Analyses for the stability of potash tailings piles

DELWYN G. FREDLUND
Department of Civil Engineering, University of Saskatchewan, Saskatoon, Sask., Canada S7N 0W0
ZAI MING ZHANG
Beijing Geotechnical Institute, Beijing, China
AND
KAREN MACDONALD[1]
Department of Civil Engineering, Queen's University, Kingston, Ont., Canada K7L 3N6

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The stability of potash tailings piles is investigated using a pore-water pressure generation and dissipation model together with a limit equilibrium analysis. It is found that a shallow toe failure mode is generally the most applicable and that the stability may be influenced by pore-water pressure migration below the pile. It is suggested that field studies would be useful in evaluating stability in the toe region of the pile.

Key words: potash tailings, slope stability, pore pressure dissipation, solutioning.

La stabilité des piles de résidus de potasse est étudiée au moyen d'un modèle de génération et de dissipation des pressions interstitielles et d'une analyse à l'équilibre limite. L'on trouve que le mode de rupture peu profonde en pied de talus est généralement le mode qui s'applique le mieux, et que la stabilité peut être influencée par la migration de la pression interstitielle sous la pile. Il est suggéré que des études sur le terrain seraient utiles pour évaluer la stabilité en pied de talus.

Mots clés : résidus de potasse, stabilité de talus, dissipation de pression interstitielle, solution.

[Traduit par la rédaction]

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Introduction

Potash is an international commodity which the Province of Saskatchewan has developed to the point where it now makes up about 26% of the world market. This share ranks Canada in second place in world potash production (Barry 1989). Nine of the ten potash mines in production throughout south-central and south-eastern Saskatchewan use conventional underground mining techniques to recover the ore and bring it to the surface for refining. Each year, some 28 Mt of waste tailings associated with the potash mining operation are piled on the ground surface.

Tailings piles are built using a spigotting technique. The waste products are delivered to the pile in the form of a slurry. The coarse fraction is deposited near the exit of the spigot while the finer fraction settles out as the slurry proceeds down the gradual slope (i.e., of a few degrees) to the brine pond. The spigot can be moved around the discharge area, and low tailings dykes are built using a crawler tractor with a dozer so as to confine the discharge to cells.

Figure 1 shows a picture of a typical tailings pile built using the procedure described above and a typical cross section is shown in Fig. 2. The slope of the steep portion of the pile generally ranges from 1:1.5 (i.e., 33.7°) to 1:1.25 (i.e., 38.7°). An angle of 35° is a typical value, and it was used for the purposes of this analytical study. The entire tailings area, including the tailings pile and the brine pond, is enclosed within an earthfill dyke to prevent contamination of surrounding agricultural land (Fig. 3). At present, the tailings piles are generally in the order of 50 m in height. It is anticipated that the salt waste will increase from 250 Mt today to some 14 000 to 28 000 Mt in the future (Hart 1989). The piles in turn may some day become as high as 100–200 m.

The long-term stability of these piles both during mining and after decommissioning is an issue of concern to the potash industry and also of interest to geotechnical engineers because of the special features associated with the waste piles.

Some of the main aspects of interest relative to the tailings piles involve: (*i*) an extremely strong and basically cohesionless material (tailings) placed over a considerably weaker material (foundation soil), (*ii*) a relatively steep slope angle of substantial height, (*iii*) the possibility of pore-water pressure migration with time which could affect the stability of the pile, and (*iv*) the possibility of solutioning within the pile (Zhang et al. 1991).

This paper illustrates how the slope-stability problem can be analysed using a pore-pressure generation and dissipation model in conjunction with a limit equilibrium analysis to provide an ongoing measure of the stability of the potash tailings piles. The suggested parametric study can be utilized as a general approach when evaluating the stability of potash tailings piles.

Analysis procedure and theory

It is important to identify potential modes of failure of the potash tailings piles, taking into account the significant differences in mechanical properties between the

[1]Present name and address: Karen Lewall, 314 MacDonald Avenue, Belleville, Ont., Canada K8N 3Z2.

FIG. 1. Tailings pile at the PCS potash mine at Lanigan, Saskatchewan.

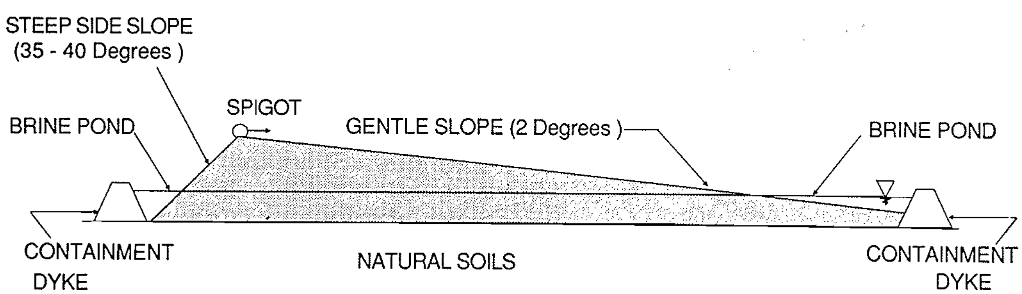

FIG. 2. Typical cross section of a tailings pile.

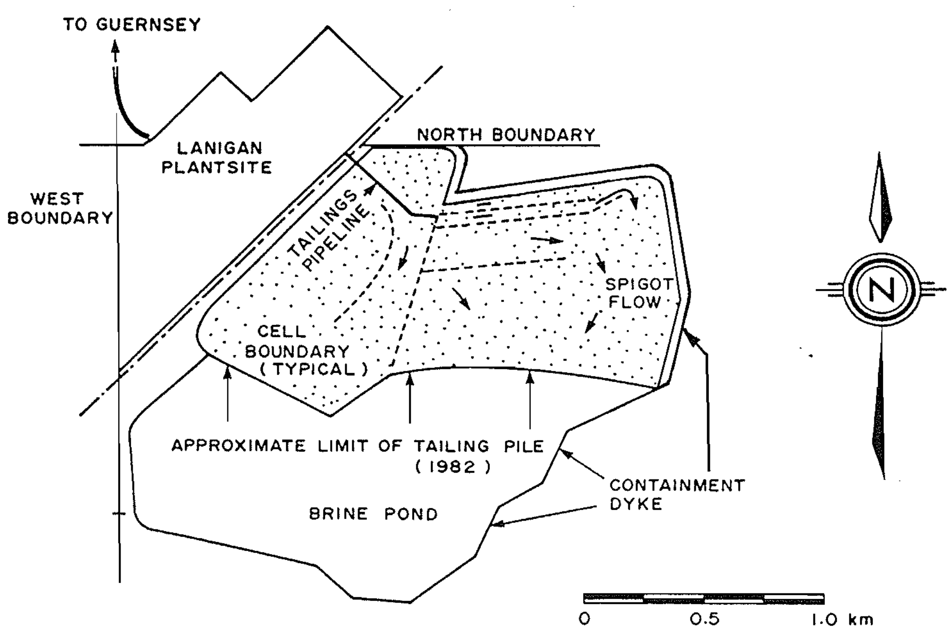

FIG. 3. Aerial layout of the PCS waste-containment facility at Lanigan, Saskatchewan.

Tailings Pile Loads Applied to Foundation Soils

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LINEAR STRESS ANALYSIS WITH PORE-PRESSURE GENERATION

INPUT
- Boundary Conditions
- Elastic Properties, E, μ
- Pore-Pressure Parameters, A, B

OUTPUT
- Distribution of Generated Excess Pore-Water Pressures

↓

UNCOUPLED, SATURATED/UNSATURATED SEEPAGE MODELLING

INPUT
- Geometric Boundary Conditions
- Initial Excess Pore-Water Pressures
- Soil Properties – Storage Coefficient, m_2^w
  - Permeability Versus Suction Curve
- Time Sequencing

OUTPUT
- Pore-Water Pressures at Various Elapsed Times

↓

LIMIT EQUILIBRIUM, SLOPE-STABILITY ANALYSIS

INPUT
- Geometric Boundary Conditions
- Soil Properties, γ, c', ϕ'
- Analysis Parameters: Interslice Force Function
- Pore-Water Pressures at Various Elapsed Times

OUTPUT
- Factor of Safety at Various Elapsed Times

FIG. 4. Analytical procedures for pore-pressure migration analysis.

tailings and the foundation soils, the possibility of thin weak layers (i.e., slimes) existing within the pile, and different pore-pressure dissipation patterns. Several potential modes of failure are defined and analysed parametrically. Three major factors, namely the strength properties of the foundation soil, the height of the pile, and the pore-water pressures, are studied for their influence on the computed factor of safety.

The stress field induced by the load of the tailings pile applied to the foundation soils was solved using a linear, elastic, finite element stress analysis program. The resulting excess pore-water pressures were obtained using Skempton's (1954) pore-pressure generation equation. Three software computer programs were used in performing the analysis, namely, PC-SIGMA to perform the linear stress analysis with pore-pressure generation, PC-SEEP to perform the seepage modelling, and PC-SLOPE to perform the limit equilibrium analysis for the factors of safety.

The problem of pore-pressure migration was modelled using a finite element program based on a saturated–unsaturated pore-water pressure dissipation type of analysis

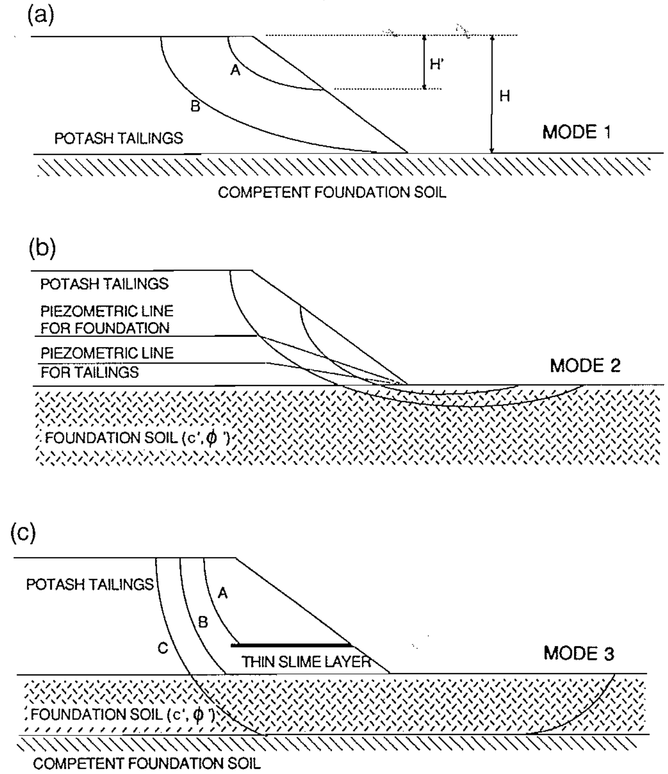

FIG. 5. Modes of failure for the slope-stability study (*a*) Potential circular slip surfaces within the tailings. (*b*) Potential circular slip surfaces penetrating the foundation soil. (*c*) Composite slip surfaces into the foundation soils.

(Fredlund 1981). Using a two-dimensional approach, the sum of the rates of change of flow in the x and y directions plus the externally applied flux is equal to the rate of change of storage with respect to time. This equality, along with the constitutive relationship for water storage, yields a governing equation for the migration of generated pore-water pressures (Wong and Barbour 1987; Wong et al. 1988; Lam et al. 1987). The governing equation is uncoupled from stress equilibrium:

$$[1] \quad \frac{\partial}{\partial x}\left(k_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(k_y \frac{\partial h}{\partial y}\right) + \Delta q = \rho_w g m_2^w \frac{\partial h}{\partial t}$$

where h is total hydraulic head, k_x and k_y are coefficients of permeability with respect to x and y directions, Δq is applied external boundary flow, m_2^w is the slope of the relationship between water content and matric suction (or pore-water pressure), ρ_w is density of water, and g is gravitational constant. The m_2^w variable is commonly referred to as the water-storage coefficient. Although the storage coefficient is somewhat stress-level dependent, it can be assumed to be a constant for the analyses in this study. For the saturated soil, the m_2^w coefficient is equivalent to the conventional coefficient of volume change m_v.

The pore-water pressures obtained from the above analysis were used in a limit equilibrium, stability analysis to compute the factor of safety. The general limit equilibrium (GLE) method was used in this study. The GLE method satisfies all equilibrium conditions and can handle any shape of slip surface (Fredlund et al. 1981). This method utilizes the same assumptions and elements of physics as the Morgenstern–Price method.

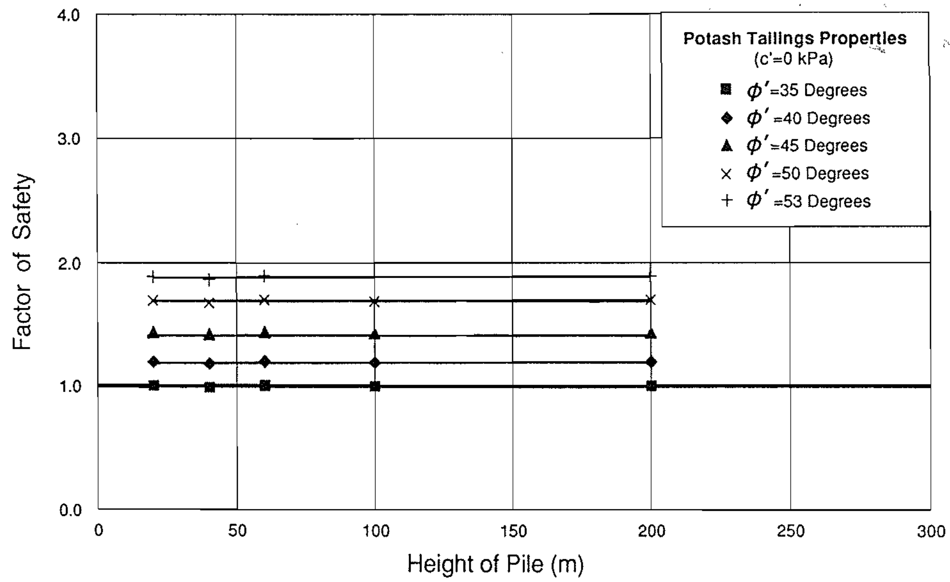

FIG. 6. Factor of safety vs. height of pile for no cohesion in tailings material (toe failure). Dry case with a firm foundation.

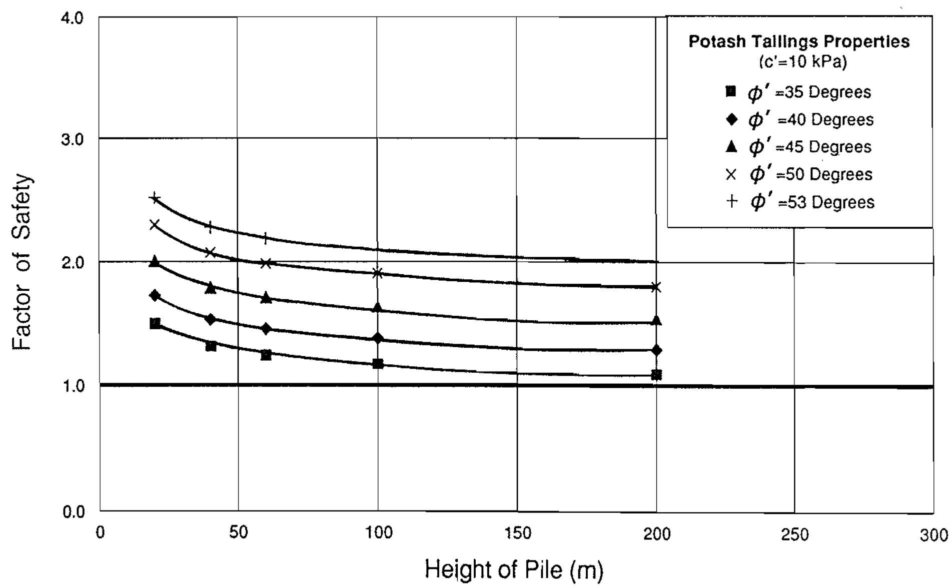

FIG. 7. Factor of safety vs. height of pile for 10 kPa cohesion in tailings material (toe failure). Dry case with a firm foundation.

The general procedures for the pore-water pressure migration analysis are outlined in Fig. 4. The degree of sophistication in the analytical procedure was kept at a level commensurate with the understanding of the soil properties and boundary conditions for the problem. For this reason, the authors felt it was unnecessary to utilize a coupled seepage and stress analysis.

Fundamental material properties

Properties of the potash tailings

Extensive research has been conducted towards a better understanding of the engineering properties of the potash tailings (Pufahl 1983; Pufahl et al. 1985; footnote 2). The main results of the research are summarized in this section.

The specific gravity (relative density) of the individual particles ranges from 2.10 to 2.19. The *in situ* density is typically about 1.6 Mg/m^3 near the surface of a pile and may increase to 1.95 Mg/m^3 near the base. Dry sieving of the salt particles indicates that the material can be classified as a fine gravel to a coarse sand. There are traces of insolubles located within the pile which are referred to as slimes. The insolubles contain silt- and clay-size particles. The saturated coefficient of permeability from *in situ* tests was found to be in the range of 1.5–4.0 $\times$ 10^{-5} m/s (Wong and Barbour 1987). These results are indicative of a specific site and in general the coefficients of permeability would vary over a wider range.

The strength characteristics under short-term loading have been evaluated using both direct shear and triaxial

[2]D.G. Fredlund 1991. Long term stability of potash tails on soft foundations. Paper presented at 2nd International Conference of Potash Technology, KALI '91, Hamburg, Germany.

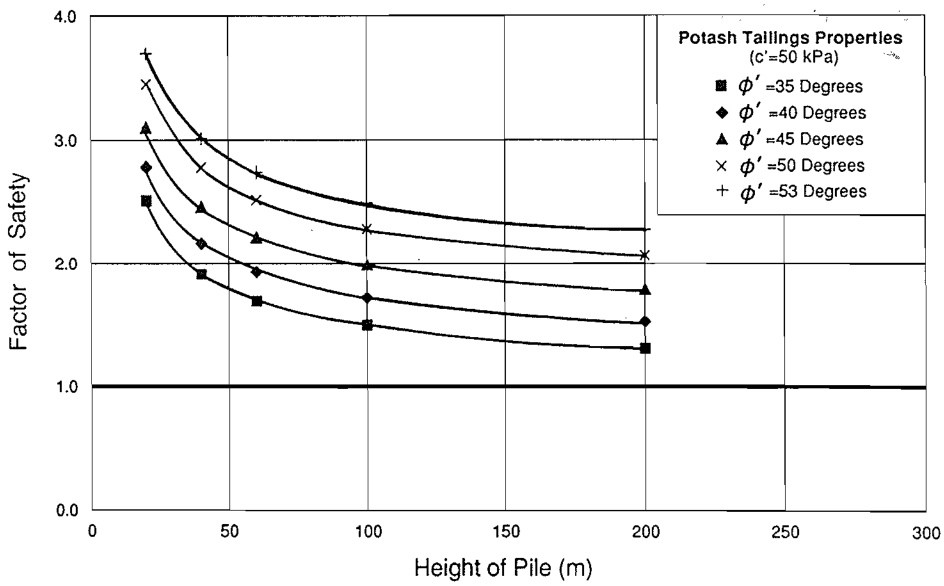

FIG. 8. Factor of safety vs. height of pile for 50 kPa cohesion in tailings material (toe failure). Dry case with a firm foundation.

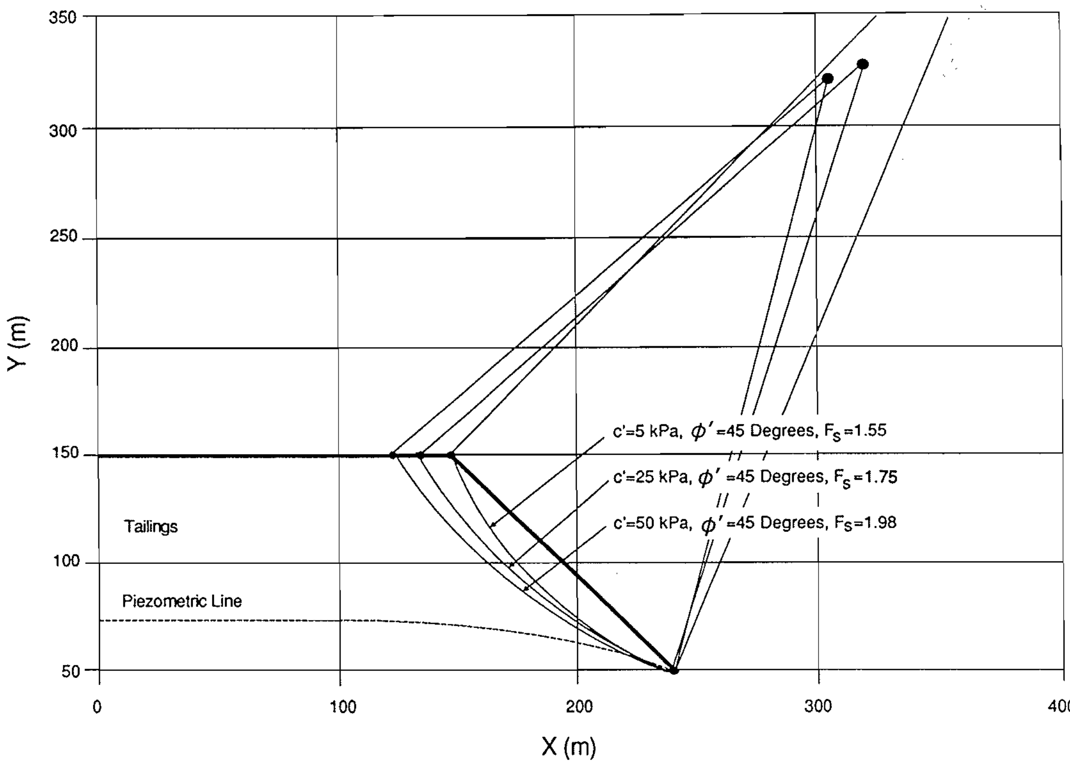

FIG. 9. Location of toe failure slip surfaces. F_s, factor of safety. Unit weight (tailings) = 19.0 kN/m^3.

compression tests. The angle of internal friction ϕ' from direct shear tests generally ranged from 39 to 45°, sometimes going as high as 56 degrees. The cohesion intercept c' was essentially zero. Shear strength parameters from triaxial tests showed an effective angle of internal friction ϕ' ranging from 49 to 55° and an effective cohesion c' ranging from 55 to 270 kPa (Chiu and Fredlund 1986). Direct shear test results on the insolubles show that the peak friction angle of this material was 30° and that the ultimate friction angle was 23°. The existence of thin layers of insolubles can thus have an effect on the stability of a tailings pile.

Geology and properties of foundation soils

In the potash mining area of Saskatchewan, the Precambrian basement is overlain by a series of relatively flat-lying sedimentary strata of up to 3000 m in thickness (Christopher et al. 1971). The upper veneer of sediments, up to 100 m in thickness, was deposited during the glacial periods. It is composed largely of various tills with sands and gravels between the till units. Locally these glacial sediments are overlain by different surficial drifts which may vary from site to site.

For the parametric study reported in this paper, three typical foundation soil conditions with the following

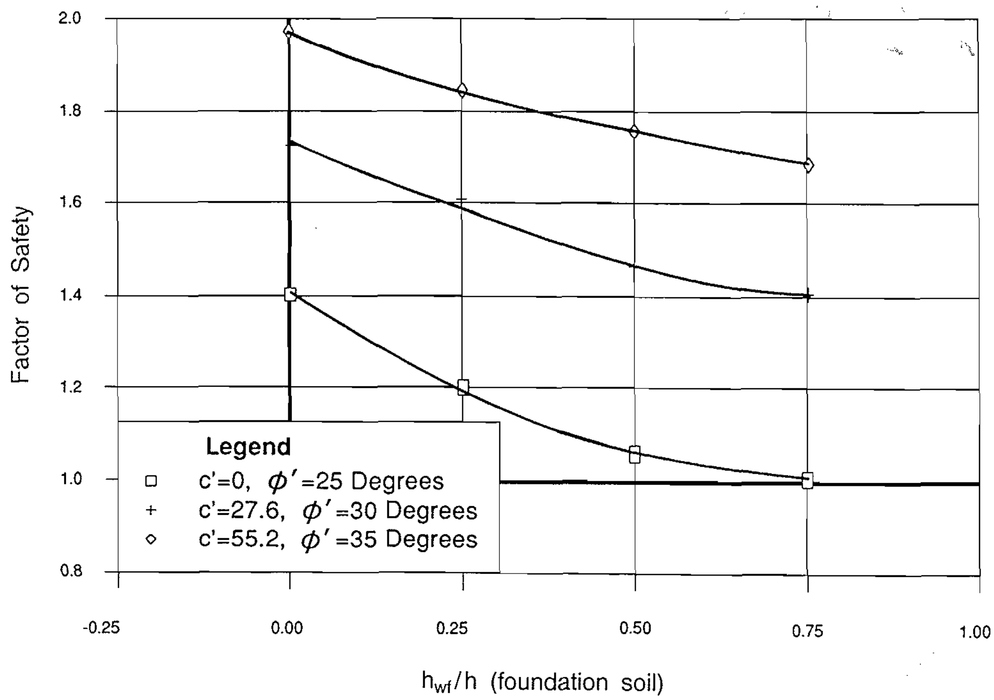

FIG. 10. Long-term stability for 60-m-high pile. Tailings water level $h_w = 0.25H$. h_{wf}, height of piezometric line for the foundation soil, relative to the base of the tailings pile.

strength parameters were used: (*i*) S1, $c' = 0.0$ kPa, $\phi' = 25°$; (*ii*) S2, $c' = 29.6$ kPa, $\phi' = 30°$; and (*iii*) S3, $c' = 55.2$ kPa, $\phi' = 35°$. These typical soils would range from a soft silty clay to a stiff clayey sand.

Long-term stability of tailings piles of extended heights

The factor of safety of a potash tailings pile is a function of the pore-water pressure distribution in the tailings and the foundation soils, the strength properties of the tailings and the foundation soils, and the height of the piles. To examine the influence of these factors on the factor of safety of the tailings piles, three potential modes of failure were studied (Fig. 5). A range of soil and tailings shear strength parameters were assumed. Assumptions were made concerning the pore-water pressures for the different modes of failure.

Mode 1

Mode 1 is assumed to be a slip surface passing through the toe of the slope (Fig. 5*a*). Any slip surface above the toe can be viewed as a special toe failure corresponding to a pile of reduced height H' (case A in Fig. 5*a*).

The curves for zero cohesion (Fig. 6) show that the factors of safety remain unchanged as the pile height increases. Cohesion has a significant effect on the factors of safety for piles of low height. As the height of the pile increases, the effect of cohesion becomes less pronounced (Figs. 7 and 8).

Figures 6–8 show that a toe failure would not likely be observed in a potash tailings pile as long as the pore-water pressures are low. If there is a water table within the pile, a shallow slip surface will likely fall in the proximity of the piezometric line, and therefore the pore-water pressures are low, having little effect on stability (Fig. 9).

Mode 2

Mode 2 constitutes a slip surface passing through the base. The slip surfaces penetrate the foundation soil below the toe of the pile (Fig. 5*b*). Pore-water pressure conditions for the potash tailings were defined using a piezometric line equal to $0.25H$ of the pile. The piezometric lines for the foundation soils were assumed to vary from the ground surface (i.e., from $0.0H$), to a height of $0.75H$. All piezometric lines for the potash tailings were assumed to drop to zero at the toe of the pile. This assumption is in keeping with piezometric observations on a number of potash tailings piles in Saskatchewan.

The analytical results for long-term stability, base-failure cases are shown in Figs. 10–12 for pile heights of 60, 100, and 200 m, respectively. And the results are summarized in Table 1.

Based on the assumption that the piezometric line for the potash tailings drops to zero at the toe of a pile, a base failure is possible only if the foundation soil is weak (i.e., $c' = 0$, $\phi' \le 25°$) and if there are relatively high piezometric conditions (e.g., tailings water level $h_w \ge 0.75H$ for piles 100 m in height and $h_w \ge 0.25H$ for piles 200 m in height, Table 1). The critical slip surfaces for the base-failure mode were relatively shallow, no more than about 15 m below the ground surface.

Mode 3

Mode 3 constitutes a composite slip surface case, where part of the slip surface follows the top of a hard surface (Fig. 5*c*). This can occur in a layered system where there are relatively weak zones. The weak zones may either be in the pile tailings or in the foundation soil.

In a previous study, Chiu and Fredlund (1986) indicated that for the case with a thin weak layer immedi-

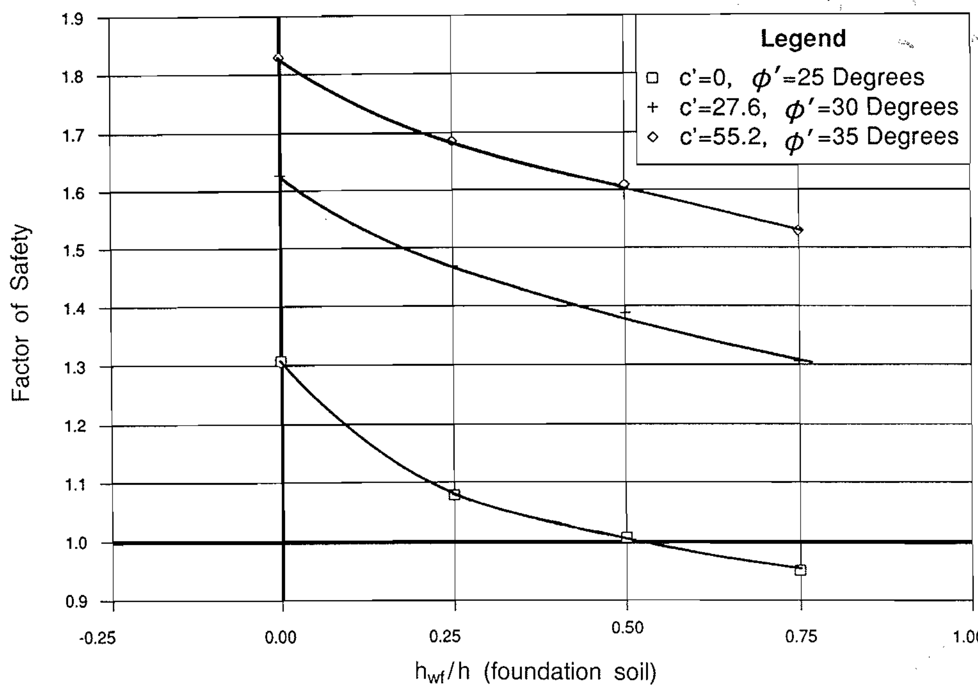

FIG. 11. Long-term stability for 100-m-high pile. Tailings water level h_w = 0.25H. h_{wf}, height of piezometric line for the foundation soil, relative to the base of the tailings pile.

TABLE 1. Summary of factors of safefy for long-term cases (tailings water level = h_w = 0.25H)

Foundation soil	c' (kPa)	ϕ' (deg.)	Piezometric line	F_S at pile heights (m)[a] indicated: <60	60	100	200
S1	0	25	0.00H	●	●	●	●
			0.25H	●	●	●	○
			0.50H	●	●	●	○
			0.75H	●	●	○	○
S2	27.6	30	0.00H	●	●	●	●
			0.25H	●	●	●	●
			0.50H	●	●	●	●
			0.75H	●	●	●	●
S3	55.2	35	0.00H	●	●	●	●
			0.25H	●	●	●	●
			0.50H	●	●	●	●
			0.75H	●	●	●	●

[a]●, $F_S \geq 1$; ○, $F_S < 1$.

ately beneath the tailings pile, the factor of safety was close to that for the case of a toe failure. Where a thin layer is located within the foundation profile at a shallow depth (e.g., less than 15 m), the resulting factors of safety would be similar to the case of a base failure. A deep-seated weak layer should not prove to be a problem in terms of slope stability for high piles, since the portion of the slip surface passing through the tailings which possesses high strength properties, would gain sufficient resistance to prevent the occurrence of a composite failure (Zhang et al. 1991). Only if the deeper layer is extremely weak would the slip surface be composite. On the basis of the above findings, only toe failures and base failures were given further considerations.

Comparison modes of failure

To establish a relationship between factors of safety for base failures and toe failures, a factor of safety coefficient Ψ was defined as follows:

$$[2] \quad \Psi = \frac{F_b}{F_t}$$

where F_b is the minimum factor of safety for a base failure and F_t is the minimum factor of safety for a toe failure, all other conditions remaining the same. The factor of safety coefficients Ψ versus the height of the pile for three foundation-soil conditions are shown in Figs. 13–15 and summarized in Table 2. It is obvious that for most of the cases studied, a toe failure would dominate. A base

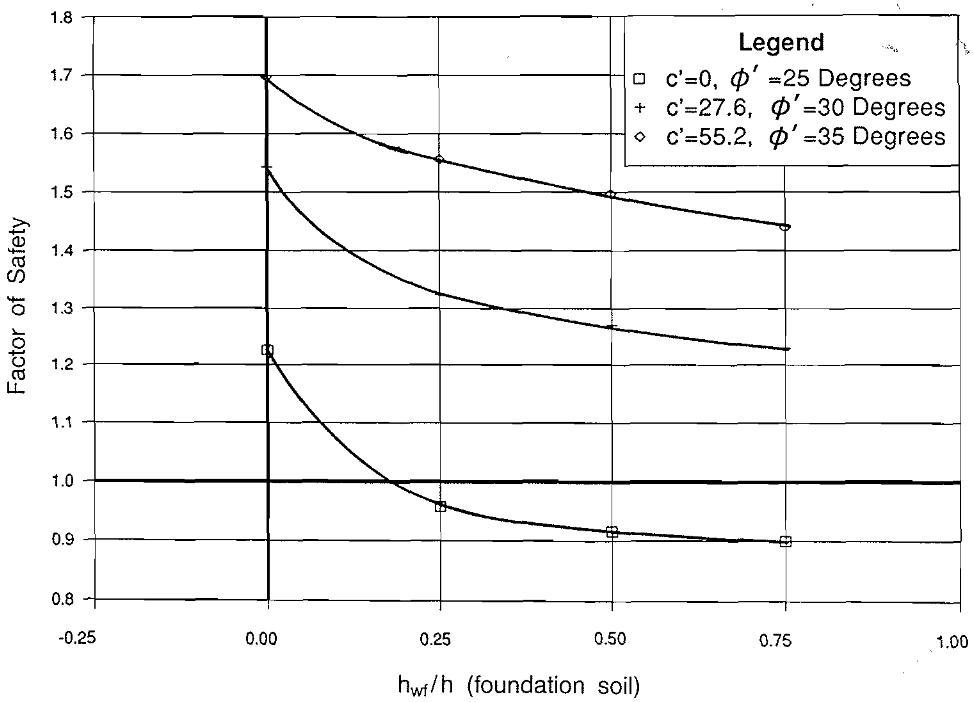

FIG. 12. Long-term stability for 200-m-high pile. Tailings water level $h_w = 0.25H$. h_{wf}, height of piezometric line for the foundation soil, relative to the base of the tailings pile.

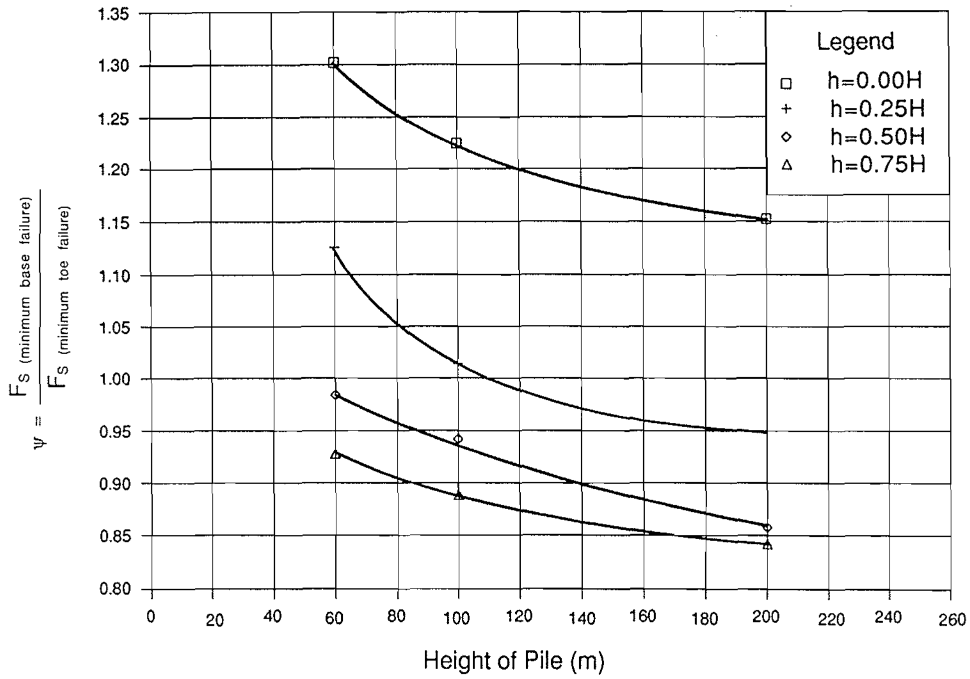

FIG. 13. Factor of safety coefficient Ψ vs. height of pile for the long-term stability case 1. $c' = 0$ kPa; $\phi' = 25°$.

failure could become a possibility only if the foundation soil is weak and the water table is higher than $0.25H$.

Effect of pore-pressure migration on slope stability

In recent years, embankment movements have been reported which are associated with a phenomenon known as "pore-pressure migration." The process involves the buildup of high pore-water pressure below an embankment because of the placement of fill, with subsequent movements occurring as a result of pore-water pressure buildup towards the toe area of the slope. The increase in pore-water pressure will substantially reduce the factor of safety. The potential for pore-pressure migration below potash tailings piles is of interest relative to the study of the stability of a tailings pile.

A 50-m-high pile of tailings was assumed. The Young's modulus for the foundation soil was assumed to be 1 MPa, and the Poisson's ratio was assumed to be 0.3. The B pore-pressure parameter was assumed to be 1.0, and the A pore-pressure parameter was assumed to be

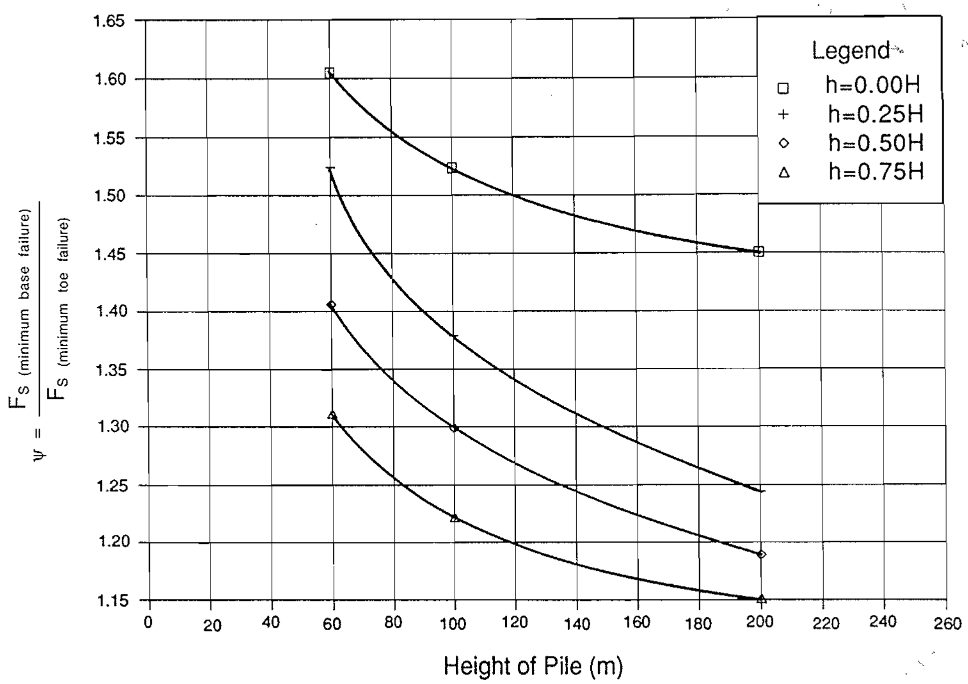

FIG. 14. Factor of safety coefficient Ψ vs. height of pile for the long-term stability case 2. $c' = 27.6$ kPa; $\phi' = 30°$.

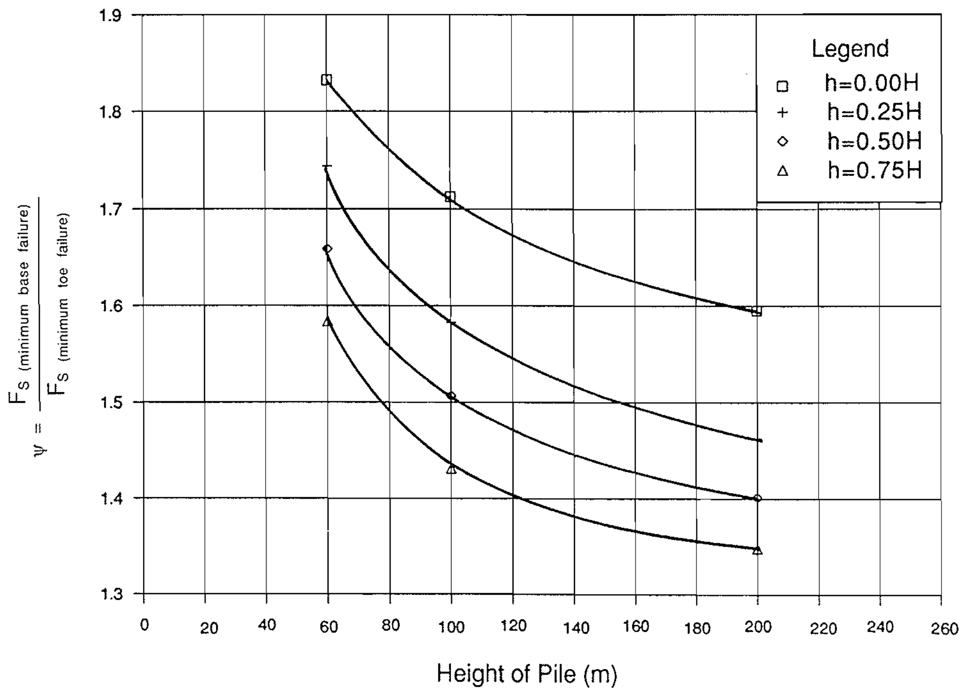

FIG. 15. Factor of safety coefficient Ψ vs. height of pile for the long-term stability case 3. $c' = 55.2$ kPa; $\phi' = 35°$.

0.33 for the foundation soil. The pore-pressure generation and dissipation process within the foundation material, due to the 50-m-high pile, was simulated using the stress analysis program with pore-pressure generation and the saturated–unsaturated seepage program for modelling the change of pore-water pressures with time (see Fig. 4). Six tailings and foundation-soil profiles were studied as shown in Table 3, with the results illustrated in Figs. 16–21. The water storage and flow properties associated with each of the soil strata are summarized in Table 4. The term sengenite in Table 3 refers to a hard, essentially impermeable material existing at the bottom of the tailings pile. These layers evolve from the potash tailings through a physico-chemical process with time.

Figures 16–21 summarize the change in total pressure head with time at an elevation of 140 m (i.e., 10 m below the ground surface) for various stratigraphic situations. The initial pore-water pressure conditions in the foundation soil correspond to a water table at the base of the tailings pile (i.e., ground surface). The results show that there are two types of pore-water pressure migration patterns. (1) Pattern 1 occurs when the excess pore-water pressures

TABLE 2. Summary of comparison of failure modes

Foundation soil	Pile height H (m)	Failure modes[a] at indicated piezometric levels for foundation soils			
		0.0H	0.25H	0.50H	0.75H
S1	60	●	●	○	○
	100	●	●	○	○
	200	●	○	○	○
S2	60	●	●	●	●
	100	●	●	●	●
	200	●	●	●	●
S3	60	●	●	●	●
	100	●	●	●	●
	200	●	●	●	●

[a]●, $\Psi \geq 1$ and toe failure dominates; ○, $\Psi < 1$ and base failures dominates.

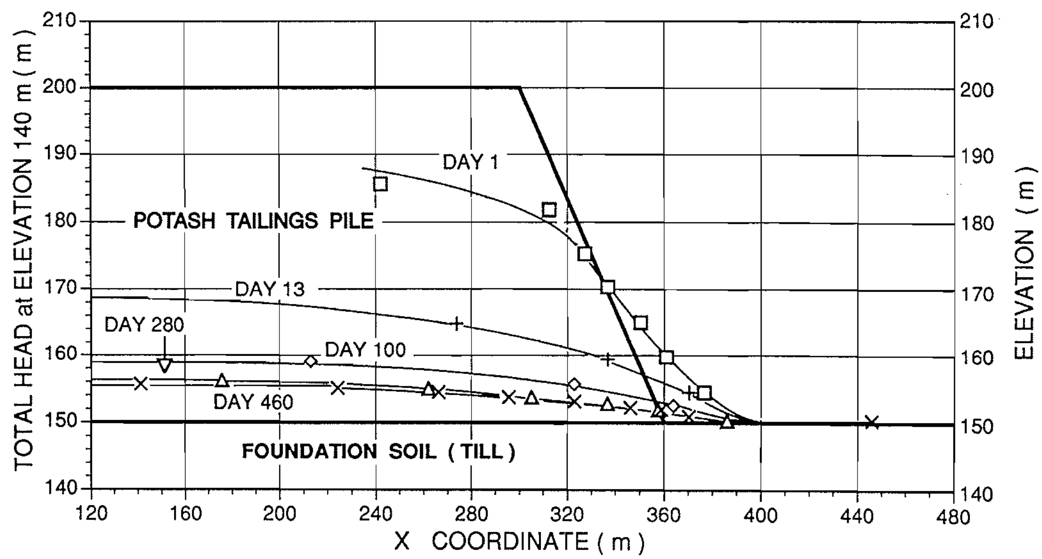

FIG. 16. Changes of total pressure head with time (case A).

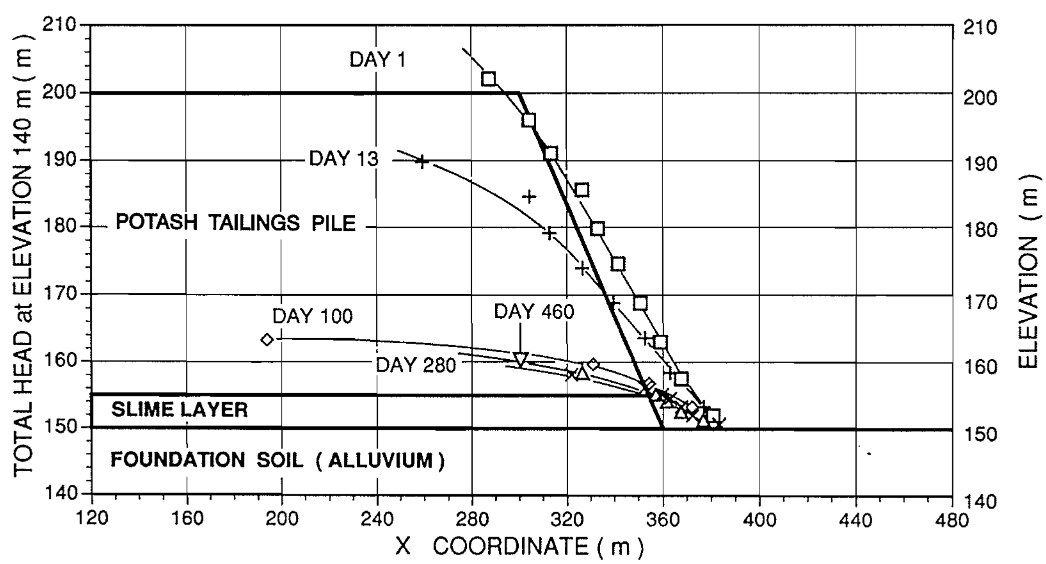

FIG. 17. Changes of total pressure head with time (case B).

drop to zero at the toe of the pile (Figs. 16, 17, 19, and 20). The initial total head curve (i.e., day 1 piezometric level) is always the highest curve. The piezometric lines drop towards the toe and remain at the ground surface outside the toe of the slope. (2) Pattern 2 occurs with the pore-water pressures building up in the region of the toe. This pattern is shown in Figs. 18 and 21 corresponding to cases C and F, respectively. These patterns have two basic

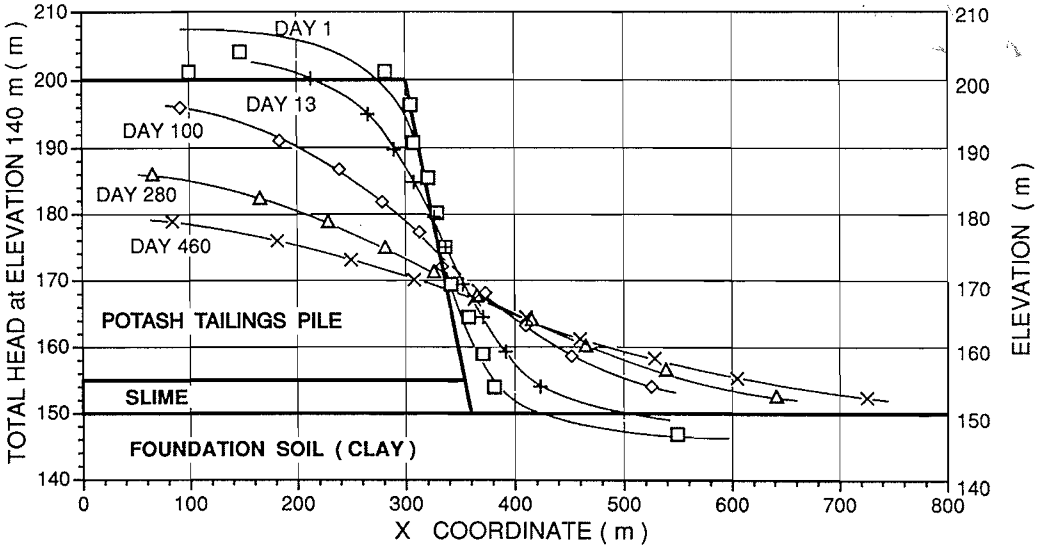

FIG. 18. Changes of total pressure head with time (case C).

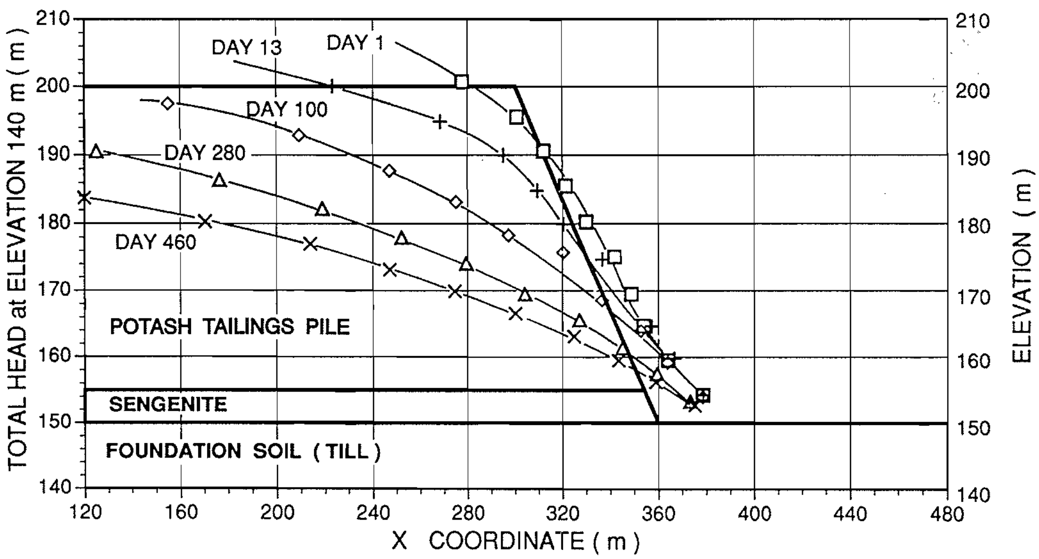

FIG. 19. Changes of total pressure head with time (case D).

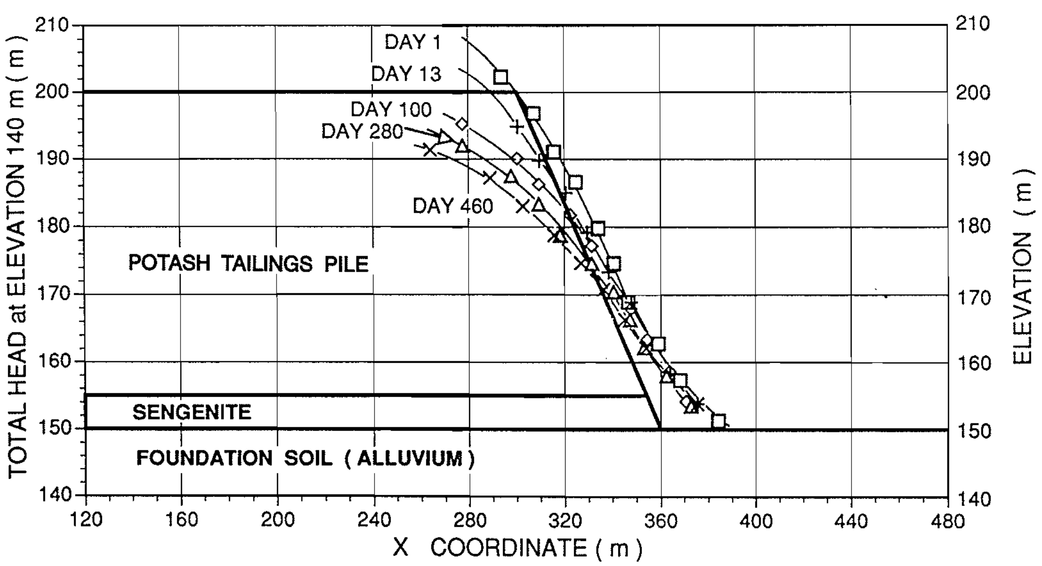

FIG. 20. Changes of total pressure head with time (case E).

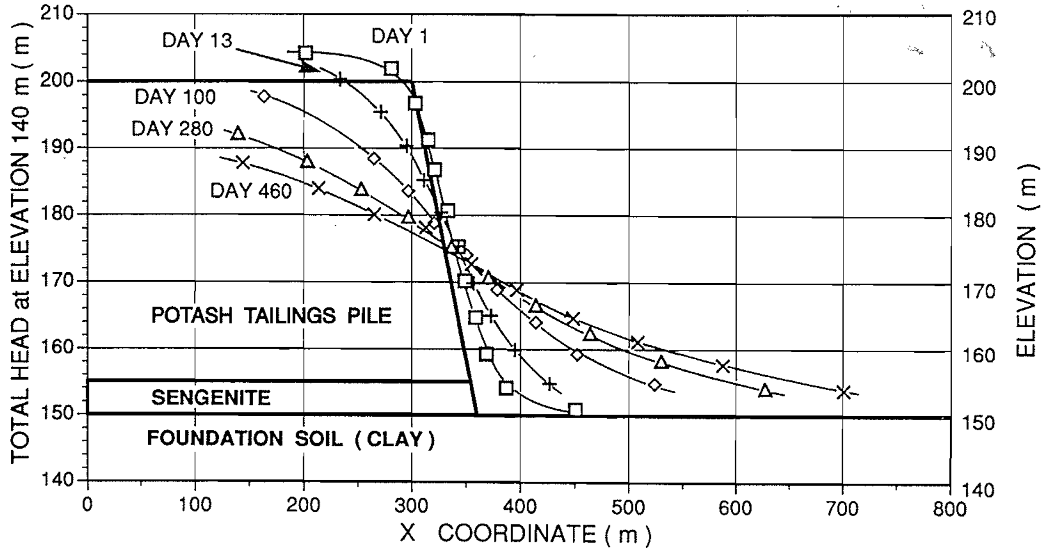

FIG. 21. Changes of total pressure head with time (case F).

TABLE 3. Profiles of tailings and foundation soils used for pore-pressure migration study

Case	Tailings	Foundation soil
A	Without interbedded layers	Glacial till
B	With a thin slime layer at the bottom	An alluvial silt layer of 10 m thickness underlain by till
C	With a thin slime layer at the bottom	Clay layer of 10 m thickness underlain by glacial till
D	With a thin sengenite layer at the bottom	Glacial till
E	With a thin sengenite layer at the bottom	Alluvial silt layer of 10 thickness underlain by glacial till
F	With a thin sengenite layer at the bottom	Clay layer of 10 m thickness underlain by glacial till

TABLE 4. Soil parameters used for the pore-pressure migration study

Material	Storage coefficient m_2^w (kPa^{-1}) For negative pressure	For positive pressure	$k_h k_v$	Saturated coefficient of permeability k_h ($\times10^{-4}$ m/s)	k_v ($\times10^{-4}$ m/s)
Potash tailings	0.056	0.011	5.0	2.21[a]	0.44[a]
Slimes	0.0082	0.0016	1.0	0.0055[a]	0.0055[a]
Glacial till	0.001	0.00001	1.0	0.001	0.001
Alluvial silt	0.005	0.005	1.0	0.01	0.01
Clay	0.001	0.00001	1.0	0.00001	0.00001
Sengenite	0.00001	0.00001	1.0	Essentially impervious	

[a]With a permeability function for desaturation from Wong et al. (1988).

features that are different from those of pattern 1. First, the pore-water pressures beyond the toe are increasing rather than decreasing with time. The pressure head at the toe can be significantly above ground level. Secondly, the pressure-head lines in the till layer beneath the top stratum show that the dominant seepage direction is essentially horizontal in the underlying strata.

A comparison of these two patterns of migration is illustrated in Fig. 22. A point 10 m below the toe of the slope was selected to illustrate the changes in pore-water pressure head with time for the various cases. The reason for the observed increases of pore-water pressure in pattern 2 is related to the existence of a layer of low-permeability material near the top of the foundation soil. The results showed that the thin layer (e.g., a sengenite layer) at the bottom of the tailings pile will delay the dissipation of the pore-water pressures but will not change the migration pattern.

Figures 23 and 24 show the factors of safety changing with time for case A (pattern 1) and case F (pattern 2).

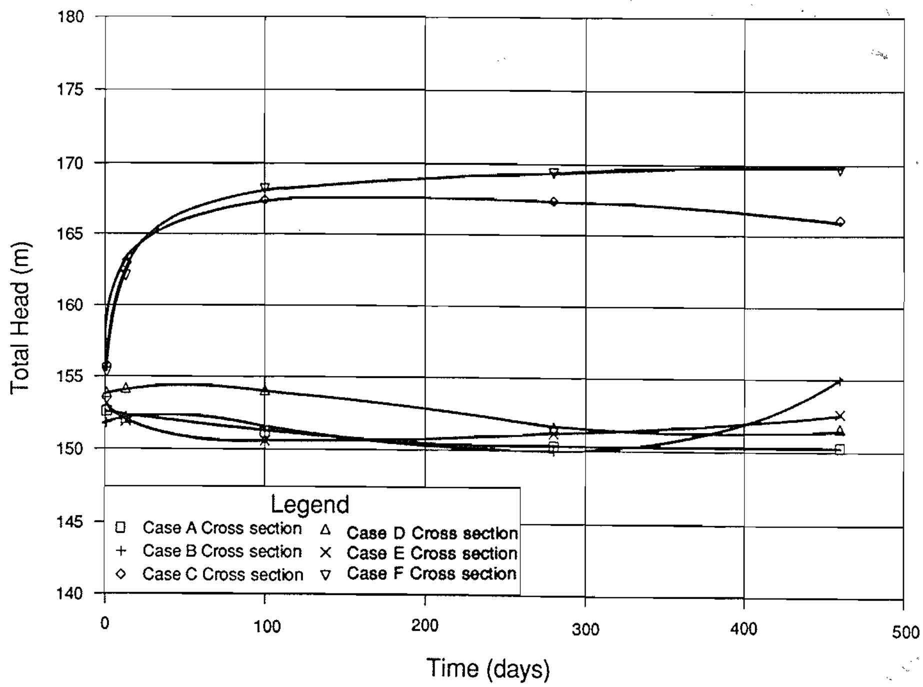

FIG. 22. Total head at elevation 140 m and an x-distance of 380 m.

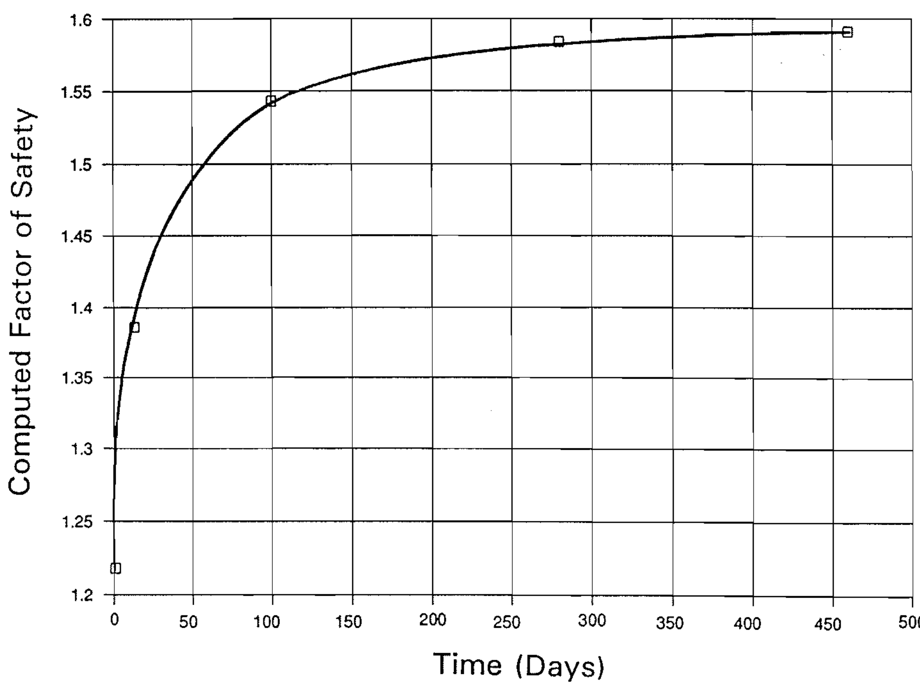

FIG. 23. Factors of safety with time for case A (pattern 1).

The computed factor of safety for case F drops dramatically from 1.42 at day 1 to significantly less than 1.0 at day 100. This illustrates how the pore-water pressures in the toe area of the slope can significantly affect the stability.

Effect of solutioning on the stability of tailings piles

There is the possibility of solutioning of potash tailings both within the pile and around the edge of the pile. Some studies were previously conducted on the effect of solutioning around the edge of a tailings pile (Chiu and Fredlund 1986; Zhang et al. 1991; footnote 2). Solutioning around the edge of the piles could result in overhanging wedges that could easily break off. The analytical results have shown that there is a significant reduction in the local factor of safety caused by solutioning below the edge. However, such failures simply produce a maintenance problem and do not result in a catastrophic failure.

The authors are not aware of any analytical investigations on the effect of solutioning within the pile. However, preliminary studies by Zhang et al. (1991)

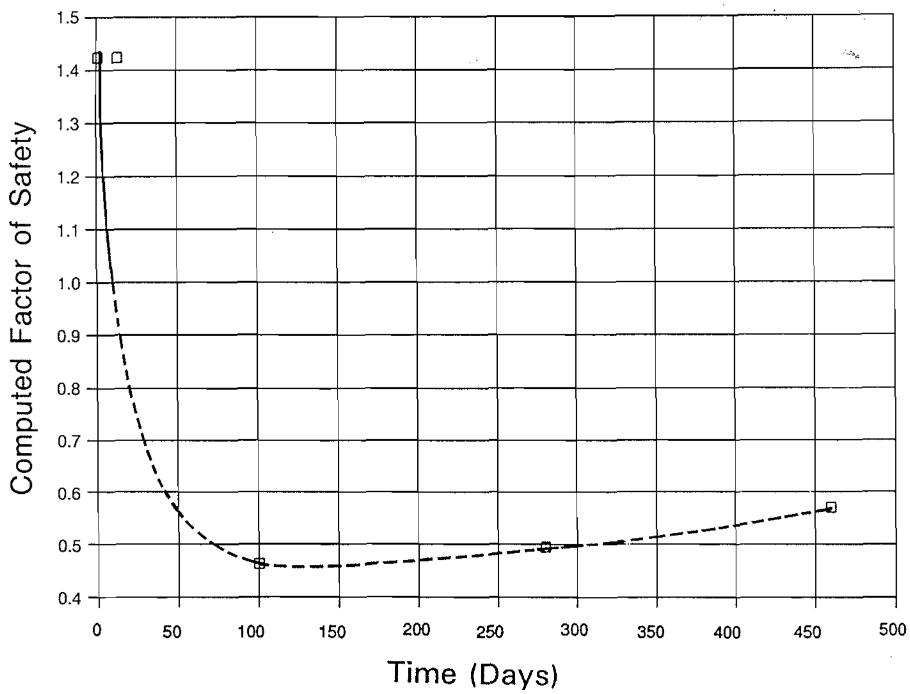

FIG. 24. Factors of safety with time for case F (pattern 2).

showed that cavities within a material mass can influence a limit equilibrium analysis in several ways. The distribution of continuous cavities can form a weakened zone. The total weight may be reduced due to the existence of the cavities. The reduction in weight will in turn affect the normal stresses on a potential slip surface. The cross-sectional area along a slip surface is reduced. The existence of cavities will dissipate excess pore-water pressures. A study of the above factors shows that when there are cavity patterns inside the pile, there will in general be little effect on the overall computed factor of safety. In other words, changes in the gravity loadings and the shear strength appear to counterbalance, and the computed factor of safety remains relatively unaffected (Zhang et al. 1991).

Conclusions

(1) The factor of safety of a potash tailings pile is influenced by the development of pore-water pressure in the tailings pile and the foundation soils, the strength properties of the foundations soils, and the height of the piles. For typical pore-water pressures and foundation-soil conditions presented in this paper, the factor of safety of a potash tailings pile was controlled by a toe-failure mode.

(2) For the case of tailings with no pore-water pressures or for cases where the piezometric line drops toward the toe (e.g., $0.25H$), the factor of safety for the toe failure mode was greater than 1.0. The relationship between factor of safety and pile height H levels off after a particular height of pile.

(3) It is only possible for a base-failure mode to occur in a situation with a piezometric line dropping towards the toe of the pile, if the foundation is weak ($c' = 0$, $\phi' \leq 25°$ in this study) and a relatively high piezometric line is present in the foundation soil (e.g., $h_w \geq 0.75H$ for piles 100 m in height and $h_w \geq 0.25H$ for piles 200 m in height).

(4) Potential slip surfaces for a base-failure mode are shallow. The slip surfaces are shallower than about 15 m below the ground surface for piles up to 200 m in height.

(5) The pore-water pressure migration analysis showed that the dissipation process and the pore-water pressure migration pattern could play an important role in the stability of the tailings piles. The effects of pore-water pressures below the toe of the slope are particularly significant. This time-dependent failure phenomenon in the toe area should be further assessed through field studies.

(6) The effect of solutioning around the edge of the pile mainly results in local slumping failures of the overhanging mass. Such failures produce a maintenance type of problem.

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