PERFORMANCE OF A TEST FILL ON SOFT SOIL STABILISED WITH PVD

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Abstract: The behavior of an instrumented test fill is analysed in order to assess the effectiveness of preloading with prefabricated vertical drains (PVD) designed to improve the soil layers at the site of Konya Wastewater Treatment Plant (WWTP) where large pool type structures are planned to be constructed. The foundation layers of the test fill were instrumented with settlement plates, magnetic settlements columns and piezometers, the readings of which are evaluated together with fill placement programme. The field measurements are compared with the results of a finite element analysis modelled to duplicate field loading and drainage conditions. In the coupled analysis performed, elasto-plastic material behaviour is considered and material properties are derived from SPT blow count numbers and laboratory oedometer and unconsolidated undrained (UU) triaxial test results. Observed field behaviour and the results of finite element analysis have shown that the proposed preloading scheme will be sufficient to obtain the required degree of soil improvement and the vertical drains will provide the degree of consolidation within the planned time period. It is also shown that the method of numerical analysis used in this study utilizing the material properties obtained from field and laboratory testing can successfully predict the field behavior.

Keywords: Prefabricated Vertical Drain (PVD), preloading , soft soil , full-scale test, coupled FEM/DEM

INTRODUCTION

In Konya province, Turkey, a waste-water treatment plant (WWTP) is under construction on an approximately 400,000m^2 area. The structures planned to be built are generally semi embedded reinforced concrete water retaining structures extending over large areas and are not expected transmit large pressures to foundation layers. Because the site soil profile comprised highly compressible layers in the upper 12meters, it was decided that soil improvement is needed to limit the total and differential settlements expected to occur under the structures. After a number of alternative soil improvement methods were studied, preloading under a 4m high surcharge loading was proposed to yield results satisfying the project's requirements. In order to reach desired level of improvement within acceptable durations, surcharge loading with prefabricated vertical drains (PVD's) at 2 m x 2 m spacing and extending to 16 m depth were planned and 90-100 days loading durations are predicted to be sufficient. The rate of consolidation during surcharge loading with PVD's is clearly dependent on horizontal permeability of soil layers. Therefore, an instrumented test fill was designed to observe in situ field behaviour and revise the preloading scheme accordingly if the need arose. In this paper the and measured settlements and pore pressures under the test fill at Konya WWTP site are studied and compared with the result of finite element analysis performed prior to and after the test fill.

SITE SOIL CHARACTERISTICS

Geology

Konya WWTP is under construction on an approximately 600m x 600m area located in on an alluvial plain covered with Quaternary deposits. The Pleistocene and Holocene Quaternary formations are up to 400m thick, and are deposited unconformably on Upper Miocene – Pliocene formations.

Soil Profile

It is known to be almost impossible to define a clear stratification and soil layering in thick alluvial deposits such as the ones on which Konya WWTP is being built. Within the depths investigated with soil borings, it is observed that the cohesive soils dominate the soil profile. Highly plastic silty clays (CH) are inter layered with sandy silty clays (CL) of low plasticity. Within these predominantly clayey layers, occasional silty clayey sand and gravel layers are encountered. The ground water level at the site is determined to be at 3.15-6.07m depth from the ground surface.

Site Soil Model

In order to be able to analyse the behaviour of foundation soil layers underneath the structures to be built, a soil model had be defined for the construction site which is covered with thick alluvial deposits showing no distinct layering. The variation of SPT blow count numbers with depth and the CPT soundings along four sections over the site were evaluated and a simplified soil model was developed consisting a soft to medium stiff silty clay layer down to 12-13 m depth, followed by an approximately 6m thick medium stiff silty clayey sand/stiff-very stiff sandy silty clay, underlain by a stiff-very stiff silty clay layer with occasional sandy gravelly deposits. The soil profile underneath the test fill considered in the numerical analyses is shown in Figure 1. The geotechnical properties of the soil layers as depicted from the field and laboratory test results are given in Table 1.

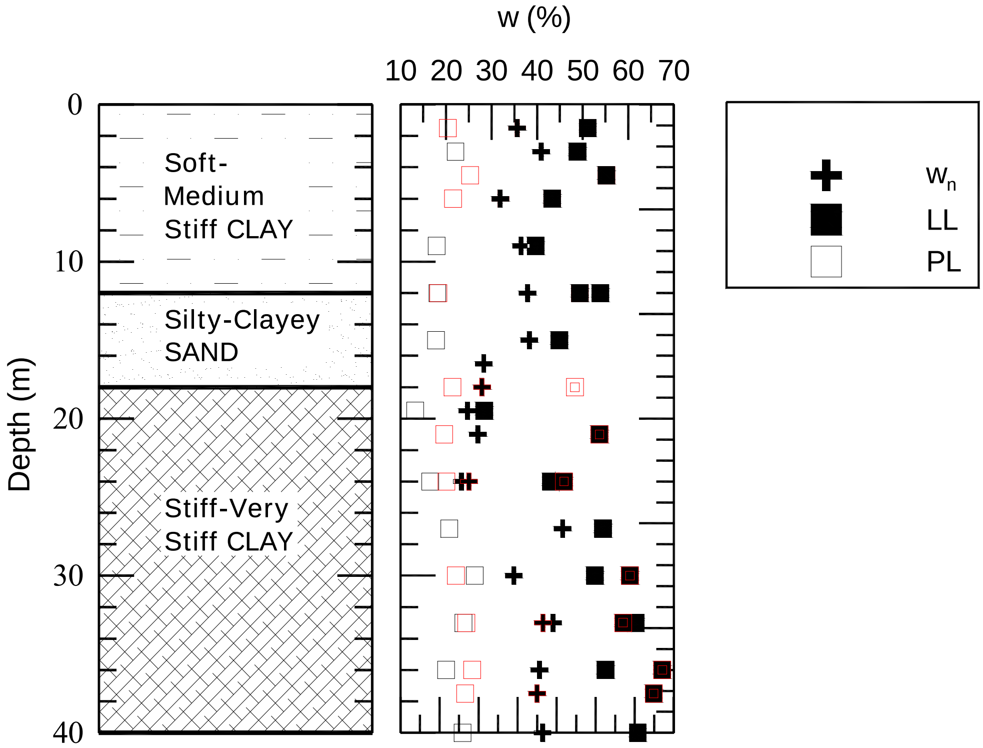

Figure 1. The soil profile underneath the test fill and the variation of natural water content, liquid and plastic limit with depth

Table 1. Geotechnical properties of soil layers as depicted from the field and laboratory tests

Layer No	Depth (m)	c_u kPa	φ′ (°)	k (cm/s)	C_c	C_s
1	0.0-12.0	35	26	5×10^{-7}	0.30	0.04
2	12.0-18-0	75	26	5×10^{-6}	0.35	0.05
3	>18.0	90	25	5×10^{-7}	0.30	0.04

c_u= undrained cohesion (φ_u=0)
φ′ = drained internal friction angle
E′ = drained Young modulus
k = hydraulic conductivity
C_c= compression index
C_s= swelling index

TEST FILL

In order to observe the in-situ behaviour of soil layers under surcharge loading, a 30m x 30m test fill embankment of 4 m height was constructed. Under the test fill PVD's of 16m length at 2m x 2m grid spacing were installed. The PVD's used are 10cm wide and 5mm thick (Colbond drain CX1000). For monitoring the behaviour of soil layers under the test fill, 9 settlement plates, 2 magnetic settlement columns and inclinometers, and piezometers at four different depths in 2 bore holes were installed. The plan of test fill with underlying PVD's and locations of instrumentation are shown in Figure 2.

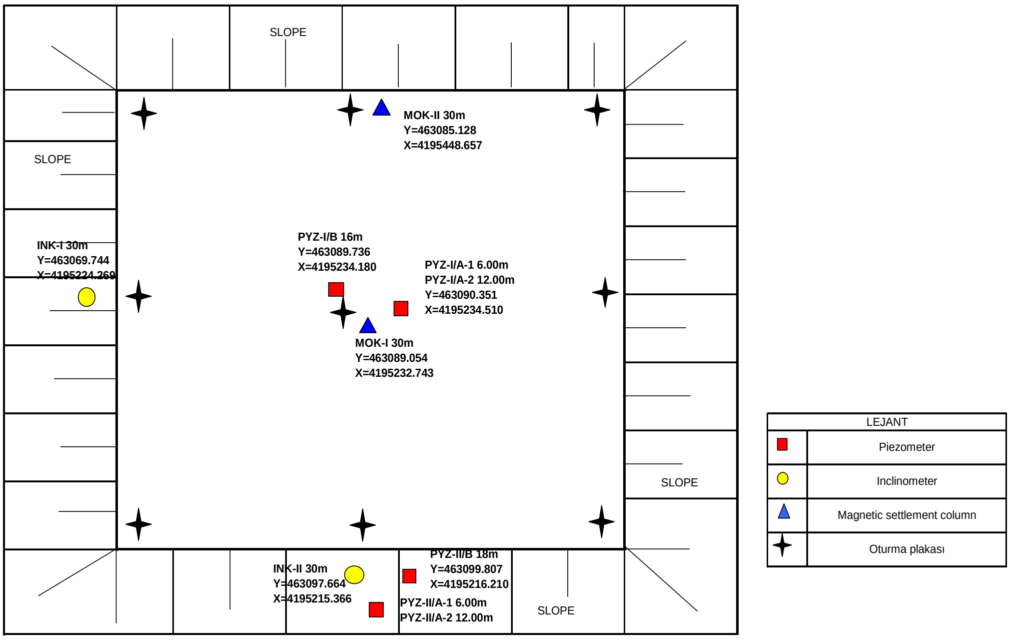

Figure 2. Layout of test fill and instrumentation

THE ANALYSIS OF TEST FILL BEHAVIOUR

In order to be able to interpret the test fill behaviour and to predict the expected settlements under the structures planned to be constructed, coupled finite element analyses were performed utilizing soil parameters consistent with the observed field behaviour. The finite element analyses were carried out using the code named PLAXIS which takes into account the loading stages and models the elasto-plastic soil behaviour.

The finite element analysis is performed with two dimensional idealization. The radial consolidation towards the PVD's installed in the ground is modelled under plane strain conditions by equating the discharge capacities for these two cases. In order to model the axisymmetric flow towards the drain in plane strain idealization (Figure 3), the relationship proposed by Indraratna and Renada (1997) has been used neglecting the smear zone and well resistance effects:

$$\frac{k_{hp}}{k_h} = \frac{0.67}{[\ln(n) - 0.75]} \tag{1}$$

where k_{hp} and k_h are horizontal hydraulic conductivity for plane strain idealization, and hydraulic conductivity for radial consolidation, respectively; n is the ratio of drain influence radius to drain radius (R/r_w). For PVD's the equivalent radius is obtained as

$$d = \frac{a+b}{2} \; and \;\; r = \frac{d}{2} \tag{2}$$

where a and b are width and thickness of the drain, respectively.

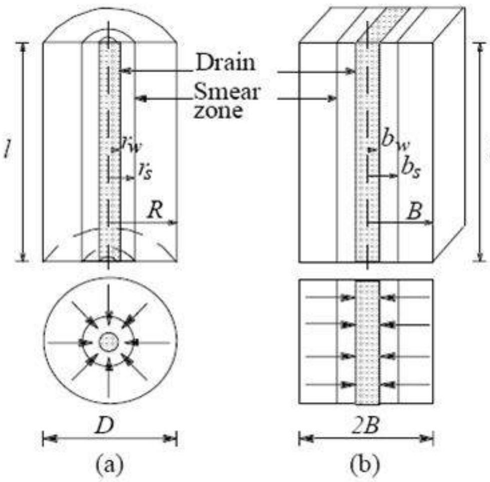

Figure 3. Drain idealization a) axisymmetric b) plane strain

The finite element model used in the analysis is shown in Figure 4 and the material properties are given in Table 2. Elasto-plastic soil behaviour is modelled with the Hardening Soil Model which is an advanced version of the Duncan and Chang (1979) model (Schanz, 1990).

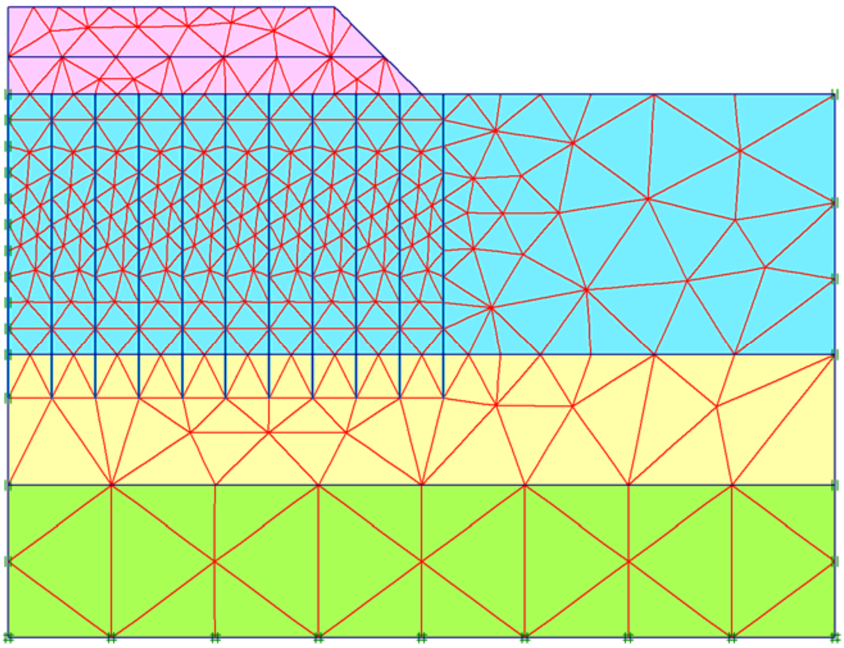

Figure 4. The finite element model used in the analysis

Table 2. Material parameters used in FE Analysis

Parameter		Unit	Layer 1	Layer 2	Layer 3
Unsaturated unit weight	γ_{unsat}	kN/m^3	18	18	19
Saturated unit weight	γsat	kN/m^3	18	19	20
Lateral hydraulic conductivity	k_x	m/day	9.34x10^{-05}	0.000934	0.000432
Vertical hydraulic conductivty	k_y	m/day	0.000432	0.00432	0.000432
Secant Young modulus	E_{50}^{ref}	kN/m^2	11000	30000	28500
Oedometer modulus	E_{oed}^{ref}	kN/m^2	11000	30000	28500
Unloading-reloading modulus	E_{ur}^{ref}	kN/m^2	33000	90000	84500
Referaence cohesion	c^{ref}	kN/m^2	10	1	10
Internal friction angle	ϕ	°	26	33	25
Dilatancy angle	ψ	°	0	3	0
Poisson rate	ν_{ur}	-	0.2	0.2	0.2
Reference stress	p^{ref}	kN/m^2	100	100	100
Power for stress level dependency	m	[-]	0.7	0.5	0.8
Earth pressure coefficient at rest	K_0^{nc}	[-]	0.562	0.455	0.577

COMPARISON OF ANALYTICAL RESULTS WITH OBSERVED BEHAVIOUR

In the coupled finite element analysis performed, the loading rates and steps are taken into account. The results are shown together with field measurements in Figures 5 and 6, for a point under the centre of the embankment and under the middle of embankment edge, respectively. It is observed that predictions based are quite compatible with the field measurements and the end of primary consolidation is reached in about 45 days. In Figure 7, the variation of settlement readings with depth at the end of different time periods are shown together with the analytical predictions for the 30 days. The test fill observations have shown that the field rate of consolidation can be increased very effectively with the use of PVD's and soil improvement with preloading can be achieved in less than the 90 days waiting period proposed initially to reach 90% consolidation. To check the spacing and capacity of PVD's for effective radial drainage, pore pressure readings from piezometers are also evaluated and compared with the computed values from coupled FE analysis. A comparison of excess pore pressures at two different levels between two PVD's underneath the centre of the test fill is shown in Figure 8.

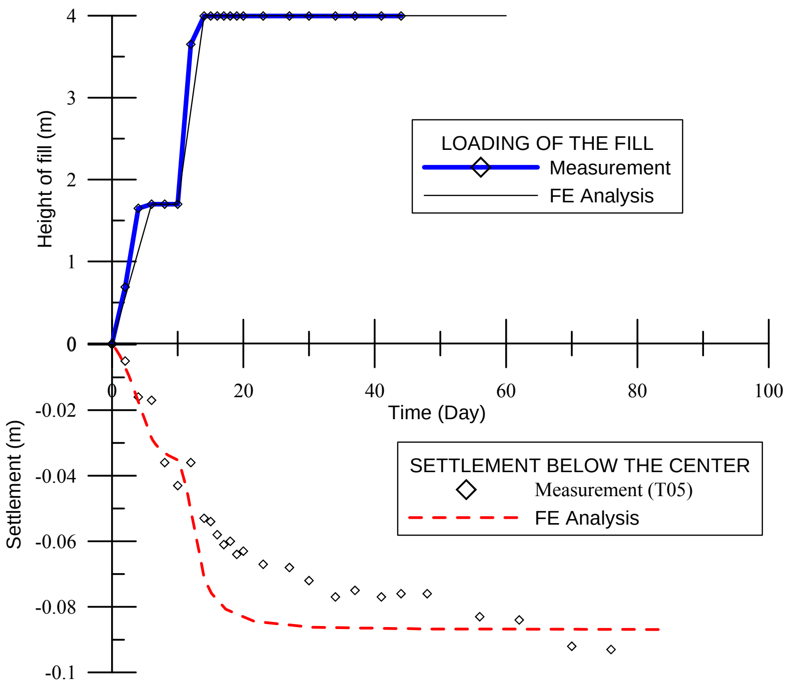

Figure 5. The embankment construction rate and variation of settlement with time below the centre of the fill

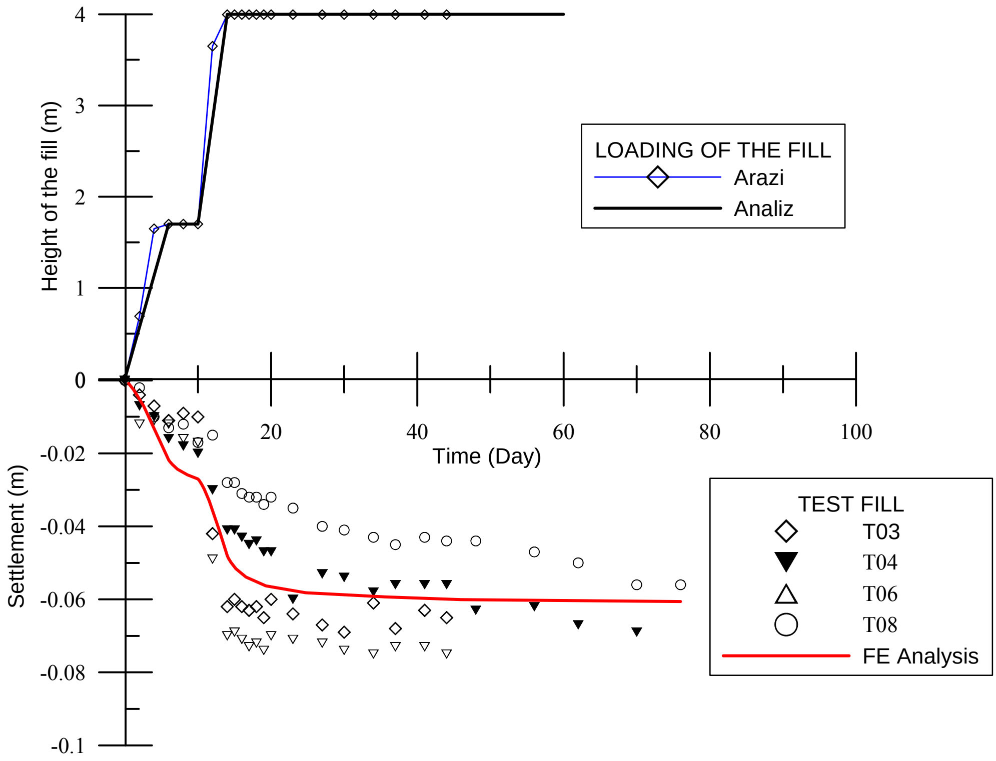

Figure 6. The embankment construction rate and variation of **s**ettlement with time below the edge of the fill

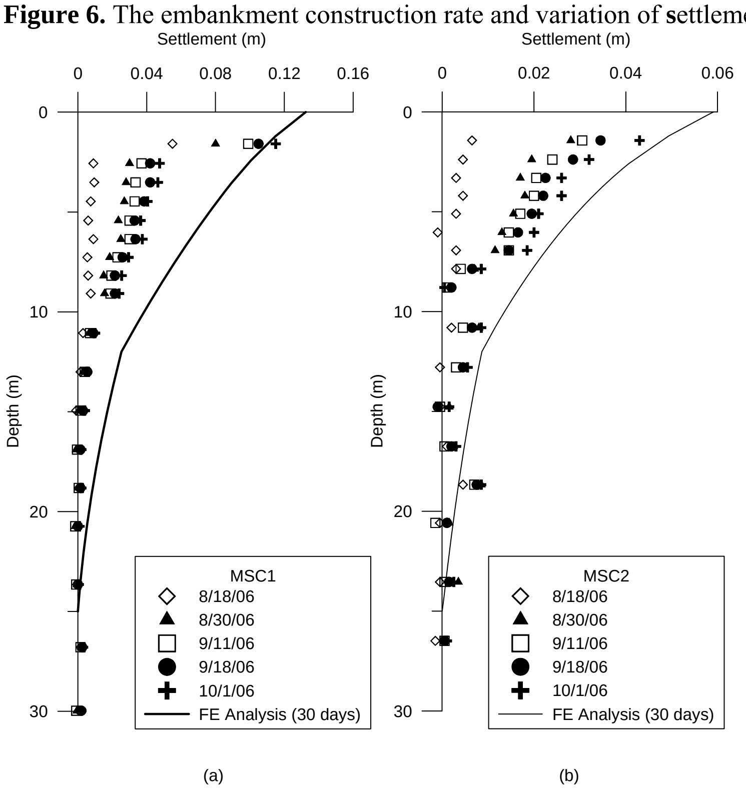

Figure 7. Settlement versus depth below centre and edge of the fill

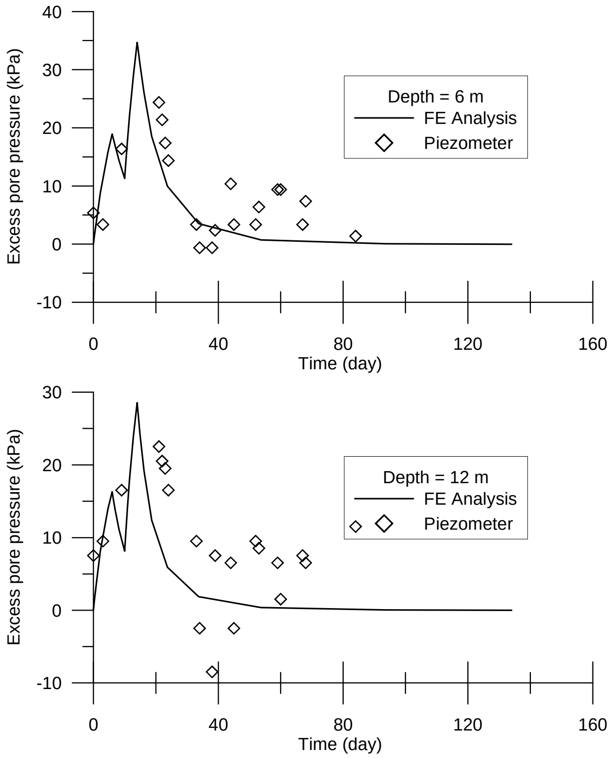

Figure 8. Computed and measured variation of excess pore pressure with time at two different depths below centre of the fill

CONCLUSIONS

The highly compressible soil layers on which the Konya WWTP structures are to be founded are to be improved with preloading. A test fill on instrumented ground was constructed to calibrate the proposed loading scheme according to the observed behaviour. In this paper, a comparison of the observed behaviour under the test fill with the finite element analysis predictions is presented.

The following conclusions are drawn from the in-situ settlement and pore pressure measurements and numerical modelling studies:

- An instrumented test fill embankment construction is believed to be very useful and necessary for soft soil sites planned to be improved with preloading.
- The installation of PVD's in the foundation layers is proved to be very effective to increase the rate of consolidation and subsequent decrease in waiting period during preloading.
- The radial drainage towards vertical drains in the ground can be modelled in a two dimensional plane strain finite element analysis with sufficient degree of accuracy.
- Provided that appropriate soil properties obtained from reliable field and laboratory investigations are utilized, a coupled elasto-plastic stress strain and consolidation finite element analysis can successfully predict the field behaviour under surcharge loading with PVD's.

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