Chapter 15

LEAP-UCD-2017 Centrifuge Test at IFSTTAR

Sandra Escoffier and Philippe Audrain

Abstract In the framework of the LEAP 2017 exercise, two dynamic centrifuge tests on a gentle slope of saturated Ottawa-F64 have been performed at the IFSTTAR centrifuge. These tests were conducted in parallel with other tests performed in nine other centrifuge centers. The objectives were to compare the experimental results, e.g., effect of the experimental procedure or of test parameters on the results, and to provide a database for numerical modeling. In this framework, all the tests were performed on the same prototype geometry and the first base shaking was a 1 Hz ramped sine with an effective amplitude of 0.15 g at the prototype scale. Among some of the tests performed in the various centrifuge facilities, several parameters were modified, such as the density and the second input motion, in order to study their impact on the slope behavior when it is subjected to base shaking. This paper details the procedure followed at the IFSTTAR center for the buildup of dense sand and medium loose sand models and the deviations from the specifications provided by the leaders of the LEAP-UCD-2017 program. The main deviations are, for both tests, the absence of the measurement of the saturation degree and, for the first, the characterization of the soil properties with CPT test and bender element measurements during the test. Some results of the tests performed are briefly presented such as the acceleration, pore pressure buildup, and the deformation of the slope surface.

15.1 Introduction

Actual researches in numerical modeling on liquefaction phenomena, for instance, advanced numerical techniques based on multiscale approach in large deformation (Callari et al. 2010), highlight the need of experimental database for the calibration and the validation processes. In this framework one of the objectives of the LEAP-UCD-2017 research program is to provide high-quality laboratory and centrifuge test

S. Escoffier (✉)
IFSTTAR, GERS, SV, Bouguenais, France
e-mail: sandra.escoffier@ifsttar.fr

P. Audrain
IFSTTAR, GERS, GMG, Bouguenais, France

data. A total of ten centrifuge teams were involved in this experimental research work.

Following the model specifications (Kutter et al. 2019), each team has performed a series of dynamic tests on a gentle slope of saturated Ottawa F-65 sand. The objectives of the specifications were to minimize the discrepancies between the experimental procedures followed in each centrifuge team in order to evaluate the quality of liquefaction centrifuge tests and the effects of procedure deviations on the obtained results through cross testing. In addition to this repeatability step, additional tests with different densities and with different second and eventually third base shaking were performed.

Two tests were performed at the IFSTTAR center with the same base shakings but with different densities. In this paper, the procedure and some of the results obtained on the dense sand model, S02, and on the medium loose sand model, S03, are presented.

15.2 As Built Model

15.2.1 Soil Material and Placement of the Sand by Pluviation

The soil selected for the LEAP-UCD-2017 centrifuge experiments is Ottawa F-65, a clean sand with less than 0.5% of fines. Control quality checks on the grain characteristics (granulometry and maximum and minimum densities of the sand delivered) were performed following the model specifications established for this project. The maximum and minimum densities were measured based on the modified Lade et al. (1998) method and the modified ASTM D4254 method, respectively. In addition, the French NF P94–059 procedure was used for both. The average values for the maximum densities are 1737 and 1775 kg/m³ for the NFP and the modified Lade et al. method, respectively. The average values for the minimum densities are 1509 and 1472 kg/m³ for the NFP and the modified ASTM methods, respectively. From the LEAP-UCD-2017 Version 1.0 model specifications, the average values of all the tests that have been performed with different methods are 1475 and 1756 kg/m³ for the minimum and the maximum densities, respectively. Due to the dispersion that has been observed, the values obtained by IFSTTAR are in the range of the previous ones (data are available in Kutter et al. 2019).

Figure 15.1 presents the granulometry curves that were obtained. The D10, D30, D50, and D60 are 0.139, 0.196, 0.226, and 0.247 mm, respectively. The D30, D50, and D60 values are somewhat higher than the one given by Kutter et al. (2017) due to the use of additional sieves for the sieves with an opening between 0.125 and 0.250 mm.

The Ottawa sand was dry pluviated through a sieve attached to the bottom of the hopper of the IFSTTAR dry pluviation system (Ternet 1999). The initial target densities for S02 and S03 tests at the IFSTTAR center were 1703 and 1651 kg/m³, respectively. Due to the lack of data concerning medium dense sand, the target

Fig. 15.1 Granulometric curves of the OTTAWA sand obtained at the IFSTTAR center

density for the S03 test has been increased to 1599 kg/m^3 . To obtain these request densities the specifications concerning the opening size of the sieve were not followed (e.g., for S02 a sieve with an opening of 1.2 mm and for the flow restriction the recommended design was three slots of 1.2 mm width with a center to center spacing of 26 mm).

First, due the standard sieves in France, the selected sieve for the calibration has an opening of 1.25 mm. In the case of the dense sand, following the recommended design for the slots, a first air pluviation test was performed but the sand flow stopped rapidly. As suggested in the specification document, the clogging effect could be due to the humidity of the room that was measured at 65%. Consequently, the geometry of the open part was modified. The width of the slots was increased up to 6 mm that corresponds to three-mesh width of the sieve and the center to center distance between the opening slots was 22 mm. Furthermore, due to the container width (i.e., internal width 200 mm) the slot opening was 240 mm long. Finally, a drop height of 50 cm and a horizontal velocity of 48 mm/s of the hopper were selected to obtain a density of $1696 \pm 4.22 \text{ kg/m}^3$ (three measurements).

In the case of the medium loose sand, the minimum density that can be obtained with the pluviation system was $1624 \pm 3.8 \text{ kg/m}^3$. It has been obtained with two slots of 2.9 cm width that correspond to 15 meshes and a center to center distance of 4.8 cm.

During the calibration procedure of the pluviation, the measurement of the density of the sand was performed using cylindrical density boxes (approximate volume 390 mm^3). Once the pluviation was made the top surface of the density box was leveled using a specific tool to avoid unintentional densification of the sand.

Fig. 15.2 Rigid steel box especially built for the LEAP project at the IFSTTAR center and placement of the rigid box inside the fixed ESB container

15.2.2 *Rigid Container Configuration and Sensor Layout*

The specifications for the model are a gentle slope (5°) with a length of 20 m, a width of at least 9 m, and a top height of the slope of 4.875 m at the prototype scale. Not one container available at the IFSTTAR center enabled the buildup of such slope geometry. Consequently, a rigid steel container was especially built for the LEAP project (Fig. 15.2). The inner dimensions are 400 mm $\times$ 200 mm $\times$ 200 mm (L $\times$ W $\times$ H) which enable to build the gentle slope at the request prototype size for a centrifuge acceleration of 50 g.

Due to the shaker configuration, this rigid container was fixed through 12 screws into an ESB. In addition, the shearing movement at the extremity of the ESB container was blocked by four rods (one at each corner, Fig. 15.2). Before the first test, a series of drives was performed in order to obtain the time histories of the commands to the servo valves of the shaker (i.e., drives) that correspond to the request base shakings. This preliminary test that was performed on the two-container assembly highlighted the presence of non-negligible high frequencies not far from the predominant one (i.e., 50 Hz at the model scale). Consequently, a sufficient amount of sand was put in place between both containers to remove these high frequencies that should be due to resonance phenomena of the embarked system (Fig. 15.2).

The relative rigidity of cables of the pore pressure transducers (Druck PDCR81) induces some problems for their positioning during the sand pluviation. In order to facilitate their positioning and limit the length of cables in the sand mass, cable glands were inserted on one of the lateral side of the container (Fig. 15.2). This configuration enables the placement of the pore pressure sensors inside the container

with the request length of cable before the starting of the pluviation. Sensors were taped up on the lateral side of the container and put in place once the request level of sand was pluviated.

Figure 15.3 details the model scale dimensions and the sensor layout.

The required accelerometers and pore pressure sensors (AH1 to AH4, AH12, AH11, AV1, AV2, P1 to P4, and P9 and P10) were all put in place during the pluviation. In order to facilitate the positioning of the sensors, the pluviation has also been calibrated to drop off a layer of approximately 1 cm thick each back and forth movement of the hopper, which is the model distance between each level of sensors of the central array (AH1 to AH4 and P1 to P4). In addition, two other accelerometers and pore pressure sensors (S02, P5 and P7; S03, P6 and P8) were installed to complete the instrumentation.

The location in the coordinate system defined in Fig. 15.3 was measured during placement and during excavation after the test. The difference between as-built and “after test” coordinates of the sensors embedded in soil could be partly due to errors, such as inaccurate measurement that was performed using a steel rule, inadvertent tugging on wires after placement or during excavation, and displacement of the sensors relative to the liquefiable soil. These location data are analyzed in the Sect. 15.4.2. of this paper and displacement vectors are provided.

A pair of bender element has also been installed. The pair of bender elements is the same as the fixed one presented by Brandenberg et al. (2006). However due to problem of amplification, the measurement of the shear wave velocity has not been performed in the model S02. In addition, as requested in the specifications no CPT test was performed during this test. These two main deviations from the model specifications do not enable the characterization of the soil properties for the S02 models. In the case of the S03 test performed on medium loose sand, both CPT tests and bender element measurements have been performed.

The shear wave velocity measurements were made just before and after each base shaking. For each soil state, 6 series of 36 measurements were made (2 series of measurement for the 3 selected input frequencies 5, 7.5, and 10 kHz). The use of several frequencies and the staking of data increase the reliability of the determined values. The CPT device used was developed and built at the University of California, Davis, and had an external diameter of 6 mm at the model scale. CPT measurements were made to characterize the initial state and the soil and the state after each base shaking. Due to the experimental configuration, the CPT was removed from the centrifuge before each shaking.

In the case of the IFSTTAR 1D shaker the direction of the solicitation is parallel to the axis of the centrifuge (Chazelas et al. 2008). In order to keep constant the radius between the surface of the soil in a transverse cross section and the center of rotation of the centrifuge, the surface should have a curved shape in the direction perpendicular to the base shaking. The distance between the axis of rotation of the centrifuge and the center of the soil surface is 5.063 m. Considering that the inner dimension of the container’s width is 0.2 m, the difference in height between the midpoint and the corresponding point at the lateral side should be 1 mm. As this

Fig. 15.3 Model scale dimension and sensor layout of the test performed at IFSTTAR (dimensions are in mm at the model scale)

value is in the range of precision of the leveling of the surface, the soil surface was not curved in the Y direction.

In addition to the sensors, surface markers were placed in a grid pattern at the surface of the soil to measure the displacement during soil liquefaction. The diameter of the surface markers used was two times smaller than the recommended design (improved design with an external diameter of 13 mm at the model scale). The location of the markers in the X and Y directions was performed with a steel rule with an estimated precision of 1 mm and the Z location was performed with a laser sensor. The use of a laser sensor to measure the soil surface required a partial immersion of sensor (at least 10 mm) and consequently the water level was between 45 and 43 mm above the top of the slope (Fig. 15.3). The precision of the Z position is smaller than 0.5 mm as requested in the specifications. The surface markers have been put in place before the saturation process and their location has been measured at 1 g before the first spin up of the centrifuge and after each base shaking (Motion#1 and Motion#2) once the centrifuge was spun down.

15.2.3 Viscosity of Pore Fluid

The viscous fluid used for the test is a mixture of tap water, HMPC (Culминаl MHPC50), and biocide that is added in order to avoid decrease of the viscosity with time. A series of viscosity measurements was performed on several mixtures with different amounts of HPMC (from 28 to 23.5 g per liter).

The viscosity was measured using a FungiLab smart series viscosimeter with a low viscosity adaptor combined with a thermostatic bath. This is a Couette-type viscosimeter. After the stabilization of the temperature, the kinematic viscosity was measured for different shear rate values (in a Couette-type viscosimeter the shear rate represents, under the hypothesis of a laminar flow and a Newtonian fluid, the gradient of the fluid velocity between the outer and the inner cylinder of the viscosimeter). The corresponding average kinematic viscosities that correspond to the centrifuge room temperature were 50.8 and 45.8 cst for the S02 and S03 tests, respectively. It should be noticed that the viscous fluid used for the test is rheofluidifying: the viscosity of the fluid decreases with the increase of the shear rate (Fig. 15.4).

15.2.4 Saturation Process

The saturation process was performed under vacuum at 1 g. Prior to the de-air fluid flow into the soil model, the rigid box was closed using an airtight lid. The container, the fluid tank, and the various connections were de-aired following the

Fig. 15.4 Evolution of dynamic viscosity for the viscous fluid used in S02 test (28 g/l) versus revolutions per minute and temperature

recommendations of Kutter (2013). Two cycles of vacuum/CO₂ were performed. Each cycle consists of, first, the application of a vacuum up to an absolute pressure of 100 to 94 mbar followed by flooding of CO₂ up to the atmospheric pressure. By this way, it is supposed that 99% of the air resting inside the container has been replaced by CO₂. After these two cycles, an absolute pressure of 94 mbar was applied and maintained during the fluid flow into the container. The fluid was dripped onto the surface near the slope tip (a sponge was put in place above the soil surface to prevent perturbations). The drip rate was not measured but once a pool was formed at the slope tip, the fluid flow was adjusted by hand to maintain the pool. The time request for saturation was around 10 h which corresponds roughly to a drip flow of 6.5 ml/min. The degree of saturation was not checked with the method proposed by Okamura and Inoue (2012) at the end of the saturation process.

One nonconformity took place during the saturation process in the case of the S02 test. Some air leakage was observed on the lateral side where the cable gland for pore pressure sensors P1 to P4 was located. However, the few air bubbles that appear concerned only a small localized zone of about 2–3 cm² at the surface.

15.3 Achieved Ground Motions

As previously mentioned the tests were performed at 50 g. The angular velocity (93.8 RPM) of the centrifuge was calculated to produce the 50 g centrifuge acceleration at a radius of 5.09 m (the radius to 1/3 depth, i.e., 26.7 mm at the model scale, below the soil surface at the central array of accelerometers).

15.3.1 Horizontal Component

Two base shakings were applied to the S02 and S03 models. Each one was a tapered sine, the first one obtained from a reference signal with a PGA of 0.1 g and the second one from a reference signal with a PGA of 0.3 g. The achieved base motions were analyzed by isolating the predominant frequency component (1 Hz at the prototype scale) from the noise. The filtering of the raw data was made by using a FIR filter with a passband frequency at the prototype scale of [0.76–1.24 Hz] with an attenuation of –40 dB. This filter type avoids introduction of phase lag between the frequency components of the signal. The time representations of the achieved base Motion#1 and Motion#2 for both tests (raw data, filtered data) are presented in Fig. 15.5.

The sampling frequency selected for the tests was 10,240 Hz except in the case of Motion#1 of the S03 test that was only 1024 Hz. The analysis of the frequency content of the base shaking for all the base shakings highlights three high-frequency components that are non-negligible especially in the case of Motion#1: 7 Hz, 9 Hz, and 16.6 Hz components. It should be noticed that due to the sampling frequency used for Motion#1 of the S03 test, only the 7 Hz and the 9 Hz components were recorded. The 16.6 Hz component corresponds to the dither associated to the servo valve of the hydraulic shaker and cannot be removed. However the velocity and the displacement induced by this high frequency are negligible due to the 16.6 Hz frequency value compared to the 1 Hz component.

For Motion#1, the reference signal has been selected to obtain a PGA_{eff} of about 0.15 g following the definition given by Eq. 15.1.

$$PGA_{\text{effective}} = PGA_{1\text{Hz}} + 0.5 \cdot PGA_{\text{hf}} \quad (15.1)$$

In this equation the PGA_{hf} corresponds to the peak value induced by the higher-frequency components that occur within one cycle of the peak of the 1 Hz component. For all the base shakings, except for Motion#2 of the S03 test, the maximum value of the 1 Hz component and the maximum value of the raw acceleration were not reached at the same time as illustrated in Fig. 15.6.

Consequently in order to determine the PGA_{eff} , the maximum value of the noise reached in the vicinity of the maximum value of the 1 Hz component has been considered. Table 15.1 gives the values obtained for the PGA_{eff} .

If the velocity is considered, the harmonic components and the frequency component induced by the dither are negligible for all the motions. An example is given in Fig. 15.7 for the case of Motion#1 of the S02 test. As the liquefaction phenomena are linked to the distortion and the velocity of the distortion, a criteria for the quantification of the base shaking based on the effective velocity could be relevant too. The velocity has been obtained by integrating the acceleration and then by applying a passband filter FIR 1 with a frequency passband at the model scale of [12.5–5070 Hz] (due to the sampling frequency error for Motion#1 of the S03 test, the passband frequency was reduced to [12.5–507 Hz]). As illustrated in Fig. 15.7

Fig. 15.5 LEAP S02 and S03—time representations of the achieved base Motion#1 and Motion#2

the maximum value of the 1 Hz component and of the raw values of the velocity is reached at the same time. Considering the definition of the PGV_{eff} , the effective peak ground velocity, given by Eq. 15.2, the values obtained for each base shaking are also given in Table 15.1.

$$PGV_{effective} = PGV_{1Hz} + 0.5 \cdot PGV_{hf} \tag{15.2}$$

Fig. 15.6 LEAP S02 Motion#1—achieved base acceleration time history (bottom). High-frequency noise isolated from achieved signal (middle). 1 Hz signal (top) (the red dot is the point at the time the maximum value of the 1 Hz component is reached)

15.3.2 Vertical Component

The time representation of vertical accelerations and displacements measured at the top of the extremity of the rigid container (AV1 and AV2, Fig. 15.1) are represented in Fig. 15.8 for the test S03. The results obtained for the test S02 have a limited interest due to the absence of data for the vertical accelerometer AV1. The displacement is obtained by double integration process of the accelerometer. After each integration, the filter previously presented to obtain the velocity was applied.

From the frequency analysis of Motion#1 of test S02, a non-negligible part of the vertical acceleration is due to components with a frequency equal of higher than 7 Hz (up to around 50 Hz). As a consequence, as previously mentioned, due to an error in the selection of the sampling frequency (1024 Hz) for Motion#1 of test S03, the acceleration data are only relevant to frequency components up to about 10 Hz at the prototype scale and consequently the maximum vertical acceleration represents only 5% of the maximum horizontal acceleration against 71% in the case of Motion#1 of test S02 (the percentage 71% is based on the value of only one accelerometer).

For Motion#2 the maximum vertical acceleration represents 66.5 and 74%, respectively, of the maximum horizontal acceleration. Even if the level of vertical acceleration is non-negligible compared to that of the horizontal acceleration applied at the base of the container, due to the frequency content of the vertical acceleration, the vertical displacements are largely lower than the horizontal displacement. For Motion#1 of tests S02 and S03 the maximum dynamic vertical displacements are 4% and 3.4%, respectively, of the maximum dynamic horizontal displacement. In the case of Motion#2 the values are 8 and 3%, respectively.

Table 15.1 Characteristics of the measured base motions for S02 and S03 tests

Test	Base shaking	PGA _{eff} (g)	PGA 1 Hz component (g)	% Due to high frequency ^a	PGV _{eff} (m/s)	PGV 1 Hz component (m/s)	% Due to high frequencies ^a	Frequency of the first harmonic
S02	Motion#1	0.153	0.102	34%	0.166	0.160	4%	7 Hz
	Motion#2	0.323	0.299	8%	0.482	0.468	3%	7 Hz
S03	Motion#1	0.113	0.1	12%	0.165	0.16	3%	7 Hz
	Motion#2	0.37	0.28	24%	0.445	0.03	3%	7 Hz

^aThis is, considering the equation, the percentage of the PGA_{eff} and PGV_{eff} due to high-frequency components

Fig. 15.7 LEAP S02 Motion#1 achieved base velocity time history (bottom). High-frequency noise isolated from achieved signal (middle). 1 Hz signal (top)

The time representation of the dynamic vertical displacement for Motion#2 of S03 test illustrates an opposition phase between AV1 and AV2 that characterized a rotation of the container. In the case of Motion#1 both vertical accelerometers are in phase opposition but the displacement amplitude varies differently with time. In both cases the rotation is limited (it is estimated around 3.10^{-4}° and 8.10^{-3}° for Motion#1 and Motion#2, respectively).

15.4 Results

In the following all the data are given at the prototype scale otherwise mentioned.

15.4.1 Pore Pressure and Acceleration Responses

As previously mentioned, due to the use of a laser sensor to record vertical displacement of the markers between each base shaking, a minimum value for height of the water table above the soil surface was necessary. The height of the water table for the S02 and S03 tests was 45 and 43 mm, respectively, at the model scale (Fig. 15.1). For both containers no system of wave limitation was used and the level of the water during the shaking was not recorded. Figure 15.9 illustrates the variation of the pore pressure during the shaking at the bottom of the container near each extremity (P9 and P10). In addition, the pore pressure at 70 mm (3.5 m at the prototype

Fig. 15.8 LEAP S03 Motion#1 and Motion#2 vertical acceleration and dynamic displacement measured at the top of both extremities of the rigid container

scale) of each extremity of the container and at about 2 m depth below the soil surface is also presented.

In this example, considering the initial position of the pore pressure sensors measured during the buildup of the model S03 and the level of the water table above the soil surface, the maximum pore pressure buildup, considering that there were no waves, was around 29.4, 47.1, 5.8, and 13.2 kPa for the P10, P0, P8, and P6 pore pressure sensors, respectively. These values are lower than the maximum pore

Fig. 15.9 S03 Motion#2 pore pressure response near and at the extremity of the container

pressure buildup measured during Motion#2 of the S03 test (i.e., 56.6, 63.6, 13.6, and 19 kPa for the P10, P9, P8, and P6 sensors, respectively). In addition, the variation of the pore pressure measured by sensors P9 and P10 is phase opposite such as for the sensors P8 and P6. Even if further analysis is requested, the amplitude of the pore pressure and the phase opposition suggest that one part of the pore pressure fluctuations recorded by these four sensors is due to the waves. This analysis suggests that a wave reduction system should be built for future tests to avoid non-negligible effect of waves near the extremities of a rigid container.

Figure 15.10 illustrates the pore pressure variation, for both tests, of the central array of pore pressure sensors for each base shaking. Referring to the as-built sensor depths, the initial effective pore pressure for the P3 and P4 sensors of the S02 test is about 23 and 11 kPa, respectively. In the case of the S03 test, the initial effective vertical stress for the P2, P3, and P4 sensors is about 28, 16, and 13 kPa, respectively.

In the case of the central array of pore pressure sensors, the shape of the pore pressure buildup such as the maximum value suggests that the effect of the wave is limited in the middle of the container.

In the case of the S02 test (dense sand, $\rho_d = 1696 \text{ kg/m}^3$) Motion#1, the pore pressure ratio (i.e., ratio of the pore pressure buildup to the initial vertical effective stress) rapidly reached about 100% at shallow depth (P4). A pore pressure ratio of around 100% was also reached at lower depth (P3) 4 s later. Compared to the pore pressure evolution near the surface (P4), in the case of the P3 sensor, the pore pressure buildup remains at the same value a shorter duration after the end of the base shaking. For both levels the dissipation of the pressure buildup took about 90 s at the prototype scale. In addition, spikes appeared once the pore pressure ratio of 100% was reached. These sharp peaks of pore pressure are more pronounced at 2 m depth. These rapid decreases of pore pressure buildup can be due to dilatancy phenomena in dense sand when the saturated soil reaches the phase transformation line. This result is in accordance with the measured accelerations presented in Fig. 15.11. As for the pore pressure buildup, acceleration spikes are more pronounced in the case of the AH4 than in the case of the AH3: it should be supposed that the dilatancy phenomena should have occurred up to the depth of 3 meters.

Fig. 15.10 S02 and S03—Motion#1 and Motion#2 pore pressure variations in the central array

In the case of Motion#2 the ratio of the excess of pore pressure over the initial vertical effective stress reached 1 for all the reliable recorded pore pressure (Fig. 15.10). Compared to the case of the P4 sensor for Motion#1, sharper peaks of the pore pressure appeared for both levels of measurement. The variation of the pore pressure reached -4.9 and -30.9 kPa for the P3 ($z = 2.2$ m) and the P4 ($z = 1.05$ m) sensors, respectively. Taking into account the initial vertical effective stress at the location of the pore pressure sensor P4 (i.e., 10.9 kPa), a negative vertical effective stress is reached on very short periods at shallow depth. Due to the main objective of this descriptive paper, a deeper analysis of this observed phenomena will be provided in another paper.

Fig. 15.11 S02 test, acceleration response of the central array for (a) Motion#1 and (b) Motion#2

The peaks that appeared in the pore pressure recordings (Fig. 15.11) are in accordance with the sharp peaks of horizontal acceleration that appear for all the accelerations measured except for the AH1 that is the acceleration at 3.5 m depth. It should be supposed that the phenomena of dilatancy occurred up to 2.5–3.5 m depth.

In the case of test S03 (medium loose sand, $\rho_d = 1624 \text{ kg/m}^3$) the pore pressure ratio for Motion#1 reached 88%, 100%, and 100% for P2, P3, and P4, respectively. The pressure buildup is more rapid than for the dense sand. Furthermore, the pressure decreases more slowly than for the dense sand.

Taking into account the difference of sampling frequency between both motions, no noticeable difference appears in the evolution of the pore pressure buildup in the case of Motion#2. In both cases, there are pore pressure spikes that coincide with acceleration spikes (Fig. 15.12).

Fig. 15.12 S04 acceleration response of the central array for: (a) Motion#1 and (b) Motion#2

Contrary to the dense sand, acceleration spikes appear at all the levels whatever the base shaking applied. However, the amplitude of the spike is generally larger and in the two directions (positive and negative) when they appear in dense sand. Furthermore, in the case of the medium loose sand subjected to large base shaking (S03 Motion#2), the number of spikes decreases with the decreasing of the depth.

15.4.2 Surface Maker Response

A cross view and a top view of the initial position and the vector of the total displacement of the surface markers and the embedded sensors are illustrated in

Fig. 15.13 S02 and S03 tests: top and cross views of the surface markers and sensor displacement induced by the two successive base shakings

Fig. 15.13. The initial position is the one that corresponds to the first location measurement before the first spin up of the centrifuge for the surface markers and during the pluviation process for the embedded sensors. The final location measurement corresponds to the location measured after the second base shaking once the centrifuge was spun down for the surface markers and during the dismantle of the container for the embedded sensors. As the displacement has remained limited in test S02, the length of the displacement vector was magnified by 3 for both tests.

Based on the surface markers, in the case of the S02 test, the settlement does not exceed 10 cm in the upper part and -2 cm near the toe of the slope. In addition, the maximum average horizontal displacement is about 10 cm and was reached in the middle of the slope. For the medium loose sand, the displacements are largely higher with a maximum settlement of 47 cm in the upper part of the slope and -17.5 cm near the toe of the slope. The maximum horizontal displacement reaches 97 cm and is also located in the middle of the slope.

15.5 Conclusion

This paper summarized the buildup and some results of the two centrifuge tests performed at IFSTTAR in the framework of the LEAP-UCD-2017 series of tests.

Two centrifuge tests were performed by IFSTTAR; the tests were done on a dense and a medium loose Ottawa-F65 sand. The main nonconformities of the experiment buildup were the absence of the measurement of saturation degree for both tests and the measurement of the soil characteristics through CPT tests and bender element measurements.

Despite all care taken during the determination of the drives, non-negligible high-frequency components were recorded for Motion#1. Motion#2 with higher amplitude was less perturbed by high-frequency components.

For the two motions the recording of pore pressure sensors highlighted sharp pore pressure peak that illustrated dilatancy phenomena. The phenomena were more pronounced in dense sand and with increasing the PGA_{eff} . The dense Ottawa sand and in a more limited way the medium loose sand should have reached under the test conditions the phase transformation line.

The density has a large effect on the observed sensors and marker displacement; in the case of the dense sand the displacement is limited contrary to the case of the medium loose sand.

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