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# Numerical Evaluation of Earthquake Settlements of Road Embankments and Its Mitigation by Preloading

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1                                   **NUMERICAL EVALUATION OF EARTHQUAKE**  
2                                   **SETTLEMENTS OF ROAD EMBANKMENTS AND ITS**  
3                                   **MITIGATION BY PRELOADING**

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5                                   **ABSTRACT**

6                                   The present paper deals with assessing the seismic road embankment response due to  
7                                   pore water pressure generation of the soil foundation. Numerical simulations are carried out  
8                                   so as to study the preloading technique as an improvement method to reduce the liquefaction  
9                                   potential and the induced settlements in a sandy soil profile. The analyses showed that the  
10                                   use of preloading reduces the induced settlements mostly because of the increase in lateral  
11                                   confinement in the superficial soil layers due to the increase of the coefficient of lateral earth  
12                                   pressure at rest ( $k_o$ ). In addition, the efficiency of the countermeasure method is limited  
13                                   to the cases where earthquakes produced a liquefaction zone lower than the depth of the  
14                                   overconsolidated soil.

15                                   **Keywords:** Liquefaction-induced settlements, Road embankments, Site mitigation, Numer-  
16                                   ical simulation.

17                                   **INTRODUCTION**

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18 Geotechnical structures such as river dikes, highway embankments, and earth dams  
19 founded on saturated loose sandy ground have been frequently damaged during past major  
20 earthquakes (e.g. the 1964 Niigata Earthquake; the 1989 Loma Prieta Earthquake; the 1993  
21 Kushiro-oki Earthquake; the 1995 Hyogoken-Nanbu Earthquake and the 2011 off the Pacific  
22 Coast of Tohoku earthquake among others). Failures of the Kushiro river embankments,  
23 Japan (Sasaki et al. 1995), the Shiribeshi-Toshibetsu river embankments, Japan (Kaneko  
24 et al. 1995) and the Rimnio Bridge embankment, Grevena, Greece (Tika and Pitilakis 1999)  
25 are typical examples. This damage was usually due to liquefaction-induced ground defor-  
26 mations of the embankment and/or foundation sandy ground (Matsuo 1996; Adalier et al.  
27 1998; Unjoh et al. 2012; Okamura et al. 2013). According to Youd (1993) and Marcuson  
28 et al. (1996) among others, lateral soil deformations (“*lateral spreading*”) due to cyclic load-  
29 ing have proven to be the most pervasive type of liquefaction-induced ground failure. The  
30 amount of lateral displacement typically ranges from a few centimetres to several metres.

31 In practice, in order to prevent the damage effects of earthquake induced liquefaction in  
32 engineering structures, the countermeasure methods such as gravel drains, soil densification,  
33 solidification and inclusions or confinement walls among others are used. Such methods are  
34 studied by several authors via centrifuge tests (Liu and Dobry 1997; Adalier et al. 1998;  
35 Brennan and Madabhushi 2002; Elgamal et al. 2002; Dashti et al. 2010b; Mitrani and  
36 Madabhushi 2012) or numerically (Elgamal et al. 2002; Elgamal et al. 2005; López-Querol  
37 and Blázquez 2006; Nishimura and Shimizu 2008; Lu et al. 2011; Bradley et al. 2013;  
38 Lopez-Caballero and Modaressi-Farahmand-Razavi 2013; Xu et al. 2013). The principal  
39 conclusion of these works is that the efficiency of each solution depends on characteristics of  
40 the input signal and the properties of the soil.

41 Preloading is a temporary loading, usually an embankment, applied at a construction  
42 site to improve subsurface soils. For construction sites where sandy layers are predominant,  
43 experience has illustrated that about three weeks suffice for soil improvement to take place.  
44 The method is frequently used to improve bad soil conditions and make them sustain large

45 static loads (Stamatopoulos and Kotzias 1985; Petridis et al. 2000). Surcharging the site  
46 increases the liquefaction strength of soil foundation and reduces the development of excess  
47 pore pressure. In addition, an increase in lateral stress is obtained due to the modification  
48 of coefficient of earth pressure at rest ( $k_o$ ) in the improved soil. The increase in horizontal  
49 stress and liquefaction cyclic strength by preloading has been studied and verified by both  
50 laboratory tests (Ishihara and Okada 1982; Wichtmann et al. 2005; Bouferra et al. 2007;  
51 Porcino et al. 2009; Liu and Xu 2013) and elaborate field tests (Stamatopoulos et al. 2005;  
52 Raptakis 2013; Stamatopoulos et al. 2013).

53 Previous works have used numerical methods to study the response of both liquefiable  
54 and improved (i.e. densified) soil profiles and their effects on the structures or dams re-  
55 sponse (Elgamal et al. 2002; Adalier and Aydingun 2003; Elgamal et al. 2005; Shahir and  
56 Pak 2010). In general, most of those numerical models are related to centrifuge modelling  
57 test results. Those researches were mainly focused on the effect of a dense region representing  
58 the improved method and placed underneath the structure on the settlement induced by soil  
59 liquefaction. Those compacted regions were modelled with constitutive model parameters  
60 calibrated for a denser soil. On the contrary, to investigate these issues, in this study at-  
61 tention is given to the construction phase of the preloading embankment. The construction  
62 and demolition of the preload embankment (i.e construction technique used in this counter-  
63 measure method) is simulated and the effects of this loading history on the seismic response  
64 of the soil profile are assessed.

65 The aim of this work is to assess numerically the efficiency of the soil densification using  
66 preloading techniques on mitigating excessive seismic ground settlements of road embank-  
67 ments on liquefiable sandy profiles. The numerical results of parametric studies performed  
68 to compare the dynamical response of the road embankment at the end of shaking for the  
69 cases with and without mitigation method are discussed.

70 First of all, a description of the used advanced constitutive model is provided. The fol-  
71 lowing section is concerned with the relevant aspects of the numerical model, namely, the

72 geometry of the road embankment, the boundary conditions, the simulation of the improve-  
73 ment method and the characteristics of the input ground motions. Then, the elastoplastic  
74 constitutive model is calibrated by simulating a series of laboratory cyclic tests. In addition,  
75 the model's ability to simulate the effect of preloading on the resistance to the sands lique-  
76 faction has been assessed. Finally, a comparison of the performance of the road embankment  
77 with and without the improvement method is presented.

## 78 SOIL CONSTITUTIVE MODEL

79 The elastoplastic multi-mechanism model developed at *Ecole Centrale Paris*, known as  
80 ECP model (Aubry et al. 1982; Hujeux 1985) is used to represent the soil behaviour. This  
81 model can take into account the soil behaviour in a large range of deformations. The model  
82 is written in terms of effective stress. The representation of all irreversible phenomena is  
83 made by four coupled elementary plastic mechanisms: three plane-strain deviatoric plastic  
84 deformation mechanisms in three orthogonal planes and an isotropic one. The model uses a  
85 Coulomb-type failure criterion and the critical state concept. The evolution of hardening is  
86 based on the plastic strain (deviatoric and volumetric strain for the deviatoric mechanisms  
87 and volumetric strain for the isotropic one). To take into account the cyclic behaviour a  
88 kinematical hardening based on the state variables at the last load reversal is used. The  
89 soil behaviour is decomposed into pseudo-elastic, hysteretic and mobilized domains. Refer  
90 to Aubry et al. (1982), Hujeux (1985), Lopez-Caballero and Modaressi-Farahmand-Razavi  
91 (2008) among others for further details about the ECP model. For the sake of brevity only  
92 some model definitions are given in what follows.

93 Adopting the soil mechanics sign convention (compression positive), the deviatoric pri-  
94 mary yield surface of the  $k$  plane in the ECP soil constitutive model is given by:

$$95 \quad f_k(\sigma, \varepsilon_v^p, r_k) = q_k - \sin \phi'_{pp} \cdot p'_k \cdot F_k \cdot r_k \quad (1)$$

96 where,  $p'_k$  and  $q_k$  are the mean and deviatoric values of stress tensors,  $\phi'_{pp}$  is the friction angle

97 at the critical state, the function  $F_k$  permits to control the isotropic hardening associated  
 98 with the plastic volumetric strain, whereas  $r_k$  accounts for the isotropic hardening generated  
 99 by plastic shearing. They represent progressive friction mobilization in the soil and their  
 100 product reaches unity at perfect plasticity. Therefore, in order to provide for any state  
 101 a direct measure of “*distance to reach the critical state*” ( $r_k$ ) and based upon the used  
 102 elastoplastic model, it is possible to define an apparent friction angle ( $\phi'_{apt}$ ) by:

$$103 \quad \sin \phi'_{apt} = \frac{q_k}{p'_k \cdot F_k} \quad (2)$$

$$104 \quad r_k = \frac{\sin \phi'_{apt}}{\sin \phi'_{pp}} \quad (3)$$

105 The parameters of the ECP model concern both the elastic and plastic behaviour of the  
 106 soil and they are separated into two categories: those that can be directly measured from  
 107 either *in-situ* or laboratory test results and those which, cannot be directly measured (Table  
 108 1).

## 109 NUMERICAL MODEL

110 For the purpose of studying numerically the effect of soil preloading technique on the  
 111 improvement of liquefiable sandy profiles to shaking, a road embankment founded on a  
 112 layered soil/rock model is considered. A scheme of the used numerical model and the FE  
 113 mesh are given in Fig. 1(a) and Fig. 1(b) respectively. The geometry used in the FE model  
 114 was inspired from the one proposed by López-Querol and Blázquez (2006).

115 The FE analysis is performed in four consecutive steps : i) Computation of the initial  
 116 *in-situ* stress state due to gravity loads; ii) A sequential level-by-level construction and  
 117 demolition of the preload embankment; iii) A sequential level-by-level construction of the  
 118 road embankment and iv) seismic loading analysis in the time domain.

119 The soil profile is composed principally of 15m of loose-to-medium sand (i.e. a relative  
 120 density  $Dr < 50\%$ ) and to take into account the stress dependency of sand behaviour it is  
 121 divided in three layers. The model parameters of each layer are adjusted so as to keep the

122 same  $D_r$  value. The shear modulus of the soil increases with the depth. The initial modulus  
123 ( $G_{max}$ ) is computed by the following relationship :

$$124 \quad G_{max} = 290 \left( \frac{\sigma'_{mo}}{p_{ref}} \right)^{0.5} \quad [\text{MPa}] \quad (4)$$

125 where  $\sigma'_{mo}$  is the mean effective pressure and  $p_{ref}$  is the pressure of reference (1MPa).

126 The low-strain frequency analysis provides a fundamental elastic period of the soil profile  
127 equal to 0.31s. It is obtained from the transfer function at free field condition, (i.e. ratio of  
128 the frequency response at the soil surface over the bedrock frequency response for a sample  
129 seismic signal at very low amplitude to ensure elastic soil behaviour). The elastoplastic  
130 multi-mechanism model briefly described before is used to represent the soil behaviour on  
131 the top 15m. For the engineering bedrock representing a half-space and placed at 15m depth,  
132 an isotropic linear elastic behaviour with a  $V_s$  equal to 550m/s is assumed. The ground water  
133 table level is placed at 1m below the surface. The model is 80 m wide so as to ensure that  
134 the effect of the boundaries on the model response can be neglected and also to satisfy the  
135 free field condition at the lateral boundaries.

### 136 **Finite Element (FE) model**

137 A 2D coupled dynamic approach derived from the  $\underline{u}-p_w$  version of the Biot's generalized  
138 consolidation theory (Zienkiewicz and Taylor 1991) was adopted for the soil. The so-called  
139  $\underline{u}-p_w$  formulation, consists of neglecting fluid acceleration terms and convective terms of this  
140 acceleration so that the unknown variables remain the displacement of the solid  $\underline{u}$  and the  
141 pressure of the water  $p_w$ . The saturated soil was modelled using quadrilateral isoparametric  
142 elements with eight nodes for both solid displacements and fluid pressures. The size of  
143 elements is 0.5m×0.5m. It was chosen in order to have 8 to 10 elements per wavelength which  
144 are sufficient to prevent numerical dispersion. An implicit Newmark numerical integration  
145 scheme with  $\gamma = 0.625$  and  $\beta = 0.375$  is used in the dynamic analysis (Kuhl and Crisfield  
146 1999), this allows an optimal high-frequency dissipation with minimal low-frequency impact

147 and adds a small numerical damping ( $\xi \approx 0.2\%$ ). A plane-strain condition was assumed in  
148 the finite element model.

149 In order to investigate the effect of the preloading method on the response of the soil  
150 profile, a comparative dynamical response analysis at the end of shaking for the cases with  
151 and without mitigation method is done.

## 152 **Boundary conditions**

153 In the analysis, only vertically incident shear waves are introduced into the domain  
154 and as the response of an infinite semi-space is modelled, equivalent boundaries have been  
155 imposed on the nodes of lateral boundaries (i.e. the normal stress on these boundaries  
156 remains constant and the displacements of nodes at the same depth in two opposite lateral  
157 boundaries are the same in all directions). The model length is 80m. It ensures that the  
158 effect of the boundaries in the model can be neglected and it satisfies the free field condition  
159 at the lateral boundaries.

160 For the half-space bedrock's boundary condition, paraxial elements simulating “*deformable*  
161 *unbounded elastic bedrock*” have been used (Modaressi and Benzenati 1994). The incident  
162 waves, defined at the outcropping bedrock are introduced into the base of the model after  
163 deconvolution. Thus, the obtained movement at the bedrock is composed of the incident  
164 waves and the reflected signal.

## 165 **Preloading simulation**

166 In order to simulate the construction and demolition of the preload embankment (Fig.  
167 1(b)), the calculations are performed in two steps. In the first step, since soil behaviour  
168 is a function of the effective stress state for nonlinear elastoplastic models, initial *in-situ*  
169 stress state due to gravity loads are computed. After this initialization, the displacements  
170 and deformations are eliminated and the initial effective stresses, pore-water pressures and  
171 model history variables are stored to be used as initial state of the second step computation.  
172 In the second one, a sequential level-by-level construction and demolition of the embankment  
173 is performed.



174 The preload embankment is supposed to be in dry condition. It was simulated using  
175 quadrilateral isoparametric elements with eight nodes for solid displacements (Fig. 1(b)).  
176 The preload embankment height ( $H_{embk}$ ) is 7.5m with a density equal to 1800kg/m<sup>3</sup>. The  
177 fill length is 38m and the length at the crest is 4m. The preload embankment is constructed  
178 and demolished at a rate of 0.22 m/day and it stays in place during 23 days before the  
179 construction of the road embankment and the application of the seismic event. After this  
180 period and according to our computations all over pore pressures are dissipated.

### 181 **Road embankment model**

182 The road embankment was simulated using quadrilateral isoparametric elements with  
183 eight nodes for solid displacements (Fig. 1(b)). It is supposed to be in dry condition. It is 5m  
184 high, 8m wide at the crest and 20m wide at the base. Its density ( $\rho_{road}$ ) is equal to 1800kg/m<sup>3</sup>.  
185 It consists of a base of dense sand (i.e.  $Dr > 70\%$ ). The soil behaviour is represented by  
186 an elastoplastic multi-mechanism model. The model parameters are summarized in Table 3.  
187 The road embankment is constructed at the same rate as that of the preload one.

188 A 0.40m thick asphaltic pavement is placed on the top of the road embankment. It is  
189 assumed to have a linear elastic behaviour, with an elastic modulus, ( $E_{asp}$ ), of 6000MPa, a  
190 Poisson's ratio, ( $\nu_{asp}$ ), of 0.35 and a density, ( $\rho_{asp}$ ), equal to 1800kg/m<sup>3</sup>. It was simulated  
191 using elastic beam-column elements.

### 192 **Input earthquake motion**

193 In order to define appropriate input motions to the non-linear dynamic analysis, a selec-  
194 tion of recorded accelerograms are used. The adopted earthquake signals are proposed by  
195 Iervolino and Cornell (2005) and Sorrentino et al. (2008). Thus, 76 unscaled records were  
196 chosen from the Pacific Earthquake Engineering Research Center (PEER) database. The  
197 events range between 5.2 and 7.6 in magnitude and the recordings have site-to-source dis-  
198 tances from 15 to 50km and dense-to-firm soil conditions (i.e.  $360\text{m/s} < V_s_{30m} < 800\text{m/s}$ ).

199 Concerning the response spectra of input earthquake motions, Fig. 2 shows the mean  
200 and the response spectra curves (structural damping  $\xi = 5\%$ ) of the input motions. It can

201 be noted that the mean response spectra is consistent with the response spectra of Type A  
 202 soil of Eurocode8 scaled to the mean maximal outcropping acceleration value. The statis-  
 203 tics on some input earthquake characteristics obtained for the strong ground motions are  
 204 summarized in Table 2. These earthquake characteristics are maximal outcropping accel-  
 205 eration ( $PHA$ ), Arias intensity ( $I_A$ ) (Arias 1970), predominant period ( $T_p$ ), mean period  
 206 ( $T_m$ ) (Rathje et al. 1998), peak ground velocity ( $PGV$ ), period of equivalent harmonic wave  
 207 ( $T_{V/A} = \alpha \cdot PGV/PHA$ ), spectral intensity ( $SI$ ), root-mean-square intensity ( $I_{rms}$ ) (Kout-  
 208 sourelakis et al. 2002), significant duration from 5% to 95% Arias intensity ( $D_{5-95}$ ) and  
 209 Specific Energy Density ( $SED$ ).

## 210 LABORATORY TESTS SIMULATIONS

211 In order to show both a global view of the response of the ECP elastoplastic model  
 212 and the coherence of the parameters set used to simulate the sand behaviour, several soil  
 213 mechanics tests are modelled. All tests are simulated with the same set of parameters.  
 214 The soil mechanics tests concern both drained cyclic strain-controlled and undrained stress  
 215 controlled cyclic shear tests paths at different consolidation pressures

216 The model parameters of three layers are summarized in Table 3. They were determined  
 217 with the methodology explained in Lopez-Caballero et al. (2007). In this methodology  
 218 a unique and coherent set of the ECP model parameters is identified by modelling both  
 219 monotonic and cyclic tests. As the model parameters are classified in two categories: directly  
 220 and non-directly measurable parameters. In the first one, correlations or *in-situ* are used to  
 221 define the parameter values. In the second one, the parameters are fixed by curve fitting.  
 222 Fig. 3 shows the responses of the drained cyclic shear tests obtained by the model of the  
 223 sand at  $p'_o = 30, 50, 100$  and  $150$  kPa. The tests results are compared with the reference  
 224 curves given by Seed et al. (1986). It is noted that the obtained  $G/G_{max} - \gamma$  curves match  
 225 relatively good though for strains less than  $0.01\%$  the damping  $D$  is underestimated while  
 226 for large strains it is overestimated.

227 The obtained curves of cyclic stress ratio ( $SR = \sigma_{v-cyc}/(2 \cdot p'_o)$ ), with  $\sigma_{v-cyc}$  the cyclic

228 vertical stress applied in the cyclic loading) in a triaxial path with isotropic consolidation  
 229 as a function of the number of loading cycles to produce liquefaction ( $N$ ) at  $p'_o = 30, 50$   
 230 and 100kPa are given in Fig. 4. As qualitative comparison, the modelled test results are  
 231 compared with the obtained curves given by Byrne et al. (2004) for Nevada sand at different  
 232 densities (i.e.  $D_r = 40\%$  and  $60\%$ ). It is noted that the obtained curves are placed between  
 233 the experimental ones. It could be noted that the ECP elastoplastic model is able to simulate  
 234 realistic liquefaction,  $G - \gamma$  and  $D - \gamma$  curves. In addition, according to these figures, the  
 235 model's response for both loading paths is coherent for similar initial conditions.

236 So as to study the effect of overconsolidation on the cyclic strength some isotropic con-  
 237 solidation tests were simulated at  $p'_o = 30$  and  $50$ kPa. Three levels of overconsolidation ratio  
 238 ( $OCR$ ) were studied (i.e. 2, 3 and 4). The  $OCR$  is defined as :

$$239 \quad OCR = \frac{\sigma'_{v-c}}{p'_o} \quad (5)$$

240 where  $\sigma'_{v-c}$  is the maximum effective stress exerted during consolidation and  $p'_o$  is the effective  
 241 stress just prior to the application of cyclic loading. It is important to remark that in this  
 242 study only the behaviour of the case when  $OCR = 1$  is defined. Then, as in a real laboratory  
 243 test, the completely loading path is simulated. It means that to reach a predefined  $OCR$   
 244 value, an isotropic consolidation loading-unloading path is first simulated followed by a cyclic  
 245 loading path. Fig. 5 displays the liquefaction curves obtained for the two initial stresses and  
 246 two  $OCR$  values. It is noted that for similar confining stress, the  $SR$  value increase as  
 247  $OCR$  increases, as shown in Adalier and Elgamal (2005), Wichtmann et al. (2005) and  
 248 Stamatopoulos and Stamatopoulos (2007). These results allow to show the ability of the  
 249 numerical code to simulate the effect of preloading in cyclic liquefaction strength.

## 250 LIQUEFACTION ANALYSIS

251 Before studying the seismic response of the road embankment without preloading, it  
 252 is necessary to assess the modification on the behaviour of the soil foundation behaviour

253 due to the presence of the road structure relative to the free field condition. As shown in  
254 Fig. 6(a) the road embankment load influences the soil behaviour down to 7.5m depth (i.e.  
255  $\Delta\sigma_{zz} > 50\text{kPa}$ ). Thus, it is expected that it produces a static bearing capacity degradation  
256 underneath the road embankment. According to the model parameters given in Table 3 and  
257 the Eq. 3, an initial value of the “*distance to reach the critical state*” ( $r_k$ ) (i.e. free field  
258 condition) equal to 0.5 is obtained. The contours of  $\Delta r_k$  value (Fig. 6(b)) show that after  
259 the road embankment construction the  $r_k$  value at shallow depths increases to near to 1.0  
260 (i.e.  $\Delta r_k \approx 0.5$ ).

261 In order to define the liquefaction reference case, the responses obtained by the model  
262 without preloading are analysed. Fig. 7 shows both a typical deformed configuration of the  
263 road embankment model and the distribution of induced horizontal displacements at the  
264 end of the earthquake loading. It could be noted that the liquefaction of soil foundation  
265 produces a lateral spreading of the foundation soil towards the free field. In addition, the  
266 liquefaction induces settlements at the crest due to the combined action of the migration of  
267 the underneath foundation soil and the deformation of the road embankment itself.

268 Fig. 8(a) displays the induced excess pore pressure ( $\Delta p_w$ ) distribution with depth in  
269 the soil profile below the embankment’s foundation at the end of the signal (i.e. coseismic  
270 analyses) for all simulations. In this study, the end of shaking is defined as the time  $t$  that  
271 corresponds to the 95% of Arias intensity  $D_{95IA}$ . It can be noted that, the flow liquefaction  
272 (i.e.  $\Delta p_w = \sigma'_{vo}$ ) or the full loss of effective stress in the soil does not occur for any input  
273 signal.

274 However, as expected, the development of excess pore pressure during the earthquake  
275 loading induces a shear strength degradation of the foundation soil which results in an  
276 accumulation of seismic settlements (Bouckovalas et al. 1991; Liu and Dobry 1997; Elgamal  
277 et al. 2002; Dashti et al. 2010a; Dashti et al. 2010b; Karamitros et al. 2013). Furthermore,  
278 co-seismic displacements are almost identical to permanent displacements. Fig. 8(b) shows  
279 the obtained co-seismic relative settlement at the base of the road embankment (i.e. point

280  $FF_{pr}$  in Fig. 1(a)) induced by the pore water pressure build-up. The relative settlement  
 281 of the road structure is computed as the difference between the settlement of the road  
 282 embankment foundation and the free field settlement (i.e. point  $FF_{Np}$  in Fig. 1(a)). It can  
 283 be seen that, the co-seismic relative settlement value increases with an increase in the Arias  
 284 intensity ( $I_A$ ) value, that is, an increase into the input seismic energy. This finding is similar  
 285 with recent works which correlate with good accuracy sliding-block seismic displacement  
 286 to Arias intensity (Chousianitis et al. 2014). It can be seen that most of the obtained  
 287 relative settlements are less than 10cm ( $\approx 60\%$  of simulations). According to the limit  
 288 states suggested by Bird et al. (2006) for rigid body settlement, it corresponds to slight ( $\leq$   
 289 10cm) damage level. According to SCDOT (2010), the maximum vertical settlement along  
 290 the profile grade over the design life of the embankment is 20cm (i.e. performance limit  
 291 EV-01).

292 In Fig. 9(a), the relative horizontal displacement ( $U_H$ ) time histories for a point near  
 293 to the slope of the road embankment (i.e. point  $Rd_{in}$  in Fig. 1(a)) are illustrated for two  
 294 typical earthquakes. As recalled before, the  $U_H$  of the road structure is computed as the  
 295 difference between the horizontal displacement of the road embankment (i.e. point  $Rd_{in}$ )  
 296 and the free field horizontal displacement (i.e. point  $FF_{Np}$  in Fig. 1(a)).

297 Finally, Fig. 9(b) displays the maximum  $U_H$  induced by the pore water pressure build-up  
 298 as a function of the ( $I_{Arias}$ ) value of input earthquake. It is observed that the maximum  $U_H$   
 299 value is about four times less than the maximum settlement value and it increases with an  
 300 increase in the Arias intensity value.

## 301 ANALYSIS OF IMPROVEMENT METHOD

302 In this section, the change in soil state due to the construction and demolition of the  
 303 preload embankment is analyzed. The reference state corresponds to the soil foundation  
 304 state after the construction of the road embankment. Normally, the application of preload-  
 305 ing produces an overconsolidation on the soil ( $OCR = \sigma'_{v-c}/\sigma'_{vo}$ ) and a residual horizontal  
 306 effective stress  $\Delta\sigma_{yy}$  appears at the end of the demolition of the embankment. However, due

307 to the adopted preloading configuration (Fig. 1) the induced  $OCR$  was concentrated at the  
308 toe of the road slopes, namely, at the place of the preload embankment (Fig. 10(a)). At  
309 these places the embankment load influences the soil behaviour down to 3 to 4m depth (i.e.  
310  $OCR > 2$ ). In addition, comparing the induced horizontal effective stress for the case with  
311 and without preloading ( $\Delta\sigma_{yy}$ ) (Fig. 10(b)), it is interesting to note that at the toe of the  
312 road slopes, a residual  $\Delta\sigma_{yy}$  appears.

313 Regarding the  $r_k$  values, the contours of  $\Delta r_k$  value (Fig. 11) show that the mitigation  
314 method produces an improvement on the static bearing capacity (i.e.  $\Delta r_k \leq 0$ ) in the  
315 same places. This final configuration corresponds to the case named “*Densification*” in the  
316 series of centrifuge tests proposed by Adalier et al. (1998). The improvement on the static  
317 bearing capacity is confirmed by the fact that the settlement evolution obtained during the  
318 construction of the road embankment is less than 20 times when the preloading was used  
319 (Fig. 12).

320 Concerning the seismic response of the soil profile, the remediation method used increases  
321 the strength at the toe of the road which contains the loose sand stratum underneath the  
322 embankment centre and in consequence, it will decrease the relative settlement (Fig. 13(a)).  
323 As illustrated in this figure, the co-seismic structural relative settlement obtained after soil  
324 improvement is greatly reduced as a consequence of soil stiffening (e.g. more than two times).  
325 This results are in agreement with those obtained by Adalier et al. (1998) and Elgamal et al.  
326 (2002) among others. Fig. 13(b) displays the variation of  $\Delta settl = settl_{mit}/settl_o$  (where  
327  $mit$  and  $o$  subscripts refer to values after and before mitigation respectively) as a function  
328 of the  $I_A$  value. It could be remarked, that the reduction is most important for the lower  
329 values of  $I_A$ . In addition, the onset of the slight damage level apparition is shifted for the  
330 higher values of  $I_A$ , passing from 0.3m/s to 0.6m/s.

331 Moreover, so as to quantify the extent of liquefaction into the profile below the embank-  
332 ment foundation, the Liquefaction Index ( $Q_H$ ) is computed. This parameter is defined as

333 follows (Shinozuka and Ohtomo 1989):

$$334 \quad Q_H(t) = \frac{1}{H} \int_0^H \frac{\Delta p_w(t, z)}{\sigma'_{vo}(z)} dz \quad (6)$$

335 where  $H$  is the selected depth (in this case,  $H = 10\text{m}$ ),  $\Delta p_w(t, z)$  is the pore water pressure  
336 build-up computed at time  $t$  and depth  $z$ .

337 Fig. 13(c) provides the variation of  $\Delta settl = settl_{mit}/settl_o$  as a function of the variation  
338 of  $Q_{H=10\text{m}}$  value obtained bellow the road embankment for the case without preloading.  
339 It appears that in general the settlement reduction is most important for  $Q_{H=10\text{m}} \leq 0.4$   
340 (i.e. red points in the figure). This  $Q_{H=10\text{m}}$  value means that the thickness of liquefied  
341 layer corresponds approximately to 4m, thus as aforementioned the effect of the preloading  
342 configuration influences the soil behaviour down to 3 to 4m depth (Fig. 11). According  
343 to Yasuda (2007), the large induced settlement is produced principally by the horizontal  
344 movement of the ground under the structure. Thus, it seems that the mitigation efficiency  
345 is due to the lateral soil stiffening effect provided by the preloading which decreases the  
346 settlement reducing the horizontal movement of soil foundation.

347 Finally, regarding the effect of soil improvement on the induced relative horizontal dis-  
348 placement ( $U_H$ ) and in accordance with Fig. 14, it seems that a clear trend could not be  
349 distinguished. In some cases, the induced  $U_H$  value after preloading exceeds the one obtained  
350 without preloading. A possible reason is that this horizontal displacement is related to a  
351 local shear failure of the embankment, which is not related to the actual state of the under-  
352 lying soil. However, in order to reduce the horizontal displacement of the road embankment  
353 other complementary mitigation methods could be used, e.g. the placement of gravel berms  
354 on both sides of the road embankment.

## 355 CONCLUSIONS

356 A series of finite element parametric analyses were carried out so as to assess the perfor-  
357 mance of preloading embankment as a liquefaction countermeasure method in foundation of

358 road embankments. A typical soil-road embankment model has been used to illustrate key  
359 results from parametric studies. Performance was assessed by means of overall road embank-  
360 ment behaviour, namely, settlements and horizontal displacements. The main conclusions  
361 drawn from this study are as follows.

- 362 ● The countermeasure method and the used configuration produce a densification of the  
363 soil layers placed at the toe of the road slopes of the embankment which is a function  
364 of the preloading embankment height.
- 365 ● The analyses showed that the use of preloading reduces the induced road embankment  
366 surface settlements. Because, the densified zones are able to limit lateral migration  
367 of foundation soil towards the free field.
- 368 ● The results show that the preloading has a beneficial effect concerning the settlements  
369 in the cases where earthquakes produced a depth of liquefaction less than the depth  
370 of the overconsolidated soil. Thus, the countermeasure's effect should reach a certain  
371 depth to be effective.
- 372 ● The threshold Arias intensity value needed to induce a slight damage level increases  
373 when the preloading method is used.
- 374 ● After soil improvement, the settlement of soil foundation could be neglected for lower  
375 levels of excitation.
- 376 ● The induced relative horizontal displacement is about four times less than the settle-  
377 ment and is not affected significantly by the preloading presumably because it may  
378 be related to shallow shear failure of the embankment, not passing through the layers  
379 of the underlying soil that liquefy.
- 380 ● The numerical results for both the simulated laboratory tests and the road embank-  
381 ment allow to show that the numerical code used in this study is able to simulate the  
382 preloading method. Moreover, for the design purposes, it could be used as a tool for  
383 assessing the effectiveness of this liquefaction countermeasure method.



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**TABLE 1. Classification of the ECP model parameters**

Behaviour domains	Directly measured	Not-Directly measured
Elasticity	$K_{ref}, G_{ref}, n_e, p_{ref}$	
Critical State and Plasticity	$\phi'_{pp}, \beta, p_{co}, d$	$b$
Flow Rule and	$\psi$	$a_1, a_2, \alpha_\psi,$
Isotropic hardening		$m, c$
Threshold domains		$r^{ela}, r^{hys},$ $r^{mob}, r^{ela}_{iso}$

**TABLE 2. Statistics characteristics for the selected earthquakes**

Parameter	Range	Mean	$CV[\%]$	Median	$\sigma_{\ln}$
$PHA$ [g]	0.04 – 0.37	0.14	53	0.11	0.49
$T_m$ [s]	0.37 – 1.27	0.67	29	0.62	0.28
$T_p$ [s]	0.10 – 1.07	0.41	53	0.40	0.57
$T_{V/A}$ [s]	0.26 – 1.54	0.56	39	0.53	0.34
$I_A$ [m/s]	0.03 – 3.13	0.40	133	0.20	0.96
$D_{5-95}$ [s]	7.16 – 51.47	18.11	42	17.22	0.36
$I_{rms}$ [m/s <sup>2</sup> ]	0.10 – 0.48	0.22	44	0.19	0.41
$PGV$ [cm/s]	3.19 – 63.70	15.74	74	0.12	0.62
$SI$ [m]	0.12 – 2.51	0.63	72	0.44	0.60
$SED$ [cm <sup>2</sup> /s]	16.64 – 6150	781	168	245	1.28

**TABLE 3. ECP model's parameters for all soil profile layers.**

Parameter	0-5m	5-10m	10-15m	Road emb.
$k_s$ [m/s]	$1 \cdot 10^{-4}$	$1 \cdot 10^{-4}$	$1 \cdot 10^{-4}$	-
Elasticity				
$K_{ref}$ [MPa]	628	628	628	708
$G_{ref}$ [MPa]	290	290	290	328
$n_e$	0.5	0.5	0.5	0.4
Critical State and Plasticity				
$\phi'_{pp}$ [°]	30	30	30	33
$\beta$	33	33	33	17
$d$	2.0	2.0	2.0	2.5
$b$	0.2	0.2	0.2	0.2
$p'_{co}$ [kPa]	18.9	39.8	60.8	$7.2 \cdot 10^3$
Flow Rule and Isotropic Hardening				
$\psi$ [°]	30	30	30	33
$a_1$	$1 \cdot 10^{-4}$	$1 \cdot 10^{-4}$	$1 \cdot 10^{-4}$	$2 \cdot 10^{-4}$
$a_2$	$2 \cdot 10^{-3}$	$2 \cdot 10^{-3}$	$2 \cdot 10^{-3}$	$1.8 \cdot 10^{-2}$
$c$	$2 \cdot 10^{-3}$	$2 \cdot 10^{-3}$	$2 \cdot 10^{-3}$	$1 \cdot 10^{-4}$
$m$	1.5	1.5	1.5	1.0
Threshold Domains				
$\gamma^{ela}$	$3 \cdot 10^{-2}$	$2 \cdot 10^{-2}$	$1 \cdot 10^{-2}$	$1 \cdot 10^{-2}$
$\gamma^{hys}$	$4 \cdot 10^{-2}$	$4 \cdot 10^{-2}$	$4 \cdot 10^{-2}$	$5 \cdot 10^{-2}$
$\gamma^{mob}$	$8 \cdot 10^{-1}$	$8 \cdot 10^{-1}$	$8 \cdot 10^{-1}$	$9 \cdot 10^{-1}$

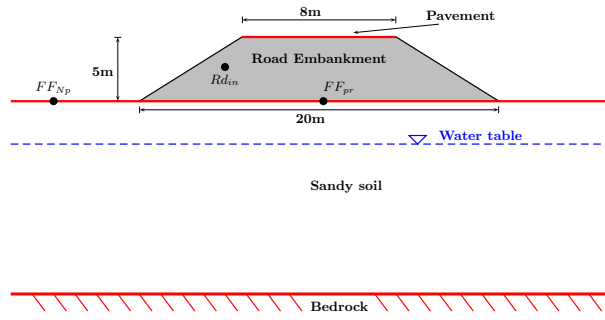
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For all cases  $p_{ref} = 1.0\text{MPa}$  and  $\alpha_\psi = 1$ .

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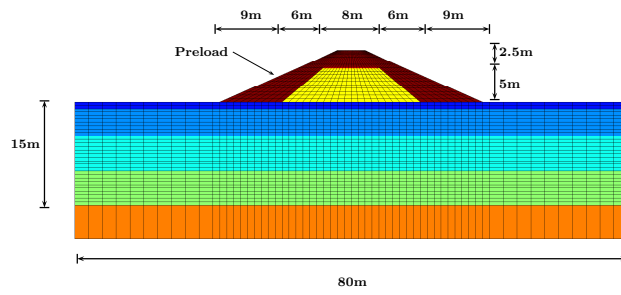
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(a)

(a)



(b)

(b)

**FIG. 1. a) Schema of the numerical model and b) Finite element mesh of the road embankment and the remediation method.**

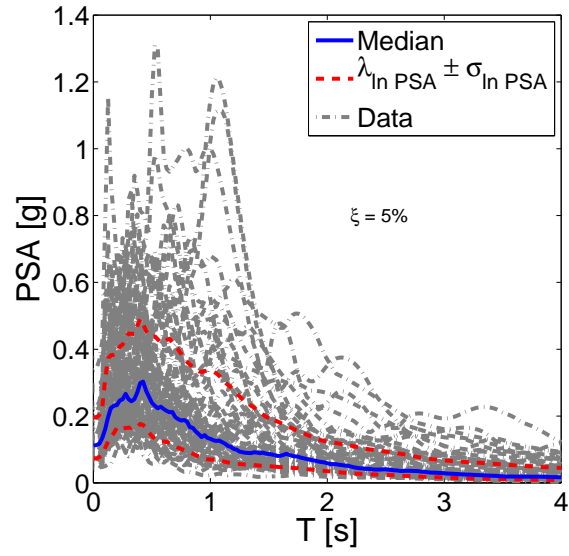


FIG. 2. Response spectra of input earthquake motions (5% damped).

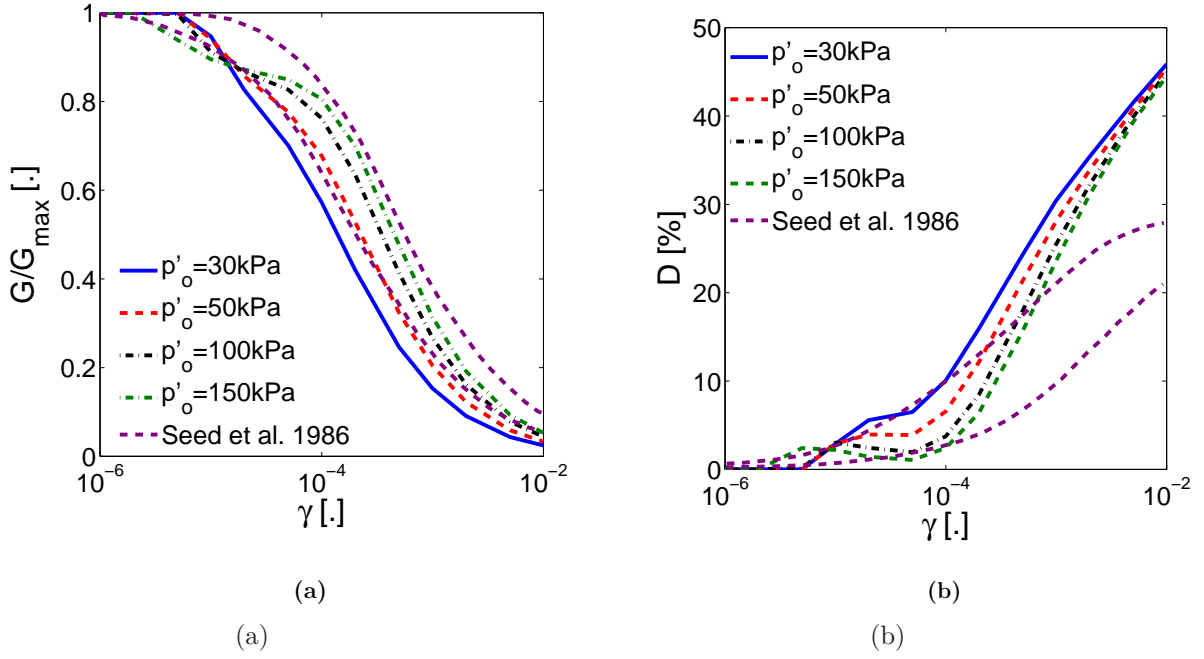


FIG. 3. Simulated a)  $G/G_{max} - \gamma$  and b)  $D - \gamma$  curves for the sand model.



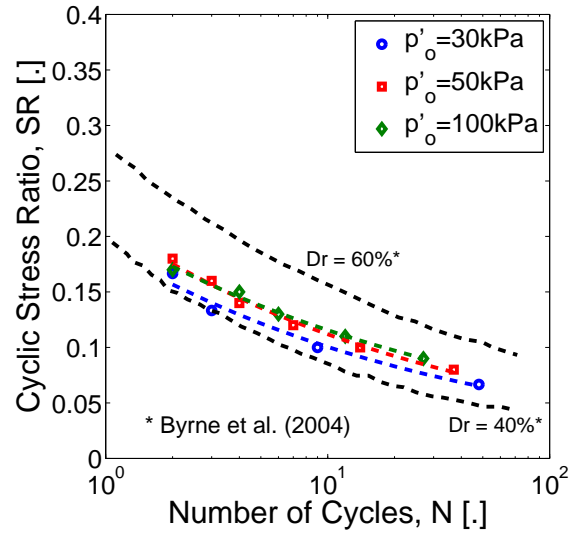
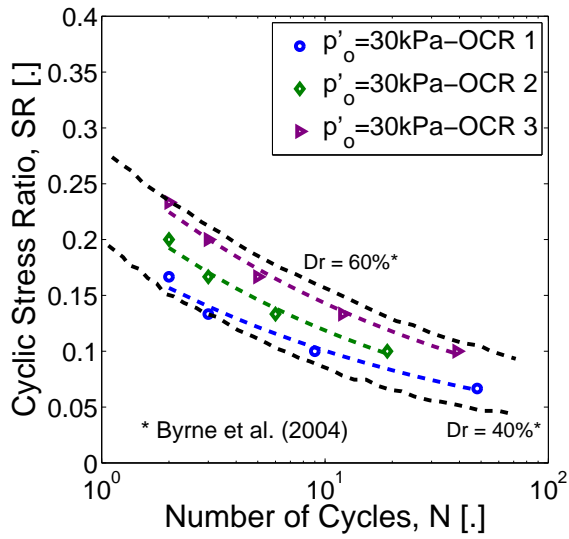
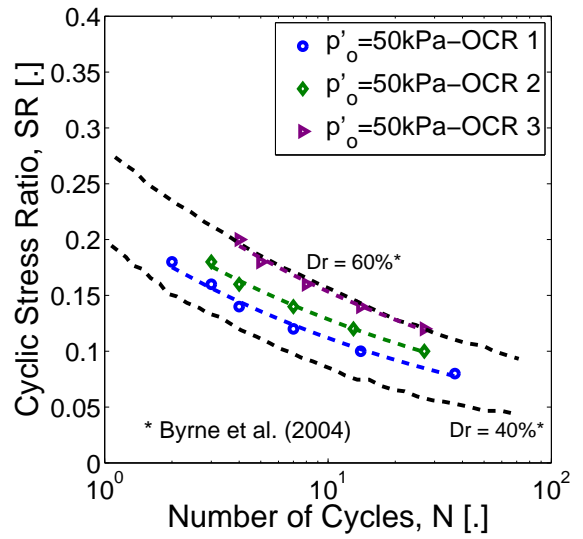


FIG. 4. Simulated liquefaction curves for the sand model.



(a)



(b)

**FIG. 5. Simulated liquefaction curves for the sand model, effect of overconsolidation ratio ( $OCR$ ) a)  $p'_o = 30\text{kPa}$  and b)  $p'_o = 50\text{kPa}$ .**

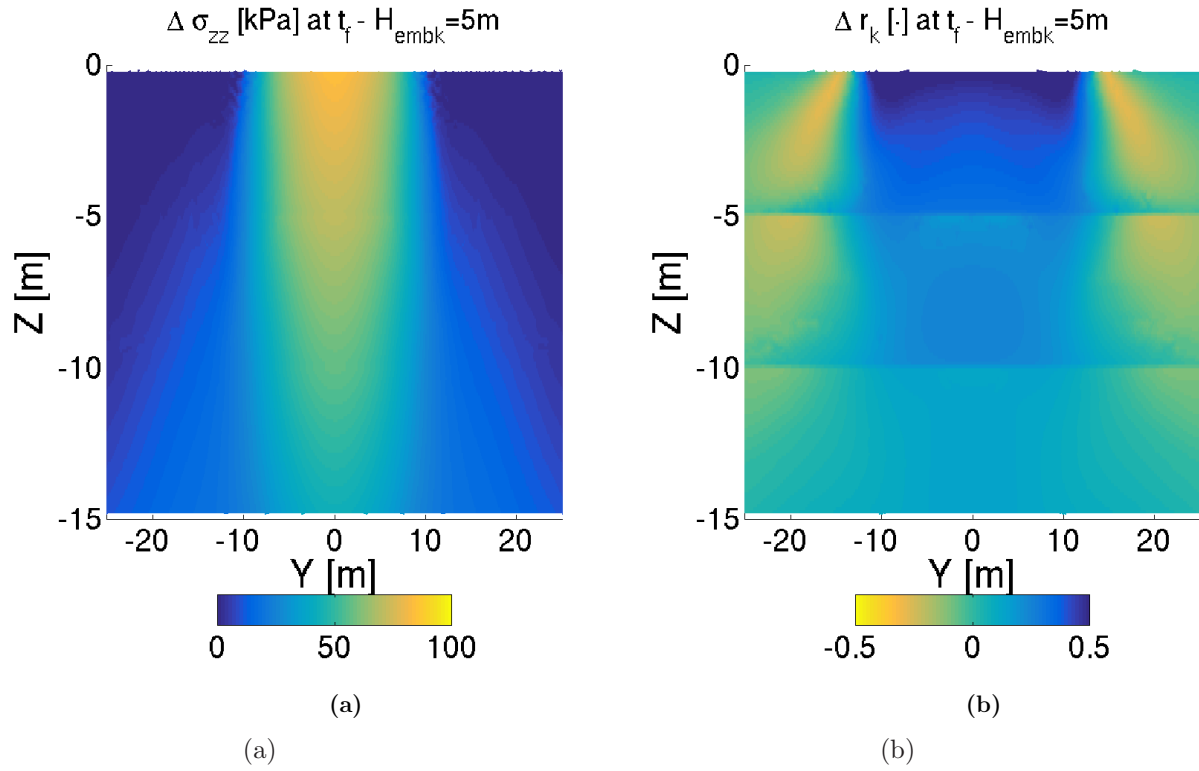
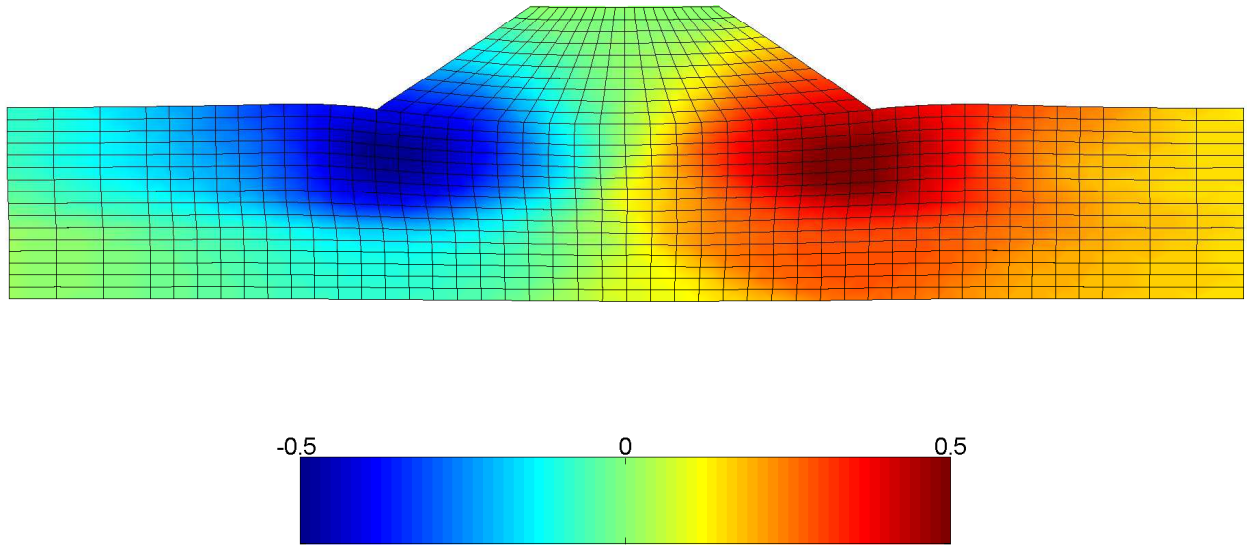
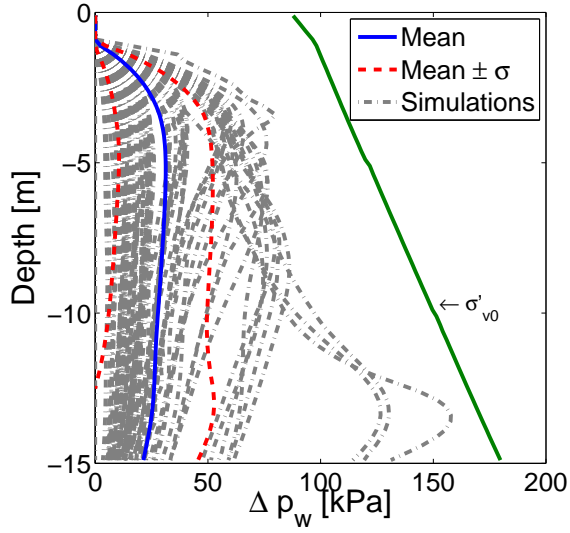


FIG. 6. Distribution of the a) induced  $\Delta \sigma_{zz}$  and b) obtained  $\Delta r_k$  parameter (Eq. 3) in the soil foundation after road embankment construction ( $H_{embk} = 5m$ ). Case before preloading.

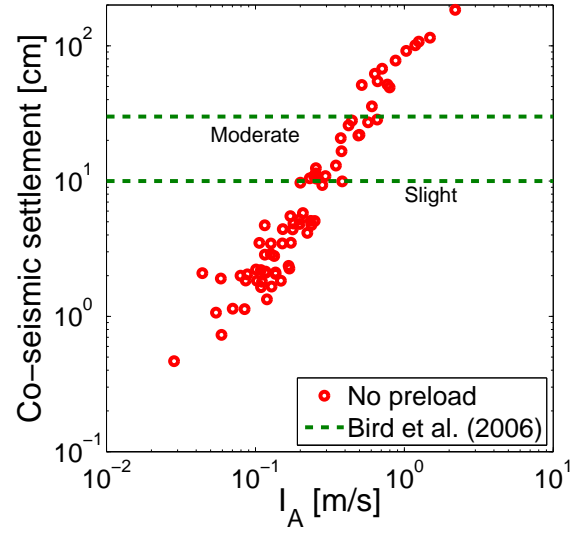


**FIG. 7. Typical distribution of induced horizontal displacements at the end of shaking (units in metres). Case before preloading.**



(a)

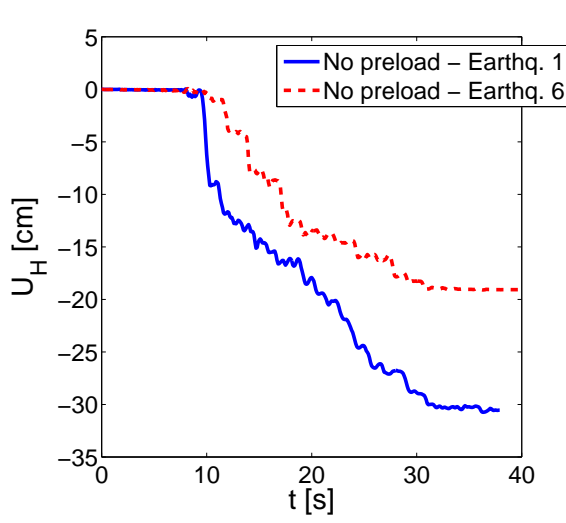
(a)



(b)

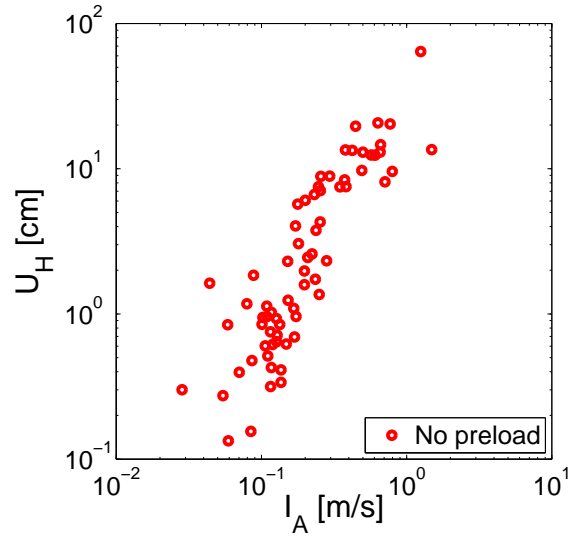
(b)

**FIG. 8. a) Obtained excess pore pressure under the road embankment centreline and b) Scatter plot of obtained co-seismic relative settlement values as a function of  $I_A$ . Case before preloading.**



(a)

(a)



(b)

(b)

**FIG. 9.** a) Relative horizontal displacement time histories of the point  $Rd_{in}$  for two typical earthquakes and b) Scatter plot of obtained co-seismic relative horizontal displacement values as a function of  $I_{Arias}$ . Case before preloading.

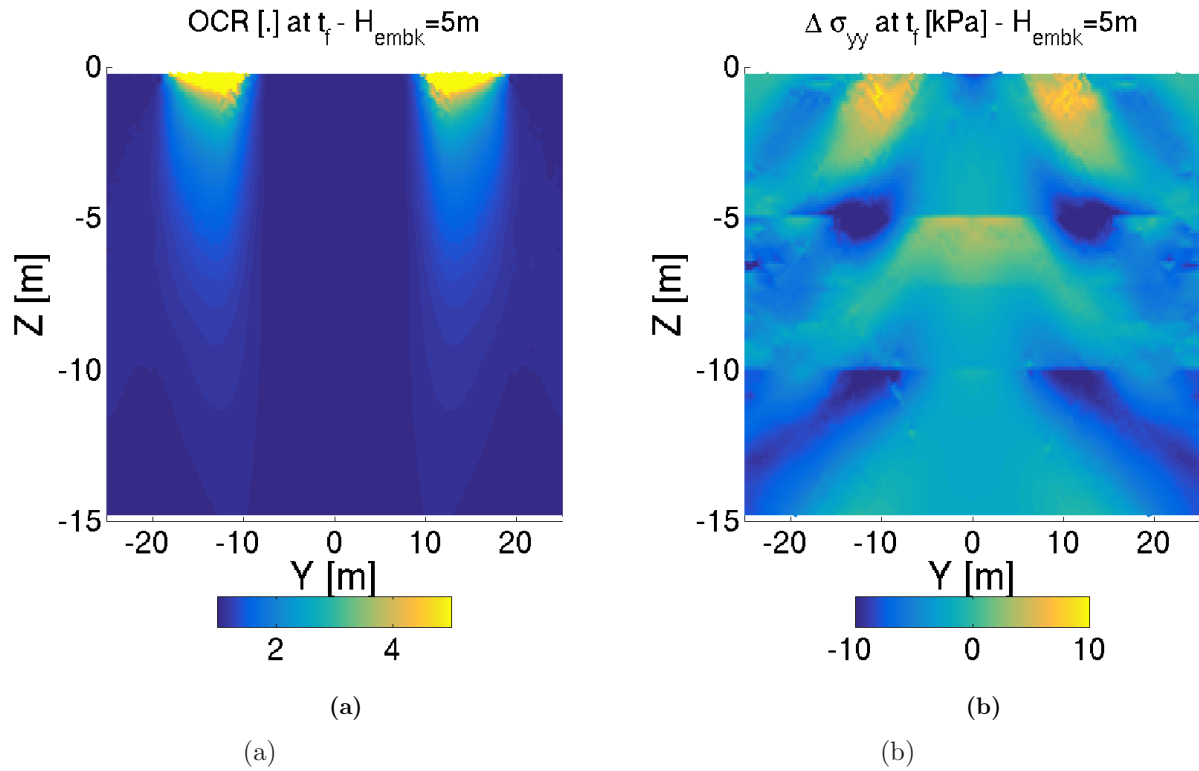
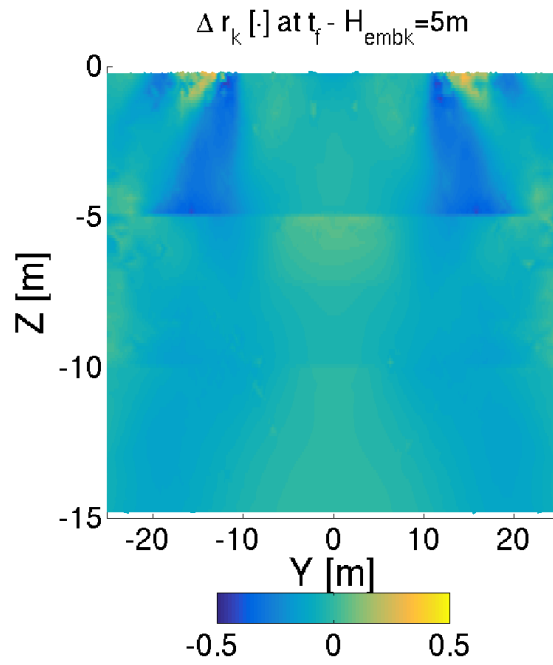
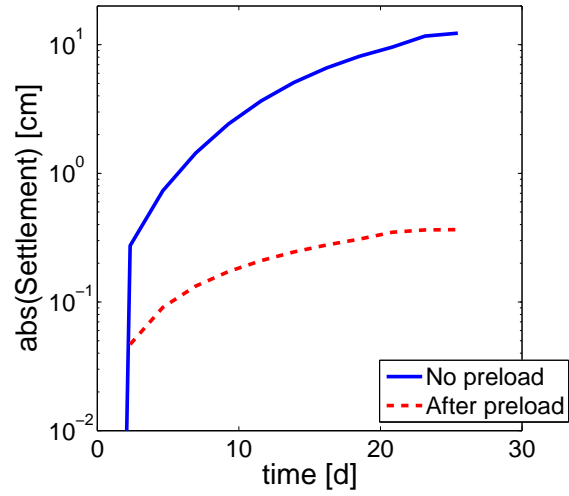


FIG. 10. a) Overconsolidation ratio ( $OCR$ ) in the soil induced by the preload and b) Change in horizontal stress  $\Delta\sigma_{yy}$  in the soil induced by the preload after road embankment construction.

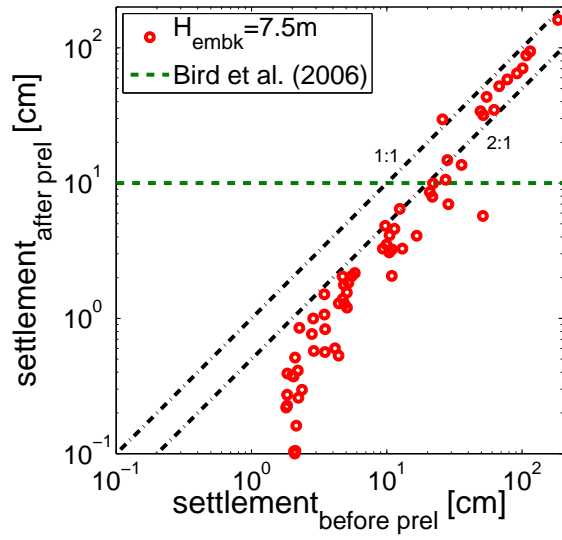


**FIG. 11. Change in the  $r_k$  parameter (Eq. 3) in the soil foundation induced by the preloading ( $H_{embk}=5m$ ).**



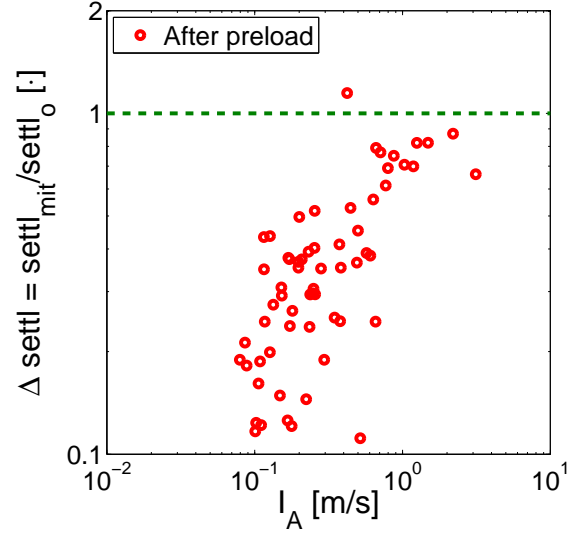


**FIG. 12.** Effect of preload on the settlement obtained during the construction of the road embankment.



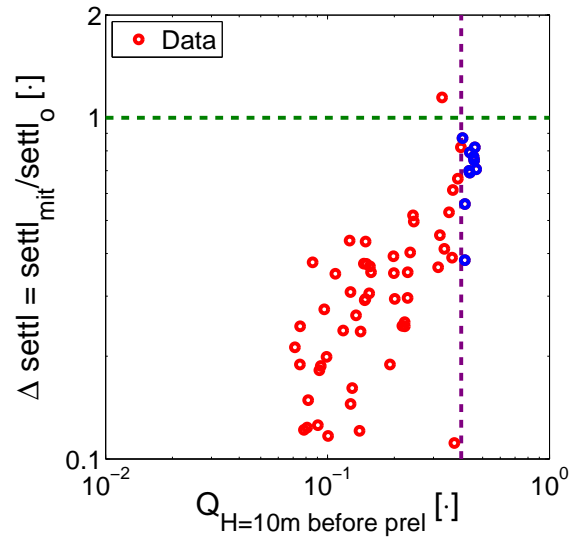
(a)

(a)



(b)

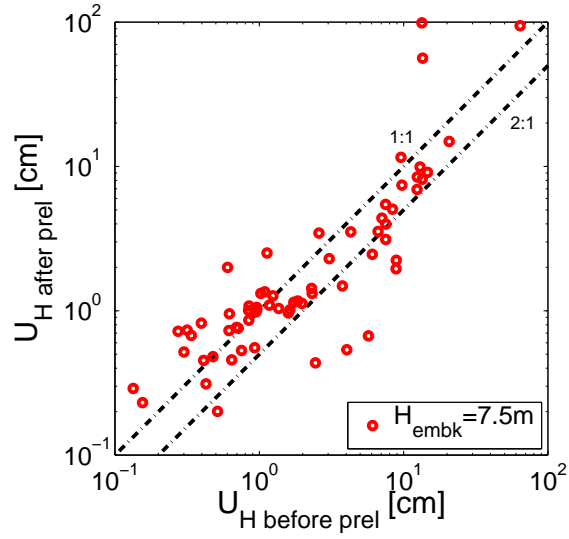
(b)



(c)

(c)

FIG. 13. Effect of preload embankment on the obtained a) co-seismic relative settlement values; b) Scatter plot of obtained  $\Delta settl$  values as a function of  $I_A$  and c) Scatter plot of obtained  $\Delta settl$  values as a function of Liquefaction Index ( $Q_H$ ).



**FIG. 14.** Effect of preload embankment on the obtained co-seismic relative horizontal displacement values.