Proceedings Paper:

Rontogianni, S and Fuentes, R orcid.org/0000-0001-8617-7381 (2016) Pile Performance Assessment under Induced Lateral Loading. In: ICONHIC 2016. 1st International Conference on Natural Hazards & Infrastructure (ICONHIC 2016), 28-30 Jun 2016, Chania, Crete, Greece. .

ICONHIC 2016
1st International Conference on Natural Hazards & Infrastructure
28-30 June, 2016, Chania, Greece

Pile Performance Assessment under Induced Lateral Loading

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ABSTRACT

This research work focuses on determining the internal force distribution in piles from displacement measurements when the piles no longer behave fully elastically. The method is based on the principle of virtual work (unit-load method) and allows the calculation of the bending moment distribution along piles, which is assumed to be the dominant internal force in bending. Four recent case studies were used, focusing either on liquefaction induced loading or on static induced plasticity. Comparison between the observed and calculated bending moments highlighted the method's flexibility to derive accurate results in various soil conditions, length-size of experiment and load conditions. The method is also equally applicable to piles and to retaining walls. The maximum error between observed and calculated maximum bending moment ranged from 30% to less than 5% whereas the location of the maximum bending moment along the length of the pile was successfully calculated in all tests which stands promising for prediction of plastic hinges. The method can be used as a reliable-rapid tool to estimate the pile state following earthquake loading. This, in turn, can be used as a resilient and vulnerability indicator of the pile ability to resist further loading and continue to perform its function safely.

Keywords: Lateral Loading, Liquefaction, Pile foundations, Plastic deformation

INTRODUCTION

Piles are the preferred type of foundation for supporting structures in seismic areas. However during past earthquakes, many important structures supported on pile foundations have been subjected to severe damage caused by large lateral displacements of liquefied ground termed as "lateral spreading". The liquefiable damage to pile foundations has clearly been demonstrated by major earthquakes during past years such as the 1964 Alaska, 1964 Niigata, 1989 Loma-Prieta, 1995 Hyogoken-Nambu, and more recently 2011 Tohoku (Zhang & Hutchinson 2012).

Field investigations and subsequent analyses after the earthquakes confirmed that kinematic effects arising from the ground movement as well as inertial effects from superstructure had significant impact on the damage to pile foundations. While the inertial force from the superstructure dominates in stiff and/or non-liquefied ground, the kinematic force also plays an important role in soft and/or liquefied grounds as well as in laterally spreading ground (Tokimatsu et al. 2005). With the commencement of liquefaction when pore water pressures build up, the soil loses its strength and stiffness, and the pile acts as an unsupported column over the liquefied depth. Piles that have high slenderness ratios will then be prone to buckling instability, which will also be amplified by imperfections, lateral forces and the dynamics of the earthquake (Lombardi & Bhattacharya 2014). Large deformations may also result in plastic hinging of the piles which currently is not incorporated in the geotechnical codes of practice (JRA, Eurocode 8). The magnitude, location and

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length of the plastic hinging has not been comprehensively investigated (Rajaparthy et al. 2008). Therefore, evaluation of pile response to lateral spreading and determining the soil-structure interaction induced by earthquakes, is an important step towards safe and resistant design (Liyanapathirana & Poulos 2005).

The mechanism of pile behaviour in liquefied soils and the design specifications have extensively been investigated (e.g. Dash et al. 2008; Prakash & Puri 2008; Dash et al. 2010; Maheshwari & Sarkar 2011). Advances in computing and computer power have intensified this effort. Many experimental studies including shaking tables, centrifuge and field tests have been conducted by different researchers for evaluation of the response of pile foundations to lateral spreading, yet some aspects have not been fully understood (Haeri et al. 2012; Chang & Hutchinson 2013).

The primary idea behind this study is to test a rapid and reliable tool to estimate extreme pile behaviour when piles are subjected to lateral loading conditions due to earthquake activity, liquefaction, or by proxy, other cases where plastic behaviour is present. It aims at determining the internal force distributions in piles using displacement measurements when the piles are no longer behaving fully elastically due to induced lateral spreading and is based on the principle of virtual work (unit-load method) with no need of defining boundary conditions. The bending moment is assumed to be the dominant internal force (as defined in the codes of practice), and the problem studied will be either plane strain or axis symmetrical, meaning that the torsion is also not relevant. The approach of determining internal forces from pile displacements measurements using the unit-load method was developed by Fuentes (2015) and will be further tested herein with an effort to expand the method to cover the non-elastic range of deformation. The method is equally applied not only to piles but also to retaining walls.

METHODOLOGY

Traditional monitoring techniques concentrate in evaluating total or relative deformation indirectly rather than enable direct conclusions about the governing internal forces of the structure itself. In this study the unit-load method (widely applied in structural engineering for determining displacement), based on the principle of virtual work is used in order to evaluate the internal forces with no need of defining boundary conditions when the conditions are not fully elastic. The validation against multiple case studies, in various ground conditions and experimental processes will test its flexibility and applicability. The method is implemented through Matlab R2014a (The MathWorks, USA).

The real structures are idealised in order for the method to apply, i.e. cantilever walls and laterally loaded piles as cantilever beams and propped walls as simply supported beams. The general assumptions are: i) linear elastic material behaviour applies; ii) cross sections that are plane before deformation remain plane; iii) only small deformations are applied to the structure. The focus for piles and retaining walls is determining bending moments therefore only the displacements perpendicular to the longitudinal axis need to be considered. The bending moment is assumed to be the dominant internal force in relation to the displacement based on the work done by Anagnostopoulos & Georgiadis (1993) and Abdoun (2003). In similar manner, the shear forces can be ignored since Gere & Timoshenko (1987) showed that their contribution to the lateral displacement is limited for thin cross-section structures.

The observed displacement can be divided into three main components (Fuentes, 2015): rigid body rotation, rigid body translation and bending. From these three, only the bending contributes to the bending moment in the structure and therefore its isolation is needed for the application of the method and should be the only component used.

The reader is referred to Fuentes (2015) for detail information related to the method's general concepts, inspiration, principles, simplifications, and main equations.

Choice of Polynomial Order

A common mistake while choosing the polynomial order is to use the higher order polynomials considering that they provide a better fit to the data. However, this may lead to over-fitting and instabilities that are important especially if the polynomials are used to derive other parameters from their derivatives such as shear forces or soil reactions.

For this study the Akaike Information Criterion (AIC) (Akaike, 1974) is used as given in Equation 1. The approach provides a formulation that agrees with the principle of parsimony by which the simplest model is selected, and reduces the risk of over-fitting.

$$AIC = -k * LN\left(\frac{SSE}{k}\right) + 2n \tag{1}$$

Where k is the number of observations-readings and n is the order of polynomials. SSE corresponds to the Sum of Square of Errors given by Equation 2, f_n is the polynomial under consideration and $\hat{f}$ an average of all polynomials used.

$$SSE = \sum_1^n \left(f_n - \hat{f} \right)^2 \tag{2}$$

The choice of this version rather than later updates as given by Hurvich & Tsai (1991) is made based on the fact that the updated approach increases the risk of under-fitting (Bozogan, 1987). The AIC approach was successfully tested by Fuentes (2015) in a number of case studies, concluding the optimal order corresponds to the lowest AIC score. Once the best order has been chosen, the final solution is taken as the average between the optimal polynomial order and the two closest orders with the lowest AIC value.

RESULTS

Four case studies were selected investigating piles in liquefaction (Abdoun et al. 2003; Ashford et al. 2006), retaining walls and piles passing into the plastic range (Bourne-Webb et al. 2011; Chang and Hutchinson 2011). Single pile (i.e. not group-pile) experiments were chosen for the unit-load method to be applied and tested. The soil conditions and experiment configuration varied for each case study.

The displacement data as found in the literature and any other information needed for comparing and contrasting the results were digitised with the use of the freely available software GetDataGraphDigitized 2.26, a program capable of digitizing graphs and plots.

Multiple polynomial orders were applied starting with 3rd order and moving up until 9th order. According to Fuentes (2015) typical starting values for polynomial order are 4th to 9th for laterally loaded piles. These are just initial estimates and iterations beyond that point were considered in order for the best order to be found. Using the AIC values it is possible to view the agreement between the polynomial orders and derive averages amongst them. It is also a way to have an initial idea of the 'best' order polynomial i.e. that with the lowest AIC score. The final solution is taken as the average of fits between the optimal polynomial order and the closest orders with the lowest AIC values. Other averages are also shown for comparison and when the AIC results did not lead to one definite solution.

Case study 1: Abdoun et al. (2003) & Dobry et al. (2003)

For this analysis the pile was treated as a cantilever wall. Figure 1 shows the AIC results for the different polynomial orders with polynomial 5 having the highest AIC value and polynomial 3 the lowest. This is an initial indication that the lower order polynomial combinations will most probably yield a more realistic representation of the pile curvature.

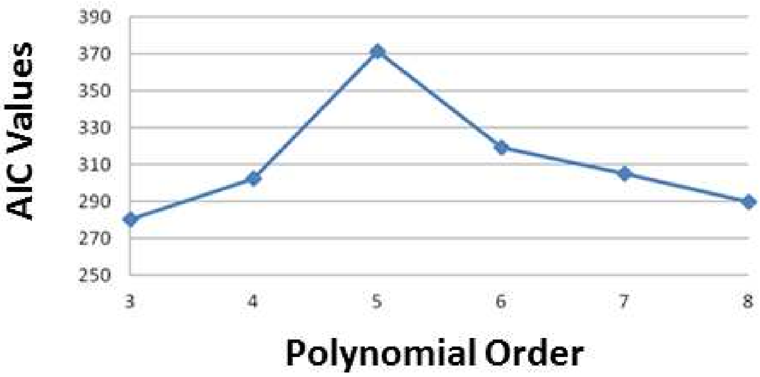

Figure 1. AIC results for the different polynomial orders. Polynomial 3 appears with the lowest AIC value.

The moment versus length plots for the different polynomial averages and the combinations that best described the experimental data are shown in Figure 2. These are shown as non-dimensional since the moment results are divided by the yield moment (M_p) as provided in the original studies. These results verify that the AIC estimation is correct since the average between 3 and 4 provide the best fit between the data from literature (i.e. Abdoun et al. (2003)) and the estimated data from implementing the unit-load approach. The EI value used for deriving the moment estimation is equal to 8,000 kNm^2 as described in Abdoun et al. (2003).

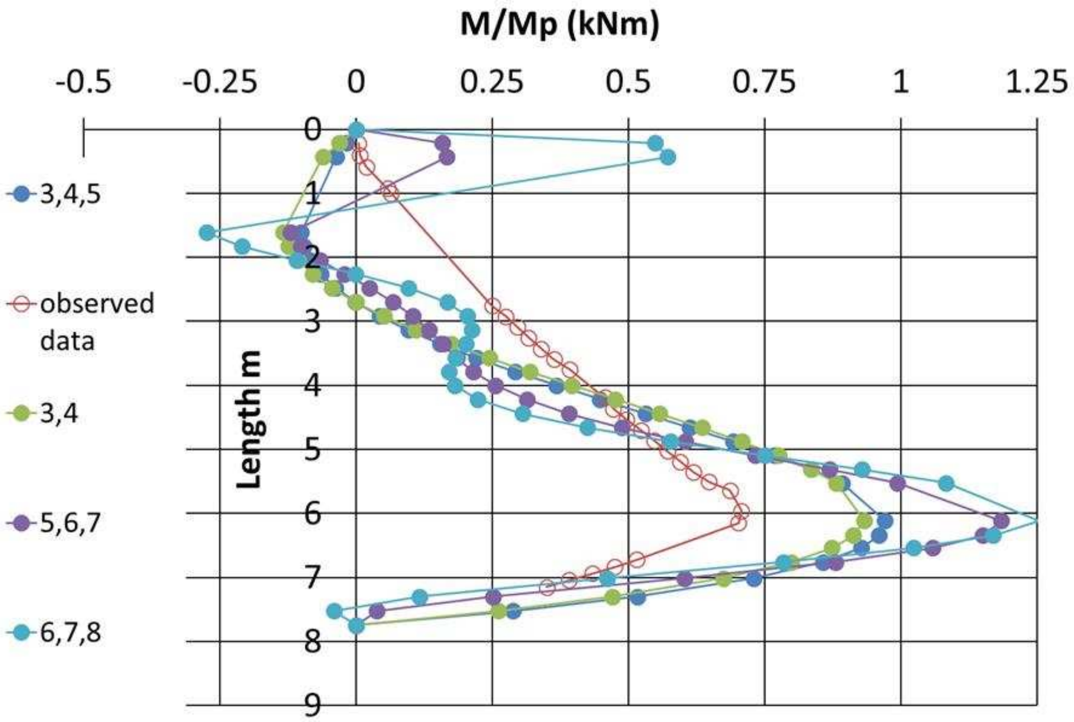

Figure 2. Moment versus length plots. Yield moment M_p = 162kNm as given in Dobry et al. (2003).

The analysis gave a reasonable agreement between the observed and calculated bending moment measurements. There is a ~25% difference between the maximum observed values from the literature and the calculated values as derived for the 3,4 average polynomial. The maximum moment is observed, for both observed and calculated data, at around 6m which is the base of the liquefied layer (reader is referred to Abdoun et al. (2003) for more information on the soil conditions). The source of the deviation between the results cannot simply be attributed to the method but issues can also be related to the limited number of instrumental readings.

Case study 2: Ashford et al. (2006)

For this experiment Ashford at al. (2006) used controlled blasting to create lateral spreading. For the unit-load method to be applied the pile was treated as a cantilever wall. Figure 3 presents the AIC results for the different polynomial orders with the 4th order having the lowest value. This gives an initial idea that the final polynomial combination can be taken as the average of fits between the 4th order and the closest orders with the lower AIC score i.e. 5 and 6.

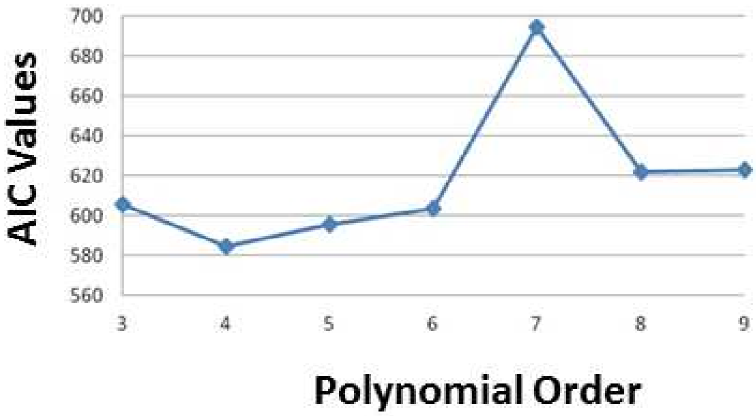

Figure 3. AIC values for different polynomial orders, polynomial 7 appears with the highest value and 4 with the lowest (optimum) value.

Figure 4 shows the moment versus length plots for different polynomial averages. The polynomial combination that best described the observed results is that of 4,5,6. This was also shown from the AIC results. The pile did not reach yielding therefore the EI value used was equal to 36,300 kNm^2 as given in Ashford et al. (2006).

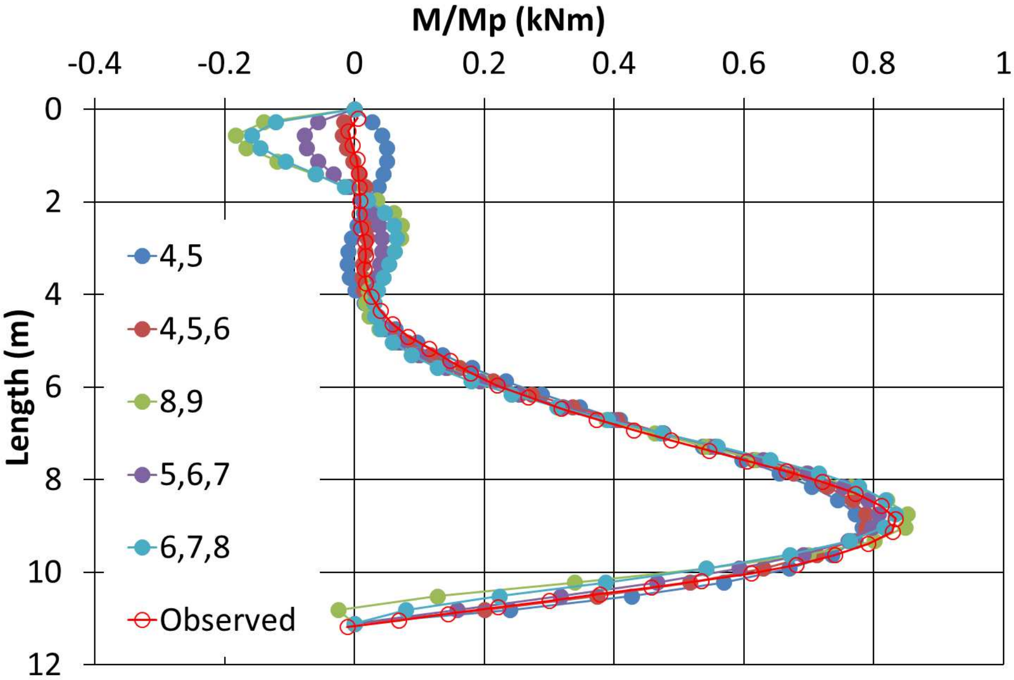

Figure 4. Moment versus length plots. Yield moment (M_p) taken as equal to 450kNm.

From the moment versus length plots it appears that there is a very good agreement between the observed and calculated measurements. The difference between the maximum moment values as observed from the literature and the calculated values as estimated herein is less than 5%. The maximum moment occurred for both the observed data and the unit-load solution at a depth of about 9 m.

Case study 3: Bourne-Webb et al. (2011), 3 Retaining Wall experiments.

The unit-load method was implemented in three of the Bourne-Webb et al. (2011) steel sheet pile wall experiments (SPWFG 14-16). Curvature versus pile length plots, AIC scores and moment versus pile length plots for different order polynomials were derived. The latter are further compared with the experimental results described in literature. For the experiments an anchor system was used therefore for the unit-load analysis the walls were treated as propped walls (the reader can refer to Bourne-Webb et al. (2011) for information in relation to the experiment configuration).

The Bourne-Webb et al. (2011) experiments took place in order to examine the effect of plastic hinging on model sheet pile walls as part of further developing and improving the Eurocode 3 requirements. The experimental results were not fully successful and according to the authors the notched wall did not reproduce the buckling effects as expected in the plastic bending response of steel sheet piles. The experiments (SPWFG 14-16) were consequent i.e. the wall was reused, leading in variations in anchor stiffness and modifications to the hinge zone. This meant that the moment-plastic curvature response was different between the experiments, because of hardening of the aluminium wall as this was bent and re-straightened.

Figure 5 gives the AIC scores for the different polynomial orders and Figure 6 the moment versus length plots for all three tests. From the AIC analysis there is an apparent difference between the high and low order polynomials for all three experiments. For SPWFG 15 the 3rd order polynomial has the smallest value, but increases rapidly when reaching the 5th order, no clear polynomial combination. For SPWFG 16 there is no clear combination even though the 3rd order has the smallest AIC value.

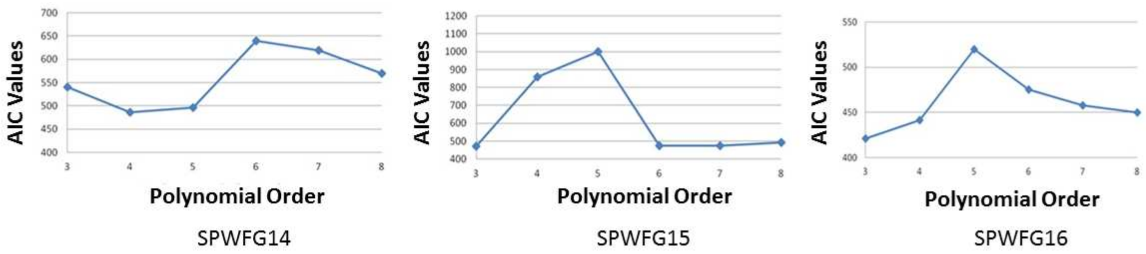

Figure 5. AIC scores for the three experiments there is a clear mismatch between the lower and the higher order polynomials.

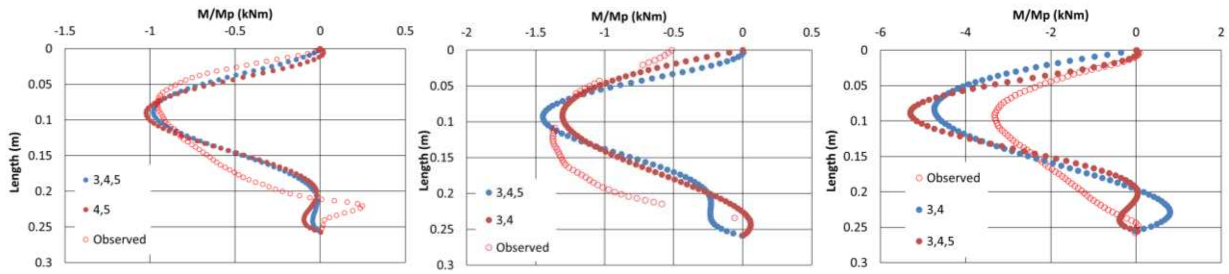

Figure 6. Moment versus length plots. From left to right: SPWFG14, SPWFG15 and SPWFG16. M_p equals 0.021kNm for SPWFG14, SPWFG15 and 0.01kNm for SPWFG16.

From the moment versus length plots for SPWFG14 the best fit is given for the 3,4,5 and 4,5 averages as also shown in the AIC scores. There is a very good agreement (less than 5% error) between the observed and calculated maximum bending moment around the hinge zone. The bending moments over the anchor is poorly predicted by the unit-load analysis. For SPWFG15 The maximum moment around the hinge zone is well described for the lower order polynomial averages (5% error) but not for the high order combinations. The bending moment around the anchor area is not well estimated. For SPWFG16 the bending moment around the hinge zone is better described by the low order averages. The agreement is not as good as in the other two examples (i.e. SPWFG 14, SPWFG 15) with the maximum bending moment for the 3,4 average polynomial being around ~30% higher than the maximum recorded value from the literature.

Case study 4: Chang & Hutchinson (2013)

In this experiment the RC pile was tested for its plasticity under cycling loading. The distribution of soil deformation at the instance of maximum positive and maximum negative accelerations of the crust was given. For the analysis the input displacement refers to the positive acceleration dataset. The pile was treated as a cantilever wall. Figure 7 shows the AIC scores for the different polynomial orders. The 3th and 5th order polynomials have the lowest score.

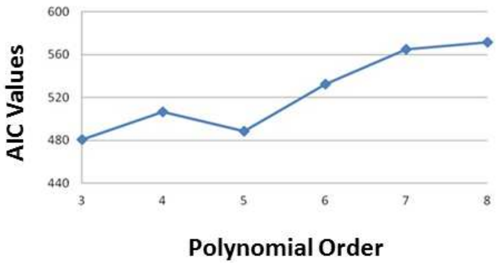

Figure 7. AIC results showing the 3rd and 5th order with the lower values.

The moment versus depth results were not provided in the Chang & Hutchinson (2013) work instead the curvature versus length results were given. In order to get an idea for the moment versus length distribution the theoretical moment versus curvature plot for the cross section was digitised and used. The plots and the equation applied for determining the values beyond yielding are given in Figure 8.

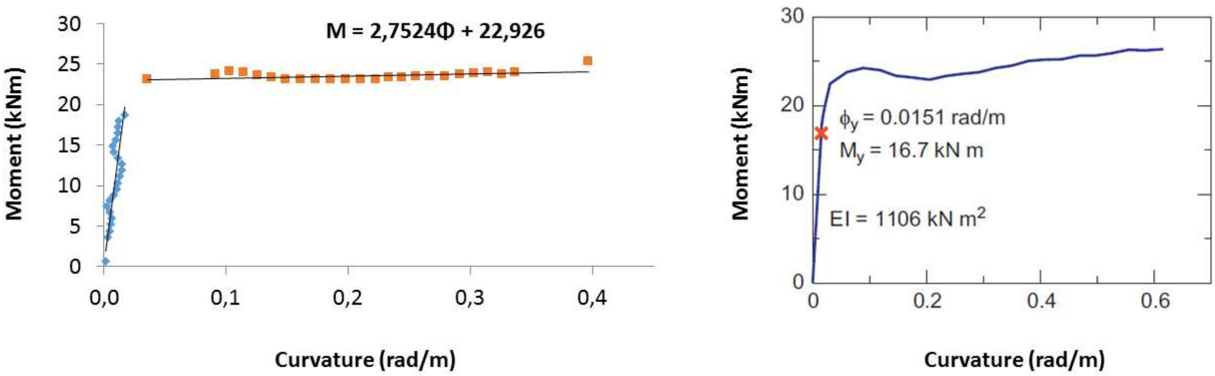

Figure 8. **Left**-Digitised Chang & Hutchinson (2013) Moment-Curvature graph. **Right**-Original plot from Chang & Hutchinson (2013).

Figure 9 shows the moment versus length distribution. The pile's plastic region seems to be ranging from 2.2D for the 3,4,5 average to 2.6D for the 4,5 average and 2.8D for the 3,4 average (where D is the piles diameter =25 cm). The literature-observed results showed an actual plastic region of ~1.5D.

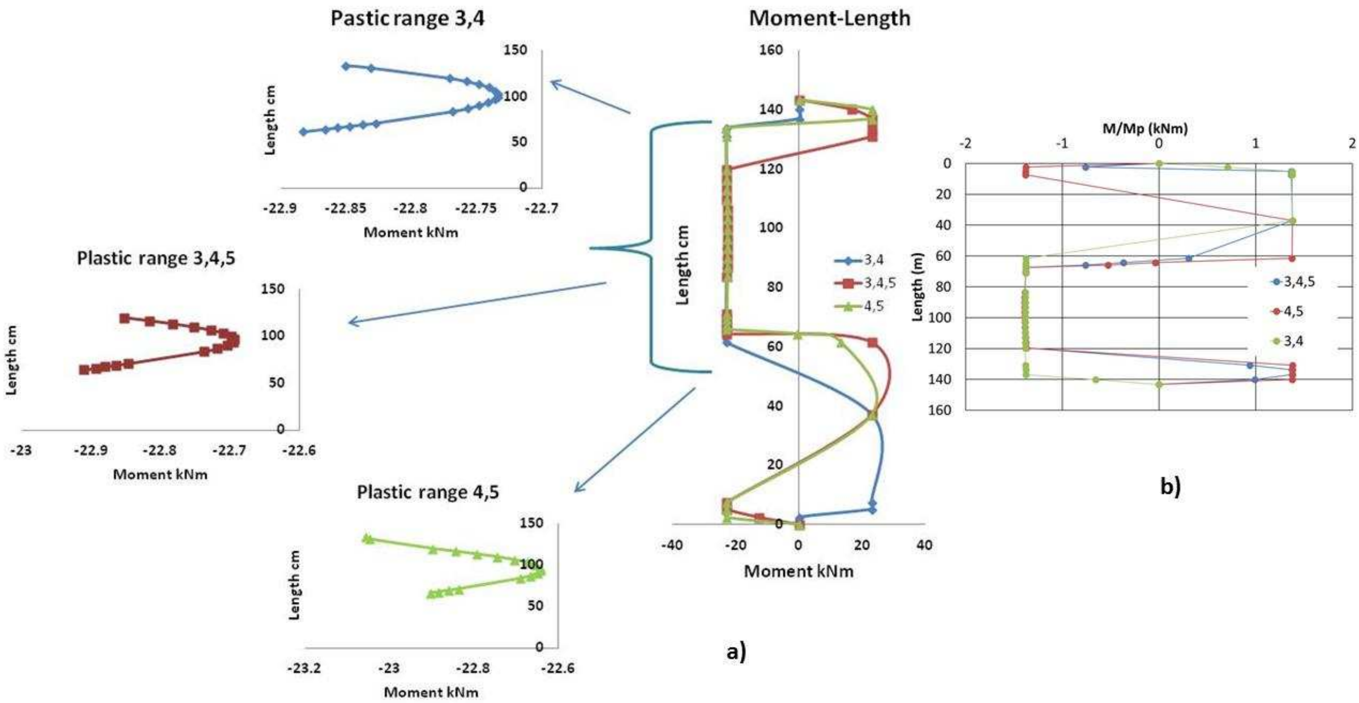

Figure 9. a) Moment versus length profile. The areas were the piles passed in plasticity have been enlarged for easiness in the description. **b)** M_p taken as equal to 16.7kNm.

This last experiment showed that there are limitations in applying the unit-load method when the pile is subjected to cycling loading that exceeds the yield curvature. For the unit-load method to be applied successfully the whole displacement of the pile needs to be inputted i.e. to capture the cumulative displacements caused by the yielding response of the structure. This is the only way to remove and characterise permanent deformations occurring in the structure for the application of the unit load method. Chang & Hutchinson (2013) provided only final displacement profiles. However, the pile reached yield point both during positive and during negative acceleration means that the cumulative yield displacement is lost from the final displacement profile. The results showed however a good agreement in evaluating the maximum curvature and its location along the pile. The small difference in the length of the plastic region i.e. 1.5D in literature and 2.2D herein can be attributed due to inability of the unit-load method to function accurately under cyclic loading where only final displacement profiles are provided. Hence displacement inputs covering the whole frequency range are needed. Table 1 gives a summary of main results from the unit-load method compared to the selected four case studies.

Table 1. Summary of main results.

Case Studies		**Optimal Polynomial Average**	**Error % Observed vs. Calculated maximum bending moment**
Abdoun et al. (2003)		3,4	25
Ashford et al. (2006)		4,5,6	5
Bourne-Webb et al. (2011)			
	SPWFG 14	4,5 & 3,4,5	5
	SPWFG 15	3,4	5
	SPWFG 16	3,4,5	30
Chang & Hutchinson (2013)		3,4,5	13

CONCLUSIONS

Pile monitoring under lateral induced deformation remains a challenging and complicated process. Therefore, there is a great need to use a method to provide an accurate and independent check on the internal forces in a structure when not fully behaving elastically. This will undeniably allow for more optimized design though greater understanding of the bending moments in piles. As a result, the main effort of this work was to use displacement measurements and apply a method suggested by Fuentes (2015), to determine internal force distribution on piles such as bending moments when the pile no longer behaves fully elastic due to induced lateral loading.

A selection of four recent case studies (from 2003-2013) was made for the method to be applied, modelling piles under extreme conditions: liquefied soils and plastic stage. The results highlighted the flexibility of the method to derive accurate results in various soil conditions, length-size of experiment, load conditions. It is also equally applicable to piles and to retaining walls.

The maximum error between observed and calculated maximum bending moment ranged from 30% to less than 5%. The location of the maximum bending moment along the length of the pile was successfully shown in all tests. Case histories and the unit-load analysis indicated that the interface between liquefied and non-liquefied soil layers play a critical role in the pile deformation. The errors observed in the calculated estimates could be improved if the following limitations were remedied: i) limited number of instrumental readings; ii) inaccuracies in the displacement measurement; iii) imperfections arising from digitisation; iv) fatigue of apparatus between consequent experiments (e.g. Bourne-Webb et al. 2011). The results were also able to determine the length and location of the plastic region. The small difference in the length of the plastic region between observed and calculated values can be attributed due to failure of the unit-load method to function accurately under cyclic loading information. The low order polynomial average fits produced better representation of the curvature and bending moment when compared with the observed plots. Therefore, a common mistake to choose higher order polynomial degrees as they provide an apparent better fit of data can lead to over-fitting and instabilities as shown herein.

FUTURE WORK

In general, from the analysis it is evident that the applicability and potential usage of the unit-load method is wide but some issues have to be addressed: i) The method assumes that the bending moment is the dominant internal force i.e. the axial load has limited impact on the lateral displacement of piles. Recent studies show that this is a simplification-assumption that might not fully stand for soils under liquefaction where piles start to lose their shaft resistance in the liquefied layer and shed axial loads downwards, as buckling failure will occur promoted by the action of lateral loads; ii) The EI values used for the application of the unit-load method were an estimate based on the model units used in each case study. From applying the unit-load method successfully beyond elastic range e.g. for the Chang & Hutchinson (2013) study, it is clear that the method is able to produce results beyond yield point if an accurate estimate of an EI value is provided. Efforts are currently taking place by various researchers towards that examining the effects of allowing for plasticity in the design at the Ultimate Limit State (Bourne-Webb et al. 2007); iii) The unit-load method as described herein models the piles as cantilever beams. An expansion to cover cases of beams fixed in both ends will make it applicable to numerous cases.

Concluding it is worth mentioning that the complexity of the problem arising both from the soil liquefaction/lateral spreading phenomenon itself and from the soil-pile-structure interaction makes it essential and urgent to continue and further expand the research into clarify mechanisms, quantify relations, calibrate analysis and improve design procedures. This will enhance our knowledge and understanding as well as the practice of seismic design and retrofitting of pile against liquefaction and reduce the risk of failure.

REFERENCES

Akaike, H., 1974. A new look at the statistical model identification. IEEE Transactions on Automatic Control, 19 (6): 716–723.

Abdoun, T. et al., 2003. Pile Response to Lateral Spreads: Centrifuge Modeling. Journal of Geotechnical and Geoenvironmental Engineering, 129(10): 869–878.

Anagnostopoulos, C. & Georgiadis, M., 1993. Interaction of Axial and Lateral Pile Responses. J. Geotechnical Engineering, 119 (4): 793–798.

Ashford, S. et al., 2006. Soil–pile response to blast-induced lateral spreading. I: field test. Journal of geotechnical and Geoenvironmental Engineering, 132(2): 152–162.

Bourne-Webb, P.J. et al., 2011. Analysis of model sheet pile walls with plastic hinges. *Géotechnique*, 61(6): 487–499.

Bozogan, H. (1987). Model selection and Akaike's information criterion (AIC): The general theory and its analytical extensions. Psychometrika, 52 (3): 345-370.

Chang, B.J. & Hutchinson, T.C., 2013. Experimental investigation of plastic demands in piles embedded in multi-layered liquefiable soils. Soil Dynamics and Earthquake Engineering, 49: 146–156.

Dash, S., Blakeborough, A. & Bhattacharya, S., 2008. Learning from collapse of piles in liquefiable soils. Proceedings of the ICE - Civil Engineering, 161: 54–60.

Dash, S., Bhattacharya, S. & Blakeborough, A., 2010. Bending-buckling interaction as a failure mechanism of piles in liquefiable soils. Soild Dynamics and EarthquakeEngineering, 30: 32–39.

Dobry, R., Abdoun, T., O'Rourke, T. D., and Goh, S. H. (2003). Single piles in lateral spreads: Field bending moment evaluation. J. Geotech. Geoenviron. Eng., 129(10), 879–889.

Fuentes, R., 2015. Internal forces of underground structures from observed displacements. Tunnelling and underground space technology, 49: 50–66.

Gere, J., M. & Timoshenko, S., P., 1987. Mechanics of materials, 2nd SI edition. Van Nostrand Reinhold (UK) Co. Ltd, UK.

Haeri, S.M. et al., 2012. Response of a group of piles to liquefaction-induced lateral spreading by large scale shake table testing. Soil Dynamics and Earthquake Engineering, 38: 25–45.

Hurvich, C. M. & Tsai, C-L., 1991. Bias of the corrected AIC criterion for underfitted regression and time series models, Biometrika, 78 (3): 499-509.

Liyanapathirana, D.S. & Poulos, H.G., 2005. Seismic Lateral Response of Piles in Liquefying Soil. Journal of Geotechnical and Geoenvironmental Engineering, 131(12): 1466–1479.

Lombardi, D & Bhattacharya, S (2014) Liquefaction of soil in the Emilia-Romagna region after the 2012 Northern Italy earthquake sequence. Natural Hazards 73(3): 1749-1770.

Maheswari, B & Sarkar, R (2011) Seismic Behavior of Soil-Pile-Structure Interaction in Liquefiable Soils: Parametric Study. International Journal of Geomechanics 11(4): 335-347.

MATLAB version R2014a. Natick, Massachusetts, USA: The MathWorks Inc., 2014.

Prakash, S. & Puri, V.K., 2008. Piles under earthquake loads. Geotechnical Earthquake Engineering and Soil Dynamics IV: 1–13.

Rajaparthy, S.R., Zhang, Z. & Hutchinson, T.C., and Lang, F., 2008. Plastic Hinge Formation in Pile Foundations due to Liquefaction-Induced Loads. Geotechnical Earthquake Engineering and Soil Dynamics IV.

Tokimatsu, K., Suzuki, H. & Sato, M., 2005. Effects of inertial and kinematic interaction on seismic behavior of pile with embedded foundation. Soil Dynamics and Earthquake Engineering, 25(7-10): 753–762.

Zhang, J. & Hutchinson, T.C., 2012. Inelastic pile behavior with and without liquefaction effects. Soil Dynamics and Earthquake Engineering, 36: 12–19.