Stability of swelling clay embankments

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A common occurrence in cuts or fills of swelling soils is their reduction in strength with time. At the time of compaction, the clay generally has a high matrix suction. Correspondingly, it has a high strength and will stand at relatively steep side slopes. With time, the soil generally tends towards saturation and the matrix suction reduces towards zero. There is a reduction in total strength and if the gravitational forces are too large, the slope fails.

During the past several years, numerous cut and fill slopes have been observed in the Regina area of Saskatchewan. Many of these slopes have remained stable for 4–6 years and then failed. There has been a 20 year history of observations on the Belle Plaine overpass west of Regina. Field and laboratory investigations have been conducted.

With a knowledge of the geometry of the slope and failure plane, the simplified Bishop method of stability analysis was used to perform a 'back-analysis' to assess the shear strength parameters. The shear strength parameters from the laboratory program are compared with those calculated from the stability analyses. The analyses indicate that the peak shear strength parameters from triaxial tests on the softened Regina clay (i.e., $c' = 5$ kPa and $\phi' = 17.5°$), with the appropriate pore water pressures, give a factor of safety of 1 for the failed surface. The effect of spring thawing appears to be to produce the condition of most serious pore water pressures.

Un phénomène courant dans les excavations et les remblais en sols gonflants est la réduction de leur résistance au cisaillement avec le temps. Au moment du compactage l'argile possède généralement une forte succion capillaire. Par conséquent, elle présente une résistance élevée permettant des pentes de talus relativement raides. Avec le temps le sol tend à se saturer et la succion décroît vers zéro. Il se produit une réduction de la résistance totale et, si les forces de gravité sont trop grandes, le talus s'effondre.

Au cours des dernières années de nombreux talus de remblais ou d'excavations ont été observés dans la région de Régina en Saskatchewan. Beaucoup de ces talus sont restés stables pendant 4–6 ans avant de se rompre. Vingt ans d'observations ont été accumulés à l'échangeur de Belle Plaine à l'ouest de Régina pour lequel des études de chantier et de laboratoire ont été réalisées.

Connaissant la géométrie du talus et du plan de rupture, une analyse de stabilité à l'aide de la méthode de Bishop simplifiée a été réalisée pour retrouver les paramètres de résistance au cisaillement mobilisés. Les paramètres de résistance au cisaillement mesurés au laboratoire sont comparés à ceux obtenus des analyses de stabilité. L'analyse montre que les paramètres de la résistance de pic mesurés au triaxial sur l'argile de Régina ramollie ($c' = 5$ kPa, $\phi' = 17.5°$) donnent un facteur de sécurité de 1 avec les pressions interstitielles convenables sur la surface de rupture. Les effets du dégel de printemps semblent produire les conditions les plus critiques de pressions interstitielles.

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Introduction

The use of overconsolidated swelling clays as a construction material commonly presents stability problems (Holtz 1959; Kassiff and Alpan 1973). The clays exhibit a relatively high strength at the time of construction; however, their strength generally decreases with time. Although the process behind the loss of strength is quite well understood, there have been limited quantitative procedures advanced to render the problem amenable to solution.

There has been a history of slope instability in cuts and fills constructed of a swelling clay in the Regina area of Saskatchewan, Canada. Cuts and fills commonly fail 4–6 years after their construction. The failure is generally a shallow essentially circular slide that exits near the toe of the slope.

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TABLE 1. Material properties

Sample	Atterberg limits: Liquid limit	Plastic limit	Plastic limit	Grain size: −0.01 mm	−0.002 mm
Test hole #3-70 (1970)					
Fill (down to 10.7 m)	80	42	38		
Natural (down to 16.8 m)	86	35	51		
Triaxial samples from Regina					
R-23	83	29	54	99	71
R-26	79	20	49	93	65
100% standard AASHO	78	31	47	94	67
Range for samples taken from Belle Plaine (Widger 1976)					
	68–76	22–25	44–53	92–95	63–65

The history of slope instability in Regina clay has prompted a study of the embankments at Belle Plaine, Saskatchewan. This paper presents the history, material characterization, and analyses of the landslides at Belle Plaine.

Description of Study Area and Material

The study area involves two overpass embankments located at Belle Plaine, Saskatchewan on the Trans Canada Highway about 45 km west of Regina. Belle Plaine is situated in the Assiniboine River Plain physiographic region and in the proglacial Lake Regina Basin. The climate of the area is classified as dry subhumid (Ellis *et al.* 1965). The annual precipitation ranges from 160–600 mm with a mean of 370 mm. The study area at Belle Plaine has a normal mean daily temperature in January of −15°C and in July of 18°C. The temperature ranges from −50 to 43°C. The average depth of frost penetration is about 1.2–2.4 m.

The surficial material in the study area is a highly plastic montmorillonite clay known as Regina clay (Christiansen 1961). The Regina clay is highly overconsolidated by desiccation and has slickensided fissures caused by desiccation and frost action. Results from standard physical tests performed on samples obtained at Belle Plaine are shown in Table 1. A standard American Association of State Highways Officials (AASHO) compaction test is shown in Fig. 2.

History and Field Investigation of the Belle Plaine Overpass

The original highway overpass (westbound lane in Fig. 1) at Belle Plaine was built in 1954 and carried traffic in both directions. The embankments were constructed of Regina clay with 2:1 side slopes and a maximum fill height of about 10.7 m. Slides occurred on the embankments in 1959, but few details are available on the extent of sliding and on the repair work performed, other than that the new slopes were approximately 2:1.

In 1965, construction was started on a new embankment for an eastbound lane. During the construction of the eastbound overpass, a deep borrow pit was excavated at the toe of the old fill (point *a* in Fig. 1). This increased the effective height of the old fill to 15.2 m. In 1966 the eastbound overpass was completed with 3:1 side slopes and a maximum fill height of about 9.1 m. It was paved the same year and overlaid in 1968.

In the fall of 1968, two landslides occurred on the northeast side of the westbound lanes (point *b*) and a minor slide occurred west of the overpass on the westbound lanes (point *c*). Cracking accelerated in the fill directly above the borrow pit (point *d*). In the spring of 1969, a landslide about 90 m long that involved about half the height of the slope occurred above the borrow pit (point *d*). The slide was accompanied by new cracking along the shoulder and in the driving surface. Cracks from 75–100 mm wide opened on the shoulder and cracks 25 mm wide opened in the driving mat.

In the fall of 1969, movements started in the east backslope region under the overpass on the eastbound lanes. The bolts in the expansion joints at the east abutment were bent and the concrete was cracked. The backslopes under the structure were 1.5:1 or steeper and constructed from relatively loose uncompacted dumped material. Some of the water from the deck drained onto this steep poorly drained slope. The drainage conditions were subsequently improved.

During the summer of 1970, the slide above the borrow pit (point *d*) was repaired and part of the borrow pit (point *a*) was filled. With the repair fill less than 3 m above the natural ground, tension