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Development, effects and mitigation of earthquake-induced liquefaction: A comprehensive study based on dynamic centrifuge modelling

Développement, effets et mitigation de la liquéfaction induite pas des séismes: Une vaste étude basée sur la modélisation dynamique en centrifugeuse

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ABSTRACT
Liquefaction remains a major threat to shallow foundations built on saturated deposits of sand, existing semi-empirical rules being inadequate for the modern requirements of performance-based design. In order to clarify the effects of liquefaction on shallow foundations and to enhance the use of mitigation methods to minimize its consequences, a broad research programme based on centrifuge modelling is being carried out at Cambridge University. This paper summarizes and critically assesses the data produced by 8 centrifuge tests, intended to describe the behaviour of bridges built on shallow foundations and to identify the different phenomena occurring in the soil during and after the earthquake loading. The deformation mechanisms of shallow foundations and the unforeseen relevance of some phenomena are discussed. The experimental results highlight some of the limitations of the information collected from case histories and have practical implications for the use of densification as a liquefaction resistance measure.

RÉSUMÉ
La liquéfaction reste une menace importante pour les fondations superficielles construites sur des sables saturés, les règles semi empiriques actuelles ne se conforment pas aux exigences modernes pour une conception basée sur la performance. Afin de clarifier les effets de la liquéfaction sur les fondations superficielles et améliorer l'utilisation des méthodes de mitigation pour en minimiser les conséquences, un vaste programme d'essais en centrifugeuse est en cours à l'université de Cambridge. Cet article résume et critique les données de huit essais modélisant le comportement des ponts construits sur fondations superficielles et les différents phénomènes se produisant dans le sol pendant et après le chargement sismique. Les mécanismes de déformation des fondations superficielles et la relevance inattendue de quelques phénomènes sont discutés. Les résultats expérimentaux soulignent certaines limitations des analyses des cas réels et ont des implications pratiques pour l'usage de la densification comme mesure de mitigation de la liquéfaction.

1 INTRODUCTION

Liquefaction is a major threat to structures built on loose deposits of saturated sand in seismically active regions. Past major earthquakes caused severe liquefaction-induced damage to hundreds of bridges, especially relatively small bridges in developing countries founded on shallow foundations. Study of case histories often suggests that these bridges collapse as a result of large settlement and/or rotation of the pier foundations, allegedly due to the loss of bearing capacity during the earthquake (Hamada et al., 1992). The mechanism of collapse can involve decks being dislodged from their pads, eventually causing a chain collapse of the superstructure. Fortunately, the dramatic collapse of these structures is not usually matched by a large number of human casualties, as the process takes place relatively slowly and failure often occurs well after the end of the earthquake (Yong et al., 1988). This does not make the failure of these structures less tragic, as it seriously affects the critical flow of aid to the catastrophe-stricken area and causes a prolonged disruption of economic and social life.

Present design is mostly based on semi-empirical rules, which fail to comply with the modern requirement of performance-based design. Numerical modelling offers large potential for the dynamic analysis of soil-structure systems, but it still faces a low success rate, due to a poor understanding of all the physics involved in liquefaction phenomena. In fact, the interpretation of case histories lacks data obtained by instruments installed in the ground prior to the event while element tests can only provide a simplified view of the behaviour of a single soil element under an idealised loading condition, from which the failure mechanism of a boundary value problem cannot be directly established. For these reasons, dynamic centrifuge modelling offers a unique tool for research on liquefaction problems.

2 RESEARCH PROJECT DESCRIPTION

In order to clarify the mechanisms involved in the liquefaction-induced failure of shallow foundations, to assess the capability of available numerical codes to simulate these problems and to optimize the use of liquefaction resistance measures, a broad research programme based on centrifuge modelling was initiated at Cambridge University. This project started in March 2002 and is funded by EPSRC and Mott MacDonald, UK. In view of the weaknesses of current design practice, the project aimed to combine centrifuge and numerical modelling as major research tools. While this paper focuses on the main experimental achievements of the project, Haigh et al. (2005) describes the concurrent progress on the numerical analyses up to the present.

The centrifuge tests were performed on the Schofield Centre's 10-m diameter Turner beam centrifuge. The models, representing deposits with a prototype depth of 18 m, were prepared by air pluviation of Fraction-E silica sand inside the flexible boundaries of an ESB container, whose behaviour is described by Brennan and Madabhushi (2002). The sand was saturated with viscous fluid to perform the so-called viscosity scaling, as clarified along with the relevant scaling laws by Schofield (1981). The models were heavily instrumented to assess the soil-system's response to an earthquake simulation, which was generated by the SAM actuator (Madabhushi et al., 1998).

Three types of centrifuge model were tested, representing:

I) level deposits of uniform and saturated sand, having varying relative density, ranging from 50% to 80%;
II) bridges built on a loose deposit of uniform and saturated sand, without any improvement under the shallow foundation;
III) bridges built on a loose deposit of uniform and saturated sand, with a densified block of varying width being created

under the shallow foundation extending down to the "bedrock".

3 MAIN EXPERIMENTAL ACHIEVEMENTS

3.1 *Level deposits*

Three uniform models of sand with relative densities (D_R) of 50%, 60% and 80% were subjected to similar earthquake loading, providing information about the influence of D_R on the free-field behaviour. Figure 1 presents some short-term data from the loosest and densest models tested.

It was observed that increasing D_R significantly reduces the liquefaction-induced settlement during the earthquake, which is, in both cases, the most significant part of the total settlement. As Table 1 shows, between 80 and 95 % of the total settlement occurs during the shaking, the post-earthquake settlement accounting for only a minor part of the total. Considering that excess-pore-pressures (e.p.p.) remain at their maximum values for some time after the end of the earthquake, especially near the surface, the results imply that the permeability of the deposit increases notably during the shaking. This fact, firstly conveyed by Ishihara (1994) based on the results of VELACS project, may only be explained by the formation of a transient system of pipes and/or cracks in the soil during the shaking. The hypothesis of formation of pipes is also supported by results of 1-g and numerical modelling presented by Gudehus (2004).

One of the most striking results obtained was the similarity of the e.p.p. generated in loose and dense sand (Fig. 1). Although the rate of e.p.p. generation seems to decline with D_R, the difference is not as large as that observed in cyclic element testing of saturated sands in undrained conditions. Furthermore, it seems that the e.p.p. can in any case increase up to the value of the initial vertical effective stress, suggesting that liquefaction can develop in both cases. The apparent inconsistency between the results of element and centrifuge tests may result from the fact that the fully undrained condition imposed in element testing does not occur in a centrifuge model, as implied by the amount of settlement occurring during the earthquake. Thus, a significant amount of upwards seepage and e.p.p. migration will occur during shaking, triggered by the hydraulic gradient generated and causing pore-pressure to rise at higher levels.

The upwards propagation of horizontal accelerations is very dependent on D_R. Peak accelerations reaching the surface in a liquefied deposit show enormous attenuation from the first cycle in loose sands, but include a period of very large short-duration spikes in dense sands (Coelho et al., 2004a). This suggests that structures built on a densified block may be more heavily loaded during an earthquake than those founded on loose sand.

Table 1. Settlement of the model surface (Coelho et al., 2004a)

Centrifuge model's D_R (%)	Absolute settlement- prototype scale (mm)			Aver. volum. strain: ε_v (%)
	During shaking	Post-shaking	Total	
50	342 (80 % [*])	88 (20 % [*])	430	2.50
80	148 (95 % [*])	7 (5 % [*])	155	0.86

[*] *% relative to the total settlement*

3.2 *Bridges built on non-improved ground*

Two models representing bridges built on loose sand (50% D_R) without improvement were subjected to earthquake simulations lasting 10 s, having a predominant frequency of 1Hz and peaks of horizontal acceleration exceeding 0.2-g. The model bridges have two symmetric 17.5-m long decks resting 6 m above the ground on a single central pier supported by a 4-m wide footing. Between the first and second tests, the pressure transmitted through the foundation base was increased from 75 kPa to 100 kPa and the centre of gravity was raised from about 0.5 to 0.7 of the bridge's height. To simplify future numerical modelling of the problem, the models were devised to replicate a plane-strain condition (Coelho et al., 2004b). A single pier was used in order to minimize the undesirable but relatively significant boundary effects observed near the container walls (Coelho at al, 2003).

Table 2 presents the settlement measured in each test. It is shown that both footings suffer large total settlements which, like final inclinations, intensify with bearing pressure. The post-earthquake settlement, which is significant in every case, increases more markedly with bearing pressure. Moreover, major footing tilting only seems to develop once the earthquake ends.

Other noteworthy events observed are the considerable attenuation of horizontal accelerations transmitted to the structure and the different stages of e.p.p. generation observed in the ground under the influence of the footing. During the first few cycles, positive e.p.p. leads to soil softening and progressive straining under the combined loading of the footing and earthquake. As soil strains reach a certain level, the soil starts to dilate, inhibiting complete loss of effective stress and capping or even reducing the e.p.p. in the soil. At this stage, a soil column stiffer than the liquefied free-field is formed under the footing,

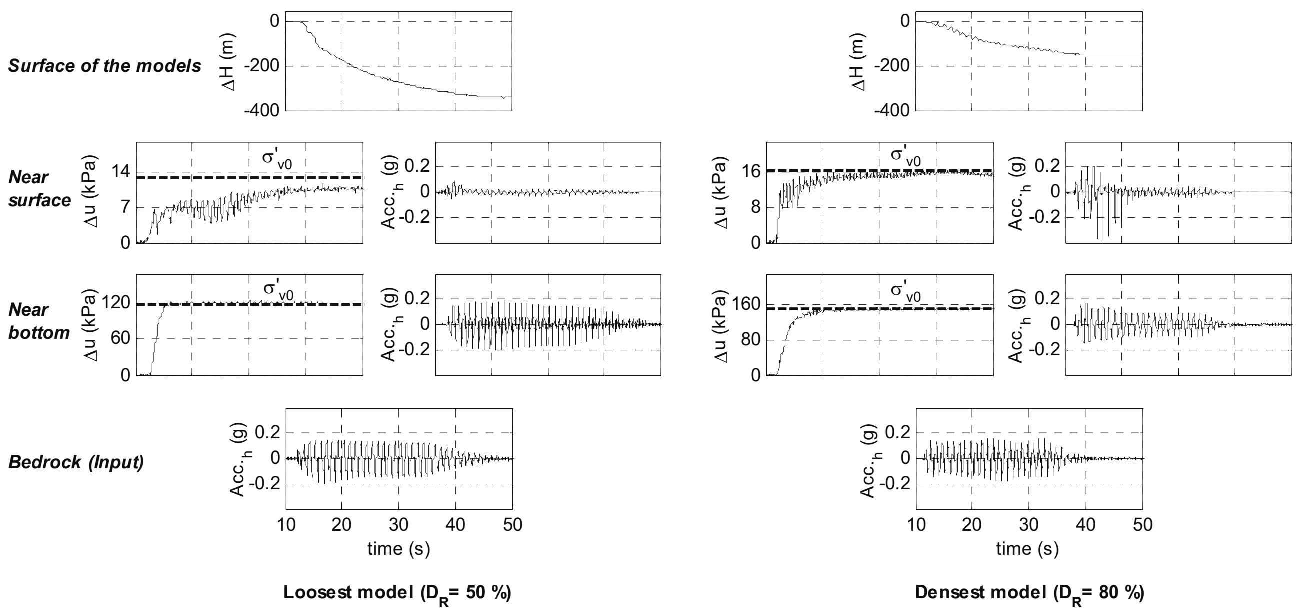

Figure 1. Settlement, excess-pore-pressure generation and propagation of accelerations in the free-field during earthquake simulation (prototype scale).

causing local stress concentration. As soon as the earthquake ends, the hydraulic gradients formed during the event lead to pore water migration from the free-field towards the soil column under the footing, which softens and redistributes to the surrounding soil part of the stresses earlier sustained.

Table 2. Settlement of the footing

Bearing pressure (kPa)	Footing side	Footing settlement- prototype scale (mm)		
		Total	During shaking	Post-shaking
75 (model 1)	Left	543	410* (76 %**)	133 (24 %**)
	Right	546	410* (75 %**)	136 (25 %**)
100 (model 2)	Left	826	530 (64 %**)	296 (36 %**)
	Right	793	533 (67 %**)	260 (33 %**)

* *approximate value*; ** *% relative to the total settlement*

3.3 *Bridges built on improved ground*

The bridge used in the second model described in section 3.2 was employed in 3 subsequent tests, intended to assess the performance of densification as a liquefaction resistance measure. The models, named as 1B, 2B and 3B, incorporated densified (80 % R_D) blocks corresponding to ratios of block to footing widths of 1, 2 and 3, respectively, and were subjected to the earthquake simulation described in the previous section.

Figure 2 shows the e.p.p. generation in the soil under the footing and in the free-field (50 % R_D), at a depth of 2 m, in test 1B, which is representative of the results obtained in other tests with wider densified zones. The results are qualitatively similar to those described in section 3.2 for the test with non-improved ground, though in test 1B dilation during shaking starts earlier and can even cause negative e.p.p. to develop in the soil, enhancing the concentration of stresses in the column under the footing. Due to the larger hydraulic gradients, post-earthquake e.p.p. migration is more significant in this case. Figure 2 clearly shows the importance of the phenomenon: in the free-field, the e.p.p. quickly grows to a value that corresponds roughly to the local initial effective vertical stress, while under the footing similar values are only attained a long time after the end of the earthquake. This phenomenon is observed at deeper levels, though the relative importance of the post-earthquake e.p.p. migration and the time to reach the maximum value of e.p.p. tend to reduce with depth. Therefore, in this case, the critical condition in terms of the foundation's bearing capacity occurs about 15 minutes after the earthquake ends, when the e.p.p. remains high in the free-field and reaches its peak under the footing.

In all the models tested, the dynamic loading of the bridge is considerably larger than in the case of non-improved ground, where substantial attenuation of the input motion was observed. Comparing the cases where densification was used as a liquefaction resistance measure, Coelho at al (2004c) show that, after liquefaction develops in the free-field, the seismic energy transmitted to the structure tends to increase with the width of the densified zone. This fact, which is compatible with the behaviour of dense sand in the free-field, should not be disregarded in structural design when densification is employed as a liquefaction resistance measure.

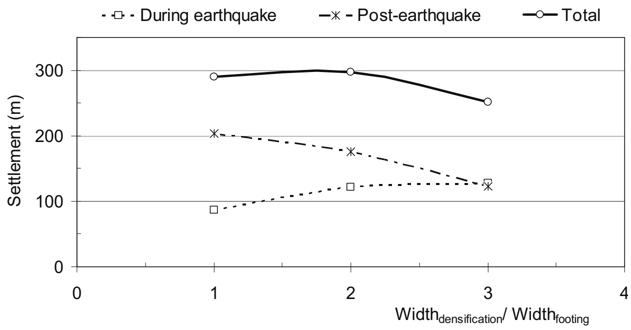

Figure 3. Settlements of the footing for different improvement widths

The average settlements measured in each test are plotted in Figure 3. By comparing these results with those of the non-improved case, it is confirmed that densification significantly reduces the total settlement, the method's performance being enhanced by the increase of the densification width, as commonly believed in practice. The analysis of the co-seismic and post-seismic fractions of the settlement shows that this improvement in performance is purely a consequence of a considerable reduction of the post-earthquake settlement, as the settlement occurring during the earthquake can even increase with the width of the improved zone. This occurs despite the fact that the e.p.p. generation and migration during and after the earthquake, under the footing, is very similar in all the models tested with densified zones. It also seems, according to the observations presented in section 3.1, that the relative density of the sand in the free-field should not account for such a difference in behaviour, as the e.p.p. generation is not so different. The information compiled suggests that, during the earthquake:

i) the performance of the soil foundation depends mainly on a narrow column under the footing, where D_R-dependent dilation induced by large straining causes local stress concentration;
ii) the firm column transiently created is surrounded by sand that, irrespective of its D_R, is critically softened by e.p.p. generation and virtually has no effect on the foundation capacity;
ii) widening the zone of densification instigates more severe inertial loading of the soil-structure system, thus increasing the co-seismic settlement of the footing;

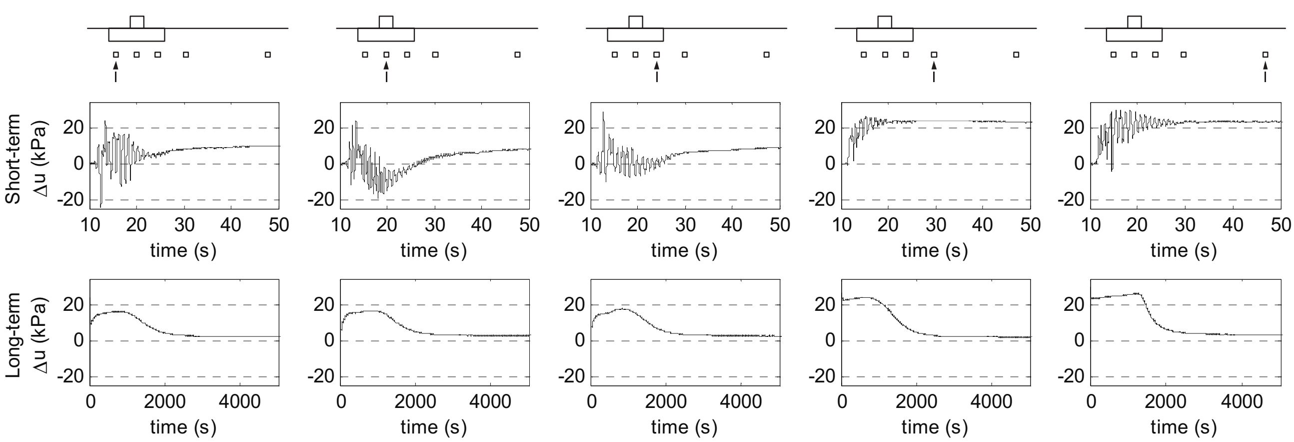

Figure 2. Excess-pore-pressure generation and migration in test 1B, at a depth of 2 m (prototype scale).

whereas, after the earthquake:

i) irrespective of the width of the densified zone, significant e.p.p. migrates towards the column under the footing, stabilization occurring long time after the end of the earthquake, only when horizontal hydraulic gradients vanish (critical situation);
ii) the new stress redistribution induced by the softening of the central column causes further loading of the surrounding liquefied sand, whose post-liquefaction behaviour strongly depends on its D_R, as illustrated in Figure 4.

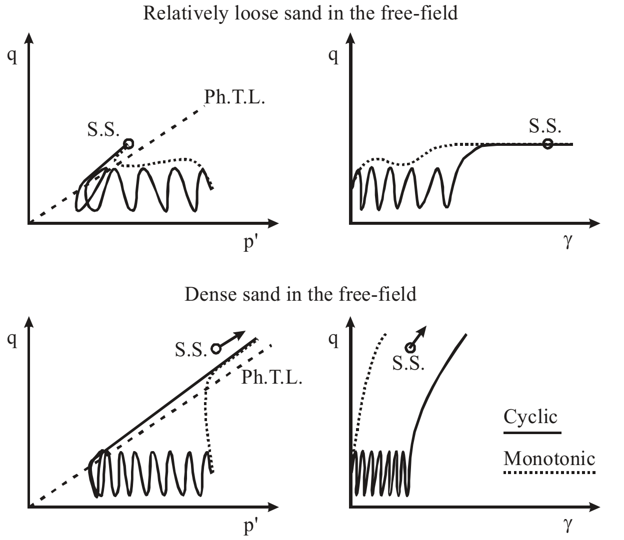

Figure 4. Behaviour of sand monotonically sheared after large cyclic e.p.p. generation.

As a corollary of the mechanism described, it can be inferred that larger densified zones perform better in practice only because of the larger residual strength of dense sand in the free-field, which is required to resist the post-earthquake loading transmitted as a result of the e.p.p migration. The different behaviour of loose and dense sands in monotonic shearing after large cyclic e.p.p. generation, which is illustrated in Fig. 4, has been characterized in detail based on element tests, as described by Ishihara (1993) and Yoshimine and Ishihara (1998). Irrespective of the amount of e.p.p. generated during cyclic loading, dense sands always dilate significantly towards the steady state (S.S.) when monotonically sheared as soon as the stress path crosses the Phase Transformation Line (Ph.T.L.). In contrast, sands in a looser condition suffer sudden collapse or, as exemplified in Figure 4, can only sustain a minor increase in deviatoric stress, accompanied by very large strains. This means that if the D_R of the loose ground used in the models (D_R =50 %) had been lower, complete failure of the structure or much larger settlements could have occurred.

4 CONCLUSIONS

The results of the centrifuge modelling performed as part of this research project intended to improve the understanding of earthquake-induced liquefaction under shallow foundations, suggest that:

i) dense sand in the free-field is easier to liquefy under a seismic simulation in a centrifuge test than is usually seen in element testing, possibly because it is not under a truly undrained condition, as the magnitude of the co-seismic settlement clearly demonstrates;
ii) whether densification is used as a liquefaction resistance measure or not, the mechanism of failure or deformation is complex and involves, at different stages in time, positive and negative e.p.p. generation under different conditions and e.p.p. migration;
iii) the co-seismic settlement is controlled by the performance of a narrow soil column under the footing, which is optimum for an improvement to footing width ratio of 1, and the amount of seismic loading transmitted to the structure, which tends to increase with the width of improvement;
iv) the post-earthquake settlement is mainly instigated by the e.p.p. migration towards the column under the footing and is deeply determined by the post-liquefaction monotonic shear behaviour of the surrounding sand, which is critically dependent on the sand's relative density.

The experimental observations highlight some of the limitations of the information obtained through analysis of case histories, from which the characteristics and the importance of the individual phenomena described herein can hardly be assessed in detail. The results provided by the research project are anticipated to have practical implications for the future advances in the use of in situ densification as a liquefaction resistance measure, as they promote essential scientific understanding of the different features of the development and effects of liquefaction in the field.

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