

Foundation solutions for large buildings using combined piled-raft foundations

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Abstract – The use of combined piled-raft foundations (CPRF) as a solution for structures with large dimension and/or constructed in poor quality soil is an asset in terms of its behaviour and economy. The raft can resist the entire load of the building while the piles limit differential and global settlements to acceptable values, contributing to an economical solution when compared to a more traditional type of foundation. In the present work, *software SAP2000* is used to model the structure of a building with mixed use (office and residential) with 33 stories in which it was used a piled-raft foundation. It is intended to study the behaviour of this type of foundation when compared with more traditional solutions, such as rafts and pile groups. The obtained values for settlements measured at various relevant points of the structure of the three solutions are compared, and it is carried out as a brief economic study in accordance with the country where it was built, Mozambique. After analysing the results, it is pointed out the superior performance of the CPRF solution, following also by the economy, even when considering the excessive design of the piles to fulfil the serviceability limit states when the ultimate limit states (ULS) are already satisfied.

1. Introduction

A combined piled-raft foundation (CPRF) is, as the name indicates, a mixed solution that joins raft and piles with the purpose of maximizing the benefits of each one of these elements.

This approach explained in Garg, Singh, and Jha (2013), resides in the application of enough piles to reduce settlements to acceptable values. The structural load will be distributed through the raft, piles, and foundation soil. The goal is to optimize the design and settlements and the number of piles to be built.

The use of CPRF is commonly associated with large buildings, in that the ultimate limit states

(ULS) are satisfied with the use of the raft alone, but the settlements are too high to satisfy the service limit states (SLS).

The fact that piles are used to confer the necessary stiffness to meet SLS criteria and the pile is robust enough to handle the loads of the building, satisfying the ULS criteria, allows an economical design in terms of the number of piles to be used when comparing to a traditional pile group. It is also possible to reduce the thickness of the raft when comparing to a traditional raft solution, contributing overall to a more economical solution without compromising safety.

While comparing with more common types of foundations, it is safe to state that CPRF:

- Economically more viable when applied in settlement sensitive soil;
- Diminishes global and differential settlements;
- Faster construction time when compared to rafts;
- Smaller quantity and dimension of piles when compared to pile group;
- Smaller stresses in raft due to the strategic positioning of piles.

The building characteristics, the elements around it, and the foundation soil are a very influential factor in the use of piled-raft foundations.

According to Poulos (2001), the use of CPRF is preferable in cases where the raft can support the entire load of the building, but the total and differential settlements are larger than admitted. Although this is a typical situation where CPRF are used, it is not the only situation where this type of foundation is preferred.

The applicability of piled-raft foundations requires an extensive understanding of soil-structure interaction and the geotechnical characteristics of foundation soil. The ideal soils for this kind of solutions are clay or sand with some density.

It is crucial that the raft and soil are in total contact to allow an increase in pressure of the soil below and optimise its resistance due to consolidation. As Katzenbach, Bachmann, Boled-Mekasha, and Ramm (2005) describe, the pressure transfer between foundation and soil increases the lateral stresses applied to the piles; therefore, allowing a better response from the piles in ULS.

A loss of contact between the foundation and the soil, which may occur during settlements caused by soil consolidation, will result in a loss of

stiffness of the whole foundation system due to the lack of contribution of the raft. Cooke, Bryden-Smith, Gooch, and Sillet (1981) claim that there will be an additional 55% to 75% load that should be carried by the piles in such a case.

There are 3 distinct design philosophies according to Randolph (1994), each one with an increase of structural responsibility on the piles compared to its prior:

- Conventional approach;
- Creep piling;
- Differential settlement control.

2. Design

There are several design methods for combined piled-raft foundations, with guidelines from Poulos, Small, and Chow (2011), Fellenius (2015), Poulos and Davis (1980), among others.

The main goal is that the load is transferred to the soil by the raft and piles, to a more resistant soil. The loads are resisted by friction between the raft and soil and by the lateral friction and end bearing capacity of the piles.

The design method should consist of 3 phases:

- Preliminary study where the viability of the solution is studied, and there's a value of the number of piles that should be applied per the design criteria;
- Mapping of the piles and definition of their general characteristics;
- Detailed study of the optimal number of piles and critical spots of application. Determination of settlements, bending moments, and shear stresses in the raft as well as loads and bending moments on the piles.

The design of CPRF solutions considers, according to Poulos (2001):

- Ultimate bearing load for axial, shear stresses, and bending moments;

- Maximum total settlement;
- Differential settlement;
- Bending moments and shear stresses in the raft;
- Bending moment and loads on the piles.

3. Model

The modelling was performed using software SAP2000.

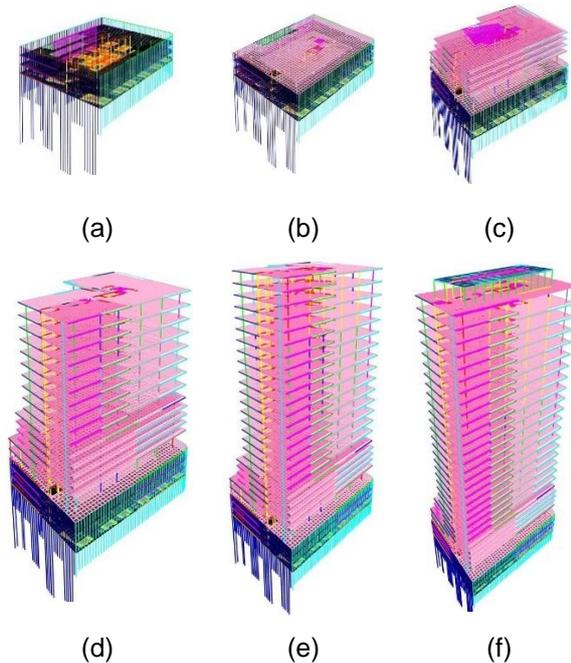


Figure 1 Several stages of analysis for the model. (a) Phase 1 (b) Phase 2 (c) Phase 3 (d) Phase 4 (e) Phase 5 (f) Final phase

According to the geotechnical report, the foundation soil consisted of “fine to medium grain sand, silty, with reddish/orange/yellow tones, with medium compaction.”

The report stated that 3 geotechnical zones were created with respect to the soil capacity:

- Zone 1: Best quality soil - consists of silty fine sand of yellow/orange tones, medium compaction (NSPT>60);
- Zone 2: Intermediate quality zone - consists of silty fine grain sand, of reddish tones and medium compaction (15<NSPT<55);

- Zone 3: Worst quality soil – consists of loose fine sand of reddish tone (NSPT<17).

The reproduction of the soil solution in the model is made by using Winkler’s method, a linear elastic theory. On the raft, there will be a distribution of area springs with the same stiffness as the one considered for the soil. The spring stiffness will be calculated according to Gazetas (1991), considering buried foundations.

4. Analysis

To classify the model as apt to be compared to a real-life building, there were some verifications:

- (1) Frequency of vibration;
- (2) Weight comparison;
- (3) Settlement values;
- (4) Vertical loads.

(1) Frequency of vibration

A frequency of vibration of 0.34Hz from the model was admitted to be a reasonable value according to the dimension of the building. The vibration modes were also adequate considering the building’s geometry and placement of rigid elements. The first vibration mode consisted of translation in the y-direction. The next relevant vibration modes consist of torsion and translation in the x-direction with slight torsion. During this simple analysis, it was possible to certify that all the elements were well connected and that the building moved according to its geometry.

(2) Weight comparison

The next point of analysis was the global weight of the structure. The results can be seen in Table 1.

Table 1 Weight comparison (kN) of real and modelled structure

Weight of the real building	Weight in the model
kN	kN
466760	474961

A variation of up to 10% between values was considered acceptable due to the approximations that were necessary when constructing the model and accounting for the elements in the blueprints. The values from the table above suggest a very close result, with only 1.8% difference.

(3) Settlement values

The displacement criteria used during monitorization of the building can be seen in Figure 7, according to JetSJ (2013).

The location of the topographical targets is shown in Figure 6 (red markings).

An iterative process was conducted, assigning each topographical target with an influence length. The total base reaction for the respective length was used to approximate the displacements, as it can be seen in Eq.(1).

$$K_e = \frac{F}{\delta \times n} \tag{1}$$

- K_e - Stiffness in each pile;
- F - Sum of the base reactions of the piles included in the influence length;
- δ - Desired displacement for that influence length;
- n - Number of piles included in the influence length.

The difference between the model displacements and the ones measured at the construction site are shown in Figure 2. Although there are some differences, it is still classified as a good approximation. The iterations were made until there was a difference of values smaller than 1mm or the approximation stagnates.

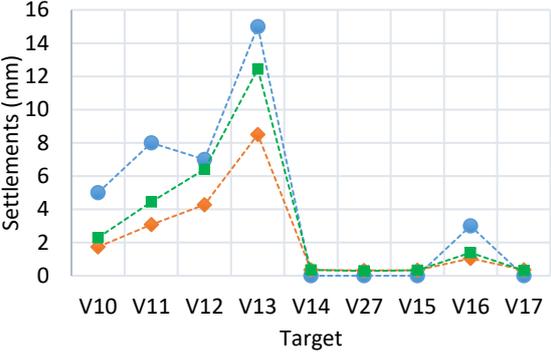


Figure 2 Comparison of vertical displacements in peripheral beam along iterations

(4) Vertical loads

A comparison between load distribution was made. Having the original mappings of loads that should be supported by the earth retaining wall, a comparison was made to see if the loads carried by the modelled elements were the same in the different zones.

The loads in the model were obtained through the sum of all the vertical reactions in the base of the elements representing the piles (from the earth retaining wall) in each zone.

Table 2 Comparison of vertical loads during design phase and model

Load mapping	Model
kN	kN
60704	48286

As it can be seen in Table 2, both values are similar. In the load mapping, during the design phase, the earth retaining wall was responsible for supporting 12.80% of the building's total load, and in the model, this value is about 10.18%. The difference between the values is disregardable. Even though the same load is sustained by the model and building, the same distribution among zones was not verified.

5. Methods

Two opposing solutions were made up in order to compare them with the combined piled-raft solution. Both are composed of more traditional

foundation types, and the methods of design used were simplified to get a realistic solution.

The alternative solutions are: (1) Pile group, and (2) A raft.

(1) Pile group

For this solution, it was admitted that:

- Pile length will be constant and enough to go through zone ZG2 with an additional length in zone ZG1 for added security;
- Being a theoretical solution, the same materials and procedures of the actual construction will be used: concrete C30/37 and bored piles using Kelly method;
- Pile dimensions will be of the same magnitude of the ones in construction, with a length of 25m and a diameter of 1000mm.

Using the soil report provided by Tecnasol (2013), It is possible to extrapolate the bearing capacity of the piles due to its likeness. For the assessment, the Bustamente e Gianceselli method was used.

From Montoya, Meseguer, and Cabré (2001), it was possible to get a relationship for determining the height of the pile cap. The height used was 2.90m.

After establishing the pile cap and pile geometry, it was necessary to access the number of piles and its placement, so that it could make a secure pile group to use as the foundation of this building.

The number of piles was established according to Eq.(2). It was necessary to have 180 piles.

$$R_{c,d} \geq \frac{F_z}{n} \pm \frac{M \times e}{\sum e^2} \quad (2)$$

- F_z - Vertical reaction;
- n - Number of piles;
- M - Bending moment;

e - Eccentricity between pile and load.

The distance between piles is the one that allows all the piles to be accommodated in the pile cap. Each pile was modelled as a vertical spring with adequate stiffness. The stiffness was dependent on the position of the pile in the pile group according to Viggiani, Mandolini, and Russo (2012). This procedure allows to segment pile solicitation per their positioning, as It is expected that piles in the centre of the pile cap will have less load solicitation than the ones in the periphery. For the pile group, there was no modelling of soil below the cap as it was intended to have only the pile group responsible for bearing the total building's load.

(2) Raft foundation

For the design of the raft, some guidelines in Montoya et al. (2001) were used concerning the thickness of the slab. The initial thickness considered was of 1.30m, subject to modifications depending on the stresses.

After designing the alternative solutions, it was important to make sure that it was prepared to support the structure. The verifications made were:

- (1) Vertical reactions and settlements in piles of the pile group;
- (2) Stresses in pile cap and raft;
- (3) Punching in pile cap and raft;
- (4) Bearing capacity of the soil for the raft

(1) Vertical reactions and settlements in piles of the pile group

Piles reactions were compared to their bearing capacity in the corresponding soil, according to the soil studies performed.

The reactions in the piles are always smaller than the bearing capacity, with a high degree of safety. The pile with the highest reaction (E166)

is still below the maximum admitted values, and it can be justified by its positioning. E166 is one of the corner piles, and its higher axial stress is boosted by the effect of the bending moments. The maximum settlement allowed in each pile was defined according to art 7.6.1.1 (3) of EC7 (2010). All the piles are far from the maximum settlement. Once again, E166 represents the highest value, as it was expected since it presented the highest reaction as well.

(2) *Stresses in pile cap and raft*

For verifying the security regarding the pile cap and raft, the stresses derived from the bending moments in both 'X and Y' directions were accessed. For the pile cap, there is Figure 3 and Figure 4, concerning the moments in the X- and Y-direction, respectively. The maximum values in the X-direction reach values of about 27000 kN.m/m. In the Y-direction, the highest values come close to 38000 kN.m/m.

That is justified due to a higher stiffness in that zone, the front and right side of the earth retaining wall, that barely have settlements prevent some movement from the slab enticing higher stresses in that vicinity. The values were considered as normal since they resulted in μ of 0.14 and 0.20, and its typical values can reach up to 0.25-0.30.

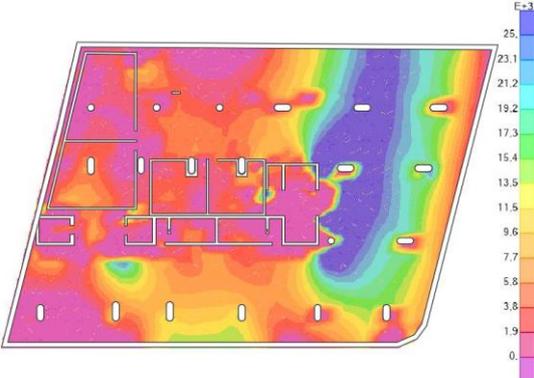


Figure 3 M_{11} diagram of pile cap, in kN.m/m

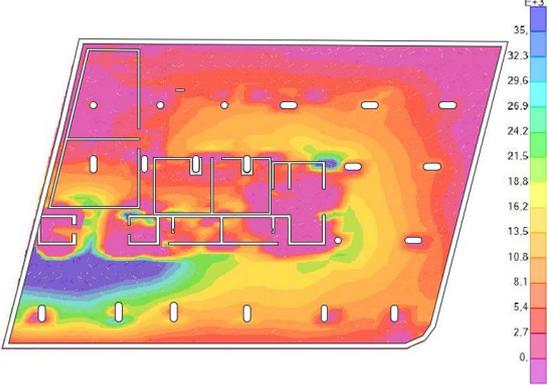


Figure 4 M_{22} diagram of pile cap, in kN.m/m

The same analysis was performed regarding the raft. The highest values for bending moments are located in the same areas as the pile cap. In this case, the magnitude of the values is considerably smaller, founding maximum values of about 13000 kN.m/m for M_{11} and M_{22} due to the difference in thickness. Even though the values are smaller, the μ values were about 0.35 for the initial thickness designed. So, in order to get to acceptable values, there was an increase of 0.20m in thickness that allowed values of μ of under 0.30.

(3) *Punching in the pile cap and raft*

For the punching verification, the column that had the highest vertical reaction was accessed, and it was then considered a puncturing area according to Eurocode 2. The dimensions varied between pile cap and raft due to the variation in thickness of both solutions.

Both solutions were well designed for punching; the pile cap is having a safety factor of over 2 and the piled-raft of about 1.40.

(4) *Bearing capacity of the soil for the raft*

The bearing capacity of the soil was verified accordingly to art. 6.5.2 of EC7 (2010). The stresses supported by the soil can be seen in Figure 5.

The only areas that came close to the soil's bearing capacity were under some columns, and due to the lack of similarity with reality (having a

column represented by a point and not a section with a designated area), it was considered that the soil could support this solution.

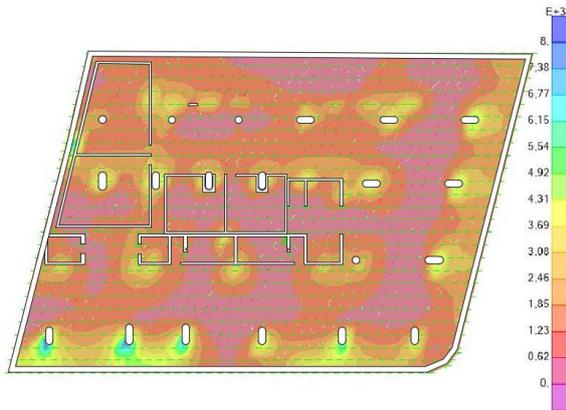


Figure 5 Stresses under the raft in kN/m²

6. Results from the model

When analysing the solutions, two different approaches were taken:

- (1) Global behaviour of the solutions along all the phases of construction;
- (2) Comparison of the 3 solutions in the same stage of construction.

For both procedures, the targets measured can be seen in Figure 6, and are a mix of actual targets measured during construction and spots where the displacements were bigger in the model, in order to check the more impactful points.

The stages considered can be seen in Figure 1.

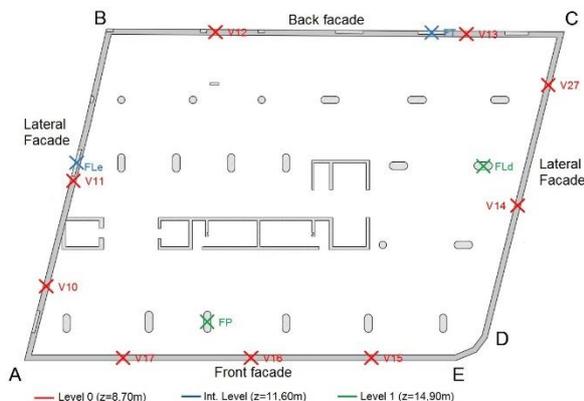


Figure 6 Location for the targets where displacements were measured

(1) Global behaviour of the solutions along all the phases of construction

As it can be seen in Figure 7 (a), the settlements in the CPRF solution are usually small apart from V13 (that corresponds to FT) that shows a higher value than expected at the late stages of the construction. The lack of measurements done in the real building at a similar phase makes it impossible to make a fair comparison. Nonetheless, it can be justified that by some simplification done to reproduce the building into the model, there's also the impossibility to model the building real behaviour in a computational model.

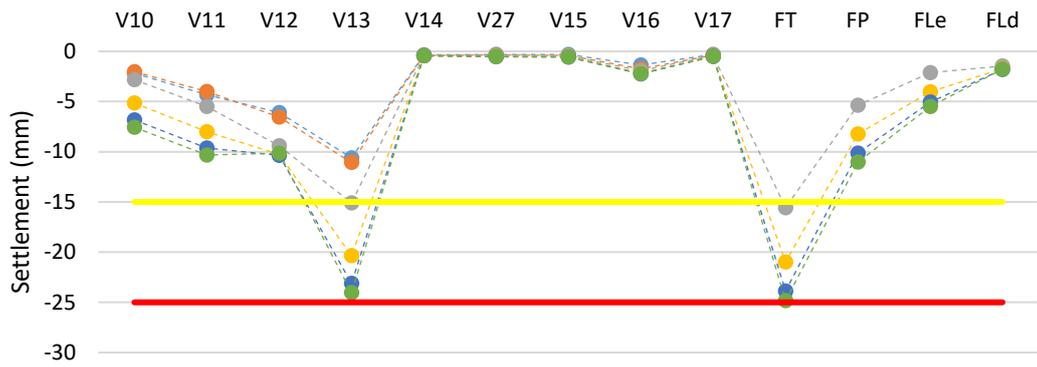
(2) Comparison of the 3 solutions in the same stage of construction

Figure 7 (b) shows the behaviour of the raft. It is easily concluded that since the early stages of construction, the settlements are well above what is expected, which leads to conclude that this is not an appropriate solution for the building's foundations.

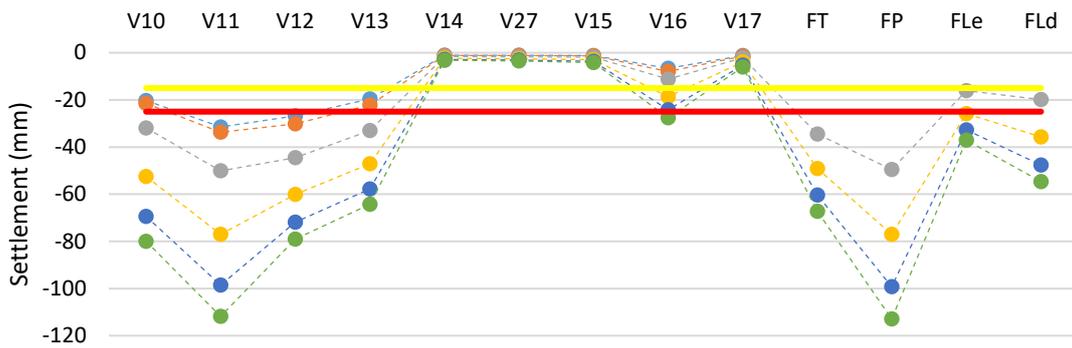
Since the settlements in FP and FLd (points situated in columns) are higher than in targets in the same region placed in the beam over the earth retaining wall (V12, V13, V14, and V27), it can be said that the displacement from the raft is higher than the one of the earth retaining walls.

The settlements from the pile group are shown in Figure 7 (c), and it shows a similarity to the raft behaviour, even though there's a slight reduction in the magnitude of the settlements. It can be said that the pile group has a slightly better behaviour than the raft, but again, it is an unusable solution for the problem in question.

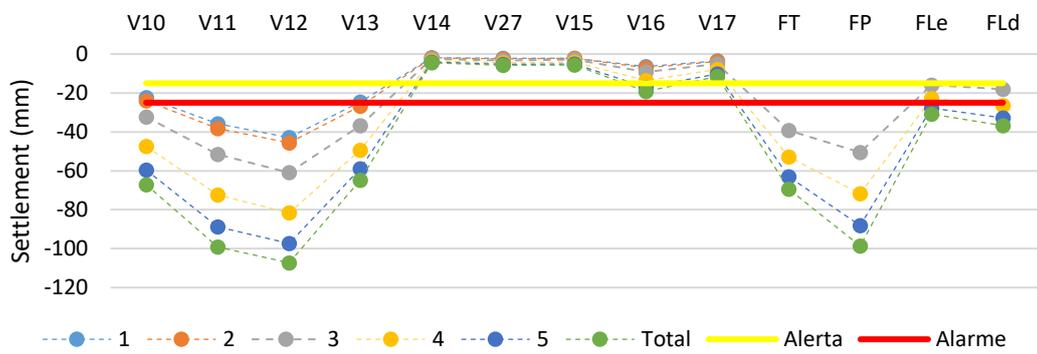
Other comparisons were made regarding the 3 solutions in each phase. For explanation purposes, one of the phases will be displayed in Figure 8.



(a)



(b)



(c)

Figure 7 Global behaviour during construction phases of (a) CPRF, (b) Raft and (c) Pile group

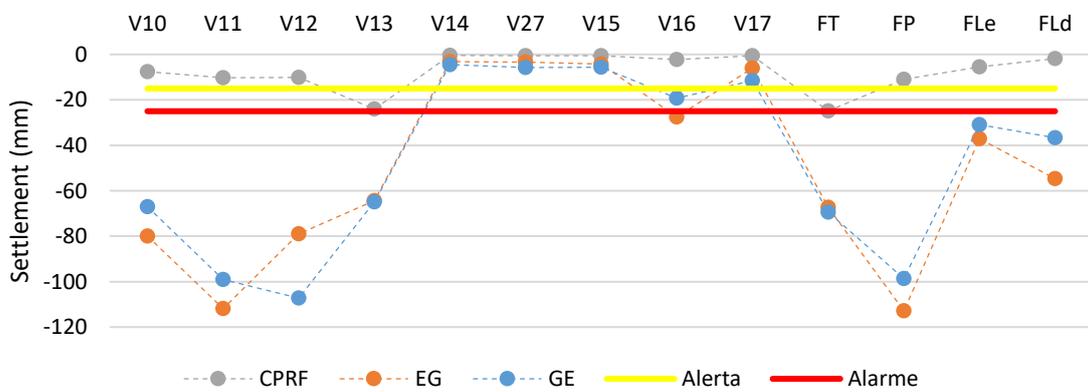


Figure 8 Comparison of CPRF, pile group, and raft after completed building

7. Economic analysis

A brief economic study was conducted. An online tool provided by CYPE Ingenieros n.d., allowed to generate prizes for different construction procedures in Mozambique. The approximations made were: When there was a lack of information, the most feasible situation was considered regarding the type and location of the construction.

As it can be seen in Table 3, CPRF is the most economical solution, followed by the raft, and then the pile group. The main savings factor for the CPRF is the capability of having a thin raft when compared to the other solutions.

Table 3 Total cost of all solution (€)

	CPRF	Pile group	Raft
Total cost	244 923,81 €	954 282,47 €	309 420,91 €

8. Final considerations

– Some simplifications were made due to the complexity that is converting finished blueprints into a model. It is believed that the simplifications made did not change the results greatly and the model is a fair representation of the real behaviour;

–The soil representation was made using vertical springs with adequate stiffness and even though there were methods that could have imposed a higher quality representation of soil, such as using rotational springs or varying spring stiffness per soil zones, etc., for vertical displacements, the method used was adequate;

–The settlements obtained in Figure 2 were not all exact. Locations V10, V12, and V13 had some variations with the displacements measured. To approximate values, the stiffness of those areas were sequentially diminished with no alteration of the settlement, which leads to believe that the stiffness present will be lower than the real one. It

doesn't seem to hurt the study performed since it could have presented settlements higher than the criteria, which didn't occur;

–To further approximate the model to reality in regard to the settlements of V10, V12, and V13, the rotational stiffness of the peripheral beam on the earth retaining wall could have been changed in different zones. Since the actual beam varied in height and was modelled using the average value, it would approximate reality and could have inflected the vertical displacements in those targets;

– Varying the axial stiffness of the earth retaining wall piles would also allow for a higher similarity between settlements. For every similar group of piles, the adopted length was the one designated as a minimum in the blueprints. In the actual construction, the value could have varied

– CPRF showed a distinct performance while being compared to the other solutions. Even though its behaviour allowed for a fair difference between the options, there were still a few settlements that were higher than expected. The lack of measurements during a similar construction phase doesn't allow a fair comparison, but that can be attributed to some degree of error both in the model and the measurements;

– The reduced capacity of the soil due to the group effect of the pile group wasn't able to be reproduced with SAP2000 due to its lack of soil modelling feature. It was verified that stiffness would alter the results completely;

– Stiffness of soil for all the piles in a group would result in an extremely stiff structure base with very small movements, which did not represent reality;

– The solutions apart from CPRF could have been more precisely designed in order to make them more competitive in the comparison.

Changes would comprise different materials and construction techniques, and soil manipulation.

9. References

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