Comparative experimental and numerical analysis of pile under lateral load in granular soil in Chilca, Perú

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Abstract. Piles are frequently used in low resistance soils and often are subjected to lateral loads from different phenomenon as wind, waves, earthquakes events, ground lateral pressure, etc. Lateral resistance of piles should be analyzed to guarantee the stability by theoretical methods or in situ tests for correct design. These investigation compares the deflection results of a lateral load test with the responses in ultimate limit state, *p-y* and finite element method to serve as an example that the different procedures, some more than others, provide a real approximation to the behavior of the pile under lateral load, therefore carry out an optimal design. The test to analyze experimental behavior was carried out in Chilca, to the down south of Lima in Peru, where full-scale load test provides a solid basis for understanding the problem. The geotechnical characterization was carried out with SPT field tests and seismic refraction results. The properties of the tubular A572 grade 50 steel pile are known for being standardized. Pile under lateral load test followed the guidelines of the ASTM D3966 standard. The analytical and numerical calculations were compared with the experimental measurements through graphs of displacements along the pile, these by performing 3D numerical analysis with the commercial program Abaqus® using elastoplastic constitutive model with Mohr Coulomb failure criterion for the FEM, using GEO5® to solve the subgrade reaction approach (*p-y*) problem using Matlock and Reese approximation as soil modulus, at last for the ultimate limit state, Broms and Meyerhof formulations for flexible piles were used.

Keywords: Abaqus, lateral load pile, numerical analysis, *p-y*

1 Introduction

Piles as foundation elements are frequently used in soils of low resistance that allows the compression, tension and lateral loads to be transferred to a resistant layer of the ground. The piles will frequently be subject to lateral loads from wind, waves, docking ships, ground pressure or seismic events [1]. This being the case of soils with high potential for liquefaction, in addition they are also used as foundations for bridges due to the possibility of undercutting, as foundations in offshore structures, as a containment measure in the construction of a pile wall.

Earthquakes are a latent geological hazard, which in this region is mainly caused by the subduction of the Nazca Plate beneath the South American Plate, where there is also a historical study of the presence of the liquefaction phenomenon in Peru [2]. There have been studies about how the Arequipa earthquake in 2001 generated liquefaction in southern Peru [3], and more recently the Pisco earthquake in 2007 had similar effects studied [4] that is why is not only important to recognize the use of piles in our media but also that they will be subjected to lateral loads.

This study performs a pile lateral load test, which is compared with analytical and numerical approaches. There are different methods in the literature to analyze laterally loaded piles, these can be classified as: Ultimate Limit State (ULS), subgrade reaction approach (*p-y*), continuum method (not developed in this article) and the Finite Element Method (FEM) [5]. The ULS method considers the pile as a rigid structure which brings the ground to a state of imminent failure, it is in this state that the maximum applicable load is calculated by equilibrium. The assumption of stiffness of the pile in its final state is far from being precise, so two variants of the method were used that allow considering the flexible behavior [6–8]. The maximum ultimate load applicable to the pile was then calculated. The subgrade reaction approach method assumes an elastic behavior of the soil, where there is a relationship between pressure (p) and settlement (y), at each of the points along the pile. This p-y relationship is defined by the horizontal reaction modulus, a corresponding relationship was used for frictional soils [9]. Finally, a 3-D numerical model was made for a laterally loaded pile using finite element software [10]. In the simulations, the initial conditions of the soil, such as its in-situ stress, are defined as a function of its depth and specific weight, the soil is considered the Mohr-Coulomb elastoplastic to model the behavior of the soil. These methods were used to model a lateral load test conducted at PSV warehouse in Chilca, 43.2 km down south Lima in Peru. The characterization of the soil was carried out using correlations with the SPT, MASW and laboratory tests, then the index and engineering properties of the soil were determined. So, this study presents the results of this full-scale test and their respective analysis, this offers the opportunity to study the behavior of piles under lateral loads due to environmental requirements.

2 Soil characterization

The geotechnical investigation and lateral pile test have been carried out at PSV warehouse in Chilca. Figure 1a shows a referential zone in the map of Chilca city and Figure 1b is a geological map of the INGEMMET (Peruvian Geological, Mining and Metallurgical Institute) where it can be seen that it corresponds to the alluvial deposit (Qh-al1) of the dejective cones of the Mala River with thicknesses of tens of meters [11], in addition, the seismicity in this area is of special importance, therefore, an analysis was carried out in Chilca (-76°67'; -12°44') where it was determined that the maximum acceleration would be approximately 0.43g, while for a pseudo-static analysis it could be considered 0.22g [12].

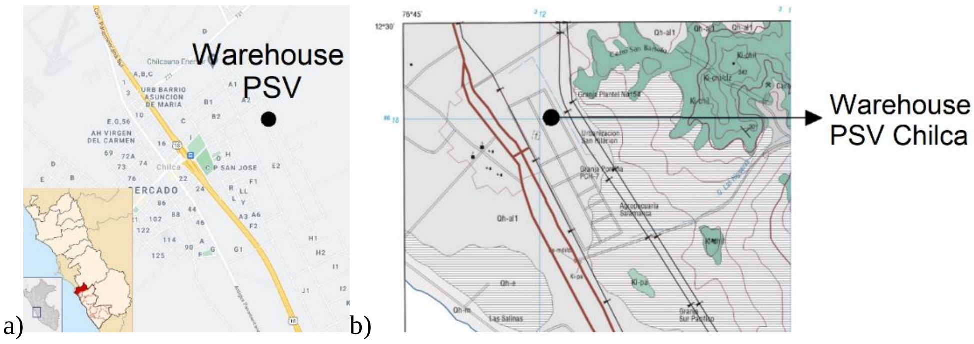


Figure 1: a) Warehouse location PSV b) Geological map from INGEMMET¨ [11].

In situ analyses were performed through the Standard Penetration Test - SPT [13]. In such studies, soil properties can be assessed through empirical correlations in terms of the number of blow count N. Table 1 shows results obtained, and water table was not found along the perforation. The $N_{1(60)}$ is the number of blow count corrected. Rod length correction is made with the table of N [14]. The overburden correction is made with the Seed equation [15].

The first value of N is not taken account for being a no representative measure. The samples were tested in the PUCP's Soil Mechanics Laboratory so we can know: moisture content [16], relative specific weight of the solids [17], granulometry [18] and USCS [19]. The USCS soil classification allows us to unify the two strata found, the first one from one to 6 m deep, which is mainly SM with a SP-SM lens at 2 m deep, the second stratum is more varied, but mainly composed of SP-SM with small CL-ML lens at 7 m depth. Terzaghi [20] estimate that soils with N values as like ours would be classified as dense and very dense.

The MASW measurements are the shear wave velocity (V_S) of 200 m/s in the first 5 m below the ground and from 5 to 13 m a V_S of 270 m/s, with those we can correlate with other properties like specific weight and elastics modulus. Results of the SPT allow us to correlate the properties of the soil. First, we verify the shear wave velocity with correlationships by Ohta and Goto, Dikmen, Marto et al [21–23].The friction angle is calculated with the equations of JSCE [24], and Peck et al. [25] The specific weight is obtained with equations by Mayne [26] and

Anbazhagan [27], both are shear wave velocity dependent. The elastic properties as the elasticity modulus and the Poisson coefficient are the result of a correlation with the blow count *N*, the equations are from Bowles [28], Kuhlawy and Mayne [29] and for the Poisson coefficient by Jaky [30].The horizontal compressibility (n_h) is calculated with the correlationship by Terzaghi [31], Décourt [32], Leoni [33], the modulus of subsoil horizontal reaction (k_h) is obtained with the Matlock & Reese equation [34], and for the passive coefficient of earth pressure we use the Rankine correlationship. The parameters were calculated from geotechnical investigation are shown in Figure 2 and Table 2.

Table 1: Measured SPT, soil properties and USCS classification.

Depth (m)	USCS	Gs	ω (%)	N	$N_{1(60)}$	γ (kN/m^3)	$N_{1(60)}$ by stratum
1 – 1.45	SM	2.69	3.8	83	-	17.05	
2 – 2.45	SP-SM	2.69	7.4	29	33.76	17.05	
3 – 3.45	SM	2.69	11.7	75	81.03	17.05	58.68
4 – 4.45	SM	2.67	14.1	72	73.68	17.05	
5 – 5.45	SM	2.69	17.2	50	46.26	17.05	
6 – 6.45	SP-SM	2.68	14.3	91	85.89	17.56	
7 – 7.45	CL-ML	2.69	32.2	71	61.56	17.56	
8 – 8.45	SM	2.66	13.2	82	65.61	17.56	
9 – 9.45	SP	2.67	15.3	81	60.02	17.56	63.68
10 – 10.45	SP-SM	2.73	19	71	51.41	17.56	
11 – 11.45	SP-SM	2.71	20.5	87	58.59	17.56	
12 – 12.45	SM	2.69	13.8	100	62.71	17.56	

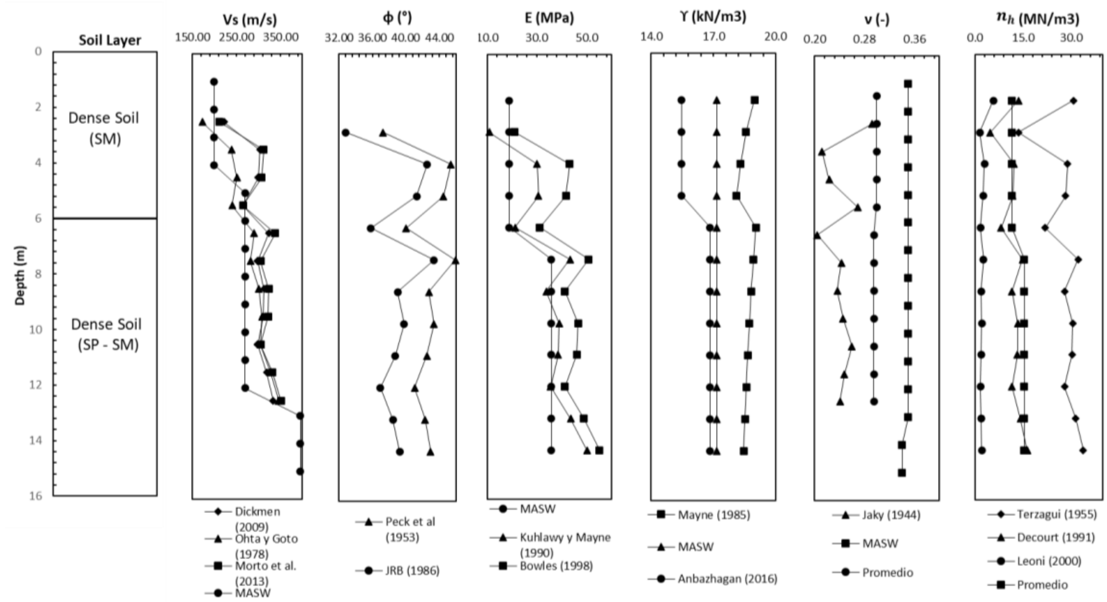


Figure 2: Shear wave velocity, friction angle, elasticity modulus, Poisson coefficient, constant of horizontal compressibility and specific weight from geotechnical investigation.

Table 2: Soil properties. Soil parameters used for modelling.

Depth (m)	ω (%)	γ (kN/m^3)	φ (°)	E (MPa)	ν (-)	n_h (MN/m^3)
0 - 6	12.60	17.05	38.11	27.71	0.3	11.52
6 - 13	18.32	17.56	39.37	40.27	0.3	15.42

3 Lateral load test on pile

The main objective of the lateral load test was to obtain the real displacement recorded over the top of the pile when being subjected to a lateral load, and to find load-displacement curve and displacement of pile body by using installed inclinometers. All the piles were installed with diesel pile hammer (Pileco Inc. D46-32) of 24'' diameter. The pile is driven at a rate of 40 to 60 strokes per minute at the rejection level. Two more piles were located far enough away from the test pile, so that it had no influence on it. These two piles act as reaction piles. In addition, a beam supported by the two reaction piles is placed in order to simulate a reaction mechanism that avoids deforming and thus obtain the real displacements of the pile. The pile is made from steel A572 Gr50 so it has normalized properties, a specific weight of 7849.05 kg/m^3, an elasticity modulus of $2\cdot10^5$ MPa, a Poisson ratio of 0.3, yield moment of 1277.72 kN·m and a moment of inertia of 0.001061 m^4. For the purpose of test, the reaction mechanism is a combination of two driven piles together with a beam supported on them and to ensure the static of mechanism a 100-ton crane is placed behind, so that the beam rests and does not generate any deformation. In Figure 3a general diagram of the entire test is shown, and reaction mechanisms installed in the field can be seen. Once the reaction mechanism has been installed in conjunction with the pile to be tested, the measuring instruments and the connection mechanism are placed as shown in Figure 4; piston, metal plates, hydraulic jack and three digital strain gauges at 0.4 m from the ground, and other two digital strain gauges at 1 and 2 m below the ground.

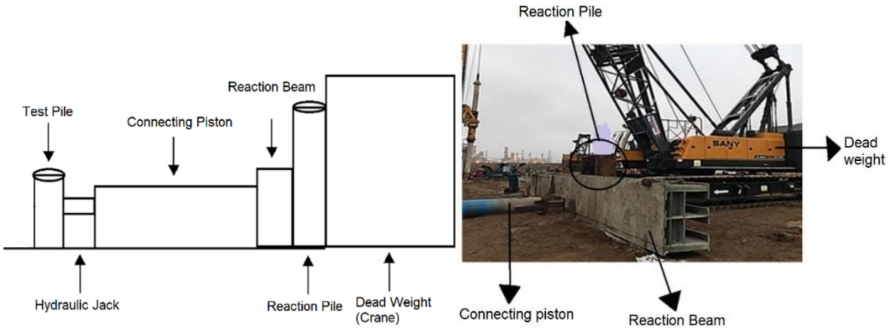
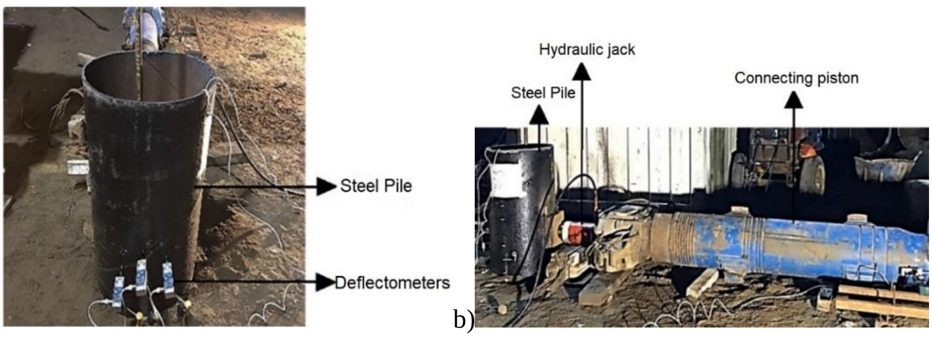


Figure 3: Scheme of the lateral load test and reaction mechanism.



a) b)

Figure 4: a) Digital strain gauges at the head of the pile and b) connection mechanism.

The strain gauges located 0.40 m above the ground, allow us to construct Figure 5b, load vs. displacement curve at the point of application of the load, it was possible to subject the pile to a load of 294.3 kN and then to unload it. Strain gauges found at 1 and 2 m below the ground surface allow us to construct Figure 5a, where the different displacements measured along the different applied loads are shown.

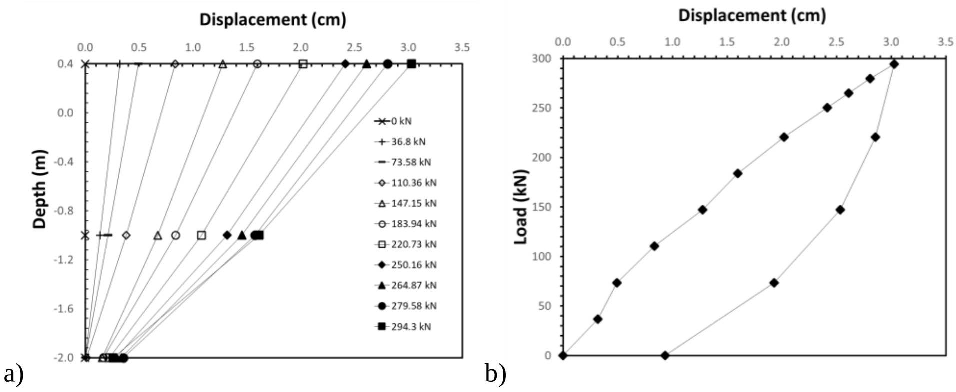


Figure 5: a) Displacements for different loads and b) load versus pile head deflection.

4 Numerical modelling

A 3D model was created to study the behavior of a pile under lateral load using Abaqus [9]. In Figure 6a, geometry of the numerical model made is presented. The pile has a tubular section with an outside diameter of 24" (thickness of ½") and a length of 9.2 m, however 8.80 m is buried and the load is applied at 0.40 m from the surface level. The geometry of the model is 5 m in diameter and 13 m deep.

In the model assumes that the interaction between the pile and the soil use of a penalty formulation. It is assumed that the pile is in perfect contact with the ground. The interaction between the pile surface and the ground can be described using in the normal with the "hard" contact type and friction contact and in the tangential direction with the "penalty" type and a value of the friction angle of 20°, following the recommendation of the NAVFAC DM 7.2 [35].

The boundary conditions define the limits up to which there is a physical response from part of the acting loads. This condition at a point in which the ground no longer exhibits displacement or rotation. The base of the sand layer is embedded in the *x, y, z* directions, while the entire lateral face (*x* and *y*) displacement is restricted.

The simulation is run in two steps, the first is known as geostatic equilibrium, where the in-situ stresses must be in equilibrium with body force and boundary conditions. This option ensures the initial stress condition in all the elements in all the strata, without the soil reaching the yield zone, which in this case is limited by the Mohr-Coulomb model. The pile is considered as a deformable body using the perfect elastoplastic model, limiting elasticity with the steel yield stress. Then, a lateral load corresponds to a value of 294.3 kN is applied as shown in Figure 6b.

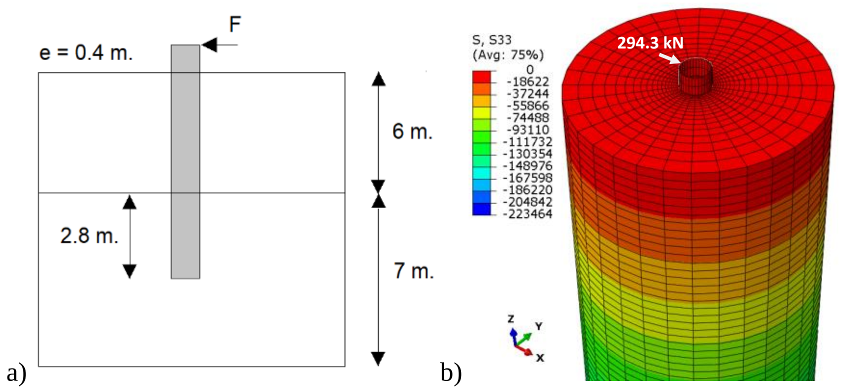


Figure 6: a) Scheme of model and b) load assign and geostatic equilibrium.

Once the previous considerations have been made, the program is executed, obtaining as a result a maximum displacement of 3.1 cm, as shown in Figure 7. In addition, it is appreciated that the displacements only reach approximately 2.5 m around the pile, so the size of the radius can be considered adequate.

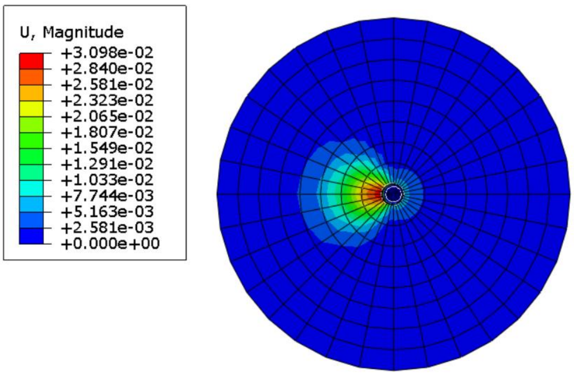


Figure 7. Displacements results.

5 Results analysis

The ultimate limit state works with a single layer of soil, so properties used correspond to weighted average of the soil along the pile. The ultimate limit state of Broms [36] uses equations (1) and (2), the maximum moment is equal to the yield moment and in an iterative resolution we calculated the value of Z_r, to later found the ultimate horizontal load of the pile.

$$M_{max} = M_y = H_u(e + \frac{2}{3}Z_r) \tag{1}$$

$$H_u = 1.5\gamma B K_p {Z_r}^2 \tag{2}$$

The ultimate limit state of Meyerhof [8] formulates the equations (3), (4) and (5) which relate soil stiffness and pile to be able to quantify the effective length of pile, to later calculate the applicable ultimate load.

$$K_{rs} = \frac{E_p I_p}{E_h L^4} \tag{3}$$

$$L_e = L1.65{K_{rs}}^{0.12} \tag{4}$$

$$H_u = 0.12\gamma B {L_e}^2 K_p \tag{5}$$

The ultimate limit state allowed calculating the ultimate horizontal load, in the case of Broms and Meyerhof were 550 kN and 210 kN, respectively. The equation developed to solve this method is the differential equation (6), the displacement obtained was 2.5 cm and the maximum moment was approximately 499 kN·m at 2.3 m below ground level.

$$E_p I_p \frac{d^4 y}{dx^4} + P_x \frac{d^2 y}{dx^2} + k_h D y = 0 \tag{6}$$

The results obtained under the different methods give as response values within an adequate magnitude range. Figure 8 shows a comparison of the displacements as a function of the load and also the displacements along the pile. The results of these analyzes can help reduce costs, to avoid oversizing by reducing the uncertainty of possible pile failure.

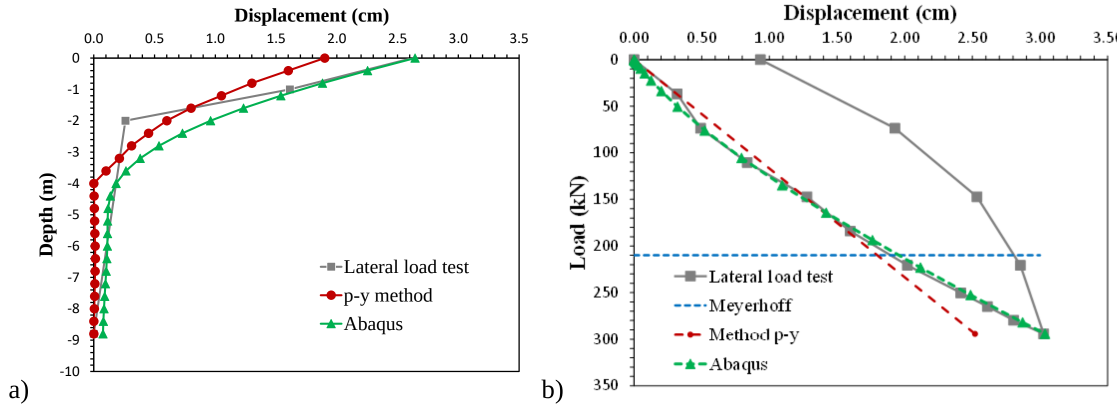


Figure 8: a) Displacement along the pile length and b) load vs displacement.

6 Summary and Conclusions

The stress-strain behavior of piles is an important aspect to consider in the design of piles under lateral loads, in places where earthquakes may occur, for example. The results of these analyzes can help reduce costs, to avoid oversizing by reducing the uncertainty of possible pile failure.

The ultimate limit methods are significantly the most inaccurate, the Broms method gives us an ultimate load significantly higher than those calculated by the other methods, while the Meyerhof method uses the ratio of modulus of elasticity between soil and steel, allowing us to find a closer and more conservative value, in this case, where we have a mostly flexible behavior, the Meyerhof method is much more appropriate.

The *p-y* method results in a similar magnitude of lateral displacement, however, the moment found is well below the yield point, which is not compatible with the test results, since the pile ended up presenting a permanent deformation so its entry into the plastic regime. Up to approximately 160 kN of load, the results are very similar to those of the model and those of the test, this could be due to the fact that when applying the load under 160 kN, the elastic assumptions of the *p-y* method are more accurate.

The numerical method is the most similar in the displacement response, this is due to an adequate choice of the diameter of the chosen ground, the characterization verified by two different tests, in addition to an adequate simulation of the initial stresses of the soil.

The difference between these three methods lies in the assumptions that were made to develop each of them, in the case of the *p-y* method the elasticity regime can simulate very well up to a load of 180 kN, after that the plastic displacements increase and the curves begin to separate, on the other hand, the numerical model can better resemble the behavior observed in the test.

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