INSIGHTS INTO SETTLEMENT MECHANISMS OF SHALLOW FOUNDATIONS ON LIQUEFIABLE LAYERS

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Abstract: Buildings with shallow foundations form the bulk of the structures at risk due to liquefaction during a seismic event. Predictions of their potential settlement are often performed using methodologies that correspond to the free-field. However, these methods often prove insufficient, as they fail to capture the mechanisms that contribute to the settlement of a building. In an effort to offer insight regarding these mechanisms, centrifuge tests were performed, examining the seismic response of a shallow foundation on liquefiable soil. The main parameter investigated was the ratio of the width of the structure's foundation over the depth of the liquefiable layer on which it rests. Velocity vector fields in combination with excess pore pressure distributions were used to identify settlement-generating processes. Mechanisms that are not accounted for by current methodologies, such as the mobilisation of bearing capacity and soil-structure interaction induced displacements proved to be prominent.

Introduction

Earthquake-induced liquefaction is a potential cause of significant damage to the built environment. Buildings with shallow foundations are the most common and often the most vulnerable to this threat. They usually suffer significant settlement and tilting. Numerous examples of failures exist. Some of the most recent, in earthquakes in Turkey (Bray et al., 2004), Chile (Bertalot et al., 2013), New Zealand (Cubrinovski et al., 2011), and Japan (Yasuda et al., 2012), showcase that even countries with advanced seismic codes are not sufficiently protected. As a result, current design methodologies need to be placed under scrutiny.

The state-of-the-practice for estimating building settlement relies on procedures developed to predict post-liquefaction, one-dimensional settlement in the free-field. The most common are those by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992). Both methodologies relate the cyclic stress ratio, and the relative density (or the SPT or CPT resistance), to the volumetric strain expected for saturated sand deposits.

The main cause of concern is that these methods were produced for the evaluation of settlement in the free-field. To use them for the prediction of the settlement of a structure is, in fact, a fallacy. Inherent in this practice is the assumption that the free-field settlement of a saturated sand layer cannot differ substantially from that of a structure that rests on it. This notion has been shown to be inaccurate (Bertalot et al., 2013; Dashti et al., 2010; Liu and Dobry, 1997). Nevertheless, free-field methodologies still prevail due to both their simplicity and the lack of a more comprehensive analytical alternative.

Free-field methods only account for settlement induced by the processes of sedimentation and consolidation. However, the seismic response of a shallow foundation is the result of the interplay between several more mechanisms. Dashti et al. (2010) identify and list several additional contributing mechanisms. Localised partial drainage as well as expansion due to a decrease in effective stress can cause volumetric-induced deformations. Partial bearing failure due to strength loss and building ratcheting due to soil-structure interaction can cause deviatoric-induced settlement.

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Even though these additional mechanisms have been conceptually identified, their interplay is difficult to predict, as it depends on a spectrum of parameters having to do with both the liquefiable layer and the examined structure. Over the past decades, researchers have focused mostly on the width of the foundation and the depth of the liquefiable layer as salient parameters. Yoshimi and Tokimatsu (1977) introduced the normalisation of both the width of a structure and its expected settlement with the depth of the liquefiable layer in question, and presented a graph that correlates the two based on the settlement of buildings during the 1964 Niigata earthquake. Backed by further post-earthquake field measurements (Adachi et al., 1992) as well as centrifuge tests (Liu and Dobry, 1997), this graph was considered an improvement to the procedures aimed for the free-field settlement of sand layers, when predicting building settlement.

However, the validity of this graph for thinner liquefiable layers has been recently challenged. Dashti et al. (2010), performing centrifuge tests, produced data points that fell well outside of the expected band, as established by Liu and Dobry (1997), when the width of a structure was more than double the depth of the liquefiable layer on which it rested. The same conclusion was reached by Bertalot et al. (2013), using data points from the settlement of real structures during the 2010 Maule earthquake in Chile. Moreover, even for deeper layers, the mere range of potential settlement given by this graph may render its use impractical.

Consequently, an analytical methodology that can be safely used in practice still remains elusive. The lack of such a reliable procedure reflects the shortcomings in our understanding of the phenomenon. The aim of this paper is to offer some novel insights into this problem, using dynamic centrifuge modelling. Of particular interest is the way that the mechanisms which contribute to the settlement of a structure change depending on two parameters, the width of the structure's foundation and the depth of the liquefiable layer on which it rests.

Methods

Three centrifuge experiments will be discussed in this paper. The experiments were performed using the Turner beam centrifuge of the Schofield Centre, at the University of Cambridge.

Plane strain problems were considered, modelling a strip foundation resting on liquefiable layers of different thicknesses. The sand layers were placed over a rigid base. Sketches for the three tests (OA4, OA6, and OA8) that will be discussed are given in figure 1. Since the ratio of the width of the foundation over the depth of the liquefiable layer is considered a salient parameter in predicting settlement (Liu and Dobry, 1997), the models tested were designed to capture a reasonable range of this parameter. The width of the foundation used corresponds to 4.6 m in prototype scale. Some important parameters regarding the tests are presented in table 1.

Table 1. Parameters of centrifuge tests (in prototype scale).

Centrifuge test	Depth of liquefiable layer, D_L (m)	Bearing pressure, q (kPa)	Relative density, D_r (%)	Width of foundation over depth of layer, B/D_L
OA4	11.5	50	39	0.4
OA6	4.6	50	42	1.0
OA8	2.3	50	40	2.0

Hostun sand was chosen and the sand layers were prepared by air pluviation using an automatic sand pourer (Madabhushi et al., 2006). In all cases, a relative density of about 40% was achieved. The instrumentation used consisted of miniature piezoelectric accelerometers, MEMS accelerometers, pore pressure transducers, and linear variable displacement transducers (LVDTs).

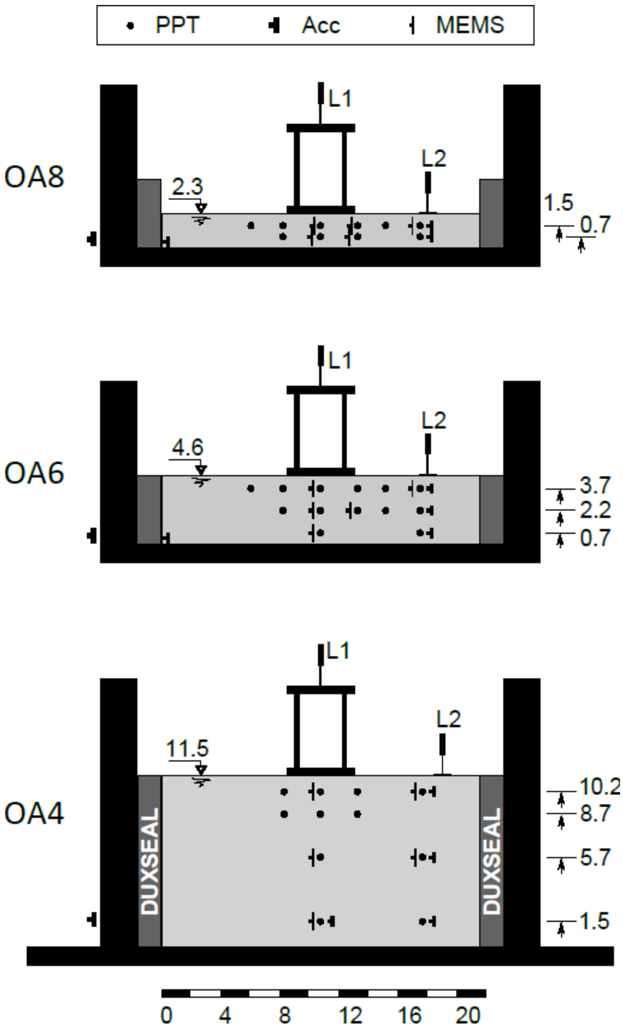

Figure 1. Models tested in centrifuge tests, presented in prototype scale. All dimensions are in meters. Pore Pressure Transducers (PPTs), Piezoelectric accelerometers (Acc), and Micro Electro – Mechanical Systems accelerometers (MEMS) were used in the soil. L1 and L2 are displacement transducers.

In order to overcome the incompatibility between the scaling of dynamic and seepage time (Madabhushi, 2015) a high viscosity aqueous solution of hydroxypropyl methylcellulose was used to saturate the sand layers. The centrifugal acceleration applied was 50g and hence the viscosity of the pore fluid was 50 cSt. The model container was a rigid box, one side of which contained a Perspex window. A high frame rate camera captured the seismic response through the Perspex. The produced images were used for PIV analysis (White et al., 2003). Layers of Duxseal at the edges of the layer were placed to limit the boundary effects (Steedman and Madhabhushi, 1991). The SAM actuator of the Schofield Centre (Madabhushi, 1998) was used to generate the earthquakes. The input motions applied are depicted in figure 2a.

Settlement evaluation methods

In an effort to elucidate the claim that a reliable analytical method for the prediction of liquefaction-induced building settlement remains unattainable, the settlement evaluation procedures described in the introduction are employed to predict the settlement of the structure in these three centrifuge experiments. The results can be seen in table 2.

Table 2. Predictions of settlement evaluation methods in comparison to experimental settlement.

Centrifuge test	Tokimatsu and Seed (1987) Settlement (cm)	Ishihara and Yoshimine (1992) Settlement (cm)	Liu and Dobry (1997) Settlement (cm)	Observed structure settlement (cm)
OA4	33	50	88 – 387	45.5
OA6	12	20	9 – 76	49.5
OA8	7	10	0 – 9	33.4

According to the method of Tokimatsu and Seed (1987), large volumetric strains of about 3% are predicted for all tests. In all cases, the method underestimates the actual settlement of the structure. As the depth of the liquefiable layer decreases, the mechanisms of sedimentation and consolidation are expected to contribute less to the settlement of the structure, leading to reduced reliability of this method. Indeed, the shallower the liquefiable layer, the further the estimations are from the actual settlement.

The method of Ishihara and Yoshimine (1992), leads to the prediction of even larger volumetric strains, of about 4.5%. Even though for the deeper layer the estimation is not far off the settlement of the structure, in the other two cases, predictions would not be reliable. As with the previous procedure, the building settlement cannot be captured because significant mechanisms that contribute to it are not accounted for.

The third method examined is the empirical estimation of settlement through the graph that relates the normalised width of a structure with its expected normalised settlement, introduced by Yoshimi and Tokimatsu (1977) and enhanced by Liu and Dobry (1997). As stated in the introduction, the range of predicted settlement can be large enough to render the prediction of limited use. Nevertheless, despite the very large range of the prediction, in the case of test OA4, the actual settlement falls outside of the predicted zone, being less than expected. For test OA6, the building settlement is within the range of the prediction, which is still quite broad. In the case of the thin liquefiable layer of test OA8, the real settlement falls well outside of the prediction band. Since this graph relies on settlements being proportional to the depth of the liquefiable layer, it is evident that volumetric mechanisms that affect the whole layer, like sedimentation and consolidation, are once again considered of increased importance. This assumption seems to not be accurate, especially for thinner liquefiable layers.

All in all, state-of-the-practice methodologies fail to offer reliable predictions for the settlement of structures. Free-field methodologies should not be discarded however, as they remain useful for the purpose for which they were created, predicting free-field settlement. Long term time histories of settlements in the three centrifuge tests examined are presented in figure 2b. The earthquake lasts for the first 20s. In the case of test OA4, the mechanism mobilised by the structure is large enough that it affects the field measurement of settlement. This measurement cannot be assumed to correspond to the free field. For the other two experiments though, the free-field settlement is well captured. The final values of field settlement, after reconsolidation has taken place, are 12.0 cm for OA6 and 9.2 cm for test OA8. These values are very close to the estimations made by the free – field methods. As a result, it is safe to deduct that when sedimentation and consolidation are the only mechanisms at play, these methods work accurately, even if, as can be seen in figure 2b, co-seismic settlement is significant, a fact that is opposed to their hypothesis of undrained seismic loading.

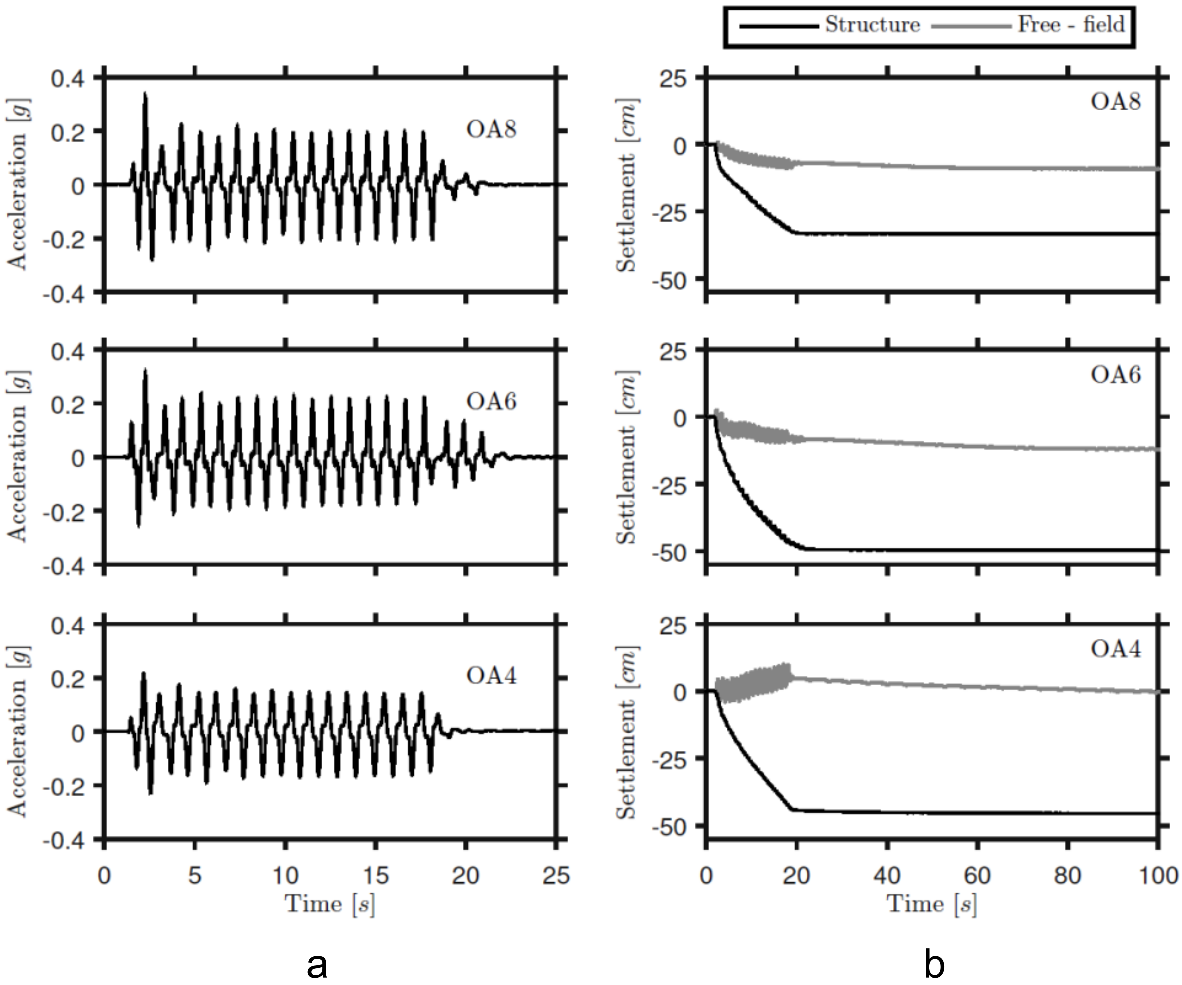

Figure 2. a) Input acceleration time histories. b) Long-term settlement time histories. “Structure” corresponds to LVDT L1 and “Free-Field” to LVDT L2.

Insights into contributing mechanisms

State-of-the-practice methods for the evaluation of the settlement of structures resting on liquefiable layers are wanting. This lack of accuracy reflects our shortcomings in understanding the mechanisms that contribute to settlement. In this section insights into these mechanisms are sought after.

Excess pore pressure contours, based on PPT measurements are presented in figure 4. A cycle halfway through the earthquake is chosen, and the distributions of excess pore pressures are shown at the point of maximum rotation of the foundation, for each test. Contours can only be drawn in areas that are surrounded by PPTs, as interpolation is necessary. Therefore, there is only enough information to partially reveal the excess pore pressure distribution within the soil layer of each test.

Figure 5 depicts velocity fields, computed from PIV-obtained displacements. The velocity fields depicted correspond to the same instant as the excess pore pressure distributions of figure 4. Velocity fields are shown as they can correspond to an upper-bound theorem mechanism. As it was impossible for the high frame rate camera to monitor the whole sand layer at once, only a section of the soil is represented in these velocity fields, as is evident by the axes of the graphs. Nevertheless, the mechanisms can be clearly seen. These mechanisms are also presented in terms of contours of the magnitude of horizontal velocity (figure 6a) and contours of vertical velocity (figure 6b).

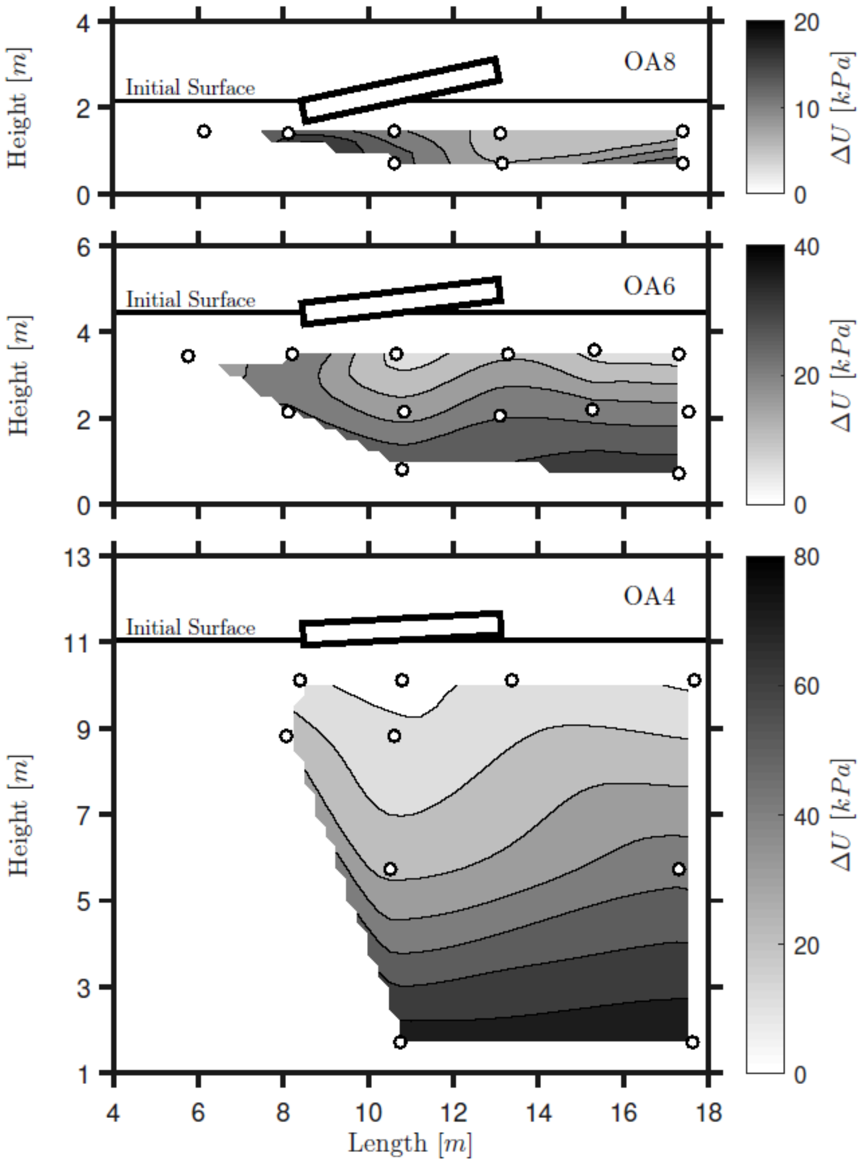

Figure 4. Excess pore pressure contours at the point of maximum foundation rotation, during a cycle halfway through the earthquake, for all three tests. The white circles depict the points where PPTs were placed. The rotation of the foundation, as presented here, has been scaled up by a factor of 10.

In the case of test OA4, where a deep liquefiable layer is modelled, the velocity field reveals a mechanism of significant extent. The soil area underneath the foundation, especially up to a depth equal to the width, B, of the foundation, is moving practically as a whole. This can be more clearly seen by examining the contours of figure 6. Due to the bearing pressure of the structure, the confining stress for the soil in this area is relatively high, compared to soil of the free – field at the same depth. Due to the increased confining stress, the developed cyclic stress ratio during the earthquake must be smaller than that of the free – field, at the same depth. This would lead to less excess pore pressure being generated. Indeed, in figure 4, it can be seen that excess pore pressures under the foundation remain lower than those of the free field. This fact, in combination with the increased total stresses due to the bearing pressure of the structure, would lead to high effective stresses in the area under the foundation, in comparison to the rest of the soil layer. As a result, within a fully liquefied layer, the mass of soil under the foundation that remains not fully liquefied, would be expected to move all together. Indeed, this mass of soil can be seen in figure 5 to push the liquefied sand to its right into a “passive” wedge that moves upwards, while forcing the soil to its left to follow its motion, by forming a downward-moving “active” wedge. These wedges are more evident in figure 6a, as areas of decreased horizontal velocity, and in figure 6b, as areas that have significant upward and downward movement, respectively. The foundation rotates slightly, following the soil beneath it. Soil-structure interaction effects are limited, though the rotating footing causes some local increase in pore pressures under its downward moving edge (figure 4). All in all, in this test, it seems that the dominant mechanism that should be examined is one of bearing capacity mechanism mobilisation, where the area of soil under the foundation and up to a depth equal to approximately the width of the foundation, moves as one. This mechanism is not captured by any of the methods described before.

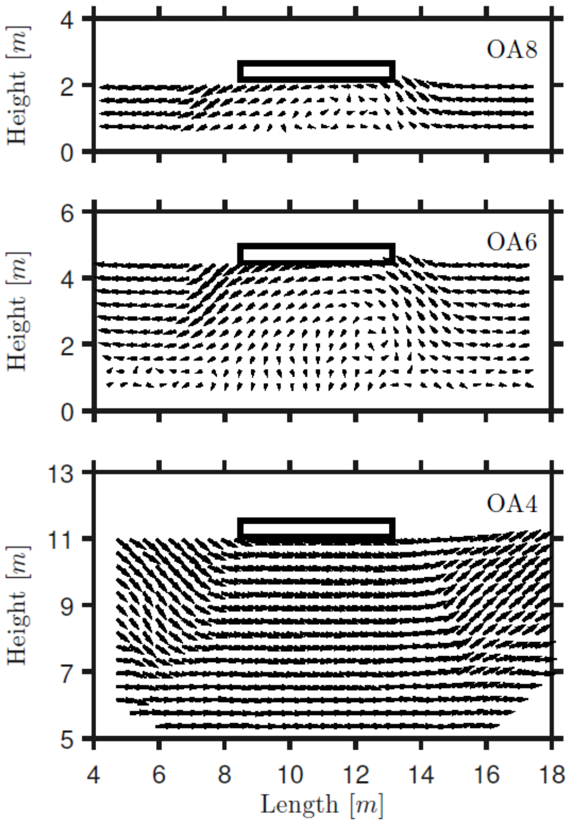

Figure 5. Velocity fields calculated from PIV displacements, at the point of maximum rotation of a cycle towards the middle of the earthquake, for all three tests.

As the depth of the liquefiable layer decreases, the mechanism changes dramatically. In the case of test OA6, where the depth of the sand layer is equal to the width of the foundation, the velocity field depicted in figure 5, at the moment of maximum foundation rotation, is different from that of test OA4. Here, the foundation itself is driving soil deformation, rather than following the mass of soil underneath it. Nevertheless, the analysis regarding the increased effective stresses under the foundation must also hold true in this case. Indeed, excess pore pressures in figure 4 are lower under the foundation, compared to those of the free-field. The difference though is not as pronounced as in test OA4. What seems to be happening is that the area of increased effective stress, which in the case of OA4 seemed to reach a depth equal to the width of the foundation, here can reach the base of the liquefiable layer. Consequently, a soil column of increased strength is formed between the rigid base and the structure. Resting on this column, the foundation can respond to the seismic motion by rocking, a behaviour that drives the displacement of the soil. Indeed, the horizontal velocity under the foundation is negligible compared to that further away (figure 6a). Furthermore, the rotation of the footing is more significant here than in test OA4. As the foundation rotates, it pushes the soil to its left (figure 5). Expectedly, pore pressures rise at his area, due to the compression imposed by the structure (figure 4). Localised drainage leading to volumetric strains must also occur. On the other hand, as the soil to the right of the foundation is unloaded, it moves upwards, since moving towards the area of increased effective stress under the foundation is more difficult. The vertical velocity contours of figure 6b depict in a clear way the localised nature of movement, at the edges of the foundation. In this test, it seems that soil-structure interaction has become the salient parameter, marking a significant change from the dominant mechanism of test OA4. Once again, this mechanism cannot be captured by the settlement evaluation methods used earlier.

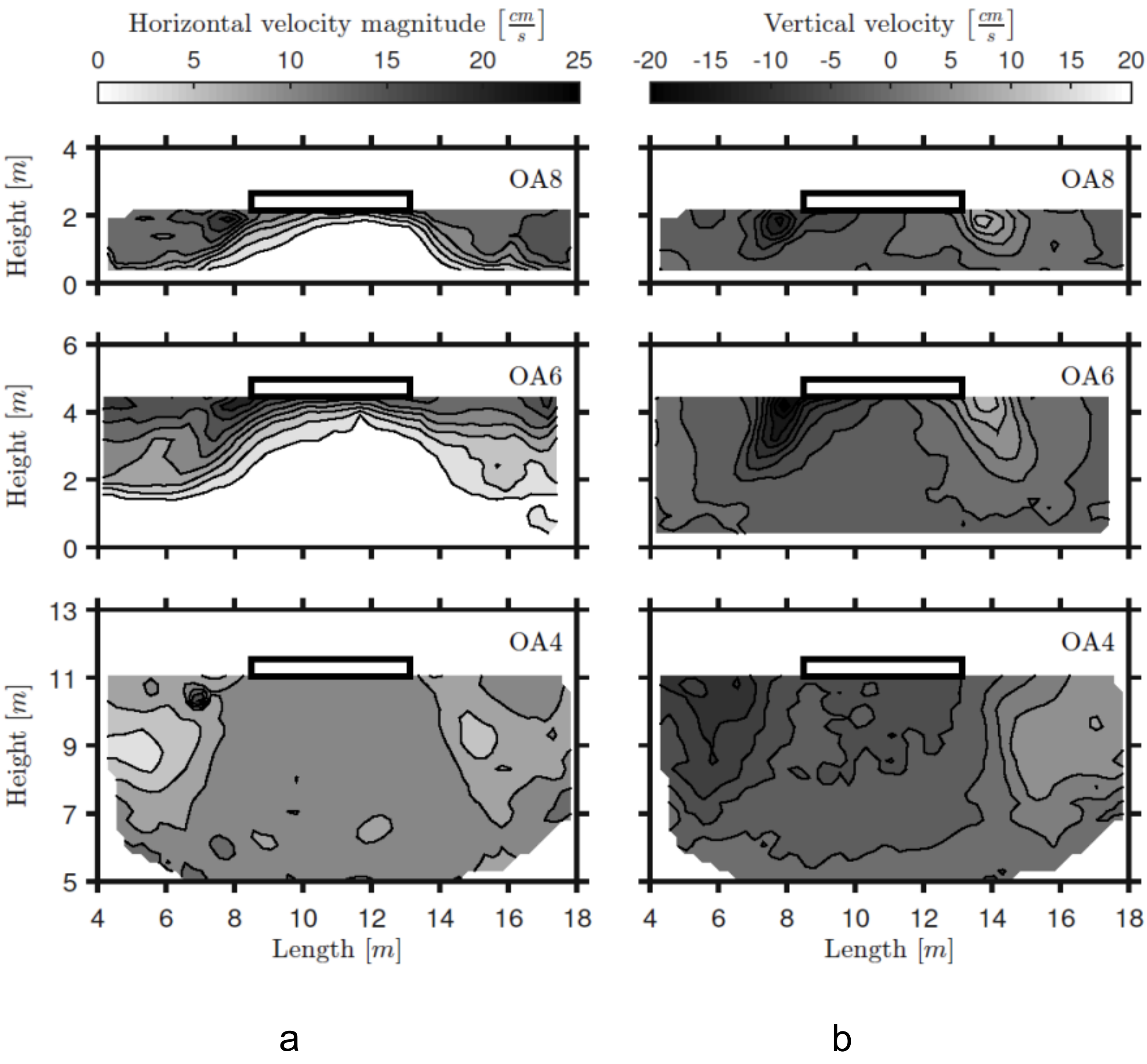

Figure 6. a) Horizontal velocity magnitude contours calculated from PIV displacements, at the point of maximum rotation of a cycle towards the middle of the earthquake, for all three tests. b) Vertical velocity contours at the same instant.

In test OA8, the depth of the liquefiable layer is half the width of the foundation. Here, the mechanism observed in test OA6 becomes even more prominent. The foundation rocks more severely, reaching larger angles of rotation, as can be observed in figure 4. The velocity contours are similar to those of test OA6, even though here, soil movements are even more localised. The increased rotation of the foundation in combination with the reduced depth of the layer leads to the pore pressure distribution being more thoroughly affected by the structural response. Under the downwards moving edge of the foundation, pore pressures increase significantly, due to compression of the soil, whereas under the upwards moving edge, they drop to almost zero. Soil-structure interaction induced ratcheting is the main cause of building settlement for this test.

Conclusions

Methodologies developed for the prediction of one dimensional, post-earthquake settlement of liquefiable sand layers in the free-field are often used in practice to estimate the settlement of structures. Such methodologies are shown to offer unreliable predictions for the settlement recorded in three centrifuge tests, modelling shallow strip foundations resting on liquefiable layers of different thicknesses.

Graphs of empirical correlation between the expected settlement of a foundation and its width, both normalised with the depth of the liquefiable layer, offer an alternative to the free-field settlement procedures. However, such a graph (Liu and Dobry, 1997) is shown to be unable to accurately predict the settlements recorded in the centrifuge tests. In the case of the deepest layer examined, it predicts a very broad and over-conservative range of settlement. In the case of the thinner layer, the settlement observed is significantly larger than the maximum of the range predicted.

Both using free-field methodologies to predict building settlement and resorting to empirical graphs that normalise settlement with the depth of the liquefiable layer inherently assume that volumetric mechanisms affecting the whole depth of the liquefiable layer are the most salient regarding settlement. In other words, according to these methods, sedimentation and consolidation are the primary causes of settlement.

Velocity vector fields and excess pore pressure distributions from the three centrifuge tests are used to elucidate the mechanisms that actually prevail. Three different ratios for the width of the shallow foundation over the depth of the liquefiable layer it rests on are examined. In all three cases the results are enlightening regarding the shortcomings of current methodologies. For the deeper layer, an extended, bearing capacity mechanism is mobilised, where the foundation and a soil mass underneath it respond in conjunction. As the sand layer reduces in depth, soil-structure interaction induced ratcheting and localised drainage are mainly responsible for the settlement of the structure. Under this light, it is not surprising that current methods fail to evaluate settlements accurately, since they do not account for the mechanisms observed in the experiments.

Further analysis is necessary in order to observe all the mechanisms that contribute to settlement and identify the parameters that affect their intricate interplay. This would be a significant step. However, the problem in question is not a conceptual one. Therefore, it is equally important to quantify how these mechanisms affect the settlement of a structure. The ultimate goal should be to move away from current methodologies and aim towards a new analytical procedure, able to offer reliable settlement predictions for practicing engineers.

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