepage forces. Available numerical codes for seepage analysis (SEEP/W, SVFLUX) provide the calculated gradients for a problem, but cannot be used with other codes directly to calculate the likely extent of seepage-induced erosion.

Research in literature (Fox et al. 2007; Zhang et al. 2005) provide examples of semi-coupled analyses, but most couple finite element seepage analysis with limit equilibrium analysis. Zhang et al. do provide a comparison of a semi-coupled finite element seepage and stress analysis with a semi-coupled finite element seepage analysis and limit equilibrium analysis but did not consider seepage-induced erosion, only rainfall as a trigger.

Seepage through an unsaturated soil (assuming the soil is not fully saturated to begin with, which is often the case due to dry periods punctuated by wet periods) require a coupled analysis solution of the governing equations describing the equilibrium of the soil structure and the mass flow of the water phase (Zhang et al. 2005). Zhang et al. provide examples from literature for formulations of coupled analyses and numerical solutions of combined seepage and deformation problems. From the examples provided by Zhang et al., all but Pereira (1996) consider unsaturated soils or partially saturated soils, which do not apply to seepage-induced soil slope instability. Pereira considers seepage but not seepage-induced erosion.

Further work is required to determine which methods best suit the modelling of seepage-induced slope instability. Work by researchers studying earth fill dam internal stability (Kenney & Lau 1985; Skempton & Brogan 1994; Tomlinson & Vaid 2000; Li & Fannin 2008; Moffat et al. 2011; Moffat & Fannin 2011; Li & Fannin 2012) can likely be used to develop numerical tools to model seepage-induced slope instability.

Erosion of Soil Due to Seepage

The case studies presented above (Williams 1966; Eisbacher & Clague 1981; Hungr & Smith 1985; Crosta & di Prisco 1999; Evans & Savigny 1994; Cavers 2003), all considered slopes consisting of layered stratum. Furthermore, even in natural soil slopes that are considered homogeneous, interbedded layers of coarser or finer material will likely be present, indicating a period in the depositional record of when an event disrupted the depositional environment in some way. The presence of layers in a slope leads to variations in permeability, which provides preferential flow paths in the slope. As such, only layered slopes will be considered in this section.

Hicock & Armstrong (1985) provide a general description of the soil stratigraphy for what they refer to as the Mary Hill gravel pit, which is likely the Gilley Brothers Gravel Pit described above (Armstrong 1984; Hungr & Smith 1985; Evans & Savigny 1994).

Groundwater seepage has been observed in the slope face of Quadra Sand exposed in the escarpments at UBC (Piteau Associates 2002). This indicates that a groundwater source is available to the Vashon Drift that was mined at the gravel pit. Furthermore, it can likely be generalized that even though the natural slopes consist of glacial sediments, they will still conduct groundwater.

Soil's Susceptibility to Seepage-induced Erosion

Internal instability of a soil is the result of the coarser fraction's inability to prevent the migration of the finer fraction, as a result of groundwater seepage (Fell et al. 2005). This is not connected to the surficial erosion that occurs due to surface water runoff (Moffat et al. 2011). A soil's susceptibility to seepage-induced erosion depends on whether it is in fact internally stable or unstable.

Muir Wood et al. (2010) define internal erosion as a process that removes fines and narrows the grading of a soil. Muir Wood et al. go on to say that the evolution of the grading accompanying internal erosion is influenced by the stress conditions present and more importantly, according to them, by the flow regime of the water seeping through the soil.

Furthermore, the gradient profile through the soil will control when a finer fraction will begin to move. Research (Kenney & Lau 1985; Skempton & Brogan 1994; Li & Fannin 2008) has shown that the critical gradient to trigger migration of the finer fraction will depend on a soil's internal stability

A soil's susceptibility to seepage-induced erosion is controlled by its 1) internal stability and 2) the groundwater gradient profile through the soil. Recognition has been given to three processes that lead to the movement of particles within a soil that is internally unstable: 1) suffusion, 2) suffosion, and 3) piping. Each process is described below.

Processes

General

A soil possesses a primary and secondary fabric, where the primary fabric supports the imposed stresses, and the secondary fabric consists of loose particles that fill the voids in the primary fabric (Kenney & Lau 1985). The particles in the secondary fabric can be moved due to applied seepage forces (Kenney & Lau 1985).

Where the constrictions between the particles of the primary fabric are larger than the particles of the secondary fabric, the secondary fabric can move from pore space to pore space (Kenney & Lau 1985). Constrictions smaller than the secondary fabric are required to stop the movement of particles and this is referred to as a filtration zone (Kenney & Lau 1985; Tomlinson & Vaid 2000).

Definitive definitions of the primary and secondary fabric are not provided, however, it should likely be considered that the matrix of the soil is carries the imposed stresses. The gradation of the soil will determine what portion of the soil – coarse or fine fraction – makes up the matrix. Where the matrix consists of the fine fraction (sand or silty sand), the constrictions for the secondary fabric to move would be too small to support movement, if at all. However, where the matrix consists of the coarse fraction (gravel) the constrictions would likely support the movement of the secondary fabric.

Suffusion

Suffusion is a process under which fine particles rearrange within the voids between the coarser fraction of the soil (Moffat et al. 2011). Only the permeability in the zone where suffusion occurs is affected (Moffat et al. 2011). Furthermore, no change in the volume of the soil in the vicinity of the suffusion occurs (Moffat et al. 2011) due to the formation of a filtration zone that allows the soil to self-filter itself and reduce the likelihood of suffosion (Tomlinson & Vaid 2000).

Based on the above, it can be expected that a change in porosity due to soil movement will lead to an increase in localized permeability. Stability of a slope would be affected if the localized increase in permeability reached the slope face, allowing for the continued movement of soil.

A filtration zone would be required behind the slope face to prevent a change in volume. The ability of a soil to develop a filtration zone, either internally or near the slope face with the addition of vegetation, would increase the resistance to seepage-induced instability.

Suffosion

Suffosion also results in the movement of the finer fraction through the voids of the coarser fraction, however, it is accompanied by a rapid change in local permeability and a rapid change in volume (Moffat et al. 2011). The rapid change in local permeability is due to the sudden loss of material and subsequent decrease in volume. Where suffosion occurs at the face of a slope, erosion occurs.

It can be expected that the occurrence of suffosion at the face of a slope will result in a pockmark like feature appearing, with the volume of lost soil deposited either on the slope face below the void, or accumulating on a bench or at the toe of the slope.

Piping

Piping, or what Bendahmane et al. (2008) refer to a backward erosion, occurs when suffosion progresses into the slope, either as suffosion continues to occur at the back of newly formed voids, or along paths where suffusion has occurred. Piping, or backward erosion, occurs when particles are detached from the slope face, or back of voids, by seepage forces acting on the soil (Bendahmane et al. 2008; Hagerty 1991a). Ultimately, piping is the formation of circular conduits in the soil (Hagerty 1991a). Where seepage erosion occurs over a wider area, the piping is commonly called sapping (Hagerty 1991a).

In cohesionless soils, or soils that exhibit some form of apparent cohesion, the soil overlying the piping can only be supported by arching effects in the soil. As the piping removes larger volumes of soil and larger voids are created, the arching effects are diminished, and the soil structure collapses, as was described in the case histories referenced above. As was noted by Hagerty (1991) and Cavers (2003), when the debris from a collapsed portion of the slope impedes the exit of seepage from the slope, the seepage will not continue until the pore water pressures increase to the point of exceeding the resisting force of the debris material. However, the impeded seepage may be redirected to another area of the slope where the process of erosion may continue (Hagerty 1991a).

Concentration of flow occurs due to inequalities in the hydraulic conductivity within a soil layer and between adjacent soil layers, such as can be seen in a layered soil slope (Hagerty 1991a). Although piping has been most often observed in alluvial sediments, as alluvial sediments most often occur in layers or lenses with variation in K between layers, piping can also occur in glacial sediments, and clayey soils with interbedded granular layers with a higher K than the clay (Hagerty 1991a).

Exit Gradients at Slope Face

Internal Stability of Soil

Soil under the influence of groundwater is subject to seepage forces. Equation 1 quantifies the seepage pressure acting on a soil.

[1] $$P_{seepage} = i \cdot \gamma_w$$

where $P_{seepage}$ has units of unit weight per unit volume of soil (Terzaghi & Peck 1948).

The governing parameter in Equation 1 is the gradient (i). As such, the gradient is used to quantify the stability of a soil through which groundwater in flowing. Lambe & Whitman (1969) postulated that a gradient greater than approximately 1 would lead to instability of a soil, as shown in Equation 2.

[2] $$i_{cr} = (1-n)(1-\rho) = \frac{\gamma'}{\gamma_w}$$

where n is the porosity, ρ is the specific gravity of the grains, γ' is the buoyant unit weight of the soil and γ_w is the unit weight of water.

Work by Kenney & Lau (1985), focusing on earth fill dam filter design, introduced the concept of an internally unstable soil. Before their work was published, the assumption was made that only gradients greater than approximately 1 would lead to the condition of piping (Lambe & Whitman 1969). Work completed after Kenny and Lau has provided further quantification of internal instability (Skempton & Brogan 1994; Li & Fannin 2008; Moffat et al. 2011). For internally unstable soils, the critical gradient to cause piping is significantly lower than 1 (Kenney & Lau 1985; Skempton & Brogan 1994; Li & Fannin 2008; Moffat et al. 2011).

Internally Unstable Soils

The onset of internal instability has been shown to be accompanied by the following as observed during large permeameter tests (Moffat et al. 2011):

- Washout of finer fraction; and
- “Boiling action” at the top of specimen under vertical flow conditions

As described above, engineers have traditionally designed earth structures such that gradients due to groundwater flow were maintained below 1. However, researchers have shown that soils can be internally unstable below a gradient of 1 (Skempton & Brogan 1994).

Soil slopes consisting of cohesionless soil, which could be considered as internally unstable, will likely be susceptible to seepage-induced erosion at gradients less than 1. If the gradient exceeds the critical gradient of a soil where groundwater exits a slope, erosion can be expected to occur. Furthermore, if the critical gradient for the soil is present within the slope, erosion will likely occur well behind the slope face.

Effect of Slope Geometry on Exit Gradients

The inclination of the soil layer acting as the preferential flow path and the slope face inclination have a direct impact on the critical gradient at which instability occurs (Ghiassian & Ghareh 2008). The condition where the flow line is horizontal and the slope face near vertical is the worst case for exit gradients, with the gradient approaching ∞. Equations 3 and 4 (Ghiassian & Ghareh 2008) provide a proof based on the infinite slope model (Griffiths et al. 2011).

[3] $$i_{exit,hort} = \frac{\sin\beta}{\sin\lambda} = \tan\beta$$

[4] $$i_{exit,parallel} = \sin\beta$$

where i_{exit} is the exit gradient at the slope face, β is the inclination of the slope measured from the horizontal, and λ is the seepage direction measured clockwise from the slope normal, as defined by (Ghiassian & Ghareh 2008). Equations 3 and 4 indicate that the exit gradient is independent of change in head from the water source to the point where seepage is exiting the slope, and the horizontal distance between those two points.

Figure 1 presents a comparison of the exit gradient for parallel flow (Fig. 1a) and horizontal flow (Fig. 1b). As can be seen from Fig. 1, the exit gradient for seepage that is considered as parallel flow is independent of the seepage direction/inclination, whereas the exit gradient for non-parallel flow or horizontal flow is dependent on both the slope face inclination and seepage direction/inclination.

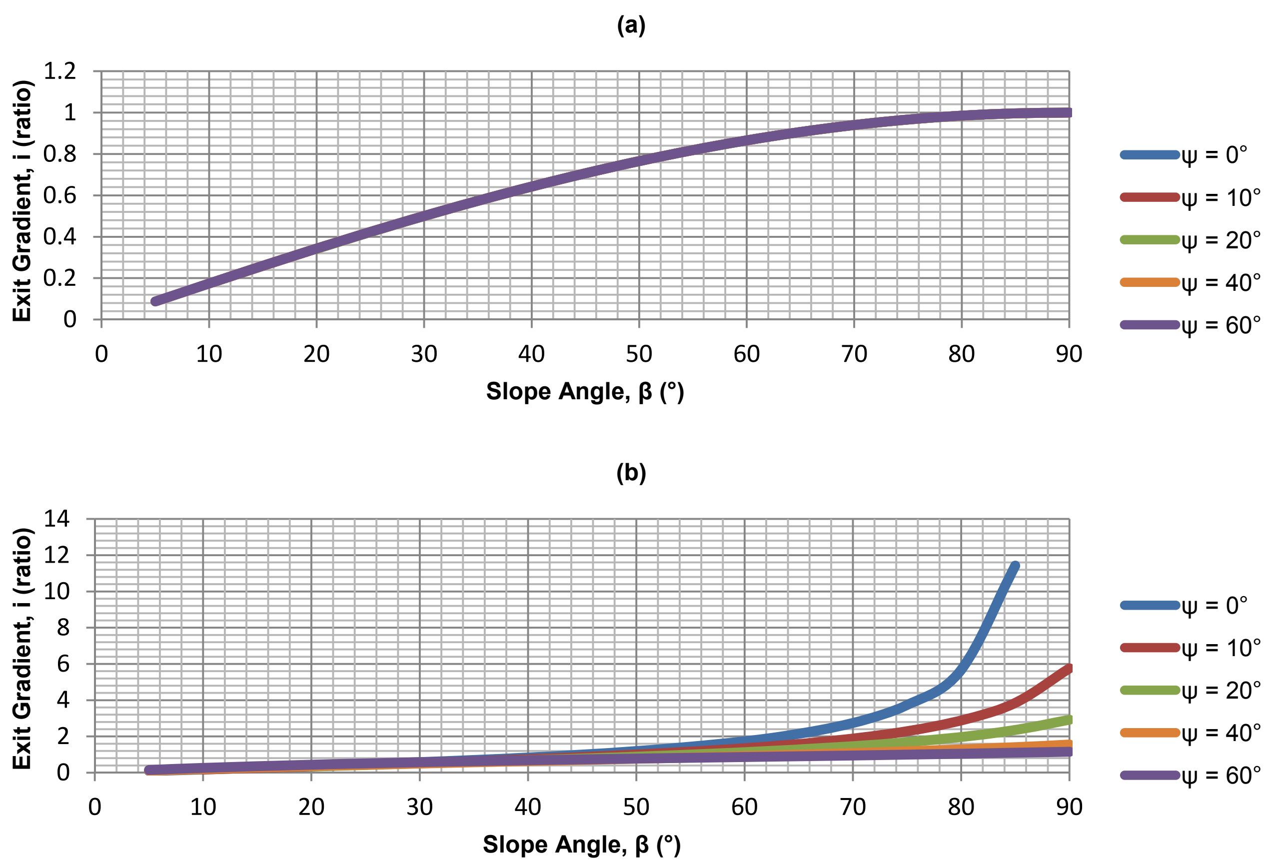

Fig 1. Plots of exit gradient (i) versus slope angle (β) for various values of seepage flow inclination (ψ) considering (a) parallel flow and (b) horizontal flow (after Ghiassian & Ghareh 2008).

Shear Failure due to Seepage Forces

Zhang & Chang (2012) have identified that when soils fail due to seepage forces, large deformations occur and the soil fails due to changes in stress and reduction in shear strength due to the loss of fine particles. For soil slopes where the soil at the slope face is loaded asymmetrically (σ_2 and σ_3 are not equal due to a lack of confinement in the face), the hydraulic gradient to cause failure will be much lower than the theoretical gradient identified by Zhang and Chang.

Zhang and Chang have, through laboratory experiments, quantified the onset of suffusion, suffosion, and piping (soil shear failure). It should be noted that the research by Zhang and Chang is in regards to dam filter stability, and their results will not directly apply to seepage-induced soil slope instability. However, they do quantify the loss of shear strength due to erosion.

Another parameter that likely should be considered in the analysis of seepage-induced slope instability is buoyancy. Wörman (1993) shows that buoyancy acts normal to the slope face just behind the slope face. This acts as a destabilizing force more so than if buoyancy is only considered to act vertically (e.g. buoyant unit weight). The shear strength of a soil will be further reduced near the slope face due to buoyancy.

Discussion

Internal Stability of Natural Slope Sediments

The work by Kenney & Lau (1985), and later furthered by Skempton & Brogan (1994), Li & Fannin (2008b), and Moffat et al. (2011), leads to a quantification for the instability of granular soils. In particular, Skempton & Brogan (1994) and Moffat et al. (2011) conducted laboratory tests on soils that may provide insight to the internal stability of natural slope sediments.

Skempton & Brogan (1994) present laboratory results from tests conducted on two soils: 1) sandy gravel (85 percent gravel and 15 percent sand) having a porosity (n) of 34 percent and coefficient of uniformity (C_u) of 24, and 2) sandy gravel (85 percent gravel and 15 percent sand) having a porosity (n) of 37 percent (moderately loose packing) and coefficient of uniformity (C_u) of 10. Skempton & Brogan (1994) observed that the first soil was highly unstable and the second was also unstable.

Moffat et al. (2011) also presented the results of laboratory testing on soils tested. Moffat et al. tested three soils: 1) gravelly silty sand (20% and 30% finer than 0.075 mm), 2) gravel and sand (no fines), and 3) gravel and sand, trace fines. Moffat et al. considered each of the three soils to be unstable for the conditions under which they were tested. Of note is the fact that of the four soil samples tested by Moffat et al., all were consolidated to varying stresses ranging from 25 to 175 kPa. These stresses are comparable to those that could be expected to be found in the soils just behind the face of a slope.

It should be acknowledged that Li & Fannin (2012) indicate that a soil's susceptibility to internal instability is independent of its gradation and actually controlled by the amount of stress carried by the primary fabric. The hydromechanical envelope suggested by Li & Fannin takes the form Equation 5.

[5] $$i = \frac{\alpha}{1-0.5\alpha}\left(\bar{\sigma}'_{vm}\frac{0.5\gamma'}{\gamma_w}\right)$$

where i is the gradient at the onset of internal instability, α is the ratio of stress in the finer fraction (α = 1 for the case of equal stress in coarse and fine fraction, and α = 0 for the case where no stress is carried by the finer fraction), $\bar{\sigma}'_{vm}$ is the normalized mean vertical stress, γ' is the buoyant unit weight of the soil, and γ_w is the unit weight of water.

Based on the hydromechanical envelope put forth by Li and Fannin (Equation 5), only the vertical effective stress is considered other than the unit weight of the soil.

The description of the Gilley Brothers Gravel Pit provided by Armstrong (1984) indicates that the silty gravel and sand being mined could have been internally unstable when compared to the results reported by Moffat et al, with the glaciofluvial deltaic sediments providing an impermeable layer to retain the eroded soil until the impermeable layer was breached, allowing the silty gravel and sand to fail, with the potential mode of failure discussed in the next section.

It is possible that progressive erosion of the slope face behind the glaciofluvial deltaic sediments, leading to the failure of some 60,000 m^3 of soil (Armstrong 1984), could have occurred, however, the author postulates that the collapse of the silty gravel and sand occurred due to the long term erosion of the finer fraction (silt and likely some portion of sand). Evidence presented by Moffat et al. indicates that mobilization of both sand and silt within a gravel is likely to occur. The loss of the glaciofluvial deltaic sediments, which were likely providing both an impermeable barrier to the movement of groundwater and a confining layer for the silty gravel and sand, would have caused the retrogressive collapse of the silty gravel and sand.

The collapse of the silty gravel and sand, along with a very high water content, would have lead to the mobilization of the soil. Observations that upwards of 60,000 m^3 of material flowed out of the slope indicate that the soil had loss all strength (liquefaction). Collapse of the silty gravel and sand structure (contractive behaviour) would lead to liquefaction (see below for further discussion).

Critical State Solution to Seepage-induced Slope Instability

The mechanics of seepage-induced failure can be explained by utilizing Critical State Soil Mechanics (Schofield & Wroth 1968; Muir Wood 1990; Budhu 2010). Muir Wood (2007) observes that soil behaviour is best understood in the context of density changes, such that stress states and density are considered together. Muir Wood (2007) further observes that a unique density and stress level relationship emerge at the eventual critical state. Critical state is a condition reached at large strain (Muir Wood 2007), which implies that for a granular soil that is only elastic at small strains, critical state is also a condition of plastic deformation.

In short, the erosion of the finer fraction out of the courser fraction of a granular soil leads an increase in void ratio (e) and subsequently a reduction in the confining stress (Muir Wood 2007). Muir Wood observes that as fines are removed, the soil will begin to feel looser and soil that becomes looser will tend to collapse (the available peak strength will fall). Failure can be progressive over a short time, or as described by Cavers (2003), a sudden release of material.

Fig. 2 presents a conceptual model of the critical state for a granular soil. Fig. 2a illustrates the relationship between void ratio (e) and mean effective stress (p', which depending on the constitutive model can be defined in a number of ways, but is a average of σ'_1, σ'_2. and σ'_3). This relationship as plotted in Fig. 2 is known as the critical state line (CSL) and when, at a particular p' value a soil reaches a value of e on the CSL, failure can occur. Soils dense of critical lie below the CSL and soils loose of critical lie above the CSL.

A corresponding plot, as shown in Fig. 3, illustrates the relationship between the deviator stress ($q = \Delta\sigma'_1 - \Delta\sigma'_3$) and p'. As can be seen, assuming a Mohr-Coulomb failure criterion model, the CSL intersects the Mohr circle at its apex. Any combination of q and p' that lies inside the Mohr circle, and below the CSL, means the soil undergoes elastic strains. Plastic strains occur when points are located outside of the Mohr circle, but lying below the CSL. Once points come into contact with the CSL, failure occurs.

However, it should be noted that failure will not occur until the soil yields (Budhu 2010). A soil must first reach the envelope bounded by the Mohr circle and then come to rest on the CSL, as shown in Fig. 3. As sands are not expected to withstand large strains before undergoing plastic deformation, it is highly likely that very small increments of strain will lead to the collapse of a soil that has had its finer fraction removed.

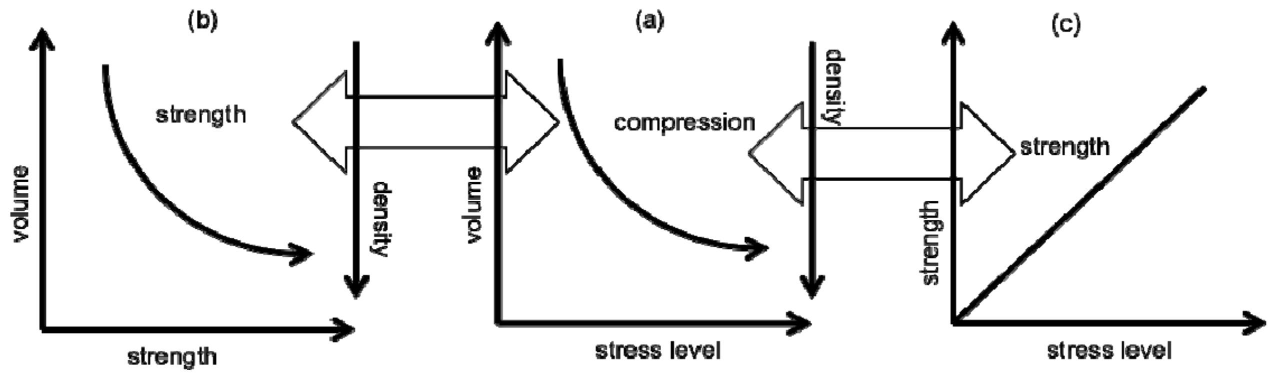

Fig 2. Critical state soil mechanics in its weak form: a) stress level and density, b) strength and density, and c) strength (after Muir Wood 2007).

A decrease in σ'_3, or an increase in e will cause the Mohr circle to decrease in size, resulting in a decrease in available peak strength (Muir Wood 2007). This indicates that as the finer fraction of the granular soil is removed, the remaining soil structure moves towards a point of collapse due to the change in σ'_3. As discussed above, the formation of a natural filter within the soil likely acts to maintain a minimum level of confinement to resist the remaining weakened soil structure from flowing out of the slope. The loss of this filter zone ultimately leads to collapse of the slope.

For a slope that has undergone seepage-induced erosion of the finer fraction, the danger then becomes that the soil, now having achieved a high void ratio, is more susceptible to a triggering event that reduces, or fully removes the remaining confinement (Muir Wood 2007). This could be the loss of the naturally formed filter zone, a seismic event, or an increase in pore water pressure due to intense surface water infiltration.

As described by various authors (Armstrong 1984; Hungr & Smith 1985; Evans & Savigny 1994), the Gilley Bothers Gravel Pit failure was the result of excavation into a slope. The excavation likely disturbed the natural formed filter zone, leading to the immediate collapse of the remaining soil structure that had its finer fraction removed by erosion.

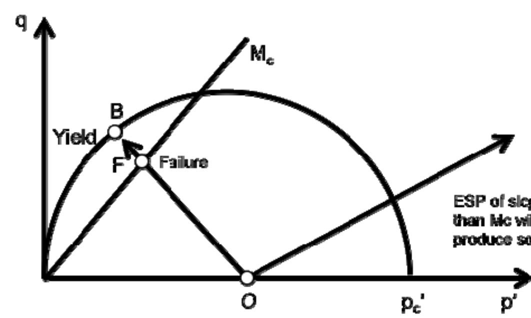

Fig 3. Conceptual critical state for a granular material. Failure would follow the O-B-F line, as the material needs to yield before it can fail (after Budhu 2010).

The blow-off failures described by Cavers (2003) are also likely the result of loss of the naturally formed filter zone, but due in part to the build-up of pore pressures behind the filter zone. Cavers' observation that slope face bulging can occur prior to a blowoff failure indicates that some tensile strength is present in the face of the slope, but is likely due in large part to vegetation root systems. However, it is likely that the same vegetation root systems are favourable to the formation of filter zones, as the roots will promote the clogging of pores with finer fraction soil particles.

Mobilization of Eroded Soil

Upon the failure of a slope due to groundwater seepage erosion, a mass of soil is deposited on the slope directly below the point of failure. The sudden release of groundwater, from the pervious layer, results in a soil mass that is heavily laden with water. An increased void ratio, due to the removal of the finer fraction, likely results in a significantly higher volume of stored water. The soil mass at the toe of the slope failure may "flow" some distance. Hungr et al. (2013) describe debris flow slides as follows:

> "Very rapid to extremely rapid flow of sorted or unsorted saturated granular material on moderate slopes, involving excess pore-pressure or liquefaction of material originating from the landslide source. Usually originates as a multiple retrogressive failure. Often occurring under water".

As described above, the collapse of the soil due to an increase in void ratio, and subsequent decrease in σ'_3, leads to contraction of the soil. Contraction of a saturated, granular soil is synonymous with liquefaction (Muir Wood 1990; Budhu 2010). Cavers (2003) also observed that debris from a seepage-induced slope failure can result in a debris flow slide.

The level of liquefaction of a soil is determined by the void ratio at the time of failure. Soils with a very high void ratio (very loose) are likely to fully liquefy, whereas soils with a moderate void ratio at the time of failure may undergo some contraction, but then begin to dilate, which ends the process of liquefaction (Li & Dafalias 2011). This will govern the movement of soil following the initial failure of the slope.

The inclination of the ground directly below the location of the slope failure will govern, along with the flowability of the debris, the runout area of the debris. Even when level ground is present, as is likely the case with the Gilley Brothers Gravel Pit failure (Armstrong 1984; Hungr & Smith 1985; Evans & Savigny 1994), the runout zone for the debris can be extensive if the full soil mass is fully liquefied. When only partial cyclic mobility (partial liquefaction) occurs, before a soil begins to dilate (Li & Dafalias 2011), the soil is likely not to achieve the same level of runout.

In the case of seepage induced cohesionless soil slope instability along the foreshore, in many cases the debris from the failure will be eroded from the toe of the slope by currents (Hagerty 1991a). This can further obscure the mechanisms of the failure.

Conclusion

As observed by Terzaghi & Peck (1948), seepage-induced slope instability is a complex phenomenon, which has not been quantified, and likely not even fully understood. The purpose of this work has been to at least narrow the gap between engineering practice and the literature available on seepage-induced slope instability.

A wide range of factors affect a slope's ability to resist seepage-induced instability, with a number of factors still yet to be fully defined. Furthermore, the available numerical codes are not currently suitable for modelling seepage-induced instability in slopes.

The development of new numerical methods to model seepage-induced erosion will provide practitioners with a means to better identify slopes susceptible to seepage-induced erosion, particularly in urban areas encroaching on sloping topography.

This work provides a possible solution to describe seepage-induced slope instability utilizing Critical State Soil Mechanics. Further work is required to determine if in fact this solution is valid for natural slopes.

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