E. Rojas and G.T. Houlsby, Oxford University

These predictions were made according to common engineering procedures:

(*a*) The initial pore pressure response of the clay was obtained from Henkel's expression $\Delta u = \alpha\Delta p' + \beta\Delta q$ where the parameters α and β were established from the results of the K_0 consolidated undrained triaxial compression tests on intact specimens of the clay ($\alpha = 1.0$, $\beta = 0.2$). The stress increments $\Delta p'$ and Δq were obtained from the elastic solution for a uniformly loaded rectangular area and employing the superposition method for the entire H-shape embankment.

(*b*) Initial settlements were also obtained according to elastic theory. The value of the shear modulus at different depths was obtained from the expression $G = 360\,\sigma'_v$ which was derived from the seismic cone test results reported by Nash, Powell and Lloyd (1992). This expression corresponds to the sole available data of the shear modulus at small shear strains and was employed as no information on the variation of the shear modulus versus the shear strain was provided. A value of 0.5 was assumed for Poisson's ratio.

(*c*) The excess pore pressure in the clay was computed considering both vertical and radial drainage and employing the finite difference method. The coefficient of consolidation was deduced from the consolidation test results, reported by Nash et al., by taking into account the stress increments induced in the clay by the embankment ($c_v = 3.2\,m^2/y$). The value of this parameter was considered to be equal in the vertical and horizontal directions as the overconsolidation ratio of the soil was reported to be small (1.5). Although this parameter is slightly higher to the one proposed in the invitation package for pressures ranging between 20 and 50 kPa ($c_v = 1.7\,m^2/y$) the former was preferred as it agreed better (although not completely) with the fact that the initial pore pressure produced by the first embankment layer of 500 mm was completely dissipated after 6 months.

(*d*) The consolidation settlement of the surface was obtained from the classic theory of consolidation. The coefficient of volume compressibility (m_v) at each level was obtained by associating the plots of the consolidation tests for three different depths with the stress increment produced by the embankment at that depth. The values of the coefficient ranged from $m_v = 0.0012$–$0.0022\,m^2/kN$.

(*e*) The profile of force with depth for friction and end-bearing concrete piles for both the isolated pile and the central pile in the group were obtained from the method proposed by Zeevaert (1973) on the basis of the initial and long-term final effective stresses computed for each case. For the short-term behaviour of the pile, a correction had to be applied as it was considered that the displacements of the soil produced by the consolidation settlements near the neutral point of the pile were too small to mobilize the total shear stress between soil and pile. This correction was made according to the theoretical stress–displacement curve of the pile as proposed by Poulos and Davis (1980).

Due to the fact that Zeevaert's analysis is based on the effective stress distribution around a single pile, this analysis may become rather complex for a group of piles as the distribution of stresses depends on the position and contribution of each one of the piles in the group. However an approximate solution can be obtained if the analysis is constrained to the tributary area of the analysed pile in the group which in this case is 4D × 4D.

For the steel piles it is considered that the forces on the shaft will be the same as those given for the concrete piles reduced by a factor of 2/3.

References

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