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CENTRIFUGE MODELING OF THE USE OF DENSIFICATION AS A LIQUEFACTION RESISTANCE MEASURE FOR BRIDGE FOUNDATIONS

Paulo A. L. F. COELHO[1], Stuart K. HAIGH[2], S. P. Gopal MADABHUSHI[3] and Tony O'BRIEN[4]

SUMMARY

Bridges built on saturated deposits of loose sand in seismically active areas are, regardless of their size, critical structures during seismic events. In fact, the location of these structures increases the risk of liquefaction-induced foundation failure. In addition, the consequence of their collapse is not limited to the direct human and economic losses but also creates a traffic impediment that severely affects post-earthquake rescue of human lives and properties and imposes a long-term disruption of social life. Small to medium size bridges found in this environment are particularly vulnerable to the effects of liquefaction, as they are frequently built, especially in developing countries, on shallow foundations.

Even though densification is frequently used as a liquefaction resistance measure for bridge foundations, design is based on poorly understood fundamentals. This paper presents the results of dynamic centrifuge modeling undertaken to investigate the behavior of a foundation-bridge system during an earthquake causing pore-pressure in the granular soil to rise. The results of a dynamic centrifuge test performed on a model of a bridge resting on liquefiable ground are compared with those obtained from similar tests where a zone beneath the bridge foundation was densified, using three different geometries. The tests demonstrate the dramatic consequences of ground liquefaction on bridge foundations and enlighten the different effects of the existence of a densified foundation soil zone on the behavior of the system. The influence of the geometry of the improved zone on the bridge performance under seismic loading is also assessed, providing important insight with respect to the optimum geometry that may be used in practice.

The results shown in this paper suggest that the present understanding on the use of densification as a liquefaction resistance measure is still limited, the issue of post-earthquake pore pressure migration emerging as a key factor to consider. New concepts that should be more deeply explored in the future are disclosed. The conclusions presented suggest that the use of densification to mitigate liquefaction effects may be improved and are particularly relevant in view of the fact that full-scale observation of the performance of improved sites is scarce and often requires considerable subjective interpretation.

[1] PhD Research Student, Engineering Dep., Cambridge University, UK. Email: pal37@eng.cam.ac.uk
[2] Research Associate, Engineering Dep., Cambridge University, UK. Email: skh20@eng.cam.ac.uk
[3] Senior Lecturer, Engineering Dep., Cambridge University, UK. Email: mspg1@eng.cam.ac.uk
[4] Project Director, Geotechnics Division, Mott MacDonald, UK. Email: aso@mm-croy.mottmac.com

INTRODUCTION

Since the first time that earthquake-induced liquefaction was responsible for widespread destruction, in the 1967 Niigata earthquake, engineers facing structural and foundation design in seismic active areas regard this phenomenon as a major threat to structure stability during an earthquake. The situation of bridges built on loose granular deposits is particularly critical, as they are usually located in flooded areas that form the ideal conditions for liquefaction to occur. Regardless of the size of these bridges, the consequences of their collapse can be devastating, not only because of the loss of human life and property directly inflicted but also as a result of the serious impediment to the circulation of persons and vehicles that will seriously affect the post-earthquake flow of aid into the catastrophe-affected region and impose a long-term disruption to normal social and economic life. Small to medium size bridges built in this type of environment are especially vulnerable to the effects of earthquake-induced liquefaction, especially in developing countries, where they are frequently supported on shallow foundations.

Hamada [1], Madabhushi [2] and Wakamatsu [3], amongst others, present comprehensive reports of a very significant number of deck-type bridge failures caused by earthquake-induced liquefaction, which occurred during the last decades in several different locations in the world. The descriptions show that, in most cases, the collapse is not related to structural failure of the bridge components, being instead determined by the behavior of the foundations and the deformations of the surrounding soil. In general, the occurrence of soil liquefaction leads to vertical and or horizontal deformations of the piers that can not possibly be accommodated by the pier-deck connections, causing quite often the decks to fall from the bearings. Other interesting remarks, some of them obtained directly from eye-witnesses of the failures, include the description of the duration of the phenomenon of liquefaction and the speed of the structure failure. According to the reports, the ground where bridges collapsed following strong earthquakes remained liquefied for quite a long time after the end of the earthquake, as indicated by sand boils in the surface. Furthermore, the failure of these structures is usually described as slow, which explains the fact that in most cases people had time to evacuate the bridge, the number of victims recorded being in consequence relatively small.

Densification has been extensively used as a liquefaction resistance measure, following the common belief that denser granular soils show lesser tendency to generate excess pore pressure during cyclic loading than loose equivalents. The merit of this technique can be irrefutably perceived from the enhanced performance of improved sites compared to adjacent non-improved sites recorded in past earthquakes, as described in detail by Mitchell [4] and Hausler [5], amongst others. Unfortunately, as with other aspects of earthquake-induced liquefaction, research and scientific clarification of the use of densification to mitigate soil liquefaction hazards have systematically followed rather than led the progress, which explains the fact that some basic issues relative to this subject are still poorly understood. Crucial questions like the geometry that results in a maximum ratio benefits/cost of the improvement or the importance of eventual undesired effects resulting from soil densification, including ground motion amplification, are not sufficiently clarified. Even the mechanism sustaining the positive effects of soil densification on structure performance seems undecided, especially in face of the results of centrifuge modeling of uniform deposits of loose and dense sand presented by Coelho [6]: although pore pressure generation is slower in denser deposits, liquefaction seems to be attained in both cases.

In view of the limitations of full scale observations during real events, resulting from temporal and spatial unpredictability of earthquakes, and the difficulties involved with numerical modeling of these problems, due to the complex behavior of liquefiable soil, centrifuge modeling emerges as a unique tool in research. In fact, taking into account that the soil physical and stress conditions are mimicked, centrifuge modeling is able to capture the true soil behavior under realistic loading, provided that the boundary conditions of the problem are appropriately set.

USE OF DENSIFICATION AS A LIQUEFACTION RESISTANCE MEASURE IN PRACTICE

In order to minimize earthquake-induced liquefaction risk for bridges foundations as well as other structures, ground remediation techniques have been extensively used to improve the performance of liquefiable soils. Densification, attained by dynamic deep compaction, vibro techniques or even deep blasting, is one of the most popular and cost effective liquefaction resistance measures. This paper considers the effects of densification irrespectively of the method employed to densify the soil and the conclusions drawn are strictly valid for situations where the soil improvement arises solely from densification, thus excluding techniques employing vibro concrete columns or grouting, for example.

Zen [7] and Mitchell [8], reviewing field applications of this technique, assert that it is usually possible to densify the soil to an average value of relative density of about 80 %, the effectiveness of the different techniques being affected by the amount of fines in the deposit. In terms of the improvement depth, Mitchell [8] recommends that densification of the ground beneath a structure should be extended to the bottom of the liquefiable layer. Zen [7], however, argues that there are no case histories of liquefaction occurring below a depth of 20 m, although the reasons behind that fact are not completely understood. Despite the scientific relevance of this issue, its practical implications are minor, as the costs of densification are much more dependant on the lateral extent of the improvement zone than on its depth. In addition, modern equipment available for vibro compaction makes possible, if required, to densify granular deposits up to a depth of 35 m, according to Moseley [9].

Because of its impact in terms of the densification costs and the alleged major influence on the enhancement of the system performance, the optimum horizontal extent of the improvement zone turned into a main interest in design practice. This problem becomes more critical in cases where the use of the surrounding space is legally or physically restricted. Based on studies conducted to date, most part of them derived from the observation of improved sites during real earthquakes, Mitchell [8] proposes that densification of the soil below the structure should be laterally expanded for a distance measured from the edge of the foundation that is equal to the depth of treatment (d). Thus, according to this author, the improvement zone under a square footing of width B should have a total width defined by:

$$\text{total horizontal dimension of improved zone} = B + 2 \times d. \quad (1)$$

The treatment width resulting from this formula can assume large proportions and involve considerable costs. However, according to Mitchell [8], its use in current practice is usually desirable in face of the poor understanding of the effects of post-earthquake loss of strength instigated by inward migration of excess pore pressure from surrounding non-densified soil as well as the need to provide adequate resistance against sliding of the densified block. Although acknowledging the lack of precise guidelines or analytical procedures to establish the optimum geometry of the densified zone, some authors, such as Kramer [10] and Cooke [11], consider that the lateral extent of the improvement zone may in some cases be reduced to 50-60 % of the treatment depth, the level of performance upgrading achieved significantly depending on the relative density of the densified zone and the width of the supported structure.

Although indubitably establishing the benefits of the use of densification as a liquefaction measure, the case histories analyzed in detail by authors like Mitchell [4] and Hausler [5] show that the performance of improved sites is not always perfect, as settlements and lateral displacements were not completely prevented in some cases. This may be just a result of an inadequate geometry of the improved zone or a consequence of some of the important aspects of the system performance that are commonly ignored. Amongst these, the influence of the structure on the stress-strain behavior and dynamic response of the treated zone and the potential effects of ground motion amplification due to soil improvement, as recently indicated by Hausler [12] or Coelho [13] based on dynamic centrifuge test results, are critical.

OUTLINE OF THE DYNAMIC CENTRIFUGE MODELING

To clarify some of the most ambiguous aspects of the use of densification to mitigate the effects of earthquake-induced liquefaction and eventually optimize its application in practice, especially for bridge foundations, a research project funded by EPSRC has been developed at Cambridge University in collaboration with Mott MacDonald, UK. This paper describes the part of the research employing dynamic centrifuge modeling to observe the behavior of a model bridge built on liquefiable ground during an earthquake. Four centrifuge tests were performed on models similarly built and submitted to analogous earthquake simulations, the geometry of a densified zone created in the soil foundation under the bridge footing being the only factor varying in each test. The characteristics of the model bridge, intended to represent a medium-size deck bridge built on shallow foundations, are shown in Figure 1.

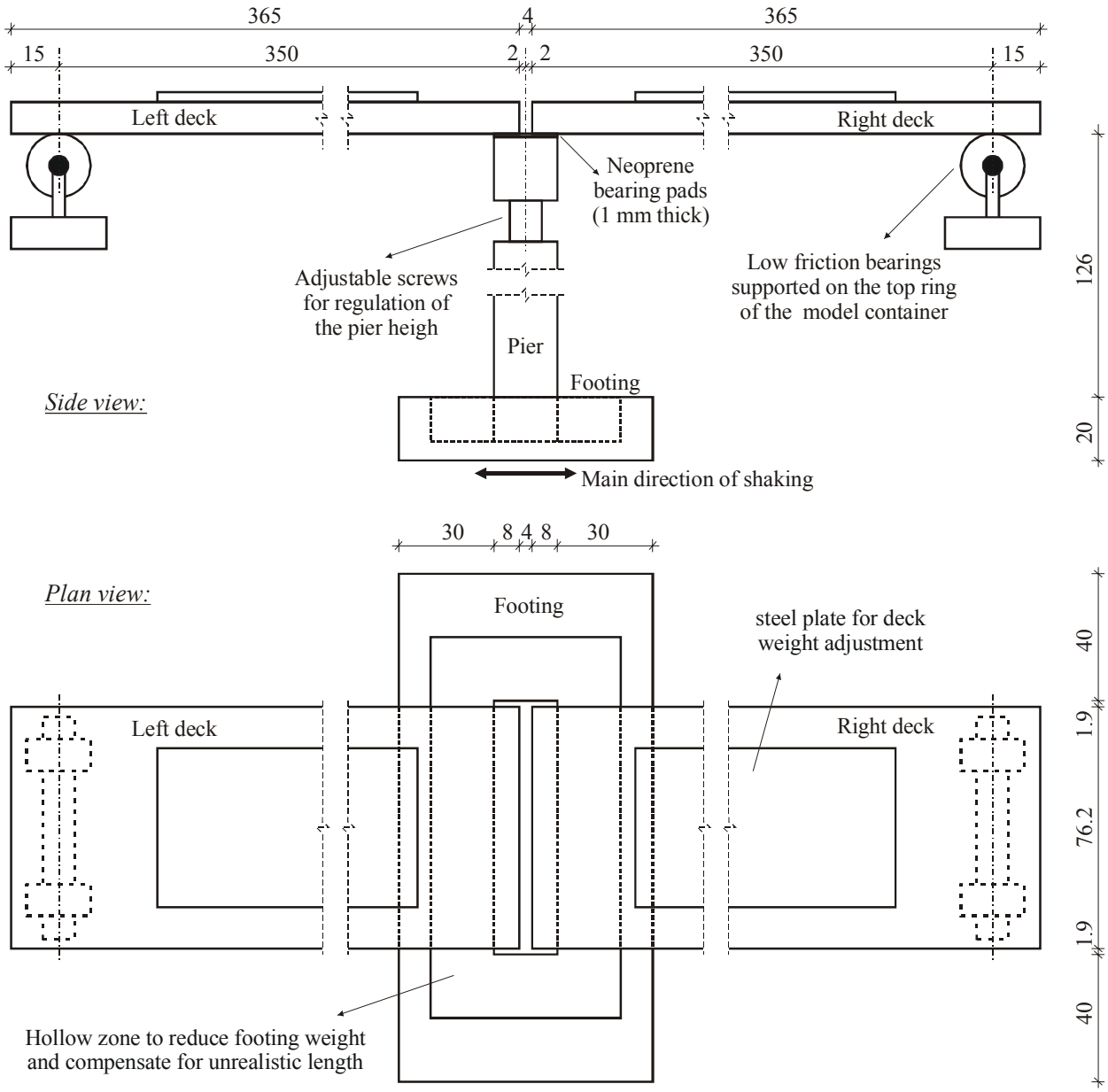


Figure 1- Characteristics of the model bridge (dimensions at model scale, in millimeters).

As described in detail by Coelho [14], the model bridge was built in such a way that the bearing pressure on the foundation base, the height of the centre of gravity and the structural behavior were representative of a common type of medium size bridge built on shallow foundations on liquefiable soil. On the other hand, the model aimed to recreate a 2-D plane-strain condition in the ground, to enhance future use of FE analysis, while avoiding placing the bridge footing near the container walls, to avoid the significant boundary effects that influence the soil behavior in that region (Coelho [15]). Thus, the bridge was designed to have a single footing, placed in the centre of the model, having a length large enough to ensure minimum deformation in the plane perpendicular to the direction of shaking. In the tests under consideration, performed at 50-g centrifuge acceleration, the pressure transmitted through the foundation basis, at prototype scale, is 100 kPa, 20 % arising from the footing, 20 % resulting from the pier and the remaining 60 % generated by the decks' weight. Hence, the bridge's centre of gravity is positioned at about 0.7 of the bridge's total height, measured from the base of the footing. The models were prepared and tested within an Equivalent Shear Beam (ESB) container, having flexible walls intended to replicate the soil dynamic behavior and minimize boundary effects. However, due to the large degradation of soil properties caused by pore pressure build-up and ensuing effective stress reduction, the container cannot exactly match the soil behavior at all times. In an attempt to cut transmission of the shaking to the bridge through the container, the decks were laterally supported on bearings placed on the container's top ring.

The soil deposit was prepared by air dry pluviation of Leighton Buzzard Fraction-E silica sand, different relative densities being achieved by varying the rate of sand pouring. In the cases where a densified block had to be created in the model, under the footing, a box made of thin metallic sheet was employed to temporarily support that zone while the model was built. This box was removed after the loose sand around the temporary support was poured, the eventual effects of disturbance caused by this procedure being compensated by using a temporary box 20 mm larger, in each direction, than the desired size of the improved block. The relative density of the sand in the models was in every case about 50 % in the loose area and 80 % in the densified zone. The bridge footing was embedded in the soil during sand pluviation, the rest of the bridge being placed in its position only before the start of the test. In order to eliminate the time-scaling conflict between dynamic and diffusion phenomena, the models were saturated with a 50 cSt solution of methylcellulose, as explained in more detail by Coelho [13].

The instruments' positions, schematically represented in Figure 2 and detailed in Table 1, were maintained in all the tests, to observe the influence of the geometry of the densified zone on the system behavior. The size of the improved block, which varies from test to test, is not displayed in the figure.

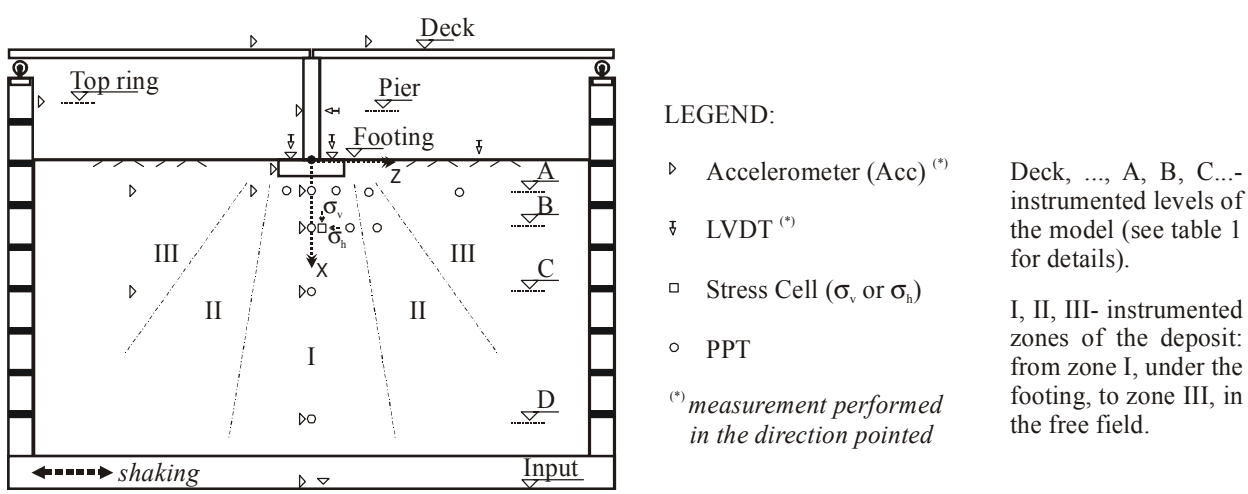


Figure 2- Schematic of the instruments location in the centrifuge model.

Table 1- Coordinates and performance of the instruments placed in the models (prototype scale)

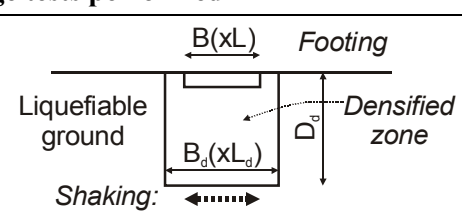
LVDTs, PPTs and Stress Cells							Accelerometers						
Instrument Reference [a]	Target Position [a,b]		Malfunctions[d]				Instrument Reference [a]	Target Position [a,b]		Malfunctions[d]			
	x (m)	z (m)	1	2	3	4		x (m)	z (m)	1	2	3	4
LVDT Foot-V^{left}	-1.3	0					Acc Deck^{left}	-3.8	-6.3				
LVDT Foot-V^{right}	1.3	0					Acc Deck^{right}	3.8	-6.3				
LVDT Pier-H	0.5	-3.0					Acc Pier	-0.5	-3.0				
LVDT FreeField	8.5	0	x				Acc Foot-H	-2.0	0.5	x		x	
PPT I-A^{left}	-1.5	2.0					Acc Foot-V^{left}	-1.1	0			x	
PPT I-A^{centre}	0	2.0					Acc Foot-V^{right}	1.1	0				
PPT I-A^{right}	1.5	2.0					Acc I-A	0	2.0				x
PPT II-A	3.5	2.0	x				Acc II-A	-3.5	2.0			x	
PPT III-A	9.0	2.0	x			x	Acc III-A	-9.0	2.0				
PPT I-B^{centre}	0	4.0					Acc I-B	0	4.0				
PPT I-B^{right}	2.0	4.0					Acc I-C	0	8.0		x		x
PPT II-B	4.0	4.0					Acc III-C	-9.0	8.0				
PPT I-C	0	8.0					Acc I-D	0	16.0				
PPT I-D	0	16.0					Acc Top Ring	-16.9	-3.0	x			
Str. Cell I-B(σ_v)	0	4.0					Acc Input-V	0	18.0 [c]			x	
Str. Cell I-B(σ_h)	0	4.0	x				Acc Input-H	0	18.0 [c]				

[a] see Fig. 2 for axis origin; [b] prototype scale (50-g); [c] total depth of the deposit; [d] in each test (1 to 4)

Three different geometries for the improvement zone were tested and compared to the behavior of the benchmark test where no ground improvement was carried out. Table 2 summarizes the characteristics of the four centrifuge tests performed. As the table shows, no densified zone was created in centrifuge test CT-0B, which pretends to establish the performance during the earthquake simulation of a bridge built directly on liquefiable ground and set up a term of comparison for the subsequent tests including ground improvement. Tests CT-1B, CT-2B, and CT-3B were carried out in similar conditions to those used in the benchmark test, except for the fact that some ground improvement was carried out under the bridge footing. In every case the densified zone had the same depth as the layer of loose sand and the same dimension in the direction perpendicular to the direction of shaking. The fact that this dimension of the densified block (200 mm) is slightly smaller than the total width of the container (254 mm) arises from the need to reduce the boundary effects that would probably affect the deformation of the footing. The only difference between the three tests including ground improvement is therefore the width of the improved zone measured in the direction of the shaking. The width of the densified block considered in the centrifuge tests ranges from a width equal to that of the footing, B (4 m), to three times the footing's width, 3xB. If the minimum width tested represents the smallest amount of improvement that would certainly be accepted in practice, the maximum width is a result of the restrictions to the container's size. The attempt to model larger densified zones would probably result in inaccurate results, due to the little volume of liquefiable ground surrounding the densified block. Nevertheless, larger improvement zones may in the future be investigated by numerical methods calibrated with the results presented herein.

Table 2- Characteristics of the centrifuge tests performed

Centrifuge Test Designation	Geometry of the Densified Zone [a]			
	B_d (m)	L_d[b] (m)	D_d (m)	B_d/B
CT-0B [c]	0	0	0	0
CT-1B	4	10	18	1
CT-2B	8	10	18	2
CT-3B	12	10	18	3



[a]Prototype scale (50-g); [b]Slightly smaller than box width to minimize boundary effects); [c]Benchmark test

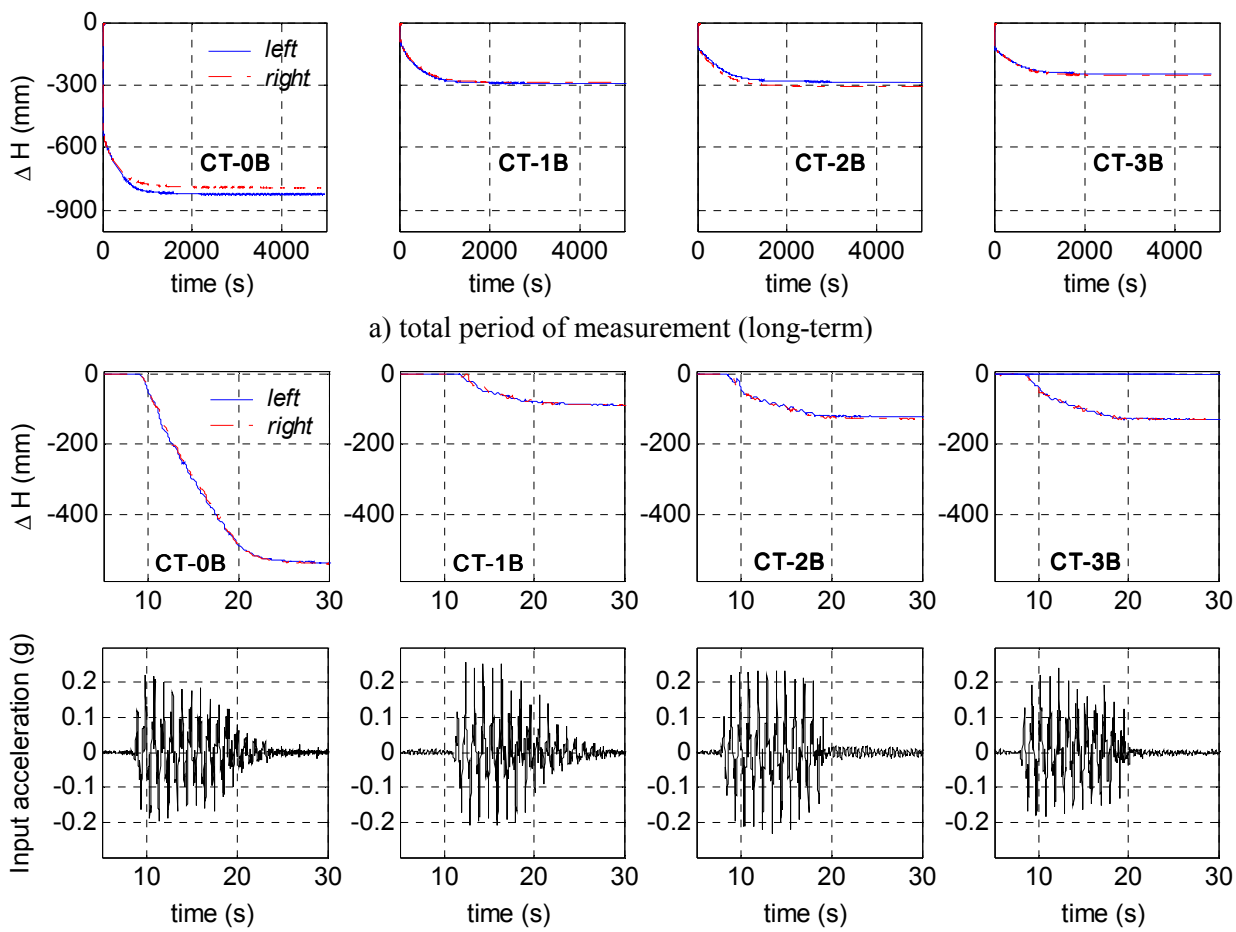
RESULTS PRESENTATION AND DISCUSSION

All the centrifuge models were submitted to a similar earthquake simulation, designed to replicate a relatively strong real earthquake motion, containing acceleration peaks around 0.2-g, lasting 10 seconds and having a fundamental frequency of 1 Hz. This section presents the data recorded in the centrifuge tests, the behavior of the models being successively compared in terms of the footing deformations, the pore-pressure generation and dissipation in the granular ground and the system accelerations. The section ends with a summary of the most important features of behavior captured by the experimental program.

Footing deformations

The major concern of designers considering a bridge built on liquefiable ground is to limit the footing deformations in order to avoid the bridge total or partial collapse during an earthquake. The deformations requiring cautious control include vertical and horizontal displacements and rotation of the footing, whether they result from the dynamic loading of the system, the soil densification following dissipation of the earthquake-induced excess-pore-pressure or the ground softening due to effective stress reduction.

Figure 3 compares the vertical deformations of the footing measured in the tests (a), magnifying the deformations occurring during the short period of shaking (b). The first conclusion is that, irrespective of the geometry of the densified block, the settlement of the footing is largely reduced, by a minimum of more than 60 %. As expected, densification proves to be an efficient liquefaction resistance measure.



a) total period of measurement (long-term)

b) period of the earthquake simulation (short-term)

Figure 3- Settlement of the footing measured by the left and right LVDTs (prototype scale).

Although current building practice uses massive densified zones around a structure to protect it from the effects of earthquake-induced liquefaction, the results presented in Figure 3-a show no substantial settlement drop as the width of improvement increases from 1 to 3 times B. The average final settlement of the footing built on the widest densified block is, in fact, just about 14 % lower than that observed in test CT-1B. The use of large densified zones can then only be understood in face of the imperative need to reduce the footing settlement, as it was still as large as 25 cm in test CT-3B. It should be noted that the trend shown by the results suggests that further settlement reduction may be achieved with wider densified zones than those tested, though the relative gain in performance would probably be minor.

More surprising than the finding previously described is perhaps the fact that the increase of the width of the densified zone results in larger settlements of the footing occurring simultaneously with the earthquake. As displayed in Figure 3-b, the so-called dynamic settlement observed in tests CT-2B and CT-3B exceeds by, respectively, 39% and 47%, the value measured in tests CT-1B. It should be noted, however, that the dynamic settlement of the footing built on densified ground is in every case just a very small part of that observed in the benchmark test, CT-0B, where no densification was carried out.

Attending to the preliminary conclusions drawn, expanding the width of improvement results primarily in a reduction of the post-earthquake settlement, whereas the portion of the settlement occurring during the earthquake may conversely increase. The plots shown in Figure 4 assist in identifying the influence of the geometry of the densified zone on the magnitude and relative importance of each contribution to the total settlement. The analysis of the absolute values of the dynamic and post-earthquake components of the settlement show that a minimum value of the dynamic settlement occurs in the model with the narrowest densified zone, the reason for the final settlement to decrease as the width of improvement grows being the smaller post earthquake settlement. Still, the comparison of tests CT-1B and CT-2B suggests that the reduction of the post-earthquake settlement may occasionally be insufficient to compensate for the increase of the dynamic settlement arising from a larger width of improvement. In test CT-2B, however, the dynamic settlement may be slightly overvalued due to an input motion that contains some more high acceleration peaks than in the other tests. Finally, the relative values of each settlement component prove the importance of the post-earthquake component and confirm that the better-than-expected performance observed in test CT-1B is mainly based on a restricted dynamic settlement.

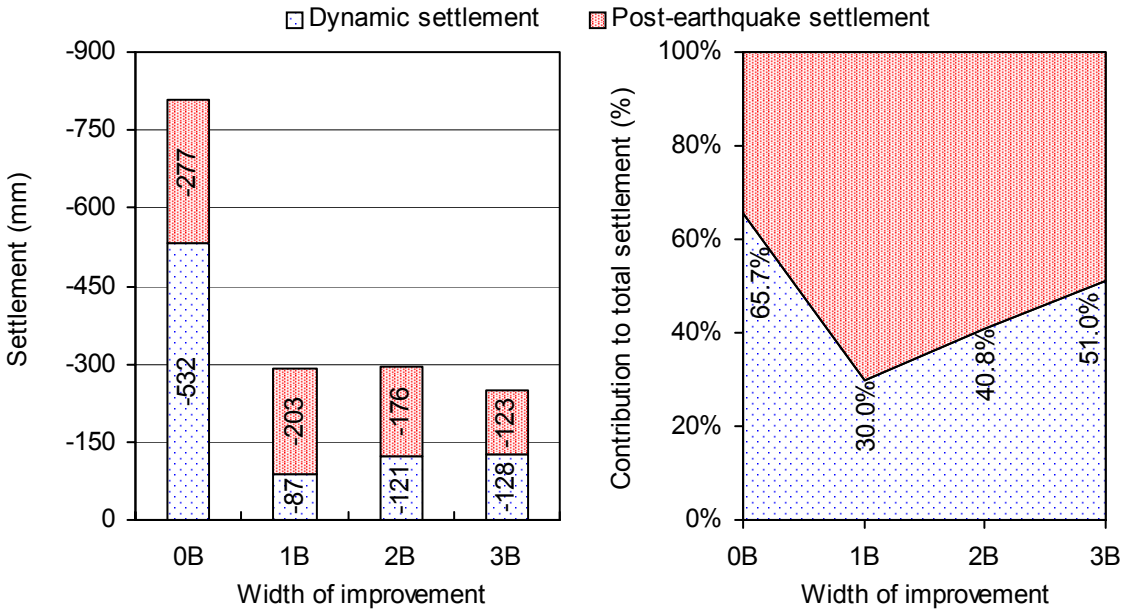


Figure 4- Absolute and relative values of each contribution to the total settlement (prototype scale).

The rotation of the footing induced by the liquefaction of the ground during an earthquake is a major concern, as a relatively small rotation may lead to the collapse of the decks, especially in tall bridges. Figure 5 displays, for each test, the final values of the rotations and those observed by the end of the shaking. The maximum total rotation, 0.77 ^{o}, observed in test CT-0B, is not catastrophic, though it may cause significant problems in bridges where the decks stand at high elevations. It should be noted, however, that the conditions employed in the test, including extremely uniform ground conditions, loads perfectly centered in the pier and a ground input motion with a relatively symmetrical time history of accelerations, may be more severe in a real situation, the consequence being a much higher rotation of the footing. It should also be noted that the rotation observed in test CT-2B is larger than could be expected in face of the results of tests CT-1B and CT-3B. The actual characteristics of the earthquake simulation applied in this case may eventually justify this result. More significant than the values of the footing total rotation is the realization that the most significant part of the rotation occurs after the end of the earthquake. The most striking piece of behavior was detected in the test with no ground densification: only about 3 % of the total rotation, mounting up to 0.77 ^{o}, occurs during the earthquake, the remaining 97 % taking place after the end of the shaking. This result may explain the fact that failures of bridges due to earthquake-induced liquefaction are usually described in case histories as a slow process, people having time to evacuate the bridge before collapse. The same qualitative behavior was presented by the models including ground improvement, though the relative magnitude of the rotation occurring simultaneously with the earthquake tends to increase significantly with the width of the improvement.

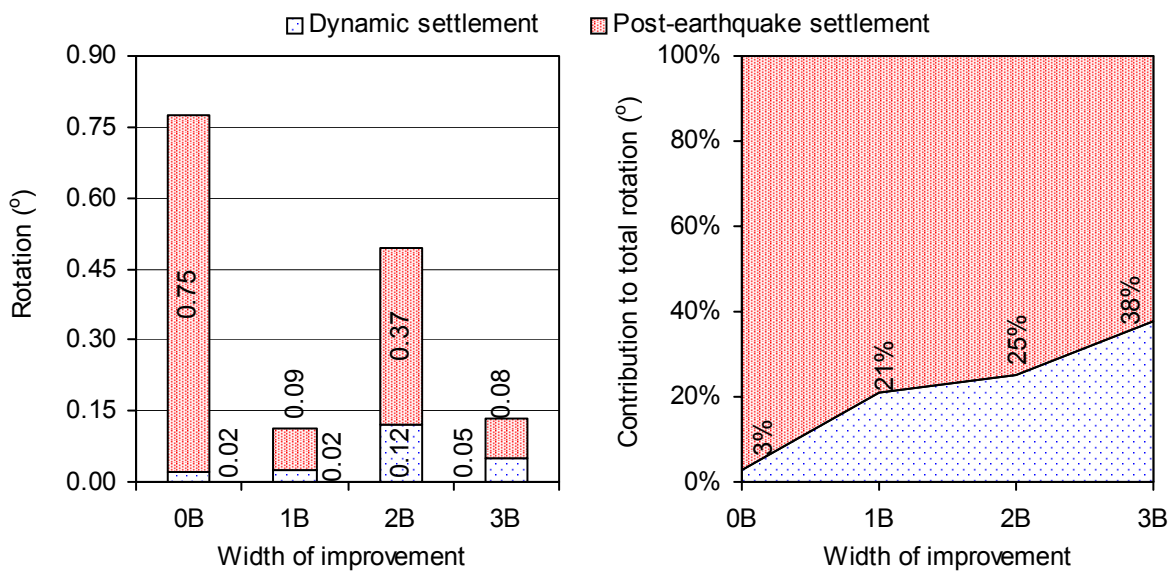
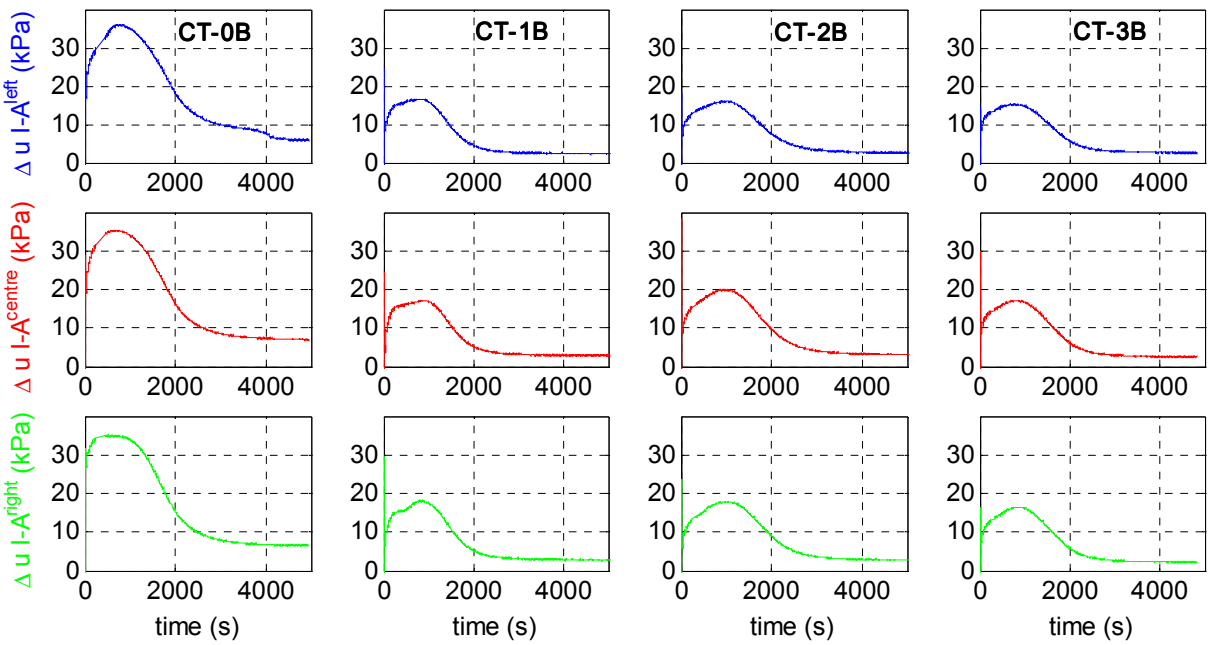


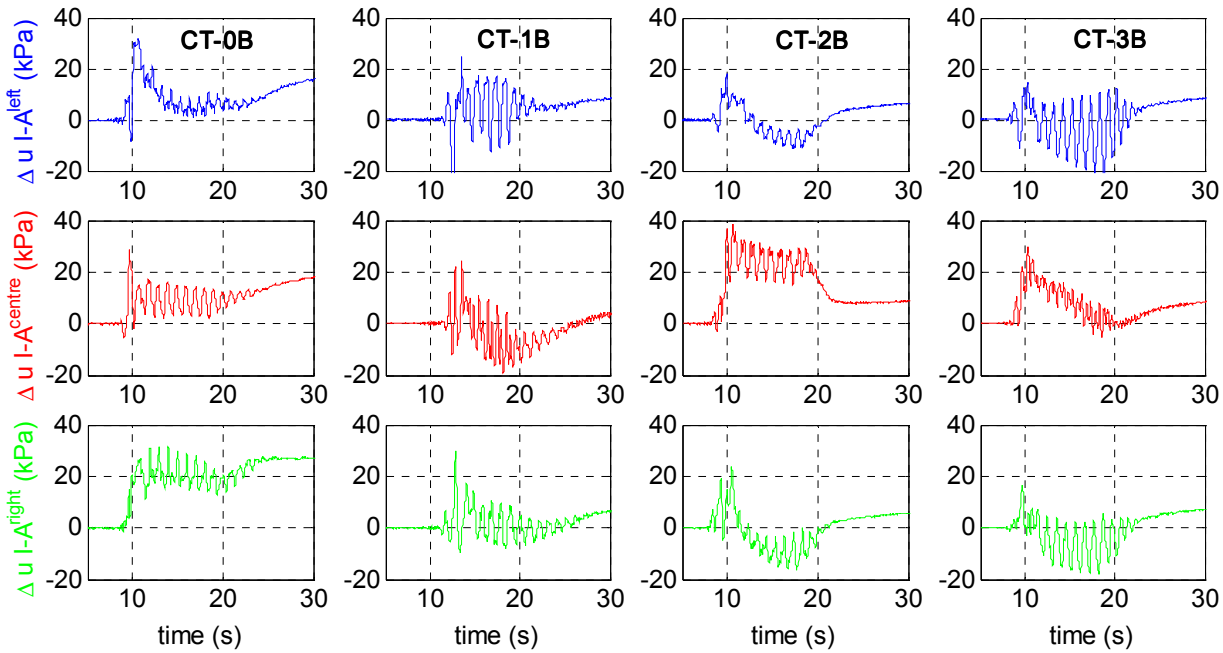
Figure 5- Absolute and relative values of each contribution to the total rotation.

Pore-pressure generation

The post-earthquake deformation of the footing, characterized by its average settlement or rotation, was shown to play a very important role in terms of the performance of the foundation. This result is contrary to the observations of Coelho [6] in uniform deposits of saturated sand during earthquake loading: according to the experimental results, the post-earthquake settlement is almost irrelevant, regardless of the relative density of the sand. This contradiction suggests that the post-shaking deformation of the footing is not solely a result of the compression of the grains following dissipation of the earthquake-induced excess-pore-pressure. Therefore, a different mechanism should account for the major part of the post-earthquake footing settlement, whose magnitude clearly exceeds that of a free-field condition. The analysis of Figure 6, which presents the excess-pore-pressure measured under the footing during and after the shaking, may enlighten the mechanism involved in the post-earthquake deformation.



a) total period of measurement (long-term)



b) period of the earthquake simulation (short-term)

Figure 6- Excess-pore-pressure under the footing, at level A, from left to right (prototype scale).

The long term readings plotted in Figure 6-a show that in all the models and in every position considered the maximum value of excess-pore-pressure occurs 10 to 15 minutes after the earthquake ends. In the models having a densified zone, the magnitude of maximum excess-pore-pressure and the period of time near maximum values of pore pressure are smaller than in test CT-0B, both seeming to decay with the width of improvement. Considering that, at level A, the free-field vertical effective stress is just under 20 kPa and the stress arising from the footing significantly increases that value, it seems unlikely that the effective stress under the footing approaches zero at any time during or after the shaking. Still, it is evident that the most critical semi-permanent situation in terms of the soil mechanical properties occurs

not during the shaking itself but long after its end. This is a consequence of the pore pressure migration from areas where the excess-pore-pressure generated is higher and causes further degradation of the soil mechanical properties and additional deformations of the footing. Wider densified zones can then reduce the effects of the post-earthquake deformations as a result of the ability to reduce the flow of water due to its lowest permeability and because the same reduction of effective stress should not cause the same degradation of mechanical properties in a soil in a loose or dense conditions.

Some interesting conclusions can also be drawn from the dynamic (short term) excess-pore-pressure generation plotted in Figure 6-b. The first one is that, under the footing built on loose sand (CT-0B), it is possible to observe soil dilation after a certain number of cycles, which is surely very important in terms of the bridge stability as it limits the risk of a bearing capacity failure during the shaking. The same is not valid after the end of shaking, as the pore pressure rises again and remains at peak values for some time, which may induce a bridge failure due to the bearing capacity reduction. The second important conclusion is that a narrow densified zone can mobilize soil dilation very effectively in the densified block, under the footing. The fact that, under the footing centre, the geometry of the improved ground in test CT-1B results in a more dilative behavior than in any other case is probably one of the reasons why the performance of such a narrow densified zone during the period shaking was so satisfactory.

The issue of the post-earthquake excess-pore-pressure migration is also a concern at deep levels, as attested by Figure 7. In fact, except at a depth of 16 m (level D) in test CT-0B, the pore pressure attains its peak some time after the earthquake ends. In general, the duration and relative importance of the post-earthquake pore-pressure rise decreases with depth, once the structure does not induce so much dilative behavior during the shaking at deeper levels. Furthermore, the phenomenon affects deeper levels when a densified block is used, giving an additional reason for the problem to be seriously considered in design.

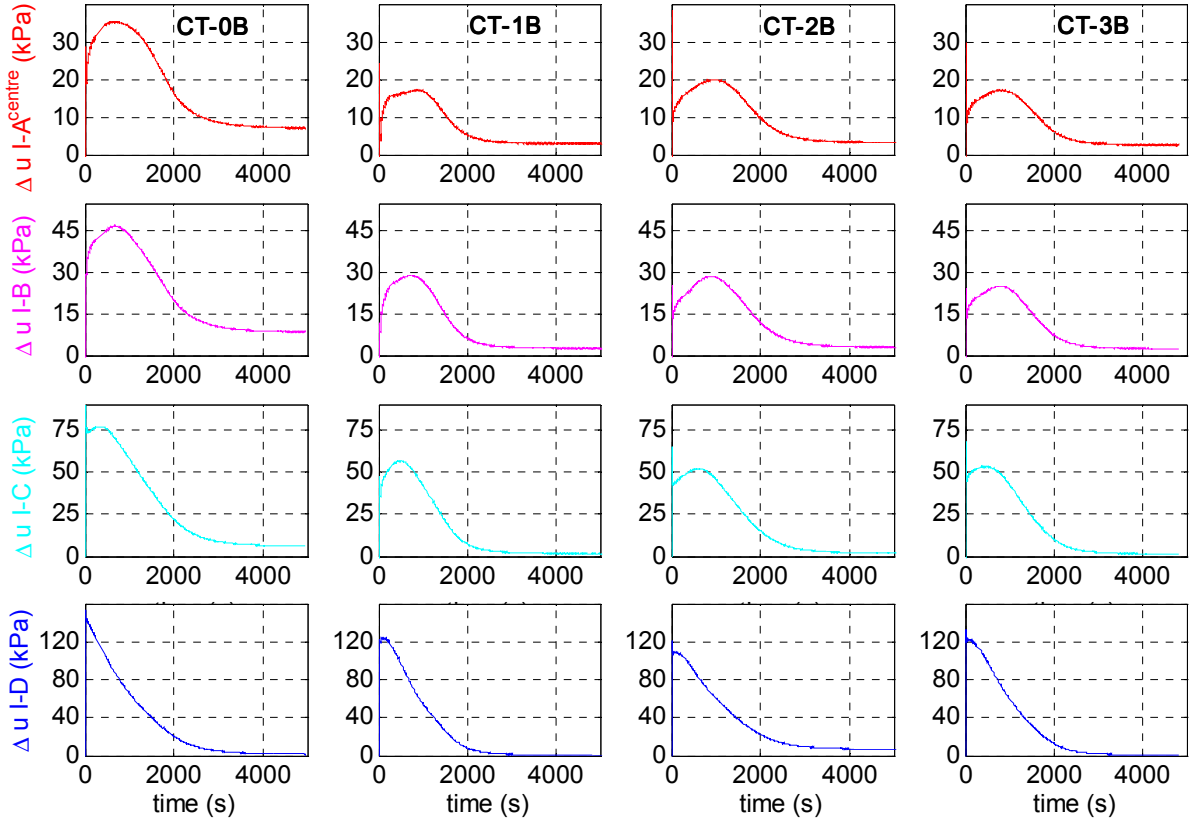


Figure 7- Excess-pore-pressure, under the footing centre, at increasing depth (prototype scale).

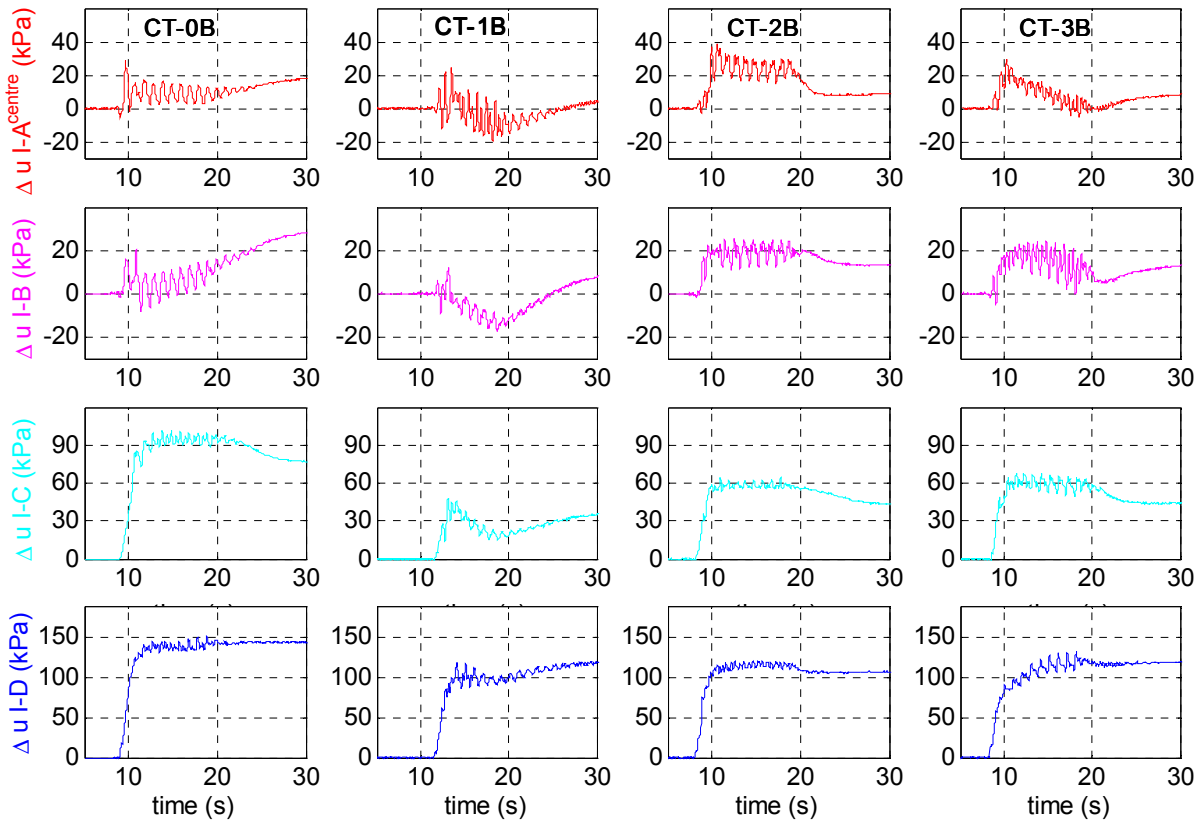


Figure 8- Dynamic excess-pore-pressure, under the footing centre, at increasing depth (prototype).

Propagation of accelerations

Centrifuge-based results presented by Coelho [6] reveal that, in a free-field condition, significant amplification of the peak accelerations occur during the upwards propagation of ground motion through a dense deposit of sand. As a corollary of that work, the authors expressed the fear that densifying the ground under a bridge footing could result in larger seismic energy transmitted to the structure, which would enlarge the footing dynamic deformations and increase the risk of structural failure.

Figure 9 plots the horizontal accelerations measured in the deposit and the bridge, at different levels, for all the tests performed except CT-2B. This test was disregarded due to the malfunction of some important accelerometers and to improve the figure's clarity. The first main conclusion is that in any case was observed an amplification of the peak accelerations as large as the one reported by Coelho [6] in a free-field condition, proving that the structure reduces the potential for large amplification of accelerations in dense sand. Still, some important differences subsist between the tests, except at deep levels like level D, where there is hardly any difference between the time histories of accelerations, which are in all cases similar to the input. At level B, where the footing effects are greater, accelerations are greatly attenuated after the first cycle in test CT-0B. The magnitude of this phenomenon is much less relevant in test CT-3B and has an intermediate value in CT-1B. Parallel study is unfeasible at level A due to an instrument failure in test CT-3B. In the footing and pier, once more, the time histories of accelerations are noticeably different: the magnitude of the attenuation of peak accelerations after the first cycles is much larger in test CT-0B whereas, in the other cases, it decreases with the width of improvement. Moreover, the spikes of maximum peak accelerations in the footing and pier last considerably longer in test CT-3B, indicating that more energy is transmitted to the structure built on a densified block. This is confirmed by Figure 10 that plots the FFT analysis of the period from the 3rd to the 10th loading cycles in tests CT-1B and CT-3B and could explain why the dynamic settlement observed in test CT-3B clearly exceeds that of CT-1B.

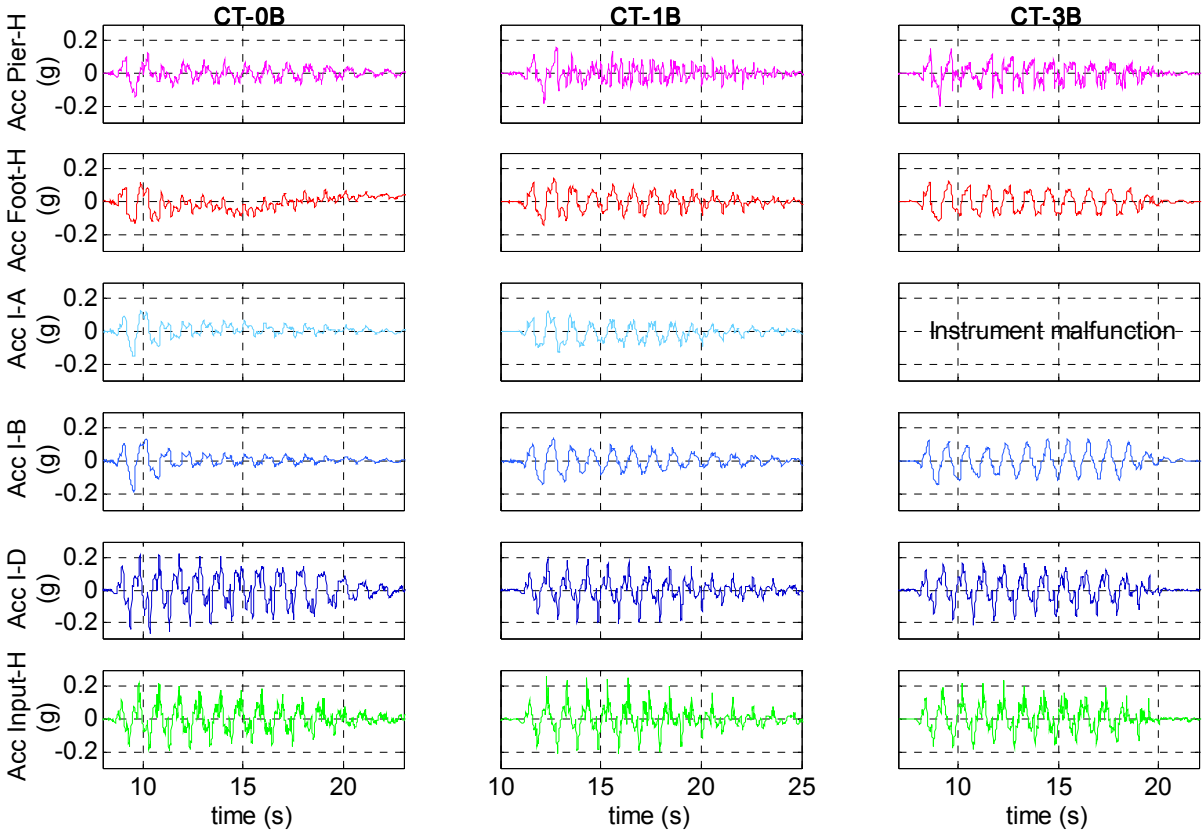


Figure 9- Horizontal accelerations at the foundation and bridge at different levels (prototype scale)

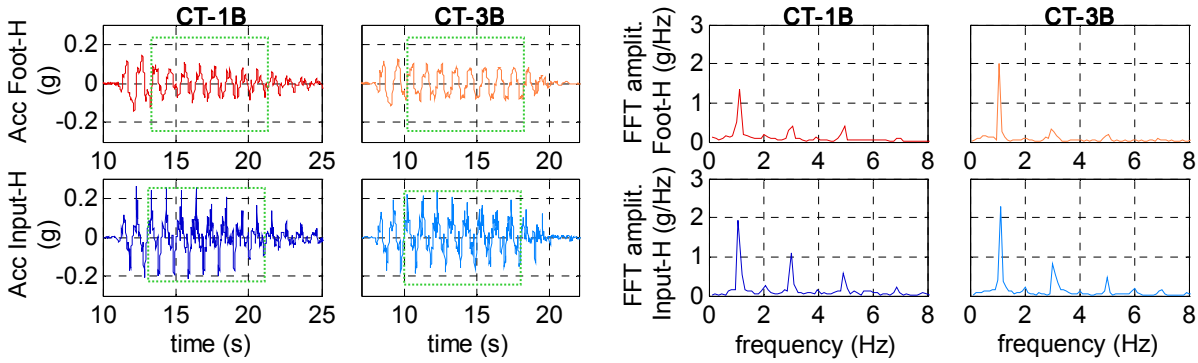


Figure 10- Influence of the improvement width on footing accelerations (prototype scale).

Main experimental achievements

The results presented confirm ground densification as a valuable liquefaction mitigation measure, its success being based on two distinct factors. Firstly, by radically reducing dynamic deformations, due to the superior ability of dense sand to exploit the soil dilation generated under the footing to resist shaking. Secondly, though much less efficiently, by limiting post-earthquake deformations, as the potential for pore-pressure migration from adjacent areas towards the region under the footing is slightly inhibited. This phenomenon proved to be decisive, as its significant magnitude determines the global performance of a specific geometry of the densified zone. In fact, the better performance of wider improved zones seems merely a result of a larger reduction of the post-earthquake deformations caused by soil softening induced by pore-pressure migration. Conversely, the wider the densified block, the more energy will be transmitted to the structure and the largest dynamic settlement will take place. Therefore, this aspect of the behavior must be considered in optimization analysis of the improvement width and structural design.

CONCLUSIONS

A centrifuge-based research program was carried out at Cambridge University to characterize the behavior of medium-size bridges built on shallow foundations on saturated deposits of loose sand during seismic events. The use of densification as a liquefaction resistance measure was assessed by considering different geometries for a densified zone under the footing. This research is encouraged by the need to optimize the use of densification to mitigate liquefaction effects and to enhance the safety of these structures during an earthquake, as its failure has devastating short and long-term consequences.

This paper discusses the results of four centrifuge tests, which compare the behavior of a deck-bridge built on a non-improved ground with that of the same bridge built on densified zones of varying width. The model bridge represents a prototype transmitting a pressure of 100 kPa through a single footing, the decks seating 6.3 m above the ground and the centre of gravity being positioned at about 0.7 of bridge's total height. The densified block extended all through the liquefiable layer, its width varying between 1 and 3 times the footing width. The experiments provide clear evidence of the serious consequences of soil liquefaction on bridge foundations when no improvement is carried out and enlighten the different effects of the existence of a densified zone on the behavior of the system. According to the results:

- in the zone of influence of the footing, the vertical effective stresses never reach near-zero values during the shaking, as the soil dilates after a small number of cycles, even in the case of loose sand;

- densification results in a large reduction of the footing settlement and rotation, which is decisive in terms of the bridge stability; the general performance is only slightly enhanced as the width of the densified zone increases, the deformations being still sizeable even for the largest width considered;

- the success of densification as a liquefaction resistance measure is a result of the combined effects of an efficient use of the soil dilation generated under the footing during the shaking, which reduces the dynamic settlement, and a slight reduction of the effects of post-earthquake pore-pressure migration;

- wider densified zones are more effective in reducing the post-earthquake settlement, caused by excess-pore pressure migration, while narrow densified zones result in smaller dynamic settlement, most probably as a consequence of the larger seismic energy transmitted to the structure;

- in every case, the largest part of the footing total rotation tends to occur after the end of the shaking;

- although the widest densified zone provides the best overall solution in terms of deformations, it also results in a more severe inertial loading of the structure, whose effects should be pondered in design;

- the narrowest densified zone showed an unexpectedly good performance, which is mainly based on a great reduction of the dynamic deformations; this is a result of the combination of the very significant soil dilation generated under the footing and the lower seismic energy transmitted to the structure;

- the effects of the structure are felt at deeper levels when densified zones are used, resulting in soil dilation and consequent excess-pore-pressure migration occurring at higher depths; this emphasizes the importance of the phenomena when using densification to mitigate liquefaction effects.

The results presented herein show that the understanding on the use of densification as a liquefaction resistance is still limited and suggest that the use of very wide densified zones in current practice may not be entirely justified. The good performance shown by a densified zone as narrow as the footing should be explored in the future, both in research and design practice, to optimize the use of densification.

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