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Evaluation of load distribution in Piled-Raft Foundation Systems (PRFS) founded on soft clay, via geotechnical centrifuge tests

Edgar Rodríguez Rincón[1#], Renato Pinto da Cunha[2], Bernardo Caicedo Hormaza[3], Juan Felix Rodríguez Rebolledo[2]

Article

Keywords
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Consolidation

Abstract

Piled raft foundation systems (PRFS) have been increasingly adopted as an alternative foundation system in developed urban areas underlain by soft soils. In such soils, a double consolidation process may occur as a result of both external loading and subsidence, leading to changes in the soil stress state. Previous studies on PRFS have primarily focused on settlement behaviour induced by external loading, whereas the effects of subsidence processes have received comparatively limited attention. Moreover, most investigations of PRFS behaviour have been conducted using numerical approaches. This study presents an experimental investigation of the behaviour of PRFS using geotechnical centrifuge modelling, with particular emphasis on the redistribution of load between the raft and piles and the associated changes in pore water pressure. Physical models at a scale 1:70 were developed and instrumented to enable direct measurements of the foundation response. The results demonstrate that the proportion of load carried by the raft and piles varies with the pile arrangement and changes in pore water pressure within the soil. The pile contribution increased from 23–26% during Stage 2 to 46–54% during Stage 3, indicating a substantial redistribution of load from the raft to the piles following groundwater extraction. Under subsidence conditions, the redistribution of load between the raft and piles was found to be consistent with trends previously reported in the literature.

1. Introduction

According to Poulos (2001), piled raft foundation systems (PRFS) are increasingly being used as an alternative foundation system for infrastructure projects. Piled rafts combine the contributions of the raft and piles with the aim of increasing the overall bearing capacity while reducing total and differential settlements. This combination can also reduce internal stresses and bending moments within the foundation system (Malviya et al., 2023; Sengupta et al., 2026). As a result, PRFS involve complex soil–structure interaction mechanisms. The bearing capacity and settlement behaviour of piled rafts have been commonly evaluated using empirical formulations, analytical approaches, and numerical analyses. Conventional design methods currently used in practice are generally based on relatively conservative assumptions regarding raft–soil–pile interaction. Consequently, these methods may not accurately capture the actual behaviour of piled raft systems and may not lead to an optimum design (Ferchat et al., 2021).

Regarding their application in soft soils, Lee et al. (2010) noted that relatively limited attention had been devoted to the behaviour of PRFS under such ground conditions, as these foundation systems had traditionally been considered unfavourable due to the potential for excessive settlements and insufficient bearing capacity. Nevertheless, Ferchat et al. (2021) reported that an increasing number of numerical and experimental studies have investigated the behaviour of PRFS in soft ground. These studies have sought to extend or modify existing approaches to account for the complex soil–structure interaction mechanisms governing the response of piled raft systems under such conditions.

In recent years, an increasing number of structures have been constructed on soft ground, and the application of piled rafts under such conditions has therefore become an important issue in foundation design (Malviya et al., 2023; Sengupta et al., 2026). PRFS have been adopted in several cities in Asia and Latin America, including cities underlain by lacustrine deposits, such as Bogotá and Mexico City (Banerjee, 2009). These deposits may contain permeable layers that contribute significantly to horizontal groundwater

#Corresponding author. E-mail address: erodriguezr@unal.edu.co
[1]Muniz e Spada Engenheiros Consultores, São Luís, MA, Brasil.
[2]Universidade de Brasília, Programa de Pós-graduação em Geotecnia, Brasília, DF, Brasil.
[3]Universidad de Los Andes, Departamento de Ingeniería Civil y Ambiental, Bogotá, DC, Colombia.

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flow and aquifer recharge (Caicedo et al., 2019). However, PRFS constructed in these areas have experienced damage associated with ground subsidence caused by groundwater extraction and/or reduced aquifer recharge (Bareño & Rodríguez, 1999). Under such conditions, subsidence can induce additional downward loads on the piles through negative skin friction, accompanied by additional settlements. Under extreme conditions, this mechanism may contribute to the failure of the foundation system (Leung et al., 2004; Auvinet & Rodríguez, 2017).

The complex soil–pile–raft interaction behaviour has been investigated extensively using laboratory-scale physical models (e.g., Thaher & Jessberger, 1991; Horikoshi & Randolph, 1996; Tran et al., 2012; Rodríguez, 2016; Elsawwaf et al., 2022), numerical models (e.g., Lee et al., 2010; Khanmohammadi & Fakharian, 2018; Ayuluri et al., 2017; Mali & Singh, 2018; Ferchat et al., 2021; Ahmed et al., 2022; Malviya et al., 2023), comparative analyses, and field investigations. However, current analysis and design procedures for piled rafts on soft clay subjected to vertical loading are still considered inadequate, particularly when additional processes such as consolidation induced by external loading and ground subsidence are taken into account (Lee et al., 2010).

Small-scale models tested under 1g conditions cannot reproduce the self-weight stresses of the prototype because of the reduced model dimensions. Geotechnical centrifuge modelling overcomes this limitation by increasing the gravitational acceleration acting on the model, thereby enabling the stress conditions of the prototype to be reproduced at reduced scale. Because load redistribution in piled raft foundations depends on the coupled response of the soil, piles and raft, physical modelling under stress conditions representative of the prototype is particularly useful for directly measuring the evolution of pile and raft loads during consolidation and subsidence. Using small-scale models in a geotechnical centrifuge, Thaher & Jessberger (1991) conducted tests at 50g to investigate the effects of pile diameter, pile length, and number of piles beneath a raft subjected to axial loading. Horikoshi & Randolph (1996) performed centrifuge tests at 100g to investigate the effects of the number of piles, raft contact pressure, settlement, and the proportion of the applied load carried by the piles. Tran et al. (2012) investigated the effects of subsidence in Ho Chi Minh City, Bangkok, Mexico City, and Shanghai, where changes in the groundwater conditions of clay deposits have contributed to increased differential settlements of structures. More recently, Rodríguez et al. (2020) investigated settlement development in PRFS constructed on soft clay, analyzing the evolution of settlement under different consolidation conditions and its variation with time.

According to the literature reviews presented by Rodríguez (2016) and Rodríguez et al. (2020), previous investigations have generally focused on the behaviour of PRFS in clay soils from a broad perspective. Only a limited number of studies have addressed the problem from a three-dimensional experimental perspective or using physical models representative of field conditions. Moreover, few studies have investigated the behaviour of PRFS when consolidation induced by external loading occurs concurrently with ground subsidence and the associated changes in pore water pressure. Sengupta et al. (2026) recently conducted a numerical investigation of groundwater drawdown and subsidence effects on PRFS, concluding that the increase in effective stress associated with groundwater drawdown induces negative skin friction along the pile shafts, resulting in increased drag loads, additional settlement, and redistribution of axial forces within the pile group.

The use of physical modelling is therefore recommended to improve the understanding of PRFS behaviour under these conditions. Previous literature reviews have also highlighted that consolidation can significantly affect the response of PRFS, particularly in soils with limited drainage. Accordingly, accurate estimation of the internal forces and deformations of PRFS requires consideration of both soil consolidation and foundation stiffness in the analysis. In addition, uncertainty remains regarding the proportion of load carried by the piles and the raft, particularly when changes in pore water pressure occur. Although several analytical approaches have been proposed to estimate this load-sharing ratio, these methods are generally based on simplified assumptions, often considering elastic soil behaviour. Experimental investigations are therefore needed to further assess the behaviour of PRFS constructed on soft soils and to provide a more reliable basis for design. In particular, physical modelling can be used to investigate how the load distribution between the raft and piles is affected by the coupled effects of external loading and subsidence-induced consolidation. This approach may provide further insight into the load-sharing mechanism and the associated changes in foundation response under a double-consolidation process.

In the present study, the physical models were subjected to two consolidation processes: one induced by external loading, representing the effect of the superstructure, and the other induced by subsidence, representing groundwater extraction from a deeper soil layer and the associated drawdown in pore water pressure. The soil models were prepared in the laboratory to reproduce a soil profile representative of the subsurface conditions in Bogotá, Colombia. Several response variables were measured using appropriate instrumentation. The behaviour of the PRFS was then analysed as a function of the number and arrangement of the piles, with particular emphasis on the resulting load distribution between the raft and piles.

This study forms part of a broader research program investigating the behaviour of PRFS constructed on soft soils subjected to a double-consolidation process, using reduced-scale physical models in a geotechnical centrifuge. In an initial study, Rodríguez et al. (2020) investigated the settlement behaviour of these systems using 1:200-scale models.

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Subsequently, Rodríguez et al. (2021) presented a comparative analysis of settlement behaviour using 1:70- and 1:200-scale models. The present study focuses on the load distribution between the raft and pile elements of PRFS based on 1:70-scale centrifuge models, thereby complementing the findings reported in the previous studies. While the previous studies focused primarily on settlement development and the effects of consolidation on the global deformation response, the present study specifically investigates the load-sharing and load-redistribution mechanisms between the raft and the piles. In particular, the present analysis quantifies the proportion of the applied load carried by the pile group before and after groundwater extraction and examines the influence of pile arrangement and spacing on this load distribution.

2. Material and methods

2.1 Geotechnical centrifuge apparatus

The testing facility used in this research is a beam-type geotechnical centrifuge located at the Geotechnical Models Laboratory of Los Andes University (Colombia), with the characteristics shown in Table 1 and Figure 1 (Rodríguez, 2016).

2.2 Measured variables

The following measurements were made in the models using appropiate instrumentation:

- The PRFS settlements;
- Variation in pore pressure;
- Load distribution among the piles.

2.3 Geometry and materials of the PFRS models

To scale the models, the similarity between models and prototypes must be guaranteed based on the flexural modulus, as shown in Equation 1 (Taylor, 1995).

$$E_m^{mm} I_m^{mm} = E_m^c I_m^c \quad (1)$$

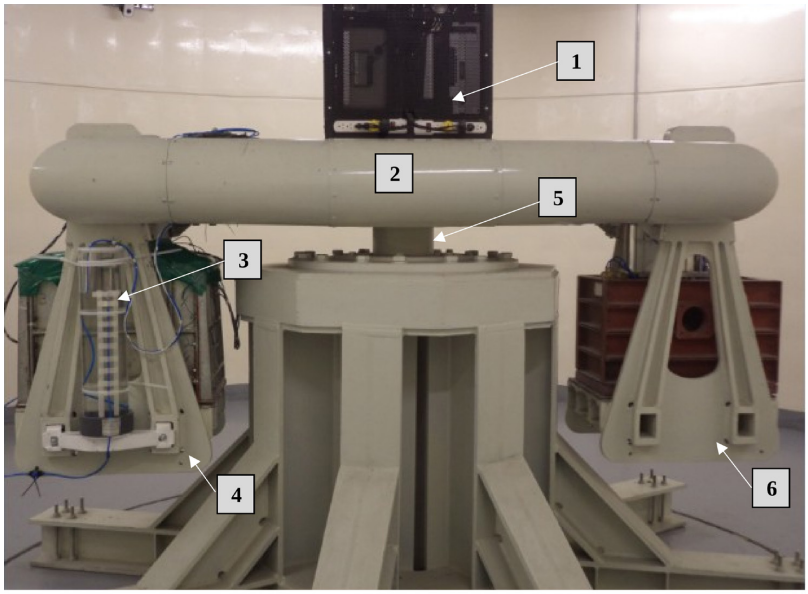
where $I_m = \frac{bt^3}{12}$: moment of inertia, b: base of the element, t: thickness of the element, E_m^{mm} : elastic modulus of the material in the model and E_m^c : elastic modulus of concrete.

Since a specific prototype was not adopted, the aim was to ensure that the modulus values were at least consistent with typical values recommended in the literature. Equation 1 was applied for the raft, as it is the main element subjected to flexural stress; for all other elements, the conversion of dimensions is proportional to the scale factor.

Table 1. Characteristics of the beam-type geotechnical centrifuge.

Turning radius (m)	1.90
Model box dimensions (cm)	40 x 50 x 60
Maximum gravitational field (g)	200
Maximum model weight (kN)	4.0
Nominal power (HP)	400
Channels for data acquisition	50

NOTE: g = Earth´s gravity; 1HP = 745.7W.



1. Data acquisition panel
2. Beam
3. Water control tank
4. Modelling boxes platform
5. Motor axis
6. Counterweight platform

Figure 1. Beam-type geotechnical centrifuge at Los Andes University.

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Taking Equation 1 into account and considering a unit base width, the dimensions of the raft-type elements were determined from Equation 2.

$$t_p^c = \frac{t_m^{mm} n_l}{\sqrt[3]{\frac{E_m^c}{E_m^{mm}}}} \quad (2)$$

where t_p^c: concrete thickness in the prototype, n_l: scale factor, t_m^{mm}: thickness of the element made from the corresponding material in the model, and E_m^c: the elastic modulus of concrete.

The load capacity of the geotechnical centrifuge and the modelling box dimensions allowed the determination of the dimensions and a scale factor to be used. The dimensions of the modelling boxes limited the size of the raft that could be investigated. Accordingly, two types of models were defined:

- Models with different numbers of piles arranged at the centre of the raft (Models M3 and M4), with a minimum pile spacing of 2*d* to minimise pile group effects.
- Models with the same number of piles but different pile spacings, ranging from a minimum spacing of 2*d* to a maximum spacing of 9*d*, depending on the raft dimensions (Model M6).

The investigated spacing range was constrained by the model geometry; therefore, the study was not intended to provide a parametric evaluation of all possible pile spacing ratios.

A scale factor of 1/70 (70 *g*) was used in this research to evaluate the behaviour of the models. Figure 2 presents the configurations evaluated, and Table 2 presents the characteristics of the models, comparing the corresponding equivalent prototype dimensions.

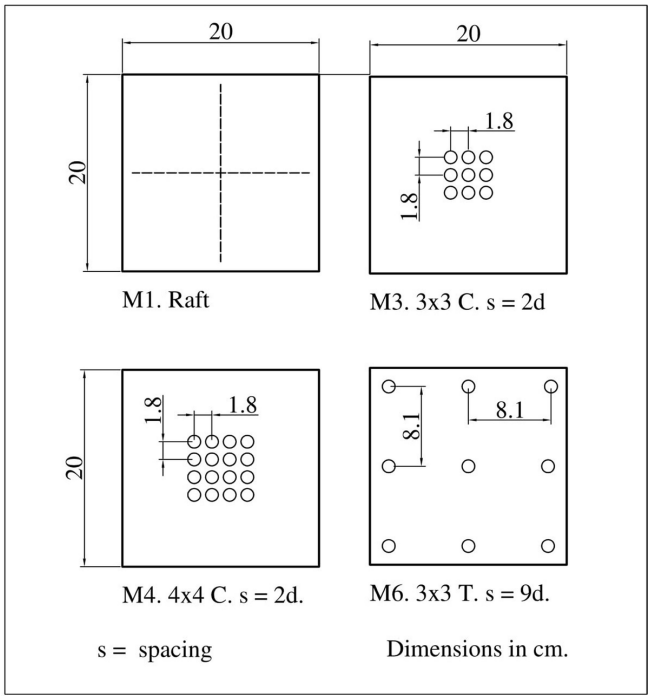


Figure 2. Geometrical configuration of PRFS models.

Table 2. Model elements dimensions and prototype equivalences.

Element	Parameter	Model characteristic	Prototype characteristic (equivalent)
Raft	Material	Aluminium	Concrete
	Thickness, e_R	13 mm	1.147 m
	*Elasticity modulus, *E*	70000 MPa	35000 MPa
	Width, *B*	200 mm	14 m
	Length, *L*	200 mm	14 m
Piles	Material	Aluminium	Concrete
	Diameter, *D*	9 mm	63 cm
	*Elasticity modulus, *E*	70000 MPa	30000 MPa
	Length, *L*	320 mm	22.4 m

*Parameter value based on supplier´s information (according to Rodríguez, 2016).

2.4 Soil profile

According to the principles of geotechnical modelling, the model must duplicate the factors that influence the soil's response, i.e., the level of stresses and resistance. In the case of clays, the void ratio and the pressure applied in the manufacture of the soil must be controlled to comply with these requirements.

The soil profile selected to represent the behaviour of soft clay soils was proposed by Rincón & Rodríguez (2001), which corresponds to a clay profile with a strength that is linearly variable with depth, between 10 kPa and 40 kPa, from the surface to the bottom of the thickness to be modelled, typical of the city of Bogotá D. C. (Figure 3).

Authors such as Thaher & Jessberger (1991), Taylor (1995), Rincón & Rodríguez (2001), and Dallos (2007) recommend preparing the soil outside of the gravitational field of the geotechnical centrifuge and subsequently carrying out an initial test or preliminary "flight", to standardise the stress conditions and the pore pressure distribution in the soil mass of the model. The procedure proposed by Rincón & Rodríguez (2001) was implemented in this research for soil preparation. This procedure is summarised in Figure 4 and detailed by Rodríguez (2016).

The soil profile is made up of three layers, as shown in Figure 3. At the bottom of the modelling boxes, a layer of filtering sand was arranged to control the pore water pressure drawdown in the model. The water pressure was controlled with an external tank connected to the bottom of the box. Intermediate sand layers with a thickness of 0.7 cm were incorporated during model preparation to facilitate construction of the soil profile and to represent relatively permeable horizontal features of the natural deposit. During centrifuge acceleration and subsequent testing, however, these thin layers underwent compression and were not considered effective drainage boundaries for the consolidation analysis. The observed pore-pressure response was consistent with pore-pressure dissipation being predominantly controlled by the lower drainage boundary, as discussed in Section 3.1. Therefore, although the intermediate sand layers were part of the physical soil profile, the consolidation-time estimation was based on an equivalent single-drainage condition.

2.5 Stages, times and pore pressure conditions during the tests

The current research focused on analyzing two working conditions of the PRFS: first, PRFS subjected to external load and, second, PFRS subjected to external loads and subsidence (by water extraction from deep soil layers). In both cases, the analysis was focused on the influence of consolidation on the response of the foundation system. Three test stages were defined and carried out, as presented in Figure 5.

As mentioned, to maintain the control over the pore pressure conditions inside the soil, an external control tank was implemented in the modelling box (Figure 1). The tank was connected to the bottom of the modelling box, where a sand filter was located (Figure 3). For Stages 1 and 2, the test was carried out keeping the water level constant in the external tank.

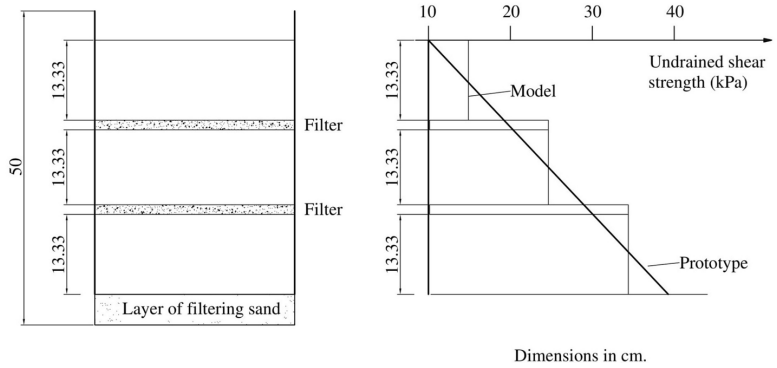


Figure 3. Proposed soil profile.

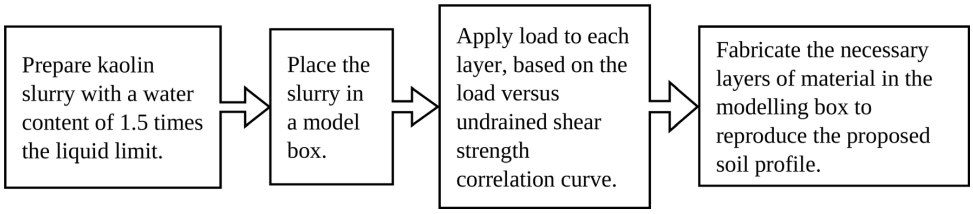


Figure 4. Procedure for soil preparation.

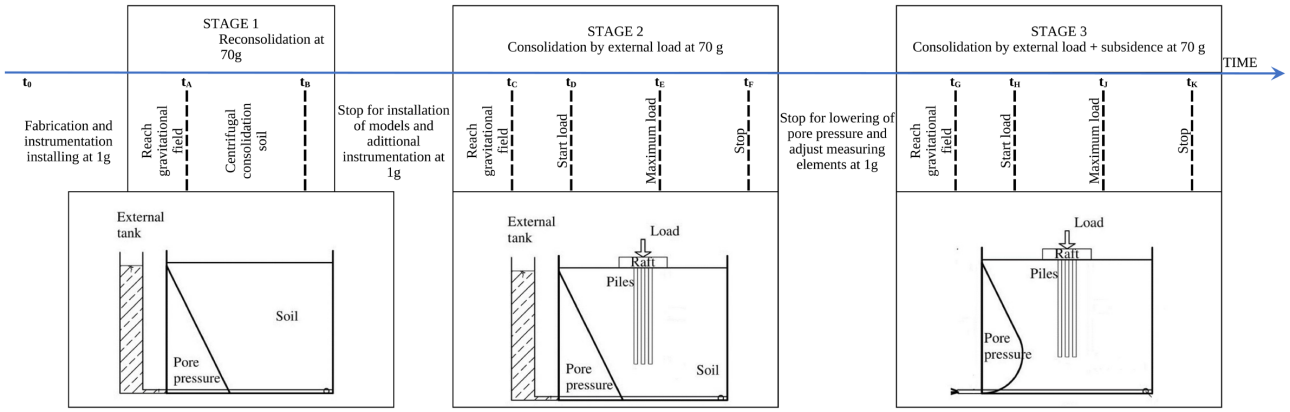


Figure 5. Stages, pore pressure conditions, and timeline to the models.

The water level in the tank was at a height similar to that of the soil surface in the modelling box. This condition allowed the hydrostatic pressures to be kept constant.

In Stage 3, the connection between the tank and the modelling box was disabled. This disconnection allowed the water to escape through the lower sand filter, causing a decrease in pore water pressure from the lower soil layer. This reduction in pore water pressure simulated a subsidence condition induced by the extraction of water from deep soil layers.

The defined test stages and pore pressure conditions are schematised in Figure 5. It is important to mention that it was not possible to make changes to the working conditions in the geotechnical centrifuge during each testing stage of the models. Therefore, flights had to be performed in stages and then stopped when a physical modification to the model had to be made that could not be replaced by an automatic process. Testing the model required a pause between Stage 2 (consolidation by external load) and Stage 3 (consolidation by external load and subsidence) to allow modifications to be made to the model. This stoppage time was short compared with the test duration, and the soil rebound was controlled.

Based on the soil material manufacturing conditions, a vertical coefficient of consolidation, Cv =0.33 cm^2/min, was considered. This value was obtained from oedometer consolidation tests performed on laboratory-prepared specimens (Rodríguez, 2016). A representative average value of Cv was adopted for the three soil layers because the oedometer tests were performed under different loading levels.

Although intermediate sand layers were incorporated during model preparation, they were not considered effective drainage boundaries during the centrifuge tests. Their limited thickness and compression during centrifuge acceleration, together with the observed pore-pressure response, indicated that pore-pressure dissipation was predominantly controlled by the lower drainage boundary. Therefore, an equivalent single-drainage condition was adopted, and the drainage path was taken as the full thickness of the corresponding clay layer (13.33 cm), rather than half its thickness, for estimating the flight times in Table 3.

Table 3. Times for each test stage.

Activity	Range	Flight time (h:min)
Model soil preparation	t_0 - t_A	
Stage 1	t_A - t_B	1:54
Stop for instrument and model installation	t_B - t_C	
Stage 2	t_C - t_F	2:54
Stop for external water tank disconnection	t_F - t_G	0:30
Stage 3	t_G - t_K	2:54

The resulting drainage path and Cv were used to estimate the flight times presented in Table 3, with a target degree of consolidation of at least 90%. This assumption represents the effective drainage condition adopted for the consolidation-time estimation and does not imply that the intermediate sand layers were completely hydraulically inactive.

2.6 Applied load and instrumentation

Based on Meyerhof's theory (Bowles, 1998) and with the undrained resistance of the proposed soil profile, the ultimate bearing capacity of the raft was determined to be 92 kPa. In the models, the load to be applied was 1.46 kN (calculated with a factor of safety of 2.5), obtained by converting the actual allowable stress into admissible stress in the model, and considering the area of the prototype raft and the raft in the models. A pneumatic actuator was used to apply the load, as shown in Figure 6. The load was applied in stages (steps) until the maximum value was reached (time intervals t_D-t_E and t_H-t_J in Figure 5), simulating a construction process of at least one year.

Considering the model scale and the parameters to be measured, the following instrumentation was employed:

- Displacement sensors (LVDT) with a reading capacity of up to 5 cm. These were placed at three points on the raft and three (3) points on the ground.

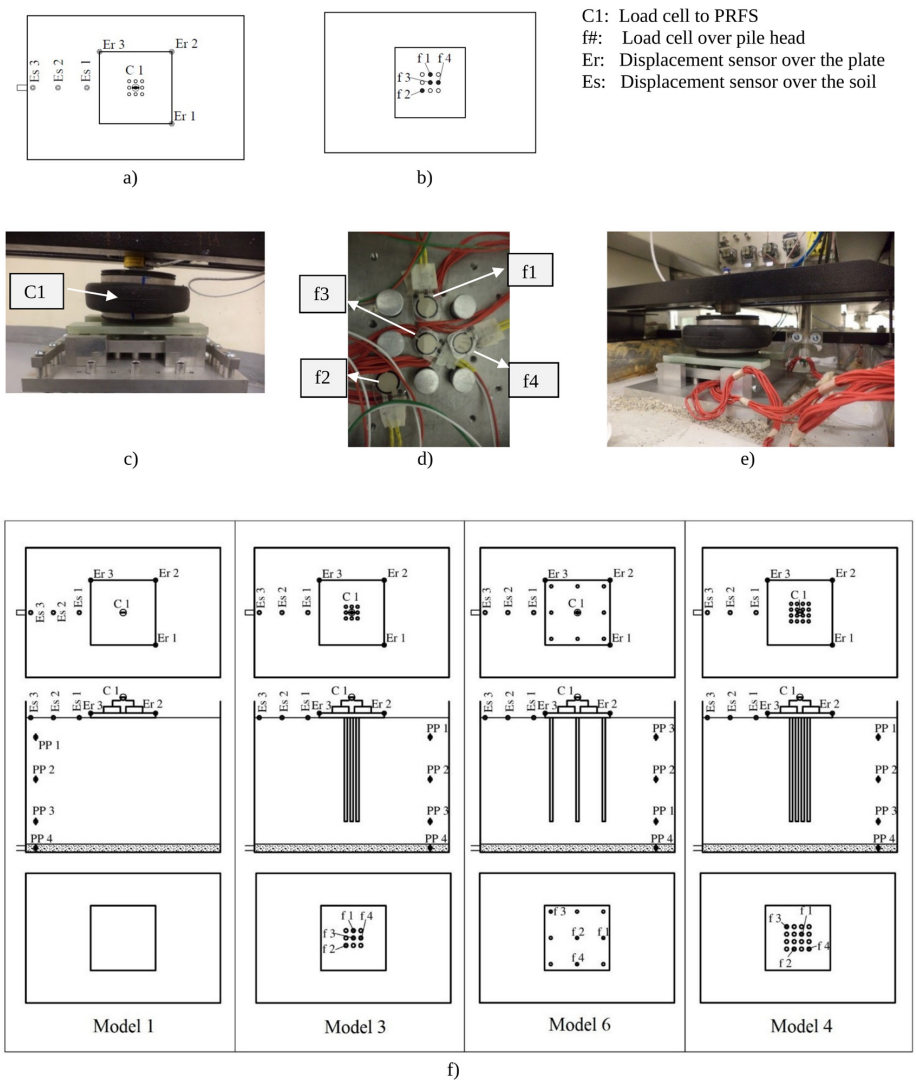


Figure 6. Details of the model assembly and instrumentation distribution: a) General instrumentation layout. b) Load cells layout over the piles dead the piles head. c) General load actuator. d) Load cells distribution at the model. e) General view of the M3 Model. f) Distribution of instrumentation on the models.

- Load cells for the measurement of load in the general system and at the heads of selected piles, to determine the global distribution of the load and its variation with the loading conditions (Figure 6).
- Piezometers to measure the variation in the pore water pressure in each flight stage.

Figure 6 shows a diagram of the physical assembly of the models. Figure 6 details the load cell placement used to measure the load on the PRFS (C1) (Figure 6b) and the arrangement of the load cells at the pile heads (f) in Model M3 (Figures 6c and 6d). The system was designed to allow an independent measurement of the load on some piles and on the PRFS. The arrangement of the instruments for each model is presented in Figure 6f, which specifies the type of instrument used and its location in plan and in depth.

3. Results and discussion

In this section, the results obtained in the models are presented and discussed; all the responses were converted to the prototype scale.

The long-term analysis condition, which represents the critical behaviour of PRFS arranged on clay materials, was carried out using the Filling Factor defined by Mandolini et al. (2013), Equation 3.

$$FF = \frac{A_G}{A_R}\frac{d}{s} \tag{3}$$

where A_G: piles group area, A_R: raft area; d: pile diameter and s: piles spacing. A_G was defined by De Sanctis et al. (2002) (Equation 4).

$$A_G = \left[\left(\sqrt{N_p} - 1\right)s\right]^2 \quad (4)$$

where *Np*: number of piles in the group. For the geometric configurations used in this research, the filling factors (*FF*) are presented in Table 4.

3.1 Pore pressure variation

Figure 7 presents the pore pressure variation in the models for each stage of the test. This figure includes the theoretical hydraulic pressure line and the values reported by the piezometers installed in the soil.

Figure 7 shows that for Stage 2 (consolidation by load), the pore pressure measurements correspond reasonably well to the theoretical hydraulic line, with a few small increments in the top layers, probably associated with the placement of the load. In Model M6, there is a small loss of pore pressure at the base of the soil, which could explain the greater settlement of this model in Stage 2. In Stage 3, the hydrostatic condition is shown as a reference for evaluating dewatering. All models showed a loss of pore pressure in the lower layer, as expected. For Model M4, the lowering of pore pressure was smaller than in the other models, which can be correlated with a lower settlement in Stage 3, and as a consequence, the relative incremental settlement at this stage tends to be positive (Table 5).

3.2 Load distribution in the system

One of the objectives of the present investigation was to determine the variation that occurs in the load supported by the piles when water is extracted as part of the process. With the results of the load readings from the cells at the pile heads, it was possible to establish the proportion of the final load that is borne by each pile (see Table 6).

To determine the total load on the piles, it was assumed that piles in symmetrical positions had similar loads. Therefore, by knowing the loads measured by the load cells placed on selected piles of the model (Table 6), the load value supported by the group of piles could be determined (Lee, 1993). The pile contribution increased from 23–26% during Stage 2 to 46–54% during Stage 3, corresponding to increases of 22–28 percentage points after groundwater extraction. The largest increase occurred in M6, which had the largest pile spacing (s/d = 9). These results indicate a substantial redistribution of load from the raft to the pile group during subsidence-induced consolidation.

In general, the following observations can be drawn from Table 6:

- In clay soils, increasing the number of piles does not necessarily reduce the proportion of load carried by the raft, as also reported by Khanmohammadi & Fakharian (2018).
- Previous studies have reported a reduction in pile contribution for larger s/d ratios (Ahmed et al., 2022). In the present study, the maximum investigated ratio was s/d=9, and within this range the pile contribution increased with spacing.
- The distribution of load among the piles is affected by both the pile configuration and the position of individual piles within the raft.
- In particular, an increase in the load carried by the central piles relative to the corner and edge piles was observed as the pile spacing increased (M6 compared with M3). Similar behaviour was reported by Lee et al. (2010).

Table 4. Filling factors for the models (FF).

Parameter	Model			
	M1	M3	M4	M6
d (cm)		63	63	63
N_p		9	16	9
s (cm)		126	126	567
s/d		2	2	9
A_R (m²)	196	196	196	196
A_G (m²)		63.5	14.28	12.86
FF	0	0.016	0.036	0.073

Table 5. Effective displacements.

Model	Stage	Es_3	Er	$Es_3 - Er$	Effective displacement (cm)
M3	2	26.4	34.1	-7.7	-10.9
	3	30.3	33.5	-3.2	
M4	2	22.4	29.3	-6.9	-5.2
	3	39.6	37.9	1.7	
M6	2	40.6	47.0	-6.4	-9.7
	3	17.3	20.6	-3.3	

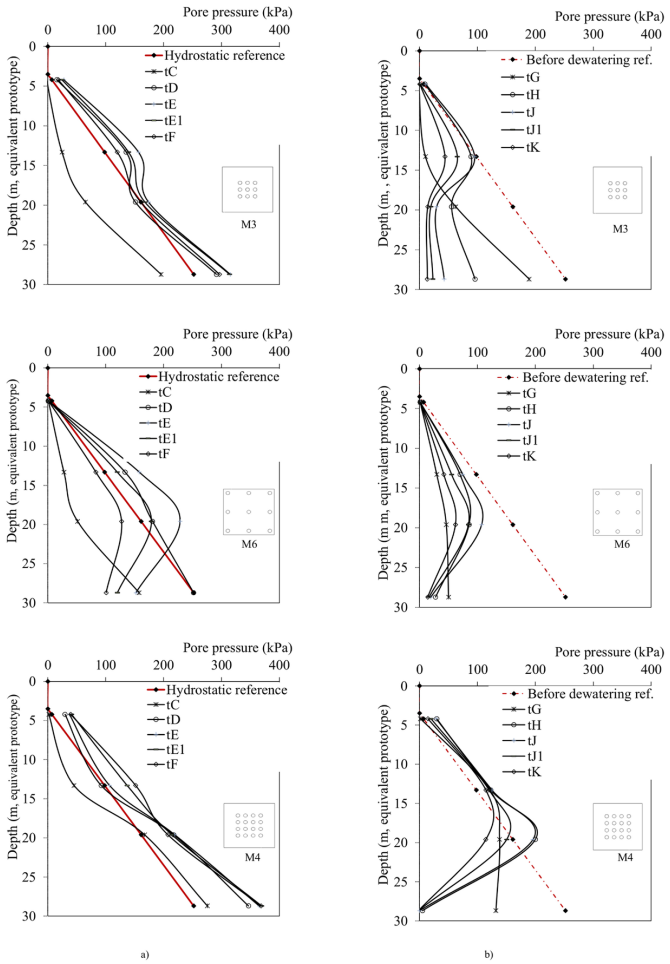
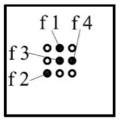
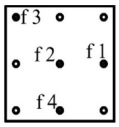
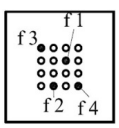


Figure 7. Pore pressure variation in the models. (a) Pore pressure evolution on the models in Stage 2 - with consolidation by load. (b) Pore pressure variations on the models in Stage 3 - with consolidation by load and extraction of water from deep layer. In Stage 3, the hydrostatic condition is present. It is the reference for observing the dewatering.

Table 6. Load distribution among the piles of the PRFS models.

Model	Load cell distribution in the model	Location by symmetry of the cells	Loads by pile (kN)		Stage 2	Stage 3
M3		f2 f1 f2	f1 = 196.2	Total load on the piles (kN)	1753.86	3423.10
		f4 f3 f4	f2 = 215.0	System load reading (kN)	7496.56	7417.85
		f2 f1 f2	f3 = 173.2	Load percentage on the piles	23%	46%
		Total piles 9	f4 = 163.9	Load percentage on the raft	77%	54%
M6		f3 f4 f3	f1 = 106.7	Total load on the piles (kN)	1916.69	3962.80
		f1 f2 f1	f2 = 307.5	System load reading (kN)	7358.82	7319.47
		f3 f4 f3	f3 = 155.4	Load percentage on the piles	26%	54%
		Total piles 9	f4 = 387.1	Load percentage on the raft	74%	46%
M4		f3 f2 f2 f3	f1 = 113.5	Total load on the piles (kN)	1862.61	3504.39
		f2 f1 f1 f2	f2 = 131.7	System load reading (kN)	7417.85	7398.18
		f2 f1 f1 f2	f3 = 35.8	Load percentage on the piles	25%	47%
		f4 f2 f2 f4	f4 = 141.7	Load percentage on the raft	75%	53%
		Total piles 16				

- The increase in effective stress during groundwater drawdown induces a redistribution of axial forces within the pile group.
- Consolidation induced by groundwater drawdown substantially modifies the load transfer to the piles, consistent with the findings reported by Sengupta et al. (2026). Variations of approximately 22–28 percentage points in the load carried by the piles were observed between Stages 2 and 3 in the tested models.

Mandolini et al. (2013) presented a graph (Figure 8) that relates the proportion of load supported by the raft to the inverse value of the Filling Factor (1/*FF*). According to the authors, for 1/*FF* values between 10 and 60, the percentage of load supported by the raft is between 50% and 70%. The results of the percentage of load assumed by the raft in the present investigation are included in Figure 8.

Figure 8 shows that the initial proportion of load carried by the raft in all models (for Stage 2) are at the upper limit of the graph and are less than 80%, with an average value of approximately 75%. When water extraction was applied, these values fell to the lower limit of the graph, with values close to 50%.

The load transfer in the pile raft is associated with the system settlements and the possible partial loss of contact between the raft and the soil that is consolidated, as shown by Rodríguez et al. (2020). When comparing the maximum load generated in the PRFS with the real theoretical load on the piles, no percentages greater than 56% were found. However, the load carried by individual instrumented piles increased by up to 16% between Stages 2 and 3.

3.3 Variation in the proportion of the load supported by the piles

Figures 9 and 10 show the proportion of the load supported by the pile group (expressed as a percentage of the total load on the PRFS) and the average settlement of the raft for the different time intervals previously defined for Stages 2 and 3, respectively. The load carried by the piles and the total load on the PRFS at each time interval were used to determine the parameter α_p, i.e., the percentage of load carried by the piles.

In Figure 9, the load percentage on the piles in the initial stage was higher and then decreased with settlement until it reached a more or less constant value. At time t_E, which corresponds to the final time of the load increment (after which it remains constant), the minimum value of the load percentage on the piles was developed. In the final interval (t_E to t_F) there was a small increase in the load ratio until time t_{E1}, becoming almost constant for all models.

For Stage 3 (Figure 10), the proportion of the load supported by the piles in the initial time interval was greater than in Stage 2, but then decreased rapidly. The final percentage of load supported by the piles was higher in Stage 3 than in Stage 2.

In both stages, the condition of the final load percentage on piles was reached at the end of the loading phase (t_E for stage 2 and t_j for stage 3) and remained constant with a displacement of the PRFS from 30 to 40% of the raft thickness. With a behaviour similar to that presented in Figures 9 and 10, Balakumar & Anirudhan (2017) concluded that the piles add load capacity to the rafts, allowing the PRFS to support a greater load, compared to a raft without piles. These results also showed that the load percentage on piles remained constant at high settlement values when the applied load was maintained.

There are no significant differences for greater numbers of piles or larger pile spacings in terms of the response at high settlement in Stage 2. When subsidence induced by deep groundwater extraction occurs (Stage 3), models with greater spacing (M6) present higher values of load distribution factor.

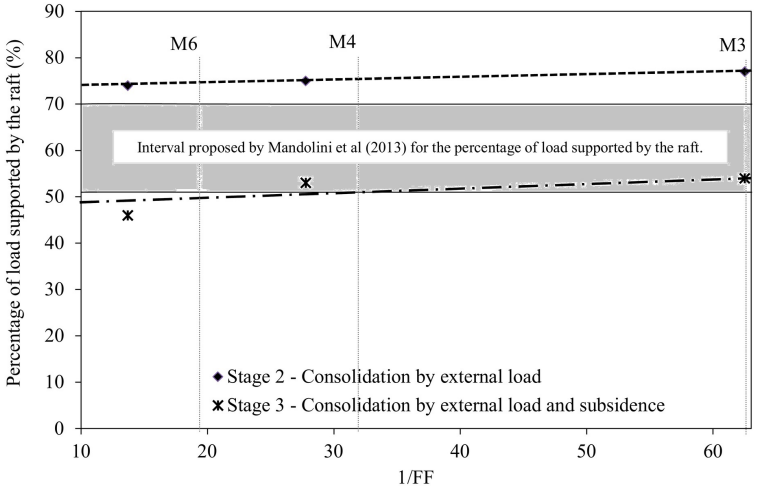


Figure 8. Proportion of load supported by the raft in the models.

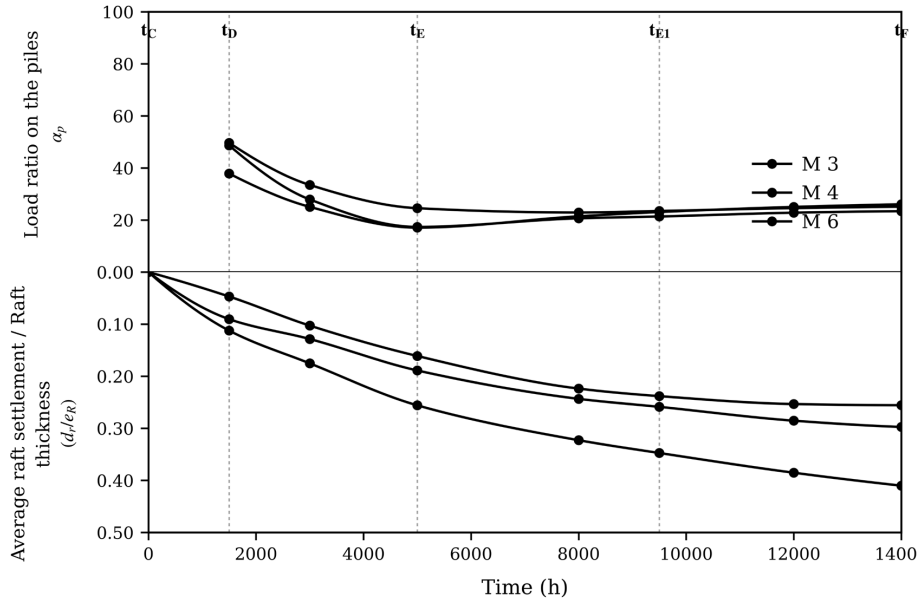


Figure 9. Proportion of load supported by the piles compared with average settlement in Stage 2.

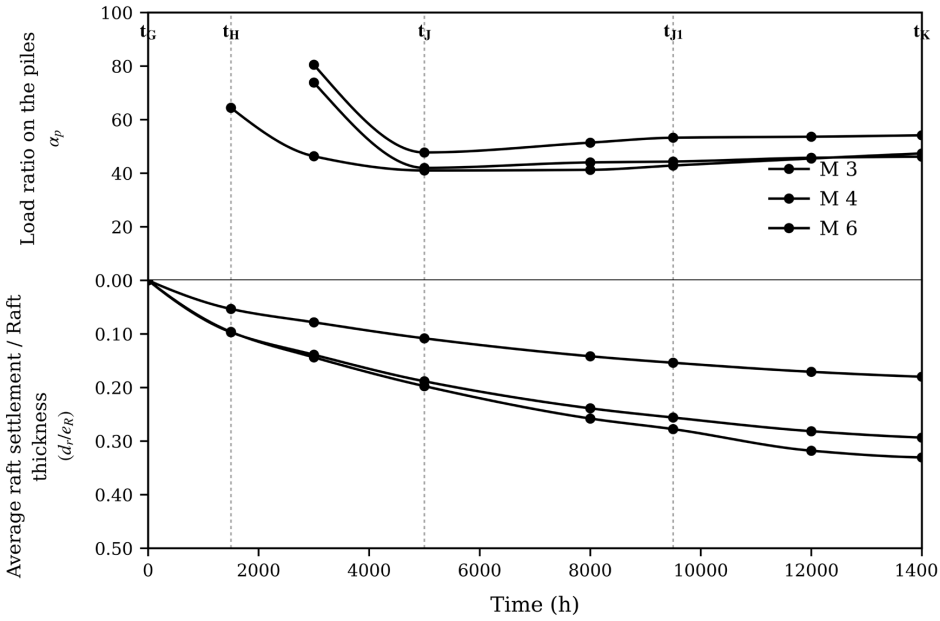


Figure 10. Proportion of load supported by the piles compared with average settlement in Stage 3.

Similar observations were made by Horikoshi & Randolph (1996), Bajad & Sahu (2009) and Balakumar & Anirudhan (2017).

The load percentage on the piles (α_p) depends not only on the geometry of the system but also on the total applied load (Lee et al., 2010). In fact, this coefficient (which expresses the proportion of the load assumed by the piles) also depends on the condition of the water in the soil, as can be observed that the values are higher for conditions where water is extracted from the soil (Stage 3) than for conditions with load only (Stage 2).

4. Conclusions

In this study, the load response of piled raft foundation systems (PRFS) subjected to a double-consolidation process induced by external loading and ground subsidence was investigated using geotechnical centrifuge modelling. Based on the previous studies and the results obtained within the range of parameters investigated, the following conclusions can be drawn:

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- Under external loading (Stage 2), the pile group carried 23–26% of the total PRFS load, while the raft carried 74–77%. Following groundwater extraction (Stage 3), the pile contribution increased to 46–54%, indicating a transfer of 22–28 percentage points of the total load from the raft to the piles.
- Increasing pile spacing from s/d=2 to s/d=9 increased the proportion of load carried by the piles and produced a less uniform distribution within the pile group, with greater loads mobilised by central piles.
- Groundwater extraction produced a substantially greater pile load contribution than external loading alone. The largest change in the tested configurations occurred in M6, for which the pile contribution increased from 26% to 54%.
- The results indicate that groundwater drawdown should be considered in the assessment of piled raft foundations because the load carried by individual piles and by the pile group may increase substantially even when the applied external load remains essentially unchanged.
- The findings are limited to the soil profile, pile configurations, stress levels and drainage conditions investigated in the centrifuge tests. In particular, the intermediate sand layers were not considered effective drainage boundaries in the consolidation-time estimation; therefore, the results should not be directly generalised to piled rafts with fully effective intermediate drainage layers.

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Declaration of interest

The authors wish to confirm that there are no known conflicts of interest associated with this publication and there has been no significant financial support for this work that could have influenced its outcome.

Authors' contributions

Edgar Rodríguez Rincón: conceptualization, methodology, investigation, validation, writing - original draft. Renato Pinto da Cunha: supervision, conceptualization, methodology, resources. Bernardo Caicedo Hormaza: supervision, conceptualization, methodology, resources. Juan Felix Rodríguez Rebolledo: supervision, conceptualization, methodology.

Data availability

The datasets generated and analysed in the course of the current study are available from the corresponding author upon request.

Declaration of use of generative artificial intelligence

This work was prepared without the assistance of any generative artificial intelligence (GenAI) tools or services. All aspects of the manuscript were developed solely by the authors, who take full responsibility for the content of this publication.

List of symbols and abbreviations

d	Pile diameter
dr	Settlement of PRFS
dri	Settlement of raft without piles
ds	Soil displacement
e_R	Raft thickness
f	Load cells at the pile heads
g	Gravitational acceleration
n_l	Scale factor
s	Piles spacing
t	Thickness of the element
t_p^c	Concrete thickness in the prototype
t_m^{mm}	Thickness of the element made from the corresponding material in the model
t_0	Initial time corresponding to specimen preparation and instrumentation installation
t_A	Time at which the maximum acceleration is reached during reconsolidation (Stage 1)
t_B	End of the centrifuge flight during reconsolidation (Stage 1)
t_C	Time at which the target acceleration is reached in Stage 2
t_D	Time at which load application begins in Stage 2
t_E	Time at which the maximum load on the model is reached in Stage 2
t_F	End of the centrifuge flight in Stage 2
t_G	Time at which the target acceleration is reached in Stage 3
t_H	Time at which load application begins in Stage 3
t_J	Time at which the maximum load on the model is reached in Stage 3
t_K	End of the centrifuge flight in Stage 3
w_L	Liquid limit
w_P	Plasticity limit
A_G	Piles group area
A_R	Raft area
B	Base of the element
B	Raft width

Cc	Compression index
Cs	Swelling index
Cv	Coefficient of vertical consolidation
C1	Load cell
E	Modulus of elasticity
E_{m}^{mm}	Elastic modulus of the material in the model
E_{m}^{c}	Elastic modulus of concrete
Es	Settlement measured for the LVDT over the soil
Er	Settlement measured for the LVDT over the raft of the PRFS
FF	Filling factor
Gs	Specific gravity
I_{m}	Moment of inertia
I_{P}	Plasticity index
L	Raft length or pile length
LVDT	Displacement sensor
N_{p}	Number of piles in the group
PRFS	Piled Raft Foundation System
Sr	Settlement reduction radio
α_{p}	Percentage of load carried by the piles

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