ICPMG 2018

Physical Modelling in Geotechnics

Volume 1

Editors

Andrew McNamara
Sam Divall
Richard Goodey
Neil Taylor
Sarah Stallebrass
Jignasha Panchal

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City, University of London, UK

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Preface

The International Conference on Physical Modelling in Geotechnics is held under the auspices of Technical Committee 104 (*TC104: Physical Modelling in Geotechnics*) of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). Early workshops on physical modelling were held in Manchester, California and Tokyo in 1984 and, as the physical modelling community grew, the first international conference was held only 30 years ago in Paris in 1988. The possibilities offered by physical modelling became apparent around the world and the conference has developed into a quadrennial event that regularly attracts researchers from over 30 countries. The last meeting of the global community was in Perth, Western Australia; a veritable feast to sate the appetite of the hungry faithful, under the very capable leadership of Professor Christophe Gaudin. Regional conferences have also become established following the first Eurofuge held at City, University of London in 2008 followed by European regional conferences at TU Delft and IFSTTAR, Nantes and Asian regional conferences at IIT Bombay and Tongji University, Shanghai. These conferences bring together a community of great innovators; the most practical and capable engineers, in an exciting and specialist field.

TC104 selected London as the destination for the 9th International Conference (ICPMG 2018) which was held at City, University of London, in July 2018. The United Kingdom is a hotspot for physical modelling activity; centrifuges are established at Cambridge University, City, University of London, University of Dundee, University of Nottingham and University of Sheffield.

The conference coincided with the 4th Andrew Schofield Lecture, established by TC104 and named after Professor Andrew Schofield, the great pioneer of geotechnical centrifuge modelling. As the highest honour that can be bestowed upon a member of our community it is fitting that the lecture was delivered by Professor Neil Taylor of City, University of London and Secretary General of ISSMGE; a former doctoral student of Professor Schofield.

The conference programme was a physical modelling extravaganza divided into plenary and parallel sessions running over four days, 17th–20th July. Four keynote lectures were given in the areas of seismic behaviour, design optimisation, new facilities and environmental engineering representing significant areas of interest of the assembled audience. Themed lectures in the areas of education, new technology, urban infrastructure and offshore engineering addressed a key aim of TC104 in showcasing research opportunities to industry who attended a specific half day event. A total of 138 oral presentations were made from 230 papers submitted, originating from over 30 countries, and included in the conference proceedings in 22 chapters. All papers that were not presented orally were presented as posters. The conference gave delegates an opportunity to experience exciting and historic aspects of London that are normally inaccessible to those visiting the city. A welcome reception was held at the historic Skinners' Hall, home to one of the Great Twelve City livery companies and delegates enjoyed a sumptuous gala dinner at the spectacular Middle Temple Hall dating from 1573; one of the four Inns of Court exclusively entitled to call their members to the English Bar as barristers. A pleasant afternoon and evening was spent on a visit to Greenwich on the River Thames, home to the Meridian Line, the famous Cutty Sark, the Royal Observatory, the National Maritime Museum and the Old Royal Naval College.

Physical modelling has come of age and advances in all areas of technology, from digital imaging to computing, electronics and materials offer exciting opportunities to push boundaries well beyond the early experimental work. Visionary and adventurous physical modellers developed the basic techniques and important scaling laws that are the backbone of our work today. Such research made possible important contributions to the understanding of complex soil/structure interaction problems long before numerical modelling was capable of even attempting to establish such insight. Present day physical modellers are just as ambitious and adventurous as their forefathers and are anxious to build ever larger facilities and undertake increasingly complex experimental work. To this end, plans are underway for a 1000 g/tonne 'megafuge' capable of modelling the very largest of geotechnical structures. Physical modelling enjoys increasing popularity as a powerful means of exploring geotechnical problems. However, it rarely finds favour over numerical modelling in the eyes of industry where results of experimental studies are required soon after commissioning the work; regardless of accuracy and at minimal cost. Current work that focuses on exploring the interface between physical modelling and numerical modelling is a particularly exciting development and has the potential to yield important new knowledge applicable to both fields.

The organisation of a major international conference is a massive undertaking. My thanks go to the Local Organising Committee and the International Advisory Board and to everyone who participated in the very thorough review process. Particular thank are due to my colleagues, Sam Divall, Richard Goodey, Jignasha

Panchal, Sarah Stallebrass and Neil Taylor at City, University of London who rolled up their sleeves to help with all aspects of the conference; but notably in managing and editing the huge volume of poorly formatted papers. For anyone reading this far, please do not alter the template when writing your conference papers.

Andrew McNamara

Chair, Technical Committee 104 on Physical Modelling in Geotechnics, 2014 – 2018

International Society for Soil Mechanics and Geotechnical Engineering

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Modelling tunnel behaviour under seismic actions: An integrated approach

E. Bilotta
University of Napoli Federico II, Naples, Italy

ABSTRACT: This paper intends to describe the integration of physical and numerical modelling, focusing on tunnels under seismic actions. It shows how numerical calculations can be used in association with centrifuge testing to model different aspects of tunnel behaviour during earthquakes. The scope of the paper has been limited to a few aspects, mainly concerning the change of internal forces in the tunnel lining during shaking and the effect of soil liquefaction. The interaction between a tunnel and a building in a soil layer undergoing liquefaction has also been taken into account.

1 INTRODUCTION

The behaviour of tunnels under seismic action and their vulnerability to earthquakes is a topic that has received increasing attention in recent years. However, evidence of tunnel behaviour during natural events of ground shaking can be observed only after an earthquake occurs. The analysis of the problem based on post-earthquake reconnaissance only may give an incomplete picture of the problem.

The study of seismic vulnerability of tunnels is therefore a typical field where small scale physical modelling in a centrifuge finds a useful opportunity of application. In fact, artificial ground shaking can be produced in a centrifuge that simulates natural earthquakes in a ground layer surrounding a model tunnel. Hence, the complex interaction mechanism that arises between the tunnel structure and the surrounding soil during shaking can be reproduced in the model. Several studies have been based on centrifuge testing on reduced scale models of tunnels in sand (e.g. Cilingir & Madabhushi, 2011; Lanzano et al., 2012; Tsinidis et al. 2015, 2016a,b,c). They have provided experimental data on the changes of structural forces in a tunnel lining undergoing ground shaking. A few studies have also modelled in a centrifuge the effects on tunnels of earthquake-induced ground failure such as fault displacement (e.g. Baziar et al., 2014) or soil liquefaction (e.g. Chou et al., 2010; Chian et al., 2014).

On the other hand, numerical modelling has often served as a tool for analysing the problem or validating simplified analytical solutions (e.g. Kontoe et al., 2014). However, it is well acknowledged that when high quality centrifuge test data are available, they can also be used to validate the results of numerical modelling (Zeghal et al., 2014). For instance, for circular tunnels under seismic loading several numerical studies originated from a set of centrifuge tests specifically designed for that purpose and a comparison among experimental data and numerical results achieved using different constitutive models and numerical algorithms provided a deeper insight into the problem (Bilotta et al., 2014).

An integrated approach, including both physical and numerical modelling, also relying on an accurate soil characterization, appears therefore the most reliable tool to analyse boundary value problems involving dynamic conditions and complex soil behaviour. Such an approach, for instance, inspired validation exercises such as VELACS (Arulanandan & Scott, 1993) that has represented for many years a benchmark for the study of seismic-induced soil liquefaction. More recently the LEAP exercise has been launched that further implements the same idea (Kutter et al., 2017).

Large research projects such as the abovementioned concerning soil liquefaction require however a significant financial support. This can be provided from public funding agencies or private sponsors, probably focusing on broad and impacting research streams only. Less appealing problems, that receive lower attention, might be excluded from the benefit of such a combined approach. Repositories of the experimental data that are produced by different facilities for different purposes all over the world, play in this case a fundamental role.

This paper intends to describe the integration of physical and numerical modelling from the point of view of numerical modellers. Focusing on the dynamic behaviour of tunnels, and in particular on the internal forces in the tunnel lining, the use of the centrifuge results to calibrate a numerical model and extend the scope of application is shown in section 2 and section 3. In the former the results of centrifuge testing are boosted by including numerically the effect of a construction process for tunnelling and a more complex structural behaviour of the lining. In section 3 the back-analysis of the centrifuge test in dry sand is used in association with the results of cyclic simple shear testing in undrained conditions, for the calibration of a constitutive model suitable for modelling pore-water pressure build-up in undrained conditions. The effect on the lining of the excess pore-pressure arising during shaking is analysed. Finally, in section 4 the process is reversed from numerical analysis to centrifuge modelling. The calibration carried out in the previous sections is used to perform a preliminary analysis of tunnel-building interaction in liquefiable soil, in order to design a series of centrifuge tests.

2 INTERNAL FORCES IN A TUNNEL LINING

2.1 *Background*

Internal forces in the tunnel lining change during earthquakes. They can be calculated following several approaches (Hashash et al., 2001; Pitilakis & Tsinidis, 2014). Pseudo-static or uncoupled dynamic analyses are usually carried out in routine design. Full dynamic analysis, that is including dynamic soil-structure interaction, must be performed however, if the influence of the existing stress state around the tunnel has to be considered. Moreover, the latter may include the irreversible behaviour of soil that is likely to produce permanent ground deformation during shaking. Since the tunnel construction process may affect the static conditions before shaking, numerical analyses can include this aspect. Compared to plane strain, three-dimensional models permit the construction phases to be simulated in a more accurate fashion, including geometrical details of the lining that may affect its structural behaviour (for instance the segmental layout of precast lining). The effect of seismic waves propagating in any direction can be also analysed in a three-dimensional numerical model.

On the other hand, direct measurements of the effect of the complex interaction between a tunnel lining model and the surrounding soil during ground shaking can be achieved in centrifuge tests. This enables a large amount of experimental data to be collected and used for validation of numerical analyses.

As part of a research within the ReLUIS project funded by the Italian Civil Protection Department, a series of centrifuge tests were carried out at the Schofield Centre of the University of Cambridge on circular tunnel in dry sand, undergoing dynamic excitation (Lanzano et al., 2012). Internal forces (bending moments and hoop forces) in the tunnel lining were measured during shakings. Hence, experimental evidence was gained on a problem that had been previously explored via analytical solutions (mainly based on the elastic theory) and numerical modelling only.

Such tests, which are briefly recalled in the next section 2.2, were later used as an experimental benchmark for numerical modelling, aimed at extending the scope of the study. In fact, three-dimensional finite element analyses were performed that take into account the non-linear and irreversible soil behaviour. The tunnel excavation process, that is neglected in the centrifuge tests, was modelled to achieve a realistic state of stress effect before shaking. Moreover, the segmental layout of a precast tunnel lining was modelled, although with a few simplifying assumptions (Fabozzi, 2017).

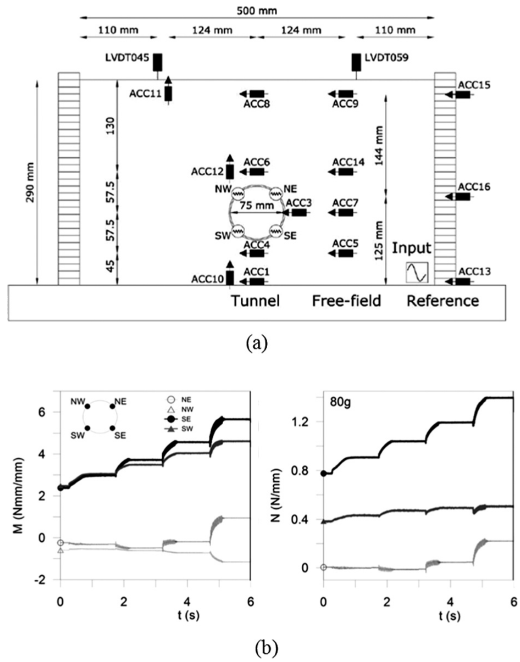


Figure 1. Model T3: (a) experimental layout; (b) measured time histories of bending moments and hoop forces (modified after Lanzano et al., 2012).

2.2 *Experimental benchmark*

The experimental benchmark used for validating the numerical model of a rather shallow tunnel (C/D = 2) in dense sand is the centrifuge model T3 (Figure 1), described in details by Lanzano et al. (2012).

In the model (Fig. 1a), an aluminium tube (diameter D = 75 mm, thickness t = 0.5 mm, cover C = 150 mm) representing a circular tunnel is embedded in a layer of dry Leighton Buzzard sand (fraction E), pluviated in the container at a relative density of 75% (Figure 1). The tube is instrumented with strain gauges in four positions along its transverse section (indicated as NE, NW, SW and SE in Fig.1a). After spin up at 80 g, a series of pseudo-harmonic signals of increasing amplitude and frequency was applied at the base of the model. The time histories of bending moment, M, and hoop forces, N measured during four subsequent dynamic events are shown in Fig. 1b at the model scale (Lanzano et al., 2012). It is worth noticing that permanent increments of internal forces arose in the tunnel lining after each events. These seem well correlated to the progressive densification of the sand layer that was observed in the experiments.

2.3 *Numerical modelling*

Numerical analyses were performed at prototype scale, using a scaling factor N=80. Hence, the corresponding

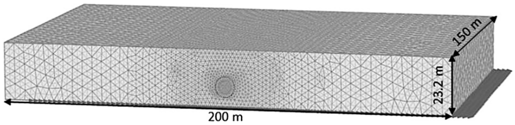


Figure 2. Numerical mesh.

Table 1. HS-small model parameters (Lanzano et al., 2016).

	sand
φ	38.6°
ψ	8.2°
c' (kPa)	0.01
$E_{ref}^{50}\,(MPa)$	18.6
$E_{ref}^{oed}\,(MPa)$	20.5
$E_{ref}^{ur}\,(MPa)$	62.2
$\gamma_{0.7}$	$0.60E^{-3}$
$G_0^{ref}\,(MPa)$	72.7
$p_{ref}\,(kPa)$	100
m	0.4

tunnel diameter is assumed 6 m, the tunnel axis depth is 15 m and the lining thickness is comparable to that of a concrete lining about 0.06 m thick.

The numerical model has been implemented in the finite element code Plaxis 3D (Brinkgreve et al., 2016). The mesh is shown in Figure 2.

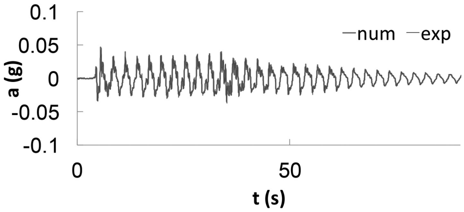
While the height of the model is 23.2 m, that is 80 times the relevant size at model scale, its width is larger than that and equal to 200 m, to minimise the influence of lateral boundaries. A reference section at the mid-span of the tunnel was assumed to be compared to the experimental results. Hence, in order to guarantee plane strain conditions in the reference section, the size of the model along the axis of the tunnel was assumed as long as 150 m. The vertical sides of the mesh were fixed in the horizontal direction in static condition; viscous dashpots were applied during shaking.

The time history of acceleration recorded by the accelerometer ACC13 at the base of the centrifuge model (see Figure 1) was scaled up to prototype scale and band-pass filtered (15–130 Hz) in order to reduce the its high-frequency content. This signal (with nominal frequency 0.375 Hz and nominal amplitude 0.05 g) was applied as dynamic input at the base of the model.

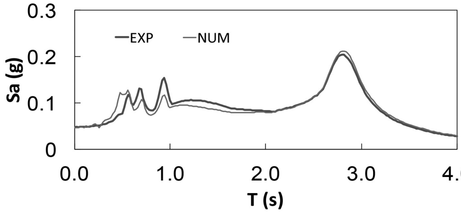
The lining is an elastic plate (EA = 2.8·106 kN/m; EI = 3.7·102 kNm2/m) with a very smooth interface (the interface factor was assumed as Rint = 0.05).

The sand has been modelled using the Hardening Soil with small strain overlay constitutive model (Schanz et al., 1999; Benz et al., 2009), with the parameters shown in Table 1, derived by Lanzano et al. (2016).

This elastic-plastic with isotropic hardening soil model is able to reproduce the decay of shear stiffness with strain level from very small strain and the increase of hysteretic damping. The initial damping ratio at very small strain was modelled through a Rayleigh formulation (αR = 0.0668; βR = 0.704 10-3).



(a)



(b)

Figure 3. ACC9, experimental and computed (a) time history of acceleration and (b) response spectra.

Figure 3a compares the time history of acceleration measured in the test by ACC9 with the corresponding computed results. In Figure 3b the corresponding response spectra at 5% of damping are shown. The dynamic response computed for the soil layer is close to the measurements, although there is evidence of a slight over-amplification of the signal at high frequencies, as observed also by Amorosi et al. (2014) in similar analyses.

Once validated against centrifuge results, the same 3D model was used to analyse the behaviour in the same sand of a different tunnel lining. This is a reinforced concrete lining with thickness t = 0.3 m (EA = 10.5E6 kN/m; EI = 78.75E3 kNm2/m; Rint = 0.7) and diameter D = 6 m.

A set of natural input signals was applied as time histories of acceleration at the base of the mesh. A few results of the study (Fabozzi, 2017) are presented in sections 2.4 and 2.5: the influence of the construction process on the seismic response of the tunnel is discussed in the former while the latter analyses the influence of the presence of joints in the segmental lining.

2.4 *Pre-seismic conditions induced by tunnel construction*

The influence of the construction process has been taken into account with reference to typical mechanized tunnelling with an earth pressure balance machine. Details of the procedure are described by Fabozzi & Bilotta (2016) and will not be discussed here. The seismic excitation was applied to the numerical model at the state of stress corresponding to the end of construction. Table 2 shows the main characteristics of the input signals applied as time history of acceleration at the base of the model. They are natural time histories of acceleration recorded on a rigid outcropping bedrock (soil type A according to EC8).

Table 2. Natural signals.

Earthquake event	Date	M_w	PGA
	-	-	*g*
Norcia	30/10/2016	6.5	0.78
Avej	22/06/2006	6.5	0.5
South Iceland (aftshck)	21/06/2000	6.4	0.36
Northridge	17/01/1994	6.7	0.68
Tirana	09/01/1988	5.9	0.33
Friuli	06/05/1976	6.5	0.35

Their mean response spectrum matches the Eurocode EC8–1 spectrum for ground type A (rock).

As an example, Figure 4a shows one of the time histories of acceleration applied at the base of the model. It is the record of the Norcia earthquake in Central Italy on 30/10/2016 (Mw = 6.5). In Figure 4b the corresponding normalized Fourier spectrum is shown.

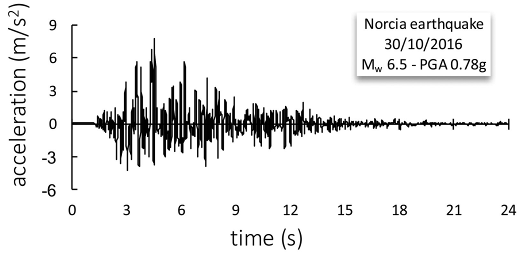
In all the analyses that have been carried out, permanent changes of internal forces in the lining at the end of shaking were calculated. In some cases, they reach values as high as 30% of the maximum transient change during shaking. As an example, in Figure 5 a pair of time histories calculated for the input signal of the Norcia Earthquake (see Figure 4) are shown. They are the time histories of the increment of bending moment (a) and hoop force (b) calculated at the point NE of the reference central section of the tunnel lining.

The experimental evidence obtained by Lanzano et al. (2012) and shown in Fig. 1b are therefore confirmed in part by numerical modelling on a different lining and for different characteristics of ground shaking: permanent changes of internal forces are calculated at the end of shaking, as observed in the experiments, although they do not exceed the transient changes calculated during the event. It should also be remarked that, in order to capture such an effect a suitable elastic-plastic constitutive model for soil must be adopted, as in this case.

Figure 6 shows the distributions of internal forces in the tunnel lining calculated under static conditions prior to (continuous lines) and at the end of shaking (dashed line). Such distributions of bending moment (Fig. 6a), hoop force (Fig. 6b) and longitudinal force (Fig. 6c) were calculated in the transverse reference section, both after simulation of the tunnel construction process (black lines) and for an ideal "wished-in-place" tunnel (grey lines).

As one would expect, the stress change due to the excavation produces lower bending moments (Fig. 6a) and normal forces (Fig. 6b) in the tunnel lining, than in a wished-in-place tunnel. Furthermore, the latter is almost not loaded in longitudinal direction (Fig. 6c).

It is worth noting that, although the maximum values of pre-shaking internal forces (continuous lines) are quite different, such differences reduce after shaking (dashed lines). This implicitly means that the calculated permanent changes of internal forces depend on



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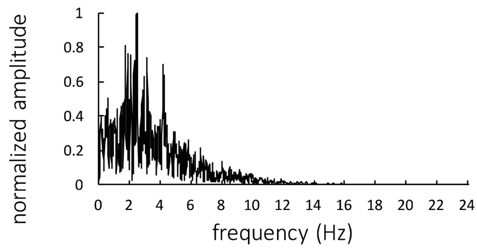
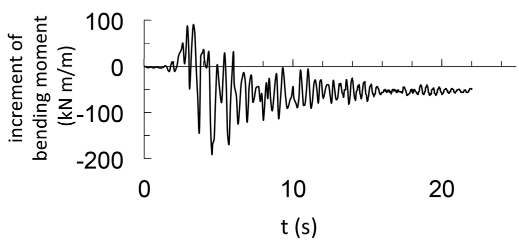


(b)

Figure 4. Norcia earthquake 2016 (M = 6.5): (a) time history; (b) Normalized Fourier spectrum.



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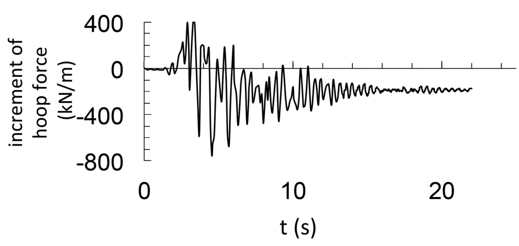
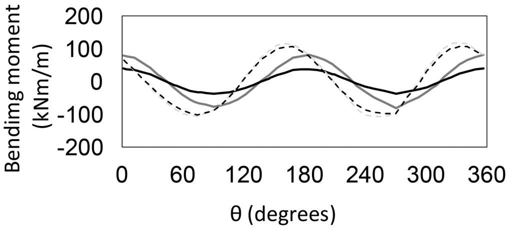


(b)

Figure 5. Time histories of the increment of internal forces in the point NE: (a) bending moment; (b) hoop force (Norcia earthquake).

the pre-seismic conditions: when the excavation process is modelled they are larger than in the case of the wished-in-place tunnel. The effect of the construction stages on the seismic behaviour of the tunnel lining is therefore evidenced by such numerical results.



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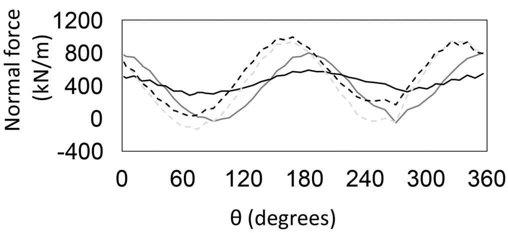

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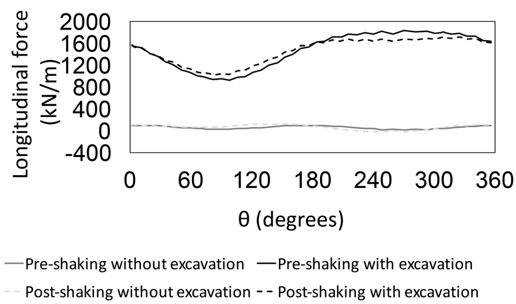


(c)

Figure 6. Distribution along the transverse reference section of (a) bending moment, (b) hoop force, (c) longitudinal force: static ‘pre-shaking’ (continuous lines) and ‘post-shaking’ (dashed lines), Norcia earthquake.

2.5 *Segmental layout of the tunnel lining*

Mechanised tunnelling in soft ground is usually associated with the use of a pre-cast concrete segmental lining to withstand external loads from interaction with the surrounding soil. Due to such a segmental layout the structural demand of the lining under static conditions is usually lower, because its flexural and axial stiffness is lower compared to a continuous lining of the same thickness.

The same numerical model that was described in section 2.4 was improved to introduce a segmental lining. The segments were modelled as elastic volumes of reinforced concrete with the same thickness as the continuous lining (EA = 10.5E6 kN/m; EI = 78.75E3 kNm2/m). Following Fabozzi (2017), the longitudinal joints between the segments were modelled as elastic-plastic elements (thickness = 0.30 m, width = 0.30 m): the values adopted for their mechanical parameters are shown in Table 3. Interface elements with the same behaviour were assumed to represent the transverse joints between rings. Figure 7 shows details of the structural model. The excavation stage was not modelled.

A lower structural requirement for the segmental lining compared to the continuous lining is evident also at the end of shaking. In Figure 8, the distributions of bending moment (Fig. 8a) and hoop force (Fig. 8b) in the transverse section at the end of shaking are shown.

Table 3. Model parameters for the lining (Fabozzi, 2017).

	γ (kN/m^3)	E (GPa)	ν	c (kPa)	φ
segments	25	35	0.15	-	-
joints	25	6	0.15	9000	42

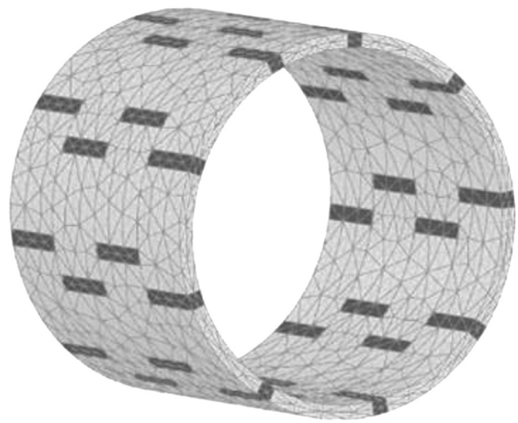


Figure 7. Detail of the numerical model of segmental lining

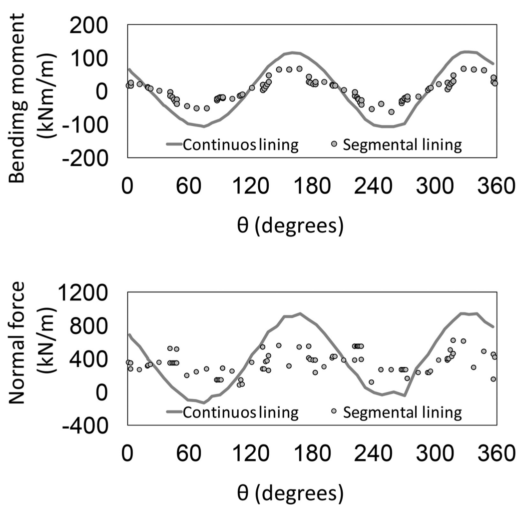


Figure 8. Distribution along the transverse reference section of (a) bending moment, (b) hoop force at the end of shaking: continuous *vs.* segmental lining (Norcia earthquake).

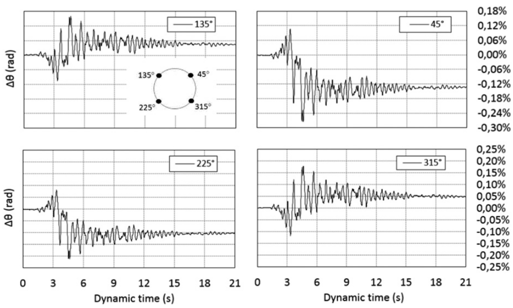


Figure 9. Time histories of relative rotation between segments during shaking (Norcia Earthquake).

The lower values of structural forces in the segments correspond to a larger deformability of the lining system at the joints, where relative displacements and rotation might be expected to occur.

Figure 9 shows the time histories of relative rotation between segments during shaking, calculated in the joints located at 45°, 135°, 225°, and 315° about the horizontal tunnel axis. At the end of shaking permanent relative rotations remain between segments. The magnitude of such permanent rotations is sometimes rather close to the peak values calculated during shaking. This result indicates a possible weakness of the segmental lining at the joints, where the rubber gaskets that guarantee water-tightness of real linings might be dislocated at the end of an earthquake.

The results in terms of relative rotations for the whole set of input signals shown in Table 2 are plotted in Figure 10. In Figure 10a a linear trend can be observed for the logarithm of the calculated peak relative rotation between segments versus the value of the peak ground acceleration of the input signal (PGA). It is worth noting (Figure 10b) that as far as the peak relative rotation increases (with increasing PGA), the permanent relative rotation increases faster. The ratio between the permanent and the peak rotation increases from as low as 10% until almost one half for the stronger earthquakes.

This further highlights the influence of the non-linear behaviour of the surrounding soil on the value of permanent rotations experienced by the segmental lining at the end of shaking.

2.6 *Remarks*

The numerical analyses presented in this section were calibrated on a single benchmark centrifuge test and then extended to model more complex cases in terms of the geometry of the lining. The numerical model also allowed a straightforward application of natural input signals and a consideration of the influence of construction process on the seismic demand of the tunnel lining.

It is worth noting that an earthquake can hit a tunnel several years after construction, hence different "pre-seismic" conditions can be considered. Moreover, in earthquake-prone regions, the same tunnel may be subjected to sequences of seismic events, with variable intensity and effects. Hence, the influence of the "initial state" should be considered in the assessment of tunnel vulnerability.

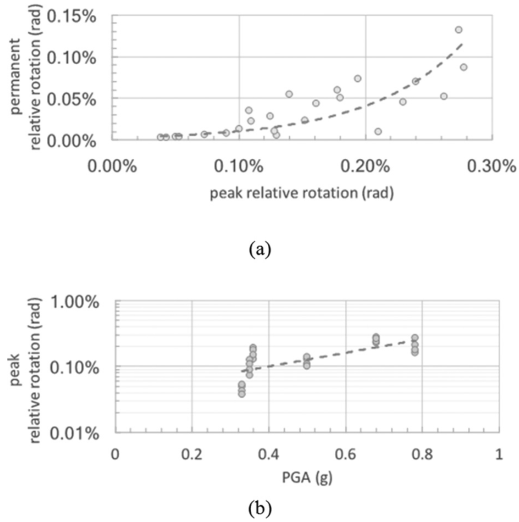


Figure 10. Peak relative rotation vs. peak ground acceleration (a) and permanent relative rotation vs. peak relative rotation (b), all input signals in Table 2.

Moreover, the numerical results from the segmental layout may create some concerns for tunnel linings in highly permeable soils, where an excessive rotation of joints may produce loss of water-tightness. This aspect may deserve attention in design and, at the same time, requires further experimental and numerical investigation.

3 TUNNELS IN LIQUEFIABLE SOIL

3.1 *Background*

Soil liquefaction may induce buoyancy of underground structures such as tanks, tunnels and pipelines. This is triggered when high excess water pressures develop, as those induced by strong motions. Several cases of uplift of underground tanks and pipelines have been observed in the past.

Although little evidence of liquefaction-induced damage to tunnels exists, physical modelling has shown that the high mobility of liquefied soil near surface would encourage floatation of very shallow or immersed tunnels. As a matter of fact, the uplift behaviour of underground structures caused by liquefaction has often been studied by physical models: for instance 1-g shaking table models of buried box structures, sewers and pipes and relevant possible mitigation measures (Koseki et al., 1997; Otsubo et al., 2014; Watanabe et al., 2016) or centrifuge models of tunnel of different shapes embedded in sand layers of different density, with several overburden and groundwater level (Yang et al. 2004; Chou et al, 2010; Chian and Madabushi, 2011; Chian & Madabhushi, 2012; Chian et al., 2014).

Experimental evidence has indicated that both the width of the underground structure and the depth of the liquefied layer have a large influence on the uplift displacement.

In general, physical modelling has been useful to collect important information and quantitative data on the phenomenon. In fact, although many numerical tools have been developed in the last decades to assess soil liquefaction, prediction of soil behaviour after liquefaction is still a challenging task. Hence, physical modelling has a further important role, that is to validate numerical models that can be used later for sensitivity analysis.

In this section, it is shown how starting from the back-analysis of the results of a centrifuge test on a model tunnel in dry sand undergoing shaking, the behaviour of the same tunnel in sand that has been saturated can be modelled numerically. The centrifuge model T4, described in detail by Lanzano et al. (2012), was used as an experimental benchmark. This centrifuge model has the same layout as model T3 in Figure 1, although the sand layer was looser (Dr = 40%).

The UBC3D-PLM constitutive model (Beaty & Byrne, 1998; Galavi et al., 2013) was used to represent the sand. It includes hardening plasticity and strain dependency of stiffness and damping. Hence it is able to capture the permanent deformation of the ground and changes in internal forces in the tunnel lining due to dynamic loading. Moreover, in undrained conditions it models the pore pressure build-up that may produce soil liquefaction and tunnel uplift.

The model is available in the 2D finite element code Plaxis (Brinkgreve et al., 2016) that has been used for the analyses. It was calibrated on the results of laboratory tests on the sand used in the centrifuge test along monotonic (Lanzano et al., 2016) and cyclic (Mele et al., 2018) stress paths.

3.2 *Numerical analyses*

A plane strain numerical model was defined in Plaxis 2D (Brinkgreve et al., 2016), at prototype scale. 'Tied degrees of freedom' between vertical sides were used as boundary conditions to simulate the laminar box behaviour during shaking. The nodes at the base of the finite element model were fixed in the vertical direction and a time history of acceleration was applied in the horizontal direction. The input signal applied at the base of the model is a pseudo-harmonic signal with nominal frequency 0.375 Hz and nominal amplitude 0.05 g at prototype scale. It was obtained after scaling up and filtering of the record of the base accelerometer in the centrifuge model ACC13 (see Figure 1).

3.3 *Model calibration*

The UBC3D-PLM is an elastoplastic constitutive model, which is a generalized formulation of the original UBCSAND model proposed for cyclic loading by Beaty & Byrne (1998). The model uses isotropic hardening and a simplified kinematic hardening rule for primary and secondary yield surfaces respectively, in order to take into account the effect of soil densification and transition to the liquefied state during undrained cyclic loading.

The constitutive model is capable of modelling cyclic liquefaction for different stress paths (Galavi et al., 2013).

Table 3 reports the input parameters used in the UBC3D-PLM model. The calibration of the model mechanical parameters was performed by Colamarino et al. (2017) using the results of laboratory tests on the sand used in the centrifuge test along monotonic (Lanzano et al., 2016) and cyclic (Mele et al., 2018) stress paths.

Table 3. UBC3D-PLM model parameters.

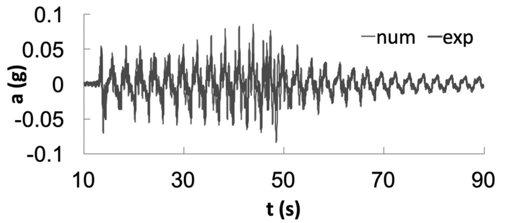
	sand
φ'_{cv}	32°
φ'_{p}	35.5°
$c'(kPa)$	0.01
K_B^e	300
K_G^e	360
K_G^p	180
m_e	0.5
n_e	0.5
n_p	0.4
R_f	0.93
$N_{1,60}$	7.36
fac_{hard}	1.6
fac_{post}	1.0

3.4 *Response of the model with dry sand*

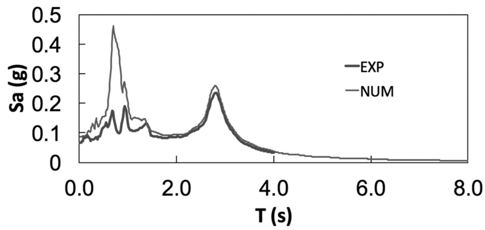
The numerical results are compared to the centrifuge test results in terms of time history of acceleration and the relevant response spectrum, 1.6 m below the ground surface (position of ACC9 in Figure 1) at prototype scale (Figure 11). For the sake of comparison, here and in the following figures, the experimental results of the centrifuge test are scaled up to prototype scale.

Other relevant comparisons between recorded data and simulation results are reported in terms of vertical displacements and bending moment in Figure 12a and b, respectively.

The main features of the experimental data are well-reproduced by the numerical predictions, both in terms of amplitude and frequency content, although an amplification larger than measured is calculated around 0.6 s, that is close to the natural period of the soil layer. Here, the agreement between the measured and the calculated amplitude achieved by using the UBC3D-PLM model is worse than by using HS-small model in similar conditions (see Figure 3). This might be in part due to the different amount of damping that the two constitutive models generate in stress-strain cycles.



(a)



(b)

Figure 11. Simulated *vs.* experimental acceleration at 1.6 m under the surface (ACC9): time history of acceleration (a) and response spectra (b).

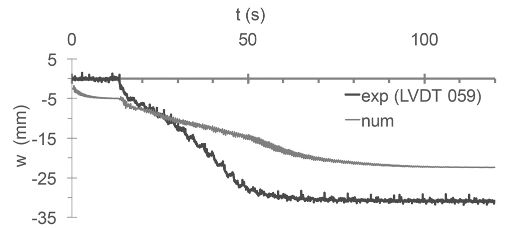
Figure 12a shows that the numerical model computes settlement at ground surface since the very beginning of the analysis, before 10 s, that is when the amplitude of the input signal is still negligible. In the same time a slight increase of bending moment is calculated (Fig. 12b). On one hand this confirms the influence of sand densification (hence plastic volumetric deformation) on the permanent change of internal forces in the lining; on the other it also shows that the numerical model tends to overpredict the plastic volumetric strain during shaking. As a consequence, the residual value of bending moment that is calculated at the end of shaking is even larger than the experimental value, although the corresponding transient cyclic changes are very similar (Fig. 12b).

3.5 *Response of numerical model in saturated sand*

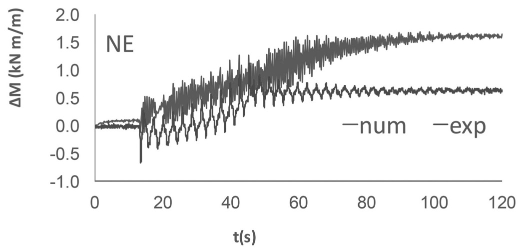
The same input motion was applied at the bottom of the mesh modelling the soil as completely saturated. In this condition, significant excess pore pressure developed in the soil layer above the tunnel, although full liquefaction was not triggered, due to the low amplitude of the input signal. The excess pore pressure ratio, r_u, defined as the ratio between the generated excess pore pressure and the initial effective vertical stress, did not exceed 0.77 (Figure 13).

Differences in the internal forces between dry and saturated conditions are shown in Figure 14.

This figure shows that the hoop force increased at both control points (NE and SE) in saturated sand compared to dry sand (Figure 14a, b), while bending moment increased in the upper part (NE) of the tunnel cross section and decreased in the lower part (SE).



(a)



(b)

Figure 12. Simulated *vs.* experimental time histories of (a) settlement at the surface (LVDT 059) and (b) bending moment in the tunnel lining at position NE

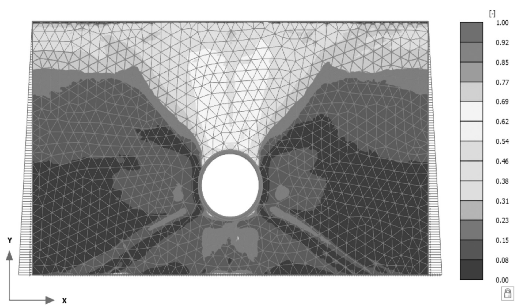


Figure 13. Excess pore pressure ratio at the end of shaking.

This indicates that the pore pressure build-up, associated with changes in effective stresses, affects the distribution of internal forces in the tunnel lining.

In general, a larger change of hoop force is induced in the lining during ground shaking if soil liquefaction approaches. The effect on bending moment depends on the position along the lining. However, for such a lining (the very flexible one used in the experiment) the values of bending moments are very low.

In order to evaluate the preliminary remarks that emerge on the basis of the comparison in Figure 14, the tunnel lining was changed, as in section 2.3, to a thicker reinforced concrete lining (EA = 10.5E6 kN/m; EI = 78.75E3 kNm^2/m).

Moreover, the numerical analyses were performed by applying at the base of the mesh the same signal ‘EQ1’ recorded in the centrifuge, two more signals obtained simply by scaling up ‘EQ1’ to twice (‘2x’)

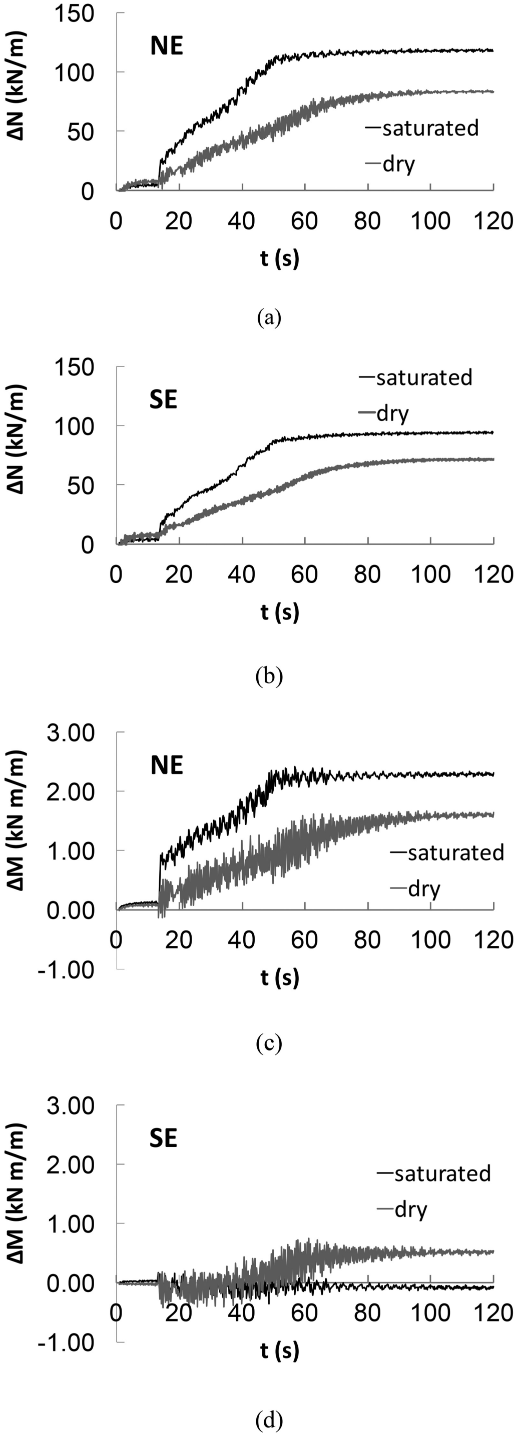


Figure 14. (a, b) Hoop force and (c, d) bending moment time histories along the tunnel on dry versus saturated soil conditions at (a, c) NE and (b, d) SE point.

and three times ('3x') its amplitude, and additionally the six natural signals shown in Table 2.

An overview of the analyses is given in Table 4.

The peak acceleration of the input signal ($a_{max,b}$), peak acceleration calculated at the ground surface ($a_{max,s}$), the maximum change of bending moment (ΔM) and hoop force (ΔN) in the tunnel lining at the end of shaking and the maximum uplift of the tunnel ($u_{v,max}$) are shown in the table. The last column of Table 4 also reports the average thickness of a continuous layer of soil (if any) where a value of the excess pore pressure ratio $r_u > 0.8$ was calculated.

In Figure 15 the time history of acceleration at the base of the model (grey line) is compared with that calculated at the surface (black line) for two cases from Table 4: 'Norcia' and 'Northridge' input. The achievement of liquefaction in the soil layer can be noticed in both cases.

Initially the signal is amplified (up to about 0.2 g, that is at about 2.5 s for 'Norcia' and 4 s for 'Northridge') at the surface compared to the base, then liquefaction occurs and the liquefied soil acts as an isolating layer: the amplitude of acceleration at the surface is lower than at the base from this point onwards. Figure 16 shows the distribution of the excess pore pressure ratio ru at the end of shaking in both cases. The shading has been limited to the range $0.8 \leq r_u \leq 1$. It can be observed that a continuous horizontal layer of soil near to the surface is very close to liquefaction if not liquefied. Moreover, the tunnel itself is partially interacting with liquefied soil, although deeper than the shallow liquefied horizontal layer (C/D=2).

In Figure 17 the ratio between peak acceleration at the surface and that at the base is plotted against the peak acceleration at the base, for all the input signals shown in Table 4. It can be noticed that in the cases where the peak acceleration of the input signal is lower than 0.2 g, such a ratio is higher than 1, indicating amplification, while for higher values of peak acceleration the ratio is lower than 1, indicating that de-amplification occurred.

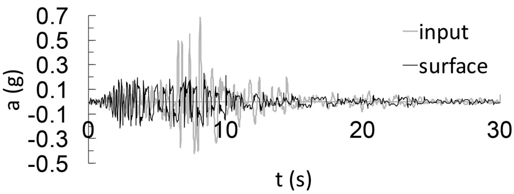
In all cases de-amplification is caused by liquefaction occurring near the ground surface. The depth of the tunnel in the ground layer does not affect the dynamic response of the soil, as shown in the figure.

The effect of soil liquefaction on the tunnel lining is analysed by looking at the maximum changes of hoop force and bending moment at the end of shaking (Table 4 and Fig. 18a, b).

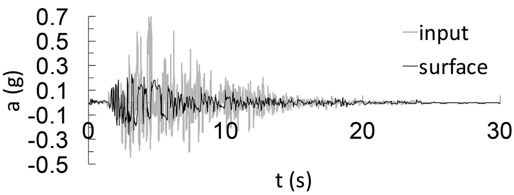
Figure 18 indicates that when the ground amplification prevails (for this ground conditions when amax,b < 0.2 g according to Figure 17) larger changes of internal forces arise for increasing amplitude of shaking. This trend is more evident for deeper tunnels (C/D=2). On the other hand, when liquefaction prevails (when $a_{max,b} > 0.2$ g), the change of internal forces is independent from the amplitude of the base acceleration.

Table 4. Overview of the analyses.

Input	$a_{max,b}$ g	$a_{max,s}$ g	ΔM kNm/m	ΔN kN/m	$u_{v,max}$ m	thickness $r_u > 0.8$ m
EQ1	0.054	0.114	43	126	0.004	-
2x (EQ1)	0.108	0.13	131	235	0.033	4
3x (EQ1)	0.162	0.197	132	210	0.238	7.5
Tirana	0.33	0.214	127	232	0.019	3
Friuli	0.35	0.25	137	240	0.026	3
South Iceland	0.36	0.315	139	234	0.087	-
Avej	0.5	0.339	189	246	0.061	-
Northridge	0.68	0.233	154	254	0.337	8
Norcia	0.78	0.26	150	233	0.128	7

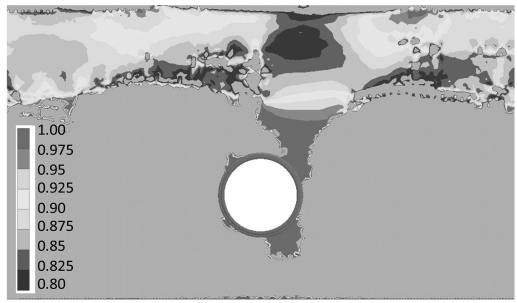


(a)

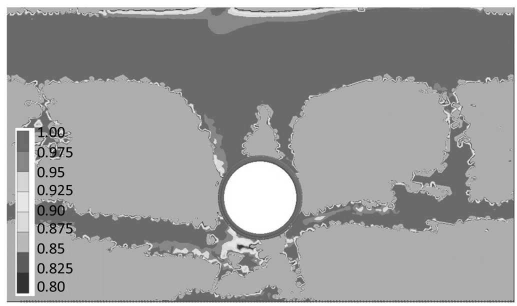


(b)

Figure 15. Time histories of acceleration at the base and at surface: input signal Norcia (a) and Northridge (b).



(a)



(b)

Figure 16. Shadings of $r_u > 0.8$: input signal Norcia (a) and Northridge (b).

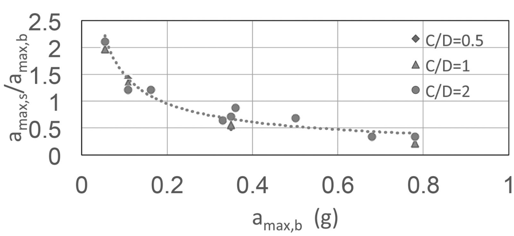
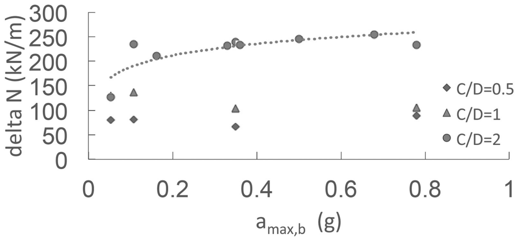


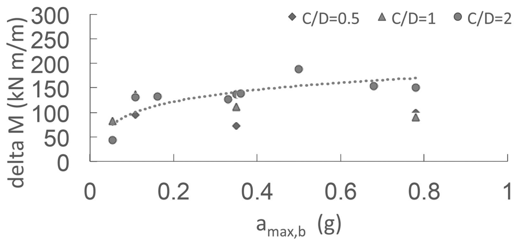
Figure 17. Ratio between peak acceleration at surface and at the base *vs.* peak acceleration at the base (all signals).

In terms of permanent displacements induced by soil liquefaction, it is worth noting that the calculations were performed by imposing undrained conditions. Pore-pressure build-up during shaking produces a very limited uplift of the tunnel, unless the soil liquefies and the liquefied ground interact with the tunnel, such as in the cases of Figure 16.

In Figure 19 the calculated uplift of the tunnel at the end of shaking is plotted as a function of the average thickness of a liquefied layer. This has been assumed to be a shallow continuous horizontal layer with $r_u > 0.8$ (see for instance the shaded areas in Figure 16). The trend in the figure shows that large amounts of liquefaction in the cover soil layer of the tunnel produces significant uplift of a shallow tunnel, although the cover upon diameter ratio is not too low (C/D=2). Similar trends were obtained for shallower tunnels (C/D = 0.5 and 1) using the pseudo-harmonic input signal only (that is 'EQ1', '2x', '3x' in Table 4) and are shown in Figure 20. Although the numerical results are limited, the effect of lower overburden can be read. The shallower the tunnel the larger the uplift associated with the mobility of the surrounding liquefied soil, as observed experimentally by Chian and Madabhushi (2011) in the centrifuge.



(a)



(b)

Figure 18. Maximum change of hoop force (a) and bending moment (b) in the lining at the end of shaking (all signals).

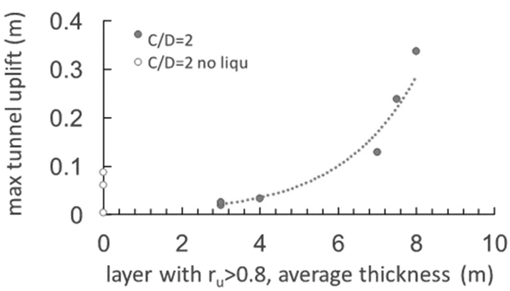


Figure 19. Maximum vertical displacement of the tunnel at the end of shaking (all signals, C/D=2).

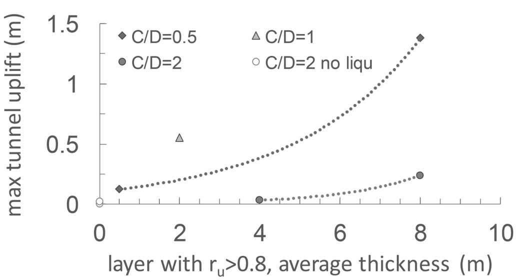


Figure 20. Maximum vertical displacement of the tunnel at the end of shaking ('EQ1', '2x', '3x')

3.6 *Remarks*

This section has shown how an advanced effective stress constitutive model, able to capture the cyclic behaviour of sand in both drained and undrained conditions, has been adapted to back-analyse a centrifuge test in dry sand in order to model afterwards a similar problem in saturated sand. The constitutive model has been calibrated using the results of laboratory tests carried out in monotonic and cyclic loading.

After comparing the numerical simulation in dry and saturated conditions, the numerical model has been used to extend the study to different conditions in terms of lining thickness, tunnel cover, input signal. This provided an insight into the behaviour of a shallow tunnel in a liquefiable sand layer. A form of limiting threshold to the change of internal forces induced by ground shaking in the tunnel lining has been observed in the numerical results once soil liquefaction occurs. At the same time, the influence of the overburden cover on the uplift induced at the tunnel in cases of extensive liquefaction has been discussed on the basis of the calculations.

Due to the lack of existing measurements for real cases, experimental campaigns using centrifuge modelling would be highly beneficial to corroborate or debate similar results.

4 TUNNEL-BUILDING INTERACTION IN LIQUEFIABLE SOIL

4.1 *Background*

Although uplift mechanisms for an underground structure experiencing soil liquefaction have been identified experimentally and numerically by several authors, the interaction of such mechanisms and the associated displacements of the underground structure with those induced in aboveground structures that may be founded nearby have not yet been investigated.

In urban areas shallow tunnels are likely to be close to the foundations of buildings and easily interact with them during earthquakes (i.e. Soil-Structure-Underground Structure-Interaction, SSUSI). Hence, the reciprocal influence of a tunnel and an adjacent building in the presence of soil liquefaction may be important.

Recent centrifuge testing on the behaviour of buildings founded in liquefiable ground layers has shown that smaller net excess pore pressures are generated within the liquefiable layer under a structure by increasing the contact pressure and height/width ratio of the building (Karimi & Dashti, 2016). Other studies have shown the reciprocal influence of adjacent buildings, affecting non-uniform settlement during liquefaction (Yasuda, 2014).

How the uplift mechanism of an adjacent underground facility is influenced by the presence of the building and how the floating of the underground structure can affect the tilt and settlement of the building are both aspects that deserve attention.

This problem appears rather important considering the rapid extension of the built environment, both above- and underground, to areas that may be subjected to risk of liquefaction. Hence an insight into such a problem may well contribute to increase the resilience of urban environment to natural hazards.

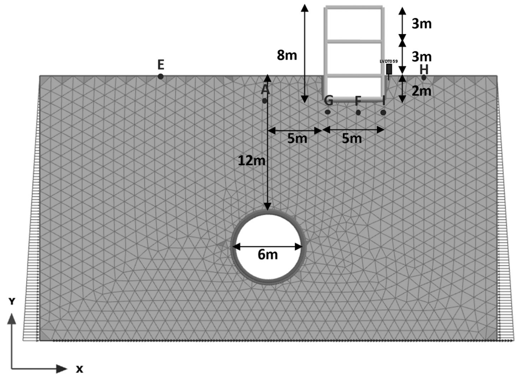


Figure 21. Numerical mesh (prototype scale).

The project STILUS, within the framework of the European funded network SERA (Seismology and Earthquake Engineering Research Infrastructure Alliance for Europe) intends to investigate this problem through a series of centrifuge tests.

In order to plan the centrifuge tests, a preliminary numerical study of tunnel-structure interaction in liquefiable soil was carried out as described in the following sections 4.2 and 4.3. A circular transverse section (modelling a bored tunnel) and a rectangular framed section (modelling a cut-and-cover tunnel) were taken into account at this preliminary stage, since they may be likely to occur in the urban environment.

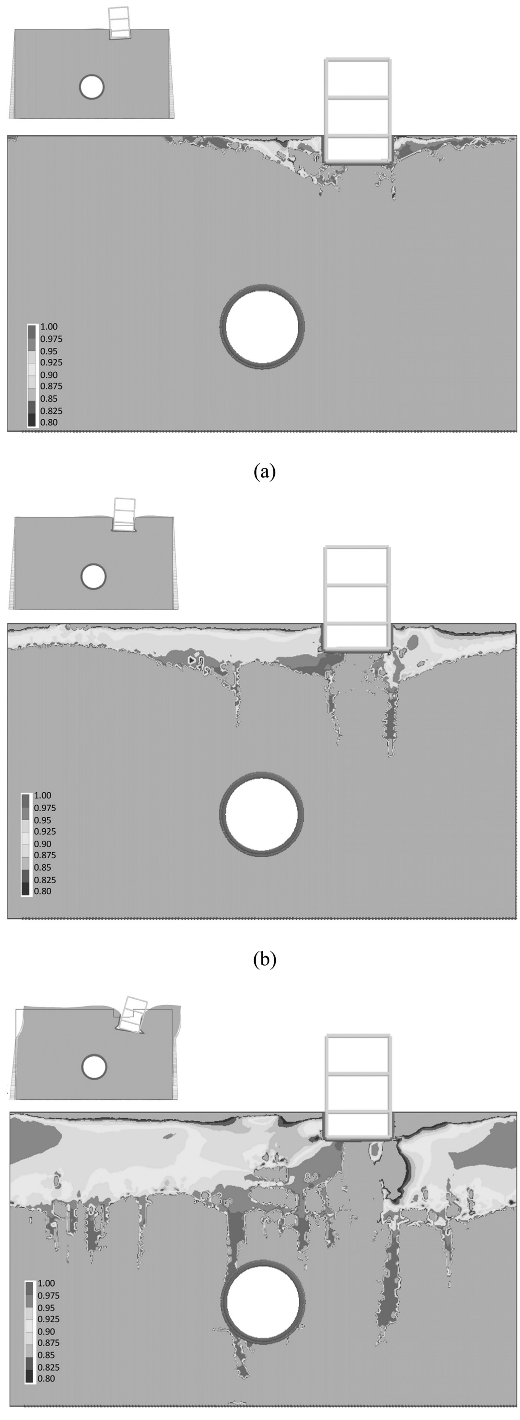
4.2 *Circular tunnel*

The numerical calculations were carried out using the same layout as shown in Figure 1, that has been analysed in section 3. The numerical model and the input signal used were the same as described in that section, being the problem modelled using the UBC3D-PLM model for the soil (Galavi et al., 2013). A simple structure was added in the model mimicking a two-storey building as shown in Figure 21 (Colamarino, 2017).

The building consists of a two-floors (3 m high each) and a basement (2 m deep). The building rests to one side of the tunnel as shown in the figure.

The building frame was modelled using linear elastic beam elements. Two different material datasets were used, one for the basement ($EI = 1.6\times105$, kNm^2/m, $EA = 1.2\times10^7$ kN/m) and the other for the rest of the building ($EI = 6.75\times10^4$ kNm^2/m, $EA = 1.6\times10^5$ kN/m). The mass assigned per unit length to the beam elements takes into account also the presence of the floors and the walls.

The same set of input signals as in the previous section was used in order to compare the results to the "greenfield" conditions considered in that section. Figure 22 shows the highest values of excess pore pressure ratio ($r_u > 0.8$) calculated in undrained conditions at the end of the shaking for the three input signals 'EQ1' (Fig. 22a), '2x' (Fig. 22b) and '3x' (Fig. 22c).

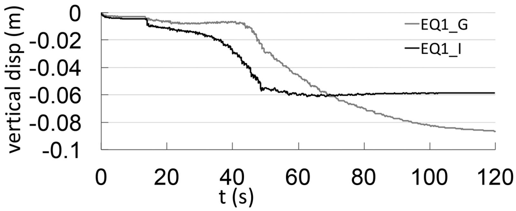


(a)

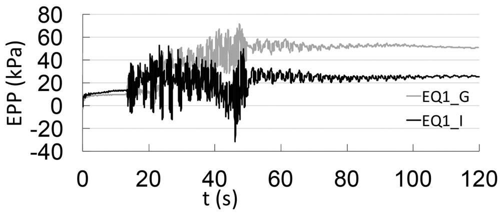
(b)

(c)

Figure 22. Excess pore pressure ratio distribution and mesh deformation (magnification 2) at the end of shakings (a) 'EQ1', (b) '2x' and (c) '3x.'

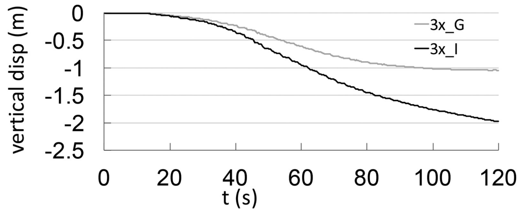


(a)

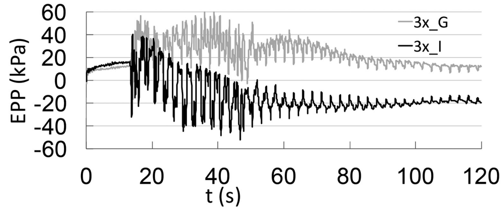


(b)

Figure 23. Time histories of settlement (a) and excess pore pressure (b) at the foundation level (‘EQ1’).



(a)



(b)

Figure 24. Time histories of settlement (a) and excess pore pressure (b) at the foundation level (‘3x’).

Insets in the same figure show the corresponding deformed configurations at the end of shaking.

As soon as the amplitude of the signal increases, larger areas of the sand layer are affected by liquefaction or are approaching it ($r_u > 0.8$). It is worth noticing that for ‘EQ1’ (Fig. 22a) the highest values of r_u are distributed in the area of maximum shear stresses around the building foundation. Instead, no evidence of liquefaction was observed in the results of the corresponding greenfield analysis in section 3 ($r_u < 0.8$, see Figure 13). The larger amplitude of the “2x” input signal (Fig. 22b) produces a continuous layer of shallow soil approaching liquefaction.

The influence of the building is still visible in this distribution but liquefaction does not affect the soil around the tunnel (C/D=2). When subjected to an even stronger shaking, a larger thickness of soil approached liquefaction (Fig. 22c). Compared to the corresponding greenfield analysis, the influence of the stresses induced by the building is evident both in terms of deviator and mean stress: calculated pore-pressure build-up are higher at the corners of the foundation due to initial higher shear stresses and lower towards the centre, where the mean stresses prevail. In this case liquefaction areas reached the tunnel below.

The building settles and tilts. Both settlement and tilt are influenced by the distribution of excess pore pressure around the foundation, that affects the degree of mobilization of shear strength in the foundation ground. However, when liquefaction reaches the tunnel depth, an increased uplift of the tunnel affects the building movements and the building starts to counter-rotate.

Figures 23 and 24 show the time histories of settlement and excess pore pressure calculated at point G and I (see Figure 21) with input signals “EQ1” and

Table 5a. Overview of the analyses with C/D = 2.

Input	$a_{max,b}$ g	$a_{max,s}$ g	ΔM kNm/m	ΔN kN/m
EQ1	0.054	0.098	52	118
2x (EQ1)	0.108	0.139	146	250
3x (EQ1)	0.162	0.203	172	235
Northridge	0.68	0.249	242	301
Norcia	0.78	0.318	164	200

Table 5b.. Overview of the analyses with C/D = 2

Input	tunnel max uplift m	thickness $r_u > 0.8$ m	building max settlmt m	building max tilt rad
EQ1	0.003	-	0.23	0.032
2x (EQ1)	0.023	3	0.709	−0.026
3x (EQ1)	0.168	7	2	−0.142
Northridge	0.175	8	1.29	−0.017
Norcia	0.146	6.5	0.793	0.021

“3x”. For the weaker “EQ1”, the larger positive excess pore pressure that arises around point G (Fig. 23b) produces a larger settlement of the building on that side at the end of shaking (Fig. 23a).

On the other hand, for the stronger “3x”, although negative excess pore pressure develop around point I (fig. 24b), the buildings settles more on the right side (fig. 24a), indicating an interaction with the uplift of the tunnel on the left side.

Tables 5a and 5b summarize the results achieved in the analyses with C/D=2. Positive tilt is assumed counter-clockwise.

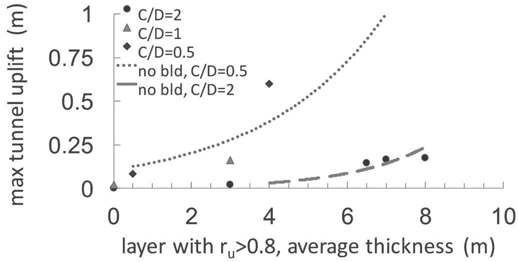


Figure 25. Tunnel max uplift in the analyses with building compared to corresponding trends calculated in analyses without.

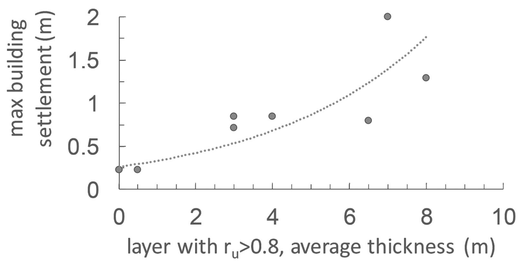


Figure 26. Building max settlement.

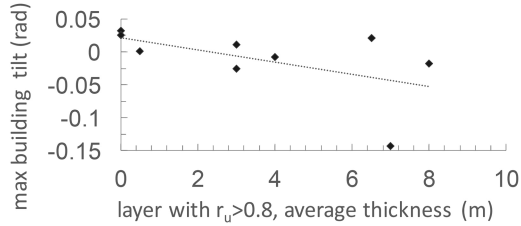


Figure 27. Building max tilt.

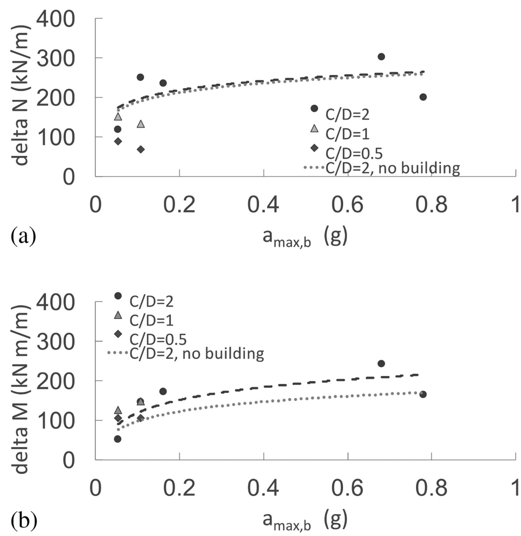


Figure 28. Maximum change of hoop force (a) and bending moment (b) in the lining at the end of shaking: all analyses in Table 5 compared to trend lines in Fig. 18 (no building).

In Figure 25 the maximum value of uplift of the tunnel axis is plotted for different values of the ratio C/D as a function of the average thickness of a continuous horizontal layer of soil where the excess pore pressure ratio r_u is larger than 0.8. As in section 3, such a value has been assumed as a proxy for the effect of liquefaction in the ground layer. In the same figure two curves are shown that represent the trends calculated for C/D = 2 and C/D = 0.5 in the analyses without buildings (section 3).

Although with some scatter, the trends are the same in both sets of analyses (with and without buildings), indicating a minor effect of the presence of a building on the amount of tunnel uplift, providing that similar distributions of pore pressure build-up affect the soil surrounding the tunnel.

Trends of increasing building settlement and tilt can be observed in Figure 26 and 27. Very low values of average thickness of the layer with ru>0.8 (close to zero) indicate that liquefaction occurs only in the proximity of the foundation of the building (e.g. Fig. 22a). This corresponds to limited settlement, although non-negligible. Much larger settlement is calculated when liquefaction is approached in larger volumes of soil, as for instance in the cases shown in Fig. 22b-c.

Correspondingly, it might be noted that in Figure 27 there is a decreasing trend of tilt towards negative values as the average thickness of the layer with ru>0.8 increases. Hence, a larger pore-pressure build-up generally induces the foundation to rotate clockwise. This indicates the effect of the upheaval associated with the tunnel buoyancy on the left side of the building.

In Figure 28 the change of hoop force (a) and bending moment (b) in the tunnel lining at the end of shaking is plotted as a function of the peak acceleration of the input signal at the base of the model. Trend lines for the case C/D=2 are shown as dashed lines and compared with similar trend lines from Fig. 18 for the 'greenfield' cases, that is without the building (dotted lines). The change of hoop force N induced by pore-pressure build-up is independent of the presence of the building. On the contrary, the change of bending moment is generally larger than in the 'greenfield' case. This finds justification in the less uniform distribution of stresses induced around the tunnel by the presence of the building (compare for instance values of r_u: for 'greenfield' conditions in Fig. 16b with 'building" conditions in Fig. 22c).

4.3 *Rectangular tunnel*

In order to analyse a typical case of a cut-and-cover tunnel in an urban environment, a rectangular section has been assumed, as shown in Figure 29. The same liquefiable sand layer and the same building as in section 4.2 are modelled.

Table 6 shows an overview of the analyses that have been carried out. The legend for the input signals has been given in Table 4. A set of analyses with input signals of increasing amplitude was carried out

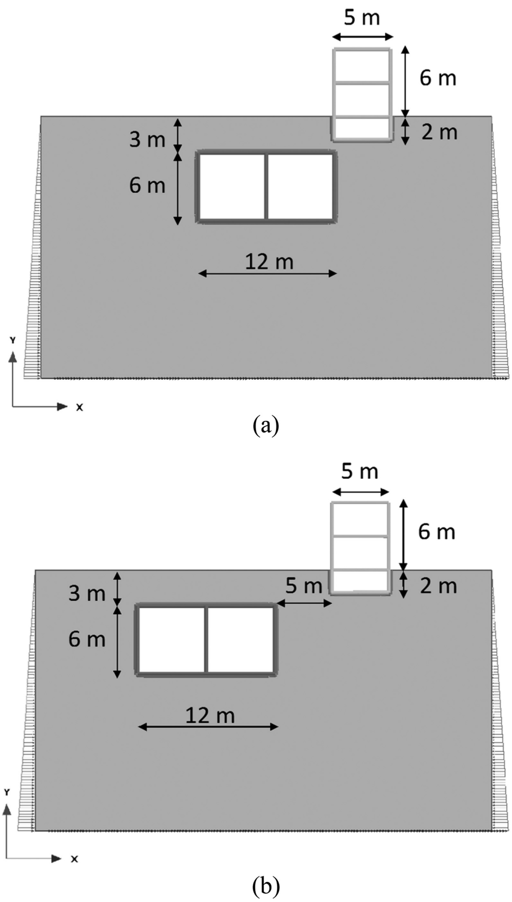


Figure 29. Models of rectangular tunnel with building: (a) building on the edge of the tunnel (d/C = 0), (b) building at a distance d = 5 m (d/C = 1.7).

in 'greenfield' conditions (Table 6a), to study the effect on the dynamic response and the pore-pressure build-up of the presence of the tunnel. Similarly, a set of analyses was carried out for models with a building and without a tunnel (Table 6b). Finally, the tunnel-building interaction was analysed with two sets of numerical models with both structures (Table 6c). In the former the building was located on the edge of the tunnel, with a distance to cover ratio, d/C = 0 (Fig. 29a). In the latter, the building was located at a distance d = 5 m on the right side of the tunnel, corresponding to d/C = 1.7 (Fig. 29b).

The tunnel was very shallow, with a cover C = 3 m, compared to the depth of the basement (2 m).

The dynamic response of the soil layer in 'greenfield' conditions (no building) with and without this tunnel is shown in Figure 30. In the figure the ratio between the peak acceleration at the surface and at the base is plotted against the peak acceleration at the base. It can be noticed that the presence of the larger rectangular tunnel reduces the amplification at the ground surface compared to the circular tunnel. In all cases the dynamic amplification calculated in 'free-field' in

Table 6a. Overview of 'greenfield' analyses without building.

input	tunnel max uplift m	thickness $r_u > 0.8$ m
EQ1	0.529	–
2x (EQ1)	0.998	2
3x (EQ1)	1.000	2
Northridge	1.300	12

Table 6b. Overview of the analyses without tunnel.

input	thickness $r_u > 0.8$ m	building max settlmt m	building max tilt rad
EQ1	3	0.572	0.058
2x (EQ1)	5	1.02	−0.009
3x (EQ1)	7	2.38	−0.1077
Northridge	7	1.2	−0.042

Table 6c. Overview of the analyses with tunnel and building.

d/C = 0

input	tunnel max uplift m	thickness $r_u > 0.8$ m	building max settlmt m	building max tilt rad
EQ1	0.095	1	0.23	−0.005
Northridge	0.421	12	1.32	−0.236

d/C = 1.7

input	tunnel max uplift m	thickness $r_u > 0.8$ m	building max settlmt m	building max tilt rad
EQ1	0.419	-	0.414	0.034
Northridge	1.340	8	1.72	−0.090

the corresponding analyses without a tunnel is much larger.

The differences between the curves reduce as the peak ground acceleration increases, when de-amplification occurs due to soil liquefaction, as discussed in section 3.5.

The presence of the building affects the distribution of excess pore pressure, as shown in section 4.2. This is confirmed by the analyses without a tunnel as shown in Figure 31a. Here the continuity of the horizontal layer approaching liquefaction is broken below the building due to the higher effective mean stresses.

The presence of the tunnel further affects the distribution of excess pore pressure, as can be seen from the comparison between Fig. 31a and Fig. 31b.

It is worth noting that liquefaction is confined most in the free-field areas at the right side of the building and the left side of the tunnel. However isolated liquefied soil volumes are identified below the tunnel,

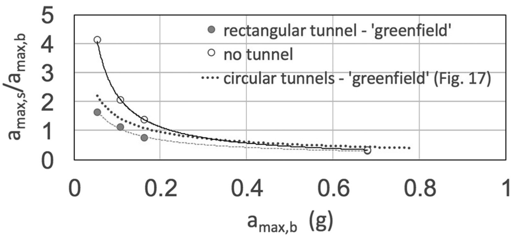
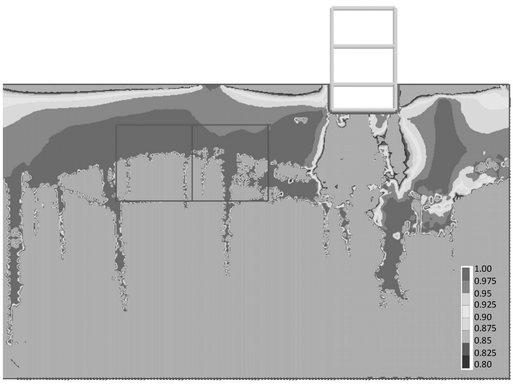
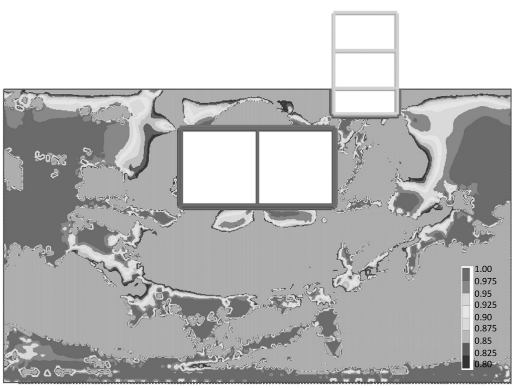


Figure 30. Ratio between peak acceleration at surface and at the base *vs.* peak acceleration at the base: with and without rectangular tunnel and comparison with trend for circular tunnel.



(a)



(b)

Figure 31. Excess pore pressure ratio distribution at the end of shaking 'Northridge': (a) 'no tunnel', (b) 'd/C=0'.

above the tunnel roof and between the tunnel and the building foundations. In the latter area the distribution of excess pore pressure depends on the relative distance d/C.

In Figure 32 the calculated values of building maximum settlement are plotted.

In the figure a trend of increasing settlement of the building with the average thickness of the layer approaching liquefaction is observed, although with some scatter. The presence of the tunnel reduces the

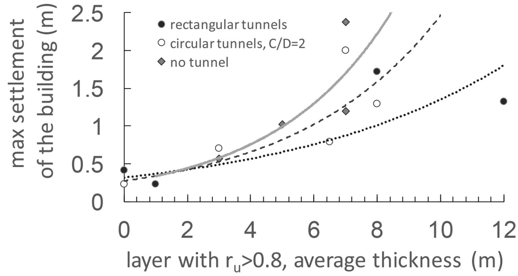


Figure 32. Max settlement of the building.

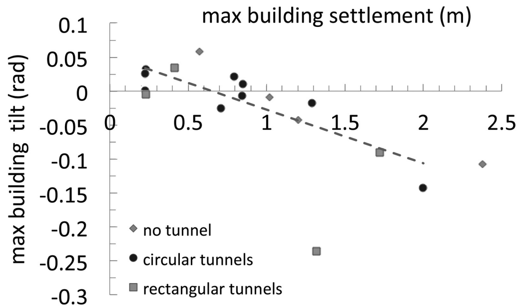


Figure 33. Max tilt vs. max settlement: analyses with building.

calculated settlement of the building. Such a reduction is more evident for the shallower and larger rectangular tunnels than for the deeper and smaller circular tunnels.

The calculated tilt and settlement of the building at the end of shaking are plotted one against each other in Figure 33. Despite some scatter, the plot shows a certain degree of correlation among the two quantities.

4.4 *Remarks*

This section has described a preliminary numerical study of tunnel-structure interaction in liquefiable soil. This study has been carried out to identify patterns of deformation that should be expected to occur in centrifuge tests to be carried out in a research project concerning such a problem.

The results have clearly shown how the distribution of excess pore pressure induced by shaking in undrained conditions is affected by the presence of the tunnel and of the building.

The relative distance between the two structures (here expressed in terms of ratio d/C of the horizontal distance between the tunnel wall and the building basement upon the tunnel cover) influences the solution both in terms of tunnel lining deformation and of building displacements. Consequence are observed in the distribution of internal forces in the tunnel lining and in the final configuration of the building.

A number of indications for implementing the physical modelling have been suggested by the results of the numerical analyses.

As far as the distribution of internal forces is concerned, since the analyses show that it is influenced by the presence of the building, it would be important to have a large number of measuring points along the tunnel lining, to get an experimental insight into this problem.

Furthermore, since building tilt is expected by the analyses, non-contact laser displacement transducers might be used in the centrifuge to measure such a tilt. They will be then associated to conventional transducers (LVDTs).

Moreover, the distribution of calculated ground movements induced by soil liquefaction may help to define areas where the addition of finer content (down to the nanoscale) may reduce the mobility of the soil. On the experimental side, this show the potential benefit of using digital imaging and particle image velocimetry in centrifuge tests, through a transparent side of the model container.

The numerical results also show that the tunnel uplift is driven by the increase of pore pressure below the tunnel invert and the concurrent reduction of effective stresses (and shear resistance) in the cover. Hence the safety factor against uplift is reduced. This will considered in the layout of the centrifuge tests by deploying transducers to measure pore pressures where the analyses show that a significant build-up may develop.

The calculated distribution of excess pore pressure induced around the tunnel and the building basement may also be useful to identify where mitigation techniques that may locally reduce pore-pressure build-up (for instance: drainage, densification, induced partial saturation) would be most effective against the effects of soil liquefaction and should be implemented in the tests.

Nevertheless, it should be pointed out that the results of the numerical predictions should be considered with care. Although the potential of the constitutive model used, and of other models of similar complexity, to predict pore pressure build-up and to identify the occurrence of soil liquefaction has been shown in several studies, their accuracy in predicting the deformation of soil approaching or experiencing liquefaction and large strain is still a matter of study. Hence the need to run tests on physical models, thus achieving an experimental assessment of the behaviour of the soil and interacting structures (tunnel and building) in such conditions.

5 CONCLUSIONS

This work has illustrated how numerical calculations can be used in association with centrifuge testing to model different aspects of tunnel behaviour during earthquakes. The scope of the paper has been limited to a few aspects, mainly concerning the change of internal forces in the tunnel lining during shaking and the effect of soil liquefaction. Tunnel-building interaction during shaking in liquefaction prone soil has also been investigated. However, those analysed are only examples of a larger number of applications where an integrated experimental-numerical modelling approach can be followed.

The point of view of this paper is on purpose slightly biased towards the numerical modellers that may benefit of centrifuge tests to calibrate their models. The use of centrifuge testing (and physical models in general) should be considered as complementing laboratory testing on single elements when it comes to study specific aspects of boundary value problems. Indeed the possibility of evaluating numerical models using well-defined and controlled experiments increase the reliability of any numerical study where advanced constitutive models are used.

On the other hand, experimental activities may benefit significantly from a preliminary numerical study that helps to define the scope of testing and the key aspects that the physical model should be able to reproduce. This permits efficient use of resources, possibly reducing the number of experiments, to focus effective efforts on the specified target.

The main achievements of this work are only partial and deserve further investigation. However, they help to show how a combined use of physical and numerical modelling is necessary to analyse earthquake-induced effects on tunnels and other similar subsystems of civil infrastructures.

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An example of effective mentoring for research centres

C.E. Bronner, D.W. Wilson, K. Ziotopoulou, K.M. Darby, A. Sturm,
A.J. Raymond, R.W. Boulanger & J.T. DeJong
University of California, Davis, California, USA

D.M. Moug
Portland State University, Portland, Oregon, USA

J.D. Bronner
GeoEngineers, Redmond, Washington, USA

ABSTRACT: Engineering centre research faculty and staff value the importance of performing educational outreach and mentoring graduate students. However, these activities are often less structured than research projects, which leads to variable and less effective results. The geotechnical group at the University of California, Davis (UC Davis), which includes research faculty and staff at the Center for Geotechnical Modeling and the Center for Bio-mediated and Bio-inspired Geotechnics, developed a Ladder Mentoring Model (LMM) for mentoring graduate students in academic environments to enrich graduate student development while minimizing additional demands on centre personnel. The LMM is a combination of several existing mentoring models and relies on six core principles where the outcome is students receiving guidance from a variety of mentors with different areas and levels of expertise or experience. This paper provides a brief overview of the UC Davis LMM and describes how it is integrated into three critical areas of graduate student development: technical training, professional skills, and educational outreach.

1 INTRODUCTION

Training graduate students is often a central objective for engineering research centres. Traditional models for training graduate students provide limited exposure to researchers other than faculty and staff related to their thesis project. There is often minimal development of non-research skills needed for successful academic careers, such as teaching, networking, and communication skills.

Centre personnel, however, have several other responsibilities including training visiting researchers on centre equipment, preparing for and performing experiments, maintaining centre equipment, and developing researchers. While centre research experiments are meticulously designed and orchestrated, a lack of structure often exists in mentoring and educational outreach activities. Despite recognizing the importance of these latter activities to prepare future engineers and scientists and broaden participation of underrepresented groups in STEM disciplines, centre researchers may feel burdened with other demands that produce timelier, more concrete results.

To improve graduate student mentoring and educational outreach effectiveness in research centres without excessive additional demands on personnel, a restructuring of these activities is needed. This paper presents a model for organizing mentoring and outreach activities to produce researchers with the technical expertise, networks of collaborators, ability to communicate to all audiences, and other professional skills that can help them achieve their career goals. After a brief overview of common mentoring practices, an overview of the UC Davis Ladder Mentoring Model (LMM) is presented along with its six core principles. The following three sections provide examples of how the LMM is applied with six core principles at UC Davis in three different areas: technical training, professional skills development, and educational outreach. The paper concludes with ideas for transferring and tailoring UC Davis's LMM model to other institutions.

2 MENTORING IN ACADEMIC ENVIRONMENTS

Table 1 summarizes the different types of mentoring models used in academic environments (Hanover 2014; Lee et al. 2015). The primary differences between the models include the distance in expertise between the mentor and mentee, the number of mentors, the combined breadth of expertise a mentee receives, and the amount of agency a mentee has in the mentoring process.

Table 1. Common mentoring paradigms used in academia (sources: Hanover Research 2014; Lee et al. 2015).

Mentoring Model	Example
Traditional one-on-one mentoring: Mentor seen as distributer of advice/help	Faculty advisor (mentor) guides graduate student (mentee) through the academic job search process
Peer mentoring: Mentoring between two or more individuals who are considered peers or have similar status	Graduate student (mentor) trains another graduate student (mentee) on how to set up a centrifuge test
Group/collective mentoring: Combination of traditional and peer mentoring	Faculty member (mentor) coaches their graduate group (mentees) on giving research presentations; students may also guide peers
Mutual mentoring: Mentoring relationships that include a wide variety of mentors and focus on specific areas of experience and expertise. Assumes that no single individual possesses all expertise that an individual needs	An assistant faculty member (mentee) mentored by a network of individuals (mentors) that may include peers, senior faculty, administrators, etc.
Reverse mentoring: The mentor in this role is often in the role of the mentee in other situations between these two individuals	Graduate student (mentor) guides a faculty member (mentee) through a new analytical approach
Mentoring up: Similar to a traditional mentoring model, however the mentee is proactive in determining the help they need and seeking it out	Graduate student (mentee) asks faculty advisor (mentor) for help on how to develop their professional network

2.1 *UC Davis Ladder Mentoring Model*

Geotechnical faculty at UC Davis encourage students to act as both mentees and mentors and to work in a collaborative environment. Often, students are mentored in research by near-peers who are just a few steps up the ladder from them (e.g., another graduate student who is one- or two-years ahead of them). Over time, the program has also developed structures that have integrated the LMM into the academic, professional development, and outreach training that graduate students receive. Through the LMM, graduate students obtain many of the benefits of traditional, peer, group, mutual, and reverse mentoring models, while practicing the pro-activeness from the mentoring up model.

Recently, the UC Davis team has started studying the LMM to evaluate its benefits and to share lessons learned with other institutions. It is posited that the model works due to the integration of the following six core principles into graduate student training in research, professional development, and educational outreach activities. Examples of how these principles are applied are provided in the next three sections.

1. Providing a sustainable structure with clear expectations
2. Tailoring mentoring to needs of the individual
3. Leveraging resources generously
4. Promoting an inclusive culture
5. Encouraging consistent assessment
6. Building networks that expand beyond the borders of the institution

The three organizations in Table 2 provide structure, vision, and resources for the sustainable implementation of the six LMM principles. The Center for Bio-mediated and Bio-inspired Geotechnics (CBBG) and Center for Geotechnical Modeling (CGM) are research centres, whereas the Geotechnical Graduate Student Society (GGSS) is a student organization.

Table 2. UC Davis geotechnical organizations.

Organization	Purpose
CBBG	Transforms geotechnical practice by developing technologies that leverage natural biogeochemical processes or leveraging principles/functions/forms from natural analogues (i.e., bio-inspired), resulting in more efficient and sustainable solutions
CGM	Provides access to geotechnical modelling facilities to enable major advances in the ability to predict and improve the performance of soil and soil-structure systems affected by natural hazards
GGSS	Promotes scholarship, service, leadership, and social events to foster collaboration within the UC Davis geotechnical group

*Abbreviations: CBBG = Center for Bio-mediated and Bio-inspired Geotechnics; CGM = Center for Geotechnical Model-ing; GGSS = Geotechnical Graduate Student Society

Many individuals in the UC Davis geotechnical group are connected to one or more of these organizations.

3 TECHNICAL TRAINING

The Center for Geotechnical Modeling (CGM) serves as a resource in the National Science Foundation's Natural Hazards Engineering Research Infrastructure program (NHERI). The facility hosts researchers from across the US and provides the technical training and oversight necessary to maintain a high standard of research quality. Currently 15 students are actively working across six projects at the CGM, including six non-UC Davis students. Typically, about 10 to 15 researchers per year will rotate through the testing facility for short durations.

New researchers start with varying skill levels, academic backgrounds, and hands-on mechanical

Table 3. Typical needs of different types of CGM researchers

Researcher Type	Typical Duration	Mentoring/ Training Need	Ability to Mentor
Undergraduate student from UC Davis	10 weeks to 2 years	Very high; transitioning to medium/high	Medium
Visiting undergraduate students	6 to 10 weeks	Very high	Low
UC Davis graduate students & post-docs	10 weeks to 6 years	Medium to high; transitioning to low or medium	High
Visiting graduate students & post-docs	2 to 6 week intervals over 1 to 3 years	Often high initially; transitioning to low or medium	High
Visiting research faculty	2 weeks to 1 year	Depends on experience	High

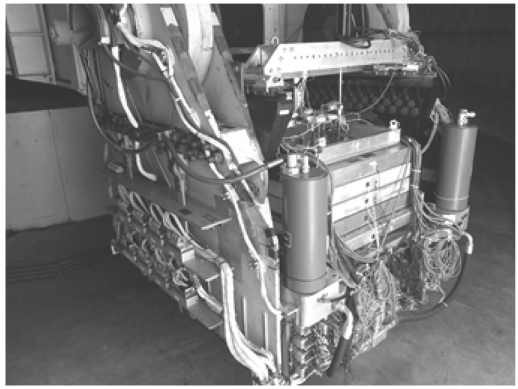

Figure 1. A typical experiment on the 9 m centrifuge at UC Davis includes 1500 kg of soil, over 100 sensors, in-flight characterization using cone penetrometers, and multiple simulated earthquake events. Experienced researchers may spend two months building, testing, and excavating such a model. New researchers learn through apprenticeships important centrifuge modelling techniques such as how to place soils, how to calibrate sensors, how to place and log sensors during model construction, how to design a test protocol, and how to manage their test schedule and facility resources, before attempting to lead an experiment.

expertise. Table 3 describes types of CGM researchers and their typical characteristics, including the amount of time they spend at the CGM.

The CGM follows an apprenticeship model to introduce new researchers to centrifuge testing. CGM staff train new users on methods directly through annual workshops and hands-on equipment training at the start of a researcher's time on site. However, new users can still be confused even after a lesson on what to do.

The apprenticeship model grew naturally from the mutual benefits gained by experienced users needing extra assistants and new users needing practice to support their training. Apprenticeship is formally integrated into current CGM operating protocols.

3.1 *CGM apprenticeship model*

At the CGM, researchers are responsible for their entire physical model test program (Fig. 1). New researchers must learn physical modelling techniques, sensor and data acquisition procedures, as well develop an engineering design of their research application. Researchers, acting as project managers, learn to supervise assistant researchers, productively direct staff, work with outside vendors, and manage non-personnel resources. Given the high cost of experiments on the 9 m centrifuge, both in terms of fees and consumed effort, projects cannot afford to let new researchers learn by failure in their first experiment. Thus, new researchers serve as apprentices to experienced researchers on other models/projects to learn how to run a centrifuge test.

The apprenticeship model requires new researchers to assist an experienced researcher during an experiment. The mentee is encouraged to participate in the experiment from beginning to end so that they can learn the entire process before becoming responsible for their own test. CGM staff still provide training on equipment, but focus primarily on personal and equipment safety. Apprentices "learn while doing" within a safe, supervised environment.

The apprenticeship model benefits both the mentee and the mentor. The mentee gains the experience and training required to design their future experiment. The mentor gains the advantage of having an extra set of hands and eyes. The CGM expects all researchers to serve as both mentees and mentors, so that all can gain experience and receive the benefit of outside help.

3.2 *Role of CGM*

The CGM has institutionalized the expectation for the apprenticeship model by incorporating the practice into facility use rates. Projects are charged a base fee for sending a "new lead researcher" to the CGM. New lead researchers require additional orientation, training, and interaction, which consumes effort of the CGM staff. Credits against this fee are given when the researcher has the tools to be self-sufficient in order to pass on the effort savings for the centre. For example, half the fee is returned if the new lead researcher has served a full apprenticeship at the CGM. Further credits are given for other forms of formal training such as attending the annual centrifuge users' workshop and taking courses in signal conditioning.

The CGM also has a fee for "basic researcher support" intended to recover costs of CGM staff providing the extra set of helping hands when a project only sends one researcher to perform a test. Credits are given if a project provides their own assistance, such as through mentoring other users.

The well-documented apprenticeship model together with the fee structure and credit incentives

Table 4. Centrifuge mentoring experience of Kathleen Darby.

Mentor or Mentee	Position and Affiliation*	Year	Role	Primary motivation in mentorship
R. Boulanger	Faculty	2014-2018	PhD Advisor	Lead research project
J. DeJong	Faculty	2014-2018	Mentor	Co-lead research project
D. Wilson	Faculty	2014-2018	Mentor	Train students on test methods
Jackee A.	GS	2014	Mentor	Transfer knowledge on NEEShub and data analysis
Mohammad K.	GS at VT	2014, 2017	Mentor	Gain assistance, train Kate and Jaclyn on test methods
Jaclyn B.	GS	2014, 2016	Peer Mentee	Co-apprentice under Mohammad. Co-lead 1 m centrifuge tests
Daniel C.	UG	2014	Peer Mentee	CGM UG employment. Experience research and assist researchers
Yunlong W.	VS from CEA	2015	Apprentice	Learn UC Davis test methods
Maggie E.	GS at OSU	2016	Apprentice	Learn 9 m test methods
Maddie H.	UG	2016	Mentee / Assistant	CGM UG employment. Experience research and assist researchers
Mohammad K.	Postdoc	2017	Assistant	Reciprocate assistance on test
Dexter H.	UG at MSU	2017	Mentee	NHERI REU to experience research
Gabby H.	GS (CBBG)	2017	Mentee / Apprentice	Gabby, Caitlyn, Alex, and Greg: Learn general 1m test methods and specific research protocols for their projects
Caitlyn H.	GS at ASU (CBBG)	2017	Mentee / Apprentice	
Alex S.	GS	2017	Mentee / Apprentice	
Greg S.	GS	2017	Mentee / Apprentice	
Jiarui C.	GS at UIUC	2018	Apprentice	Jiarui and Soham: Learn centrifuge testing methods (shared project)
Soham B.	GS at UV	2018	Apprentice	

* Institutional affiliation is UC Davis unless otherwise listed. Abbreviations: ASU = Arizona State University; MSU = Morgan State University; OSU = Oregon State University; UCD = UC Davis; UIUC = University of Illinois – Urbana-Champaign; UV = University of Vermont; VT = Virginia Tech; CEA = China Earthquake Authority; GS = graduate student; UG – undergraduate student; VS = visiting scholar; REU = Research Experience for Undergraduates.

have proven effective in getting 100% participation by project teams from UC Davis and near 100% by external users. External teams have an added burden of paying travel costs, which reduces their apprenticeship participation rate. When possible, external teams apprentice on the 1m centrifuge, where mentees can participate from beginning to end over a shorter time. The CGM has implemented parallel operating protocols across the 1m and 9 m centrifuge so that procedural training is consistent, which has improved the apprenticeship participation of external research teams.

The CGM use fees are located on the CGM website under the "information for users" area. https://cgm.engr.ucdavis.edu/information-for-users/

3.3 *Connection to the LMM*

The apprenticeship model for training researchers in centrifuge techniques aligns with the LMM framework and its six core principles as described below.

Providing a sustainable structure with clear expectations: The centrifuge test pricing incentives provide the primary structure for the success of the apprenticeship model. This structure offers users a price incentive to participate both as a mentee and as a mentor, and has helped the apprenticeship model of training to become "the norm" at the CGM.

Tailoring mentoring to needs of the individual: The model allows researchers to be paired with individuals who are their near-peers with respect to the experiment they will be performing. Researchers actively work with someone performing experiments using similar techniques to those they need to learn in addition to general training. As external researchers have additional housing costs, the CGM implemented parallel operating protocols for both the 9 m and 1 m centrifuge to allow researchers to train on either centrifuge. This flexibility reinforces the structure by making the program feasible for internal and external researchers.

The graduate students involved in mentoring develop advising skills, which is particularly important for those who plan to enter academia or serve in leadership roles. The high number of mentees a CGM graduate student mentors provides them more opportunity to develop their teaching style. Table 4 provides an example of doctoral student's mentoring experiences.

Leveraging resources generously: Leveraging of resources occurs between UC Davis and visiting centrifuge researchers. Through the apprenticeship program, a researcher is provided a necessary assistant at no cost, while another researcher receives training in centrifuge methods and a credit towards the cost of their centrifuge tests. The two projects benefit from reduced costs and the CGM staff can better utilize their expertise in centre operation and technical research advancement.

Promoting an inclusive culture: The apprenticeship model provides the opportunity to involve researchers from a broad range of backgrounds, abilities, expertise, and development levels. For example, the apprenticeship model allows for the inclusion of undergraduates in centrifuge research. Typically, undergraduates are not able to commit the time or flexible schedule needed to participate in centrifuge experiments. They can, however, offer valuable assistance as the third member of a centrifuge team while gaining valuable research experience. Provision of the primary assistance by the apprenticeship program produces more opportunities for undergraduates to work as an extra assistant when their schedule permits.

Encouraging consistent assessment: The CGM has a stated performance goal of developing its members for the future workforce. Objectives toward this goal include providing ladder mentoring toward the development of independent researchers, engaging researchers in education and outreach activities (EOT) (to be discussed later), and providing technical training on all facets of geotechnical centrifuge testing. Progress is assessed by tracking the percentage of teams with ladder-mentored lead or assistant researchers (target >90%, actual 10 of 11 since 2016), percentage of users engaged in EOT (target >50%, actual >75% since 2016), and through user satisfaction surveys (target > 90% of users satisfied or very satisfied with training, actual surveying has been informal to date). Our user surveys to date have indicated strong support for the apprenticeship model, but also a consistent desire for improved documentation.

The UC Davis geotechnical group is now working to improve and expand assessment of the ladder mentoring program across all activities in an effort to better quantify its impact on preparing its members for the twenty-first century workforce.

Building networks that expand beyond the borders of the institution: The CBBG and CGM both include participation by researchers across the US. These activities give users valuable opportunities to work with people from diverse institutions and academic backgrounds (Fig. 2). Anecdotal observations indicate that knowledge, beyond centrifuge testing skills, is being broadly disseminated and wide-reaching networks are being developed.

3.4 *Example: Experience of a graduate student*

To demonstrate the potential impact of the apprenticeship model, Table 4 highlights the centrifuge-related mentoring experiences of a graduate student participating in both the CGM and CBBG, Kathleen Darby. Her research included centrifuge tests over a period of five years. As Ms. Darby progressed through her graduate work, she worked with 17 different researchers (three faculty, one visiting scholar, one post-doc, nine graduate students, four undergraduate students) from eight different institutions covering a range of research roles, as described in the table.

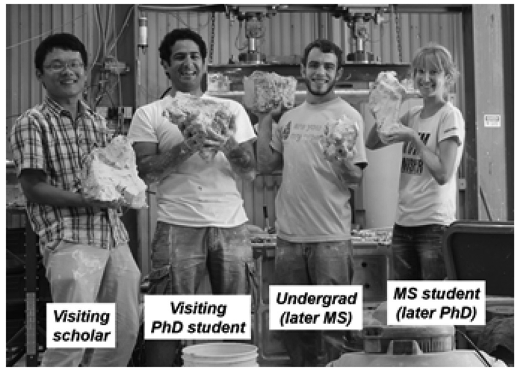

Figure 2. Ladder mentoring in practice. Visiting PhD student Mohammad K. (VaTech) led an experiment looking at ground improvement using soil cement. He mentored three engineers during the test and benefited from the depth of support available for a complicated test. Dr Wang, a visiting scholar, gained experience in how to perform centrifuge testing that he would take back to his new centrifuge in China. Kate D., as an MS student, apprenticed during the experiment so that she could lead her own tests on the 1 m centrifuge and eventually the 9 m centrifuge as a PhD student. Daniel C. gained valuable research experience as an undergraduate and ultimately decided to further pursue his education as an MS student.

Due to the CGM's apprenticeship model, Ms. Darby's contact with researchers at several institutions allowed her to gain and distribute centrifuge-related skills beyond the boundaries of the CGM. She received mentorship from researchers within and outside of UC Davis, including initially serving as an apprentice under a visiting graduate student. As a graduate student, she mentored undergraduate and graduate students from seven different institutions, including several who apprenticed with her or were supervised by her.

4 PROFESSIONAL SKILLS DEVELOPMENT

The UC Davis geotechnical graduate program typically consists of about 30 graduate students and six full-time faculty, serving as their graduate advisors. A traditional mentoring system where knowledge transfer occurs only from faculty member to student would lead to limitations on mentoring in professional skills, such as restrictions based on faculty time constraints and variability based on an advisor's individual sense of importance for specific skills. Expansion of a mentoring system to include knowledge transfer between peers and research staff increases development of and feedback on professional skills.

At UC Davis, the Geotechnical Graduate Student Society (GGSS) provides an additional structure for graduate student professional development. The GGSS program actively fosters leadership, outreach, and mentorship skills in its members, making them better qualified and well-rounded to graduate to professional or academic careers. The organization's practices align with the LMM core principles and expand support originally provided through geotechnical faculty members and the CGM.

4.1 *Geotechnical graduate student society*

In 2007, the UC Davis geotechnical engineering faculty members guided the graduate students in initiating the GGSS to formalize and focus the activities used to develop the professional skills of graduate students. The goal of the GGSS is to promote scholarship, service, leadership, and social events for the geotechnical group at UC Davis. The intention is to foster community and collaboration, and provide opportunities to promote graduate student education and professional development.

The GGSS is governed by a board consisting of six officers: President, Treasurer, Seminar Coordinator, Social Events Coordinator, Field Trip Coordinator, and Outreach Coordinator. Each officer has clearly defined responsibilities and opportunities. For example, the seminar coordinator recruits and hosts seminar speakers, which allows them to develop a professional network that they can leverage for employment opportunities as they near graduation. The GGSS board is mentored by the faculty advisor.

Faculty members rotate the responsibility of GGSS faculty advisor so that the workload is fairly distributed. The faculty advisor provides historical context and advice to the students as they navigate their new roles. While the faculty advisor will always be a critical role, the GGSS board retains continuity of some members from year to year and draws on advice from past officers. The officers are usually established senior graduate students who in turn serve as mentors to junior officers and new GGSS members. New officers are elected in April and current officers end their terms the following June to ensure there is training time for new officers.

The GGSS organizes a variety of events including a weekly seminar series, field trips, educational outreach activities, and social outings, which diversifies the expertise and experiences to which graduate students are exposed. The largest GGSS event is the annual Round Table where about 80 geotechnical professionals from government and industry are invited to a full day of student presentations, poster sessions, panel discussions, and closing social. The goal of the Round Table is to foster connections between UC Davis researchers and leading professionals by providing opportunities for open conversations, exposure and feedback on current research, exchanges or collaborations, and connections among future colleagues.

4.2 *Role of CGM*

The CGM supports the goals of the GGSS by providing connections and institutional knowledge. Networking opportunities include interacting with visiting scholars at the CGM, utilizing the growing network of professional contacts when planning GGSS field trips and seminars, and connecting GGSS members with long-term educational contacts for outreach events.

The institutional history provided by the CGM was instrumental for the GGSS when developing its educational outreach program as it could build off the centre's previous experience and existing connections. Graduate students learned whom to contact and which activities had been the most successful.

4.3 *Connection to the LLM*

GGSS mentoring relationships strongly rely on characteristics of the mutual mentoring, peer mentoring, and mentoring up models. The GGSS structure relies on the six core principles in the LMM to provide effective professional skills development for graduate students.

Providing a sustainable structure with clear expectations: The GGSS provides a structure, outlined in its bylaws, with clear roles and responsibilities of officers. The election process and officer overlap period provide continuity for the organization and minimize the possibility of knowledge loss when students graduate.

Geotechnical faculty and current graduate students set a clear expectation that all graduate students in the group should be active participants in the GGSS. If students are not attending seminars, their faculty advisor is responsible for strongly encouraging their attendance, often through a reminder of the benefit they are missing out on. The importance of participation in the GGSS is highlighted from their first day on campus; prospective graduate student campus visits include attendance at a GGSS organized activities such as the Round Table event or a weekly seminar.

Tailoring mentoring to needs of the individual: Students in the UC Davis geotechnical group vary based on their experiences, career ambitions, and desired professional skills. The GGSS offers a variety of involvement levels, which requires students proactively decide how much they can or want to contribute and gain from the GGSS at a given time in their graduate study.

As a baseline, all students are expected to attend the weekly seminars and the Annual Round Table, which together provide essential exposure to professional practice and opportunities for networking. Note that all seminar speakers are taken to lunch by a group of two to four GGSS members, so all students have opportunities for establishing personal connections with various professionals during the year. In addition, GGSS members can participate in some combination of the field trips and outreach events held throughout the year, with that mix varying from year to year. For example, an MS/PhD student may only have time to participate in one or two outreach events in their first year (due to class workload), may participate more heavily for the next year or two, and then participate less frequently in the last year or two depending on other commitments or roles they assume. The same MS/PhD student may serve in an officer role (e.g., seminar coordinator) in their second or third year, followed by a second officer role (e.g., president) in the fourth or fifth year.

Additionally, the GGSS structure allows students to work on specific professional skills that they want to improve. For example, a student who has difficulty communicating their research to non-technical audiences may choose to participate in outreach activities to practice these skills. Another student who struggles in professional networking situations may become the seminar coordinator to hone these skills in a supportive environment.

Leveraging resources generously: Both the CGM and CBBG have responsibilities related to the professional development of graduate students. By these centres supporting the GGSS and encouraging their students to be active members, they leverage the enthusiasm of graduate students and provide a structured approach to professional development.

The GGSS, CGM, and CBBG also leverage resources for providing professional development. The Round Table event provides the majority of the funds for GGSS activities; the event's success is partially due to the reputation of the CGM and research faculty. CBBG resources (e.g., webinars) for supporting professional development of its students are often shared with other GGSS members. The CBBG also provides funding resources to support the outreach activities of the GGSS; these activities are further discussed in Section 5.

Promoting an inclusive culture: GGSS members actively recruit new graduate students as members. Their commitment to inclusivity is demonstrated by the policy in their bylaws that automatically makes any registered UC Davis geotechnical graduate student a voting member of the GGSS. The GGSS also has a practice of inviting visiting students and scholars to participate in GGSS activities as honorary members while they are in Davis.

The culture of inclusion is demonstrated through diverse leadership in the GGSS. Nationally, 20% of civil engineering graduate degrees are earned by women. Currently, 50% of the GGSS officers are women and 33% are underrepresented minorities. Three of the past seven presidents have been women. These statistics indicate that women and other underrepresented groups in engineering are supported and actively participating in the GGSS. GGSS students further stress the importance of inclusion by including presentations on topics such as inclusion in engineering education and impostor phenomenon in their seminar program.

Encouraging consistent assessment: After every large GGSS event, students host a debriefing session to identify strengths, weaknesses, and opportunities for improvement. Feedback on weekly seminars and social events is provided during quarterly GGSS board meetings. This consistent assessment followed by action to address concerns leads to ever-improving, high-quality events. For larger events, surveys are distributed to collect participant feedback and include their input in the debriefing meetings.

Building networks that expand beyond the borders of the institution: The GGSS members have helped expand the influence of the UC Davis geotechnical program beyond the institution's borders. Due to the GGSS's success at UC Davis, CBBG faculty and students used the GGSS as the model when designing the engineering research centre's Student Leadership Council (SLC). The SLC consists of graduate student and undergraduate student representatives from all CBBG partner institutions: Arizona State University, Georgia Institute of Technology, New Mexico State University, and UC Davis. To help establish similar expectations and a culture of inclusion in the SLC, UC Davis students, Michael Gomez and Alena Raymond, served as the president for the first and second years of the centre, respectively. Additional plans for expansion include collaborating with GGSS alumni now working at other universities to help establish a similar graduate student organization at their universities.

The professional network for UC Davis researchers has expanded through positive interactions of geotechnical professionals with students during seminars, field trips, professional and K-12 outreach activities, and the Round Table event. This reputation has helped a large percentage of students secure jobs before graduating; about 90% of master's students are hired by companies who attend the Round Table.

4.4 *Example: Round Table event*

The GGSS's Annual Round Table event foster connections between leading geotechnical professionals and UC Davis faculty and graduate students by providing opportunities for open conversations, exposure and feedback on current research, exchanges or collaborations, and connections among future colleagues. During the event, geotechnical graduate students present their research to professionals from industry, consulting firms, and government organizations through poster and oral presentations. The event also includes an industry panel discussion and social activities.

Round Table guests provide gifts that go to an account overseen by the civil and environmental engineering department, but controlled by the GGSS, and those funds support the GGSS activities throughout the year. These generous gifts reflect the fact the community has embraced the Round Table as an event they look forward to, they like to support the broader educational experience of graduate students, and they like the personal connections that lead to either hires or connections with future colleagues.

GGSS students plan and run all portions of the Round Table, which requires students to interact with professionals, plan out all logistics for the event, and develop an engaging program. Each year the GGSS President leads the event, however successful implementation requires a coordinated effort from all GGSS members. In their first year at UC Davis, students' participation at a minimum includes creating an abstract and poster presentation, informal conversations with professionals, and observations of their senior GGSS peers. By their second year, students will take on more responsibilities and may eventually lead the event or

Table 5. Different levels of GGSS member participation during Round Table.

Level of Involvement	Description of Mentoring	Role
First year graduate student	Mentoring focuses primarily on preparing individuals to present their research to a professional audience in a clear and engaging manner, including through their design of a research poster. Mentoring comes from faculty advisors and fellow GGSS students. Students make minimal contributions to larger planning efforts, mainly observing their peers.	Mentee
2+ years as graduate student	With respect to interactions with industry and poster preparation, students transition from mentee to mentor roles. Students receive mentoring from faculty advisors and fellow GGSS students on poster and/or oral presentations. Students make minimal contributions to larger planning efforts, mainly observing their peers.	Mentee & Mentor
GGSS Officer	Mentored by GGSS faculty advisor and provides mentoring to junior GGSS officers and members. Students contribute to larger planning efforts, such as program design and implementation and contacting professionals	Mentee & Mentor
GGSS President	Mentored by GGSS faculty advisor and provides mentoring to junior GGSS officers and members. Student is responsible for the event.	Mentee & Mentor

Table 6. Mentoring interactions initiated due to Round Table.

Mentoring Interactions at Round Table
Prior to Event • GGSS past/senior officers mentor new officers on logistical processes involved, as well as how to handle moments of stress (near-peer mentoring) • GGSS faculty advisor mentors GGSS president through check-in meetings and advising on logistics, especially those related to industry (traditional mentoring) • GGSS senior members mentor new members on preparing research posters and how to interact with industry (near-peer mentoring) • GGSS members give feedback to each other on their posters and presentations (peer mentoring)
During & Post Event • Industry members and faculty members provide feedback and advice to graduate students on their research projects (form of mutual mentoring) – potentially forming new research contacts • Faculty and GGSS members provide constructive feedback to each other on Round Table execution – strengths, weaknesses, opportunities (form of collective mentoring)

give one of the keynote presentations. Table 5 provides a potential Round Table path for GGSS members over their academic journey.

Table 6 lists examples of ladder mentoring interactions that occur during the preparation and implementation of the Round Table.

5 EDUCATIONAL OUTREACH

In addition to the technical training and professional development of graduate students, the mission of engineering research centres often includes providing service to the profession in the form of educational outreach activities. Despite good intentions, outreach activities are often ad hoc and their impact is seldom assessed. Funding agencies, such as the US National Science Foundation (NSF), are increasing the burden of evidence for demonstrating the impact of outreach efforts. Throughout its history, the CGM has and continues to provide hands-on tours of facilities to K to 12 students (US primary and secondary school levels, typical ages 5-17). Over time, these outreach events have added structure by rotating attendees through discrete stations, each led by a volunteer geotechnical graduate student. After its establishment, the GGSS took over the organization of outreach activities at the CGM. The post-activity assessment of outreach events includes discussion of what worked and what did not after each tour, but does not include assessment of activity learning outcomes.

In 2015, the UC Davis geotechnical group began transitioning to a more strategic approach to educational outreach due to three factors: the start of the CBBG, the hiring of a department faculty member with expertise in assessment, and the creation of a GGSS outreach officer position. One program in development is a graduate-level engineering education course in which students design educational activities to be implemented in annual outreach activities performed by the GGSS.

5.1 *History of UC Davis geotechnical engineering outreach program*

Before the GGSS began, CGM faculty, staff, and students developed relationships with local secondary schools and invited them on tours of CGM facilities (Fig. 3). They developed a series of modules for participants to rotate through. Modules are tailored to the needs of the participants, and more formal presentations on geotechnical earthquake engineering can be included. The most successful modules include significant physical interaction, while a tour of the 9 m centrifuge can impress students simply with its scale.

Current modules include a shake table where participants build structures with K'nex, a create your own earthquake station where participants jump on

Figure 3. Kathleen Darby (centre) leading an outreach module during a tour by middle school students during one of her experiments on the 1m centrifuge. Jaclyn B. (peer/mentee) and Mohammad K. (mentor/visiting graduate student) also participated in this tour event.

an instrumented pad, a CGM module explaining the centrifuge and how it works, a CBBG module with bio-cemented sands, and a liquefaction module where users liquefy soil in a bucket to induce foundation failures.

With the creation of the GGSS, the students took over organizing the outreach events with the assistance of CGM staff. In 2014, the GGSS created an officer position for outreach coordinator. The result of these efforts was a time-efficient outreach system where new geotechnical students were trained on how to run different stations as they became involved in research. The participation in the activities provided opportunities for students to communicate technical topics to an audience with no or limited understanding of engineering. The direct interaction with K to 12 educators also exposes the graduate students to the curricular requirements of K to 12 education in the US.

The geotechnical group, however, did see a need for more intentional outreach that maximized impact without exhausting CGM staff and GGSS students. In 2015, the funding of the CBBG increased external demands for inclusive educational outreach and assessments of outreach efforts. This change coincided with the department hiring of a faculty member with an expertise in pedagogy and assessment.

Early steps have included the design of a two-course sequence for engineering graduate students in *Engineering Education Design (*discussed in section 5.4), intentional targeting of outreach activities to where they will have the most value, and developing tools for assessing outreach.

5.2 *Role of CGM*

As noted earlier, the CGM was the catalyst for early outreach efforts. Most connections with educators occurred organically. For example, one CGM development engineer, Tom Kohnke, initiated a now annual visit from a local high school where his daughter was attending. CGM personnel and students developed the first versions of the educational modules, and the facility attracted groups to UC Davis. The CGM currently support GGSS graduate students by providing access to the facility for tours and providing maintenance on outreach equipment (e.g., the shake table).

Table 7. Contributions to UC Davis geotechnical educational outreach.

Organization	Structure provided
CGM	Access to physical facility; institutional memory; technical support for demos
CBBG	Funded education-focused project; graduate course in engineering education; expectation of CBBG students to participate in two events per year
GGSS	Annual outreach coordinator; supply of volunteers
Department	Supporting tenure-track faculty hire in civil engineering education

5.3 *Connection to the LMM*

Aligning the educational outreach program with the LMM maintains the sustainability of the program and trains graduate students to communicate their research to non-technical audiences.

Providing a sustainable structure with clear expectations: The structure for the outreach efforts are provided by the three geotechnical organizations and the UC Davis Civil and Environmental Engineering Department (Table 7). One of the most important factors is the expectation that graduate students participate in educational outreach, which allows more outreach to occur than if it were performed only by centre personnel.

Tailoring mentoring to needs of the individual: As with other GGSS activities, the level of involvement in educational outreach activities is flexible. Students with minimal interest may only participate in a couple of outreach events each year and receive basic training from more experienced GGSS members. However, students with a strong interest in outreach or teaching may enrol in the graduate course sequence and serve as GGSS outreach coordinator. More active students will have multiple mentors coaching them, including both geotechnical engineering faculty and a faculty member with expertise in engineering education.

Leveraging resources generously: For outreach programs and associated mentoring interactions to be sustainable, they must leverage funding, equipment, space, time, and expertise. The CGM and CBBG both contribute funding related to outreach activities. The CGM primarily funds equipment maintenance, some supplies, and contributes staff effort. The CBBG funds workshops, undergraduate assistants to help design and organize outreach events, and new module development, and provides faculty support.

Expertise is leveraged in the design of modules and training of graduate students. Modules depend on the technical expertise of the geotechnical graduate students and faculty and the engineering education expertise of an environmental engineering faculty member. By finding someone with an educational design and assessment background, the geotechnical group can more efficiently train their students and assess the impact of their activities. The CGM provides expertise and support in maintaining the equipment used for outreach and providing a facility for on-campus outreach activities.

Both the CGM and CBBG are required to perform educational outreach and contribute to broadening participation of underrepresented groups in geotechnical engineering. By working together and with the GGSS and pooling resources, different types of expertise are exchanged and activities are more strategically designed with respect to time and impact.

Promoting an inclusive culture: All three organizations are committed to an inclusive culture, both for participants in the outreach activities and for the graduate students, staff, and faculty involved.

Outreach activities typically are targeted at populations underrepresented in engineering, including students who are female, from an underrepresented minority or ethnicity, from low-income families, have a disability, or who would be the first in their family to go to a four-year university or graduate school. Examples of inclusive actions include partnering with schools where many students come from low-socioeconomic backgrounds and a one-week sustainable engineering academy designed for girls entering grades seven to nine.

Outreach activities are an opportunity for students to see role models with similar backgrounds to their own, and to envision themselves in similar roles. For example, in California, where approximately 50% of elementary students are Hispanic or Latino, it is important that some of our participating graduate students are Hispanic or Latino. The diverse group of geotechnical graduate students allows students to find someone who shares some characteristics with them. Currently 75% of UC Davis CBBG graduate students are female and 25% are Hispanic or Latino. The US averages for civil engineering graduate students are 24% and 12%, respectively (National Science Foundation 2017). Additionally, some of our outreach activities highlight the impact of less-known female civil engineers (e.g., Emily Roebling) to provide historical role models.

Recognizing and valuing the different areas of expertise needed for effective outreach, graduate students receive mentoring from each other, faculty, and secondary teachers in how to integrate the culture of inclusion into their educational modules. Examples of inclusive designs include designing flexible lesson components or challenges that can be increased or decreased in complexity and incorporating best practices for inclusive teaching in both the design and implementation of the module.

Encouraging consistent assessment: As with research experiments, assessment and evaluation are necessary to understand the results and make improvements. Assessment data has been collected from outreach participants through observations, surveys, and engineering assignments. For example, some of the modules ask participants to answer questions before and after the activity to determine if the learning outcomes are reached. In addition to these methods, assessment of secondary teacher feedback was collected through discussions on specific modules and on overcoming barriers to productive collaborations between the university and secondary schools. Graduate students are assessed in the engineering education course through reflection assignments and the process they use to design their educational module.

Through the assessment process there have been numerous lessons learned. Evaluation based on assessment data from the Sustainable Engineering Academy for Girls led to a modified recruitment plan, increasing the ages targeted, changing the duration from four to five days, adjusting the target number of participants to 15, and modifying educational modules for future implementations. The increased target age group was observed to be appropriate as students had the fundamental math skills desired for some activities (e.g., a Life Cycle Assessment activity). The older students also had a larger attention span and were all highly interested in science.

The recruitment strategy, based on conversations with middle school teachers, was modified in 2017 to have teachers nominate students for participation. Students came from five different schools and three different grades. Students were more racially and ethnically diverse than in 2016; 33.3% of students in the 2017 cohort were from underrepresented minorities and two of the students had disabilities.

As graduate students work with faculty in the assessment phase, they are mentored in the iterative process that is required when designing instructional activities. Graduate students also learn of the great impact of non-technical factors on the success of educational activities (e.g., the length of student's attention span, emotional needs of students, and preparing for sometimes random remarks/questions from students).

Building networks that expand beyond the borders of the institution: Through CBBG partner institutions, best practices and lessons learned are exchanged with respect to outreach design and implementation. That network also allows for an expanded library of educational modules.

While the CGM already had a network of secondary teachers, the revised outreach program has expanded the network and provided the teachers agency. They now are mentors and mentees in the overall LMM of the geotechnical group. By providing interactions during the academic year, hopefully these relationships will be strengthened and sustained. One mechanism for maintain relationships with secondary teachers is through the development of K to 12 educational modules.

Table 8. Summary of LLM Core Principles integration into the UC Davis geotechnical program.

Principle	Implementation in UC Davis Geotechnical Group
Providing a sustainable structure with clear expectations	Each program has multiple structures that provide clear roles or expectations
Tailoring mentoring to needs of the individual	Flexible options for participation depending on interests and needs of individuals
Leveraging resources generously	Financial, time, space, and expertise resources are leveraged
Promoting an inclusive culture	Common focus on increasing access to broaden participation
Encouraging consistent assessment	Assessment occurs in all activities and is increasing in rigor with time
Building networks that expand beyond the borders of the institution	Partners include other academic institutions and personnel, industry partners, secondary education teachers, etc.

5.4 *Development of educational modules*

A two-course sequence in *Engineering Education Design* was designed for graduate engineering students to offer guidance in intentional engineering educational design. The first course introduces students to engineering education topics (e.g., student learning outcomes (SLOs) and assessment, types of learning and communication styles, active learning strategies, project-based learning, and creating inclusive environments).

In the second course, students design educational outreach modules related to their research that target specific age groups, align SLOs with state or national education standards, and include SLO assessment strategies. After developing a draft of their modules, students pilot their designs for a sample target audience. In the past, pilot events have included a public outreach event and a one-week engineering academy for secondary school girls. The course has been offered twice, with plans to offer it annually.

It is necessary that students designing educational modules for elementary and secondary school levels receive feedback from teachers at these levels, as they are most knowledgeable on what would work and what is most important to cover in their classrooms. To provide this input, secondary school teachers participated in a one-week summer workshop in 2016 and 2017.

The workshop format included one to two graduate students teaching their educational modules each day followed by discussion on those modules. Other workshop activities introduced participants to the topics of engineering, civil engineering, geotechnical engineering, sustainability, and underrepresented groups in engineering (especially women). At the end of each day, facilitated discussions with teachers led to: 1) developing strategies for integrating workshop activities and content into lesson plans, 2) strategizing methods for involving underrepresented groups in outreach activities, 3) identifying potential partnerships between UC Davis and local schools, and 4) obtaining feedback for graduate students on the modules they presented.

The workshops achieved three main outcomes: 1) graduate students increased teachers' confidence to teach engineering in their classrooms, 2) teachers provided practical feedback on the modules designed by the graduate students, and 3) partnerships between teachers and the geotechnical group were nurtured. After incorporating feedback from teachers, graduate students revised their modules for future implementations.

Current modules are in the iterative revision and testing phase familiar to most engineers. Although some students have graduated, the modules remain part of the GGSS/CBBG/CGM library of activities. When the final educational modules are complete, they will be submitted to TeachEngineering (https://www.teachengineering.org/), a web-based digital library of standards-based engineering K to 12 curricula.

After evaluating activities from the past two years, an adjustment has been made to encourage more continuous interactions with secondary teachers (e.g., student visits to UC Davis, teachers attending some of the classes in the improved graduate student course, visits to science classes in the teachers' schools, teachers providing direct feedback on modules during the graduate course). One improvement implemented is a Google Form created in which teachers can submit requests for borrowing outreach equipment, touring UC Davis facilities including the CGM, and having undergraduate and graduate students visit their classrooms. There have also been improvements to assessing outreach activities and their impacts. For example, an online outreach form that the GGSS outreach coordinator fills out after each event maintains a record of all information needed for reporting to NSF and observations about the activity's implementation (e.g., features that could be improved).

6 SUMMARY AND FUTURE WORK

The Ladder Mentoring Model presented herein has provided a formal structuring of mentoring and outreach activities toward producing researchers with the technical expertise, networks of collaborators, ability to communicate to all audiences, and other professional skills that can help them achieve their career goals. While the specific mechanisms vary, the three different programs described herein address the six core principles of our LMM (Table 8).

The results in Table 8 are an initial effort to characterize the LMM at UC Davis. However, CGM and CBBG researchers continue to investigate impacts and perceptions of the LMM through surveys and interviews of current and past geotechnical engineering graduate students. The goal of these studies is to evaluate how and why the LMM model has been successful at UC Davis. Factors under investigation include quantifying mentoring interactions, understanding graduate student participation in program activities, and student perception of mentoring activities.

Future work will include piloting the LMM framework beyond UC Davis. GGSS alumni are now in faculty positions at other universities and we are making plans with them to pilot programs featuring the core principles at their institutions.

ACKNOWLEDGEMENTS

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Physical Modelling in Geotechnics – McNamara et al. (Eds)

Geotechnical modelling for offshore renewables

C. Gaudin, C.D. O'Loughlin & B. Bienen
Centre for Offshore Foundation Systems, The University of Western Australia, Perth, Australia

ABSTRACT: Centrifuge modelling has been used extensively over the last five decades to address offshore geotechnical challenges associated with oil and gas developments. In recent years, the development of offshore renewable energy devices and structures, including wind turbines and wave energy converters has increasingly mobilised the offshore geotechnical engineering community. This paper revisits the use of centrifuge modelling for offshore geotechnics in the light of the new challenges raised by offshore renewable energy developments. This is illustrated through some aspects of foundation loading regimes such as dynamic tensile loading and multidirectional loading over a large number of cycles, which are specific to offshore renewable energy applications. The emphasis is on the modelling techniques developed to address these challenges and the opportunities provided by centrifuge modelling.

1 INTRODUCTION

1.1 *Historical background*

Centrifuge modelling for offshore geotechnics has historically been driven by the needs and requirements of the oil and gas industry. The large size of offshore infrastructure, the emphasis on failure in design and the complexity of loading regimes have been key challenges to address, for which centrifuge modelling is particularly well suited. The modelling undertaken, focused first on phenomenological and site-specific studies (see the first use of centrifuge modelling in offshore geotechnics in Manchester University in 1973 and reported in Rowe & Craig, 1981) and developed progressively towards more general investigations to better understand soil-structure interaction, observe failure mechanisms or provide performance data.

The development of centrifuge modelling for offshore geotechnics has been well documented through the years, notably by Murff (1996), Martin (2001) and Gaudin et al. (2010). They present in particular a comprehensive review of the use of centrifuge modelling that can be categorised as:

- *Identification/Observation*: to develop an initial understanding of the engineering concern, to identify a particular failure mechanism such that an appropriate analytical solution can be developed, or to observe a particular mode of soil behaviour (e.g. is the response drained or undrained, does the soil flow or collapse?).
- *Validation*: of a technical solution (type and geometry of structure) or of a mode of behaviour upon which the design was based (i.e. mode of collapse).
- *Generation*: of performance data that can be used to calibrate numerical models, generate design charts, or understand the relative importance of particular parameters in the global geotechnical response.

Acceptance and awareness in the offshore oil and gas community (both industry and academic) of the benefits of centrifuge modelling has grown significantly over the past two decades. This is partly due to scientific and technical developments associated with motion control, instrumentation and data acquisition that has enabled more realistic and sophisticated modelling, but also due to an increasing need for performance data and understanding of offshore soil structure interaction. This awareness appears however to be limited within the offshore renewable energy community, evident by the very limited body of literature of offshore renewable energy studies involving centrifuge modelling that are driven by industry.

This paper revisits the use of centrifuge modelling for offshore geotechnics in the light of the new challenges raised by offshore renewable energy developments. This is illustrated using examples of foundation loading that are specific to offshore renewable energy applications, including dynamic tensile loading, and multidirectional loading over extremely high numbers of loading cycles. The emphasis of the paper is on the modelling techniques developed to address these challenges and the opportunities provided by centrifuge modelling, notably with respect to the economical constraints that are faced by the renewable energy industry.

1.2 *The transition to renewables*

In an era of escalating energy demand and climate change, securing the supply of low-emission energy is one of the major challenges of our generation. The world's oceans offer a largely untapped resource, with enormous potential for energy solutions. The most rapid development in offshore renewable energy has been in offshore wind (+2.2 GW in 2016, for a total of 14.4 GW, +3.5 GW in 2017), with most of the installed offshore wind capacity in European waters (+1.57 GW across 7 windfarms, +813 MW in Germany alone). The worldwide wind capacity reached 486 GW by the end of 2016, with a growth rate of 11.8 % (WWEA 2017). In the majority of these offshore developments monopile foundations are favoured (constituting 97% of the foundations for wind turbines installed in 2015) due to the shallow water depth (<30 m). The developments went hand in hand with ever increasing pile diameters, with 8 m diameters now relatively common, constantly redefining the boundaries of what is possible.

As the industry evolves, offshore wind farms will be sited further from the coast and in deeper waters (>50 m), requiring floating facilities that are moored to the seabed with anchors. The advantages of floating systems are (i) better energy resources can be tapped due to winds becoming higher and more consistent with distance offshore, (ii) larger wind turbines (8-10 MW) can be installed, thus increasing energy production, and (iii) maintenance costs can be potentially reduced, as turbines can be untethered and towed to shore. Similar trends are forecast for wave energy converters (WECs). The industry will need to transition from single or small-array demonstrator units (of moderate scale and power capacity) towards integrated arrays of larger, full-scale devices to realise commercially viable energy generation. This introduces a need to design multiple closely spaced foundations in water depths able to accommodate the typically larger draft of full scale WECs (up to depths of ~100 m).

Renewable energy generation from floating systems has been proven, and includes for example: (i) the Hywind spar floating wind turbine, which has been in operation in 198 m of water off the southwest coast of Norway since 2009, (ii) the WindFloat semi-submersible floating wind turbine that has been tested in 40-50 m water depths off the coast of Portugal since 2011, (iii) the Ocean Power Technology floating wave energy device, which has been tested off the coasts of Hawaii, USA and Scotland in water depths of up to 30 m since 2005, and (iv) the Perth Wave Energy Project from Carnegie Wave Energy, with three 240 kW WECs operating over 12 months offshore Garden Island in Western Australia in 2015. These small projects have aimed to demonstrate concept feasibility such that commercial developments can be expected in the coming decades.

1.3 *The economic constraints*

Previous offshore wind farm developments were subsidised, but now need to prove themselves to be competitive with other energy sources. This is indeed the case, with the cost of offshore wind power reported to be lower than that of nuclear power in the UK (BBC 2017). As these wind farms are large, with perhaps 200 turbines, even small improvements in design translate to large economic savings. Similarly, offshore floating renewables require reliable and economical anchoring systems that can perform in the type of seabed sediments encountered on the continental shelf, where floating renewables are expected to operate. Anchoring systems can contribute up to 22% of the total installed cost of an offshore wind turbine (Willow & Valpy 2011), and up to 30% of the total installed cost of a wave energy converter (Martinelli et al. 2012). This is one order of magnitude higher than for oil and gas structures (Kost et al. 2013), and contributes significantly to the high levelised cost of offshore wind and wave energy. The US Energy Information Administration forecasts a levelised cost of offshore wind energy of US$158/MWh (US$64.5/MWh for onshore wind) for plant entering service in 2022 (Energy Information Administration, 2017), while the cost of natural gas ranges from US$56/MWh to US$105/MWh. Without large commercial scale installations, the cost of wave energy is harder to forecast and varies significantly between the various types of converters (and capital cost) and between forecasters. For an array of 100 point absorbers, the cost has been estimated at around US$800/MWh (Neary et al. 2014), although a case study using an oscillating water column offshore Portugal estimated a cost as low as US$86/MWh (Castro-Santos et al. 2015).

A large volume of research is being undertaken to improve the efficiency of wind turbines and WECs, but considering the significant fraction of the capital cost they represent, savings in foundation engineering could potentially have a significant impact on the LCOE, provided a step wise improvement in technology and design is achieved (Gaudin et al. 2017).

Foundation design for offshore renewables is currently based on the knowledge and technology developed for and by the oil and gas industry over the last 50 years. This is reflected in the large number of rules and guidelines applicable to floating renewables, which overwhelmingly refer directly to oil and gas guidelines, such as API (2008) for mooring analysis for station keeping and API (2014) for foundation design. Neither guideline suggests adaptation for renewable energy.

A number of publications have emerged over the last few years (Stevenson et al. 2015; Knappet et al. 2015; Diaz et al. 2016), listing the existing anchoring solutions and design methodologies that would be suitable for floating renewables, highlighting particularities that require further investigation. Recently, an additional body of research has started to focus on the specific aspects of offshore renewables that are fundamentally different than oil and gas, such as changing loading direction (Rudolph et al. 2014) or ratcheting behaviour under very large number of cyclic loads (Houlsby et al. 2017). Foundation alternatives that could provide significant cost savings also start to be considered such as helical piles (Byrne & Houlsby 2015), or active suction caissons (Fiumana et al. 2018). These new developments will be required to generate the step change cost reduction in foundation engineering that is required to make offshore renewables economically viable at commercial scale.

Centrifuge modelling can play a significant role in assisting these developments, similar to how it has enabled some of the more significant advances in offshore geotechnics for oil and gas applications. New geotechnical challenges associated with offshore renewable energy applications often requires the development of new modelling techniques. This paper presents a snapshot of some of these developments, associated with:

1. Modelling dynamic tensile loading resulting from extreme storm loading on a floating wave energy converter.
2. Modelling very large number of loading cycles and multidirectional loading.

2 MODELLING DYNAMIC TENSILE LOADING

2.1 *Motivation*

Point absorbers are a category of wave energy converters that produce electricity by using the foundation as a reaction point. They are designed to resonate at the peak frequency of the energy in the wave spectra to ensure optimum power take-off (PTO), resulting in in magnitude tensile loads on the foundation, that are of the order of several MN under extreme (e.g. storm) loading. An approach to limit the loads on the foundation involves using a Coulomb-Damping PTO that caps the load to a maximum value, above which the PTO experiences continuous extension. The drawback of such an approach is the snatch load that occurs if the PTO reaches its maximum extension. Hydrodynamic analyses have demonstrated that this snatch load is dynamic in nature, such that the foundation experiences acceleration. Hence, an opportunity arises to design the foundation to satisfy two different loading states. The first is an operational loading state, where the foundation is designed to withstand the cyclic tensile loads due to movement of the WEC at the water line that translates (via the PTO) to loading on the foundation. Although this continuous tensile cyclic loading on foundations is relatively uncommon, and raises questions associated with drainage response (for coarse-grained seabeds) and the potential for ratcheting behaviour (e.g. Houlsby et al. 2017), from a modelling perspective it can be addressed using existing techniques. The second loading state is the extreme condition, associated with maximum extension of the PTO. This will result in a high magnitude but short duration tensile load on the foundation, such that geotechnical response is not only expected to be undrained, but may also include strain rate and inertia effects. Centrifuge tests can assist in identifying and quantifying these capacity components, although modelling such a loading event requires new modelling techniques.

2.2 *Apparatus and preliminary results*

An experimental arrangement designed to model the extreme loading condition described above is shown in

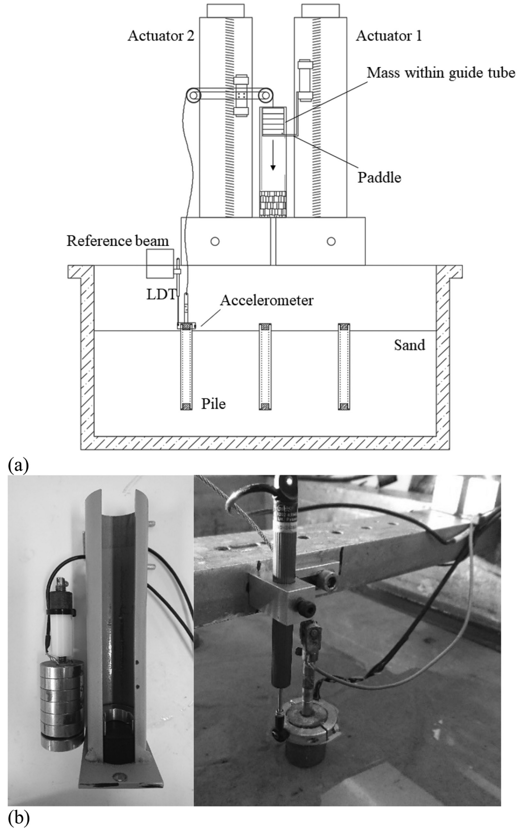


Figure 1. Experimental arrangement: (a) schematic representation, (b) falling mass and guide (left), instrumentation at the pile head (right).

Figure 1. Rather than using actuation systems to load the foundation, the foundation is loaded by allowing a mass to fall over a short distance in the elevated acceleration field of the centrifuge to generate a very short duration, high magnitude load.

Referring to Figure 1, a pile foundation is connected to a mass using a steel wire via two pulleys such that the dynamic tensile load is applied vertically to the pile. The mass falls within a slotted guide tube, with rubber foam at the base to absorb the impact if the tension in the steel wire does not arrest the fall. The mass is initially held in position using the 'paddle' located on the vertical axis of an actuator, where the slot in the guide tube allows access for the paddle. The pulley assembly is located on the vertical axis of a second actuator, which is adjusted to control the tension in the steel wire before the test. A load-cell, connected in series with the steel wire, is located just above the pile head, and a linear displacement transducer (LDT), located on an independent reference beam measures the pile head displacement. Two Microelectromechanical systems (MEMS) accelerometers, one on the falling mass and one on the pile, allow the acceleration and (through integration) the velocity of the mass and pile to be established.

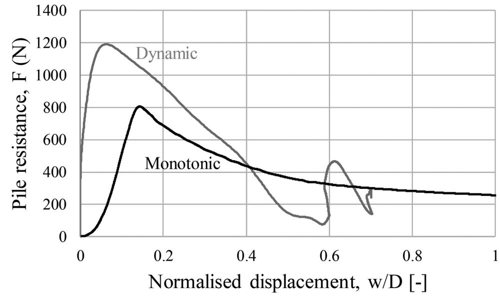
The above experimental arrangement was adopted for tests conducted in dry and saturated silica sand conducted in the 1.8 m radius fixed beam centrifuge at The University of Western Australia. Details of the testing are due to be reported elsewhere, with snapshots presented here to illustrate some of the highlights from the tests.

A typical test programme using the falling mass loading system involves a monotonic test to establish the drained tensile capacity of the pile, followed by a series of dynamic tests to explore the pile response when loaded beyond the drained tensile capacity. Monotonic tests involve using the vertical axis of an actuator to displace the pile vertically slowly (via the steel loading wire) until a peak anchor capacity is measured. The dynamic tests would then select a mass such that the weight is a percentage of the drained capacity, but would result in a dynamic load that is considerably higher than the drained capacity. Each dynamic pile test could involve a single or multiple dynamic load events, depending on the pile response. Consecutive dynamic loads can be applied without stopping the centrifuge, by manipulating the paddle on the second actuator to raise and hold the mass, whilst raising the pulley assembly to control the initial tension in the steel wire.

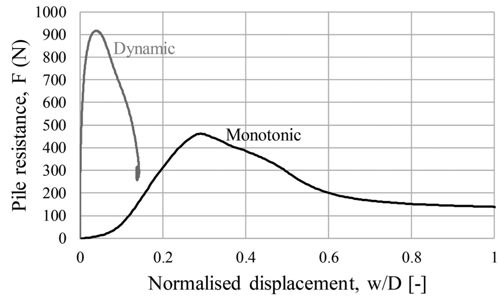
Example test results are provided in Figure 2a and 2b for dry and saturated sand respectively. In each instance the weight of the falling mass (at the initial drop elevation) was approximately 50% of the drained tensile capacity. Figure 2a shows that in dry sand the dynamic pile capacity is approximately 50% higher than the drained monotonic capacity, and that the response in the dynamic test is much stiffer. As the sample is not saturated the additional resistance cannot be due to drainage, but must reflect an inertial component of resistance.

Figure 2b shows an equivalent comparison between monotonic and dynamic responses for a saturated sample (at the same relative density). In this instance the dynamic pile capacity is almost double the monotonic capacity, noting also that the monotonic capacity is lower than in the dry sample, reflecting the lower effective stress level in the saturated sample. As with the test in dry sand the pile response to dynamic loading is much stiffer than to monotonic loading, such that the pile displacements associated with these snatch loading events can be expected to be sufficiently low that the pile has sufficient residual capacity for additional operational or extreme loading events. The much higher ratio of dynamic to monotonic capacity for the saturated sample is due to the undrained response in the sand. This is to be expected, as the pile velocity reaches a maximum velocity, $v = 5$ m/s, such that the strain rate – approximated here as v/D, where D is the pile diameter – is 227 s^{-1}.

Returning to the test result from the dry sample, Figure 3 shows that the difference between the monotonic and dynamic resistance is close to the inertial resistance, calculated as the sum of the measured pile acceleration and the pile mass. This result suggests



(a)



(b)

Figure 2. Example results from monotonic and dynamic tensile loading tests on a pile in sand: (a) dry sand, (b) saturated sand.

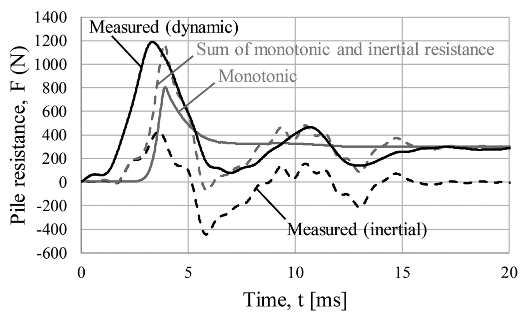


Figure 3. Interpretation of a dynamic tensile pile test in dry sand.

that for dry sand there are no other components of pile capacity, such as the pile impedance considered in pile driving. Consideration of the time duration of the dynamic load in these tests (∼10 ms) relative to the likely time taken for a stress wave to travel along the pile and back (∼0.06 s), supports this conclusion. An extension of the logic used in the interpretation of the tests in dry sand is that the dynamic resistance in saturated conditions is the sum of the undrained resistance plus an inertial component that is simply the product of the pile mass and acceleration.

These example results not only show that a pile in sand is capable of withstanding a short duration dynamic load, of a magnitude that is considerably in excess of the monotonic capacity, but also reveal how relatively simple measurements and permutations of test conditions reveal the components of capacity that are generated during dynamic loading, allowing for the development of appropriate prediction tools.

3 MODELLING CYCLIC MULTIDIRECTIONAL LOADING

3.1 *High numbers of loading cycles*

While design for oil and gas structures typically involves consideration of the order of 10^3 loading cycles, the design of an offshore wind turbine generally requires the consideration of 10^6 to 10^7 load cycles undertaken a high frequency. This is not only to fulfil similitude requirements in terms of loading frequencies in the field and resulting drainage regimes, but also to minimise testing time while maximising the number of cycles. The drainage regime is important (Zhu 2018; Bienen et al. 2018) and can be controlled through pore fluid viscosity.

While it may not be economically feasible, nor indeed necessary, to perform long-term cyclic loading tests entirely in the centrifuge, it has been shown to be important to capture effects including installation history, stress level and drainage regime on the initial response under cyclic loading (Zhu 2018). This allows quantification of the initial rotation at field scale, which, when considered holistically with data from single gravity tests providing the accumulation trend over large numbers of load cycles enables evaluation of the long-term full-scale foundation response. An example from Zhu (2018) is provided in Figure 4 for monopod suction buckets in sand. The rate of accumulation in the single gravity and centrifuge tests is the same, although the low stress level in the single gravity tests results in a higher and incorrect initial rotation. Both the initial rotation and the rate of rotation accumulation with cycle number are important for the design of these dynamically sensitive structures, with strict limitations on non-verticality strictly enforced (DNV 2016).

3.2 *Accuracy and resolution of displacement measurements*

Offshore wind turbines are very sensitive to out-of-verticality, to the extent that rotation is typically limited to 0.5° over the design life of the structure (DNV 2016). This implies that in investigations of the foundation performance for offshore wind turbines, the expected displacements are very small. Further, precise knowledge of foundation stiffness is important as this affects the system stiffness, which is typically designed to fall in the narrow range between the blade and rotor forcing frequencies so as to avoid excitation of resonance. Experimental investigation therefore requires high accuracy and resolution of displacement measurements. Figure 5 illustrates the effects of average vertical stress and drainage regime (achieved through the use of different viscosity pore

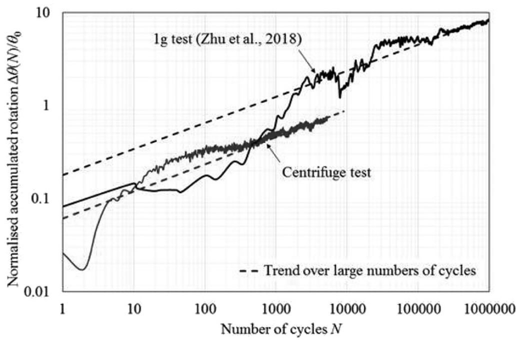


Figure 4. A combined approach of single gravity and centrifuge tests to predict the response of foundations subjected to large numbers of loading cycles (Zhu 2018).

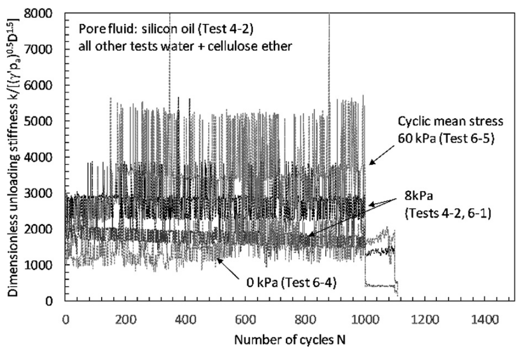


Figure 5. Unloading stiffnesses of suction buckets under vertical cyclic loading into tension (Bienen et al. 2018).

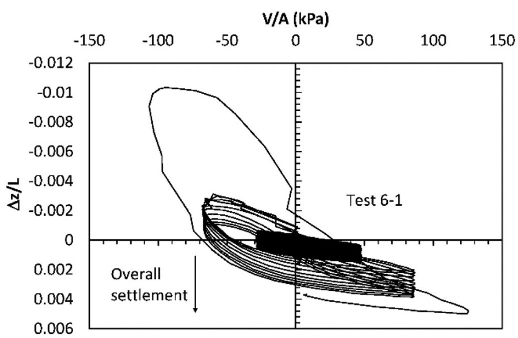


Figure 6. Displacement of suction bucket under vertical cyclic loading into tension (Bienen et al. 2018).

fluids) on the unloading stiffness of suction buckets in dense sand under vertical cyclic loading. While the displacement measurements (shown as an example for Test 6-1 in Fig. 5) are captured well, the deduced unloading stiffness (Fig. 6) shows some scatter, which is introduced by the division by very small differences in displacement.

Where investigations focus on foundation response under lateral cyclic loading, the displacement measurement technique is ideally non-contact as any

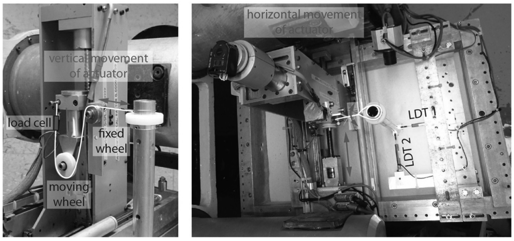


Figure 7. Centrifuge experimental arrangement that enables cyclic lateral loading with changing directionality (Rudolph et al. 2014).

resistance of the sensors may impact the measured displacements, in particular under low loading magnitudes.

3.3 *Multidirectional loading*

Loading of an offshore renewable energy device is expected to vary in cyclic loading magnitude, load eccentricity and even directionality. The latter has different origins, depending on the type of renewable energy installation. For offshore wind turbines, this relates closely to changes in metocean conditions over the design life of the structure.

The experimental apparatus hence needs to be sufficiently flexible to accommodate changes in loading characteristics, with an example shown in Figure 7 that enables cyclic lateral loading with changing directionality to be applied at an eccentricity above the soil. This is achieved by fixing a wheel at the top of the pile within which the pile can move freely in the vertical direction. A wire is connected to the wheel and to the vertical axis of an actuator. The vertical axis of the actuator is used to apply the load on the pile via the wire, while the horizontal axis modifies the direction of the application of the loading. The system enables either displacement or load control to be applied, while a set of two LDT sensors connected to the pile and separated by an angle of 90° provides information of the displacement history of the pile in the horizontal plane. The change in loading direction was found to significantly increase the monopile displacements (Rudolph et al. 2014), rendering consideration of uni-directional loading un-conservative.

Floating renewables are also subjected to multidirectional loading, but of a different type. One strategy currently considered for floating renewables (either wave energy converters of floating wind turbines) to significantly reduce the foundation costs involves sharing foundations across multiple devices assembled in an array (Karimirad et al. 2014). Different pattern of arrays can be considered as a function of the power output and shadowing effects between devices (see Child & Venugopal 2010), but in all cases, the number of foundations can be reduced significantly. For instance, for an array of 13 point absorbers each with three mooring lines, the honeycomb array pattern (Fig. 8a), allows the 13 devices to be anchored by 20 foundations (instead of 39).

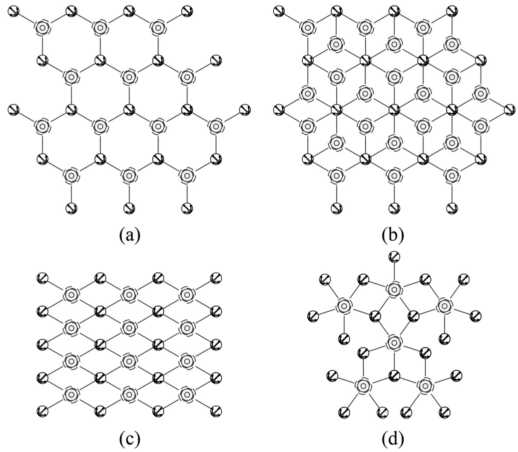


Figure 8. Array of wave energy converters (after Herduin et al. 2018).

A direct consequence of the foundation sharing strategy is the complexity of the loading regime on the foundation that can come from 2, 3 or 4 different directions (see Fig. 8). Depending on the wave spectra and period, and the spacing between the floating devices, the foundation can be subjected to loads coming from different directions that can be in phase (resulting in alternate loading along each of the mooring line) or out of phase (Herduin et al. 2018). This complex multi-directional loading mode, is fundamentally different from typical design considerations for floating oil and gas infrastructure, on which most of the design methodologies are based.

A first step in investigating the performance of foundations under multidirectional loading is to accurately and comprehensively define the load distribution and history acting on the foundation.

An initial development to define this load distribution and history (Herduin et al. 2016) used an analytical framework to establish the characteristics of the resultant load from multiple mooring lines, as a function of the individual load characteristics. The purpose is to characterise the variation of load magnitude and direction for a given array configuration, in order to define potential best and worst case loading scenarios from a foundation design perspective. Fig. 9 presents an example result that assumes an array of floating bodies assembled in a hexagonal configuration in constant water depth and subject to regular waves coming from a single direction. The wave series produces three harmonic loads of equal period and amplitude on the anchor from directions separated by 120°. Fig. 9 illustrates the variety of loading configurations applied to the anchor. As the phase difference θ between the three loads varies from 0 to π, resultant contours transform from a thin ellipse suggesting nearly bi-directional loading to a circle centred at the origin indicating constant amplitude and large variation in direction. This variety is further exacerbated when consideration is given to irregular waves and wave direction

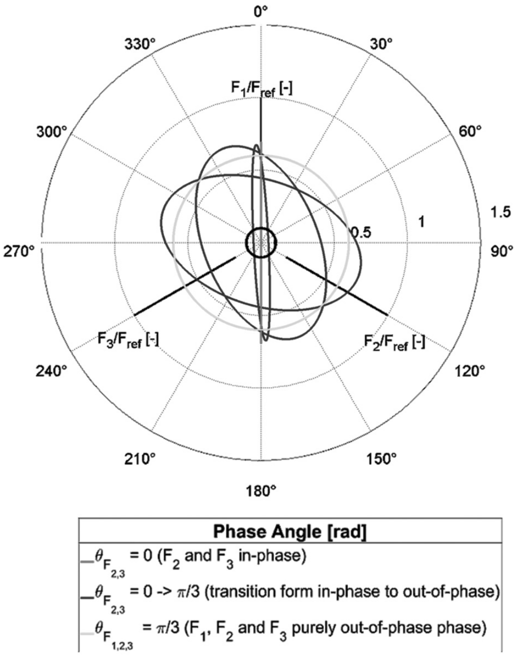


Figure 9. Variation of magnitude and direction of the load resultant from three harmonic loads of phase (after Gaudin et al. 2017).

with respect to the array configuration (Herduin et al. 2018). Fig. 9 is important from a geotechnical design perspective, considering that it is uncertain whether a resultant load of high magnitude and limited variation of direction is more detrimental than a resultant load of low magnitude varying over a wide range of directions. It is also important from a modelling perspective to define the requirements for a testing setup.

Indeed, the performance of traditional anchoring systems such as piles, skirted circular foundations or plate anchors, under multi-directional cyclic loading is poorly understood, but starts to receive attention. A few studies on multi-directional loading of offshore foundations suggested that a change in loading direction can increase plastic displacements and can reduce foundation performance as discussed in the previous section (Rudolph et al. 2014).

More recently, preliminary tests have been undertaken to better understand the performance of foundations under multidirectional alternate loading. Tests were performed on a suction caisson in sand with a setup that enabled two mooring lines separated by a planar angle ranging from 60° to 120° to be loaded alternatively (Fig. 10). The caisson was first installed at 1g (assuming wished in place conditions for this more fundamental study), and the two mooring lines were connected to two independent actuators that could apply either controlled displacement or loads to mimic any multi-directional loading scenario. Displacements are measured along the loading direction via encoders on the actuator, although this technique

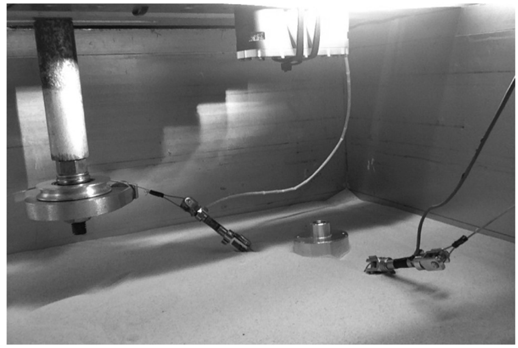


Figure 10. Multi-directional loading setup (after Herduin et al. 2016).

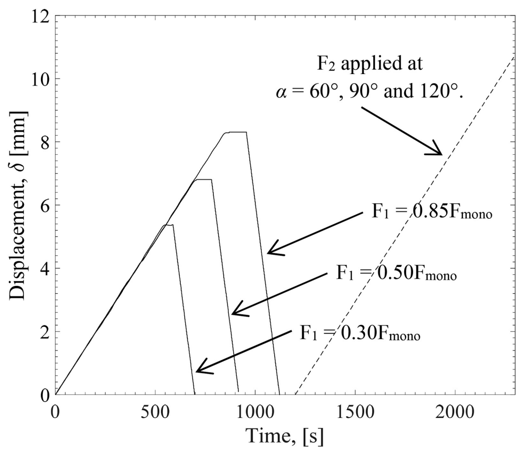


Figure 11. Multi-directional loading regime (after Herduin et al. 2018).

does not capture the whole displacement and rotation of the caisson in the six degrees of freedom. Further refinement, using PIV techniques have enabled measurements of the vertical horizontal and rotational displacements in one vertical plane (Gomez-Battista 2017) and developments are currently undertaken to expand the technique to the whole 6 degrees of freedom.

The loading regime applied in these preliminary tests is presented in Figure 11. A load at a fraction of the monotonic ultimate capacity was first applied in direction 1, and subsequently released. Immediately after a load was applied to failure along a direction 2, which is separated from direction 1 by a planar angle 30°, 90° or 120°.

Results are summarised in Figure 12 for initial loading of 30%, 50% and 85% of the monotonic capacity. They indicate a reduction in capacity of up to 10% for an initial loading higher than 50% of the maximum capacity and a loading direction of 120°. While this reduction may seem limited at first glance, it should be noted that this reduction occurs after one single episode of loading and that such a loading regime will be repeated multiple times over the life of the structure.

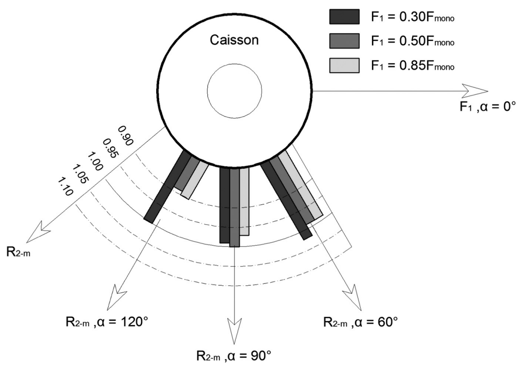


Figure 12. Change in caisson capacity under multi-directional loading (after Herduin et al. 2016).

The experimental approach is currently being improved to allow loading to be applied in three directions to mimic complex loading regimes with cycle numbers exceeding 10^5. In parallel, a macro-element model, capable of evaluating the change in capacity of the foundation under cyclic multi-directional loading, is being developed (see Gaudin et al. 2017). In this particular case, centrifuge modelling is used to provide insights into the behaviour of the foundation under multi-directional loading and to provide performance data to calibrate a theoretical model.

4 CONCLUSIONS

The paper presents a brief overview of some of the challenges faced by the offshore renewables community when designing foundations for fixed or floating structures. Because of the nature of these structures, the challenges are different than those faced by the oil and gas community over the last five decades and some of the standards and guidelines commonly used in design must be revisited.

Centrifuge modelling has an important role to play in addressing these challenges. Interestingly, the aims and objectives of centrifuge modelling remain identical; providing performance data, validating new concepts, identifying mechanisms, etc., but new modelling techniques are required to model new loading regimes, installation processes and serviceability constraints that are specific to offshore renewable energy applications. A few examples are provided, highlighting technological constraints and requirements and a snapshot of results demonstrate that centrifuge modelling is well positioned to assist the development of offshore renewables.

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Physical modelling applied to infrastructure development

R.J. Goodey
City, University of London, London, UK

ABSTRACT: The physical modelling conference series has served as a primary means of sharing practise and disseminating current research in experimental geotechnics. Each conference highlights the trends, techniques and direction of current research. This paper summarises contributions to the 9th International Conference on Physical Modelling in Geotechnics from researchers broadly in the field of infrastructure development. This themed paper aims to identify innovative approaches to geotechnical problems, advances in experimental techniques and equipment in order to address new research questions and future trends in infrastructure research that might feature more significantly in future conferences. Some reflection on past conference proceedings is included with the hope that the community appreciates the scale of our achievements since the first conference in the series.

1 INTRODUCTION

Now in its ninth iteration, the International Conference on Physical Modelling in Geotechnics is the pre-eminent forum for the dissemination of research in all areas related to experimental geotechnics. From the early days, the conference series has grown and matured and this is reflected in the contributions submitted from researchers which have increased both in number as well as in complexity of topics addressed and range of techniques adopted.

The aim of this paper and the accompanying lecture is to highlight some of the contributions and advances made in the area of infrastructure development. Within this field there are many areas of interest and this is reflected in the high number of papers submitted to the conference. These papers detail work carried out using a wide variety of experimental techniques including large scale testing, centrifuge modelling and comparisons with field data.

The organisation of this paper follows the broad theme of the papers with the aim of identifying advances in the field as well as highlighting areas of future interest.

2 URBAN DEVELOPMENT

In urban areas there is a high demand to maximise available land and other resources. This has led to taller buildings with larger foundations and deeper basements. These types of structure can be difficult to construct in urban environments, there can be issues of noise during construction, interaction with existing infrastructure, as well as the need to ensure protection from earthquakes and other natural events. There are also problems of increasing subsurface congestion on new construction i.e. as buildings are redeveloped there are existing piles to consider as well as the need to avoid damage to existing infrastructure.

2.1 *Driven piles*

Conventional installation methods for driven piles (i.e. impact or vibration driving) cause undesirable noise and vibration in urban environments. One solution investigated by El Haffar et al. (2018) and Frick et al. (2018) is to use rotary jacked piles whose installation is much lower in both noise and vibration. These studies use coarse grained soils and investigate the influence of the installation parameters (forces and jacking stroke). Both studies produce broadly similar conclusions in that the installation forces and final capacity of the piles are strongly linked to the installation method adopted.

2.2 *Deep basements*

Deng & Haigh (2018) and Chan & Madabhushi (2018) both present preliminary work relating to deep basements. These papers investigate more efficient basement design (recognising that urban development now routinely incorporates deep basements) and both studies aim to investigate the underlying mechanisms. Deng & Haigh (2018) describe experimental work on soil movements behind a retaining wall. In this work, the wall movements are controlled and DIC (Digital Image Correlation, e.g. White et al. 2003) is used to monitor the soil response. This approach has been adopted as a more fundamental investigation of movements around excavations when compared with existing guidance (e.g. Clough & O'Rourke 1990) which is often based on empirical data and may not be universally applicable. Chan & Madabhushi (2018) also present work under development but here focussing on the heave behaviour of basement slabs founded on overconsolidated clay. The aim of the work is to study the influence of slab and basement stiffness on heave whereas previous work has focussed on specific cases or mitigation techniques. The results presented, whilst at an early stage, highlight not only the potential outcomes of the project, but also the complexity of the centrifuge modelling being carried out currently and the difficulties encountered.

3 ROADS AND PAVEMENTS

In the area of transportation infrastructure it is interesting to note that the majority of papers submitted to the conference are concerned with maintenance and prediction of long term performance. This is understandable given that many countries have well developed transport systems, elements of which may have originally been constructed more than a century ago.

3.1 *Pipelines buried beneath roads*

For ease of installation and maintenance, utility pipes are often buried beneath roads. This results in shallow pipelines which are subjected to significant cyclic loads from above. Bayton et al. (2018) report centrifuge tests of model pipelines subjected to simulated traffic loads. The motivation for the study is to minimise leakage from pipe networks with water supplies being highlighted. The work concentrates on the accumulation of bending moment within the pipe with repeated load cycles which could eventually result in damage to the pipe causing subsequent leakage. The effects of that leakage in the form of development of sinkholes are investigated in other papers (Kuwano et al. 2018, Indiketiya et al. 2018, Kearsley et al. 2018). Both Kuwano et al. (2018) and Indiketiya et al. (2018) use 1g testing and DIC to investigate the formation and propagation of a void above a simulated pipe with a defect. The experiments simulate the situation where soil is washed into the pipeline via the defect. It is interesting to note that in all cases very little movement is observed at the ground surface before the cavity collapses. The experimental arrangements in these papers are similar and in all cases soil below the water table is more prone to development of a cavity (as it is washed into the pipe via the defect). Indiketiya et al. (2018) conclude that cavity development is a function of the soil size when compared with the pipe defect but that it is difficult to identify a relationship between volume of soil lost and the size of the defect in the pipe. Kuwano et al. (2018) draw a similar conclusion with respect to the ratio of defect versus soil grain size but, due to the pipe defect having a fixed size in this study, do not comment on this aspect.

Laporte et al. (2018) report the development of apparatus to investigate the effect of wetting and drying cycles on expansive soils. Differential soil displacements can result in cracked pavements, damage to buried pipes and foundation movements. The experimental arrangement incorporates a model pavement and associated drainage ditch. This work shows significant differential movements between areas exposed directly to the elements and those shielded by the pavement surface. Again the work presented highlights the level of technical complexity that is being achieved in centrifuge modelling as well as the precise measurements that can be made via conventional instrumentation and DIC.

3.2 *Pavement design*

Two papers (Smit et al. 2018a,b) detail experimental investigations of Ultra-Thin Continuously Reinforced Concrete Pavement. This is proposed as a cost effective alternative to traditionally designed pavements and comprises a thin layer of heavily reinforced concrete. These papers highlight the problems associated with applying conventional design approaches to this innovative pavement design. The authors rightly highlight differences in design approach that would need to be accounted for given the observed difference in behaviour between this and traditional pavements.

4 PILES AND PILED FOUNDATIONS

Piled foundations are utilised in a wide range of applications; as foundations for medium to large size structures (both in isolation and as part of a piled raft), reinforcement under embankments or existing structures, to form walls and, more recently, as energy piles. This wide range of applications is reflected in the significant number of papers relating to pile performance. There is a particular focus on seismic and cyclic performance with the majority of the studies using centrifuge modelling techniques although 1g and shaking table tests are also utilised.

4.1 *Piles under seismic action*

A number of papers investigate the performance of piles under seismic loading. The experiments consider single piles (Yao et al. 2018, Ebeido et al. 2018, Chen et al. 2018, Pérez-Herreros et al. 2018) or small pile groups (Imamura 2018, Egawa et al. 2018). One paper (Garala & Madabhushi 2018) considered a comparison between a single pile and a small pile group (containing three piles). A range of soil conditions are used although sands and coarse grained soils are predominant. The majority of the papers investigate pile response in terms of bending moments and pile movements.

Many studies apply seismic loading either utilising input motion recorded during real earthquakes (e.g. Pérez-Herreros et al. 2018) or idealised sinusoidal motion, however Yao et al. (2018) have created an experimental apparatus to directly model the movement of a fault and used this to investigate the effect this has on piles close to and within the fault zone. This work also uses laser displacement transducers to measure the movement of the pile head and presents a good comparison between these measurements and those obtained from DIC. Significant bending moments and movements are observed in all piles, even those some distance from the fault zone. The authors conclude that, as in the real case, piles within the fault zone would most likely be completely sheared.

Egawa et al. (2018) present a series of tests on piles within layered soil models. The soils used are sand and volcanic ash and a variety of arrangements were tested (with respect to number and thickness of each layer). Despite the variation in test arrangements it was shown that large bending moments were consistently produced in the piles at around mid-height. It would be interesting to investigate whether this observation was repeated with different arrangements or sizes of pile. Pérez-Herreros et al. (2018) also use layered soil samples in their experiments on pile response, in this case the model is overconsolidated clay overlying dense sand. The model pile is predominantly embedded within the clay with its base just penetrating the sand layer beneath. Of great interest in this work is the observation that bending moments in the pile are strongly influenced by the amount of embedment into the dense sand layer. This obviously has great significance in areas where soft soils overly sands and end-bearing piles might be used.

4.2 *Static behaviour of piles and piled foundations*

Bisht et al. (2018) and Rodríguez et al. (2018) both present work investigating the performance of piled raft foundations. Bisht et al. (2018) show how the total load capacity is affected by the arrangement of piles the lengths of each pile within the group. A better understanding of how piles within the raft interact and contribute to the overall capacity could lead to more efficient designs. Rodríguez et al. (2018) concentrate on how the piled raft performs during changes of pore water pressures. These changes could arise from consolidation processes or by pumping from deep aquifers which is common in many cities. The authors present their results in terms of proportion of load carried by the raft or the piles. As pore water pressures decrease the proportion of load carried by the piles increases significantly, accompanied by a separation of the soil from the underside of the raft. This would have an impact on the design of both the piles and the raft although, as Bisht et al. (2018) point out, the contribution of the raft is often ignored in conventional design even though that implies poor economy.

Panchal et al. (2018) present a small study on a hybrid foundation system combining sheet piles with a pile cap. The aim is to produce a sustainable foundation design that can be used in already heavily developed urban areas. This type of foundation could be easily removed and recycled in the future which is not possible when dealing with bored, cast in-situ piles that are often found during site redevelopment. The results show a strong influence of geometry on the load capacity but that easily constructed square sheet pile groups could be a viable alternative to traditional bored piles.

4.3 *Energy piles*

Two papers highlight how physical modelling can be applied to new and emerging technologies. Energy piles combine structural requirements with the thermal performance of a ground source heat pump. The challenge is to assess the influence of the temperature changes on the structural performance. In a clay soil temperature changes will generally result in pore pressure variations due to the low permeability of the clay. The resulting change in effective stress is presumed to affect the interaction between soil and pile. Parchment & Shepley (2018) present a fundamental study of the influence of temperature on a soil-structure interface. A large number of direct shear tests between clay and a structural element (an aluminium block) are carried out. The study concludes that, for the rnage of temperatures that might be expected in a thermal pile, there is little effect in overconsolidated clay. Any effects seen relate to adhesion between pile and soil and may, in fact, be structurally beneficial. In normally consolidated clays the effect of heating is negligible.

Ghaaowd et al. (2018) present work on how heating affects the properties (undrained strength) of a clay sample and the resulting effect on pullout strength of a heated versus unheated pile. Significant increases in the pullout capacity are observed after heating. Only one (extended) cycle of heat was applied and it would be interesting to investigate how cycles of the type that would be expected in a thermal pile influenced the soil and pile behaviour.

5 SLOPES AND EMBANKMENTS

Slopes (both engineered and natural) present potential hazards primarily related to their long term performance and stability. A significant number of the papers submitted in this area deal with assessing and predicting the response of a slope to changes in pore water. The slopes studied are generally clay or clay dominated.

5.1 *Slopes subjected to wetting and drying*

Slopes and embankments are generally subjected to cyclic variation of wetting and drying. This could be due to seasonal variation, tidal variation or changes in reservoir level amongst other phenomena. Ahmed et al. (2018) present work investigating the movement of a slope subject to cyclic variation representative of tidal cycles. Similarly, Luo & Zhang (2018) simulated more extreme variations of wetting and drying more representative of the changing levels in a reservoir. Both of these studies adopted a similar, centrifuge based, approach and used DIC to monitor the movements. Movements are shown to accumulate with increasing numbers of cycles which obviously has implications for the long term stability of the slope. Ahmed et al. (2018) also carried out in-flight measurements of the soil strength within the slope and demonstrated that strength changed quite significantly with only a relatively small number of cycles. Again, this has implications for the long term stability of the slope.

Variations in water content can also be affected by vegetation on the slope. Vegetation is often cleared from embankments near roads and railway lines in order to reduce the potential for accidents and disruption. The effect of vegetation removal is investigated by Kamchoom & Leung (2018). These effects are twofold; firstly, removal of the vegetation would halt transpiration, potentially increasing pore water pressures in the embankment and secondly, the live roots act like reinforcement, the effectiveness of which will be reduced as the root decays. Kamchoom & Leung (2018) create a centrifuge model of a slope with artificial roots connected to a vacuum system. In this way, plant transpiration can be simulated and, by control of the suction from the artificial roots, plant removal can also be simulated. The results of these tests show that removal of plants from the upper portions of the slope has minimal effect on slope stability but stability is significantly compromised when vegetation is removed from the lower portion of the slope.

As well as fluctuations that might be interpreted as relatively easy to predict, if not account for, slopes and embankments are often subject to flooding. Saran and Viswanadham (2018) detail centrifuge tests on model levees which are subjected to flood events. Given that the potential for catastrophic failures to occur is well recognised and documented (e.g. Steedman & Sharp 2011) this work is significant and timely. Saran and Viswanadham (2018) perform centrifuge tests comparing the efficiency of horizontal and vertical (chimney) drainage layers within the levee. The experimental work is compared with numerical models. The experimental results suggest that the chimney drain increases the stability of the levee more effectively than the horizontal drain. This conclusion is not necessarily borne out by the numerical model which suggests that either drainage system results in a similar factor of safety against failure.

5.2 *Embankments on soft soils*

The problem of constructing an embankment over regions of soft soils is addressed by a number of papers. Founding an embankment on a soft underlying layer will generally result in long term settlements as the soft soil consolidates under the embankment load potentially damaging road and rail infrastructure. One solution to this problem is to improve the soft soil layer prior to embankment construction for which there are a number of approaches. Shiraga et al. (2018) detail centrifuge experiments on an embankment constructed using vacuum consolidation. Comparisons are made between embankment construction on the soft layer with and without the vacuum consolidation process. The results indicate that the vacuum consolidation process returns the pore water pressures in the ground to their original levels more quickly than simple embankment construction alone. Careful construction sequencing using this technique could reduce the possibility of having to undertake remedial works by ensuring settlements are mostly complete prior to installation of infrastructure on the embankment.

Another technique is to construct a piled embankment. The aim of this method is to reduce the load applied to the surface of the soft soil layer. This can be achieved by use of a geosynthetic such that the load spans between the piles beneath the embankment. Almeida et al. (2018) carried out multiple centrifuge experiments to investigate the influence on embankment performance of number geosynthetic layers, geosynthetic pretension, and pile size and arrangement. Their results concluded that use of a geosynthetic layer was extremely efficient in transferring load to the piles but there was no benefit to multiple layers and that the pretensioning effect was minimal. Blanc et al. (2018) also investigated the load transfer from the embankment (or granular mattress) to the piles below the embankment. In this work there was no geosynthetic reinforcement and only the pile spacing, size and height of embankment was varied. The work was carried out with the aim of validating previously published analytical models although the results suggest that each model is capable of representing some features of the system better than others.

6 TUNNELS AND PIPELINES

In previous conferences in this series, research and experimentation into tunnelling has been particularly well represented. It is interesting to note that for the 9th ICPMG the number of papers in this field is limited, perhaps indicating that researchers are adopting other techniques in this area.

6.1 *Tunnelling*

Tunnels are generally used in urban areas for mass transit systems. This is generally because of surface space constraints. Once constructed, tunnels can be subject to a variety of load conditions. Hajialilue-Bonab et al. (2018) describe 1g shaking table tests on an instrumented tunnel representative of a section of the Tabriz subway. Historically, underground structures have been subjected to lower levels of damage during earthquakes although the work presented here indicates that there may be a significant effect, particularly in strong ground shaking events. De & Zimmie (2018) investigated the effect of surface explosions on a tunnel. Measurements are presented on the basis of additional strains imparted to the tunnel lining during the event and some techniques for mitigation are investigated. The test arrangements are all shallow tunnels and it might be inferred that deeper cover (although not always possible) would result in more attenuation of the energy imparted by the explosion.

Xu & Bezuijen (2018) investigate the tunnelling construction process, specifically use of bentonite slurry to support the tunnel face during shield tunnelling. This is a 1g element testing study of how the bentonite filter cake develops as pore fluid infiltrates the soil surrounding the tunnel cavity. It was shown that different concentrations of bentonite affect the permeability of the surrounding soil with an almost direct relationship. The quantification of this would be useful information when designing tunnelling schemes through sandy strata.

6.2 *Pipelines*

Pipelines are used both onshore and offshore for the distribution of, for example, oil and gas. Extremely long networks are vulnerable to many hazards as they may cross earthquake zones, faults, slopes and many different soil conditions. Eichhorn & Haigh (2018) use the mini-drum centrifuge to examine the uplift resistance of pipes positioned parallel to the fall of a slope. Careful consideration is given to the scaling of the pipe model such that it is representative of a high pressure transmission pipeline that is currently in use. The experiments highlight deficiencies in the current methods used by industry.

The resistance to uplift is of critical importance during earthquakes or when large ground movements occur such as in a landslide. Wang et al. (2018) present a novel solution to the problem of uplift during earthquakes which is to strengthen the soil overlying the pipeline with vegetation. Model plant roots were created using a 3D printer and used to strengthen the soil in the upper layer of their experimental model. Results were presented in terms of the relative uplift of the pipeline with respect to the ground under the action of three earthquakes varying in intensity. Compared with a baseline case were there was no reinforcement, the introduction of roots to the soil did reduce pipeline uplift. The magnitude of reduction was related to the size of the roots. It was noted that uplift forces did not appear to be reduced and therefore the reduction was attributed to the increase in soil strength obtained from the root systems.

7 RETAINING WALLS

Retaining walls are found in a wide range of engineering projects including basement construction, retained embankments, and quays. There are also a wide variety of construction methods and materials. This diversity of application is represented by a number of papers investigating a range of topics including sheet piles, nailed walls and earth walls.

7.1 *Earth walls*

Walls constructed from earth offer advantages over other wall types such as low cost and ease of construction. To ensure stability, earth walls must contain some element of reinforcement and these systems are variously referred to as Geosynthetic Reinforced Soil Walls, Mechanically Stabilised Earth Walls and Reinforced Earth Walls. The performance of these types of walls relies on the interaction between the soil and the reinforcing elements. Mirmoradi & Ehrlich (2018) report large scale experiments on two Geosynthetic Reinforced Soil walls. Each wall was similar in design however one was faced with blocks and the other faced by wrapping the geosynthetic fabric around the soil. The wrap-faced wall was overall more flexible and transferred more of the applied surcharge load into the geosynthetic reinforcement. It is inferred that, given the good performance of both wall types, that the wrap-faced wall may be a preferred design solution on the basis of cost.

As stated earlier, the behaviour (and analysis) of these types of wall is dependent on the interaction between reinforcement and soil. Loli et al. (2018) have presented a large scale device for testing (and therefore characterising) reinforcement buried within soil. This device allows control of the overburden stress and is of a size sufficient to test many types of reinforcement. The performance of the device is compared with numerical modelling and the design choices justified on this basis. Whilst the focus of the paper is the device itself, it is clear that better understanding of the interaction between soil and reinforcement will enable more efficient and economical earth wall designs.

7.2 *Soil-nailed walls*

Two papers from the same group investigate walls reinforced with soil nails. Sabermahani et al. (2018) investigate the optimal arrangement of nails within an irregularly shaped excavation whereas Akoochakian et al. (2018) consider a more regular, rectangular excavation. The motivation for both these pieces of work relates to maximisation of available space during urban development. The work presented in both of these papers highlights that, even in relatively simple cases of regularly shaped excavations, the spacing of the nails, stiffness of the wall facing and presence of surcharge behind the wall all greatly influence the movements observed around the excavation.

8 SHALLOW FOUNDATIONS

As highlighted in the section described the papers submitted in the area of tunnels and pipelines, it is interesting to note how research activity changes over the years. In earlier iterations of the ICPMG there were many papers concerning the behaviour of shallow foundations but this number is very much reduced for the 9th ICPMG.

Qi & Knappett (2018) and Ghalandarzadeh & Ashtiani (2018) both present work relating to the response of shallow foundations under seismic loading. In the paper of Ghalandarzadeh & Ashtiani (2018) a similar approach to that taken by Yao et al. (2018) is adopted whereby an apparatus is developed that induces a predefined fault plane into the soil model and the response of the foundation to this is monitored. The footing load was generally maintained constant with embedment and distance from the fault plane being varied.

Results are presented in terms of footing rotation. In general, footings founded on the surface experience less rotation compared with footings that are initially embedded. The magnitude of the footing load appears to have little effect upon this result. As with Yao et al. (2018) the zone of influence of the fault is quite large so foundations are affected wherever they are placed within the experiment.

Qi & Knappett (2018) investigate the influence of soil permeability on shallow foundations (supporting a low-rise structure). The time histories of real earthquakes were applied sequentially. The results showed that, even if the potential for liquefaction could be identified, the prediction of structural damage is extremely difficult to estimate. In particular the effect of strong aftershocks seemed to place higher demands on the structure whilst not necessarily resulting in significant additional settlement or rotation.

9 GROUND IMPROVEMENT

There are many techniques available for ground improvement. The term generally refers to increasing soil strength but may also refer to the improvement of drainage. A number of papers submitted have considered ground improvement as a means to mitigating the effects of earthquakes. In particular, the use of drains (of various types) as mitigation against the effects of liquefaction is the subject of several papers. Paramasivam et al. (2018), García-Torres et al. (2018), Marques et al. (2018) and Kirkwood & Dashti (2018) all consider the use of vertical drains whilst Apostolou et al. (2018) consider the use of stone columns although their tests did not represent structural loads but rather, investigated dissipation of excess pore water pressures within the soil model. Finally, although not strictly speaking a ground improvement technique, Nigorikawa et al. (2018) describe a base isolation system as a mechanism to mitigate liquefaction effects.

All of these studies utilise centrifuge modelling techniques, highlighting the applicability of this method to studying earthquake related problems. The four papers that considered vertical drains underneath model structures (Paramasivam et al. 2018, García-Torres et al. 2018, Marques et al. 2018 and Kirkwood & Dashti 2018) all demonstrate a reduction in earthquake induced rotation and settlements when drains were used. There appeared to be a cost associated with this improvement however, in terms of the motion transferred to the superstructure. Both Kirkwood & Dashti (2018) and Paramasivam et al. (2018) saw increased accelerations of the structure when mitigation by drains was included. Additionally, in the tests of Kirkwood & Dashti (2018) an adjacent structure without drains was present which experienced an increase in rotations. The implication here is that this solution would either need to be applied to all structures in an area or that some sort of isolation or cut-off wall would be required to protect unmitigated structures.

10 SUMMARY AND CONCLUSION

Approximately fifty papers submitted to the 9th ICPMG have been reviewed in order reflect upon the contributions made, both in terms of experimental techniques being adopted and research questions currently being addressed. These contributions have been discussed with a view to identifying future trends and research questions whilst keeping in mind the progress that has been exhibited in the field over the entire conference series.

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The role of centrifuge modelling in capturing whole-life responses of geotechnical infrastructure to optimise design

S. Gourvenec
University of Southampton, UK

ABSTRACT: Whole-life design relies on scrutinizing the geotechnical responses to whole of life loading sequences, through installation and operation or service, and partnering appropriate 'current' operational soil parameters with corresponding 'current' loading to optimize design outcomes. Whole-life design offers efficiencies over established design methods that are based on in situ soil parameters. In the current environmental and economic climate, established paradigms of design are being challenged to make way for enabling technologies to deliver projects of greater scale and complexity for less risk and cost. Whole-life design can be applied to a range of geotechnical boundary value problems - and can best be practically investigated in a centrifuge environment. This paper demonstrates the role of centrifuge modelling to identify governing mechanisms of whole-life response as a critical activity in the trajectory from design concept to implementation in engineering practice. The role of geotechnical centrifuge modelling in capturing whole-life response to optimize offshore foundation design is illustrated, although the overarching concepts put forward in the paper have much broader application.

1 INTRODUCTION

1.1 *Whole-life response*

Geotechnical centrifuge modelling has enabled field-scale infrastructure, soil stresses and geotechnical processes to be modelled realistically at small scale for over 5 decades (Roscoe 1970, Lyndon & Schofield 1970) – or a bit under 2 days at 100 g. Many hundreds of papers reporting significant insights into a range of field-scale geotechnical boundary value problems from centrifuge modelling are collected in the 8 sets of proceedings of the *International Conference on Physical Modelling in Geotechnics* and elsewhere in the literature.

A temporal spectrum can be used to describe how geotechnical centrifuge modelling can assist understanding of different geotechnical responses:

1. Short-term '*events*' (e.g undrained installation or failure of a geotechnical structure, or a change in load, water table level or other state);
2. Longer '*episodes*' that may be comprised of a series of events (e.g. construction or operational processes such as an excavation sequence or extreme weather event);
3. The '*whole life*' of a structure (e.g. a lifetime of weather episodes, such as freeze-thaw seasonal cycles or storms, or an operating life of changes in load level, such as tanks repeatedly filling and emptying, or thermal expansion loads from operation of equipment or facilities).

These temporal concepts are illustrated in Figure 1. Whole-life geotechnical response is the least investigated of the three classes of activity and is the focus of this keynote paper.

The whole-life concept partners whole-life loading sequences with whole-life soil responses to optimize geotechnical design outcomes. For example, this approach can lead to reduced foundation size where the soil strength rises through the design life, with knock on effects of reduced costs through

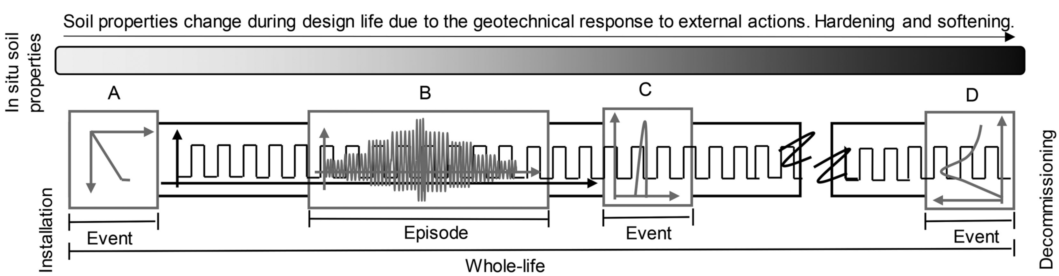

Figure 1. Temporal spectrum of geotechnical processes.

fabrication, transport and installation (Gourvenec et al. 2017a).

Consideration of the whole-life response requires identification of the dominant loading sequence that governs the geotechnical response of the soil throughout the design life. This inevitably leads to idealization of a field situation but is essential to obtain the effect of the whole-life loading on the whole-life capacity. In this way, the evolution of the soil strength, stiffness and other geotechnical parameters can be 'banked' where it is beneficial in meeting the design criteria throughout the whole life.

Events or episodes can be superimposed on a background of the whole-life response to enable greater scrutiny of specific activities or environmental influences. It is essential to understand current operative soil strength as well as the current position of a structure to inform predictions of the geotechnical response for an event during the life of the structure.

Questions such as "what is the soil strength and stiffness at the start and end of the episode 'B' in Figure 1" determine the (true) geotechnical stability of the structure during the episode and subsequently for future events, such as 'C'. Identifying the 'true', or current, operative shear strength and stiffness to inform that calculation enables a more realistic prediction of geotechnical resistance and optimized design, compared to assuming that the initial (in situ) properties apply throughout the life.

An example, topical in offshore engineering at present, is the need to predict retrieval loads to lift a structure from the seabed for decommissioning. Installation, self-weight consolidation, and a life time of operational loading and consolidation change the seabed state and strength, as well as the foundation position since installation. These conditions need to be predicted to assess the uplift resistance, which is likely to be (potentially much) greater than the resistance during installation (Small et al. 2015, Gourvenec & White 2017). This whole-life behaviour has clear implications in terms of crane requirement for vessels for planning decommissioning.

1.2 *Offshore facility architecture and foundations*

1.2.1 *Field development architecture*

Offshore developments for oil and gas are increasingly diverse in terms of architecture and support an increasingly diverse range of activities. Offshore structures range from single fixed platforms to fixed or floating hosts supporting a subsea development that may extend tens of kilometers from the host. Alternatively, subsea developments may be tied directly back to shore (Figure 2). Subsea developments comprise a network of flowlines (in-field pipelines) connected by structures to transport fluids to or from wells (Figure 3).

1.2.2 *Fixed and floating platforms*

Foundations for fixed offshore platforms are subjected to multi-directional loading derived from self-weight and cyclic lateral loads and moments dominated by

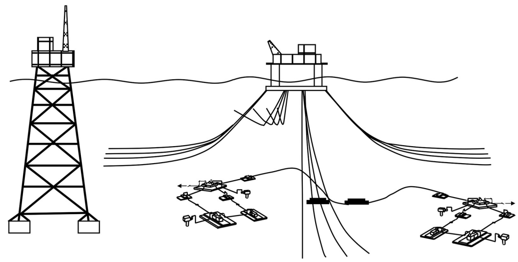

Figure 2. Examples of offshore structures; (L) single fixed platform and (R) floating host and subsea development.

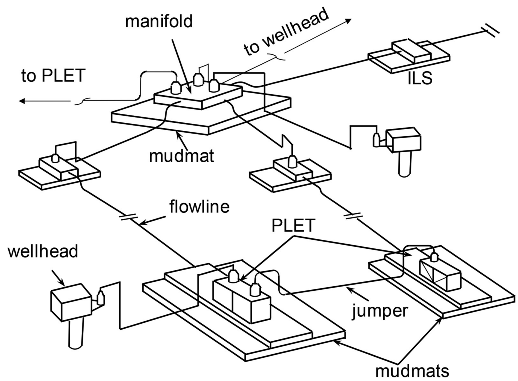

Figure 3. Examples of subsea architecture.

environmental forces such as wind, waves, current and in places sea ice. A permanent monotonic moment component may also arise from eccentricity of the supported superstructure relative to the foundation footprint. Foundations for fixed platforms are typically deep piled foundations or gravity bases.

Foundations, or anchors, for floating platforms are subject to vertical loads determined by the buoyancy of the floater and to multi-directional loading derived from environmental forces. One-way cyclic loading is typical for mooring anchors compared to the two-way cyclic loading seen by fixed platform foundations. Deep piled foundations, suction caissons and drag anchors are the most common anchoring systems for floating platforms although a number of novel and developmental anchors exist.

1.2.3 *Subsea structures*

Foundations for subsea structures supporting pipeline infrastructure are subjected to self-weight loading, but in contrast to fixed and floating platforms, the multi-directional horizontal loads, moment and torsion are dominated by installation and operational activities, rather than by environmental conditions.

Tie-in or 'metrology' loading occurs when pipelines and jumpers are connected to the subsea structure. Episodic monotonic loading occurs from thermal expansion and contraction of the attached pipelines and jumpers from start up and shut down cycles that form the operation of an offshore development.

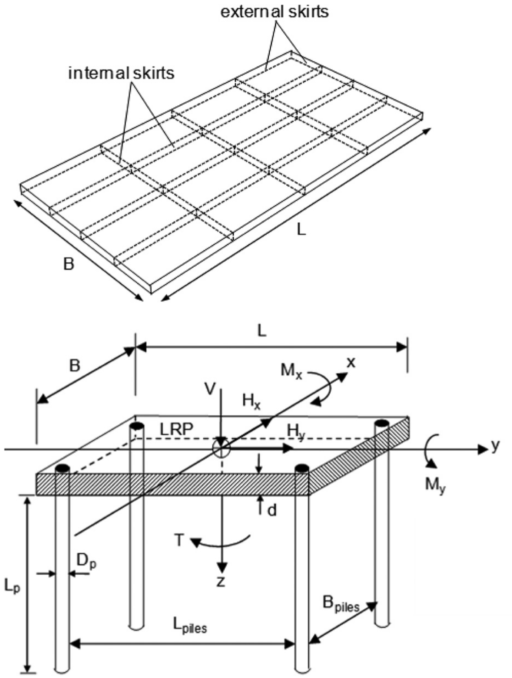

Figure 4. Options for enhancing capacity of subsea mudmats (upper) skirts and (lower) pinpiles.

Figure 5. Subsea pipeline end termination structure (Image from Subsea7).

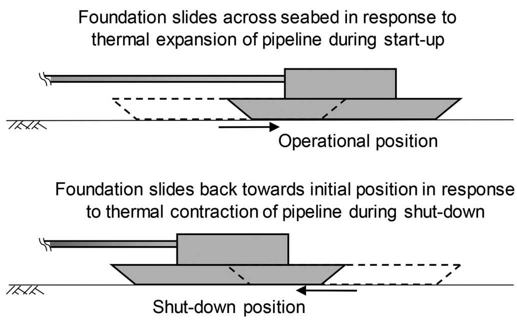

Figure 6. Concept of a tolerably mobile subsea foundation.

Pipelines expand as hot hydrocarbons pass through the pipelines during start up, remain expanded during an operational cycle, and contract when a well is shut down and the pipeline no longer contains hot hydrocarbons. Multi-directional lateral loads, moments and torsion are imposed to the foundation due to vertical and horizontal eccentricities of the pipeline and jumper connections to the orthogonal axes of the foundation.

Subsea structures are often supported on shallow foundations, or 'mudmats', due to the attractiveness of relatively straightforward self-weight installation, often performed from the pipe-laying vessel.

Increasingly, shallow foundations for subsea structures designed with traditional methods are too large for standard installation vessels. This is because of the more demanding operational requirements – capacity and stiffness – set by the supported structures and due to the softer seabeds found in deeper waters. This leads to increased project costs associated with heavier lift vessels.

Shallow foundations for mudmats can be augmented with skirts, caissons or pinpiles (Figure 4) that penetrate the seabed to increase resistance and reduce displacements (Dimmock et al. 2013, Feng et al. 2014, Hossain et al. 2015, Demel et al. 2016, Gourvenec et al. 2017b, Dunne & Martin 2017, Wallerand et al. 2017). However, these modifications lead to increased cost and risk in fabrication and installation (risk of failure to install) and cannot always deliver the reduction in mudmat footprint required. A photograph of a pipeline end termination structure on a skirted mudmat is shown in Figure 5.

1.2.4 *Challenging the design paradigm*

The basis of design for a mudmat is that it will spread the supported loads to the seabed with limited settlement and without geotechnical 'failure' – currently defined in practice by a required material factor on the mobilized soil strength (ISO 2016, API 2011). Traditional design methods for shallow foundations require a sufficiently large footprint to resist all applied loading and remain stationary in order to meet the basis of design – i.e. to avoid 'failure'.

To meet the demand for smaller subsea foundation footprints, the traditional design paradigm of static foundations has been challenged with concepts of 'tolerable mobility' or 'on-seabed sliding' (Cathie et al. 2008, Cocjin et al. 2014a, 2015, Deeks et al. 2014, Stuyts et al. 2015, Wallerand et al. 2015, Feng & Gourvenec 2016).

The concept of tolerable mobility is that the foundation is designed to move across the seabed to relieve the displacement-sensitive tie-in or operational loads in a way that is tolerable in relation to the function of the structure. However, foundations designed to slide across the seabed violate the current code definition of 'failure' (e.g. API 2011, ISO 2016). Nonetheless, sliding foundations have been designed and deployed for projects and centrifuge modelling has been a key element in making this possible (Client confidential).

Figure 6 illustrates the concept of a tolerably mobile mudmat for a pipeline structure. The mudmat rests on the seabed (i.e. without skirts) and is equipped with 'skis' to resist against overturning during sliding.

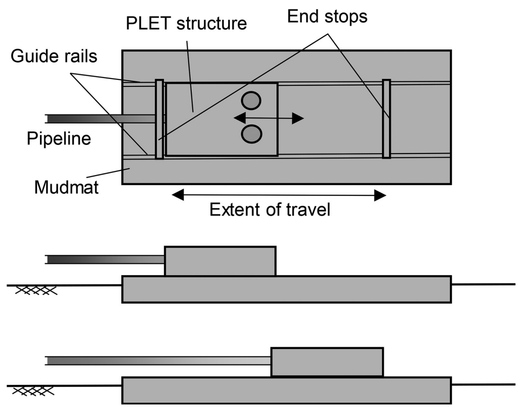

Figure 7. Concept of a static mudmat with a sliding mechanism.

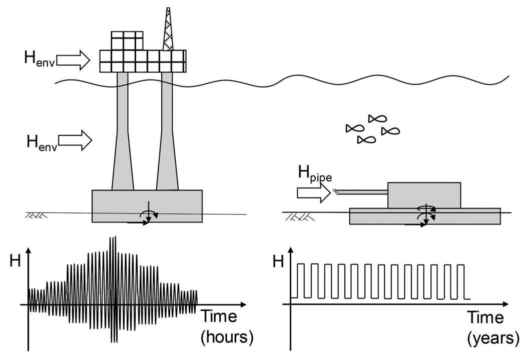

Figure 8. Comparison of loading sequences relevant to shallow foundations for (left) a fixed platform and (right) a subsea pipeline structure.

The foundation slides across the seabed in response to thermal expansion of the pipeline during start up and remains in the operational position while the well is producing, which may be for a few days to a few months before the next shutdown. The pipeline then slides back towards the initial position in response to thermal contraction of the pipeline on shut down. Shutdown is brief, typically a day or two. The process repeats episodically over the life of the structure leading to cycles of shearing (during sliding) and reconsolidation of the seabed at the operational and shut-down positions. The sliding foundation concept is similar to a snow sleigh or ski but is intended for only small distance of travel, of the order of a few meters.

It is worth noting that a 'sliding foundation' in this context is different to static subsea foundation equipped with a sliding mechanism (e.g. Jayson et al. 2008). In the latter case, a mechanical slider is mounted on the mudmat to absorb, to some extent, pipeline expansion and contraction movements (Figure 7). In contrast, a 'sliding foundation' is taken to mean a foundation that slides across the seabed.

Whole-life concepts are an additive tool that can be applied to reduce foundation footprints, whether designed to be static or tolerably mobile, and can yield particular efficiencies in subsea foundations.

1.3 *Application of whole-life concepts to subsea foundations*

Whole-life response can provide significant efficiencies to subsea foundation design outcomes due to the nature of loading, which is quite different to that for a fixed or floating platform.

A subsea mudmat may be set down some weeks or months in advance of a field becoming operational at which point the multi-directional operational loads are 'switched on'. The geotechnical foundation design can then rely on the enhanced shear strength of the seabed due to consolidation under self-weight of the foundation and structure for the operational load case. This is not so straightforward for platform foundations as the multi-directional loading is driven by environmental forces that are less predictable, which makes reliance on enhanced self-weight consolidated strength for the operational design load case more challenging. Nonetheless, the broadest concepts of whole-life response underpin the established practices of reliance on set-up in pile foundation and anchor design, active suction programs for gravity based platforms, staged installation processes (for GBS, embankments or artificial islands) and reuse of existing foundations. Moving beyond these examples of self-weight and post-installation consolidation, additional whole-life response benefits can be harnessed to optimize subsea foundation design.

Figure 8 illustrates schematically the load sequences transmitted to the shallow foundation for a fixed platform and a subsea pipeline structure. For the fixed platform, peak loading corresponds to extreme weather events that involve high amplitude and frequency cyclic loading. The duration of an extreme weather event, typically a few hours or days, prevents significant dissipation of excess pore pressure in fine grained seabeds. Excess pore pressures therefore accumulate, leading to a reduction in effective stress and undrained shear strength of the seabed, i.e. cyclic softening (e.g. Andersen 2015, Zografou et al. 2016) and a subsequent reduction in foundation capacity (e.g. Andersen et al. 1988, Xiao et al. 2016).

The whole-life loading sequence of a shallow foundation supporting a subsea structure will depend on the function of the structure and the environmental conditions, but in many cases will be dominated by operational activities, i.e. the thermal expansion and contraction of the attached pipelines and spools during start up and shut down operations. The duration of these operational activities are orders of magnitude longer than storm loading (months rather than days), such that excess pore pressures generated during the loading event may dissipate, even in fine grained seabeds, prior to the subsequent cycle.

Intervening reconsolidation between cycles of loading can lead to an increase in the shear strength of the seabed, i.e. cyclic hardening. Cyclic hardening has been demonstrated with in situ characterization