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► **To cite this version:**

Fernando Lopez-Caballero, Arézou Modaressi-Farahmand-Razavi, Constantine A. Stamatopoulos. Numerical Evaluation of Earthquake Settlements of Road Embankments and Its Mitigation by Preloading. *International Journal of Geomechanics*, 2016, 16 (5), pp.C4015006. 10.1061/(ASCE)GM.1943-5622.0000593 . hal-01186317

HAL Id: hal-01186317

<https://hal.science/hal-01186317v1>

Submitted on 12 Mar 2020

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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, ϕ'_{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 (ϕ'_{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 σ'_{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/m³. 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 (ρ_{road}) is equal to 1800kg/m³.
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, (ν_{asp}), of 0.35 and a density, (ρ_{asp}), equal to 1800kg/m³. 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 σ_{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 σ'_{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 Δr_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 (Δp_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 Δr_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.

384 **ACKNOWLEDGEMENTS**

385 This work was funded by the Seventh Framework Programme of the European Commu-
386 nity, European Commission Research Executive Agency under Grant agreement FP7-SME-
387 2010-1-262161-PREMISERI. The research reported in this paper has been supported in part
388 by the SEISM Paris Saclay Research Institute.

APPENDIX I. REFERENCES

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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	ψ	$a_1, a_2, \alpha_\psi,$
Isotropic hardening		m, c
Threshold domains		$r^{ela}, r^{hys},$ r^{mob}, r^{iso}

TABLE 2. Statistics characteristics for the selected earthquakes

Parameter	Range	Mean	$CV[\%]$	Median	σ_{\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 ²]	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 ² /s]	16.64 – 6150	781	168	245	1.28

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

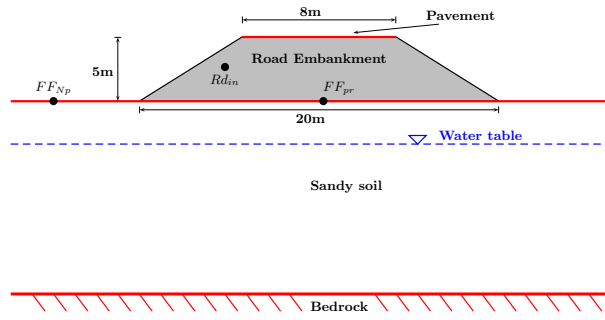
Parameter	0-5m	5-10m	10-15m	Road emb.
k_s [m/s]	$1 \cdot 10^{-4}$	$1 \cdot 10^{-4}$	$1 \cdot 10^{-4}$	-
Elasticity				
K_{ref} [MPa]	628	628	628	708
G_{ref} [MPa]	290	290	290	328
n_e	0.5	0.5	0.5	0.4
Critical State and Plasticity				
ϕ'_{pp} [°]	30	30	30	33
β	33	33	33	17
d	2.0	2.0	2.0	2.5
b	0.2	0.2	0.2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

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

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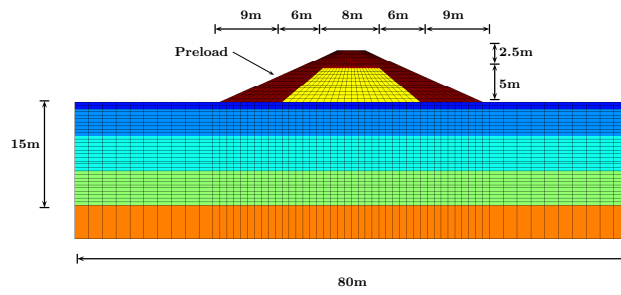
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578		displacement values.	40



(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.

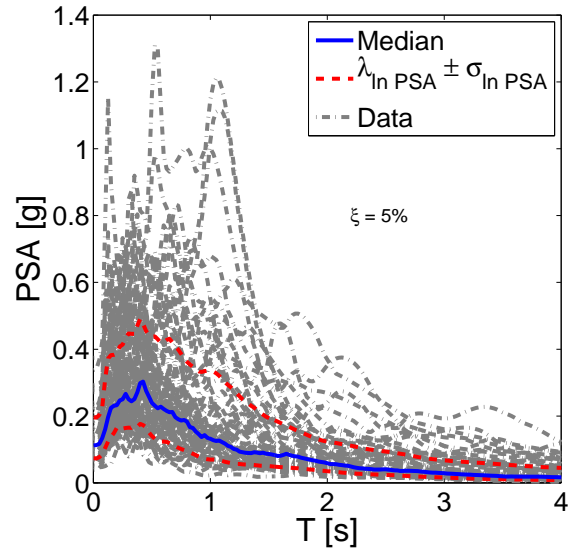


FIG. 2. Response spectra of input earthquake motions (5% damped).

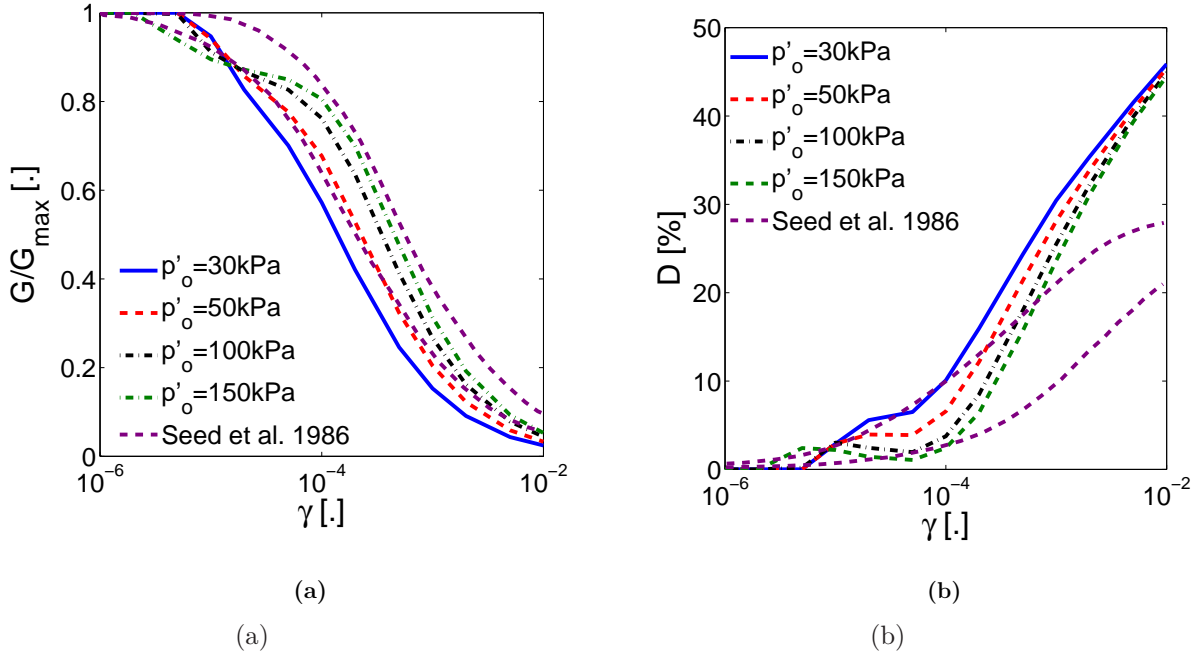


FIG. 3. Simulated a) $G/G_{max} - \gamma$ and b) $D - \gamma$ curves for the sand model.

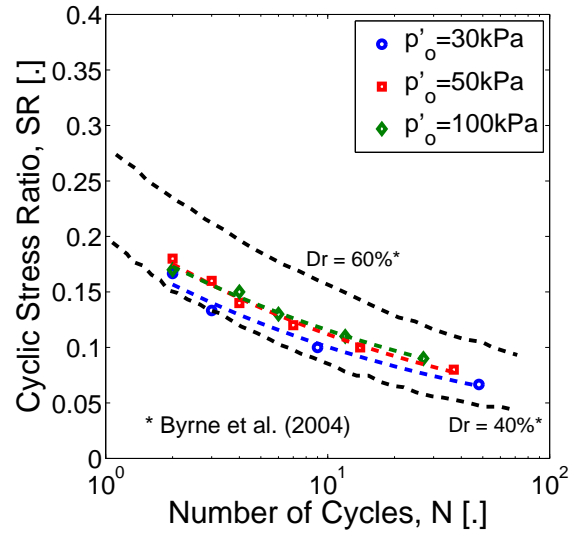
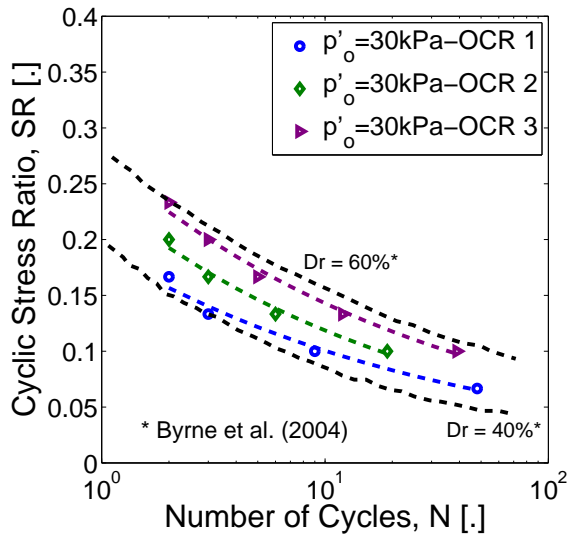
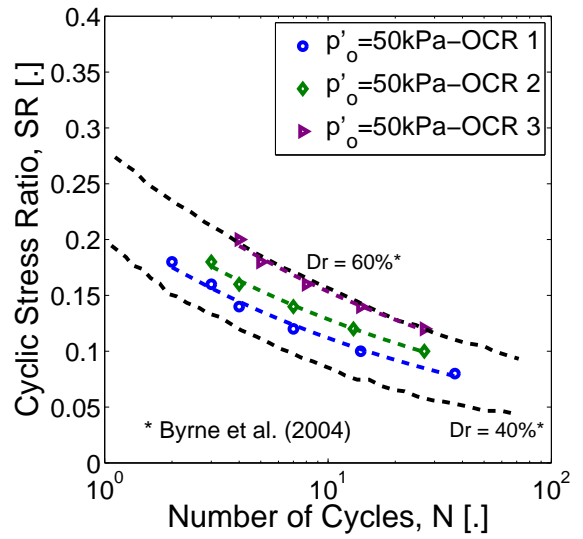


FIG. 4. Simulated liquefaction curves for the sand model.



(a)



(b)

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

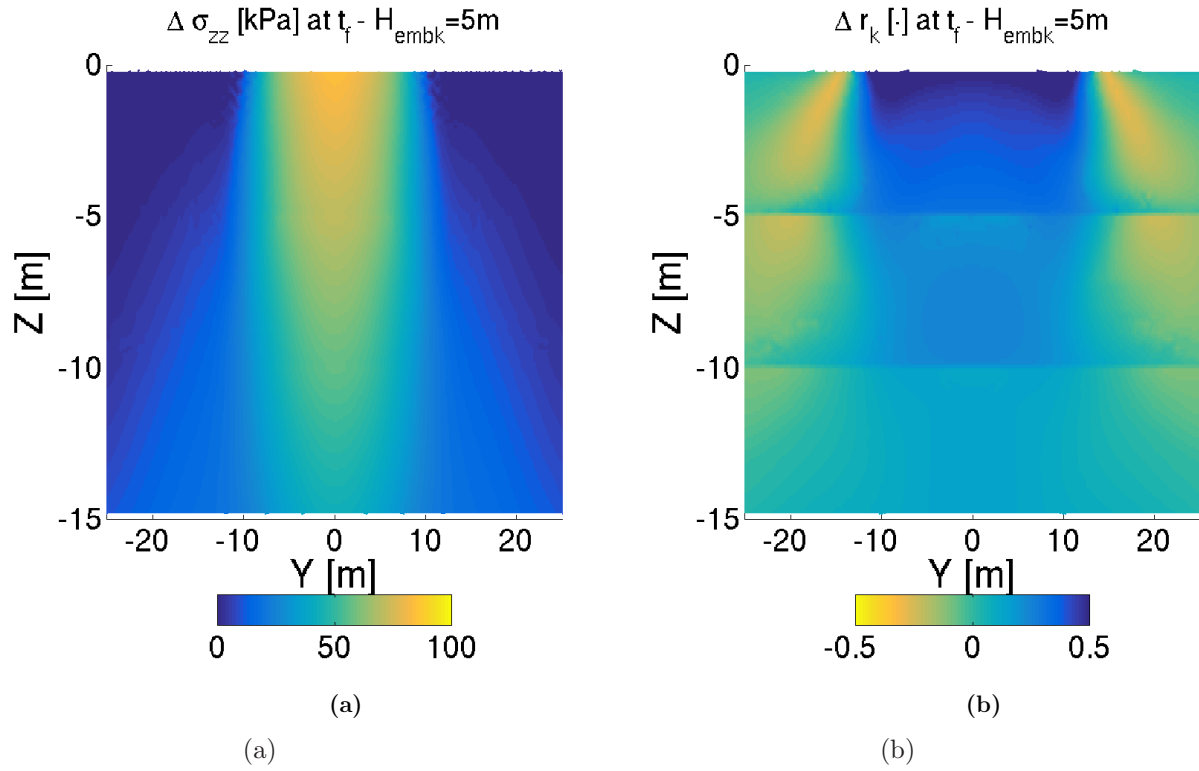


FIG. 6. Distribution of the a) induced $\Delta \sigma_{zz}$ and b) obtained Δr_k parameter (Eq. 3) in the soil foundation after road embankment construction ($H_{embk} = 5m$). Case before preloading.

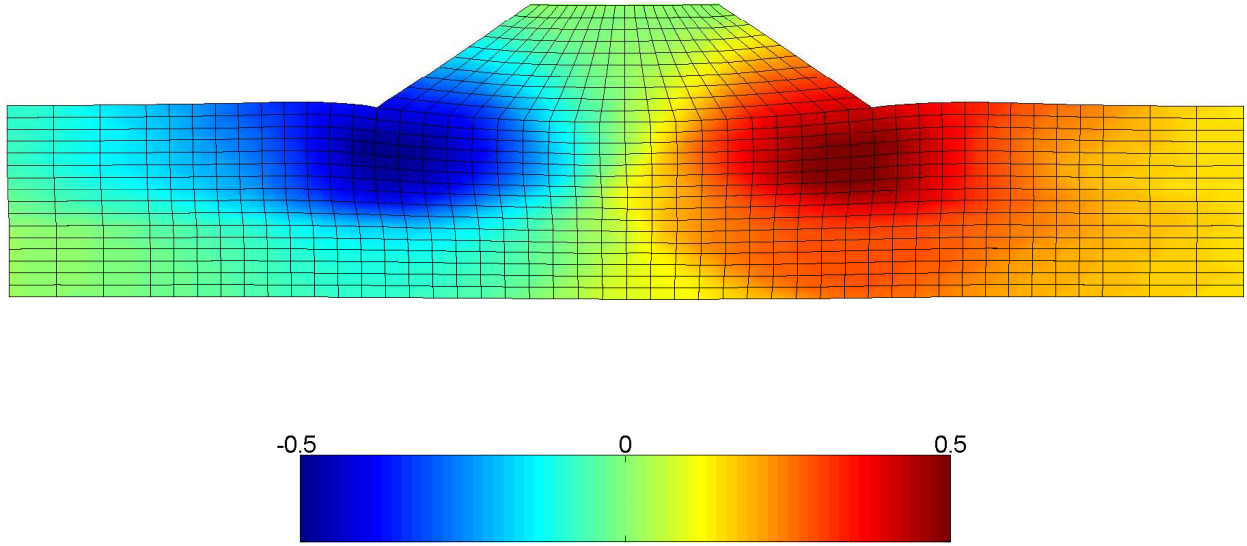
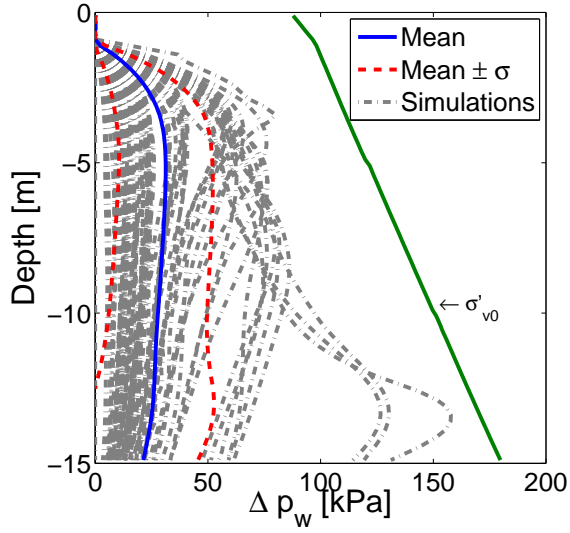
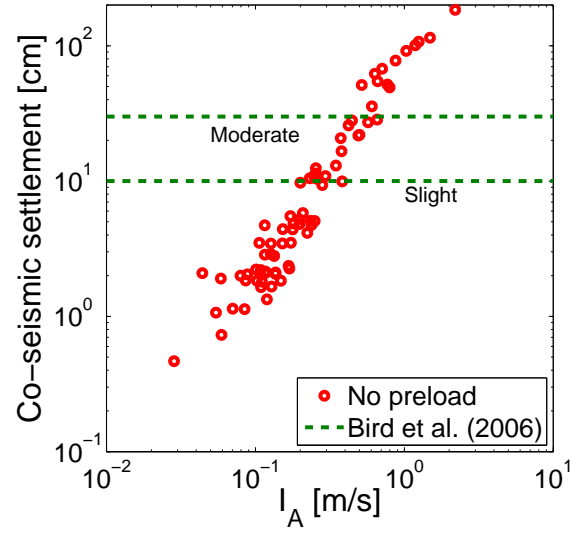


FIG. 7. Typical distribution of induced horizontal displacements at the end of shaking (units in metres). Case before preloading.



(a)

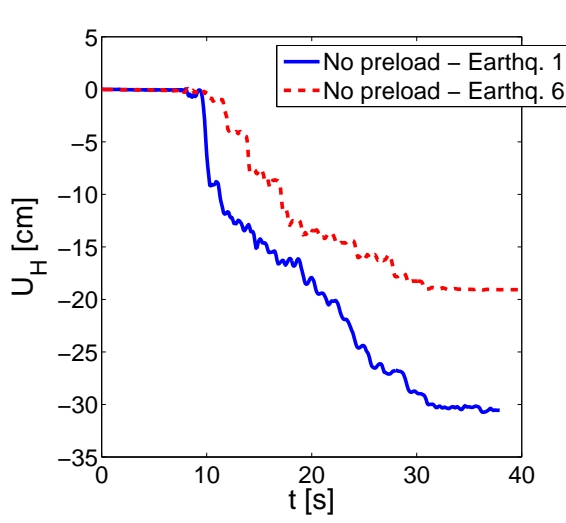
(a)



(b)

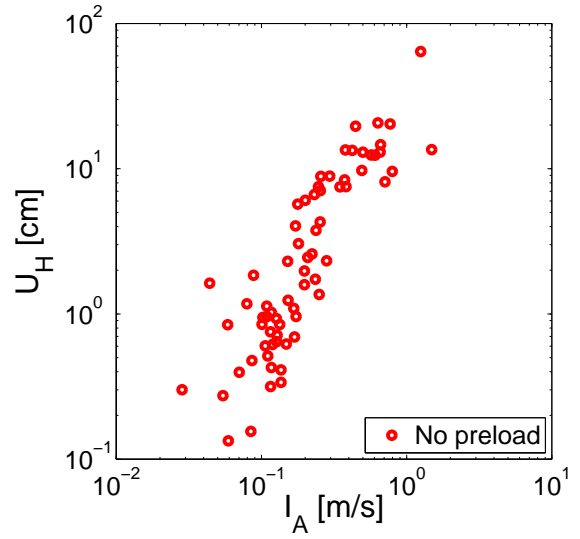
(b)

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.

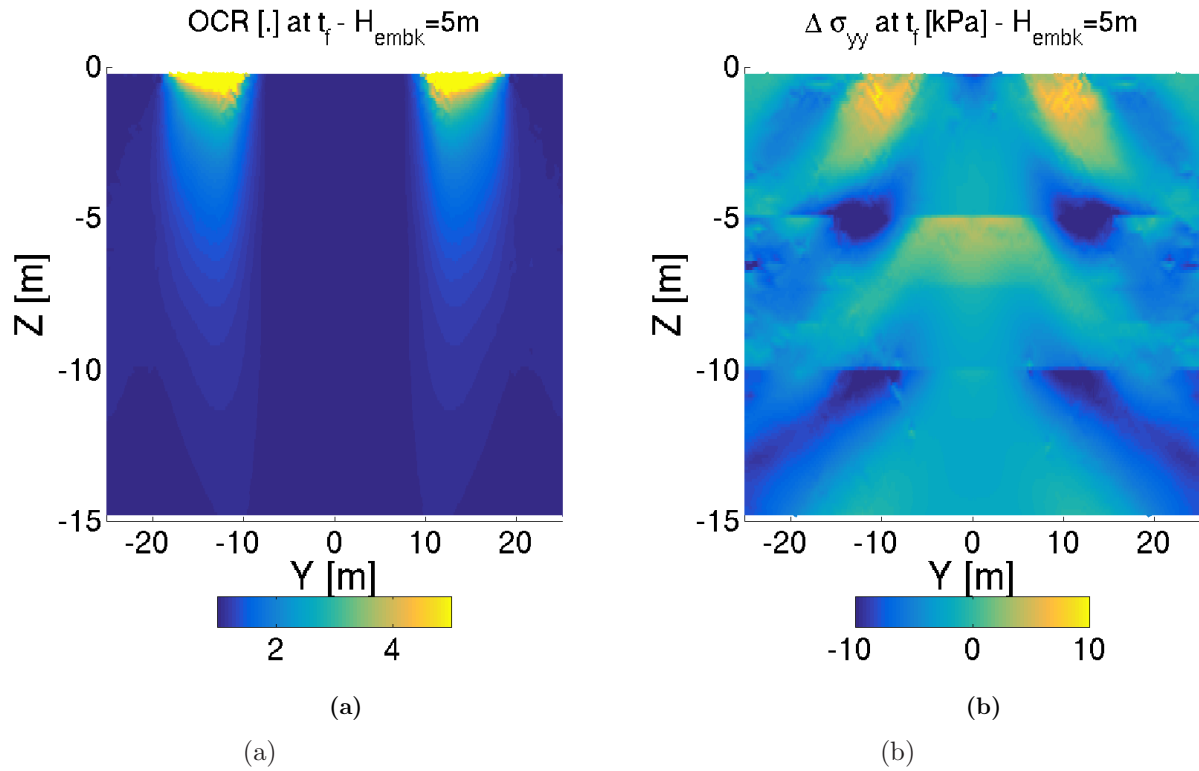


FIG. 10. a) Overconsolidation ratio (OCR) in the soil induced by the preload and b) Change in horizontal stress $\Delta\sigma_{yy}$ in the soil induced by the preload after road embankment construction.

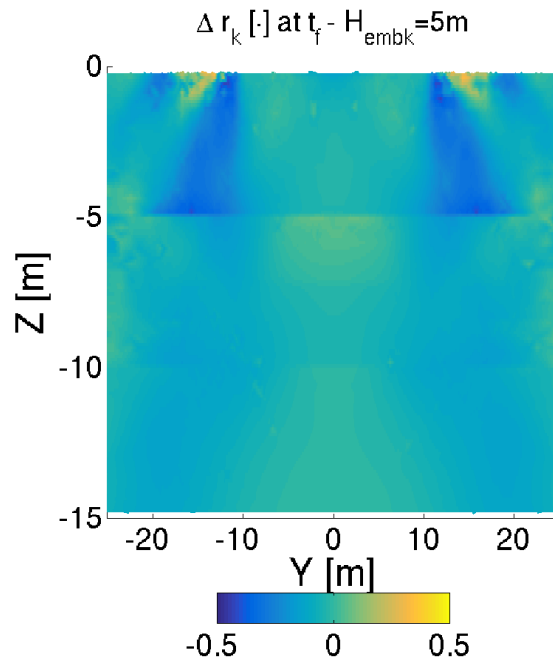


FIG. 11. Change in the r_k parameter (Eq. 3) in the soil foundation induced by the preloading ($H_{embk}=5m$).

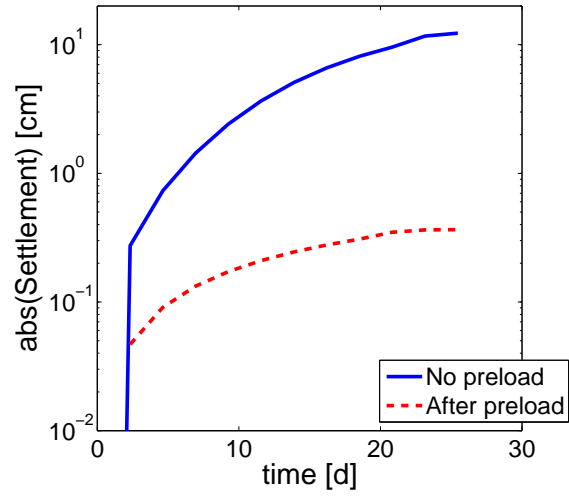
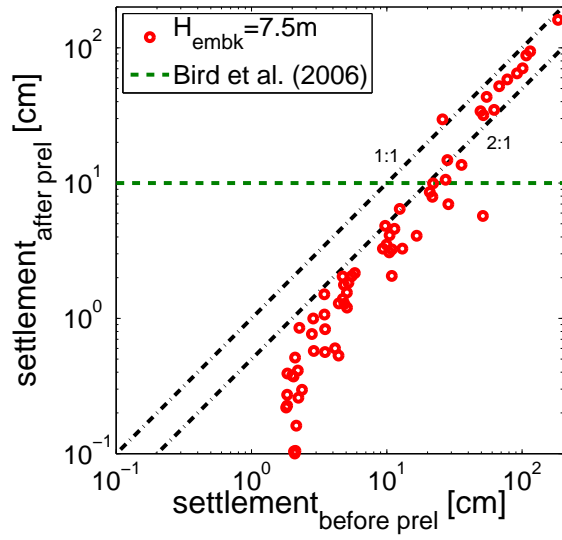
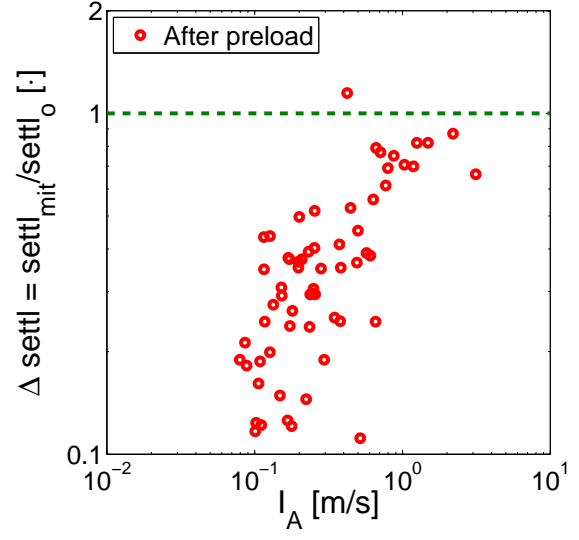


FIG. 12. Effect of preload on the settlement obtained during the construction of the road embankment.



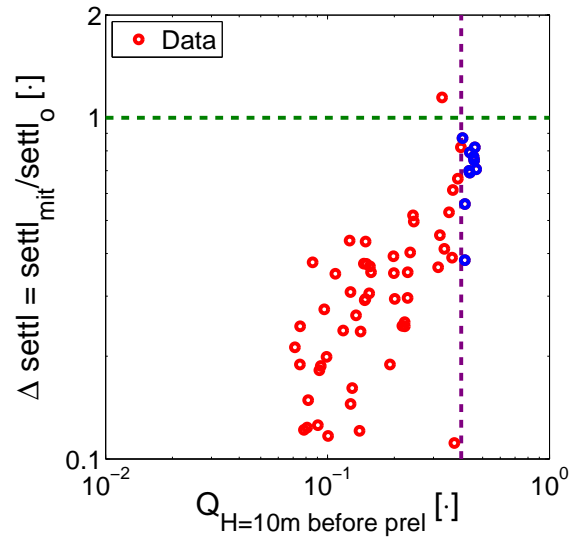
(a)

(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).

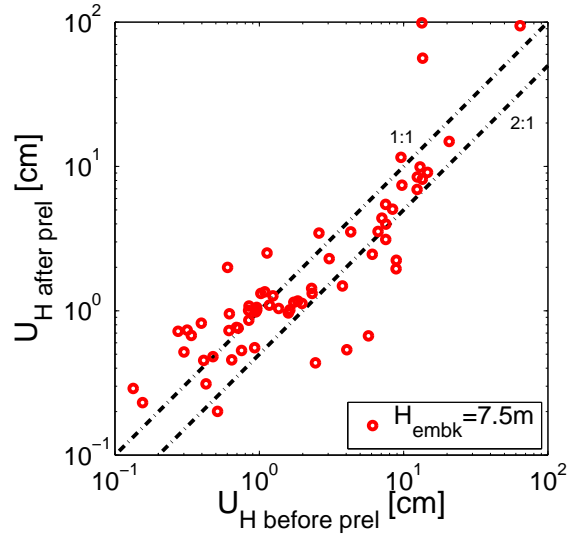


FIG. 14. Effect of preload embankment on the obtained co-seismic relative horizontal displacement values.