Geotextiles and Geomembranes 38 (2013) 51–67

Contents lists available at SciVerse ScienceDirect

Geotextiles and Geomembranes

journal homepage: www.elsevier.com/locate/geotexmem

A case study on soil settlements induced by preloading and vertical drains

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A R T I C L E I N F O

Article history:
Received 31 October 2012
Received in revised form
24 March 2013
Accepted 3 May 2013
Available online 7 June 2013

Keywords:
Soil improvement
Preloading
Vertical drains
Settlement monitoring
Dissipation tests
Allowable steel tank settlements

A B S T R A C T

The method of preloading associated with prefabricated vertical drains was used to accelerate the consolidation process and anticipate the long-term settlements of the foundation soil of two cylindrical oil tanks founded on an alluvial deposit mainly consisting of silty clays.

In-situ investigations, including boreholes and cone penetration tests (*CPTs*), and laboratory tests were carried out to define the geotechnical profile of the construction site and the soil mechanical properties. Dissipation tests were also carried out during the *CPTs* and allowed evaluating the horizontal consolidation and permeability coefficients through several procedures.

An extensive field monitoring of the site was carried out during the embankment construction, the preloading period and, after the embankment removal, during the hydraulic leakage test of the tanks. Differential settlements and angular distortions of the tank foundation evaluated from the measured settlement profiles were compared with expected profile shapes for tanks overlying homogeneous compressible soil layers and with available empirical relationships. A general fair agreement was observed even if the heterogeneity of the alluvial soil deposit affects the tank response.

Observed absolute and differential settlements and distortions are consistent with the allowable limits provided by the literature and with the design prescriptions, thus confirming the effectiveness of the preloading and drainage technique adopted in the project and envisaging a satisfactory performance of the tank under service conditions.

1. Introduction

The performance of heavy structures founded on soft normally consolidated or lightly overconsolidated clays is governed by large stresses induced in the soil and by poor soil mechanical properties. In this case the geotechnical design is aimed to ensure stability and keep settlements within allowable limits. For shallow foundations small to medium strains, causing excessive settlements, or large plastic strains, eventually culminating in a bearing capacity failure, may arise in the foundation soil.

Both stability problems and the occurrence of excessive settlements can be coped with different techniques, among which deep foundations, stabilising loading berms, soil replacement, reinforcement or improvement. In many projects soil improvement through the well-known technique of preloading associated with vertical drains proved to be effective in accelerating soil consolidation, thus increasing the shear strength of soil and reducing the post-construction settlement (e.g. Holtz et al., 1991; Rowe et al., 1984; Maugeri et al., 1994; Indraratna and Redana, 2000; Almeida et al., 2000; Li and Rowe, 2001; Shen et al., 2005; Chu et al., 2006; di Prisco and Buscarnera, 2010; Moraci and Gioffrè, 2010; Saowapakpiboon et al., 2010; Moraci, 2011). In fact, soft clay deposits are usually characterized by low permeability and may be considerably thick with respect to the foundation width. Therefore, the use of preloading alone to achieve a desired settlement or increase in shear strength may require a long time, possibly incompatible with the work yard schedule. Vertical drains allow drastically shortening the drainage path to half of the drain horizontal spacing and, due to the inherent anisotropy in soil permeability, the rapid radial drainage accelerates the consolidation process (Jamiolkowski et al., 1983).

Analytical solutions for the consolidation of homogeneous soils by vertical drains installed in a regular pattern have been derived considering a soil column containing a single vertical drain which is assumed to dewater the column under suitable boundary

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http://dx.doi.org/10.1016/j.geotexmem.2013.05.002

conditions. Barron's (1948) theory is still the basis for most analyses of radial consolidation. The theory accounts for soil smear due to drain installation, drain hydraulic resistance, vertical and radial flows and permeability anisotropy. A number of studies provided analytical or approximate solutions accounting for the effects of soil smear, transition between smeared and undisturbed soil, drain resistance and discharge capacity (e.g. Hansbo, 1981; Tripathi and Nagesha, 2010; Abuel-Naga et al., 2012; Deng et al., 2013), time-dependent loading (e.g. Tang and Onitsuka, 2000; Zhu and Yin, 2004), heterogeneous soils (e.g. Tang and Onitsuka, 2001; Tang et al., 2013), the destruction of soil structure and the decaying characteristics of vertical drains during consolidation of sensitive clays (Chen et al., 2007), vacuum preloading and variable permeability (Indraratna et al., 2005, 2012), partial penetration of drains (e.g. Chai et al., 2009; Ong et al., 2012). Some of these effects on radial consolidation were also investigated in experimental studies (e.g. Indraratna and Redana, 1998; Sharma and Xiao, 2000; Sathananthan and Indraratna, 2006; Saowapakpiboon et al., 2011).

Numerical approaches allow accounting for general stress and strain conditions and have also been applied to radial consolidation problems (e.g. Rowe and Soderman, 1987; Indraratna and Redana, 1997; Rowe and Li, 1999; Castelli et al., 2008; Chai et al., 2010). Such methods of analysis require an appropriate description of mechanical soil behaviour and a significant effort in problem formulation and parameter calibration and are frequently used in back-analysis of case-histories.

Other studies addressed different issues relevant to analysis and design of preloading and vertical drains projects such as the evaluation of the equivalent diameter of *PVDs* (e.g. Long and Covo, 1994; Abuel-Naga and Bouazza, 2009), new types of drains (Liu and Chu, 2009; Karunaratne, 2011), staged construction of high embankments (Sinha et al., 2009) and verification of installation depth (Liu et al., 2009).

In the framework outlined by the above-mentioned studies dealing with the technique of preloading and drainage through prefabricated vertical drains, this paper describes a well-documented case history concerning the improvement of the foundation soil of two oil tanks. The case history allows highlighting the influence of soil heterogeneity and time-dependent loading on the results of the soil improvement technique in terms of differential settlements and angular distortions of the shallow foundation of the tank.

In-situ investigations, including boreholes and cone penetration tests (*CPTs*), and laboratory tests were carried out to define the site geotechnical profile and soil mechanical properties. Dissipation tests were also carried out during the *CPTs* and were interpreted in order to evaluate the horizontal consolidation and permeability coefficients.

An extensive field monitoring of the site during the preloading period and during the hydraulic leakage test of the tanks is also presented.

From the settlement profiles measured, differential settlements and angular distortions of the tank foundations were evaluated. Comparison of field data with expected profile shapes for tanks overlying homogeneous compressible soil layers and with available empirical relationships is presented, showing a general fair agreement. The lithological and mechanical heterogeneity of the foundation soil deposit affects the tank settlement response. However, absolute and differential settlements as well as angular distortions are consistent with the allowable limits provided by the literature and with the design prescriptions, thus confirming the effectiveness of the preloading and drainage technique adopted in the project and envisaging a satisfactory performance of the tanks under service conditions.

2. Site investigations and geotechnical characterization

Two cylindrical oil tanks, namely tank No. 1 and No. 2, were built in a site located in the alluvial plain south of the city of Catania, Italy.

The site is characterised by alluvial soils to a depth of about 80 m, transported by the small rivers crossing the plain, overlying a thick layer of Pleistocene stiff clays of marine origin. Due to its alluvial nature, the foundation soil of the tanks is heterogeneous both in vertical and horizontal directions.

In-situ investigations and laboratory tests were carried out to define the soil profile and to evaluate soil physical and mechanical properties. Investigations consisted of 8 boreholes (*BH*), 5 cone penetration tests (*CPTs*), two of which with measure of pore pressures (*CPTU*), and laboratory tests. The location of the geotechnical investigations relevant for this study is shown in Fig. 1 with respect to the position of the two tanks. In the figure the area occupied by the preloading embankments is also shown.

2.1. Soil profile

The soil columns retrieved from boreholes along with results from cone penetration tests allowed defining the soil profile shown in Fig. 2. The soil layers encountered during boring from ground surface downward, are:

L1: a 6–8 m thick (depth $z < 6$–8 m) layer of high plasticity (liquid limit $LL > 70\%$ and plasticity index $PI > 35\%$) and lightly overconsolidated clay and silt;
L2: a 12–15 m thick layer (varying locally from $z = 6$–8 m to $z = 21$–22 m) of high plasticity ($LL > 60\%$, $PI > 35\%$) clay and sandy silt;
L3: a 1 m thick ($z = 20$–22 m) discontinuous layer of sand;
L4: a 4–6 m thick ($z = 21$–27 m) layer of high plasticity clay and sandy silt;
L5: a thin (less than 1 m) layer of silty sand ($z = 27$–28 m);
L6: a silt with sandy clay layer extending from about $z = 28$ m to the maximum investigated depth of 40 m, with a small fraction of gravel in the last 2 m observed in *BH*8.

According to the piezometric measurements carried out at the site, the water table is located at a depth of 3 m from the ground surface.

Fig. 3 shows the profiles of the cone and sleeve resistance, q_c and f_s, of pore water pressure u, of the pore pressure ratio $B_q = \Delta u/(q_t - \sigma_{vo})$, of the normalized cone resistance $Q_t = (q_t - \sigma_{vo})/\sigma_{vo}'$ and of the normalized friction ratio $F_r = f_s/(q_t - \sigma_{vo})$ as obtained from the two piezocone tests *CPTU*1 and *CPTU*2.

In the definition of Q_t and B_q, Δu is the excess pore pressure induced by the cone penetration, q_t is the cone resistance corrected for pore pressure effect and σ_{vo} and σ_{vo}' are the vertical total and effective stress, respectively.

The profiles of the cone resistance (Q_t, q_t) combined with the friction ratio (F_r) and the pore pressure ratio B_q, were used to detect soil stratigraphy and for soil classification according to the procedure proposed by Robertson (1990).

Fig. 4 shows the classification charts proposed by Robertson (1990) adopted herein with the data obtained from *CPTU* tests.

The profiles and the charts of Figs. 3 and 4 show that the upper part of the soil deposit consists mainly of lightly overconsolidated clayey and silty–clayey soils (zone 3 in Fig. 4); however, the presence of levels of silty and sandy soils (zones 4–5 in Fig. 4b) or even sands (zones 6–7 in Fig. 4b) is apparent in the charts.

The data derived from *CPTU* tests points out the overall heterogeneity of the soil deposit under consideration where the clayey and the silty–clayey soils are frequently alternated to