Comparative study of design methodologies regarding a shallow foundation of a pipeline end manifold (PLEM)

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1. Introduction

The Pipeline End Manifold (PLEM - Figure 1) is an equipment employed in the subsea systems of oil and gas production. The main objective of the equipment is to split the production flow into multiple routes. In general, the production reaches the PLEM, and later split the oil or gas into multiple routes to a FPSO (Floating Production, Storage and Offloading), for offshore works.

The PLEMs are often constructed in rectangular geometry of mudmat foundations. During installation and operation, these foundations are subjected to simultaneous vertical, horizontal, moment and torsional (twist) loading, mainly due to their self weight, pipeline imposed loads, and environmental loads.

An important point of a mudmat foundation design is the evaluation of the bearing capacity (MOREIRA, 2011), (HERAVI, 2018) and (ABDALLA & HOSSAIN K, 2013), the sliding effect, the short term settlement, etc. All calculations are based on numerous load combinations, in order to represent the installation and operation conditions. Usually, the calculation methodology are in accordance to the design codes, based on analytical calculations. Alternatively, another methodology calculation based on the Finite Element Method (FEM) can be employed on the analysis to check the critical loads, and aiding the Structural Engineers to assess the better geometry of the mudmat, where the foundation will not fail.

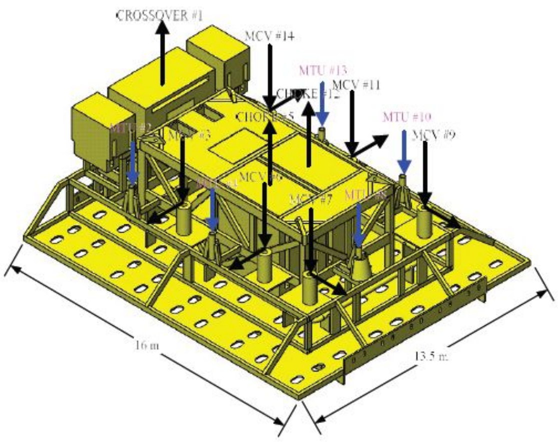
The aim of this paper is to carry out a comparative study regarding three different standards (ISO 19901-4, 2016), (API-RP-2A WSD, 2000) and (DNVGL-RP-C212, 2017) related to Shallow Foundation Design of Subsea Structures, as a design methodology, to analyze an analytical model of an interaction between a Pipeline end Manifold (PLEM) and the soil (Seafloor). Furthermore, it was conducted a numerical analysis about the soil structure interaction employing the Finite Element Method (FEM), based on equations of Elasticity and Plasticity Theory. The FEM is able to generate highly accurate solutions (DUNNE, MARTIN, MUIR, BROWN & WALLERAND, 2015) and (MARTIN, DUNNE, WALLERAND & BROWN, 2015), and it can consider highly complex geometry. Moreover, the Method is able to represent complex material behavior and boundary problems.

In several cases, the engineer has to decide, which rule is more suitable, then, this work presents the results of an analytical model regarding three different rules, therefore, aiding the engineer to understand the differences between the three design condes employed. In addition, the numerical model of the structure considered (PLEM) was verified using the FEM, and the results were compared to the analytical model. Thus, the work can contribute to the choice of the better design code that fit to the project.

Keywords: Shallow Foundation; Mudmat Foundations; Subsea Structure Foundation.

The results in terms of short term settlement, sliding capacity and bearing capacity were very close, by using the calculation methodologies presented on the standards regarded on the work (ISO 19901-4, 2016), (API-RP-2A WSD, 2000) and (DNVGL-RP-C212, 2017). All responses related to Analytical and Numerical analysis were compared to, and the results were discussed and presented.

Figure 1 - Pipeline End Manifold - PLEM.



Source: Extracted from (DANDOULAKIS, MARIA & NASCIMENTO, 2017).

2. Methodology

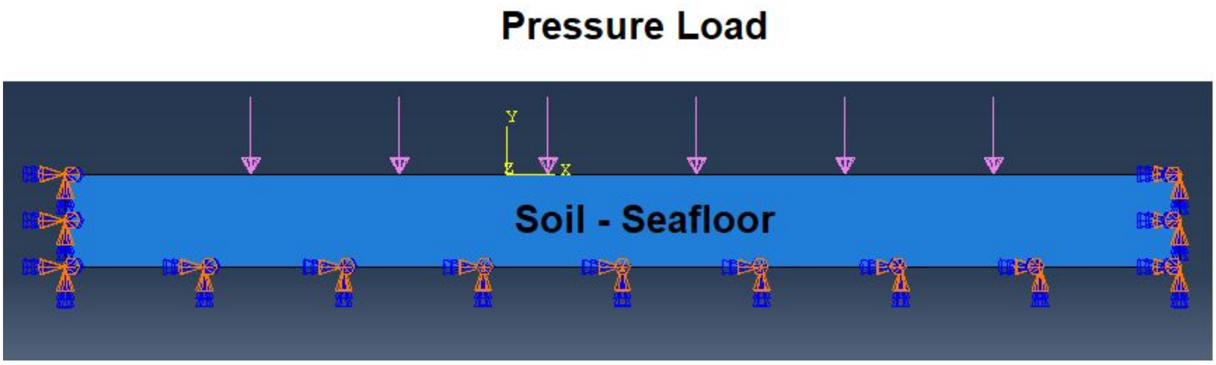
2.1. Analytical model

In the current work, it was employed the standards (ISO 19901-4, 2016), (API-RP-2A WSD, 2000) and (DNVGL-RP-C212, 2017), to develop the analytical calculations related to the bearing capacity, settlements and sliding capacity of the foundation. Furthermore, the work presented on (DANDOULAKIS, MARIA & NASCIMENTO, 2017) was taken as a reference, which was based on the standard (API RP 2GEO, 2011).

2.2. Finite element model

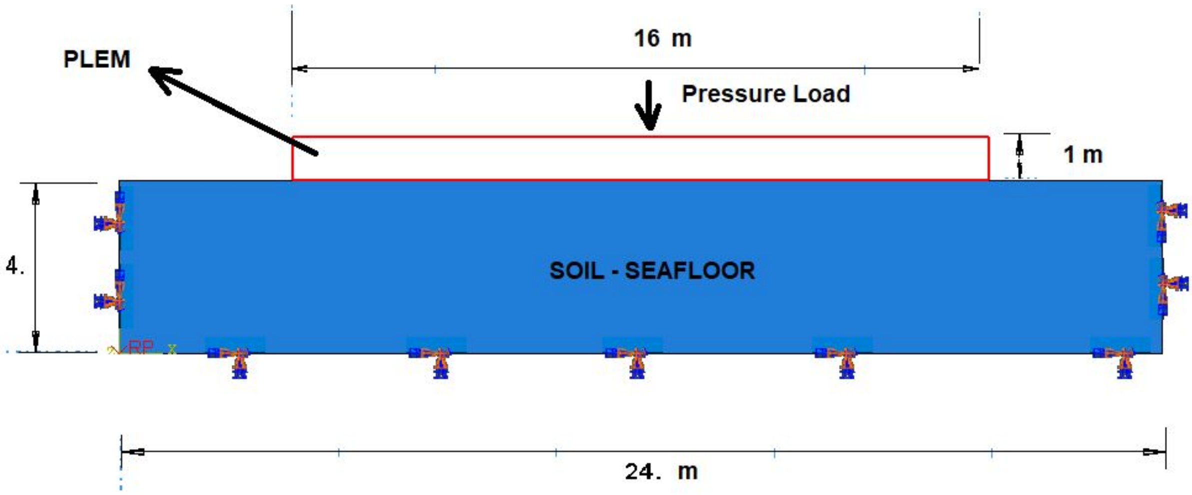
The numerical model was developed using the software (ABAQUS, 2011). Initially, it was developed a model regarding only the soil, and applying the pressure directly on the seafloor, as shown on Figure 2. However, that kind of modeling is not advisable, because the results do not represent the real situation satisfactorily. Thus, it was developed a model including the PLEM, Figure 3. The seafloor was modeled as a shell element, and the PLEM, as a wire, rigid element. Moreover, the elastic theory was employed for the soil model (Young's modulus, E = 4339 kPa (DANDOULAKIS et al., 2017), and the Mohr Coulomb Plasticity model was regarded (Undrained shear strength, Su = 2 + 1.5 x h kPa, h is the depth of soil). The numerical contact model was set as contact hard. Furthermore, no gap between the soil and the rigid element was allowed in the numerical simulations.

Figure 2 - First Model Regarding only the Soil.



Source: Extracted from (ABAQUS, 2011).

Figure 3 - PLEM-Soil Interaction Model.

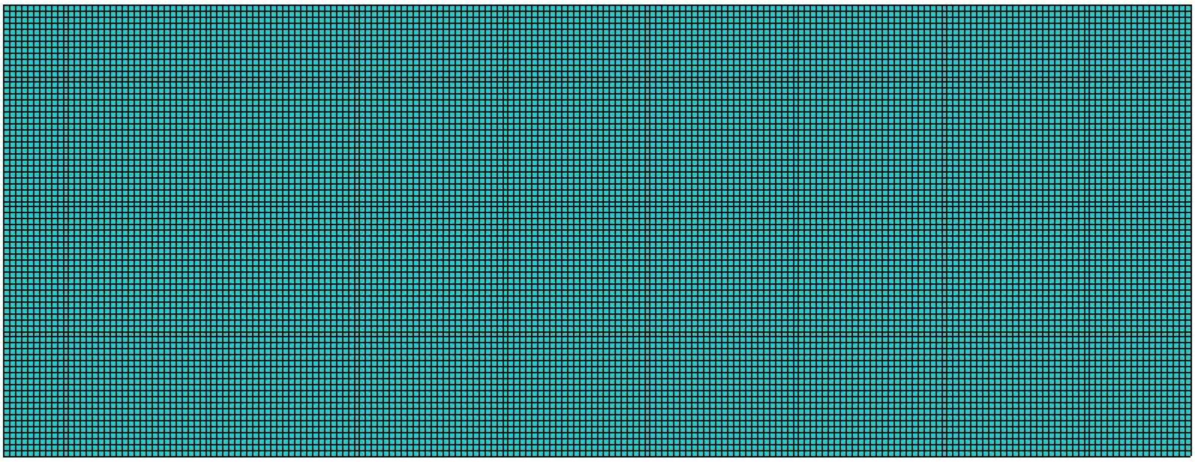


Source: Extracted from (ABAQUS, 2011).

2.2.1. Model mesh and boundary conditions

The PLEM mesh was built with element size of 0.2 m, and the Soil with 0.5 m. The dimensions of the PLEM and Soil (Figure 3) were adapted from reference (DANDOULAKIS et al., 2017). They were used squares elements for the soil mesh (Figure 4). Furthermore, the displacements of soil were restricted on the vertical and horizontal direction, for the lateral side and bottom (Figure 3). With respect the rotations, they were restricted too, on the same mesh nodes of the restricted displacements.

Figure 4 - Soil Mesh..



Source: Extracted from (ABAQUS, 2011).

3. Soil data

The soil was modeled as a Soft Clay, and all data (Table 1) related to were extracted from (DANDOULAKIS et al., 2017). Furthermore, all data with respect the applied loads (15 Cases) were extracted from (DANDOULAKIS et al., 2017), too.

Table 1 - Soil Data.

Parameter	Unit	Value
Submerged unit weight of the soil	kN/m^3	6.1
Undrained shear strength	kPa	$2 + 1.5 \times h^*$
Young's modulus	kPa	4339
Elastic shear modulus	kPa	1446
Poisson's ratio	---	0.5
Ground inclination angle	Degree	0

* h = depth of soil

Source: Adapted from (DANDOULAKIS et al., 2017) apud (FAGUNDES, 2012).

4. Results

4.1. Analytical model

To develop the Analytical Model, it was used the equations of the standards (ISO 19901-4, 2016), (API-RP-2A WSD, 2000) and (DNVGL-RP-C212, 2017). Thus, the Short and Long Term Settlements, the soil stiffness, Sliding Stability and the Undrained Bearing Capacity were assessed. The main results due to the applied vertical loads are presented as follows.

Table 2 - Short Term Settlement.

Case	Vertical Load (kN)	ISO-19901-4 Short Term Settlement (m)	API 2A WSD Short Term Settlement (m)	DNVGL-RP-C212 Short Term Settlement (m)	Percentage Difference ISO-DNV	Percentage Difference API-DNV
1	-2,315.80	-0.024	-0.024	-0.03	18.86%	18.86%
2	-2,260.90	-0.023	-0.023	-0.029	19.13%	19.13%
3	-2,260.90	-0.023	-0.023	-0.029	19.13%	19.13%
4	-2,161.80	-0.023	-0.023	-0.028	18.59%	18.59%
5	-2,161.80	-0.023	-0.023	-0.028	18.59%	18.59%
6	-2,315.80	-0.024	-0.024	-0.03	18.86%	18.86%
7	-2,164.10	-0.023	-0.023	-0.028	18.59%	18.59%
8	-2,260.90	-0.023	-0.023	-0.029	19.13%	19.13%
9	-2,260.90	-0.023	-0.023	-0.029	19.13%	19.13%
10	-2,161.80	-0.023	-0.023	-0.028	18.59%	18.59%
11	-2,161.80	-0.023	-0.023	-0.028	18.59%	18.59%
12	-2,352.00	-0.025	-0.025	-0.03	18.59%	18.59%
13	-2,421.90	-0.025	-0.025	-0.031	18.59%	18.59%
14	-2,421.90	-0.025	-0.025	-0.031	18.59%	18.59%
15	-2,315.80	-0.024	-0.024	-0.03	18.86%	18.86%

Source: Produced by the author.

Table 3 - Effective Area.

Case	Vertical Load (kN)	ISO-19901-4 Effective Area (m^2)	API 2A WSD Effective Area (m^2)	DNVGL-RP-C212 Effective Area (m^2)	Percentage Difference ISO-DNV	Percentage Difference API-DNV
1	-2,315.80	217.424	217.424	183.685	15.52%	15.52%
2	-2,260.90	218.848	218.848	184.958	15.49%	15.49%
3	-2,260.90	218.848	218.848	184.958	15.49%	15.49%
4	-2,161.80	216	216	182.25	15.63%	15.63%
5	-2,161.80	216	216	182.25	15.63%	15.63%
6	-2,315.80	217.424	217.424	183.685	15.52%	15.52%
7	-2,164.10	216	216	182.25	15.63%	15.63%
8	-2,260.90	218.848	218.848	184.958	15.49%	15.49%
9	-2,260.90	218.848	218.848	184.958	15.49%	15.49%
10	-2,161.80	216	216	182.25	15.63%	15.63%
11	-2,161.80	216	216	182.25	15.63%	15.63%
12	-2,352.00	216	216	182.25	15.63%	15.63%

Source: Produced by the author.

Table 3 - Effective Area (**Continuation**).

Case	Vertical Load (kN)	ISO-19901-4 Effective Area (m²)	API 2A WSD Effective Area (m²)	DNVGL-RP-C212 Effective Area (m²)	Percentage Difference ISO-DNV	Percentage Difference API-DNV
13	-2,421.90	216	216	182.25	15.63%	15.63%
14	-2,421.90	216	216	182.25	15.63%	15.63%
15	-2,315.80	217.424	217.424	183.685	15.52%	15.52%

Source: Produced by the author.

Table 4 - Bearing Capacity.

Case	Vertical Load (kN)	ISO-19901-4 Bearing Capacity (kN)	API 2A WSD Bearing Capacity (kN)	DNVGL-RP-C212 Bearing Capacity (kN)	Percentage Difference ISO-DNV	Percentage Difference API-DNV
1	-2,315.80	6,397.22	10,043.07	4,972.73	22.27%	50.49%
2	-2,260.90	6,418.27	10,048.86	4,980.86	22.40%	50.43%
3	-2,260.90	6,418.27	10,048.86	4,980.86	22.40%	50.43%
4	-2,161.80	6,380.87	10,049.11	4,966.00	22.17%	50.58%
5	-2,161.80	6,380.87	10,049.11	4,966.00	22.17%	50.58%
6	-2,315.80	6,397.22	10,043.07	4,972.73	22.27%	50.49%
7	-2,164.10	6,380.87	10,049.11	4,966.00	22.17%	50.58%
8	-2,260.90	6,418.27	10,048.86	4,980.86	22.40%	50.43%
9	-2,260.90	6,418.27	10,048.86	4,980.86	22.40%	50.43%
10	-2,161.80	6,380.87	10,049.11	4,966.00	22.17%	50.58%
11	-2,161.80	6,380.87	10,049.11	4,966.00	22.17%	50.58%
12	-2,352.00	6,380.87	10,049.11	4,966.00	22.17%	50.58%
13	-2,421.90	6,380.87	10,049.11	4,966.00	22.17%	50.58%
14	-2,421.90	6,380.87	10,049.11	4,966.00	22.17%	50.58%
15	-2,315.80	6,397.22	10,043.07	4,972.73	22.27%	50.49%

Source: Produced by the author.

The Standards (ISO 19901-4, 2016) and (API-RP-2A WSD, 2000) present equations very similar for the calculation of the Short Term Settlement. As a consequence, the results shown on Table 2 were the same. The same comment is valid for the results presented on Table 3. Otherwise, it is not valid for the Bearing Capacity which the Standards present different equations to calculate the correction factors. Consequently, presenting different results.

In general, Table 2, Table 3 and Table 4 presented results were very similar for all standards regarded, except for the comparison between (API-RP-2A WSD, 2000) and (DNVGL-RP-C212, 2017) with respect to the Bearing Capacity (Table 4), presenting percentual differences of the order of 50%. The calculations from (DNVGL-RP-C212, 2017) presented more conservative values. Probably, the reason is due to the differences in the Effective Area (Table 3), and the different approach to calculate the correction factors. However, the Usage Safety Factors for all standards presented suitable values, although the results have not been presented here. The Required Safety Factor was considered equal to 2 (API RP 2A-WSD, 2000).

The complete results with respect Long Term Settlements, Soil Stiffness and Sliding Stability were not shown on the present work. However, they were determined and the summary is discussed hereafter.

The results in terms of Long Term Settlements were very similar for all the standards considered, and around 0.094 m (First Load Case). It is important to mention that the Long Term Settlement is calculated in function of the compressibility index (Cc), specially for the rules (ISO 19901-4, 2016) and (API-RP-2A WSD, 2000). For example, if an index of Cc = 1.17 (See reference (BARROS, SILVEIRA & AMARAL, 2009), for Jubarte field) were applied, instead of Cc = 0.515 (DANDOULAKIS et al., 2017), the Long Term Settlement would be around 0.213 m (First Load Case).

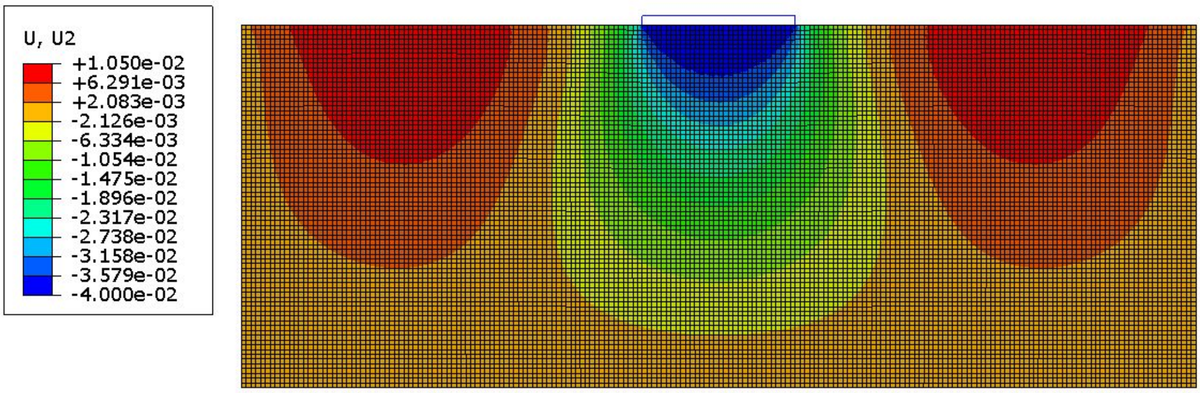
The results in terms of Soil Stiffness were very close for all rules considered on the current work, and around 0.44 MN/m³.

For the Sliding Stability assessment, the results were very close, presenting values around 288.00 kN. The Usage Safety Factors for all standards presented suitable values (The Required Safety Factor was considered equal to 1.5 (API RP 2A-WSD, 2000).

4.2. Finite element model

To develop the Numerical Model, it was employed the Software (ABAQUS, 2011). Thus, the vertical displacements, due to the applied vertical loads (Section 3), were determined, see Figure 5 and Figure 6. The Figure 6 shows a detail of the interaction between the PLEM and Soil (Figure 5). The vertical displacements were obtained at the middle of the PLEM, between the PLEM and Soil, node 101. Moreover, the von Mises Stress were estimated according to the results presented on Figure 7. Finally, the graphic shown on Figure 8 was created, in order to compare the Analytical and Numerical results. The responses are related to the Short Term Settlement (Vertical Displacements) in function of the applied Stress.

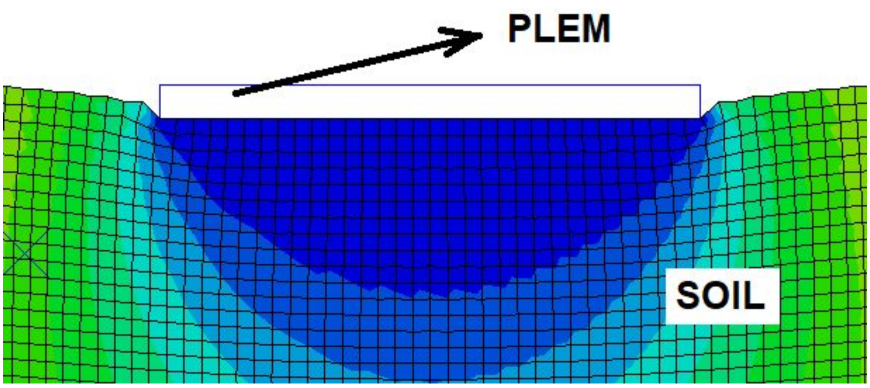
Figure 5 - Short Term Settlement (m).



Source: Extracted from (ABAQUS, 2011).

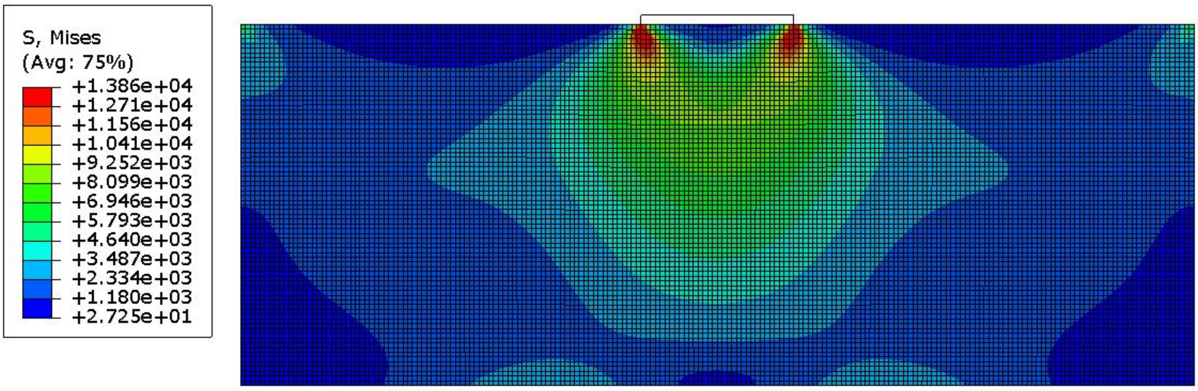
The Figure 5 shows that the Short Term Settlement were more significant at the middle of the model, and below the PLEM, as expected, once the loads were applied on the PLEM. For the level of loads applied, the displacements on that region were about -0.04 m.

Figure 6 - Detail of PLEM-Soil Interaction (Scale 30 times actual size).



Source: Extracted from (ABAQUS, 2011).

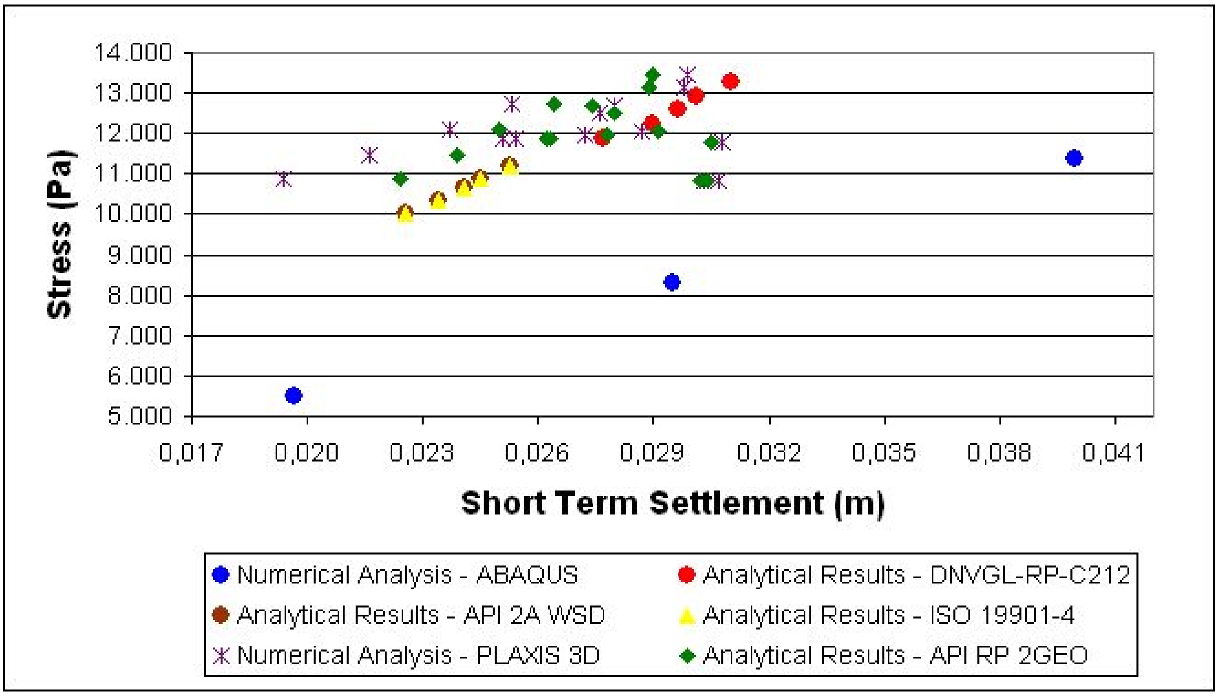
Figure 7 - von Mises Stress (Pa).



Source: Extracted from (ABAQUS, 2011).

It can be observed, from Figure 7 that the von Mises Stress presented the higher values at the extremities of the PLEM, as expected, because of the stress concentration at sharp corners, where the stress normally are more significant. Values around 13.86 kPa were registered. It is important to mention that the von Mises Stress shown on Figure 7 is different from the Stress presented on Figure 8, once the last Stress is the due to the applied loads on PLEM.

Figure 8 - Short Term Settlement.



Source: Produced by the author.

The Figure 8 shows the results in terms of Short Term Settlement for all standards regarded on the present work: 1) Analytical Model, 2) for the Numerical Model developed on (ABAQUS, 2011), and 3) for the results presented on the reference (DANDOULAKIS et al., 2017), on the graph, the sequence of values: a) Numerical Analysis - PLAXIS 3D and b) Analytical Results (DANDOULAKIS et al., 2017) apud (API RP 2GEO, 2011). It can be observed that for the value about 12 kPa (Stress), some design methodologies presented results very similar, around 0.0275 m (Vertical downward), one exception was the numerical analysis developed on (ABAQUS, 2011) that presented a value about 0.04 m (Short Term Settlement). The percentage difference was about 31.25 %, showing that the (ABAQUS, 2011) numerical model of the soil is less resistant than the other design methodologies presented. Despite of the differences, it is important to note that the responses were in the same magnitude order, showing that all design methodologies are in good agreement.

5. Final comments

Nowadays, there are many design methodologies applied to the subsea foundation design, and the engineers have to decide which standard and which software they will choose to develop their analytical models, and to carry out their numerical analysis. The present work shows results related to four standards (ISO 19901-4, 2016), (API-RP-2A WSD, 2000), (DNVGL-RP-C212, 2017) and (API RP 2GEO, 2011), and two softwares (ABAQUS, 2011) and PLAXIS 3D). The results with respect (API RP 2GEO, 2011) and PLAXIS 3D were extracted from (DANDOULAKIS et al., 2017). Analysing that results the engineers are able to decide which standard is more suitable to develop their foundation design. Specially, in relation to the project to be more or less conservative.

About the Analytical results presented on Section 4.1, it could be observed that the results were very close for all standards regarded, except for the comparison between

(API-RP-2A WSD, 2000) and (DNVGL-RP-C212, 2017) with respect to the Bearing Capacity (Table 4). The others parameters analyzed showed close responses, as discussed previously on that section.

In general, all results shown on Figure 8, about the Analytical models, presented results in the same magnitude order. Some of them more conservative than others, however, with little percentage difference (close to 19.13%), for example, comparing the standard (ISO 19901-4, 2016), and (DNVGL-RP-C212, 2017). Furthermore, with respect the numerical models, it was observed that, although, the model developed in (ABAQUS, 2011) to be simpler than the PLAXIS 3D (DANDOULAKIS et al., 2017), the percentage difference was about 23.00%. That percentage difference was determined regarding the Stress about 11.80 kPa, and the displacement about 0.031 m, for PLAXIS 3D. And the Stress about 11.39 kPa, and the displacement about 0.040 m, for (ABAQUS, 2011). Another important point to highlight is that the region of analysis is very critical, once little variations in terms of Stress induces big distortions about displacements. Observe, for example, that the region analyzed presents percentage difference about 3.60% for the Stress comparison. The (ABAQUS, 2011) model is simpler than PLAXIS 3D, because (ABAQUS, 2011) model is a 2D modelling, while PLAXIS 3D is a 3D modelling. In Addition, PLAXIS 3D were developed including more parameters related to the soil stiffness.

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