Geotechnical instrumentation in practice

Purpose, performance and interpretation

Proceedings of the conference *Geotechnical instrumentation in civil engineering projects* organized by the Institution of Civil Engineers and co-sponsored by the British Geotechnical Society, the British Tunnelling Society and the British National Committee on Large Dams and held in Nottingham on 3-5 April 1989.

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Earthworks

Landslides and excavated slopes

P7. Behaviour of an instrumented reinforced embankment

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SYNOPSIS. The paper deals with the case of an overpass, composed of two reinforced earth embankments, linked by a three-span bridge. One of them has been instrumented with electric strain gauges. The Authors present a comparison between experimental and theoretical values of the tensile tension on three characteristic reinforcing elements.

INSTRUMENTATION AND MEASUREMENTS.

1. The soil, used as granular fill, is a sandy gravel. A series of C.I.D. triaxial tests with increasing confining pressure were carried out. The triaxial specimens were compacted, with a rubber membrane around them, in a specially developed split mold (200 mm wide; 410 mm high). The failure curve on the Mohr plane was not linear. For the theoretical determination of the stresses in the reinforced embankment, an internal friction angle of 50° and an apparent cohesion of 0 were assumed.

2. Electric resistance strain gauges, for measuring the stresses developing on the reinforcing elements, were installed on a cross-section of the smaller embankment. Three characteristic reinforcing elements were chosen, respectively at +4.40 m, +2.90 m and +1.40 m from original ground level. The gauges were placed at various distances from the vertical facing, 3 on the upper strip, 4 on the middle one, and 3 on the deep one. The gauges were bonded in pairs to the top and bottom of the reinforcing elements to permit separation of axial and bending tension.

3. The distributions of axial tension on the instrumented reinforcing elements are presented in Fig.1. The values of the peak tension appear to be indipendent of depth. This behaviour conflicts with the conventional design approaches (tie-back analysis and coherent gravity analysis)in which it is assumed that, for constant spacing of the elements, the tension will increase in direct proportion to the depth of the fill. This could be due to the stresses induced by compaction, having a dominating influence on those produced by the level of the fill, or to the rod, to which all the elements in a vertical profile are connected, that causes a redistribution of the tensile forces.

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4. The line of maximum tension cannot be located. Anyway, apart from the deepest reinforcing element, the highest value of the axial stress was recorded at the point nearer the facing. The simple schemes, used for the calculation of the maximum axial stress and corresponding tensile force on each reinforcing element, gave values greater than experimentally recorded ones. This could be due to the extremely conservative values provided by the calculation scheme. Only on shallow element theoretical values match with experimental ones; proceeding downwards such differences tend to increase more and more.

5. Considering the shallow reinforcing element it has been determined the average mobilized coefficient of the interface friction soil-strip on the basis of the gradient between the peak tension and the rear of the element, where the tension must be equal to zero: we obtained a value of 0.29 much less than 1.2 proposed by Schlosser for ribbed strips.

6. Lastly, corrosion checking rod, 600 mm long, was positioned in the fill in 1984 and removed four years later. The weight loss was negligible and equal to 0.06% of its original weight.

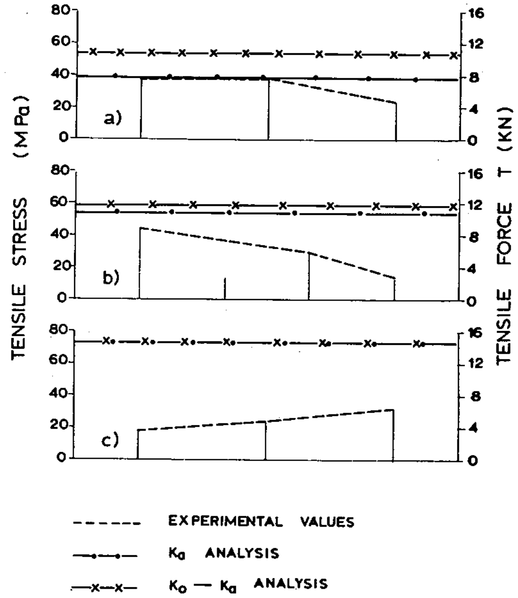

Fig. 1