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Centrifuge and numerical modeling of the behavior of homogeneous embankment on liquefiable soil subjected to dynamic excitation

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Abstract

Centrifuge and numerical modeling are developed to investigate the behavior of a homogeneous embankment constructed on a liquefiable sand ground, which was prepared with wet under-compaction method. The physical centrifuge modeling directly highlights the response of the centrifuge model in terms of excess pore pressure, acceleration response, settlement, and the deformation pattern of the embankment model. In parallel, a numerical simulation using FEM is conducted. The advanced constitutive model, PM4Sand, is adopted to simulate the liquefaction of the sand ground. The performance of the numerical model is then verified against the results of the centrifuge test. Overall, the numerical simulation aligns well with the centrifuge test. Finally, the verified numerical model is refined to include two improved boundaries, and the potential boundary effects in centrifuge and numerical models are discussed.

Keywords: Centrifuge modeling; Numerical modeling; Embankment; Ground liquefaction

List of symbols

CRR Cyclic resistance ratio	t_2 Start time of decreasing stage of permeability
CRR_{TX} Cyclic resistance ratio of cyclic triaxial test	t_3 End time of decreasing stage of permeability
D_{50} Mean particle diameter	G_{max} Maximum shear modulus
e Void ratio	G_o Shear modulus coefficient
e_{max} Maximum void ratio	G_s Specific gravity
e_{min} Minimum Void ratio	g Earth gravity
t Time	h_{po} Contraction rate parameter

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h_o	Secondary input parameter that adjusts the ratio of plastic modulus to elastic modulus	V_s	Shear wave velocity
I_D	Relative density	X	Horizontal position
k	Permeability	Y	Vertical position
k_i	Initial permeability	α	Constant controls the variation of excess pore pressure
k_b	Permeability in excess pore pressure build up stage	α_2	Constant controls the variation of excess pore pressure
k_d	Permeability in excess pore pressure decreasing stage	β_1	Constant controls the variation of excess pore pressure
k_l	Permeability in liquefaction stage	β_2	Constant controls the variation of excess pore pressure
K_0	Lateral earth pressure coefficient	Δu	Excess pore pressure
N	Number of cycles required for liquefaction	ν	Poisson's ratio
p_A	Atmospheric pressure	ρ_{sat}	Saturated density of the soil
p'	Mean effective stress	ρ_{dmin}	Minimum dry density
p_e	Experimental value	ρ_{dmax}	Maximum dry density
p_n	Numerical value	σ_v	Initial vertical stress
Q	Bolton's constant in PM4Sand model	σ'_v	Initial vertical effective stress
R	Bolton's constant in PM4Sand model	ϕ	Critical state friction angle
r_u	Excess pore water pressure ratio		
S	Anderson's score		

1. Introduction

Earth dams, levees, berms, and embankments have played an important role in the development of infrastructure throughout history. Considered among the oldest forms of construction in civil engineering, these structures serve multiple purposes, including transportation, irrigation, flood control, and power generation. Their failure may cause a huge disaster and loss of lives and facilities (Wu *et al.*, 2012). Under dynamic loading, dam failures like cracks, huge settlements or internal erosion can occur (Veylon, 2017) even in low to medium seismic zones (Kawakami and Asada, 1966; Seed *et al.*, 1980). Historical cases, for example the Chilean earthquake in 1960 (Duke, 1960), the

8 San Fernando Earthquake in 1971 (Seed *et al.*, 1975) and the Tohoku earthquake in 2011 (Yamaguchi *et al.*, 2012;
9 Oka *et al.*, 2012), reveal that earthquake-induced liquefaction was the most significant cause of embankment failure.
10 Therefore, the study of the damage or even failure of embankment due to the earthquake-induced liquefaction is of
11 considerable importance from both public safety and financial standpoints.

12 In recent decades, physical tests have been conducted to directly observe and study the behavior and failure mech-
13 anisms of embankments under dynamic loading. Shaking table tests have been utilized to investigate the embankment
14 response to soil ground liquefaction. For instance, studies by Iwasaki (1986), Koga and Matsuo (1990) and Park *et al.*
15 (2000) have reported significant deformations in the embankment, including crest settlement and lateral displacement
16 of embankment sides. Besides the 1g shaking table tests, Ng centrifuge modeling could be regarded as a better option
17 in the study of embankment failure due to liquefaction. Based on the theory of scaling laws and the principle of stress
18 similitude (Garnier *et al.*, 2007), centrifuge modeling allows to interpret the behavior of a large-scale prototype model
19 with a reduced-size model that subjected to the artificial macro-gravity. Many centrifuge tests in the literature (e.g. Ng
20 *et al.* (2004), Okamura *et al.* (2013), Higo *et al.* (2015) , Adapa *et al.* (2021) and Izawa *et al.* (2022)) have focused
21 on the responses of embankments resting on non-liquefiable ground under earthquakes actions. Centrifuge tests were
22 conducted by Adalier and Sharp (2004), Pramaditya and Fathani (2021a) and Pourakbar *et al.* (2022) to investigate
23 the response of the embankment to soil ground liquefaction. These studies found that liquefaction typically occurs
24 beneath the ground surface near the toe of the embankment, while the soil beneath the embankment is less susceptible
25 to liquefaction. Furthermore, Adalier *et al.* (1998) investigated the effects of the thickness and position of the lique-
26 fiable layer on embankment behavior. Additionally, Park *et al.* (2000), Okamura and Matsuo (2002), Tiznado *et al.*
27 (2020), Li *et al.* (2021), Pramaditya and Fathani (2021b) and Pourakbar *et al.* (2022) have highlighted the effects of
28 liquefaction remediation on embankment response, taking into account different types of liquefaction reinforcement.

29 Numerical modeling or simulation is a versatile and important technical approach that has been utilized in the
30 geotechnical earthquake engineering community to study soil liquefaction. The numerical simulations of earthquake-
31 induced liquefaction were mainly conducted on the continuum domain by exploiting either Finite Element Method
32 (FEM) (Finn, 1999; Elgamal *et al.*, 2002; Aydingun and Adalier, 2003; Okochi *et al.*, 2015; Bhatnagar *et al.*, 2016;
33 Gobbi *et al.*, 2017; Rapti *et al.*, 2018; Chakraborty and Sawant, 2022) or Finite Difference Method (FDM) (Wu *et al.*,
34 2009; Bouraoui and Benmebarek, 2018; Ziotopoulou, 2018; Boulanger *et al.*, 2015; Boulanger and Montgomery,
35 2016; Dinesh *et al.*, 2022). The main objectives of these numerical simulations were to predict the behavior or the
36 failure of the embankment due to earthquake-induced liquefaction.

37 In order to have a more accurate and realistic prediction of the complex liquefaction phenomena, in the literature,
38 many constitutive models were proposed and then used in the simulation of liquefaction-related problems. Examples
39 of the constitutive models for liquefaction analysis include the multi-surface plasticity model (Elgamal *et al.*, 2003),
40 the bounding surface plasticity model – SANISAND (Dafalias and Manzari, 2004), the UBCSand model (Beatty
41 and Byrne, 2011), the coupled effective stress WANG model (Wang and Ma, 2019) *etc.* Recently, Dinesh *et al.*
42 (2022) used an advanced constitutive model PM4Sand implemented in FLAC 2D to simulate the same centrifuge

43 test. It has been shown that the numerical model with PM4Sand constitutive law was able to reproduce correctly
44 the responses in terms of accelerations and excess pore pressures. In recent years, the PM4Sand constitutive model
45 has proven to be successful in simulating soil liquefaction (Rahmani *et al.*, 2012; Chiaradonna *et al.*, 2022) and in
46 investigating its effect on the behavior of various geotechnical structures, such as dams (Boulanger and Ziotopoulou,
47 2015; Boulanger and Montgomery, 2016; Lu *et al.*, 2022), traffic embankments (Oblak *et al.*, 2020), structures with
48 shallow foundation (Kassas *et al.*, 2021b) and offshore pipelines (Seth *et al.*, 2022). PM4Sand is also one of the
49 recommended models by United States Society on Dams (USSD) for the seismic analysis of embankments, dams,
50 and levees (USSD, 2022).

51 This study investigates the behavior of a homogeneous embankment on liquefiable sand ground under dynamic
52 loading, using a combined experimental and numerical approach. The liquefiable ground was constructed with the
53 wet under-compaction method and the effect of the humid tamping on the liquefaction resistance of sand was care-
54 fully considered. Although a rigid (or strong) container without any additional absorbing materials was used in the
55 centrifuge test, the impact of boundary effects was found to be limited. The outline of this paper is as follows. Firstly,
56 the experimental set-up including the model geometry, model preparation and instrumentation is presented. Then,
57 the numerical modeling of the centrifuge test is discussed in detail, including FEM mesh, boundary conditions, con-
58 stitutive model calibration, and modeling execution procedure *etc.* Next, the experimental results in terms of the
59 model responses and liquefaction pattern are presented and discussed. The performance of the numerical model is
60 then verified by comparing the experimental and numerical results in terms of acceleration, excess pore pressure,
61 model deformation pattern, settlement *etc.* Finally, the effects of the boundary conditions on the model responses are
62 evaluated numerically with models with larger and different boundaries.

63 2. Experimental set-up of the dynamic centrifuge test

64 In the present study, the dynamic centrifuge test was conducted at the University Gustave Eiffel (Nantes Campus).
65 The centrifugal acceleration level was 60 g. To excite the model, a 1D shaking table embedded in the swinging basket
66 was employed, as previously described by Chazelas *et al.* (2008). This section provides a detailed description of the
67 experimental set-up utilized in the dynamic centrifuge test.

68 2.1. Geometry and dimensions of the centrifuge model

69 The geometry of the centrifuge model is shown in Fig. 1. According to the scaling laws (Garnier *et al.*, 2007), the
70 reduced-size centrifuge model represents a prototype embankment of 6 m in height, which rests on a liquefiable layer
71 9 m thick. The slope of the embankment is determined to be 1/1.5, which is closed to the friction angle of the sand
72 that was used in the embankment of the centrifuge model.

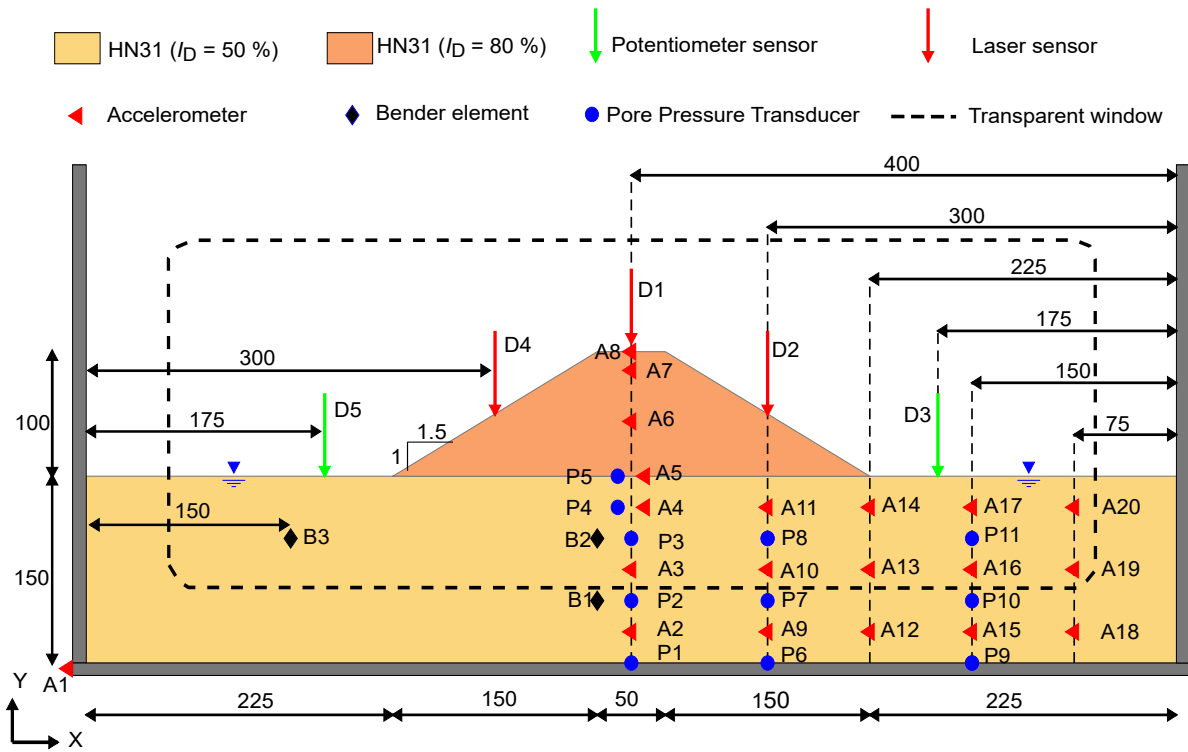


Figure 1: Centrifuge model at model scale, $N=1/60$ (Dimensions in mm)

2.2. Materials of the centrifuge model

The sand used to reconstitute the ground and the embankment was Hostun HN31 sand. Hostun HN31 sand is a fine sub-angular to angular siliceous sand (Benahmed, 2001; Benahmed *et al.*, 2004) and its main properties are listed in Table 1. The ground layer is fully saturated with a fluid that has a viscosity 60 times (60 cSt) higher than pure water to satisfy the scaling laws in the centrifuge test at 60g (Adamidis and Madabhushi, 2015). The viscous fluid was prepared by a mixture of water with hydroxypropyl methylcellulose (HPMC) (Dewoolkar *et al.*, 1999; Escoffier and Audrain, 2020).

Table 1: Properties of Hostun sand HN31 (Benahmed, 2001; Benahmed *et al.*, 2004)

Sand	D_{50} (mm)	e_{\min}	e_{\max}	G_s	$\rho_{d\min}$ (g/cm ³)	$\rho_{d\max}$ (g/cm ³)
Hostun HN31	0.35	0.656	1.049	2.65	1.33	1.6

2.3. Soil container

The model was built in a rigid container (see Fig 2) which has the inner dimensions of 800 mm (length) \times 400 mm (height) \times 340 mm (width). The soil container has a transparent on one side that allows the direct observation by cameras during the test. Doubled-face tapes were attached at the bottom of the container to enhance interface friction and ensure good transmission of the shear waves in the sand.

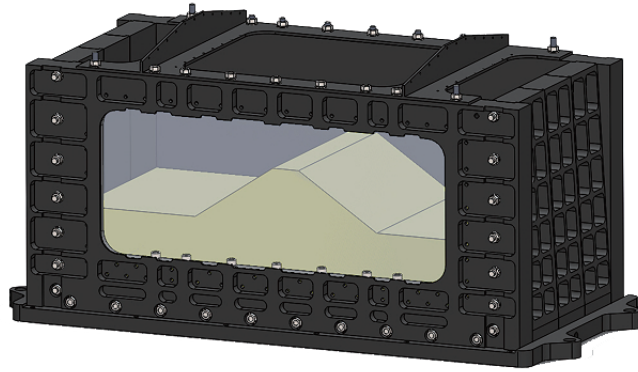


Figure 2: Illustration of the rigid container

2.4. Model preparation

The centrifuge model was prepared in two different steps. The ground layer and the embankment were prepared by different techniques proposed by [Adalier and Sharp \(2004\)](#). In this section, a detailed procedure for preparing the soil ground and the embankment is presented.

2.4.1. Liquefiable ground layer

The liquefiable ground layer which had a total thickness of 150 mm was prepared by the under-compaction method proposed by [Ladd \(1974\)](#). The moist Hostun HN31 sand was used with a 5% content of viscous fluid for tamping. The ground soil was then compacted layer by layer using a rectangular tamper of 1.4 kg which was dropped from a fixed height of 70 mm. As the tamper completed one compact at each position, it moved on to the next, until the height of the soil layer reached the controlled under-compaction depth. The levelness of each soil layer after under-compaction was carefully checked. Totally, six layers of sand were prepared for the ground layer. The relative density I_D was controlled at around $50.0\% \pm 2\%$.

2.4.2. Embankment

After completing the ground layer, a flat layer of dry dense Hostun HN31 sand with a thickness of 100 mm was then prepared using air pluviation. The pluviation was carried out with an automatic sand hopper ([Garnier, 2001](#)) with a slot opening equal to 4 mm, a falling height of 750 mm and a constant horizontal moving speed of 100 mm/s. With these parameters, the relative density I_D was estimated around $80.0\% \pm 1.5\%$. In order to achieve the desired shape without disturbing the dry sand of the embankment, the flat layer of dry sand was carefully and gradually trimmed using a vacuum cleaner (Fig. 3). Finally, the shape of the embankment was double-checked with a wooden mould.

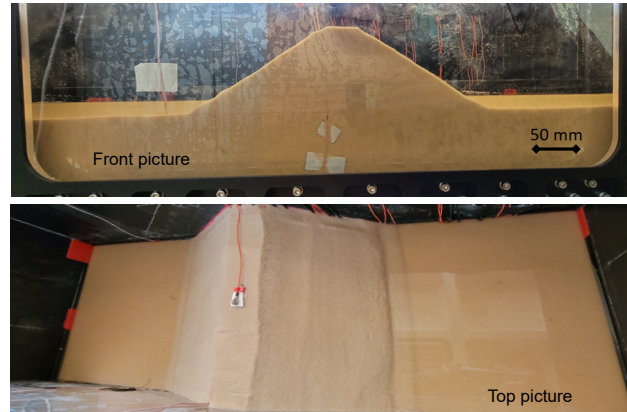


Figure 3: Final shape of the instrumented centrifuge model

104 2.4.3. Model saturation

105 The centrifuge model was saturated with viscous fluid at 1g in a vacuum chamber following the saturation procedure proposed by Ueno (1998) and Mulilis (1975). The air in the model was removed by applying vacuum and then
 106 flushing CO₂ by two times. Then, viscous fluid was injected into the centrifuge model under an absolute pressure of
 107 50 mbar from the bottom of the container. After the saturation process was completed and the water level reached
 108 5 mm above ground surface, the degree of saturation of the ground layer was checked by using the “laser-floater”
 109 method proposed by Okamura and Inoue (2012). The global degree of saturation was estimated to be approximately
 110 99.0%.
 111

112 2.5. Instrumentation

113 The type and location of the sensors are shown in Fig. 1 and the detailed information about the sensors is listed
 114 in Table 2. Totally, 20 accelerometers (A) and 11 pore pressures transducers (P) were installed to follow the soil
 115 motion and the pore pressure evolution during the dynamic event, respectively. For example, Fig. 4 illustrates the
 116 positioning of a 1D accelerometer at a certain depth in the sand. There were also three pairs of bender element located
 117 at three different depths. Three laser displacement sensors were installed to track the settlements of the crest and
 118 mid-slope of the embankment. Two potentiometer sensors were also placed at the free ground surface near the toe of
 119 the embankment.

Table 2: Instrumentation of the centrifuge model

Sensor type	Sensor notation	Working range	Manufacturing precision (%)	Number
Accelerometer	A	1 Hz-20 kHz	0.3 %	20
Pore pressure transducer	P	0-2 bars	0.2 %	11
Laser displacement	D (in red)	16-120 mm	0.1 %	3
Potentiometer	D (in green)	0-100 mm	0.1 %	2

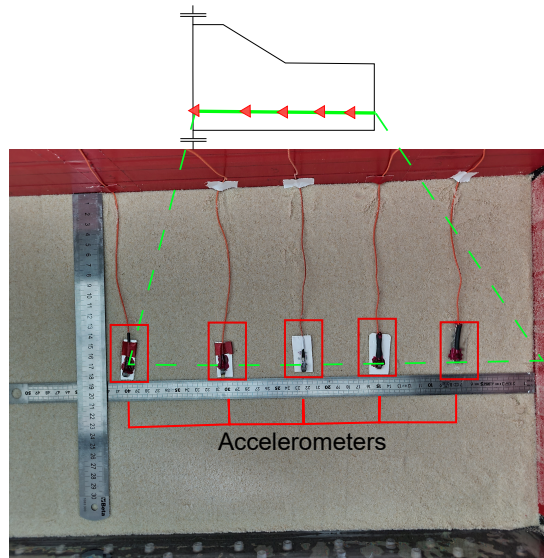


Figure 4: Centrifuge model in preparation - installation of accelerometers

120 2.6. *Experimental program*

121 The dynamic centrifuge test was performed in three stages: in-flight consolidation, bender element test and shak-
 122 ing/excitation. For the in-flight consolidation, the centrifuge model was gradually spun up 10 g by 10 g until up to
 123 60 g. The model was then kept at 60 g for 10 mins to fully consolidate the model in K_0 condition and stabilize the
 124 initial geo-static stress and pore pressures. Fig. 5 shows the initial pore pressure profiles obtained from the theoretical
 125 calculation, FEM solution (its details were introduced in Section 3 hereafter.) and experimental measurement. The
 126 pore pressure profiles were also compared under the embankment and under the free ground. The experimental mea-
 127 surement agreed well with the numerical model and the theoretical calculation. The theoretical profile was calculated
 128 by assuming an absolute dry state of the embankment, with the water level exactly at the surface of the ground. How-
 129 ever, during the centrifuge test, the embankment was in an unsaturated state (with an estimated fluid content of around
 130 3.5%). At the interface between the embankment and ground, the actual water level in the center of the embankment
 131 was slightly higher than that of the ground surface. This effect was observed in the numerical model and experimental
 132 measurement after consolidation, as it is shown in Fig. 5.

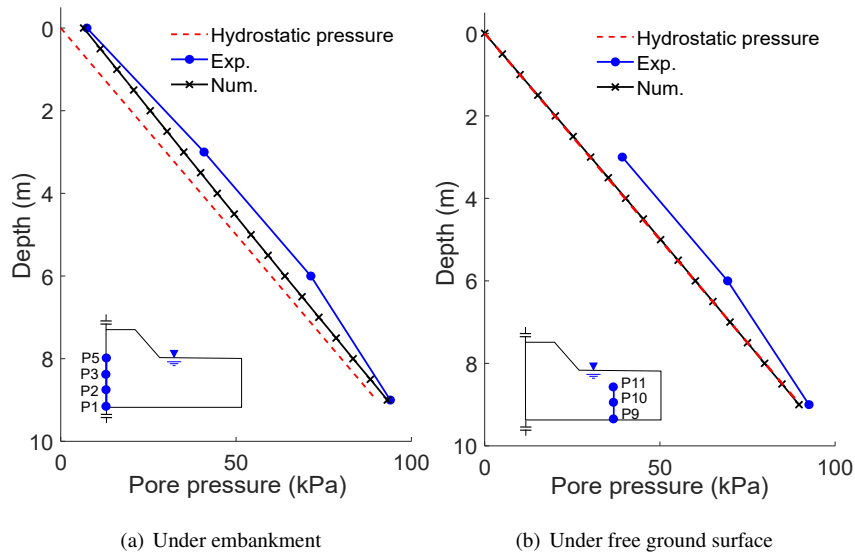


Figure 5: Comparison of initial static pore pressures of the centrifuge and numerical model after consolidation with the theoretical hydrostatic pressures: (a) under the embankment and (b) under the free ground surface

133 After in-flight consolidation and before the shaking, bender element tests were performed to measure the shear
 134 wave velocities at three different depths, see Fig. 1. Then, the model was subjected to a 1D horizontal sinusoidal base
 135 shaking. As it is shown in Fig. 6, the target input signal used in the test consisted of 20 full cycles with a dominant
 136 frequency of 1.5 Hz and an amplitude of 0.2 g in prototype scale. Due to the non-perfect response of the hydraulic
 137 shaker, the input signal was unavoidably noised by multiple high-frequency contents with small amplitudes. Fig. 6
 138 also shows the actual input signal and its frequency contents measured at the base of the container (A1). As a result
 139 of the presence of these small amplitudes at higher frequencies, the peak acceleration of the actual input signal is
 140 therefore slightly higher than that of the target amplitude.

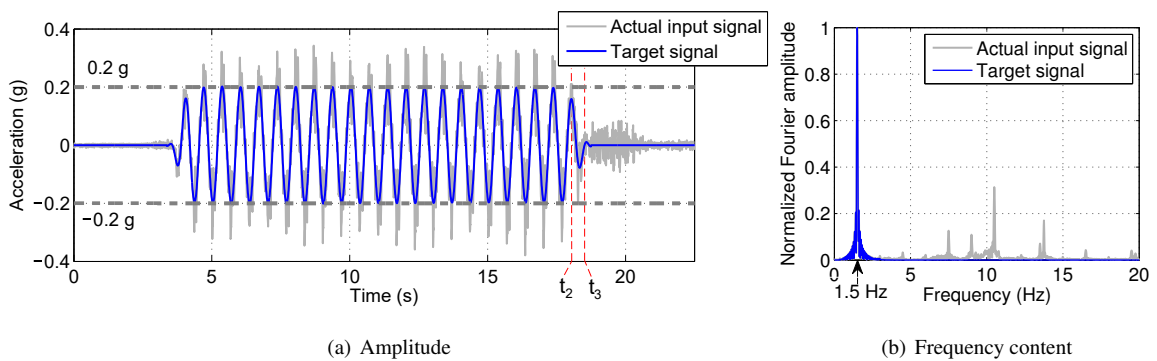


Figure 6: Time and frequency representations of the input signal (target and actual) in prototype scale (a) amplitude and (b) normalized frequency content

141 **3. Numerical modeling**

142 The numerical simulation of the centrifuge test was carried out on the OpenSees platform – “*Open System for*
 143 *Earthquake Engineering Simulation*”, which is a well known finite element method based open source code (McKenna
 144 *et al.*, 2006). In this section, a detailed description is provided regarding the FEM mesh, boundary conditions, consti-
 145 tutive laws, and calibration of the constitutive model parameters.

146 *3.1. FEM mesh and boundary conditions*

147 The numerical model was built at the prototype scale and the FEM mesh is shown in Fig. 7. The ground soil and
 148 embankment were modeled using 4-node quadrilateral SSPquadUP elements. The SSPquadUP element which has a
 149 mixed displacement-pressure $u - p$ formulation enables the dynamic plane strain analysis of fluid saturated porous
 150 media (McGann *et al.*, 2012). There were 1728 elements and 756 elements for the soil ground and the embankment,
 151 respectively. According to the mesh sensitivity analyses, the results of the model with the current mesh density (mesh
 152 size ≈ 0.5 m) are accurate enough.

153 The boundary conditions in the numerical model were chosen to accurately reflect the actual conditions in the
 154 rigid container which was used in the dynamic centrifuge test. All the nodes at the base were fixed in both horizontal
 155 X and vertical Y directions. While the lateral sides of the model were fixed only in the horizontal X direction. Drained
 156 conditions were assigned to the surfaces of the ground and the embankment (blue dashed line); while undrained
 157 conditions were assigned to the bottom and the lateral sides.

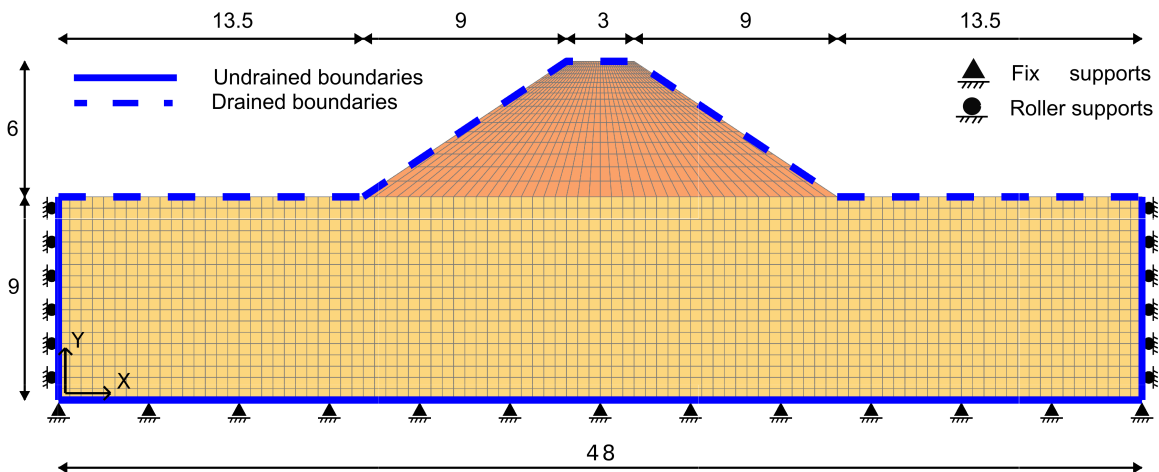


Figure 7: FEM mesh of the numerical model (Dimensions in m)

158 3.2. Constitutive model and parameter calibration

159 In this study, the behavior of liquefiable sand was modeled with the PM4Sand constitutive law which was de-
160 veloped by Boulanger and Ziotopoulou (2015). This model describes the plasticity of sand controlled by the stress
161 ratio related to the critical state and the bounding surface (Dafalias and Manzari, 2004). The PM4Sand model has
162 the capability of predicting the behavior of soil under cyclic loading for various relative densities. PM4Sand model
163 requires 27 input parameters: 6 primary parameters and 21 secondary parameters (Boulanger and Ziotopoulou, 2015).
164 The primary parameters include the relative density I_D , small-strain shear modulus coefficient G_o , contraction rate
165 parameter h_{po} , the atmospheric pressure p_A and two 'flag' parameters (FirstCall and PostShake). There are variety
166 of methods to estimate the small-strain shear modulus either by field tests (through empirical correlations with SPT
167 or CPT tests) or from laboratory element tests (for example bender element test). In this study, the small-strain shear
168 modulus coefficient G_o was directly estimated through the equations proposed by Azeiteiro *et al.* (2017) and Kassas
169 *et al.* (2021b) for Hostun HN31 sand. The contraction rate parameter h_{po} can be calibrated with the experimental
170 results in terms of the cyclic resistance curve ($CRR - N$) of the sand. While the other secondary parameters can be
171 either left with their default values or calibrated against the laboratory tests data (Boulanger and Ziotopoulou, 2015).
172 In this study, the variable h_o that adjusts the ratio of plastic modulus to elastic modulus was taken as the minimum
173 value 0.3, while the other secondary input parameters were left to their default values.

174 In the centrifuge model, although the same sand was used for the embankment and the liquefiable ground, ac-
175 cording to the literature (Mulilis *et al.*, 1977; Tatsuoka *et al.*, 1986; Benahmed, 2001; Sze and Yang, 2014), the soil
176 preparation method affects the liquefaction resistance. Therefore, when calibrating the PM4Sand model, it is essential
177 to consider the effects of the preparation method *i.e.* the under-compaction method used for the liquefiable ground
178 layer.

179 For the model parameters of PM4Sand which are not dependent on the sand preparation method *i.e.* G_o , ϕ , Q ,
180 R , the values proposed by Kassas *et al.* (2021b) were adopted (see Table 3). On the other hand, the contraction rate
181 parameter h_{po} , which plays an important role in stimulating the liquefaction resistance of sand under cyclic loading,
182 has to be calibrated carefully by taking into account the effect of the sand preparation method.

Table 3: Parameters used in the numerical model

Parameter	Value	
Relative density	[%] I_D	50
Void Ratio	[-] $e = e_{\max} - I_D(e_{\max} - e_{\min})$	0.83
Shear modulus coefficient [†]	[-] $G_o = 293 \times \frac{(2.97-e)^2}{(1+e)}$	712
Critical state friction angle [†]	[°] ϕ	33.8
Atmospheric pressure	[kPa] p_A	101.3
Lateral earth pressure [†]	[-] $K_0 = 1 - \sin \phi$	0.44
Poisson's ratio [†]	[-] $\nu = \frac{K_0}{1+K_0}$	0.3
Bolton's Constant [†]	[-] Q	8.4
Bolton's Constant [†]	[-] R	0.78
Permeability [†]	[m/s] $k = 2.92 \times 10^{-3} \times \frac{e^3}{(1+e)}$	9.68×10^{-4}
Contraction rate parameter	[-] h_{po}	0.07
Secondary input parameter	[-] h_o	0.3

Note: The parameters marked with [†] were proposed by [Kassas et al. \(2021b\)](#) for Hostun HN31 sand prepared by dry air pluviation method.

183 The calibration of h_{po} was carried out against the experimental data of $CRR - N$ curve of Hostun HN31 sand
184 prepared by moist tamping method. For the $CRR - N$ curve, since the direct data of undrained cyclic simple shear test
185 of Hostun HN31 sand with humid under-compaction method was rare, thus the experimental data has to be obtained
186 indirectly from other type of cyclic test *e.g.* triaxial cyclic test. In this study, the isotropically consolidated undrained
187 cyclic triaxial tests on Hostun HN31 sand of [Gobbi \(2020\)](#) and [Gobbi et al. \(2022a,b\)](#) were converted to undrained
188 simple shear $CRR - N$ form by Eq. (1) proposed by [Idriss and Boulanger \(2008\)](#).

$$CRR = \frac{1 + 2K_0}{3} CRR_{TX} \quad (1)$$

189 where CRR is the cyclic resistance ratio in simple shear form; CRR_{TX} is the cyclic resistance ratio of cyclic triaxial
190 test; K_0 is the lateral pressure coefficient for simple shear test.

191 An empirical equation Eq. (2) was used to fit the converted undrained simple shear $CRR - N$ data. Eq. (2) is based
192 on the equation originally proposed by [Kassas et al. \(2021b\)](#), with modifications on the parameters. The contraction
193 rate parameter, h_{po} , was then calibrated by conducting a series of numerical simple shear tests and matching the results
194 with the $CRR - N$ curve obtained from Eq. (2).

$$CRR = (-1.119 e + 1.14) N^{(0.24e-0.44)} \quad (2)$$

195 In this study, the simple shear test was carried out on a single element (SSPquadUP) under the initial vertical stress
196 $\sigma_v = 100$ kPa. The optimized value of h_{po} was found to be 0.07 and the calibration results are shown in Fig. 8. It is

197 shown that the calibration fits well with the curve globally. In case of higher *CRR* ratio (*N* ranges from 1 to 30) where
 198 the liquefaction is triggered after a few cycles, the fitting remains satisfactory. The main parameters of PM4Sand
 199 model used in this study are summarized in Table 3.

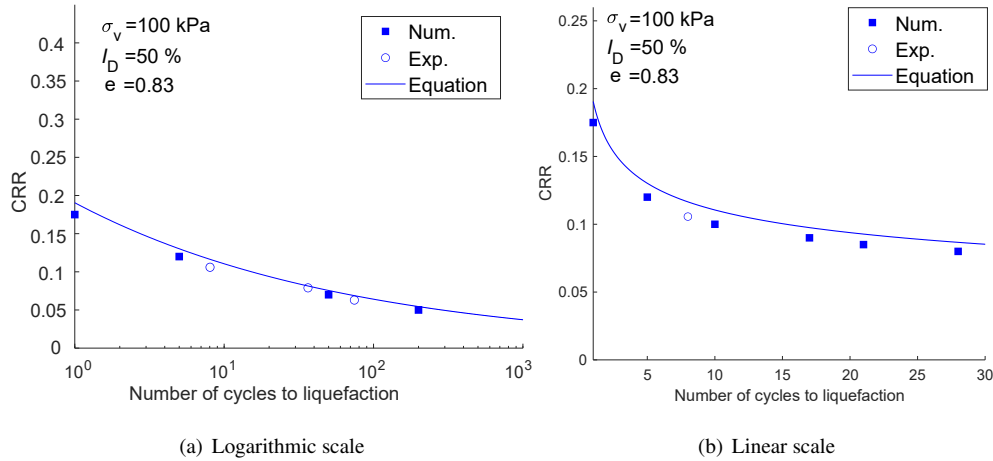


Figure 8: Calibration of PM4Sand constitutive model: Cyclic stress ratio vs. the number of cycles for liquefaction triggering

200 For the embankment soil, the Drucker-Prager model was adopted. As it was mentioned in section 2.4.3, the
 201 centrifuge model was saturated at 1g with viscous fluid. Due to the capillary effect, when the soil ground was fully
 202 saturated at 1g condition, the 100 mm height embankment was also partially saturated with viscous fluid, of which
 203 the water content is around 25%. According to Caicedo and Thorel (2014), when spin-up the centrifuge, due to the
 204 macro-gravity, the water content in the embankment can drop to around 2.0~3.5%, which was also confirmed by
 205 measuring the water contents of the samples taken from the embankment after the test. As a result of the unsaturated
 206 embankment and the viscous fluid, the cohesion and strength of the embankment soil increased. Therefore, in this
 207 study the peak friction angle 38° , the bulk modulus 173 kPa and the shear modulus 76 kPa were assumed as the
 208 parameters of the Drucker-Prager model for the unsaturated embankment soil according to Chen and Saleeb (1994).
 209 The cohesion of soil was set at 65 kPa in order to fit the settlement of the embankment as presented in Fig. 18 (see
 210 Section 5.4).

211 3.3. Procedure of numerical simulation

212 The numerical simulation was conducted on the FEM platform OpenSees – Version 3.2.2 64-Bit (OpenSees, 2020)
 213 in three steps. Firstly, the consolidation of the model was performed. At this stage, all elements' material properties
 214 were switched to elastic. Then, the material properties were switched from elastic to plastic and the "Update FirstCall"
 215 is activated and a "plastic gravity analysis" step was performed to initialize the internal variables and to reach the initial
 216 stress state of the model (see Section 2.6). After the consolidation, the whole model was subjected to dynamic shaking
 217 by "UniformExcitation" (Mazzoni et al., 2006). The numerical model was excited by the actual input signal applied
 218 in the dynamic centrifuge test, see Fig. 6. During the dynamic excitation, a Rayleigh damping ratio of 3.0% was

219 applied. The damping ratio was estimated through bender element test by the method proposed by [Karl et al. \(2008\)](#).
 220 The damping parameters were calculated with two modal frequencies 2.7 Hz and 6.7 Hz. These two frequencies were
 221 estimated by modal analysis with elastic material properties in Table 3.

222 In order to properly model the variation of the excess pore pressure, the hydraulic conductivity in the numerical
 223 model varied in two different stages: first, during the base shaking; and second, during the dissipation of excess
 224 pore pressure after shaking. During base shaking, the procedure proposed by [Shahir et al. \(2014\)](#) was adopted. The
 225 permeability was updated at each step according to Eq. (3), in which the permeability varied in function of the excess
 226 pore pressure ratio ([Shahir et al., 2014](#); [Dinesh et al., 2022](#); [Basu et al., 2022](#)).

$$k = \begin{cases} k_b = k_i (1 + (\alpha - 1)r_u^{\beta_1}) & (\text{if } r_u < 0.95) \\ k_l = \alpha k_i & (\text{if } r_u \geq 0.95) \end{cases} \quad (3)$$

227 where k_i is the initial permeability of the soil; k_b is the permeability during the build up of pore pressure ($r_u < 0.95$); k_l
 228 is the permeability during liquefaction ($r_u \geq 0.95$). The two constants α and β_1 were taken as 1.2 and 1, respectively,
 229 as they were proposed by [Basu et al. \(2022\)](#) for Hostun sand.

230 After the shaking, the "PostShake flag" in PM4Sand model was activated and the hydraulic conductivity was
 231 considered to decrease earlier before the excess pore pressure dissipation ([Shahir et al., 2012](#)). It is recommended by
 232 [Tobita \(2020\)](#) that the permeability should decrease to a certain value lower than its initial value during the dissipation
 233 process. Therefore, Eq. (4) was used for the reduction of hydraulic conductivity, which starts at the middle of the last
 234 cycle of the input signal ($t_2 = 18.1$ s) and ends at the last cycle ($t_3 = 18.7$ s) as presented in Fig. 6(a). This equation is
 235 based on the one proposed by ([Shahir et al., 2012](#)), but takes into account the reduction of permeability by a constant
 236 factor of α_2 .

$$k_d = \frac{k_i}{\alpha_2} (1 + (\alpha\alpha_2 - 1) \left(\frac{t - t_2}{t_3 - t_2} \right)^{\beta_2}) \quad (4)$$

237 where k_d is the permeability in the decreasing stage; t_2 and t_3 are the start and end time of decreasing stage, respec-
 238 tively; and the constants α_2 and β_2 were taken as 40 and 1 in this study to have the best fit with the experimental
 239 results.

240 4. Experimental results analysis

241 This section presents and discusses the main results of the dynamic centrifuge test. Firstly, the bender element
 242 test results before the base shaking are presented. Then, during the base shaking, the results are presented in terms
 243 of acceleration responses, the excess pore pressure ratio, liquefaction paths, *etc.* Finally, the model deformation and
 244 embankment failure pattern are discussed. All the experimental results hereafter are presented at the prototype scale
 245 unless otherwise mentioned.

246 4.1. Bender elements results

247 The shear wave velocity V_s is calculated from the distance and the travel time of the shear wave between the
 248 transmitter and receiver. Concerning the determination of the travel time, different definitions of first arrival point
 249 are proposed in literature (Jovičić *et al.*, 1996; Lee and Santamarina, 2005; Murillo *et al.*, 2011; Kumar and Shinde,
 250 2019). The travel time considered in this study is the time between the zero point of the input signal and the zero
 251 point after the first bump as proposed by Lee and Santamarina (2005) and adopted later by Kassas (2021). After the
 252 calculation of the shear wave velocity (V_s), the small-strain shear modulus (G_{\max}) was calculated using Eq. (5).

$$G_{\max} = \rho_{\text{sat}} V_s^2 \quad (5)$$

253 where ρ_{sat} is the mass density involved in the wave propagation. The estimated small-strain shear modulus was then
 254 compared with the empirical equation (Eq. (6)) proposed by Azeiteiro *et al.* (2017).

$$G_{\max} = 29300 \frac{(2.97 - e)^2}{1 + e} \left(\frac{p'}{100} \right)^{0.49} \quad (6)$$

255 It is shown in Fig. 9 that the experimental results and the empirical equation of G_{\max} were comparable, although
 256 dispersion was observed, which could probably be due to the precision of bender element measurement in the cen-
 257 trifuge test.

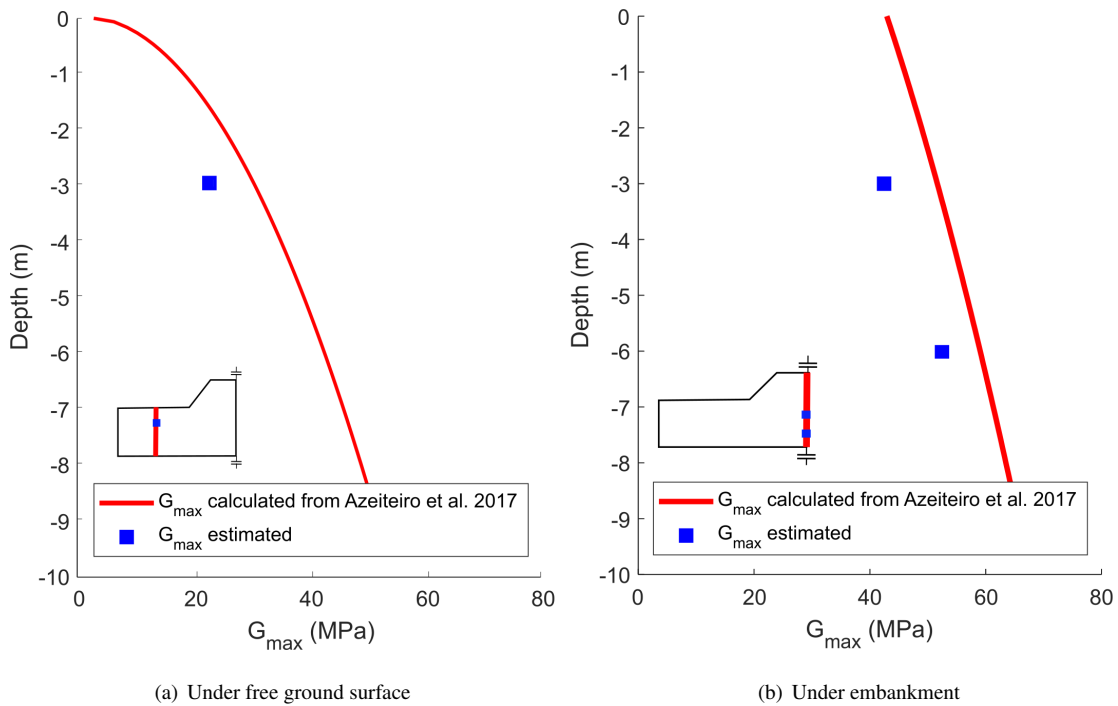


Figure 9: Comparison of the calculated small strain shear modulus with the empirical equation (Azeiteiro *et al.*, 2017)

258 *4.2. Acceleration response*

259 The time-histories of acceleration response at different positions are compared to the base shaking input signal
260 (Fig. 6), and the results are shown in Fig. 10. In this study, the noises and the unexpected frequency components in
261 the signals (Fig. 6) were filtered by a band pass filter with the cut-off frequencies from 0.33 Hz to 6.67 Hz (at the
262 prototype scale).

263 Different patterns of response were observed with the acceleration profiles. The first pattern was shown at bottom
264 of the model (A2, A9, A12, A15, and A18) where the acceleration responses were very close to the input signal. This
265 pattern proves the good transmission of the shear motion from the base of the container to the sand. An acceleration
266 amplification effect is observed for A6, A7 and A8 which were embedded in the embankment, and the amplification
267 increases from the bottom to the top. At A8, the amplification ratio with respect to the input signal is 2.12, with
268 a phase lag of about 0.5 s. For the acceleration profiles beneath the toe of the embankment (A11, A14, A13, and
269 A16), asymmetrical acceleration responses are observed, which indicates a horizontal movement of the embankment.
270 These acceleration responses are in accordance with the work of [Adalier and Sharp \(2004\)](#). These positive spikes
271 were caused due to the irregular geometry of the model above these positions. The acceleration profiles in Fig. 10
272 also reveal that liquefaction has occurred in the free ground rather than in the soil beneath the embankment. It is clear
273 that the accelerations at A17 and A20 show a significant attenuation after two cycles, where the liquefaction occurred
274 and the shear waves cannot be transmitted. This behavior is in accordance with the study of [Koga and Matsuo \(1990\)](#)
275 for embankments resting on liquefiable sandy ground.

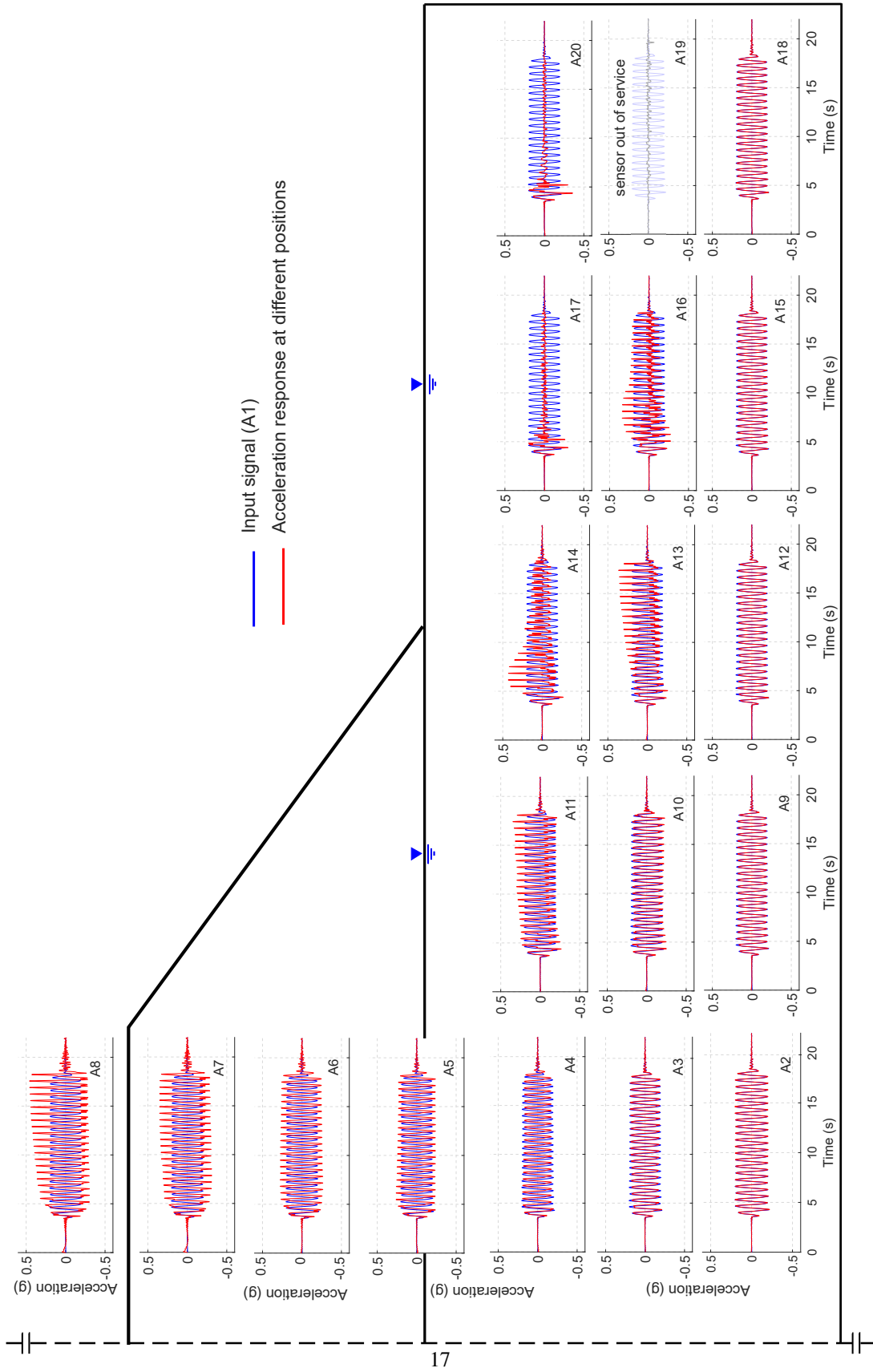


Figure 10: Acceleration responses at different positions in the model during dynamic excitation

276 *4.3. Excess pore pressure response*

277 The variation of excess pore pressure ratio r_u (ratio between excess pore pressure and initial vertical effective
278 stress) at various locations in the model is shown in Fig. 11. Below the embankment (P1 to P8), the evolution of the
279 pore pressure ratio follows the same tendency. In terms of r_u , the excess pore pressure reaches approximately 50%
280 of the initial vertical effective stress, which indicates that the soil beneath the embankment was not liquefied. While
281 for P10 and P11 under the free ground, the r_u increased rapidly after approximately two cycles and reached 1, which
282 indicates the occurrence of liquefaction. The responses of excess pore pressure at P10 and P11 are in accordance with
283 the acceleration responses presented in section 4.2 and confirm again the preference of liquefaction at the free ground
284 (Koga and Matsuo, 1990).

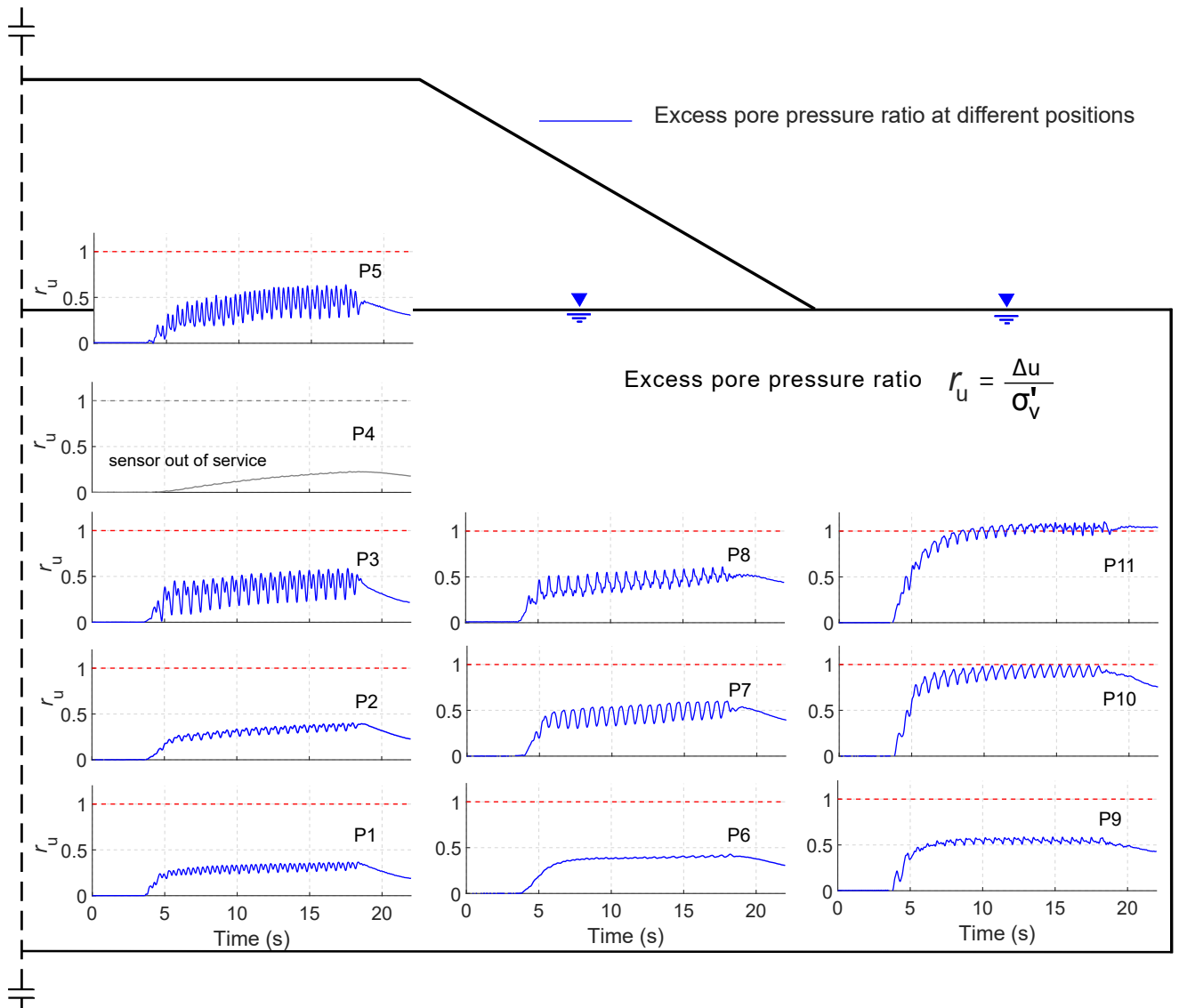


Figure 11: Excess pore pressure ratio at different positions in the model during dynamic excitation

285 *4.4. Stress path of liquefaction*

286 Fig. 12 presents the stress paths of the central array under the embankment and the array under the free ground
 287 surface, respectively. The liquefaction stress paths were calculated from the acceleration and excess pore pressure
 288 records by the method proposed by Zeghal and Elgamal (1994). Fig. 12(a) highlights that the stress paths below
 289 the embankment stabilized, and the vertical effective stress did not reach zero. On the other hand, for the stress paths
 290 under the free ground, the effective stresses show a significant decrease which tends to zero after two cycles of loading,
 291 see Fig. 12(b). The dramatic decreases of the effective stress are more pronounced for the soil with a depth of less
 292 than 6 m. The responses of the liquefaction stress paths confirm the liquefaction pattern mentioned in sections 4.2

293 and 4.3. Due to the presence of the embankment, which results in higher effective vertical stress, liquefaction did not
 294 occur under the embankment. In contrary, at a distance from the embankment where the effective confining pressure
 295 was relatively low, liquefaction occurred.

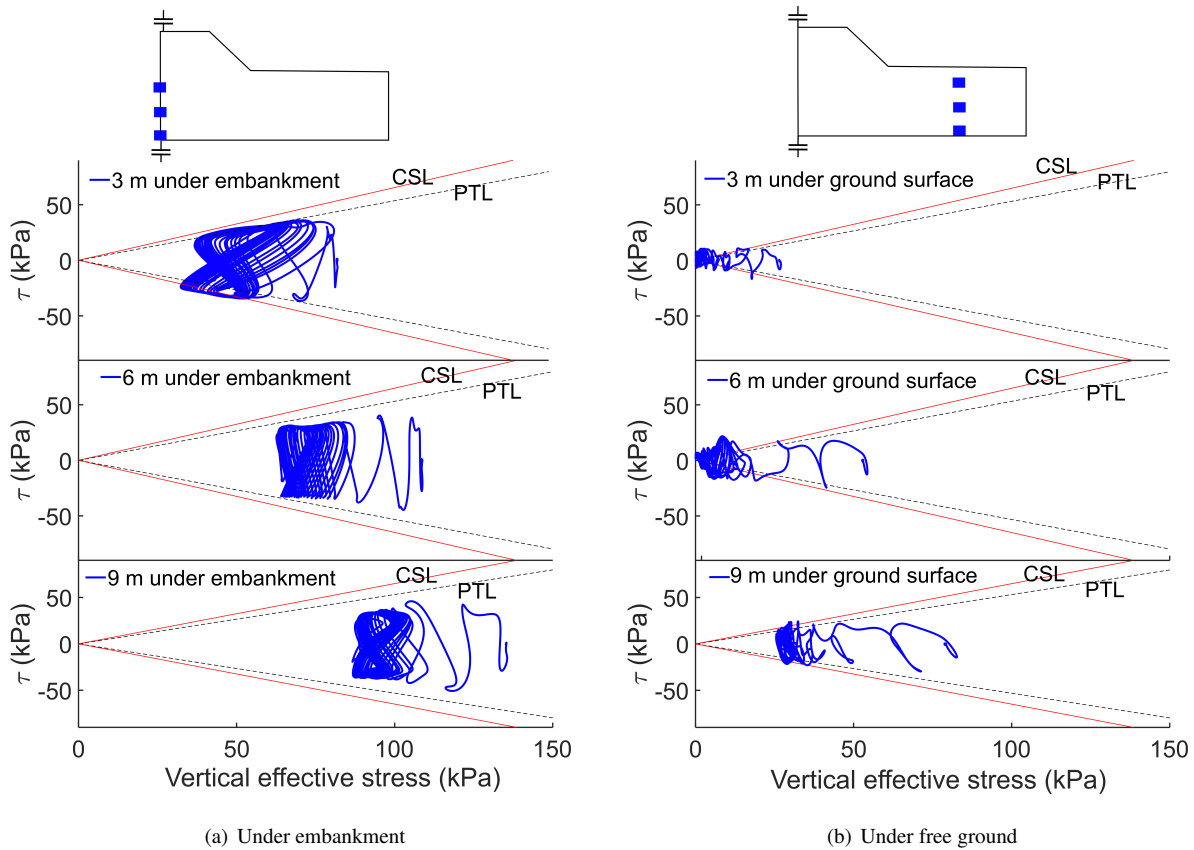


Figure 12: Stress paths at different positions (a) under embankment and (b) under the free ground surface. CSL and PT are Critical state Line and Phase Transformation Line for Hostun HN31 sand (Benahmed, 2001), respectively.

296 4.5. Model deformation and embankment failure pattern

297 Fig. 13 (a) shows the profiles of the maximum dynamic lateral displacement calculated by the double integra-
 298 tion (Li et al., 2013) of the recorded accelerations at different vertical arrays: center of the model (black line), under
 299 the toe of the embankment (red line) and under free ground surface (blue line). As it is shown in Fig. 13 (a), in the
 300 ground soil layer, the lateral displacement increased significantly around 6 meters under the ground surface. On the
 301 other hand, in the embankment, the slope of the maximum displacement profile above the ground surface decreased,
 302 which indicates a small relative displacement between the soil layers in the embankment. The maximum shear strains
 303 at different arrays in Fig. 13 (b) confirm a peak value at the depth around 6 m under the free ground; while for the
 304 embankment, the maximum shear strain decreased. As it was explained in section 4.4, limited and full liquefaction
 305 were observed under the embankment and under the free ground, respectively. The liquefaction of the soil caused a

306 decrease in the shear resistance, which results in a “soft” and “slippery” ground, on which the embankment moved
 307 approximately as a rigid block and exhibited a lateral sliding behavior.

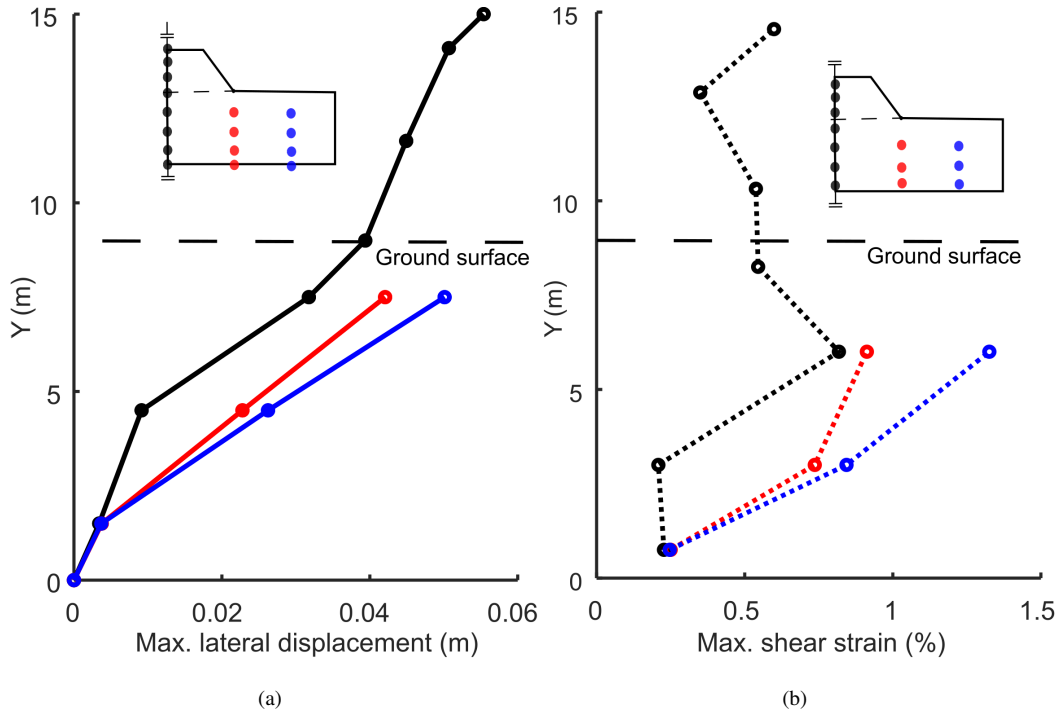


Figure 13: Displacement and shear strain analyses: (a) Maximum dynamic lateral displacement and (b) maximum dynamic shear strain at different positions in the model

308 Fig. 14 shows the deformation of the embankment before and after the base shaking and the settlements recorded
 309 by D1, D2 and D3 (Due to the symmetry of the model, the results of D4 and D5 are close to those of D2 and D3).
 310 A great crest settlement was observed, which is noted by the dashed lines in Figs. 14 (a) and (b). The recorded
 311 crest settlement was recorded by the laser sensors D1 (Fig. 14 (c)). The crest settled around 0.7 m which was nearly
 312 12.0% of the embankment height. This settlement was accompanied by a 0.2 m settlement of the middle slope of the
 313 embankment shown in Fig. 14 (d). In addition, a heaving of 0.04 m was observed at the free ground surface near the
 314 toes, see Fig. 14 (e). The heaving observed near the toes indicates a possible lateral spreading of the toes embankment
 315 (Fig. 14 (f)). The deformation pattern agrees well with the results reported by Doboku (1986), Oka *et al.* (2012),
 316 Adalier and Sharp (2004), Shahbodagh *et al.* (2020) and Li *et al.* (2021).

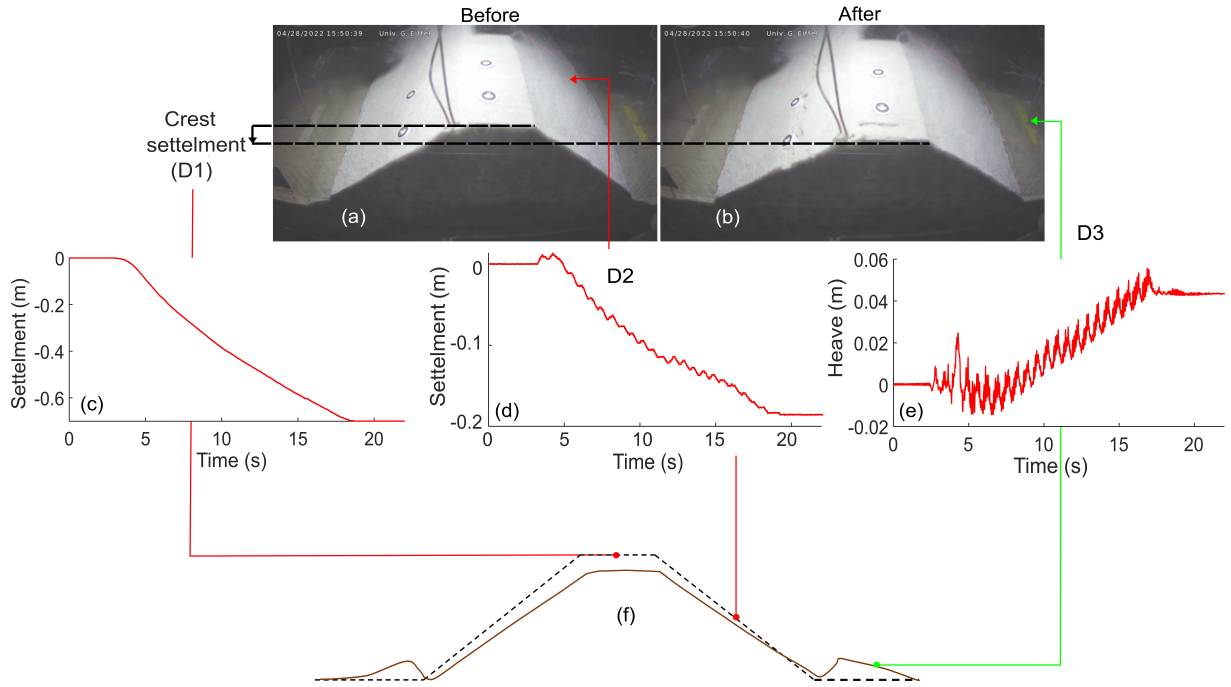


Figure 14: Deformation and settlement analyses: photos (a) before and (b) after the base shaking; (c) D1, Settlement of embankment crest; (d) D2, settlement of the middle slope of the embankment; (e) settlement at the free ground surface measured by potentiometer D3 and (f) embankment deformation pattern

317 5. Performance verification of the numerical model – comparison of numerical and experimental results

318 The performance of the numerical model was verified by comparing the numerical results with the experimen-
 319 tal results. The comparisons were conducted for the time-histories of acceleration, excess pore pressures and the
 320 deformation of the embankment, which are presented hereafter.

321 5.1. Acceleration response

322 Based on Anderson's Goodness of Fit (GoF) criteria (Anderson, 2004), the acceleration times histories computed
 323 with numerical model were compared to the experimental data. Quantitative scores were estimated to classify the
 324 agreement into poor fit (score lower than 4), fair fit (score between 4 and 6), good fit (score between 6 to 8) and
 325 excellent fit (score higher than 8) (Santisi d'Avila and Semblat, 2014). The score S related to the parameter p
 326 was estimated between numerical (p_n) and experimental results (p_e) using Eq. (7)

$$S\{p_n, p_e\} = 10 \exp\left(-\left(\frac{p_n - p_e}{\min(p_n, p_e)}\right)^2\right) \quad (7)$$

327 The items used for comparison of the time-histories acceleration at different positions are the peak acceleration
 328 (Spga), peak velocity (Spgv), peak displacement (Spgd), arias intensity (SIa), energy integral (SIv), acceleration

329 spectral (Ssa, 5 % damping) and the Fourier spectra (Sfs). The comparison of numerical and experimental results was
330 carried out for signals filtered with a band pass filter (0.33 Hz~6.67 Hz). The total score for a specific position is
331 considered as the mean value of all scores calculated at this position.

332 Fig. 15 and Table 4 show the comparison of the acceleration responses between experimental and numerical results
333 at different positions: at embankment crest (A8), under the embankment (A3), near embankment toe (A14) and under
334 the free ground surface (A17). Overall, the computed acceleration time histories are in good agreement with the
335 measured acceleration in the centrifuge test.

336 For A3 under the embankment, a symmetrical acceleration profile is observed in the centrifuge and in numerical
337 model which is classified as an excellent fit. The numerical model accurately simulated the experimental results
338 in the central part of the model beneath the embankment, where liquefaction did not occur. At the embankment
339 crest, the acceleration (A8) computed with numerical model is in good accordance with the experimental results.
340 Overall, the scores are good, despite of the fair score of the acceleration spectral (Ssa). This low score can be
341 explain by the underestimation of the acceleration amplitude after the third cycle which is observed in time histories
342 acceleration. The observed difference at A8 could be attributed to the constitutive law utilized for the embankment.
343 As mentioned before, the embankment was in an actual unsaturated state, and the Drucker-Prager model employed
344 can only approximately simulate the behavior of the unsaturated embankment as a simplified “ $c - \phi$ ” soil. Thus,
345 the transmission of the shear wave in the unsaturated embankment cannot be effectively captured by the numerical
346 model. In addition, the response under the toe of the embankment (A14) which is characterized by an asymmetrical
347 acceleration is also captured in a good way by the numerical model. The numerical model successfully captured
348 the asymmetrical response, but it underestimated the acceleration amplitude in the positive part. This discrepancy
349 may be attributed to the same cause that affects the prediction of the acceleration at the embankment crest (A8).
350 The difference in amplitude after the first three cycles could be attributed to the limitations of the Drucker-Prager
351 model when simulating the unsaturated soil. Under the free ground surface, the comparison between numerical and
352 experimental results at position A17 shows some differences. These differences were classified as an overestimation
353 of the acceleration amplitude after two cycles which was also reflected by the scores of the peak acceleration (Spga)
354 and arias intensity (SIa). These differences of the amplitude after two cycles, when the initiation of the liquefaction
355 occurred, could be attributed to the boundary effect of the numerical model.

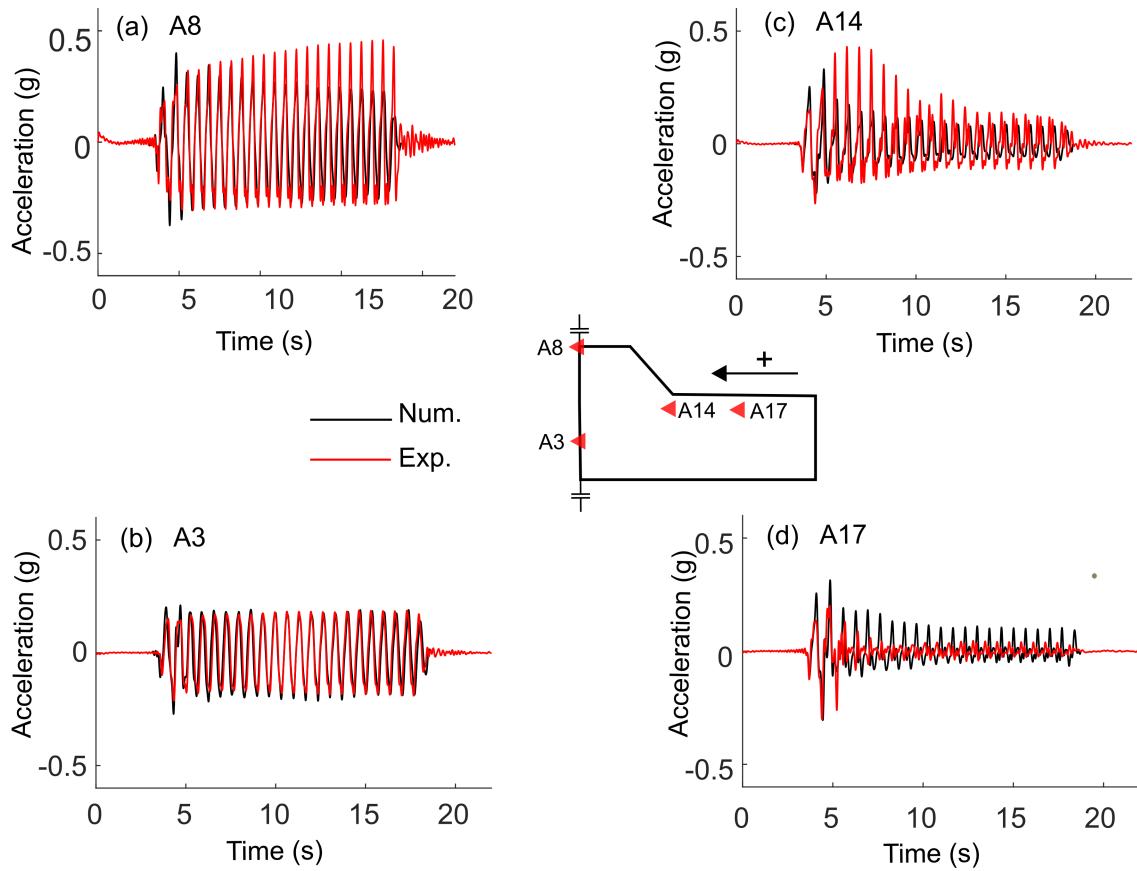


Figure 15: Comparison of numerical results with experimental results in centrifuge test in terms of acceleration response (a) at the embankment crest (b) under the embankment (c) near the embankment toe and (d) under the free ground surface

Table 4: Scores of Anderson's Goodness of Fit classification at different positions (A17, A14, A8 and A3). Comparison of different parameters S_{Ia} (Arias intensity), S_{Iv} (Energy integral), S_{pga} (Peak acceleration), S_{pgv} (Peak velocity), S_{pgd} (Peak displacement), S_{sa} (Acceleration spectra (5% damping)) and S_{fs} (Fourier spectra). (A) Excellent (B) Good (C) Fair and (D) Poor.

Position	S _{Ia}	S _{Iv}	S _{pga}	S _{pgv}	S _{pgd}	S _{sa}	S _{fs}	Total score
A17	D	A	B	A	B	A	D	B
A14	D	A	A	A	B	A	D	B
A8	A	A	A	B	A	D	D	B
A3	A	A	A	A	C	B	D	A

356 5.2. Excess pore pressure ratio

357 The numerical and experimental results of excess pore pressure ratio are compared in Fig. 16 for two positions
 358 under the embankment (Figs. 16 (a) and (b)) and two other positions under the free ground surface (Figs. 16 (c)
 359 and (d)). The numerical model captured the variation of excess pore pressure at the positions P5 and P8 under

360 the embankment. In the case of pore pressure P5 under the embankment, the excess pore pressure ratio was well
 361 simulated for the first six cycles, but for the subsequent cycles, the model underestimated the excess pore pressure
 362 ratio. This may be attributed to the limitations of the current numerical model in accurately simulating the changes
 363 in pore pressure in the zone (near P5) where the soil transitioned from a fully saturated to a partially saturated state.
 364 At the positions P10 and P11, the numerical model captured well the build-up and also the dissipation of excess pore
 365 pressure under the free ground surface.

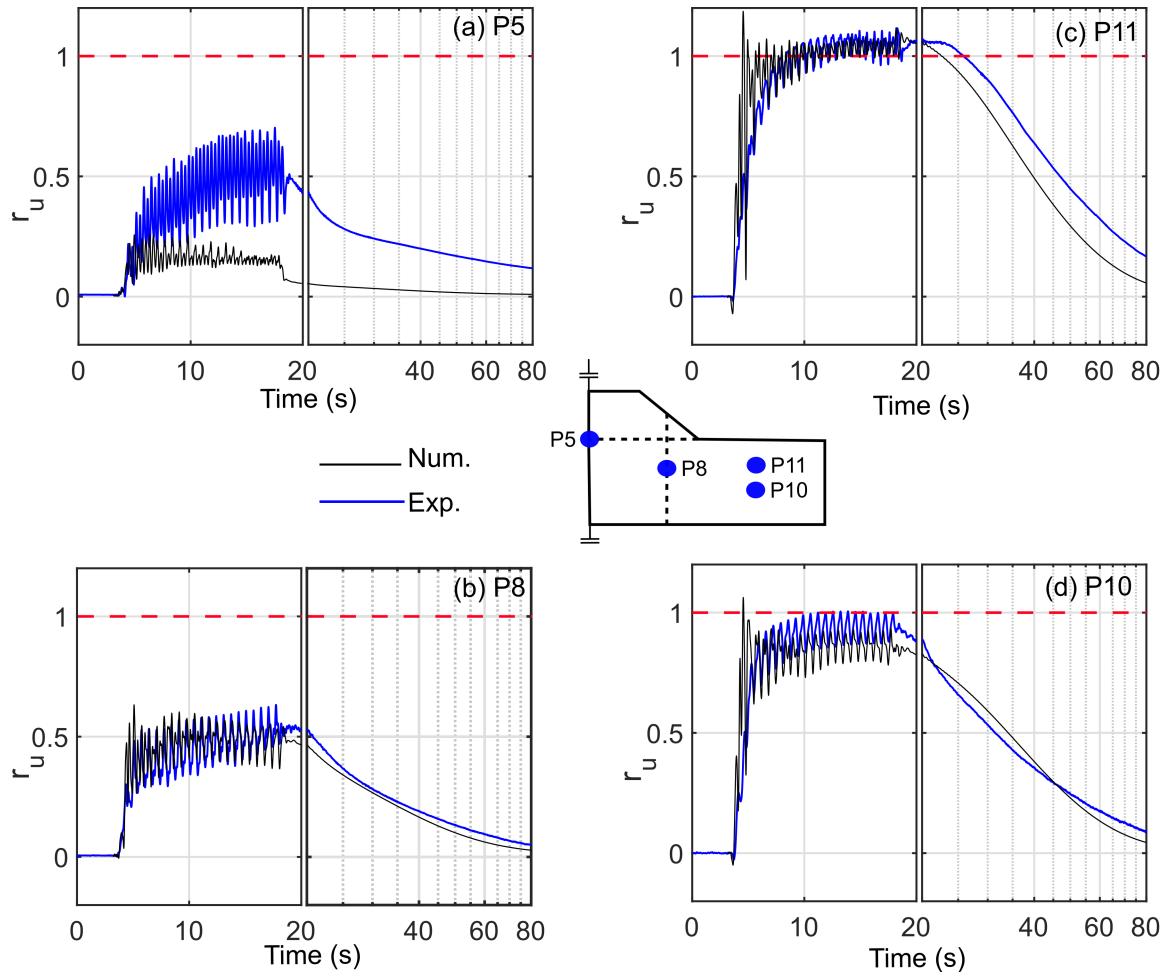


Figure 16: Comparison of numerical and experimental results in terms of excess pore pressure ratio under the embankment (a) P5 and (b) P8; under free ground surface (c) P11 and (d) P10

366 5.3. Stress path of liquefaction

367 The stress paths calculated from the experimental data were compared to those extracted directly from the elements
 368 of the numerical model in Fig. 17. The numerical model captured a rapid decrease in the effective stress under the
 369 free ground surface as well as the mobilization and stabilization of the effective stress under the embankment. In
 370 terms of the number of cycles before liquefaction occurs, the numerical results indicate that liquefaction initiates

371 after one cycle. This is earlier than what was observed in the centrifuge test, where liquefaction occurred after two
 372 cycles. It should be noticed that the liquefaction stress paths in the experiment were calculated from the 1D horizontal
 373 accelerations and the excess pore pressure nearby, which were not as precise as the stress paths obtained directly from
 374 the individual elements of the FEM model.

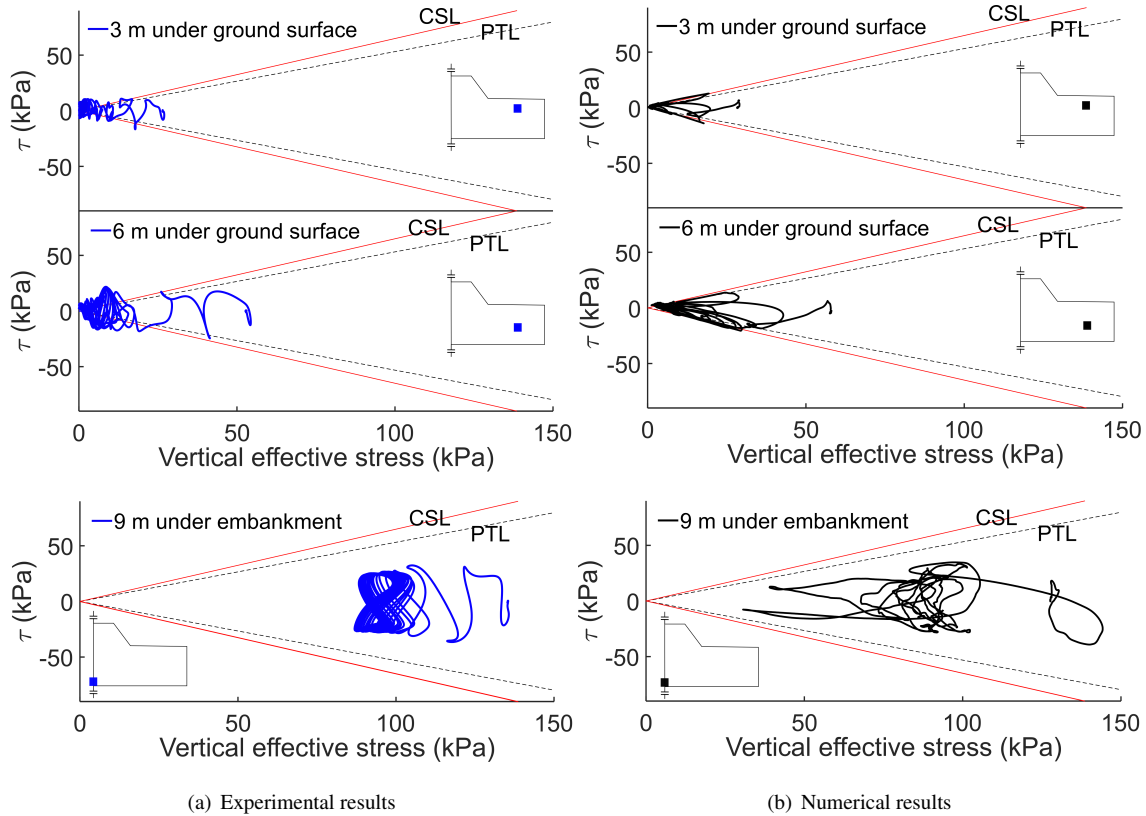


Figure 17: Stress paths of liquefaction under the free ground surface and under the embankment (a) obtained from the centrifuge test and (b) computed by the numerical model

375 5.4. Settlement and deformation

376 Fig. 18 compares the numerical and experimental results for the settlements of the embankment crest and free
 377 ground surface, including a parametric study that investigates the influence of the cohesion of the embankment soil
 378 on embankment settlement. In this study, the optimum value was selected for a good fit with experimental results.
 379 According to the parametric study shown in Fig. 18, it is found that the cohesion does not only influence the defor-
 380 mation of the embankment itself, but also the settlement of the free ground surface. The numerical model reflects the
 381 settlement and deformation trends that observed in the centrifuge test. Quantitatively, the settlement of the embank-
 382 ment crest measured in the centrifuge test is about 0.7 m, which is also captured by numerical model with 65 kPa of
 383 the cohesion. However, the heaving of the free ground surface from the numerical model using the selected cohesion
 384 (about 0.25 m) was larger than the experimental result (about 0.04 m). The significant difference in ground surface

385 heaving may be due to the influence of the rigid numerical boundary conditions. Additionally, the difference may be
 386 attributed to the presence of the self-weight of the potentiometer which can increase the settlement and at the same
 387 time limit the heaving during the centrifuge test. For the model deformation pattern, the response of the numerical is
 388 presented in Fig. 19 at the end of shaking ($t=18.26$ s). As explained before, there was no important liquefaction of soil
 389 beneath the embankment as it is shown in the contour plot of the excess pore pressure distribution. For the soil under
 390 the free ground surface, the excess pore pressure increased largely and reached the maximum value, which indicates
 391 the liquefied zones (in yellow). The liquefied zones, which extend from the toes of the embankment to the free ground
 392 surface, result in lateral displacement of the soil beneath the embankment and significant embankment settlement. In
 393 addition, a lateral displacement of the toes of embankments was observed followed by the heaving of the free ground
 394 surface.

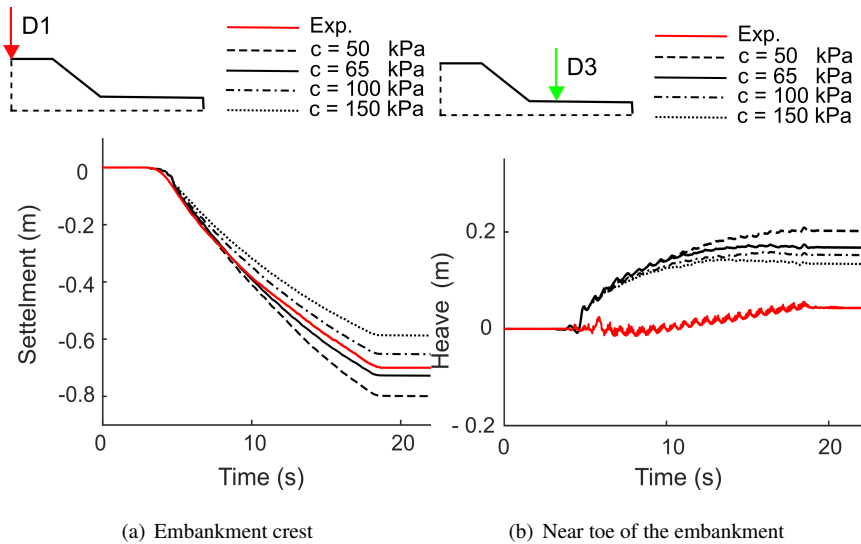


Figure 18: Settlements from numerical simulation and centrifuge test at the: (a) embankment crest and (b) free ground surface

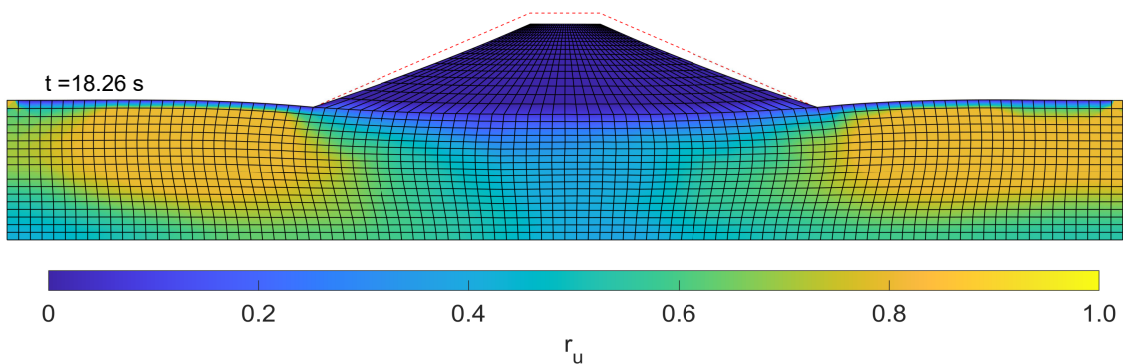


Figure 19: Deformation of the model and the distribution of excess pore pressure at the end of shaking ($t=18.26$ s)

395 6. Study of the influence of the boundary conditions on the responses of the model

396 The numerical model presented in section 5 was built exactly based on the dimensions and the boundary conditions
397 of the centrifuge model. The model shows good performance in terms of acceleration, excess pore pressure, settlement
398 and deformation. However, due to the limited dimension of the model, the results could be possibly influenced by
399 the boundaries. In order to clarify and quantify these influences and their impacts on the model responses, two new
400 models were built in this section (see Fig. 20): the first model with the free field conditions and the second model with
401 extended lateral boundaries.

402 The first model is similar to the original model presented in section 3, but the boundaries on the lateral side
403 were changed from rigid to flexible conditions. The flexible free-field conditions (or periodic boundary conditions
404 (Basu *et al.*, 2022)) were realized by imposing the same displacement on the nodes at the same depth by command
405 “EqualDOF” in OpenSees.

406 For the second model, the original model was extended by 150 m for each side of the ground soil as shown in
407 Fig. 20. According to the analyses of Kassas (2021) and Kassas *et al.* (2021a) on the boundary effect of a rigid
408 container, when the ratio of the boundary distance to the soil depth is larger than 8, the boundary effect of a rigid
409 container is negligible. In this study, for the model with extended boundaries, the ratio between the distance of
410 the embankment toe to the side and the ground soil depth is around 18.2. Therefore, it could be regarded that the
411 boundaries do not affect the response of the embankment or the soil in its vicinity in the extended model. The new
412 extended model keeps the same boundary conditions as the original model. The extended parts were modelled with
413 the same type of element and material constitutive law. The same analysis procedure explained in section 3.3 was
414 adopted.

415 In order to investigate the influence of the boundary condition on the response of the model, this section focuses
416 on the comparison of experimental results with the three numerical models in terms of acceleration, excessive pore
417 pressure, and settlements.

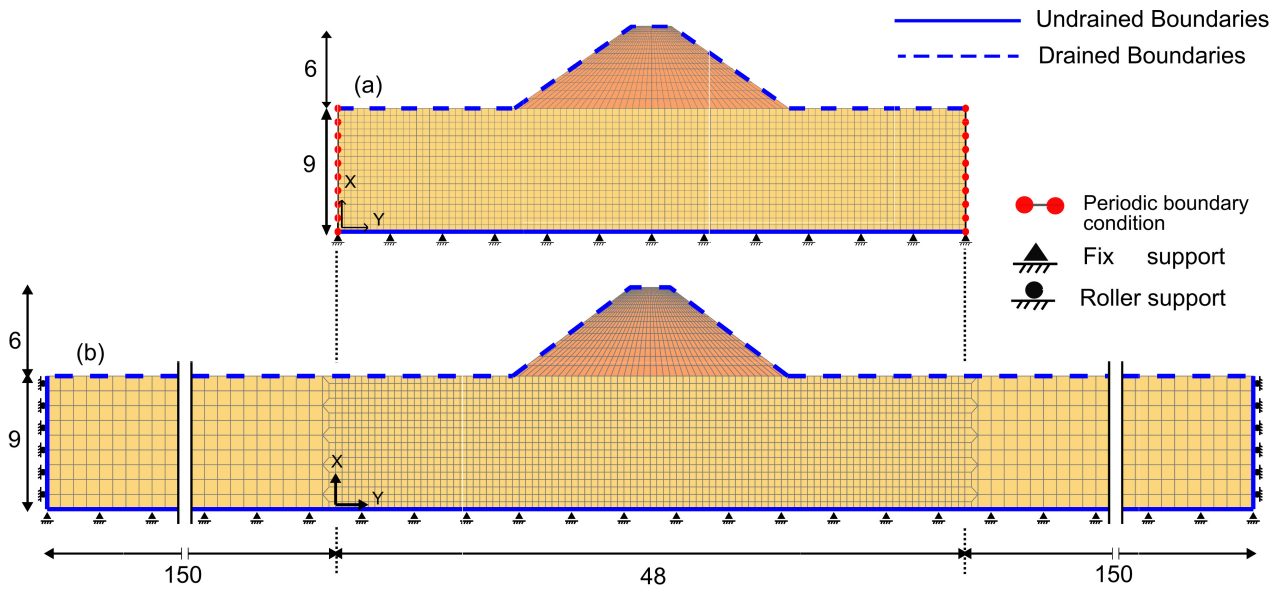
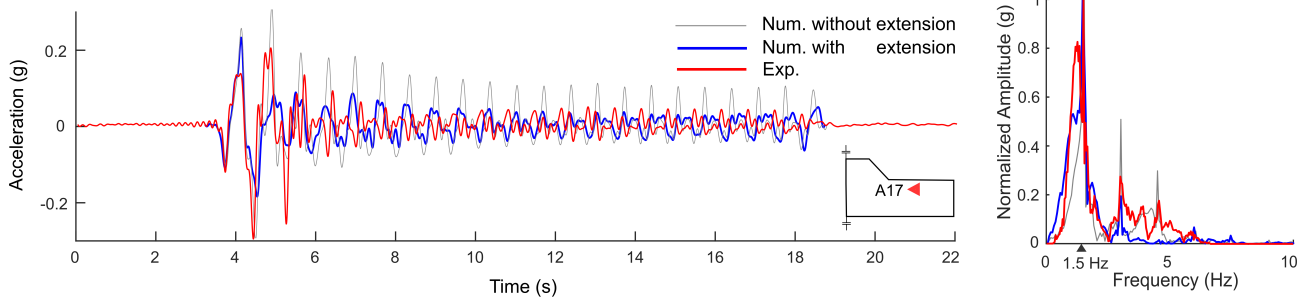
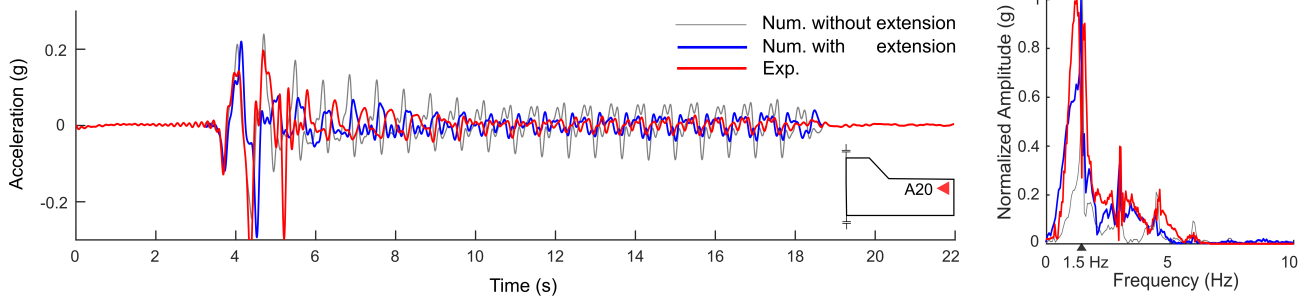


Figure 20: FEM mesh of the new models: (a) with free field conditions and (b) with extended boundaries of the ground soil (unit in prototype scale)

418 In terms of acceleration responses under the free ground surface (A17 and A20), Fig. 21 presents the comparison
 419 between the results obtained from the original model and the extended model; while Fig. 22 shows the comparison
 420 between the results obtained from the original model and the model with flexible free-field conditions. The numerical
 421 results from different models were also compared with the experimental results. The accelerations recorded during
 422 centrifuge test and those computed with numerical models show the same dominant frequency around 1.5 Hz in
 423 prototype scale. In terms of the time-history representation, the accelerations show similar trends for experimental
 424 and numerical models in which the amplitude decreases after two cycles. By quantifying the amplitude, the results
 425 obtained by the two new models (with extension and with free field condition) are closer to the experimental results
 426 and are better than the results from the original model. Overall, the original model can capture the main features
 427 of the experimental results, its responses are not significantly different from the two new models which have the
 428 more realistic boundaries. This indicates that (a) for the original numerical model, although certain boundary effects
 429 were observed, the numerical boundary conditions do not greatly affect the acceleration responses and (b) the rigid
 430 container used in the test did not affect significantly the behavior of the centrifuge model.



(a) Accelerations at A17



(b) Accelerations at A20

Figure 21: Comparison of experimental and numerical results (original model and model with extensions) in terms of amplitude and normalized frequency content of the acceleration at different positions: (a) A17 below the soil surface near embankment toe and (b) A20 near the container boundary

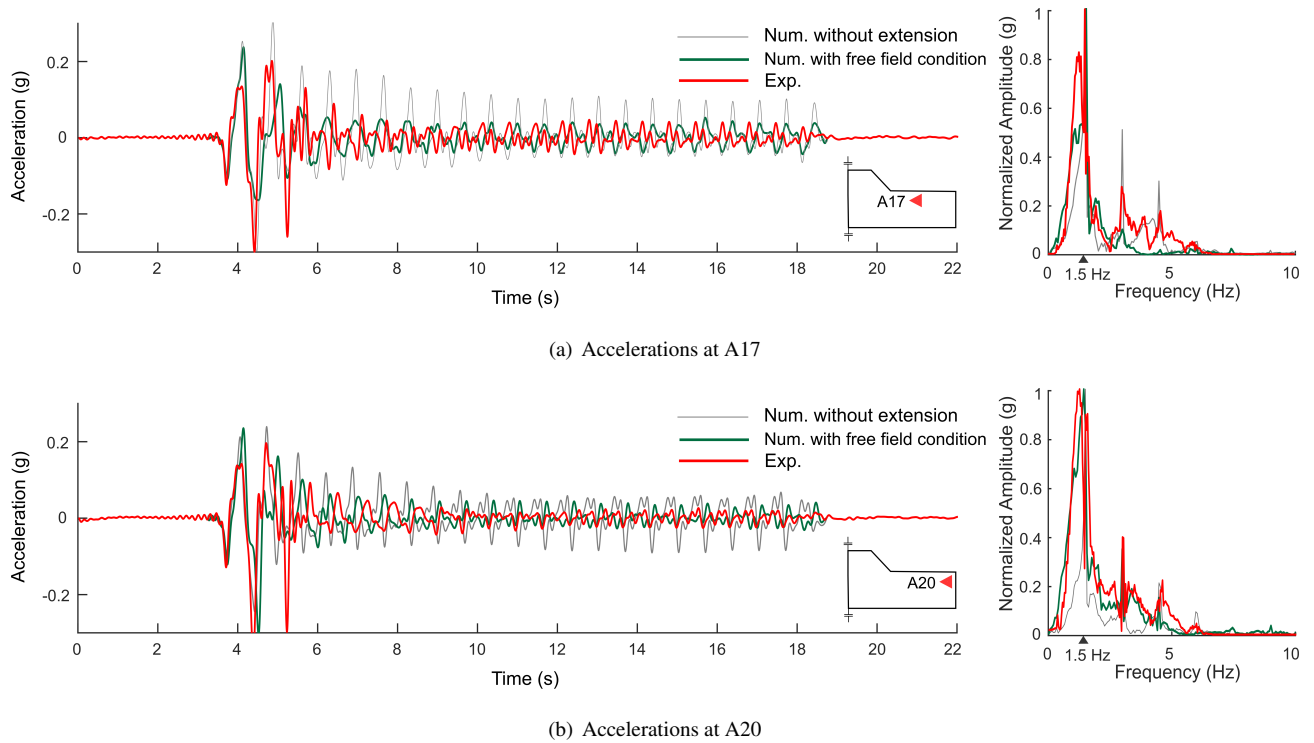
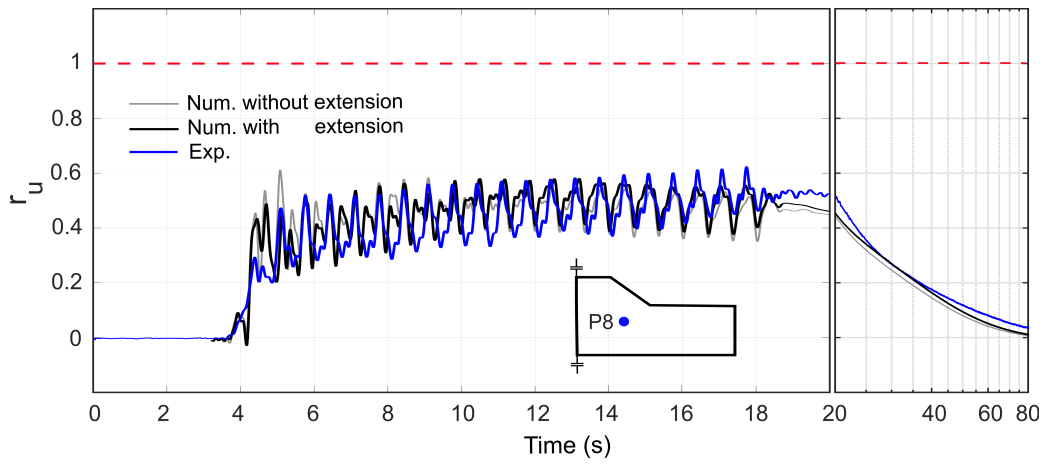


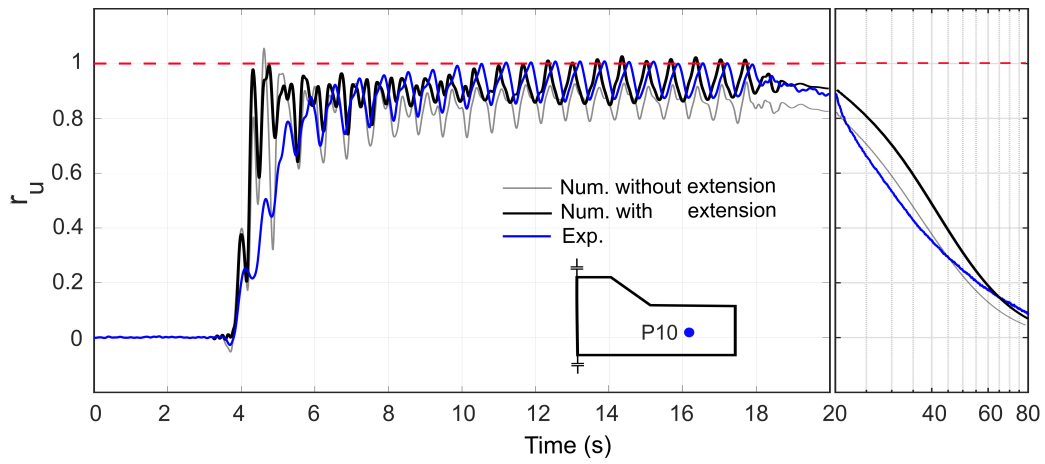
Figure 22: Comparison of experimental and numerical results (original model and model with free field condition) in terms of amplitude and normalized frequency content of the acceleration at different positions: (a) A17 below the soil surface near embankment toe and (b) A20 near the container boundary

431 The excess pore pressures obtained from the two new models were compared with experimental results in Fig. 23
 432 and Fig. 24. Under the embankment (P8), the excess pore pressure ratios obtained by the extended model and model
 433 with free field conditions are similar to the original numerical model and the experimental results. It indicates the
 434 numerical boundary conditions or the original model and the rigid lateral sides of the container used in the centrifuge
 435 test did not affect significantly the pore pressure response under the embankment.

436 For the position near the boundary of the container (P10), the build-up and dissipation of excess pore pressure
 437 computed with the three numerical models are comparable to each other and are also comparable to the experimental
 438 results. Under the free ground surface, the pore pressure increased rapidly (with $r_u \geq 0.9$) after 2 cycles. The
 439 liquefaction initiation was captured by the numerical models which is in accordance with the experimental results.
 440 This indicates that the numerical boundary conditions of the original numerical model and the rigid container used in
 441 centrifuge test did not have significant impacts the variation of pore pressure for the locations under the free ground
 442 surface.

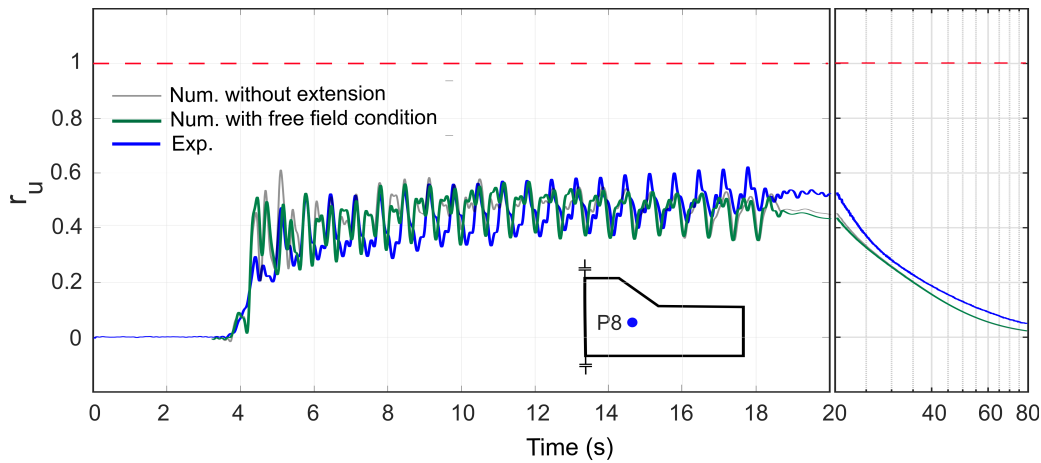


(a) Excess pore pressure at P8

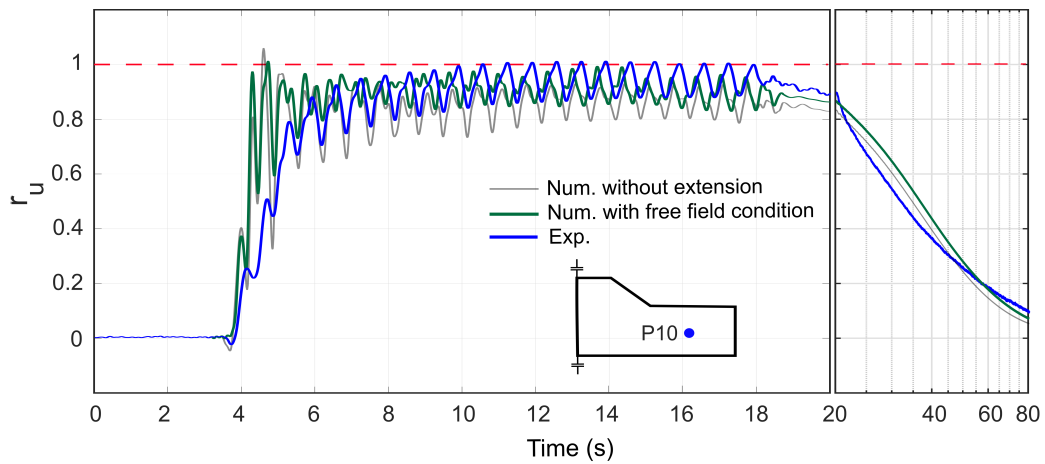


(b) Excess Pore pressure at P10

Figure 23: Comparison of experimental and numerical results (original model and model with extensions) in terms of excess pore pressures at different positions: (a) P8 under embankment and (b) P10 below the ground surface



(a) Excess pore pressure at P8



(b) Excess Pore pressure at P10

Figure 24: Comparison of experimental and numerical results (original model and model with free field condition) in terms of excess pore pressures at different positions: (a) P8 under the embankment and (b) P10 under the free ground surface

443 The embankment deformation and ground surface displacement recorded during the experimental test were also
 444 compared to the results of the three numerical models (with free field condition, with and without extension). The
 445 comparison is shown in Fig. 25. The deformation pattern was similar in all models: the embankment crest settled,
 446 accompanied by the heaving of the ground surface at a distance from the embankment toe (also observed in Fig. 26).
 447 At the embankment level, the settlement of the crest obtained by the model with free field condition (0.72 m) is similar
 448 to those of the model without extensions (0.70 m) (Fig. 25 (a)).

449 For the free ground surface near the toes of the embankment, the heaving simulated by the free field condition
 450 model (0.21 m) is close to the heaving in the original numerical model (0.23 m). For the model with extended
 451 boundaries, the crest settlement (0.82 m) is slightly larger than the others (experimental, original numerical model and
 452 free field condition model). The computed heave (0.14 m) is improved but still higher than the experimental results

453 (0.04 m).

454 Therefore, for the two models without extensions, the numerical boundary conditions affect slightly the crest
455 settlement and the heaving of free ground surface as shown in Fig. 25 (b). However, for the numerical model with
456 extensions, although the simulation of heaving on the free ground surface is improved, a slight overestimation of the
457 crest settlement is observed.

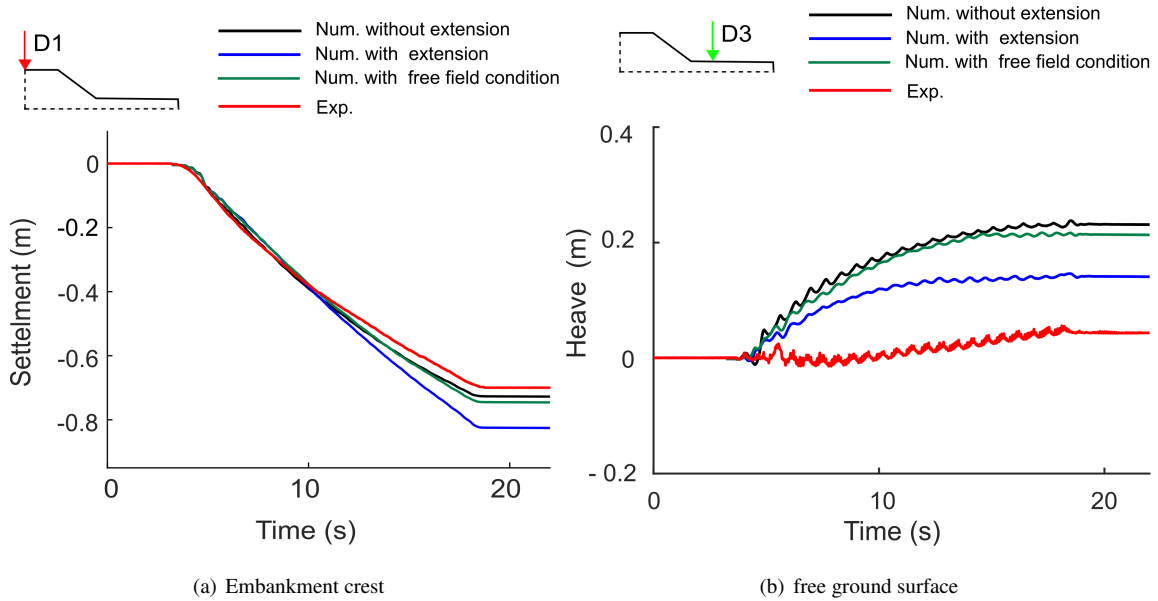


Figure 25: Vertical displacement at: (a) embankment crest and (b) free ground surface near the toe of embankment

458 Furthermore, the deformation and the distribution of the excess pore pressure ratio of the three numerical models
459 were presented in Fig. 26 after the shaking ($t=18.26$ s). A similar deformation pattern was observed in the three
460 models. Under the embankment, the excess pore pressure distribution has the same pattern for the three models.
461 Under the free ground surface, the pore pressure distribution was similar in the three models up to a distance of
462 around 5 m away from the lateral sides (marked in red in Fig. 26 (a)). In this zone, the pore pressure ratio obtained by
463 the numerical model was less than those of the other two models. This difference allows to define the zone affected
464 by the numerical boundaries (Saade *et al.*, 2022). For the positions close to the lateral sides of the container, due to
465 the influence of the boundaries, the liquefaction potential decreases.

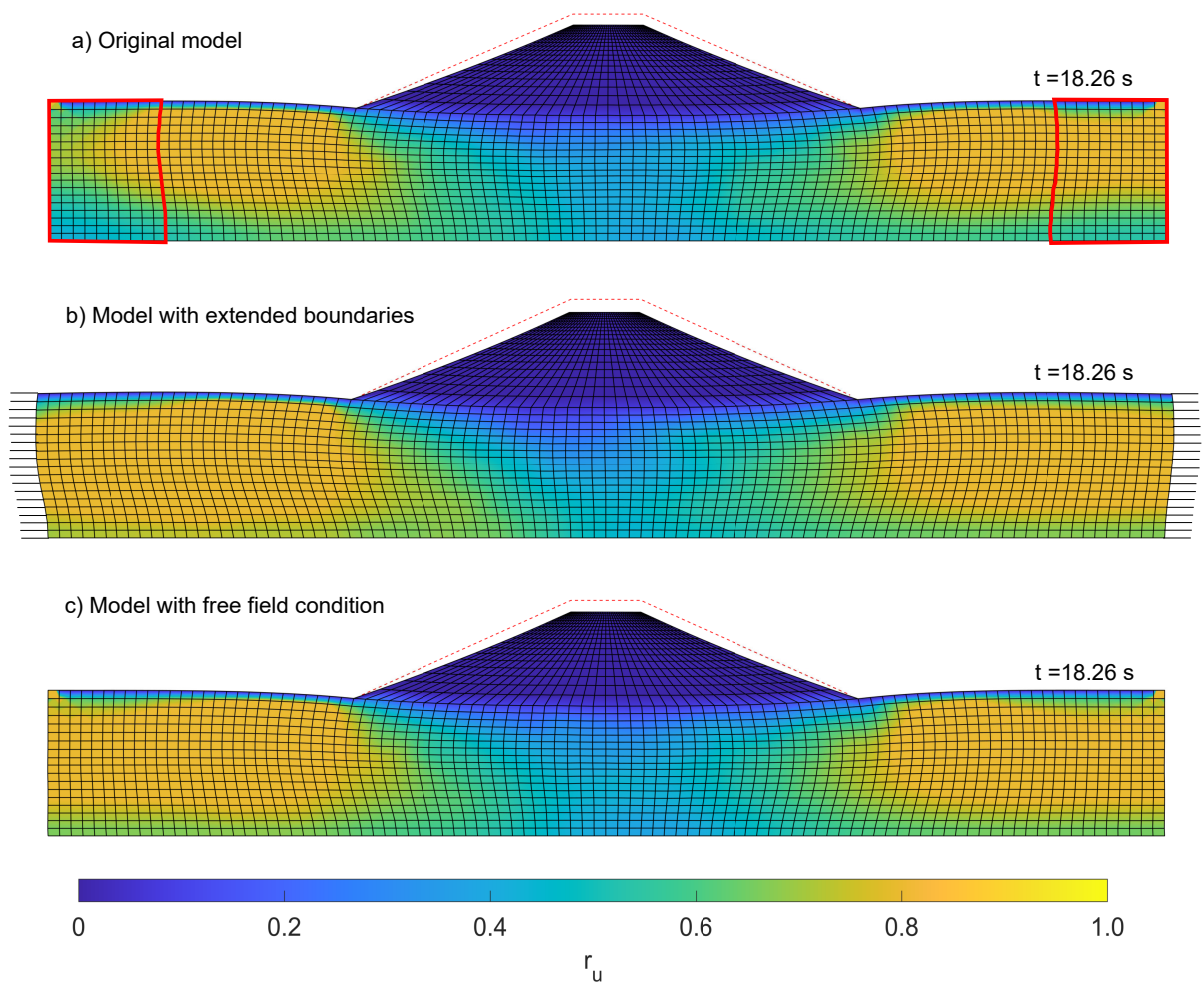


Figure 26: Deformation and the distribution of excess pore pressure at the end of shaking of the 3 numerical models: a) original model b) model with extended boundaries and c) model with free-field conditions

466 7. Conclusions

467 In this study, a dynamic centrifuge program was performed to have the direct observation and analysis of the
 468 behavior of a homogeneous embankment resting on liquefiable ground. Additionally, FEM numerical modeling was
 469 carried out using an advanced constitutive model – PM4Sand, and the performance of the numerical model was
 470 verified against the experimental results. With the verified numerical model, the boundary effects in the numerical
 471 simulation and dynamic centrifuge test were analysed and discussed. Based on the experimental and numerical results,
 472 the following conclusions can be drawn:

- 473 • In the centrifuge test, different patterns of time histories of the acceleration were observed. The signal was
 474 amplified with a ratio of approximately 2.12 from the bottom of the model to the crest of the embankment.
 475 The positions under the toes of the embankment had an asymmetrical acceleration profile due to the irregular

476 geometry of the model above those positions. For the positions under the free ground surface, the liquefaction
477 initiation was observed after two cycles of loading.

478 • The soil liquefaction was confirmed by the excess pore pressures and the stress paths at various positions in the
479 model. For the positions under the embankment with high confining pressure where liquefaction did not occur,
480 a limited excess pore pressure ratio was observed (around 0.5) and the stress path stabilized before complete
481 liquefaction occurred. For the position at a depth less than 6 m under the ground surface where the soil liquefied,
482 There was a significant increase in excess pore pressure after two cycles, accompanied by a notable decrease in
483 vertical effective stress.

484 • Soil liquefaction resulted in a soft ground that allowed the embankment to slide approximately as a rigid block.
485 A settlement of 0.7 m was observed at the crest of the embankment, along with a 0.2 m settlement at the middle
486 slope. Furthermore, a heaving of 0.04 m was observed at ground surface level near the toes, indicating a possible
487 lateral spreading of the toes embankment.

488 • The performance of the numerical model was verified by comparing the acceleration, excess pore pressures and
489 deformations against the experimental results. The numerical model provided reasonable simulation for lique-
490 faction and deformation responses. The symmetrical responses under the embankment and the asymmetrical
491 responses under the toes of the embankment were well captured. Furthermore, the numerical model accurately
492 simulated the attenuation of acceleration near the ground surface after two cycles. Additionally, the build-up
493 of the excess pore pressure was also well reproduced by the numerical model. The deformation trends of the
494 embankment were reflected in the numerical model, which were characterized by settlement at the embankment
495 level and heaving at the free ground surface.

496 • The boundary effects were analyzed using two new numerical models: an extended model (with over 150m on
497 each side) and a model with free field conditions. Regarding the results, the results from the centrifuge test
498 slightly differ from those from the extended model and the model with free-field conditions. This indicates that
499 the boundaries of the rigid container do not have significant impacts on the responses of the centrifuge model.
500 Based on the numerical results, a zone can be defined in which the behavior of the soil is slightly affected by
501 the numerical boundaries in the original model.

502 **CRedit author statement**

503 **Chedid SAADE:** Experiment execution, Numerical modeling execution, Data Analysis, Writing- Original draft
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505 **COFFIER:** Conceptualization, Co-supervision, Investigation, Reviewing and Editing. **Luc THOREL:** Conceptu-
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507 **Declaration of competing interest**

508 The authors declare no conflict of interest.

509 **Data availability statement**

510 The experimental data presented in the study are available upon direct request to the authors.

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