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ORIGINAL ARTICLE

Comparison of measured and calculated consolidation settlements of thick underconsolidated clay

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Abstract This study investigates the consolidation settlement of thick deep deposit of underconsolidated clay encountered east of Port Said in Egypt. The foundation soil of the studied area includes a 35 m thick deposit of very soft to medium stiff silty clay. Calculated settlements for a container terminal constructed in this area are compared with two years of field measurements. Consolidation parameters were defined for this site from laboratory and cone penetration tests (CPT). Upper and lower bounds of calculated settlements were calculated using one-dimensional consolidation theory for the range of working container loads. Settlement monitoring was conducted using settlement plates at eight (8) locations. Field measured settlements were compared to calculated settlements to validate the soil properties and evaluate the rationality of the calculated settlements. Field measured settlements fell within the upper and lower bounds of the calculated settlements. The results of this study confirmed that the deep clay deposit is underconsolidated, which poses a geotechnical challenge to potential construction in this area due to expected excessive settlements. In addition, the study showed that applying the one-dimensional consolidation theory using consolidation parameters estimated from CPT and laboratory tests for underconsolidated clays reasonably estimated the magnitude and rate of consolidation settlement.

1. Introduction

East of Port Said is considered an important logistical area due to its location at the northern end of the Suez Canal with high volumes of trade traffic. The area currently includes a large container terminal and is envisaged to grow into a large hub for trade and industry, which entails large infrastructure projects and heavy construction. The foundation soils in the area of the container terminal provide a geotechnical challenge due to the underconsolidated nature of the deep clay deposits that have thicknesses in excess of 35 m. The deep clays have been loaded with dredged soils from an adjacent bypass canal. This

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exposes potential construction activities to challenges of excessive settlements and associated safety and serviceability consequences.

Evaluation of expected settlements depends on the determination of consolidation parameters, which could be evaluated by laboratory tests, field tests, and empirical correlations. Consolidation parameters interpreted from laboratory tests could be subject to inaccuracies resulting from factors such as sample disturbance, sample size, and strain rate [9,16,18]. For field tests, existing correlations are typically used to interpret consolidation parameters. To evaluate inaccuracies in settlement calculations using laboratory and field testing, field settlement monitoring has been conducted at different sites, and was used to compare actual to calculated settlements based on laboratory and field tests [1,2,7,13,17,20].

Several attempts have been made to calculate consolidation settlements using parameters interpreted from in situ cone penetration tests. Oakley and Richard [17] found that calculated settlements using cone penetration test (CPT) data compared well to actual settlements, but the calculated time rates of settlement were within 150% of the actual field measurements. Crawford and Campanella [7] compared measured settlements of earth embankments with settlements calculated from laboratory consolidation tests, in situ Piezocone tests, and dilatometer tests. The authors mentioned that there was good agreement between the settlements calculated using the three methods; however, the actual settlement was approximately 60% greater than the average calculated value.

Liu et al. [13] compared measured settlements at eight embankments sites to calculated settlements based on laboratory tests. The authors found that the calculated settlements based on laboratory tests underestimated the actual settlements in six sites by 13–72%. Abu-Farsakh et al. [2] and Abu-Farsakh and Yu [1] compared calculated settlements based on laboratory tests and Piezocone penetration tests to field measured settlements of instrumented embankments. The authors found that the settlement calculation based on laboratory and Piezocone tests tended to over predict the actual settlement. However, Abu-Farsakh et al. [2] and Abu-Farsakh and Yu [1] reported that values of coefficient of consolidation measured in the laboratory and back-calculated from field measurements showed some scatter but were within the same log cycle. In the first month, the laboratory calculation showed the largest settlement rate; while in the following months, laboratory and field measurements showed almost similar settlement rates. Purzin et al. [20] mentioned that values of coefficient of consolidation may vary by two orders of magnitude from the field values, with the field exhibiting more pervious behavior.

2. Geotechnical data

The site understudy is located near the Mediterranean Sea along the east side of the Port Said East Canal, a side channel east of the Suez Canal referred to as "Sharq El Tafreea" in Port Said, Egypt (Fig. 1). The area surrounding the project was known before the construction of the Suez Canal as "El-Tinah" Plain. Recent silty and clayey materials have been deposited at the area of the site during dredging of the nearby canal. The site covers an approximate area of 184 m by 850 m. Site investigation was conducted in 2010, which included drilling six (6) boreholes from which samples were extracted using 3-inch diameter Shelby tubes for laboratory testing. In addition, six (6) cone penetration tests (CPT) were conducted. Boreholes extended to depths ranging from 40 m to 70 m; and CPTs extended to depths ranging from 30 m to 40 m.

The stratigraphy based on the interpretation of the boring logs, CPTs, and laboratory tests consisted of silty fine sand resulting from dredging nearby canal with thickness ranging from 1.2 m to 6.1 m (Unit 1), followed by very soft to soft silty clay also resulting from dredging with thickness ranging from 4.1 m to 11.9 m (Unit 2). The clay layer is underlain by medium dense to dense silty fine sand with thickness ranging from 0.8 m to 7.3 m (Unit 3), followed by alternating thin layers of silty clay, silt, and silty fine sand with thickness ranging from 3.2 m to 7.0 m (Unit 4). These alternating thin layers are underlain by very soft to medium stiff silty clay with thickness ranging from 34.5 m to 36.8 m (Unit 5). The lower thick clay layer is underlain by very dense sand with thickness ranging from 9.2 m to 13.8 m (Unit 6).

Based on the conducted boreholes and CPTs, it was noted that there is significant spatial variability within the site regarding the thicknesses of the first four (4) units overlying

Figure 1 Location of site under study in Egypt.

the thick clay deposit of Unit 5. The groundwater table is recorded at an average depth of 6.7 m (at water level in nearby canal), which corresponds to an elevation of 0.5 m above chart datum. A representative CPT sounding recorded at the site along with the profile from the nearest borehole is presented in Fig. 2.

Soil replacement was applied at the site to remove the dredged deposits forming Units 1 and 2. Compacted sand was placed on top of Unit 3 to reach the target design elevation of the container terminal. Thus, consolidation settlements only result from compressibility of Units 4 and 5. Variation of liquid limit, plastic limit, and water content with depth for Units 4 and 5 is shown in Fig. 3.

The saturated unit weight of the entire soil profile was evaluated from laboratory testing, and from the CPT data using the following correlation [15]:

$$\gamma_t = \gamma_w[1.96 + 0.25\log(\sigma'_{vo}/p_a) + 0.265\log(f_s/p_a)] \quad (1)$$

where γ_t is saturated unit weight, γ_w is water unit weight, σ'_{vo} is the effective overburden stress, and p_a is the atmospheric pressure. The interpreted saturated unit weights of the top five layers from top to bottom are 17, 17, 18.5, 17.5, and 17 kPa.

Compressibility parameters of Units 4 and 5 were evaluated from CPT and/or laboratory tests. Cone penetration tests were used to evaluate the overconsolidation ratio (OCR) based on the correlation proposed by Lunne et al. [14] as follows:

$$\mathrm{OCR} = k\ Q_t \quad (2)$$

$$Q_t = (q_t - \sigma_{vo})/(\sigma'_{vo}) \quad (3)$$

where q_t is the cone tip resistance corrected for pore pressure effect, σ_{vo} is the total overburden stress, and k is a constant ranging between 0.2 and 0.5 with a recommended value of 0.3 [3,14]. A total of eleven (11) one-dimensional consolidation tests were performed in accordance to ASTM D2435 [4] to

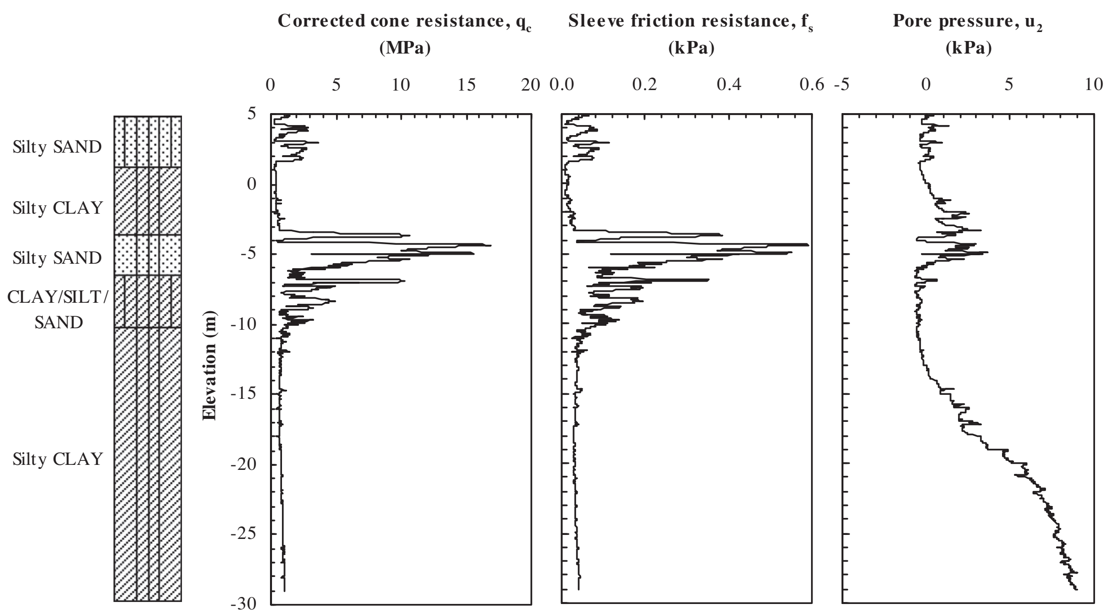

Figure 2 Representative CPT sounding along with soil profile from nearest borehole.

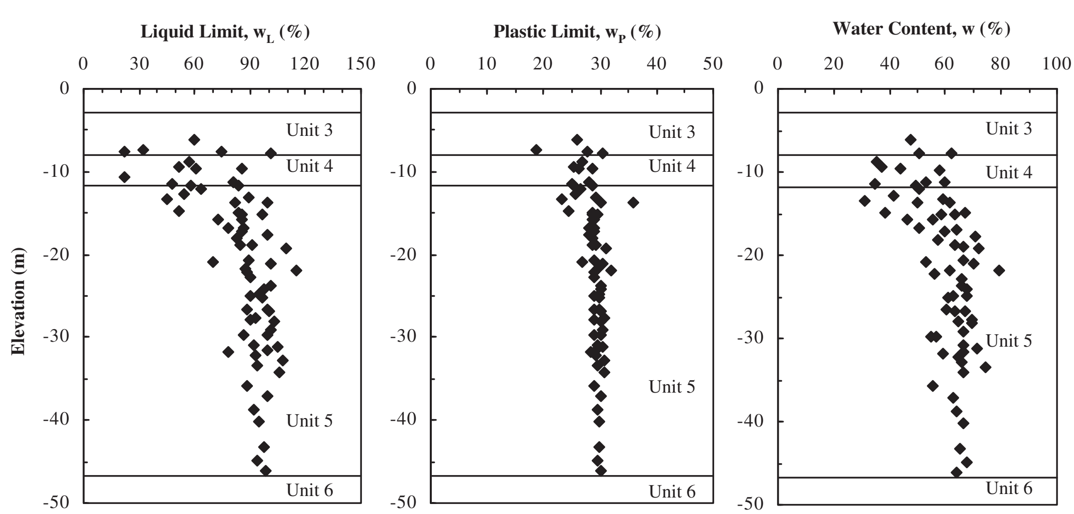

Figure 3 Variation of liquid limit, plastic limit, and water content with elevation.

evaluate the compressibility of Unit 5. The laboratory test results were used to evaluate the clay void ratio (e), compression index (C_c), OCR, and coefficient of consolidation (c_v). It is noted that the recompression index (C_r) of Unit 5 is not of importance as this clay layer is underconsolidated.

In addition, well established empirical correlations were used to determine C_c, C_r, c_v, and $C_{\alpha\varepsilon}$. Compression and recompression indices were calculated from the liquid limits and plasticity indices using the following correlations [12]:

$$C_c = 0.009(w_L\text{-}10) \tag{4}$$

$$C_r = PI/370 \tag{5}$$

where w_L is the liquid limit in percent and PI is the plasticity index in percent. Compression and recompression indices were normalized with respect to void ratio (Eqs. (6) and (7), respectively).

$$C_{c\varepsilon} = C_c/(1 + e_o) \tag{6}$$

$$C_{r\varepsilon} = C_r/(1 + e_o) \tag{7}$$

The coefficient of consolidation was determined based on the soil liquid limit [12]. The coefficient of secondary compression for Unit 4 was determined using correlation with natural water content presented as follows [12]:

$$C_{\alpha\varepsilon} = C_\alpha/(1 + e_{100}) = 0.0001w \tag{8}$$

where w is the water content in percent and e_{100} is the void ratio at end of primary consolidation. In addition, $C_{\alpha\varepsilon}$ was calculated using correlation with C_c, where $C_\alpha/C_c = 0.05$ as reported by Terzaghi et al. [21] for clays and silts. It is noted that the coefficient of secondary consolidation of Unit 5 was not considered in the settlement evaluation as this thick clay layer is underconsolidated and is not expected to reach end of primary consolidation within the time frame under study.

Based on the CPT and laboratory tests as well as the empirical correlations, compressibility parameters were concluded for Units 4 and 5. Variation of normalized compression index with depth is shown in Fig. 4, and variation of OCR with depth from CPT and laboratory tests is shown in Fig. 5. It is noted that the very soft clay of Unit 5 was not tested in the laboratory for consolidation properties. The characteristics of the very soft clay within Unit 5 were evaluated using either the CPT or empirical correlations based on index properties.

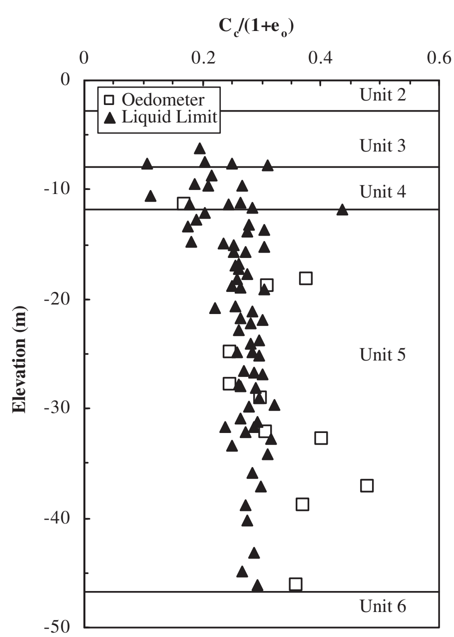

Figure 4 Variation of normalized compression index with elevation.

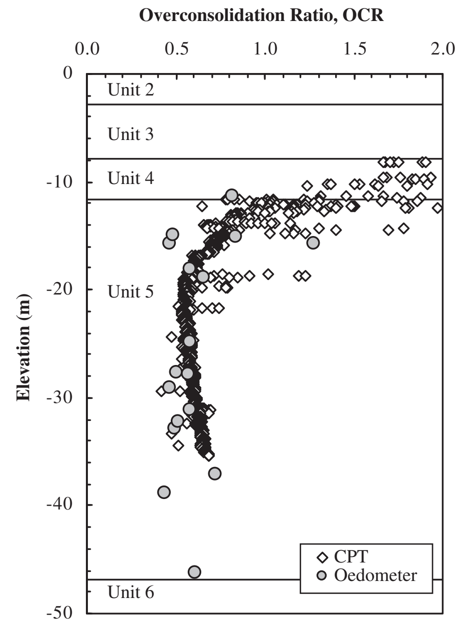

Figure 5 Variation of overconsolidation ratio with elevation from CPT and oedometer tests.

Based on laboratory and field tests conducted on Unit 4, the representative void ratio (e) was estimated to be 1.22, and normalized compression ($C_{c\varepsilon}$) and recompression ($C_{r\varepsilon}$) indices were 0.23 and 0.052, respectively. The representative overconsolidation ratio (OCR) was estimated to be 1.8, coefficient of consolidation (c_v) was 4.42 m^2/year, and the coefficient of secondary compression ($C_{\alpha\varepsilon}$) was 0.0047.

For Unit 5, based on laboratory and field tests, the void ratio (e) increased with depth and ranged from 1.6 to 1.8. Similarly, the compression index (C_c) increased with depth from 0.58 to 0.82. The representative normalized compression index ($C_{c\varepsilon}$) was estimated to be 0.27. It was noted that the overconsolidation ratio varied with depth, as it increased within the top and bottom few meters of Unit 5 due to proximity to freely draining layers causing accelerated consolidation. The top and bottom six meters of Unit 5 had an OCR of approximately 0.8, while the rest of clay layer had an OCR of 0.6.

Based on oedometer tests conducted on Unit 5, coefficient of consolidation (c_v) corresponding to the range of initial and final stresses in the clay layer ranged from 0.31 m^2/year to 0.42 m^2/year. Settlement calculations were conducted twice using the two values of c_v to calculate range of expected settlements. Representative consolidation parameters for Units 4 and 5 are summarized in Table 1. Evaluated consolidation parameters were used to estimate the magnitude and rate of settlement under design loads.

3. Methodology of settlement calculation

The area under study was preloaded with soils from Units 1 and 2, which are removed and replaced with compacted back-

Table 1 Representative consolidation parameters of Units 4 and 5.

Parameter	Unit 4	Unit 5
Void ratio (e)	1.22	(1.6–1.8)
Normalized compression index ($C_{c\varepsilon}$)	0.23	0.27
Normalized recompression index ($C_{r\varepsilon}$)	0.052	–
Overconsolidation ratio (OCR)	1.8	0.8 for top and bottom 6 m, and 0.6 for middle of the clay layer
Coefficient of consolidation (c_v)	4.42 m^2/year	(0.31–0.42) m^2/year
Coefficient of secondary compression ($C_{\alpha\varepsilon}$)	0.0047	–

fill to carry a range of working container loads ranging from 0 to 60 kPa. Dewatering was applied before commencement of excavation to lower the groundwater table below the bottom of the target excavation level by approximately 0.5 m. The dewatering lines were removed as the backfilling elevation exceeded the natural groundwater elevation. The load distribution due to containers stacking is shown in Fig. 6. The load due to containers stacking was modeled as infinite strip of distributed uniform load (q) on the surface of a semi-infinite mass. The stresses increment due to container loading at any point in the soil mass underneath the container terminal were calculated assuming superposition of stresses from infinite strips on an elastic semi-infinite mass [19].

Immediate settlements of the different layers were calculated as follows [6]:

$$S_i = \Sigma \frac{1}{E_{si}} \Delta p_i H_i \tag{9}$$

where E_{si} is the constrained modulus of each layer, H_i is the layer thickness, and Δp_i is the stress at centerline of each layer due to additional design load. The constrained modulus (E_s) was calculated as follows:

$$E_s = \frac{(1-v)E}{(1+v)(1-2v)} \tag{10}$$

where E is the modulus of elasticity and v is Poisson's ratio. For saturated clay layers, Eq. (10) would produce negligible settlements as Poisson's ratio approaches 0.5.

Consolidation settlements of the clay layers were calculated using equations for one-dimensional consolidation settlement. Consolidation was divided into two phases: primary consolidation and secondary consolidation. Note that the removal of the fill layer and placement of backfill was assumed to occur in a relatively short period, i.e. undrained conditions.

Settlements resulting from primary consolidation were calculated using the general form of the settlement equation as given below [10]:

$$S_p = \frac{C_r}{1+e_o} H_o \log\left(\frac{\sigma'_{vo} + \Delta\sigma}{\sigma'_{vo}}\right), \text{ for } \sigma'_{vo} + \Delta\sigma < p_c \tag{11}$$

$$S_p = \frac{C_r}{1+e_o} H_o \log\left(\frac{p_c}{\sigma'_{vo}}\right) + \frac{C_c}{1+e_o} H_o \times \log\left(\frac{\sigma'_{vo} + \Delta\sigma}{p_c}\right), \text{ for } \sigma'_{vo} < p_c \text{ and } (\sigma'_{vo} + \Delta\sigma) > p_c \tag{12}$$

where S_p is the primary settlement; C_c is the compression index; C_r is the recompression index; e_o is the initial void ratio; H_o is the initial thickness of compressible layer; σ'_{vo} is the initial vertical effective stress; p_c is the preconsolidation pressure; and $\Delta\sigma$ is the increment of vertical effective stress due to additional load.

Settlements resulting from secondary compression of Unit 4 were calculated according to the following equation [10]:

$$S_t = C_{\alpha\varepsilon} H_{100} \log\left(\frac{t_2}{t_1}\right) \tag{13}$$

where S_t is the time dependent secondary settlement; $C_{\alpha\varepsilon}$ is the secondary compression index; H_{100} is the thickness of compressible layer at end of primary consolidation; t_1 is the time when secondary compression is assumed to begin (assumed to be t_{100} = time to reach end of primary consolidation); and t_2 is the time for which secondary settlements are calculated (assumed to be 50 years).

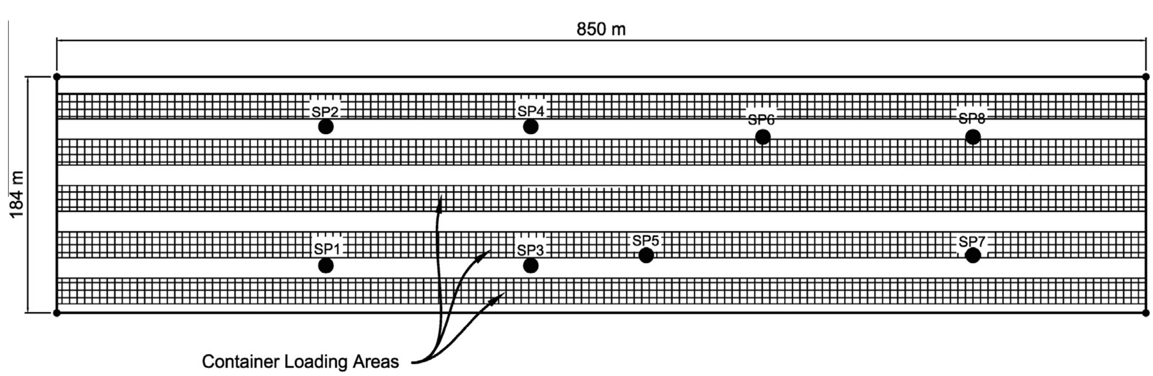

Figure 6 Load distribution due to containers stacking and settlement plate locations.

4. Calculated settlements

In this paper, settlements were calculated at eight (8) locations within the site where field measurements were conducted using deep settlement plates (Fig. 6). The thickness of each soil layer at studied points was interpreted based on the conducted (6) boreholes and (6) CPTs. The interpreted soil profiles at the eight locations are presented in Fig. 7. For settlement calculations, the thick clay layer (Unit 5) was divided into 6 equal sublayers. Variation of settlement with time was calculated for the two c_v values that cover the range of measured values for Unit 5 for container loads of 0 and 60 kPa, and are presented for the eight (8) studied locations in Fig. 8.

The calculated settlements at the eight locations over the period of 50 years ranged from approximately 180 mm to 560 mm for the considered range of loads and c_v values. Even in cases of no container loading (q = 0 kPa) and using minimum values of measured c_v, the calculated settlements over 50 years ranged from approximately 180 mm to 410 mm. This is due to the underconsolidated nature of soil Unit 5, which results in significant settlements under the own weight of the existing soil profile. The assumed values of c_v and OCR seem to be reasonable based on the loading history of Unit 5. Unit 5 was loaded with dredged soil Units 1 and 2 approximately 25 years ago. Assuming that Unit 5 was originally normally consolidated, and evaluating one-dimensional consolidation under the added stress from dredged soils over the period of 25 year period using the above deduced c_v values, the estimated average degree of consolidation (U) of Unit 5 is approximately 15%. This degree of consolidation corresponds to an average OCR on the order of approximately 0.65, which is in reasonable agreement with estimated values based on laboratory and field tests.

Comparing the calculated settlements at the eight (8) settlement plate locations, it is noted that Settlement Plate 2 (SP2) exhibited minimum settlements, while SP5 showed maximum settlements. By investigating the soil profile at the locations of SP2 and SP5, it was found that the elevation of

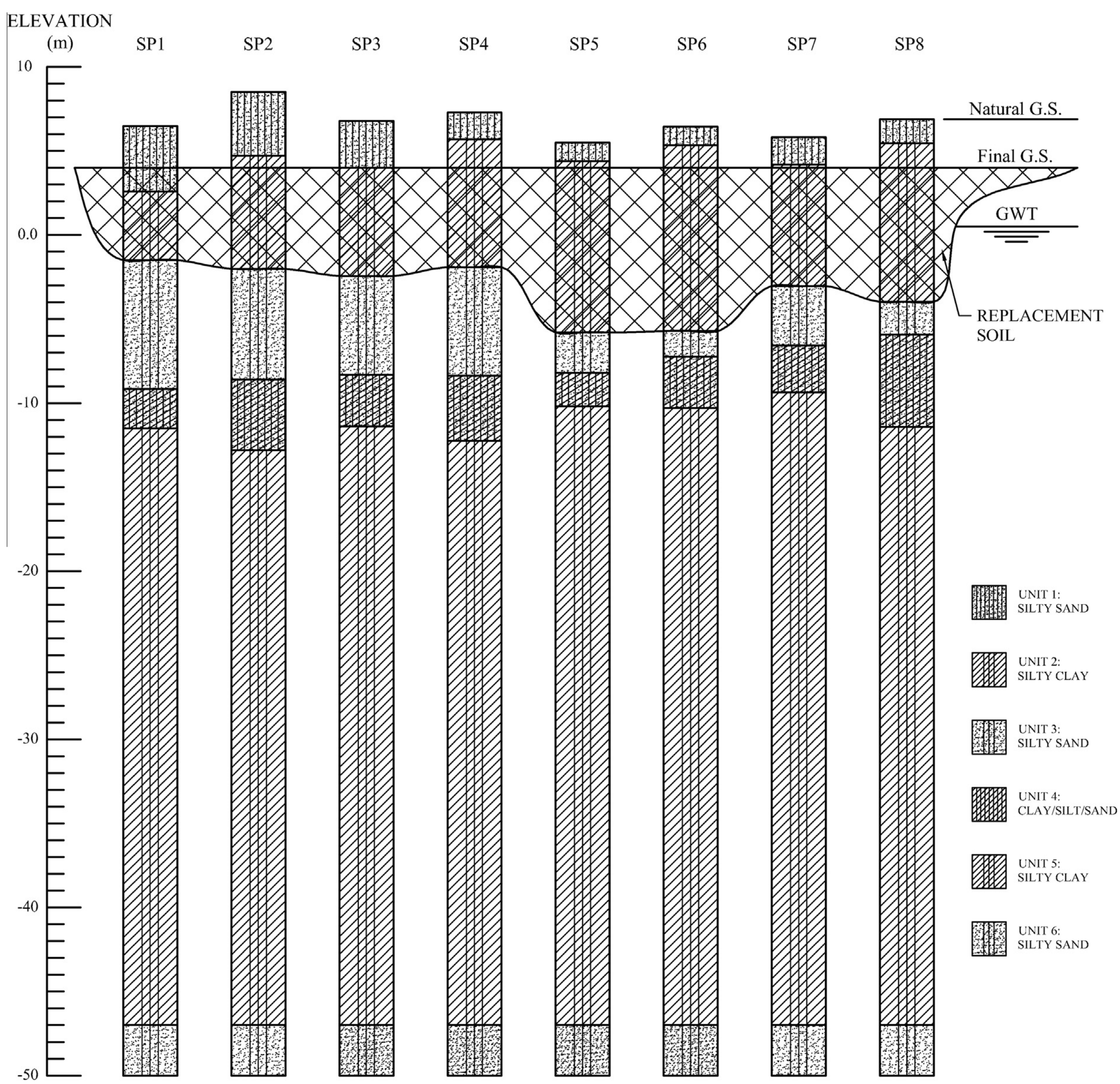

Figure 7 Interpreted soil profile at locations of eight deep settlement plates.

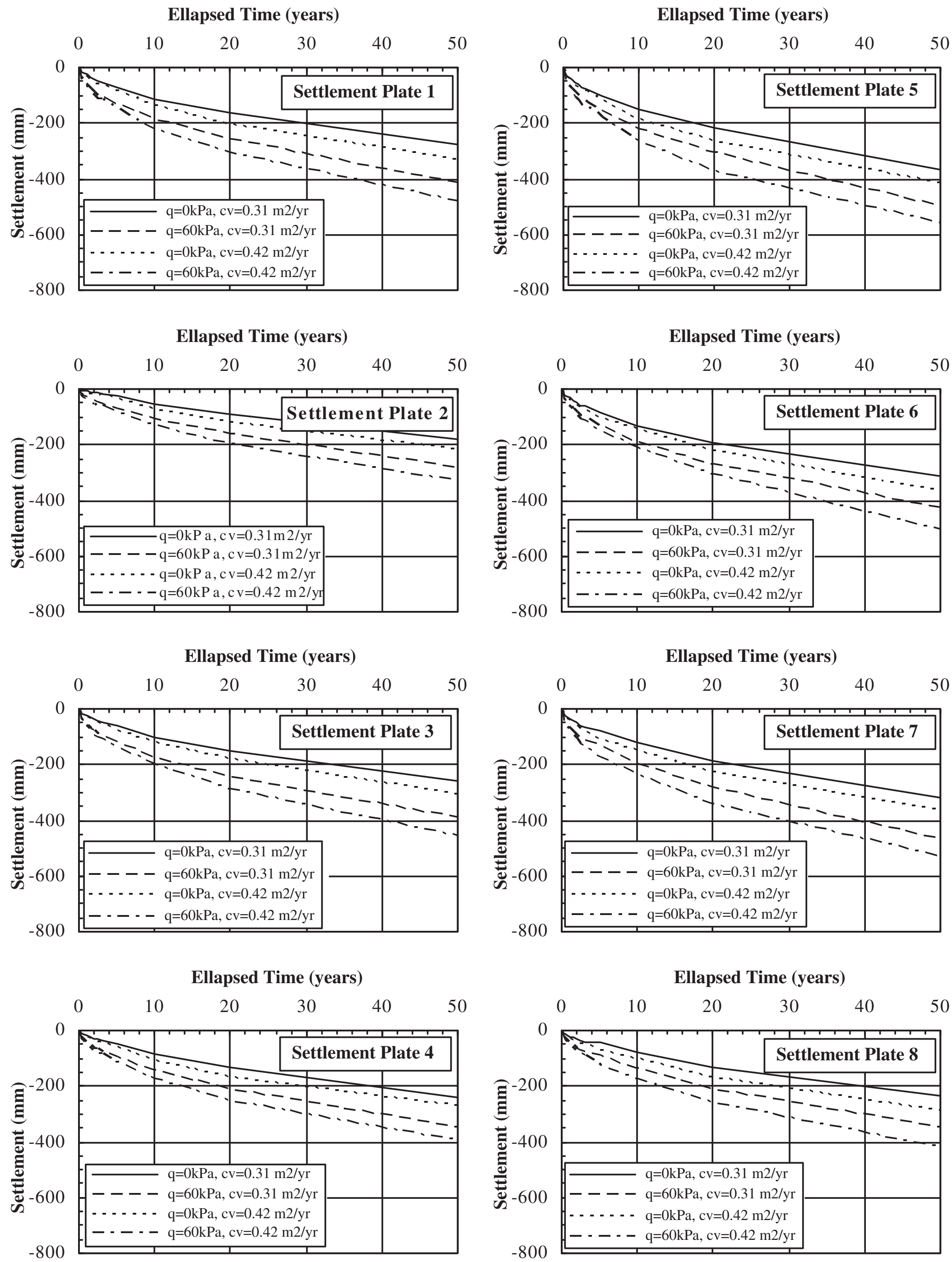

Figure 8 Calculated settlements at locations of 8 settlement plates.

the original ground surface at SP2 was the highest (+8.5 m) and at SP5 was the lowest (+5.5 m). The finished elevation of the container terminal is at elevation (+3.3 m) on average. The higher original ground surface at SP2 resulted in higher preconsolidation pressure and thus lower settlements compared to other settlement plates; and vice versa for SP5. Other less effective factors causing lower settlements at SP2 location are related to the depth and thickness of Unit 5 being deeper and thinner compared to other locations. In addition, the location of SP2 and SP5 relative to the container stacks resulted in lower additional loading acting at SP2 relative to SP5. However, it is worth noting that the impact of differences in container stacking configuration diminishes at depths close to the mid-depth of Unit 5. For example, under a surcharge load of 60 kPa, the additional stresses due to container stacking at the mid-depth of Unit 3 equaled 18.7 kPa and 39.4 kPa at SP2 and SP5, respectively; while at the mid-depth of Unit 5, the additional stresses due to container stacking equaled 30.5 kPa and 31.5 kPa at SP2 and SP5, respectively.

5. Field monitoring

Deep settlement plates were installed on top of Unit 3 prior to placing any fill to monitor settlements of soils underlying the soil replacement layer. Different plate sizes have been reported in the literature, typically square plates with sides ranging from 0.5 m to 1.0 m [5,8,11]. The deep settlement plates consisted of 0.6 m × 0.6 m galvanized steel plate welded to a galvanized steel riser pipe having a diameter of 25 mm (1-in.) extending from the base to the top of the fill as shown in Fig. 9. The riser pipe was extendable in 0.6 m increments as layers of fill are placed and compacted. The riser pipe was incased in a sleeve (2-in. galvanized steel pipe) through the full thickness of the replacement soil and super-imposed pavement layers. Thus,

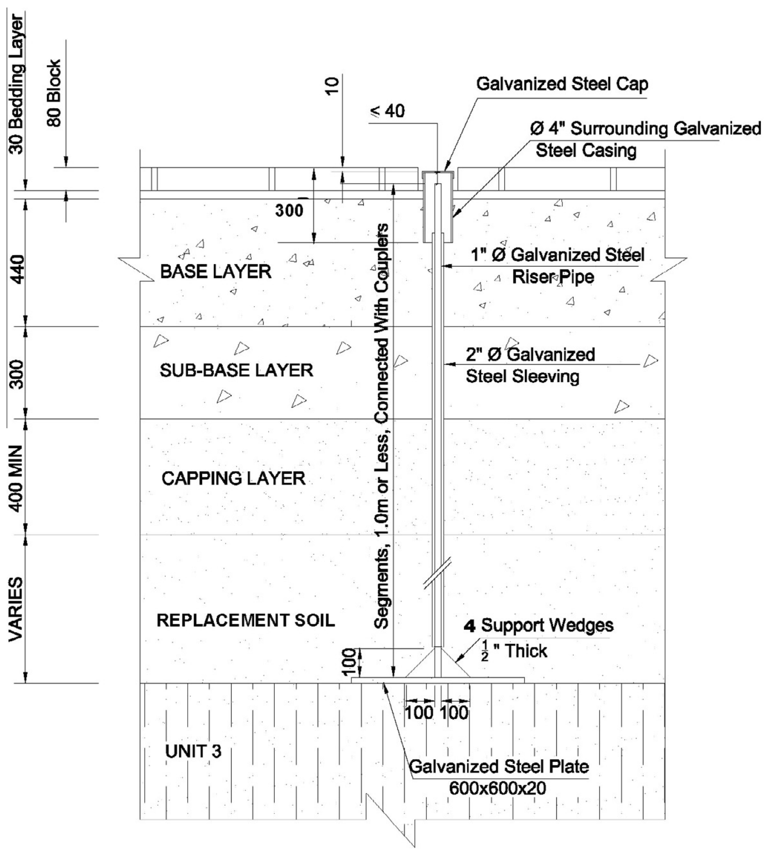

Figure 9 Details of deep settlement plate.

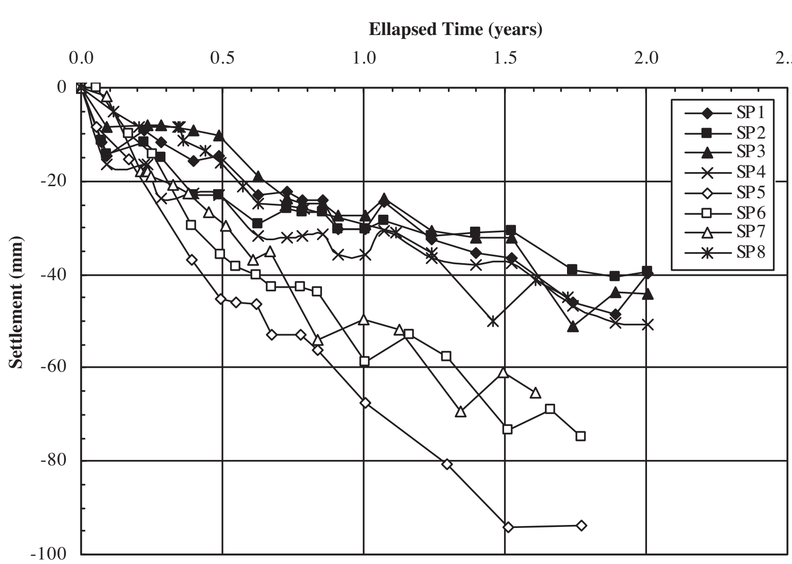

Figure 10 Measured settlements at locations of eight settlement plates.

friction and interaction between the riser pipe and the surrounding earthworks were minimized. An outer casing 100 mm (4-in.) in diameter surrounded approximately the top 300 mm of the sleeved riser pipe. The casing was equipped with a galvanized steel cap to protect the riser pipe from accidental interference or damage due to site activities, construction plant, site traffic, or other causes. The steel cap was removed to record the elevation of the top of the riser pipe as needed (relative to fixed survey monuments remote from the loaded areas). Surveying the top of riser elevation provided a record of the plate settlement. Settlement readings were obtained by means of precise leveling to an accuracy of 1 mm. The locations of the eight settlement plates are shown in Fig. 6.

Field monitoring of settlements is reported herein starting from the end of soil replacement works. Monitoring points were surveyed after the completion of the final grade for a period of two years at a frequency of approximately one month. Variation of recorded settlements at the eight locations with time over the monitored period is presented in Fig. 10.

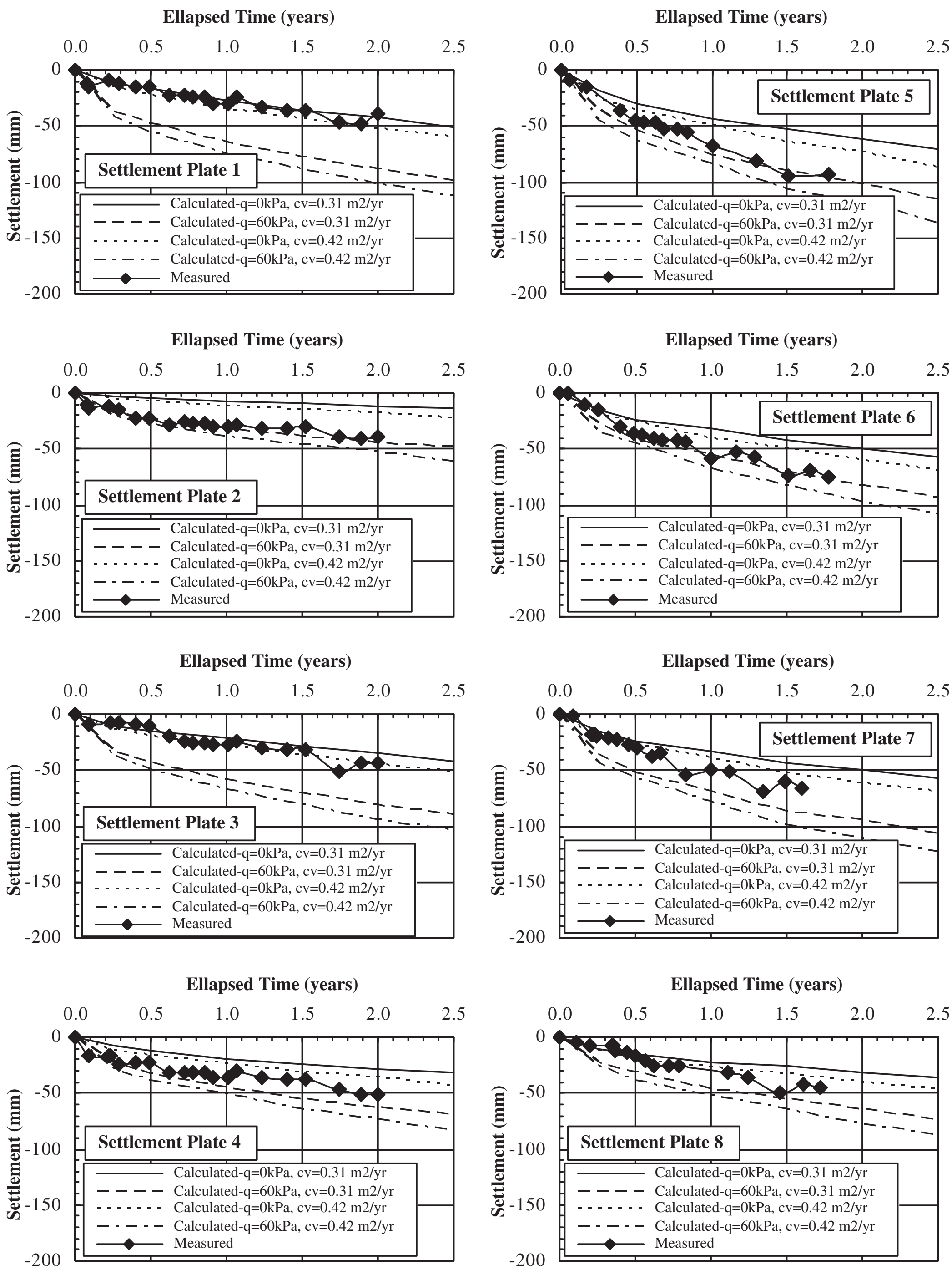

Figure 11 Comparison between calculated and measured settlements at locations of eight settlement plates.

As expected, the recorded settlements indicate increasing settlement with time at an overall decreasing rate. Based on the observed settlements, the settlement plates can be clustered into two groups. The first includes SP1, SP2, SP3, SP4, and SP8, with relatively lower settlement rates. The second includes SP5, SP6, and SP7, with relatively higher settlement rates.

The lowest settlements were observed at the location of SP2 especially after 0.75 years; while the highest settlements occurred at the location of SP5. The lowest and highest settlements at SP2 and SP5, respectively, are attributed to the same reasons discussed in the previous section related to the original ground surface elevation in addition to the soil profile and loading conditions. It is worth noting that the loading from the container stacks was not constant with time nor consistent among all settlement plates locations, but was limited to 60 kPa. This might explain the irregularities observed in the settlement curves of the different settlement plates.

The monitored settlements showed some heave at the locations of SP2, SP3, SP4, SP6, and SP7 after approximately 1 year. The observed heave could be related to the reduction in final stresses acting on Units 4 and 5 due to the unloading caused by soil removal from the original ground surface to reach the final terminal level. This unloading resulted in swelling of the normally to slightly overconsolidated Unit 4. However, Unit 5 remained underconsolidated, which resulted in continuous compression of Unit 5. The settlement records from the deep settlement plates were taken at the top of Unit 3; thus, the measured settlements included the swelling of Unit 4 and compression of Unit 5. The swelling of Unit 4 and compression of Unit 5 occurred at different rates due to the variation in layer thicknesses and coefficients of consolidation, which may have resulted in the observed heave after approximately 1 year.

6. Comparison between calculated and measured settlements

Calculated settlements during the first two (2) years of loading are compared to measured settlements at the locations of the eight settlement plates in Fig. 11. Measured settlements seemed to be within the range of settlements calculated using results of field and laboratory tests under the range of container loads and measured range of c_v values. Settlements monitored at the locations of SP1 and SP3 were closer to the lower bound of calculated settlements corresponding to q of 0 kPa. On the other hand, recorded settlements at SP5 and SP6 were closer to the upper bound of the calculated settlements corresponding to q of 60 kPa. It is evident that the ranges of container loads and c_v values provide reasonable boundaries to the expected settlement.

The reasonable agreement between the calculated and measured settlements increases the confidence in the interpretation of the state of the underconsolidated soil Unit 5, and the selected compressibility parameters for Unit 4 (OCR, $C_{c\varepsilon}$, c_v, $C_{r\varepsilon}$, and $C_{\alpha\varepsilon}$) and Unit 5 (OCR, $C_{c\varepsilon}$, and c_v). Note that Unit 5 is underconsolidated, therefore, $C_{r\varepsilon}$ and $C_{\alpha\varepsilon}$ do not affect the results. Similar agreement between calculated settlements using CPT data and actual settlements were reported by Oakley and Richard [17]. However, Crawford and Campanella [7] and Liu et al. [13] reported higher actual settlements than those calculated based on laboratory and field tests. On the other hand, Abu-Farsakh et al. [2] and Abu-Farsakh and Yu [1] found that calculated settlements based on laboratory and CPT tests tended to over predict the actual settlements.

Over the monitored period, measured and calculated settlements presented in Fig. 11 seem to follow similar settlement rates. Similar observation was reported by Abu-Farsakh et al. [2] and Abu-Farsakh and Yu [1]. However, Purzin et al. [20] mentioned that field values of coefficient of consolidation may be higher than laboratory values by two orders of magnitude.

7. Conclusions

The foundation soils in the investigated coastal area of east of Port Said impose a geotechnical challenge due to the presence of underconsolidated thick deep clay deposit. This clay deposit exposes potential construction activities to challenges of excessive settlements and associated safety and serviceability consequences. In this study, calculated consolidation settlements of thick deposit of very soft to medium stiff silty clay with thickness of about 35 m underlying a container terminal located east of Port Said in Egypt was compared to field measurements recorded over a period of two years. The properties of the underconsolidated clay were assessed using field and laboratory measurements. Measured settlements over the initial period of two years seemed to be within the range of settlements estimated based on field and laboratory tests under the range of working container loads and predicted range of c_v values. The agreement between calculated and measured settlements increased the confidence in interpretation of the state of the deep clay deposit and the selected soil compressibility parameters based on laboratory and field tests. To date, calculated and actual settlements seemed to follow similar settlement rates. The elevation of the original ground surface had a significant impact on calculated and actual settlements as it directly affected the preconsolidation pressure of the soil deposit. Assessing consolidation behavior and parameters based on field measurements for this site location enables better understanding and prediction of behavior of other structures constructed in the nearby area planned for industrial use with foundation soil of similar characteristics.

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