Experiences with Two Fills on Clay Shale Foundations

Experiencias con Dos Rellenos de Arcillas Comprimidas en Terreno Firme

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SYNOPSIS Highly plastic clay shale of Cretaceous age underlies much of the central plains of North America. Many of the river valleys of western Canada have eroded through the glacial overburden and into the Cretaceous bedrock. Where this occurs, the valley walls are often flattened by slumping processes which shear and soften the upper portion of the bedrock strata. As a result, poor foundation conditions exist along many of the river valleys. Many transportation corridors must cross these valleys, and the associated earthworks are often designed with factors of safety with respect to stability of less than 1.25. This paper reports observations on two such earth fills constructed at bridge crossings. Both were constructed with factors of safety of less than 1.25 and have been observed for more than 10 years. The rate of strain experienced was a function of the factor of safety, varying from large strain at low factors of safety to less than 1 mm per year at a factor of safety of 1.2.

1. INTRODUCTION

The Cretaceous shale that underlies a large area of the central plains of North America is montmorillonitic and exhibits high plasticity. In the unweathered, intact state, these materials are hard and exhibit high cohesion and friction angles consistent with the nature of the clay minerals. Upon softening, the loss of strength, particularly through loss of cohesion, is substantial (Skempton, 1964). When subjected to shear strain, these materials exhibit marked residual behaviour with a reduction in the effective angle of internal friction and reduction of effective cohesion to near zero.

Extensive landslides exist along many of the river valleys of western Canada where a valley has been eroded into highly plastic clay shales. As a result, poor foundation conditions exist along many of the river valleys. A methodology for design, construction and monitoring of earth fills founded on clay shales has been developed in western Canada (Clifton, 1994). This paper examines two such developments where approach fills for major bridges have been constructed with factors of safety less than 1.25. The two sites lie approximately 40 km apart along the valley of the North Saskatchewan River in west central Saskatchewan, Canada, as indicated in Figure 1. The paper examines the geology,

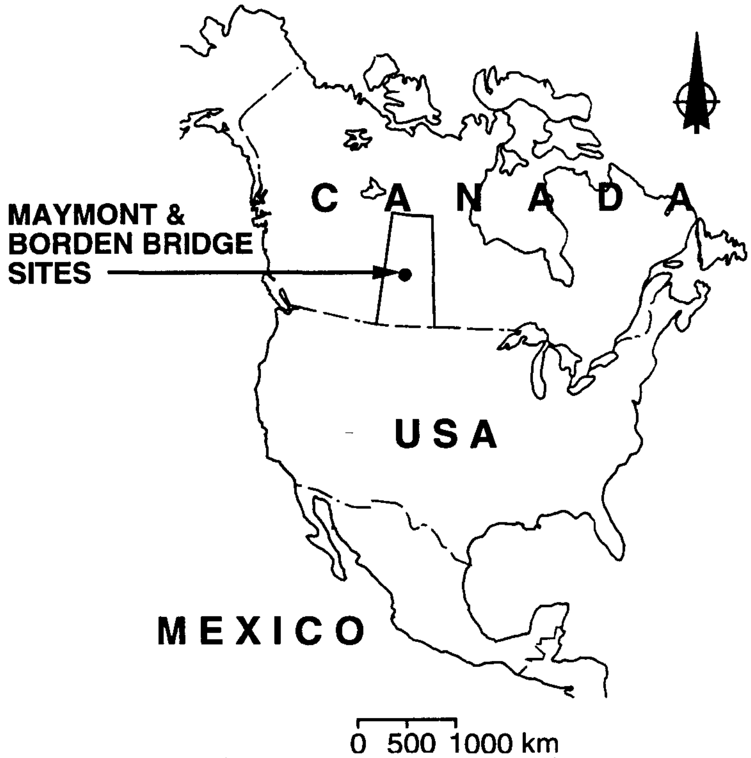

Fig. 1 General Location of Maymont and Borden Bridges

briefly outlines the design methods, and reviews monitoring results for earth fills constructed in 1978 and 1982, respectively.

2. REGIONAL SETTING

During the Cretaceous epoch, the interior basin of North America was infilled with marine clay (Caldwell, 1968). The Lea Park formation was part of the Upper Cretaceous marine sediments deposited in west central Saskatchewan and east central Alberta, Canada. These sediments were subsequently overlain before the surface was modified by at least four major glaciations during the Pleistocene epoch. Ice from the latest (Wisconsinan) glaciation retreated from the area approximately 12,000 years ago (Christiansen, 1979). Most of the major river valleys were meltwater channels formed during deglaciation. Successive processes of erosion of the valleys, slumping of the valley walls, and agrading of the river channels have resulted in broad valleys with extensive floodplains and large reaches with slumped valley walls. The processes of slumping, along with ice thrusting (Stauffer *et al*, 1990) have reduced the shear strength of valley soils. The valleys are commonly more than 100 m deep and have an overall valley wall slope as flat as 10 per cent.

3. MAYMONT BRIDGE APPROACH FILL

Conventional drilling and sampling techniques were utilized to define the site conditions including the stratigraphy, piezometric conditions and geometry of the landslide mass. A generalized cross section illustrating soil conditions in the valley is shown in Figure 2. The base of shearing was identified in highly plastic clay shale, and the landslide mass consists of clay shale and glacial till. The lower block of landslide debris extended into the alluvial sand of the floodplain.

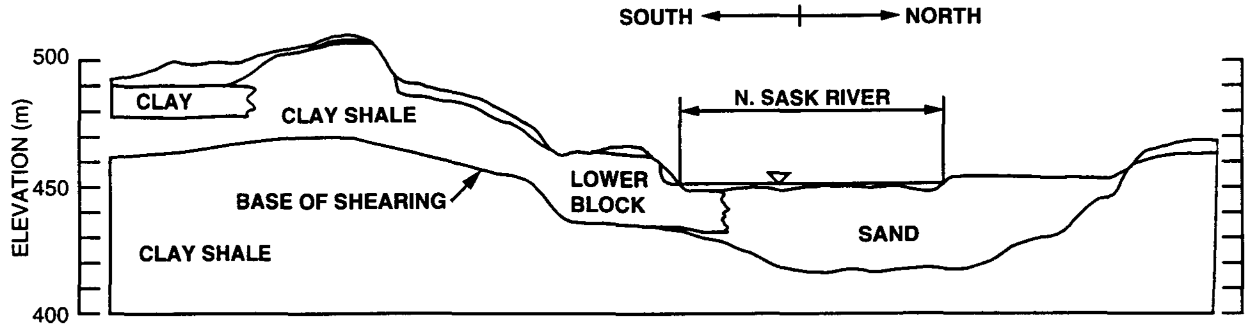

Fig. 2 Cross Section of River Valley at Maymont Bridge (Clifton, 1994)

Shear strength parameters for the clay shale were determined both by laboratory testing and by back analysis techniques. Conventional laboratory testing on both intact and pre-sheared samples resulted in effective residual friction angles varying between 11° and 19.5°. Back-analysis, using the Simplified Bishop Method, concentrated on the lower block of the south valley wall. This analysis, along with experience of other investigators, indicated the effective residual angle of internal friction should be 10° along with a nominal cohesion of 0 kPa to 2.5 kPa (Johnson, 1978).

The approach fill had a maximum height of approximately 15 m. The most crucial section of the foundation was comprised of disturbed clay shale which overlaid and was overlain by alluvial sand as indicated in Figure 3.

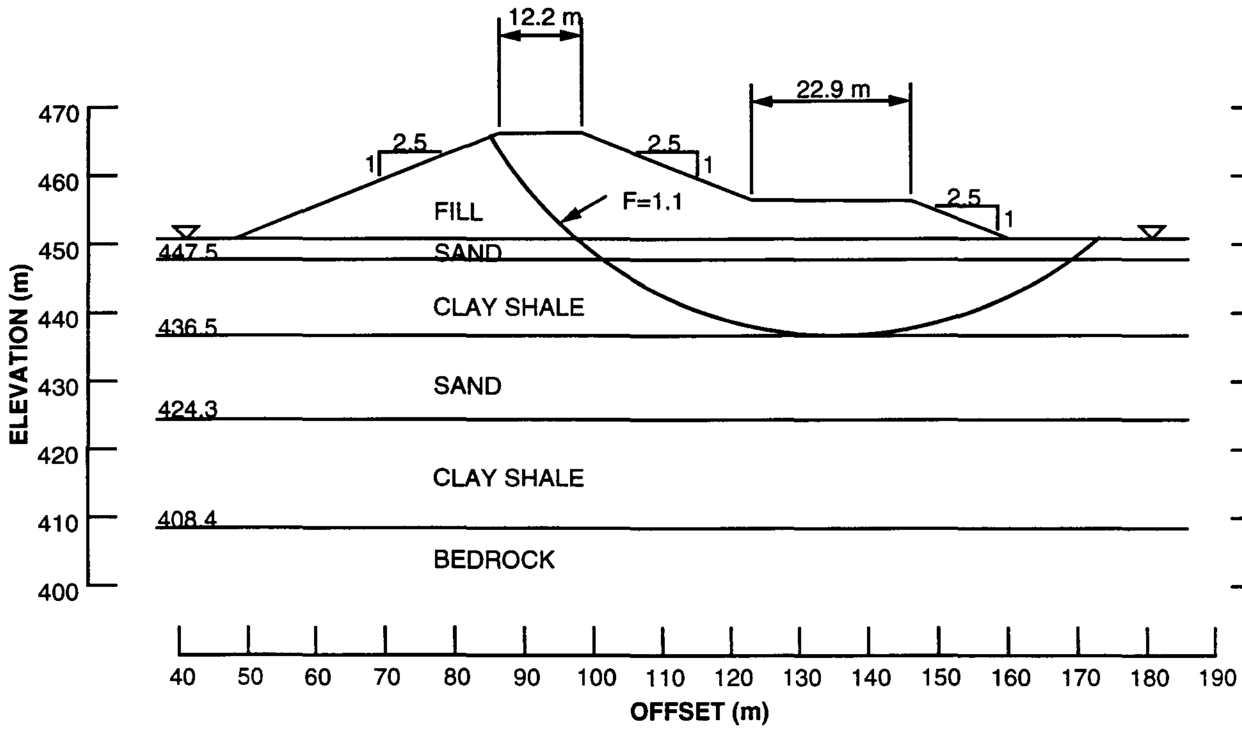

Fig. 3 Typical Geometry of South Approach Fill, Maymont Bridge (Clifton, 1994)

Earthwork in the vicinity of the bridge abutment and other critical areas were designed with conventional factors of safety, but a lower factor of safety, approximately 1.1, was utilized for lateral stability of fills where the consequence of failure was low. Typical fill geometry is illustrated in Figure 3.

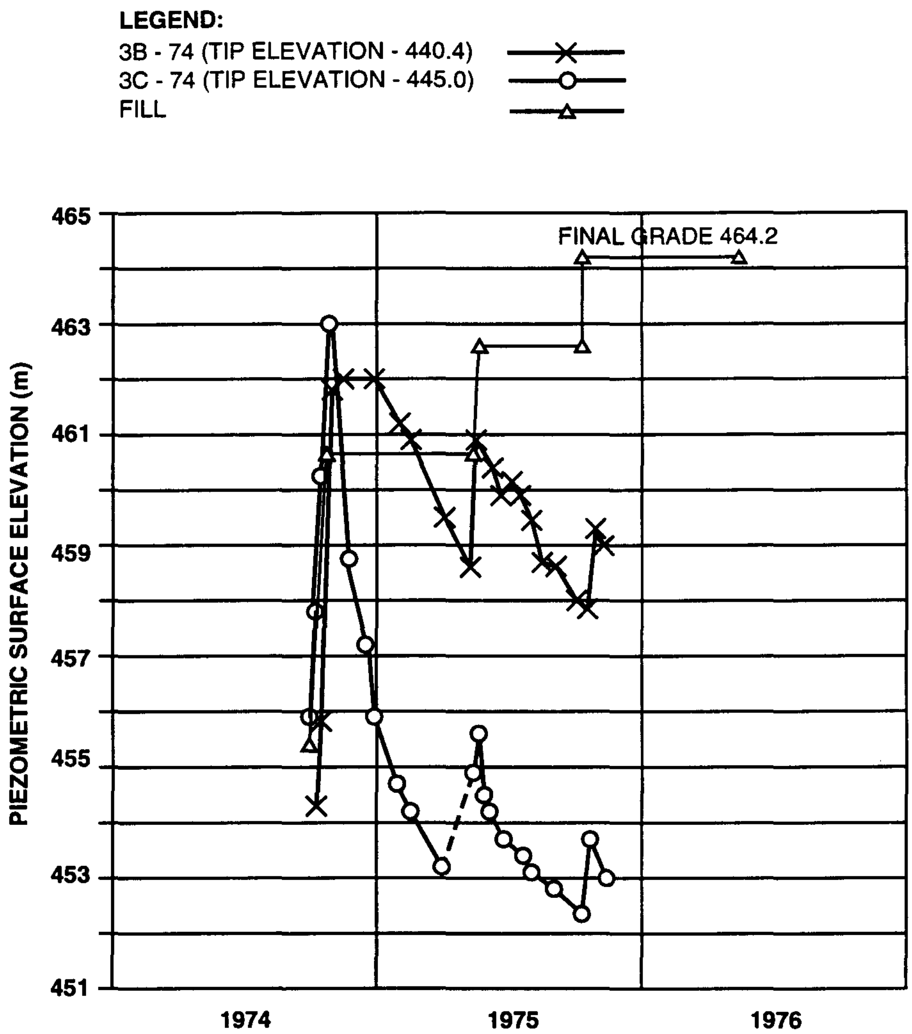

Fig. 4 Typical Fill Elevation and Construction Pore-water Pressures, South Approach Fill, Maymont Bridge (Clifton, 1993)

Construction of the fill commenced in 1974 and was completed in 1975. The typical rate of fill advance and resulting foundation pore-water pressures are illustrated in Figure 4.

The stability of the fill was monitored using slope movement indicators which were responsive to the rate of fill placement and pore-water pressure conditions. Figure 5 illustrates the rate of change of movement across the shear plain in a typical slope movement indicator installed through the fill. Strain was detected at several elevations in the foundation, with the more exaggerated movement at shallower depth. The rate of movement exceeded 120 mm, on an annual basis, when pore pressures were elevated and the calculated factor of safety was 1.04. Once the pore pressures dissipated to less than the design condition, the calculated factor of safety was 1.15 and the shear strain across the shear zone reduced to an average rate of 2 mm per year.

4. BORDEN BRIDGE APPROACH FILL

A concrete arch bridge was constructed across the North Saskatchewan River in 1937. The south abutment of that bridge was constructed, with little cut or fill, on a landslide block on the south valley wall. The abutment foundation performed adequately with no reported distress (Wilson *et al*, 1986).

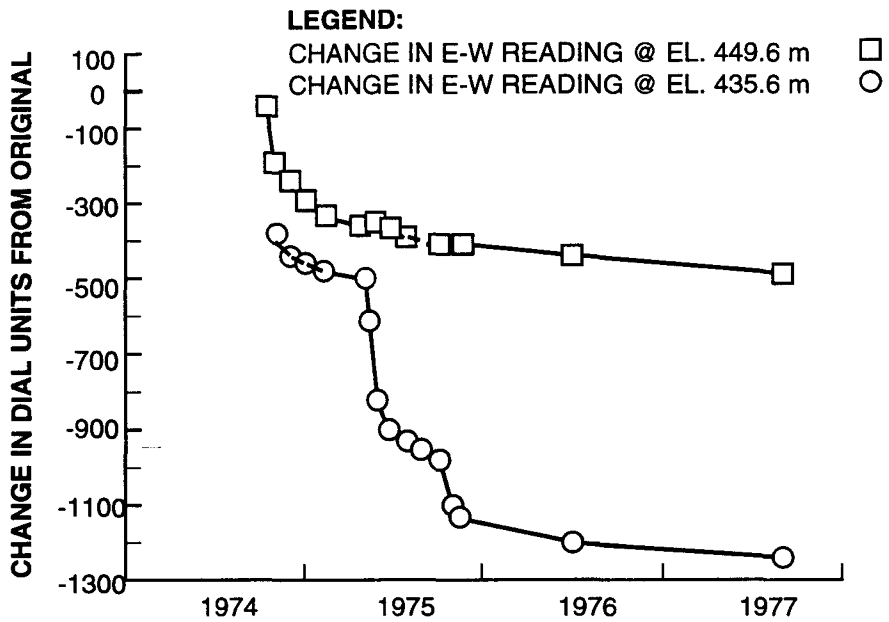

Fig. 5 Change in Rate of Slope Movement, South Approach Fill, Maymont Bridge (Clifton, 1993)

Construction of approach fills for a new bridge immediately upstream of the existing structure commenced in 1984. The cross section of the river valley at the bridge location is shown in Figure 6 and typical critical slope section near the south abutment is shown in Figure 7. The cross section shows the valley wall stratigraphy consisting of glacial till overlying shale. A layer of bentonitic, highly plastic clay shale existed at an elevation equivalent to the base of the alluvial sand, approximately 40 m below natural ground at the south abutment.

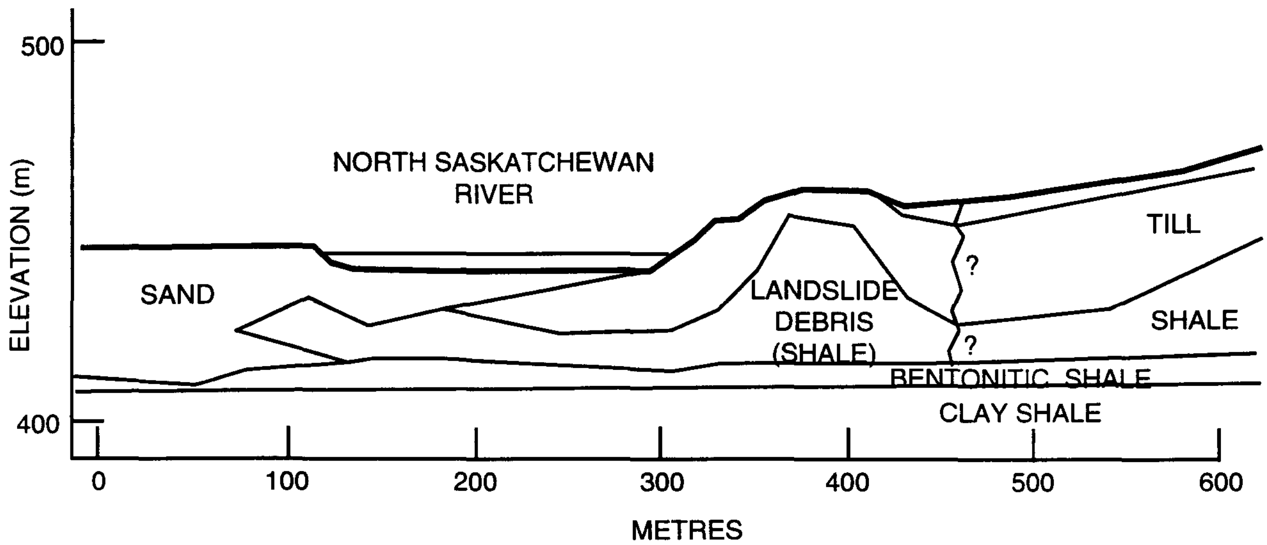

Fig. 6 Cross Section of River Valley at Borden Bridge (Wilson *et al*, 1986)

Laboratory tests on the highly plastic clay shale indicate it had a plastic limit in the range of 32% to 36%, a plastic index of 60% to 80%, and a liquid limit of 90% to 120%. Natural water content in the undisturbed shale was approximately 23%. This material exhibited a residual effective friction angle of between 5° and 9°. The water content of disturbed clay shale was typically close to the plastic limit. Back-analysis of existing slopes yielded effective angles of internal friction of 6.0° with zero cohesion; this value was chosen for design calculations.

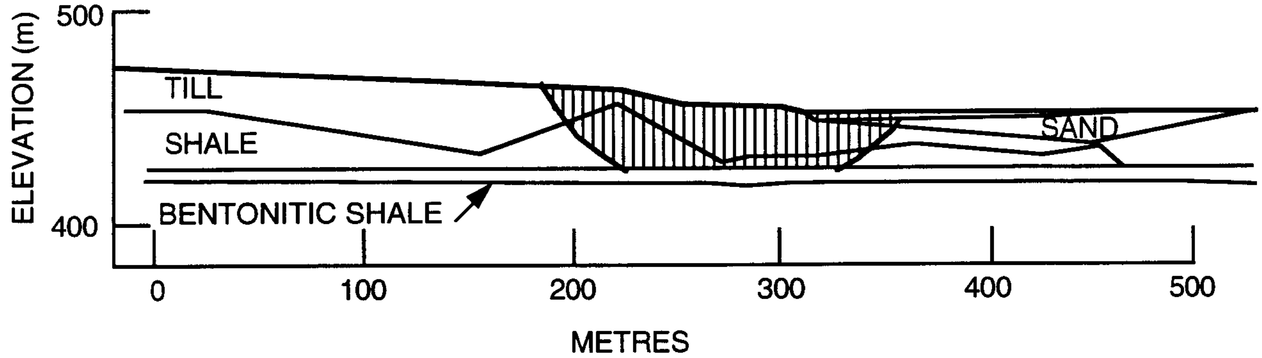

Fig. 7 Typical Critical Slope Section, South Abutment, Borden Bridge

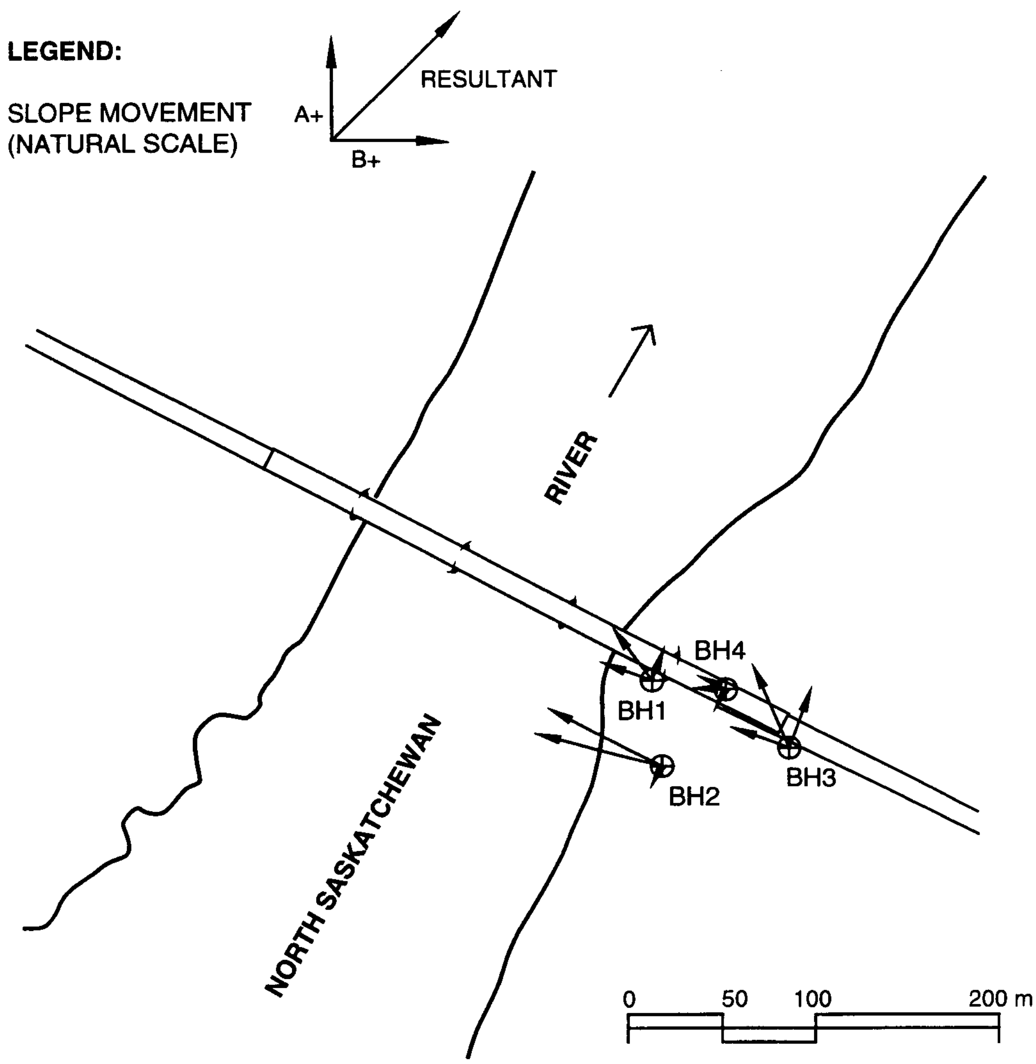

Fig. 8 Slope Movement Vectors 1983-1994, South Approach, Borden Bridge

Construction of the approach fills began in 1984 and the bridge was opened to traffic in 1985. Slope movement indicators were installed in four bore holes near the south abutment. The location of slope movement indicators and vectors indicating the magnitude and direction of movement observed across the shear plane between 1984 and 1993 are indicated in Figure 8.

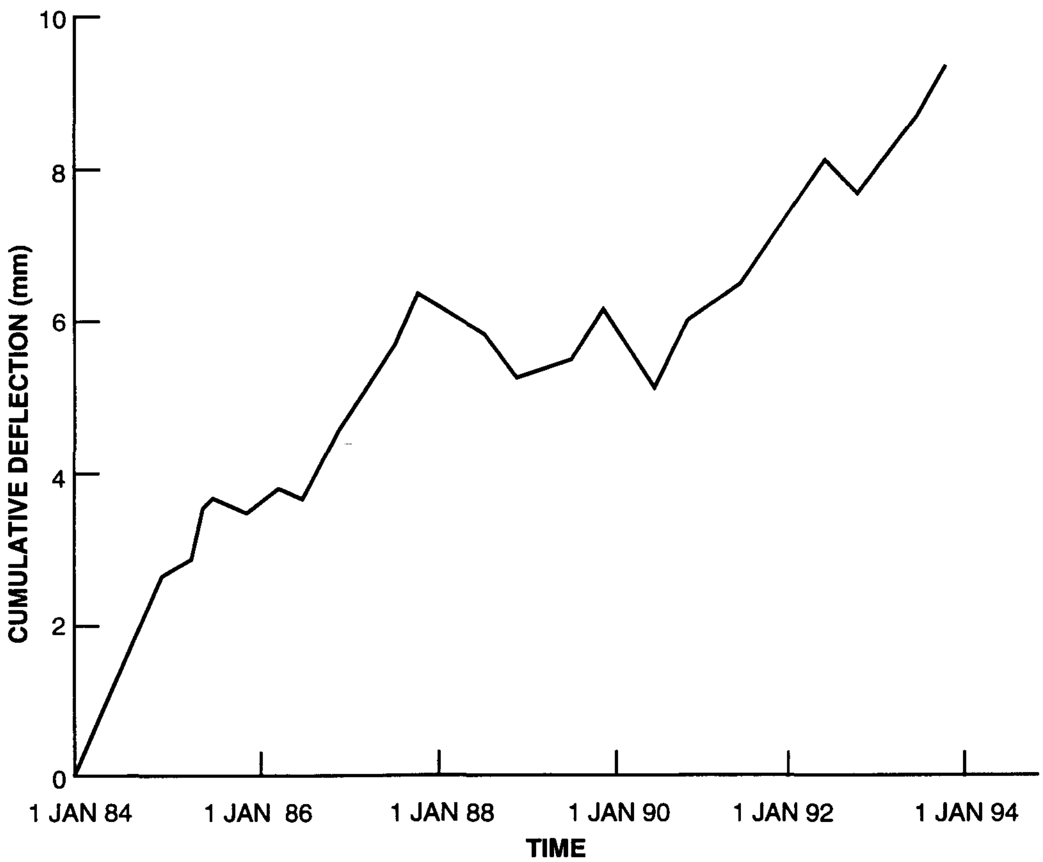

Fig. 9 Cumulative Displacement in the Downslope Direction, Instrument BH3

Typical measured cumulative deflection in the downslope direction is illustrated in Figure 9, which plots movement parallel to the fill centreline against time. The measured cumulative displacement over the same period for the same instrument (BH3) is illustrated in Figure 10.

The analysis of stability was conducted using the Simplified Bishop Method. For the conditions analyzed, it was calculated the factor of safety would vary between 1.06 and 1.20. The principal zone of movement was observed in the slope movement indicators between elevation 412.2 and 416.2 and the strain rate varied from an average rate of movement of 0.8 mm to 2.0 mm per year in the post construction period.

5. DISCUSSION

Construction of earth fills on sheared highly plastic shale foundations can be expected to cause shear displacement as load is applied and foundation pore-water pressures increase. As the factor of safety with respect to stability approaches unity, the rate of observed movement in the foundation may increase dramatically. With dissipation of pore-water pressure, reduction of the rate of movement usually occurs.

Similar results were observed at the two sites in this study. Initial fill placement resulted in movement, but the initial rate quickly subsided to a long term rate of 2 mm or less per year even though the calculated factors of safety were less than 1.2.

It is interesting to compare the observed results with results of excavated slopes. Krahn *et al* (1979) reported on the failure of a large cut slope on the

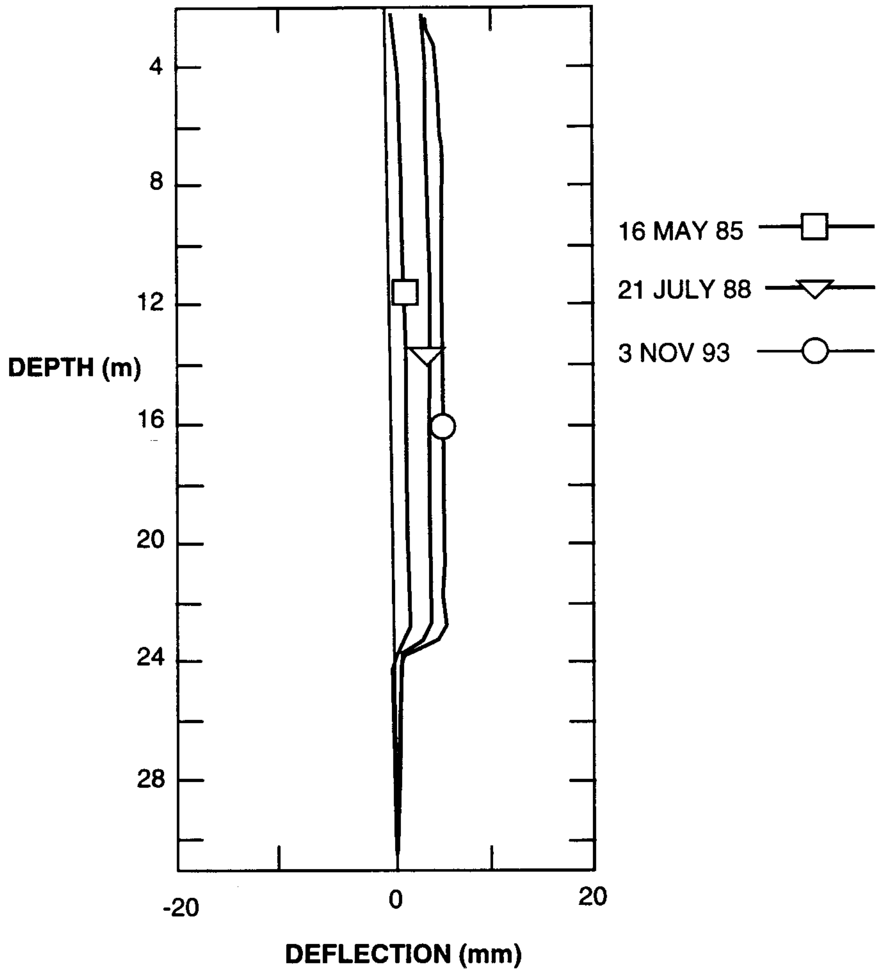

Fig. 10 Measured Displacement with Depth, BH3

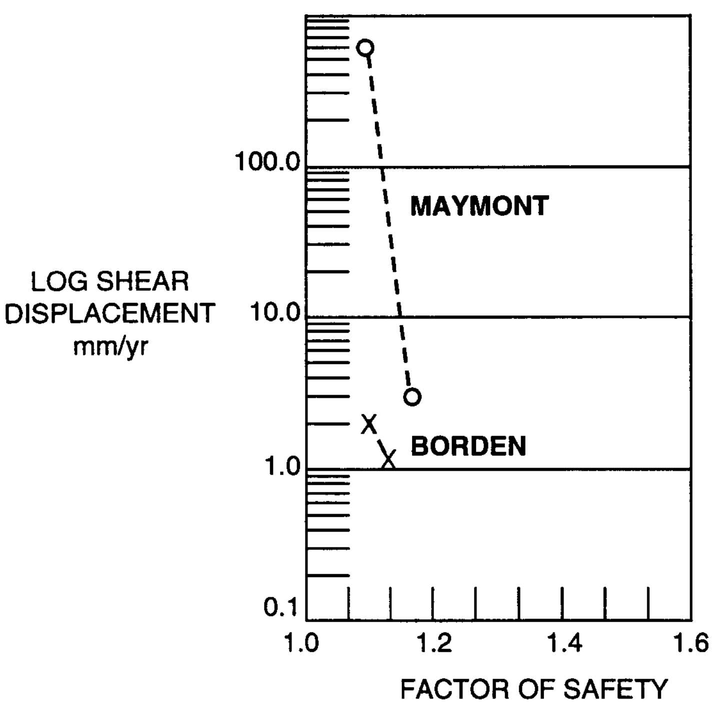

Fig. 11 Average Annual Displacement at Various Factors of Safety, Maymont and Borden Fills

south valley wall at Maymont where the design factor of safety was unity for a slope founded on sheared clay shale. Large rapid movement of the slope was observed as the design grade of the excavation was reached.

By contrast, the fills behaved in a relatively predictable manner, with the observed strain rate a function of the calculated factor of safety. The observed relationship between average long term shear displacement and factor of safety is illustrated in Figure 11. For the conditions observed, the rates of shear displacement varied from 0.8 mm to more than 120 mm per year. The calculated factors of safety varied from 1.04 to 1.2. Relatively small increases in the factor of safety were effective in reducing the rate of movement of the fill, even when available shear strength in the foundation was very low. Careful construction control and proven analytical methods are necessary to predict slope behaviour.

6. CONCLUSION

Experience with the Borden and Maymont fills has indicated that, under certain conditions, fills with a low factor of safety can exhibit satisfactory performance with respect to stability. Highly plastic clay shales have low shear strength, but fills can be effectively designed and constructed to provide adequate long term performance if the geometry, shear strength and pore-water pressures are adequately defined. Sensitive instrumentation, strategically placed, is required to monitor both shear strain and pore-water pressures during construction. Construction conditions can be expected to be less stable than long term conditions, and staged construction may be necessary. Adequate design and construction control practices are available. Experience indicates that fills with a low factor of safety may be acceptable in certain circumstances.

7. ACKNOWLEDGEMENTS

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