



HAL
open science

Effect of coupling excess pore pressure and soil deformation on nonlinear SSI in liquefiable soil deposits

Silvana Montoya Noguera, Fernando Lopez-Caballero

► **To cite this version:**

Silvana Montoya Noguera, Fernando Lopez-Caballero. Effect of coupling excess pore pressure and soil deformation on nonlinear SSI in liquefiable soil deposits. *Bulletin of Earthquake Engineering*, 2018, 16 (2), pp.681-705. 10.1007/s10518-017-0218-3 . hal-01576000

HAL Id: hal-01576000

<https://hal.science/hal-01576000v1>

Submitted on 12 Mar 2020

HAL is a multi-disciplinary open access archive for the deposit and dissemination of scientific research documents, whether they are published or not. The documents may come from teaching and research institutions in France or abroad, or from public or private research centers.

L'archive ouverte pluridisciplinaire **HAL**, est destinée au dépôt et à la diffusion de documents scientifiques de niveau recherche, publiés ou non, émanant des établissements d'enseignement et de recherche français ou étrangers, des laboratoires publics ou privés.

1
2 Effect of coupling excess pore pressure and soil deformation on nonlinear SSI in liquefiable
3 soil deposits

4
5 S. Montoya-Noguera^{1,2} and F. Lopez-Caballero¹

6 ¹Laboratory MSSMat CNRS UMR 8579, CentraleSupélec, Paris-Saclay University,
7 Châtenay-Malabry 92290, France

8 ²Present institution: Department of Architecture and Civil Engineering, City University of
9 Hong Kong, Tat Chee Avenue, Kowloon, Hong Kong

10
11
12 Abstract

13 The current seismic design philosophy is based on nonlinear behavior of structures where the
14 foundation soil is often simplified by a modification of the input acceleration depending on
15 the expected site effects. The latter are generally limited to depend on the shear-wave
16 velocity profile or a classification of the site. Findings presented in this work illustrate the
17 importance of accounting for both soil nonlinearity due to seismic liquefaction and for soil-
18 structure interaction when dealing with liquefiable soil deposits.

19 This paper concerns the assessment of the effect of excess pore pressure (Δp_w) and
20 deformation for the nonlinear response of liquefiable soils on the structure's performance.
21 For this purpose a coupled Δp_w and soil deformation (CPD) analysis is used to represent the
22 soil behavior. A mechanical-equivalent fully drained decoupled (DPD) analysis is also
23 performed. The differences between the analyses on different engineering demand parameters
24 are evaluated. The results allow to identify and to quantify the differences between the
25 analyses. Thus, it is possible to establish the situations for which the fully drained analysis
26 might tend to overestimate or underestimate the structure's demand.

27
28 Keywords: Seismic liquefaction ; Effective stress analysis ; Soil-structure interaction ;
29 Nonlinearity

30 31 1 INTRODUCTION

32 Sufficient evidence has been identified to proof the importance of site effects on the surface
33 ground motion. Still, in the earthquake engineering practice, the effects of nonlinear soil
34 behavior on the dynamic soil-structure interaction (SSI) are either neglected or simplified.
35 Moreover, when these effects are taken into account, in most cases the nonlinear soil
36 response is modeled with fully drained analysis that do not account for the excess pore
37 pressure (Δp_w) generation which controls the behavior of liquefiable soils. In contrast,
38 coupled analysis allows the modeling of the generation, redistribution and eventual
39 dissipation of Δp_w during and after earthquake shaking. While small or no volume change
40 associated with shear strain is presented, use of fully drained analysis may be justified. For
41 typical liquefiable soils at relatively shallow depths, a threshold shear strain below which no
42 Δp_w will develop has been identified at about 10^{-4} (Drenevich and Richart 1970; Dobry and
43 Swiger 1979; Dobry et al. 1982). Above this threshold effective mean stress changes which
44 will affect the material properties. In such situations, coupled effective stress analysis seem
45 more appropriate (Yoshida and Iai 1998).

46
47 Recent studies, performed by Hartvigsen (2007), Yoshida (2013) and Montoya-Noguera and
48 Lopez-Caballero (2016) have evaluated the effects of coupling Δp_w and deformation on the
49 soil behavior. In these studies, focus was given mainly to the effects on the ground motion
50 amplitude and frequency content. A summary of this research is briefly presented in the

51 following section. Concerning peak ground acceleration (PGA), all studies agree that
52 accounting for Δp_w will in general incur in lower values; thus without it the results will be
53 conservative. However, regarding the response spectra of acceleration (PSA), higher values
54 could be presented for long periods; which will be prejudicial for flexible buildings. All of
55 these analyses have been performed in free-field 1D conditions and these effects have not
56 been analyzed on the performance of structures founded on cohesionless soils.

57
58 The aim of this work is to assess the effects of Δp_w generation on the performance of a
59 structure founded on liquefiable soil. For this purpose a nonlinear 2D soil-structure
60 interaction (SSI) model is analyzed under a wide range of unscaled earthquake signals. Two
61 structures with different predominant periods are used in order to account for different cases
62 of SSI. In the interest of quantifying the effects, two mechanically-equivalent models -one
63 with coupled Δp_w and soil deformation (CPD) and one fully drained (DPD) - were performed
64 with the same effective-stress analysis. Three aspects were evaluated on the structure: i) the
65 response spectra of acceleration , ii) the relative settlement and iii) the inter-story drift.

66 67 1.1 Summary of previous findings

68 In this work, the comparison is made between analyses with and without Δp_w in the interest
69 of evaluating solely the effect of coupling Δp_w and deformation. To address this subject, only
70 a limited number of studies have been published. Each study used a different technique but in
71 all, the fundamental variation is in the presence or lack of Δp_w . For clarity the two analyses
72 will be referred to as CPD and DPD.

73
74 Hartvigsen (2007) performed a series of CPD and DPD analyses of 1D wave propagation in
75 different soil columns and subjected to a variety of earthquake motions. The main
76 conclusions of the parametric study were also published by Kramer et al. (2011). In total,
77 nine 20-m thick soil profiles with varying depths of liquefiable soil were subjected to 139
78 input motions. The model used is written in effective stresses but for the DPD analysis, the
79 bulk modulus of the pore fluid was reduced to a very low value. Thus, while the weight of the
80 water contributes to the dynamic soil response (i.e. the inertial effect), there is no generation
81 of Δp_w . As a result, they presented a correction function for the surface PSA obtained by the
82 ratio of CPD and DPD analyses. This correction, called the response spectra ratio ($RSR =$
83 PSA^{CPD} / PSA^{DPD}), is a function of period and of the loading parameter (L) that is the inverse
84 of the factor of safety (FS) defined by the cyclic stress approach (Seed and Idriss 1971). RSR
85 can then be applied to the PSA values obtained with a total stress analysis to obtain a more
86 accurate response spectrum for liquefiable soils. Among the main features of this factor is
87 that even for very high FS, an amplification is evidenced -i.e. $RSR > 1$ - above a certain
88 period that increases as FS decreases. Interestingly, as FS decreases a peak of amplification
89 and of deamplification evolves. The combination of the mid-period deamplification trough
90 and long period amplification evidences the shift to low frequencies that occurs as a result of
91 soft-site amplification (Beresnev and Wen 1996). For the lowest liquefaction resistance
92 shown (FS=0.5), maximum RSR is approximately 1.3 at $T \approx 10s$ and the minimum is 0.3 at
93 $T \approx 0.5s$. A regression analysis was used to obtain RSR but the correlation between the
94 residuals was low, hence the applicability of the function is unlikely. Additionally, the use of
95 a loading parameter reduces the analysis to a simplified procedure where the input motion
96 variability is only expressed by PGA and the magnitude scaling factor while the soil response
97 is only described by the SPT blow count and the reduction factor.

98
99 Similarly, Yoshida (2013) compared the CPD and DPD analyses. A total of 268 sites were
100 subjected to 11 earthquake motions. Again the only difference between the analyses is the

101 consideration of excess pore water pressure generation or not; however, no information on
102 how this was performed was stated. The comparison was focused on five frequently used
103 intensity measures of the surface time history: the peak acceleration (PGA), peak velocity
104 (PGV), peak displacement (PGD), instrumental seismic intensity (I_{JMA}) and spectral intensity
105 (SI). The PGA results from the CPD analysis were consistently smaller than the ones
106 obtained by the DPD one. Whereas for PGD, the CPD results were greater, which could
107 affect the design of underground structures such as piles and pipelines. The differences
108 between the analyses were smaller for the other intensity measures. Again, the program used
109 for these analyses only took into account the SPT blow count to describe the soil behavior. In
110 contrast with Hartvigsen results, no clear trend between the two analyses was identified,
111 possibly due to the great variability of sites tested, some of those with no liquefiable soil.

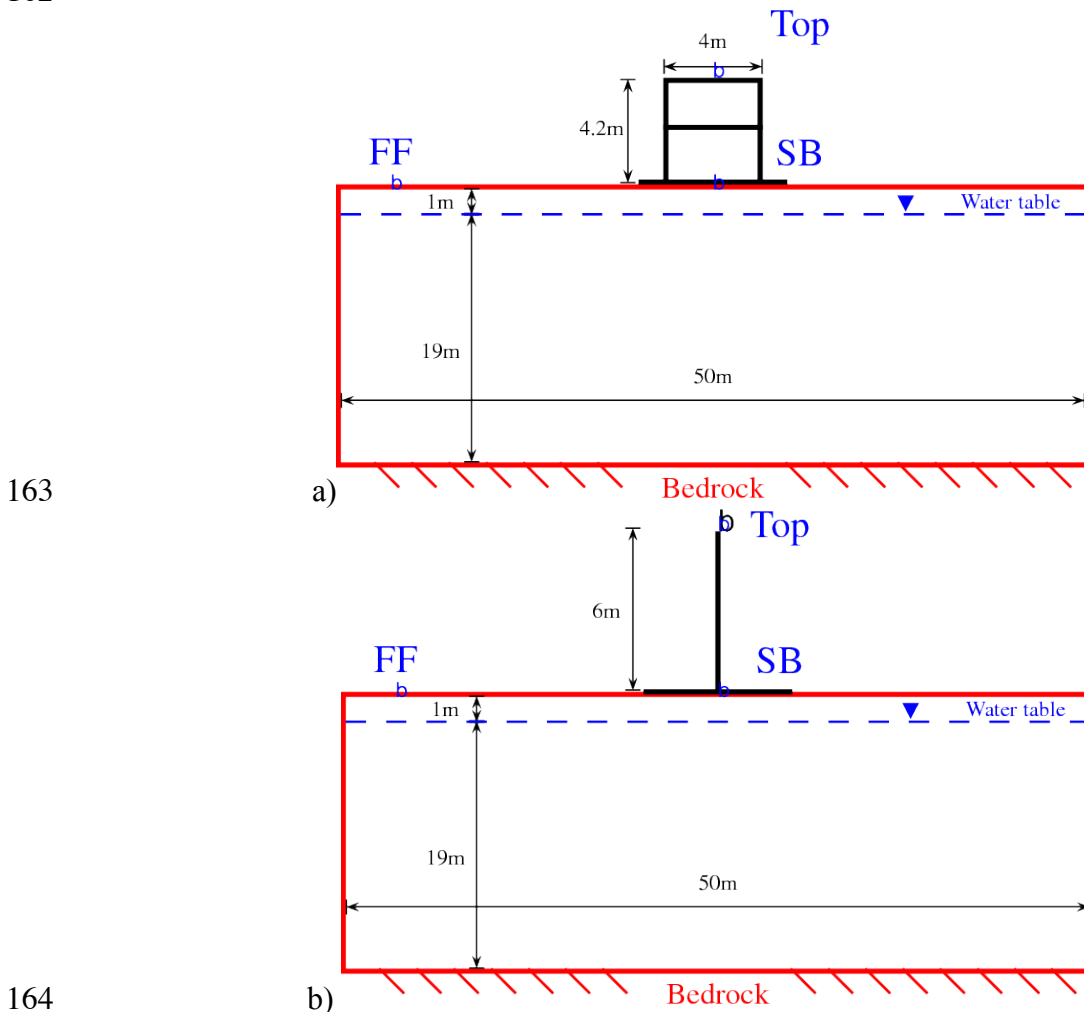
112
113 Gingery et al. (2014) followed a similar procedure as that of Hartvigsen (2007) but applied to
114 ground motion prediction equations (GMPEs) that do not account for liquefaction. The four
115 GMPEs used were the updated attenuation relationships for the western U.S. from the Next
116 Generation Attenuation (NGA) project (Chiou et al. 2008) that accounted for soil conditions.
117 An amplification function (AF) for each GMPE was calculated from the logarithmic ratio of
118 PSA of recorded motions in 19 liquefiable soil deposits and the evaluated NGA GMPE, i.e.
119 $\ln(AF) = \ln(PSA^{rec}/PSA^{NGA})$. The correction functions presented in general a similar trend
120 as the RSR proposed by Hartvigsen (2007) but with higher maximums and minimums;
121 although these factors are hard to compare, as they are applied to different analyses (i.e. CPD
122 vs DPD and measured vs. GMPEs). However, an interesting aspect of these results is that
123 added to the known long period amplification, a second amplification was observed for short
124 periods (around $T \leq 0.05$ seconds) which could be the result of acceleration spikes that occur
125 in association with the dilatational part of liquefaction phase transformation behavior.

126
127 Finally, the work presented by Montoya-Noguera and Lopez-Caballero (2016) also analyzed
128 the effect of coupling Δp_w and soil deformation on 1D wave propagation. Besides evaluating
129 PSA and other intensity measures at the surface; a detailed analyses of time evolution and
130 maximum profiles was also presented. Concerning PSA, the CPD results present an
131 amplification in long periods and also at periods around 0.05s, as evidenced by Hartvigsen
132 (2007). The differences between CPD and DPD response spectra increase with the amount of
133 liquefied soil in the deposit. One interesting finding was that although for most motions
134 tested PGA is greater for the DPD analysis, some cases present lower values, even down to
135 40% smaller. In contrast, for the Arias intensity of surface acceleration and the co-seismic
136 settlement, DPD values were up to six times greater than CPD ones. Regarding the soil
137 profile, clear relations were found between acceleration, shear strain and liquefaction ratio for
138 the CPD analysis; however, shear strains evolution with depth do not relate with acceleration
139 in the DPD analysis. It should be mentioned that this paper was the first phase of a larger
140 study and that the present work is the scale up to a 2D more complex model. To the best
141 knowledge of authors, no study has been published that accounts for the effects of coupling
142 Δp_w and soil deformation on SSI. While PGA and other intensity measures can be lower with
143 CPD analysis in 1D soil profiles, the added effect of the 2D pore pressure's migration and
144 distribution, and of the structure's load and seismic performance, it could be prejudicial for
145 the analysis.

146 147 2 NUMERICAL MODEL

148 Two soil-structure models are considered in this work. They consist of reinforced concrete
149 (RC) buildings with a shallow rigid foundation, standing on saturated cohesionless soil. A
150 schema of the models is shown in Figure 1. One structure is modeled as an RC frame of one

151 span and two stories and the other is a single-degree-of-freedom (SDOF). The dynamic
 152 performance of the structures is different as they have different height, weight and
 153 predominant periods. Both structures lay on a 6m rigid foundation. All structural elements are
 154 elasto-plastic. Concerning the soil model, a 50m wide and 20m thick deposit of loose-to-
 155 medium (LMS) sand is overlaying an elastic bedrock. The shear modulus increases with
 156 depth and the equivalent shear wave velocity of the first 20m ($V_{s,20}$) is 200 m/s. The
 157 fundamental elastic frequency (f_0) of the soil profile, calculated with a low-strain frequency
 158 analysis, is equal to 2.63Hz. An elastoplastic multi-mechanism model is used to represent the
 159 soil behavior. Under the deposit, an engineering bedrock representing a half-space medium is
 160 modeled with an isotropic linear elastic behavior and a shear wave velocity (V_s) equal to
 161 550m/s. The ground water table is located 1m below the surface.
 162



164 Figure 1: Schema of the numerical model for a) B01 and b) T040

166 2.1 Finite element model

168 As the soil is assumed to be horizontally homogeneous, a 2D finite element computation was
 169 performed. The general purpose finite element code GEFDyn (Aubry and Modaressi 1996)
 170 was used. The saturated soil was modeled using quadrilateral isoparametric elements with
 171 eight nodes for both solid displacements and fluid pressures. The soil model is 20m depth,
 172 50m wide. The size of the elements is $0.5 \times 0.5 \text{m}^2$, which is in agreement with the suggestions
 173 made by Foerster and Modaressi (2007) to prevent numerical dispersion even for strong
 174 motion excitations. For a maximum frequency of 15Hz (of engineering interest in this study)
 175 and a minimum initial V_s of 135m/s (at the surface), a minimum of 36 points per wavelength

176 is allowed in the elastic domain. The number of points is high, although it will decrease as the
 177 soil softens and the V_s decreases. For the time discretization in the dynamic analysis, an
 178 implicit Newmark numerical integration scheme is used with parameters $\gamma_N = 0.611$ and $\beta_N =$
 179 0.301 . This induces a minimal numerical damping that affects principally the elastic response
 180 of the model, but allows an optimal high-frequency dissipation with minimal low-frequency
 181 impact (Hughes, 2000; Montoya-Noguera and Lopez-Caballero 2016). To take into account
 182 the interaction effects between the structure and the soil, a modified width plain-strain
 183 condition was assumed in the model. This approach was developed and verified with 3D
 184 models by Saez et al. (2013). In this case an out-of-plane dimension of 4m is used for the soil
 185 profile.

186 2.1.1 Coupled and Decoupled dynamic approach

187 A coupled dynamic approach derived from the $\mathbf{u} - p_w$ formulation of the Biot's generalized
 188 consolidation theory (Zienkiewicz and Shiomi 1984; Zienkiewicz and Taylor 1991) was
 189 introduced in the code by Modaressi (1987). This formulation consists of neglecting fluid
 190 acceleration terms and its convective terms so that the unknown variables remain the
 191 displacement of the solid \mathbf{u} and the pore water pressure p_w . As further simplifications, soil
 192 grain compressibility is assumed to be null and thermal effects are ignored. The behavior of
 193 the solid skeleton is derived assuming the principle of effective stress as proposed by
 194 Terzaghi (1943), where the total stress tensor ($\boldsymbol{\sigma}$) is separated into two components: the
 195 effective stress tensor ($\boldsymbol{\sigma}'$) and the pore pressure (p_w). Which reads: $\boldsymbol{\sigma} = \boldsymbol{\sigma}' - p_w \mathbf{I}$ with \mathbf{I} the
 196 identity second order tensor. Under such hypotheses the set of governing equations is:

- 197 • Overall equilibrium for the soil-fluid mixture

$$200 \quad \text{div} \boldsymbol{\sigma}' - \text{grad}(p_w) + \rho \mathbf{g} = \rho \ddot{\mathbf{u}} \quad (1)$$

201 where ρ is the total average unit mass ($\rho = n\rho_w + (1 - n)\rho_s$); n is the soil porosity; ρ_w , the
 202 fluid mass; ρ_s , the soil particle mass; \mathbf{g} , the gravity acceleration vector and \mathbf{u} , the solid
 203 skeleton displacement.

- 204 • Equilibrium of water and flow conservation equation using generalized Darcy's law.
 205 Assuming each phase as homogeneous:

$$206 \quad \text{div} \dot{\mathbf{u}} - \text{div}(\mathbf{K}(\text{grad}(p_w) - \rho_w \mathbf{g})) - \text{div}(\mathbf{K} \rho_w \dot{\mathbf{u}}) + \frac{p_w}{Q} = 0 \quad (2)$$

207 where \mathbf{K} is the permeability tensor defined by $\mathbf{K} = \boldsymbol{\kappa} / \rho_w \mathbf{g}$, $\boldsymbol{\kappa}$ being the kinematic
 208 permeability tensor, and Q is the compressibility parameter ($Q^{-1} = n/K_w + (1 - n)/K_s$),
 209 K_w and K_s being the fluid and solid compressibility, respectively.

210 Note that the pore water pressure generation and dissipation will depend on the permeability
 211 tensor and the compressibility parameter. Hence, the DPD analysis can be done by either
 212 reducing the compressibility to nearly zero as was done by Hartvigsen (2007), or by
 213 increasing the permeability to infinity. In this study another approach was chosen. It consists
 214 in keeping the same inertial effects, while no excess pore pressure is generated. For the
 215 inertial effects, the DPD analysis has a total unit mass equal to the coupled model, so the
 216 weight of the water contributes to the dynamic soil response (equation 1). Thus, in the DPD
 217 analysis the pore water pressures are computed based on a fixed water table level. It is worth
 218 noting that both CPD and DPD are performed with effective-stress analysis and with the
 219 same constitutive model.

225
226
227
228
229
230
231
232
233
234
235
236
237
238
239
240
241
242
243
244
245
246
247
248
249
250
251
252
253
254
255
256
257
258
259
260

2.1.2 Boundary conditions

Concerning boundary conditions, as the signal propagation is one-dimensional and as the response of an infinite semi-space is modeled, equivalent boundaries have been imposed on the lateral nodes (i.e. the normal stress on these boundaries remains constant and the displacements of nodes at the same depth in two opposite sides are the same in all directions) (Lopez-Caballero and Modaressi 2011). As the lateral limits of the problem are considered to be far enough from the structure in the middle of the model, periodic conditions are verified. For the bedrock's boundary condition, paraxial elements simulating "deformable unbounded elastic bedrock" have been used (Modaressi and Benzenati 1994). The paraxial elements efficiently evacuate outgoing (diffracting) waves in a local domain. While the vertically incident shear waves, defined at the outcropping bedrock, are introduced into the base of the model after deconvolution. Thus, the obtained movement at the bedrock is composed of both incident and reflected waves.

For the CPD analysis, during the dynamic loading, p_w are allowed to change below the groundwater table as a result of soil contraction and dilation due to shear strains. For the bedrock's boundary, the pore pressure conditions are assumed to be impervious. Therefore, no flux occurs across the interface boundary between the studied domain and the underlying semi-infinite space.

2.2 Soil Model

The soil deposit of cohesionless sand presents a maximum shear modulus (G_{max}) profile dependent on the mean effective stress (p'), as follows:

$$G_{max} = G_{ref} \left(\frac{p'}{p'_{ref}} \right)^{n_e} \quad (3)$$

where G_{ref} is the shear modulus at the reference stress (p'_{ref}) and n_e is the degree of nonlinearity. In this case, G_{ref} equals to 290MPa, p'_{ref} to 1MPa and n_e to 0.5. Thus, at free field G_{max} increases with depth as shown in Figure 2. The ground water table is placed 1m below the surface and for this superficial layer, G_{max} is constant (i.e. $n_e = 0$). The engineering bedrock is assumed to be deformable. It has an isotropic linear elastic behavior with a shear-wave velocity (V_s) equal to 1700m/s and a shear modulus (G) of 5.8 GPa. The impedance ratio between the bedrock and the deposit (I) is 7.8, evaluated as $I = (V_s^r \rho^r) / (V_s^s \rho^s)$, where superscripts r and s denote bedrock and bottom of the soil deposit. This value is sufficiently large to assume an elastic behavior of the engineering bedrock compared to the soil deposit.

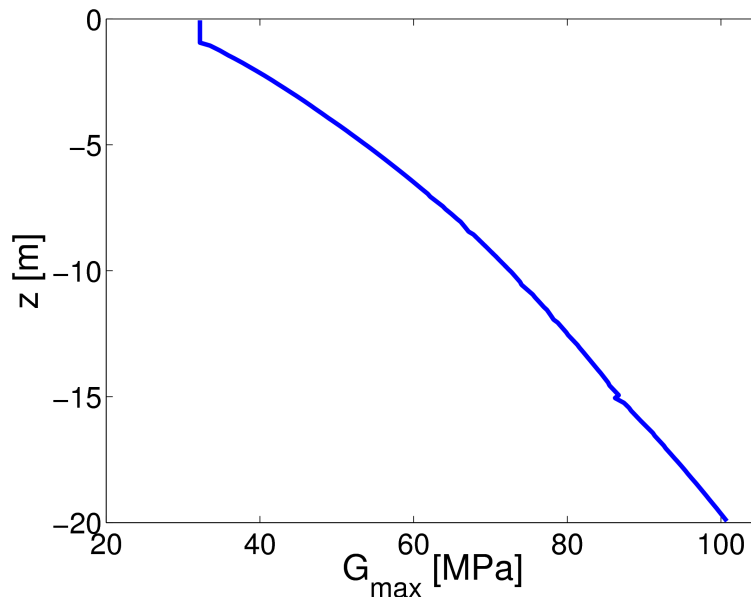


Figure 2: Initial shear modulus profile at free-field

2.2.1 Soil constitutive model

The elastoplastic multi-mechanism model used was developed at CentraleSupélec, formerly called École Centrale Paris (ECP), in the early 80s (Aubry et al. 1985; Modaressi 1987) and has been improved thereafter. The family of ECP models has been validated in a number of studies to simulate different kinds of loadings: seismic soil response of vertical arrays (Foerster and Modaressi 2007b; Regnier et al. 2015b), seismic response of soil structures (Sica et al. 2008) and pile installation (Berenguer 2014) among others.

The model uses a Coulomb type failure criterion and follows the critical-state concept. It can take into account a large range of deformations due to the decomposition into three domains (pseudo-elastic, hysteretic and mobilized) and the evolution of hardening based on the plastic strain. To model the cyclic behavior it uses a kinematic hardening which relies on the state variables at the last load reversal. For a complete description of the model refer to Aubry et al. (1982), Hujeux (1985), and Lopez-Caballero and Modaressi-Farahmand-Razavi (2010) among others. The soil model parameters were determined with the procedure defined by Lopez-Caballero et al. (2007) and are found in Montoya-Noguera and Lopez-Caballero (2016).

2.3 Structural model

Two models were used in order to analyze the effect of the coupling Δp_w and soil deformation for different predominant periods. The structures used will be called B01 and T040 and are reinforced concrete buildings with different size, weight and stiffness. In order to simulate the B01 structure, plastic hinge beam-column elements are used which take into account axial force (P) and bending moment (M) interaction (Prakash et al. 1993). This structure is a large-scale, one-span, two-story frame model proposed by Vechio and Emará (1992). In contrast, the T040 is a single-degree-of-freedom (SDOF) modeled with an elastic perfectly plastic behavior and is equivalent to a one-span, three-story building (Saez et al. 2013). It is a simplified model of a moment frame mid-rise structure according to the building type classification of HAZUS-MH MR3 (2003). This additional structure is used in this study to highlight the effects of the soil behavior in the structure performance near resonance. The main characteristics of both structures are shown in Table 1.

Property	B01	T040
Total height [m]	4.2	6
Width [m]	4	N/A
Mass [ton]	45	120
Fundamental fixed-base period T_0 [s]	0.24	0.40

296

297 Table 1: Structures' properties

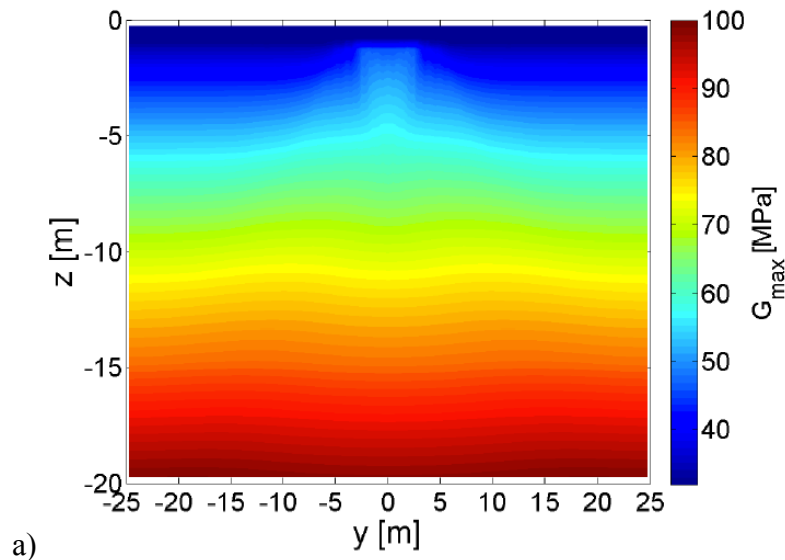
298

299 The structure foundation is modeled as a rigid block of 10cm thick and 6m wide. Between the
300 foundation and the soil, interface elements are used to allow relative movement of the
301 structure with respect to the soil, in order to avoid the traction effect. These elements follow a
302 Coulomb-type plastic criterion.

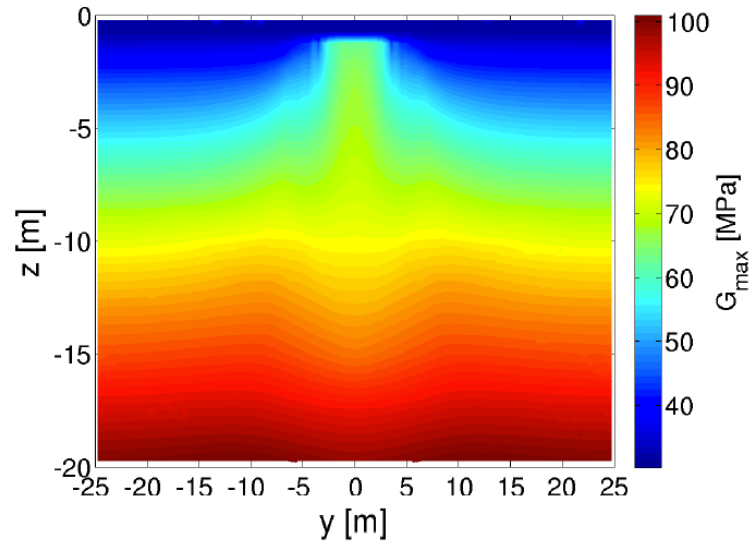
303

304 The building's weight affects the stress state (horizontal, vertical and shear) and also the
305 volumetric deformations, hence it affects the maximum shear modulus (G_{max}) defined
306 previously by equation (3). Thus, the effect of the structure's inertial load on the soil behavior
307 is seen on the differences in G_{max} along the horizontal axis which is shown in Figure 3 for
308 both structures. Note how the soil under the building presents higher values compared to free-
309 field for shallow depths until around 6m but for deeper soil, the G_{max} values are decreased.
310 Additionally, because T040 is heavier, this increase and decrease are greater and affect lower
311 depths. As the coefficient of earth pressure at rest (k_0) also increases with the building load,
312 the surrounding soil is stiffened but without the effect of the shear stress. Thus, a slight
313 increase in G_{max} is visible in the soil on this region, specially visible below 10m in T040
314 model (Figure 3b). It is worth noting that areas where the G_{max} increased might present
315 higher liquefaction resistance.

316



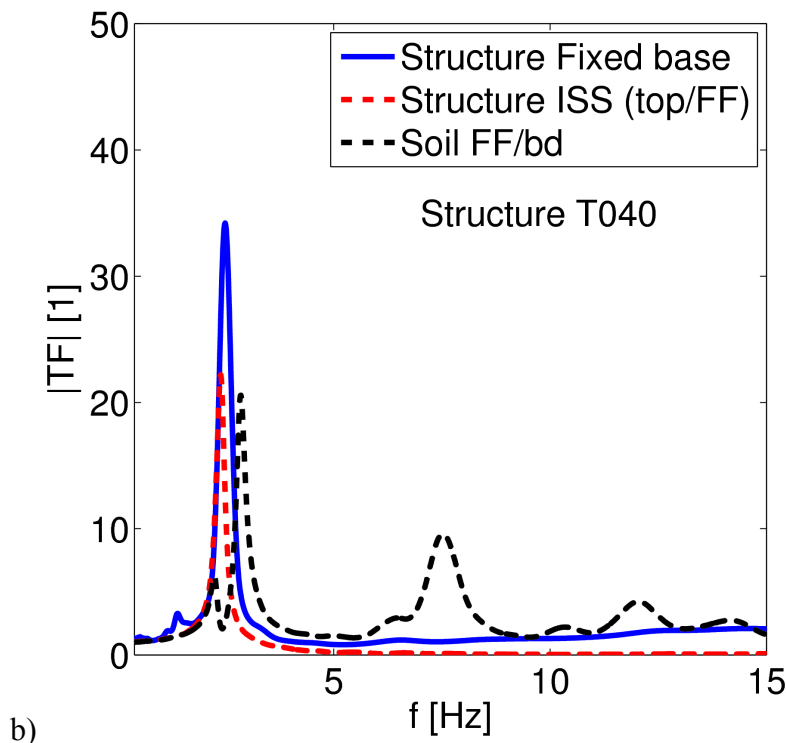
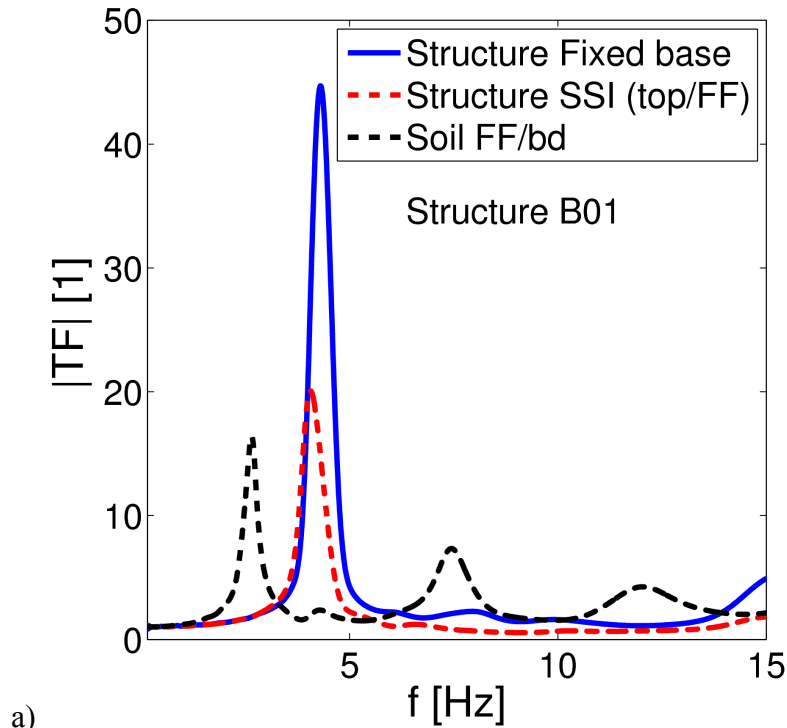
317



318 b)
 319 Figure 3: Shear modulus G_{max} distribution in the deposit before shaking for a) B01 and b)
 320 T040

321
 322 Concerning the initial seismic behavior, a scaled motion with a very low amplitude (i.e.
 323 $PHA \approx 10^{-5}g$) was used to evaluate the pseudo-elastic behavior of both soil and structure.
 324 Figure 4 shows the transfer functions ($|TF|$) - with fixed base and with soil-structure
 325 interaction (SSI) effects (top/FF) for the two structures. In addition, $|TF|$ of the soil deposit
 326 (FF/bedrock) is also shown. Firstly, it is interesting to note that, even if B01 has two stories,
 327 only one resonant frequency (f_{str}^{FB}) is observed - as the other one is above 15Hz. Hence both
 328 structures behave as SDOFs. The SSI effects consist of a shift to lower frequencies due to the
 329 flexibility of the foundation soil and a deamplification due to the material and radiation
 330 damping added by the soil (Mylonakis and Gazetas 2000). As expected, structure B01 being
 331 more rigid presents a higher interaction with the soil foundation; hence, the deamplification
 332 and the frequency shift are greater. Hence, the relative position of the fundamental frequency
 333 of the soil with respect to that of the structure and the frequency content of the input motion
 334 is very important for the inelastic dynamic SSI effects (Saez et al. 2013). Thus, as T040 main
 335 frequency is lower than that of the soil, higher SSI effects are expected when nonlinear soil
 336 degradation causes a shift of the soil frequency to lower values.
 337

338



339

340 Figure 4: Transfer function of the free-field and the structure for a) B01 and b) T040

341

342 2.4 Input earthquake motions

343 Ninety unscaled records were chosen from the Pacific Earthquake Engineering Research
344 Center (PEER) database. The signals used were recorded near the source - with a site-to-
345 source distance below 70km- and in dense soil conditions - i.e. 30m averaged shear-wave
346 velocity ($V_{s,30}$) above 600m/s. These signals are supposed to have minimal noise and are
347 appropriate for an outcropping bedrock condition. The events range between 6.2 and 7.7 in
348 moment magnitude.

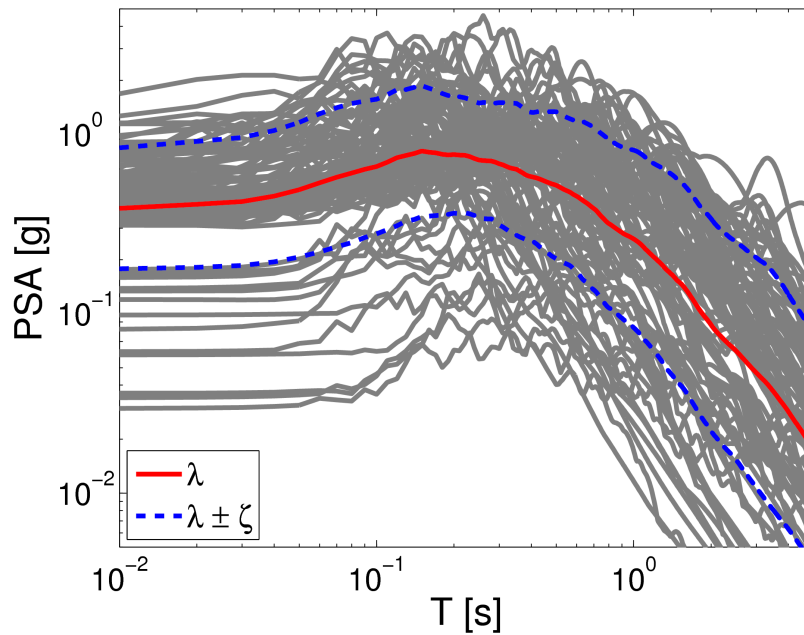
349

350 The statistics of some earthquake parameters calculated at outcropping conditions are shown
351 in Table 2. These intensity measures are peak horizontal acceleration (*PHA*), peak ground
352 velocity (*PGV*), mean period (T_m), predominant period (T_p), period of equivalent harmonic
353 wave ($T_{V/A}$), Arias intensity (I_A), significant duration (D_{5-95}), root-mean-square intensity
354 (I_{RMS}), spectral intensity (*SI*) and specific energy density (*SED*). The coefficient of variation
355 (*CV*) is high for all parameters and it is about 130% for I_A . A high variation in I_A is of great
356 importance given that after the sensitivity analysis performed by Lopez-Caballero and
357 Modaressi-Farahmand-Razavi (2010), it was proved to be the most influential input variable
358 on the average liquefaction developed on the upper 10m of the deposit, known as $Q_{H=10m}$.
359 Figure 5 shows the response spectra of all the input earthquake motions; accelerations were
360 filtered to 20Hz and the spectral amplitude has a 5% structural damping. Similarly, a great
361 variation is presented on the response spectra.

363 Table 2: Earthquake characteristics
364

Parameter	Range	Mean	CV [%]	Median
PHA [g]	0.03 λ 1.16	0.34	85	0.26
PGV [cm/s]	3.15 λ 121	30.52	92	21.80
T_m [s]	0.22 λ 0.87	0.49	31	0.47
T_p [s]	0.09 λ 0.65	0.30	51	0.25
$T_{V/A}$ [s]	0.21 λ 0.85	0.48	35	0.48
I_A [m/s]	0.02 λ 11.8	2.17	136	0.64
D_{5-95} [s]	4.12 λ 36.47	17.04	58	14.59
I_{RMS} [m/s ²]	0.04 λ 1.48	0.46	79	0.41
<i>SI</i> [m]	0.04 λ 1.16	0.37	80	0.30
<i>SED</i> [cm ² /s]	28.7 λ 34098	2705	241	499

365
366



367 Figure 5: Response spectra of acceleration for the outcropping motions. The geometric mean
368 (λ) and standard deviation (ζ) are shown in red and blue, respectively.
369

370

371 3 ANALYSIS AND RESULTS

372 The effect of coupling Δp_w and soil deformations on a soil-structure model is highly complex
373 and will affect several aspects of the response. First the analysis for one input motion will be
374 shown and then some results concerning all the motions will be addressed.

375

376 As recalled before, Montoya-Noguera and Lopez-Caballero (2016) have already analyzed the
377 effects of such coupling on the nonlinear seismic soil response. In that study, a 1D model of
378 the same soil deposit was used and compared to a denser soil deposit. As differences in the
379 response are given by the liquefaction apparition and the pore pressure generation, in this
380 study, only the more liquefiable soil is used for the SSI model.

381

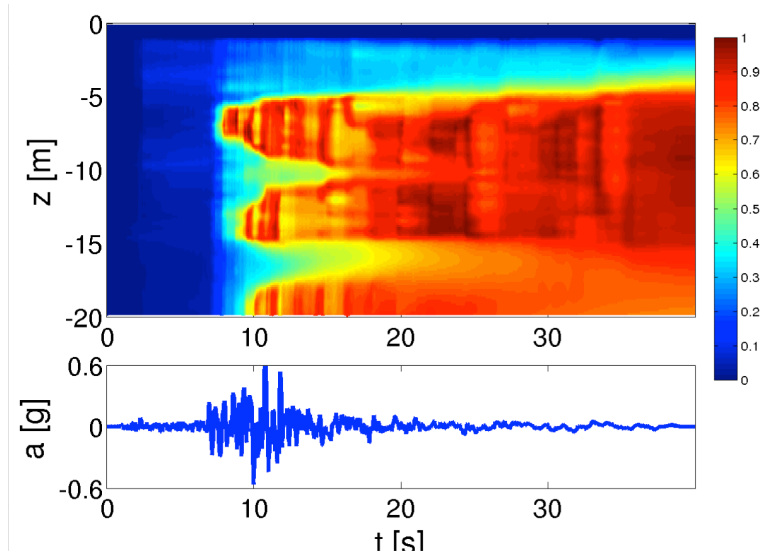
382 As explained in section

383 2.1.1, the main difference in the analyses will be given by the strains due to pore pressure
384 generation. As the latter increases, the difference in the soil deposit in terms of acceleration,
385 shear strain and settlement increases. Figure 6 and Figure 7 show the time histories of these
386 parameters for one motion tested.

387

388 First, in Figure 6 the evolution of the liquefaction ratio ($r_u = \Delta p_w / \sigma'_0$) is shown for a profile
389 far from the structure -i.e. in free field. In this analysis, liquefaction is described as the
390 significant reduction of the effective stress and therefore accounts for both phenomena: *true*
391 *liquefaction* (when $r_u = 1.0$) and *cyclic mobility* (i.e. $0.8 < r_u < 1.0$ with development of
392 large strains) (Koutsourelakis et al. 2002; Popescu et al. 2006). Thus, liquefaction is said to
393 be triggered when r_u equals 0.8. The location of liquefaction is dependent on the frequency
394 content and the soil stiffness evolution. In general, high frequency accelerations tend to
395 liquefy shallow soil while low frequency ones tend to liquefy deeper soil (Popescu, 2002;
396 Lopez-Caballero and Modaressi-Farahmand-Razavi, 2008). Real motions have complex
397 frequency content that varies in time thus liquefaction can be triggered at different locations.
398 For the case shown in Figure 6, liquefaction is first triggered at shallow soil as initially the
399 motion contains primarily high frequency amplitudes, and then liquefaction appears in deeper
400 soil when the motion contains low frequency content. The pore pressure generation and
401 dissipation through depth evolves as the motion's frequency content as well as the soil
402 stiffness changes. The frequency content of the motion is modified due to pore-pressure
403 generation as has been shown in previous studies (Kramer et al., 2015; Montoya-Noguera and
404 Lopez-Caballero, 2016). Additionally, the soil stiffness is reduced in zones where
405 liquefaction is triggered first and pore-pressure might migrate to deeper as well as shallower
406 depths. It can be noted that although the same soil properties are used throughout the deposit,
407 due to the increase in mean stress and the stress ratio difference, pore pressure generation
408 does not increase simultaneously through depth. Additionally, the time evolution of the
409 frequency content of the motion causes liquefaction at different depths in different instants.
410 For this motion, liquefaction first appears between 6 and 7m depth at around 8s, then it starts
411 in two other deeper zones. After the predominant duration of the motion, i.e. at around 15s,
412 the pore pressure migrates to other depths. At the end of shaking, r_u is almost evenly
413 distributed and it is still above 0.8 for depths between 5 and 15m.

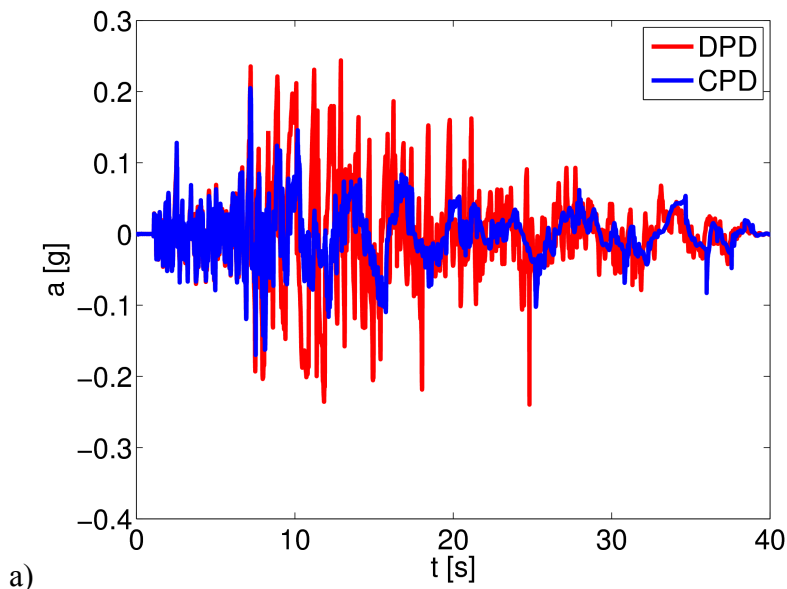
414



415
416
417
418
419

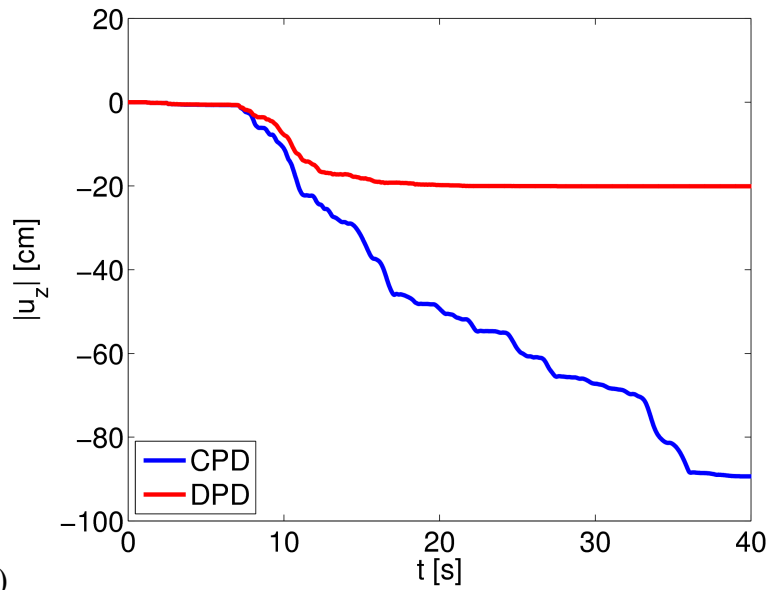
Figure 6: Time history liquefaction ratio at free field for one motion tested. The outcropping acceleration time history is shown below.

420 The time history of the acceleration at the structure's base is shown in Figure 7 for both
421 analyses. With respect to the outcropping motion shown in the previous figure, the peak
422 amplitude is almost one third for the DPD model and is slightly lower for the CPD one. The
423 differences in time between the analyses is easily related to the liquefaction evolution with
424 time also shown previously. After the first liquefaction triggering at 8s, the responses start to
425 differ. The CPD model presents a shift towards low frequencies after approximately 10s,
426 while the DPD shows higher spikes at higher frequencies. Similarly, the relative settlement of
427 the structure with respect to free field ($|u_z|$), shown in Figure 7, evolves differently starting
428 from 8s. Due to the pore pressure migration, evidenced in Figure 6, differences in $|u_z|$ are
429 greater after the predominant duration of the motion and by the end of the shaking, the CPD
430 model presents results more than 4 times higher than the DPD.
431



432

a)



433

b)

434

Figure 7: Time history analysis for one motion tested: a) Surface acceleration at structure's base and b) relative settlement of the structure with respect to free field

435

436

437

438

439

440

441

442

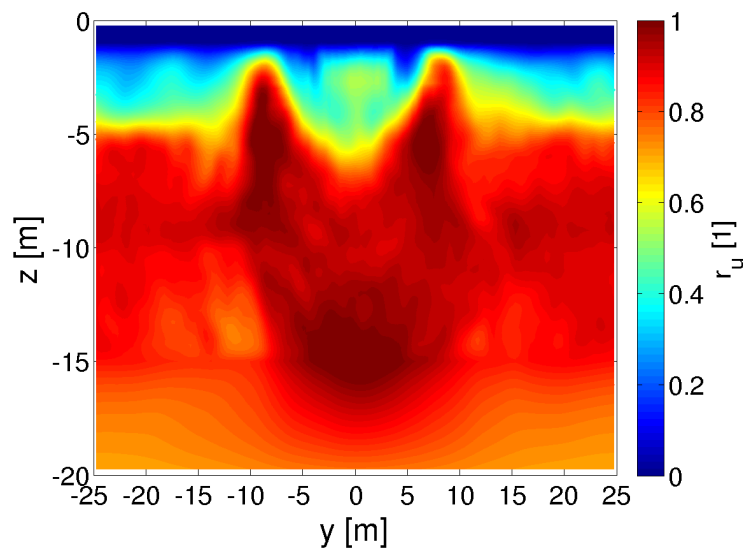
443

444

445

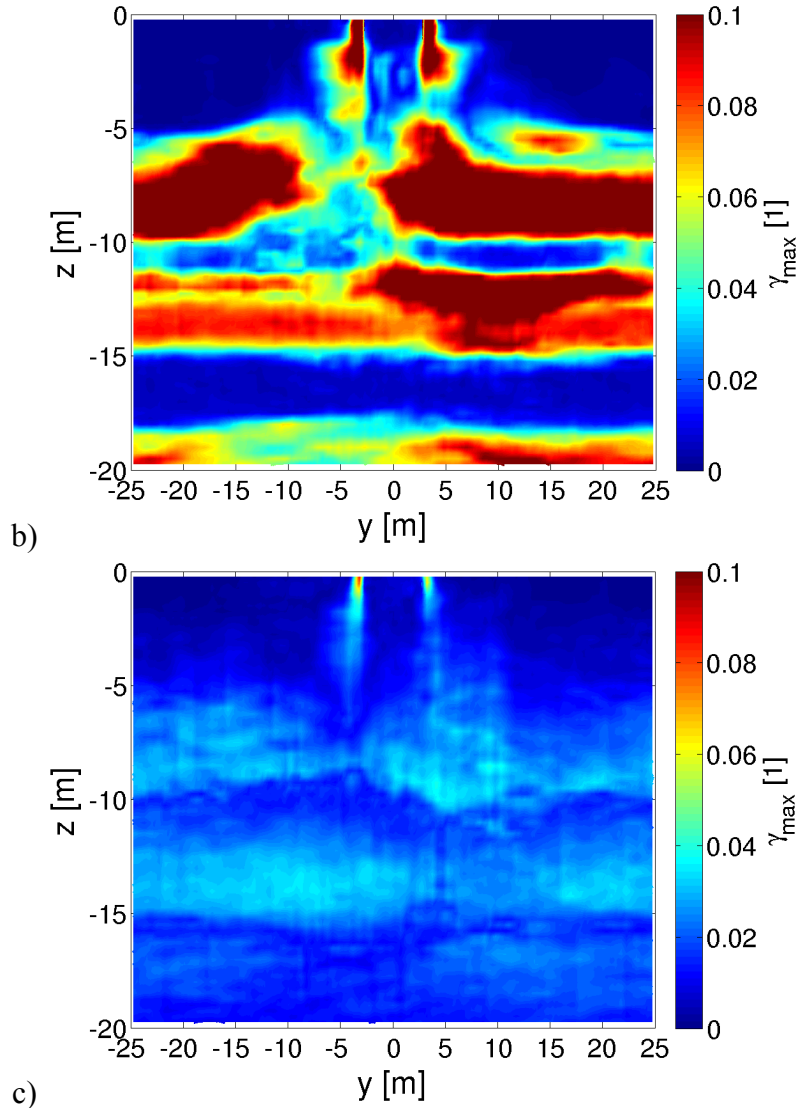
446

Concerning the entire deposit an important issue is the building load and the effects on the soil behavior. Figure 8 shows the liquefaction ratio at the end of shaking and the maximum shear strain for all the deposit. As the soil under the structure is stiffened, liquefaction is not presented there; however, around this area and because of the differences in the stress state, high r_u values appear beyond the structure's influence. On that account, the maximum shear strains are higher for the CPD model; often twice the value of the DPD one. Furthermore, while for the CPD model, the maximum values appear to be related to the places where maximum r_u values are located; for DPD, the maximum values are more evenly distributed through the deposit.



447

a)



448

449

Figure 8: Soil deposit analysis for one motion tested: a) Liquefaction ratio at the end of shaking and maximum shear strain for b) CPD and c) DPD models.

452

453

454

455

456

457

458

459

460

461

462

463

464

465

466

467

468

In terms of time history and in the entire deposit, the response for each motion varies greatly, due to the differences in frequency, energy and duration content of the motion. Hence to analyze all the motions tested, focus will be given to three engineering demand parameters: (1) peak ground acceleration (PGA) at the structure's base, (2) co-seismic relative settlement of the structure with respect to free field ($|u_z|$) and (3) maximum inter-story drift (ISD). These parameters are often used for seismic structure performance when SSI effects are taken into account and are often used for numerical analysis.

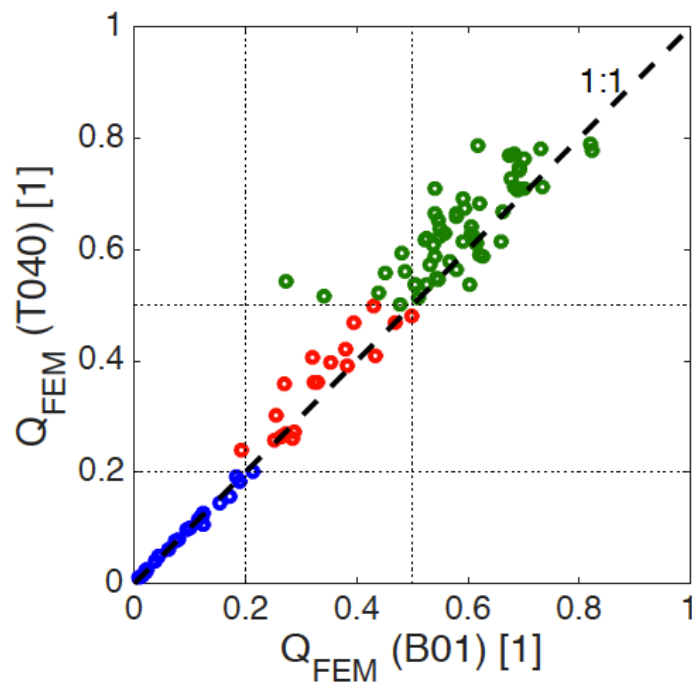
To relate the differences between the CPD and DPD analyses with the level of liquefaction, results are ranged by the liquefaction index (Q_{H-L}). This parameter was firstly introduced in 1D by Shinozuka and Ohtomo (1989) and it is extended to a 2D model as:

$$Q_{H-L} = \frac{1}{H-L} \int_0^H \int_0^L r_{u, \text{end}}(y, z) dy dz \quad (4)$$

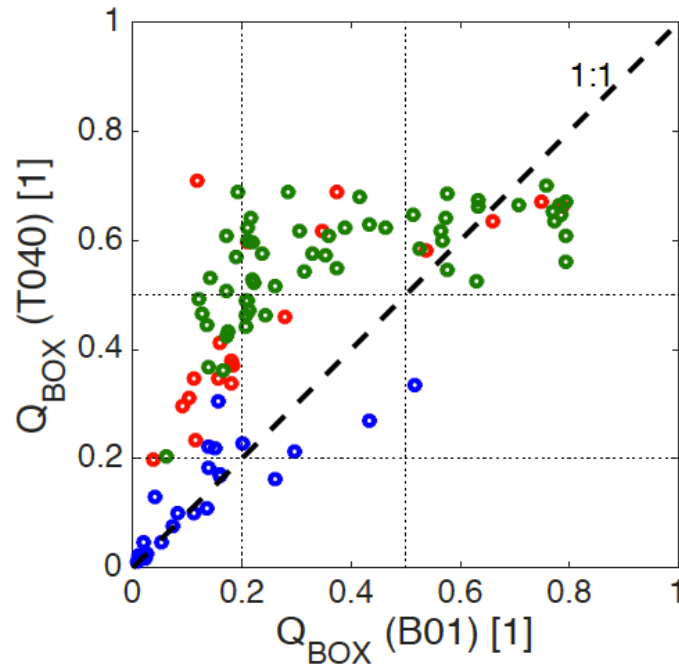
where $r_{u, \text{end}}$ is the liquefaction ratio, previously defined, evaluated at the end of shaking.

469 When $Q_{H \cdot L}$ is equal to unity, it means that liquefaction is present throughout the thickness H
 470 and the length L and thus gives information of the liquefaction ratio as well as the total
 471 liquefied area. As was seen previously, the structure has a significant effect on the
 472 liquefaction distribution in the soil deposit. Hence different values of H and L were used to
 473 evaluate $Q_{H \cdot L}$. Figure 9 compares the $Q_{H \cdot L}$ values between the models of the B01 and that of
 474 the T040 structures for two sets of H and L values : a) $H=20\text{m}$ and $L=50\text{m}$, which is the size
 475 of the entire finite element model (FEM) and b) $H=4\text{m}$ and $L=20\text{m}$, a box under the structure
 476 and below the water table level. Note that the results are divided in three groups by
 477 liquefaction levels: low (for Q_{FEM} below 0.2), moderate (between 0.2 and 0.5) and high
 478 (above 0.5). When the liquefaction index is evaluated throughout FEM, in Figure 9a, results
 479 are slightly higher with T040 but in average are fairly similar. This is to be expected since the
 480 motions energy is the same and, while under the structure the soil has been modified, other
 481 areas will be affected, as was observed in Figure 8a. In general, the differences between the
 482 Q_{FEM} results for the two models generally increase for higher Q_{FEM} . However, in average this
 483 difference is less than 8%.

484
 485 In contrast, the Q_{BOX} results shown in Figure 9b differ greatly between the two models. In
 486 most cases, the area tested presented higher levels of liquefaction when the T040 structure is
 487 used. Note that for the model with B01 structure, several cases present a Q_{BOX} value about
 488 0.2, while for the same cases, Q_{BOX} in the T040 model range between 0.2 to 0.7. On the other
 489 hand, in the T040 model, cases that presented Q_{BOX} values about 0.6, can present values
 490 ranging from 0.1 to 0.8 in the B01 model. In other words, in the same area, three motions can
 491 produce the same level of liquefaction in one model and on the other model, produce either
 492 low, moderate or high levels. In order to have a better relation with the motions energy and
 493 less influence on the structure used, the liquefaction levels defined by Q_{FEM} will be used in
 494 the following section. For further details on this analysis please refer to Montoya-Noguera
 495 (2016).



496 a)



b)

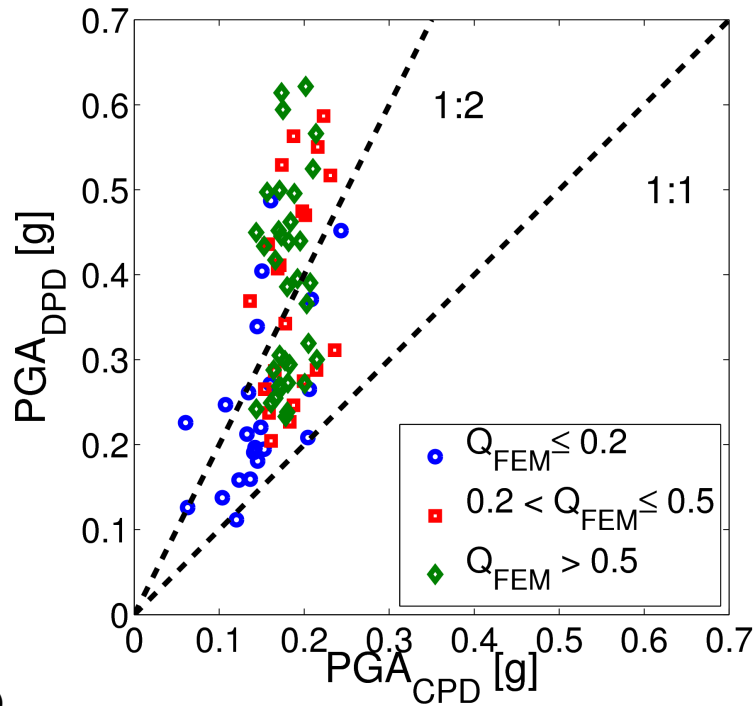
Figure 9: Effect of the structure on the Liquefaction index (Q_{H-L}) evaluated at: (a) the entire deposit (Q_{FEM}) and at (b) a box below the structure of size 4x20 (Q_{BOX})

3.1 Effect on the surface acceleration

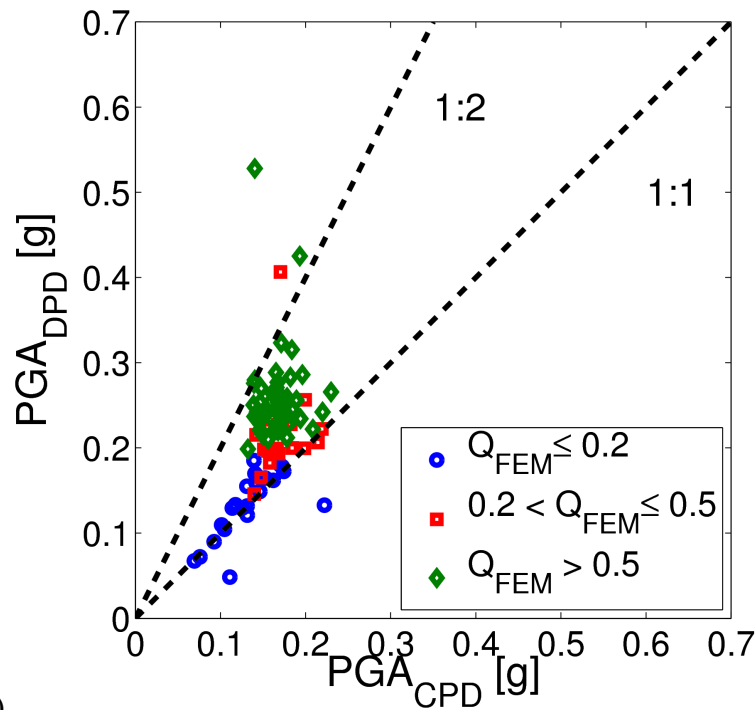
The amplification (or deamplification) of an earthquake motion due to site effects has been studied for many decades. Some researchers as Trifunac and Brady (1975) and Seed et al. (1976), among others, focused on the correlations between distance from source, earthquake magnitude and site conditions. In engineering practice, site amplification factors are often used to take into account these effects. An example of these factors, is the pioneering work of Idriss (1990) and Dickenson and Seed (1996) based on empirical measurements and engineering judgment of ratios between the peak surface acceleration of a soft deposit (PGA) and the peak horizontal acceleration at outcropping rock (PHA). Advanced numerical models of seismic soil behavior are useful to analyze the nonlinear effects on the ground motion; however if the site is susceptible to liquefaction, when no coupling of Δp_w and soil deformation is present, the model might largely overestimate the peak accelerations at surface.

Concerning the peak ground acceleration (PGA) at the structure's base, Figure 10 shows the comparison between the CPD and DPD values for both structures. For most motions tested the coupling produces a deamplification on the acceleration as it accounts for soil softening due to the pore pressure increase. However, for two motions with T040, where the value of Q_{FEM} is low, the CPD results are greater. These values correspond to high frequency peaks related to the dilatation phase. In general, the differences between the analyses are higher and more dispersive with the B01 structure, specially results for the DPD analysis are doubled for this structure. CPD results of PGA are similar for both structures and appear to be limited by the liquefaction triggering to a maximum value of about 0.25g. The structure weight has an influence on the liquefaction susceptibility but does not seem to affect the PGA value with CPD analysis. In contrast, DPD results of PGA are twice as big with B01 structure which is lighter. As shown in Figure 7a, in the DPD analysis the acceleration at the structure base presents high amplitude spikes at high frequencies which are controlled in the T040 model

529 because of the structure weight.
 530



531 a)

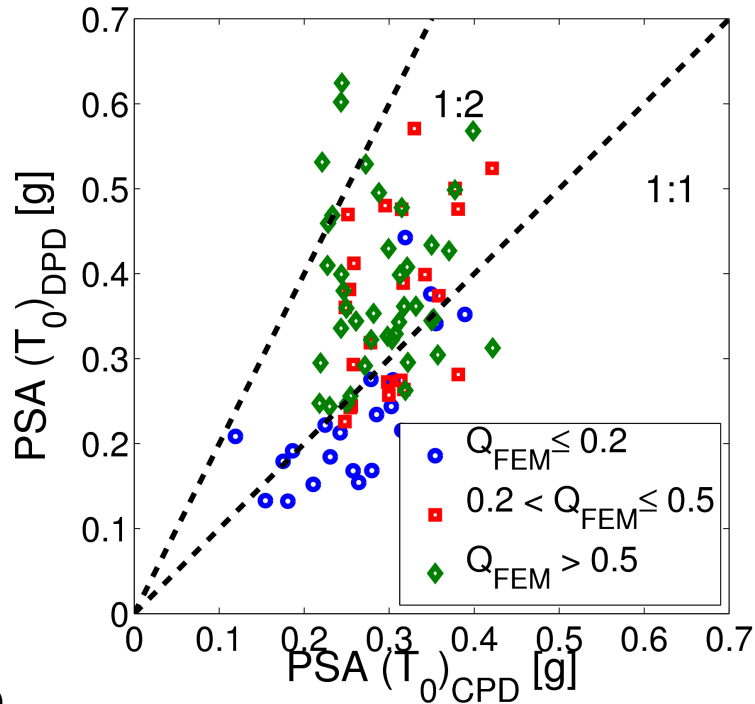


b)

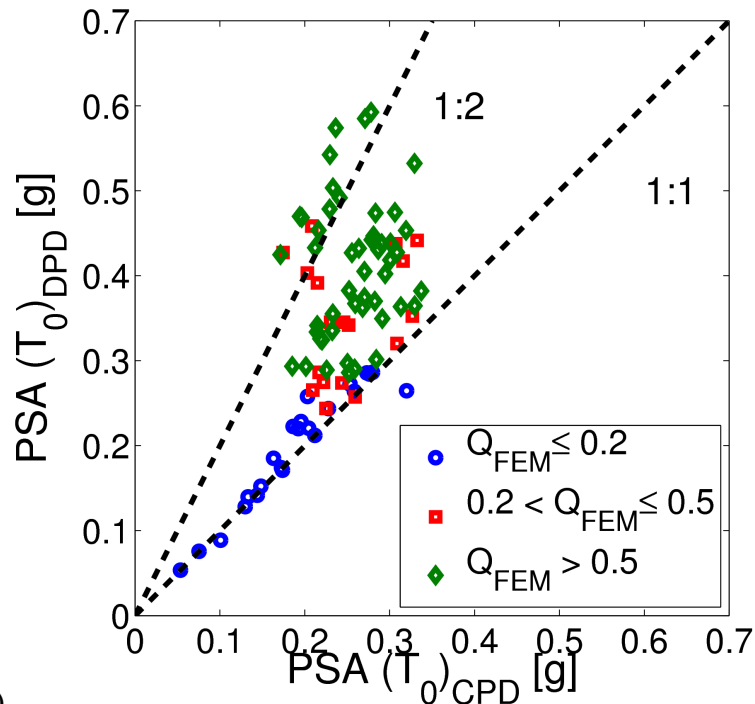
532
 533 Figure 10: Mean peak ground acceleration (PGA) at structure's base for CPD and DPD
 534 analyses for : a) B01 and b) T040 structures
 535

536 Considering that the structures tested have different predominant periods (T_0), the
 537 acceleration response spectra (PSA) is evaluated at each T_0 value and the comparison
 538 between CPD and DPD is shown in Figure **Error! Reference source not found.** As the T_0
 539 value of T040 is near the fundamental elastic period of the soil, the behavior presents less
 540 dispersion and higher PSA values for the decoupled model. On the contrary, with B01 higher
 541 dispersion is evidenced specially for the CPD analysis. The combined effect of SSI and

542 liquefaction is highly variable. With this structure, for almost all cases with low levels of
 543 liquefaction and even for some cases with intermediate and high levels, CPD results are
 544 higher. Although with the majority of the motions, the results were greater with DPD
 545 analysis. Hence, in general, as surface acceleration is concerned, the lack of coupling will
 546 produce a conservative analysis.
 547



548 a)



549 b)

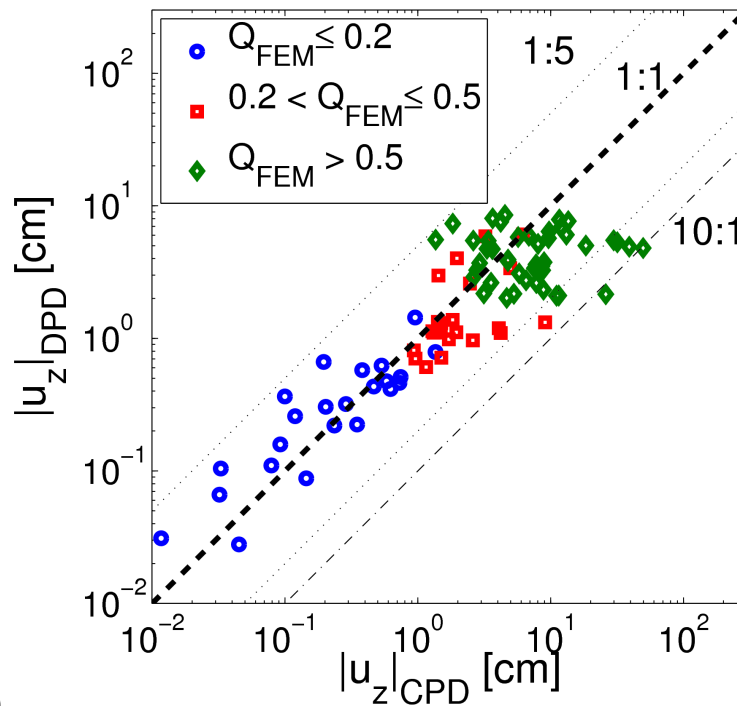
550 Figure 11: Comparison of CPD and DPD results for the response spectra of acceleration at
 551 structure's base evaluated at the predominant structure period: a) B01 and b) T040
 552

553 3.2 Effect on the structure settlement

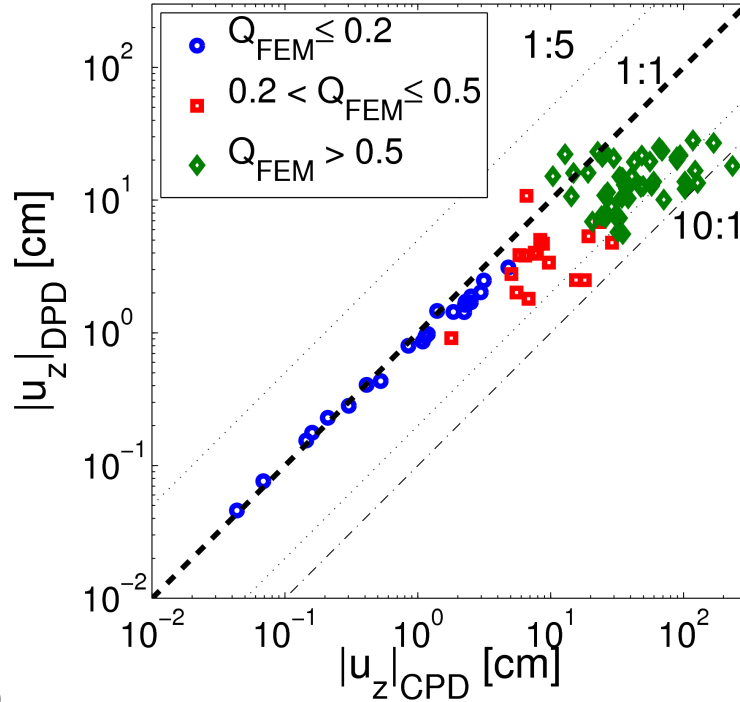
554 In contrast, regarding the relative settlement of the structure with respect to free-field ($|u_z|$),

555 shown in Figure **Error! Reference source not found.**, taking into account the effect of
 556 coupling in most cases will result in higher values. This could be prejudicial for the
 557 structure's performance. However, as for PSA, some motions presented lower values when
 558 CPD is used. Specially for low levels of liquefaction and with the B01 structure. In contract,
 559 when liquefaction levels are high and $|u_z|$ is the greatest, the differences between the
 560 analyses increase. Note that for some cases, the CPD results are about 10 times greater than
 561 the DPD ones.

562
 563 As T040 is heavier, $|u_z|$ are more important and with CPD results are above 10cm for
 564 approximately half of the motions tested. This value corresponds to the limit for slight
 565 damage state of reinforced concrete frame buildings given by Bird et al. (2006). Above it,
 566 cracks in structural elements appear and damage has to be repaired. If no coupling was
 567 present, this fundamental aspect would be overseen and results will be unconservative or
 568 even dangerous for the structure.
 569



570 a)



571 b)

572 Figure 12: Comparison of CPD and DPD results for the relative settlement with respect to
 573 free-field: a) B01 and b) T040

574

575 3.3 Effect on the structure seismic demand

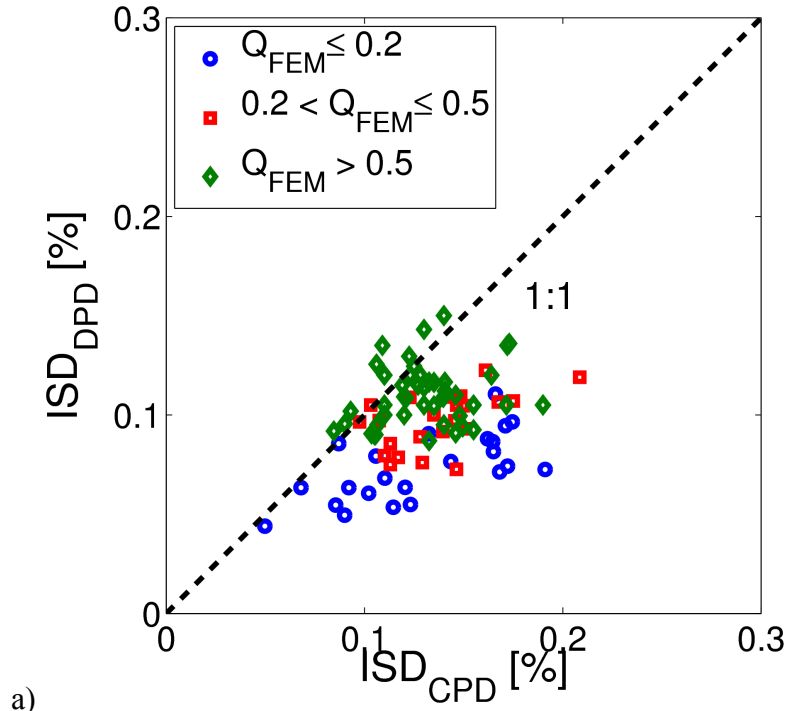
576 Regarding the seismic demand on the structure, the comparison of CPD and DPD results of
 577 the maximum inter-story drift (ISD) is shown in Figure **Error! Reference source not**
 578 **found.** This parameter is appropriate to show the effect on the drift in structures with
 579 different height as it normalizes the maximum horizontal displacement evaluated at each
 580 level by its corresponding height. It is interesting to note how the two structures present very
 581 different results. For B01, the CPD values are greater than the DPD ones for almost all cases;
 582 while for T040, it is the contrary.

583

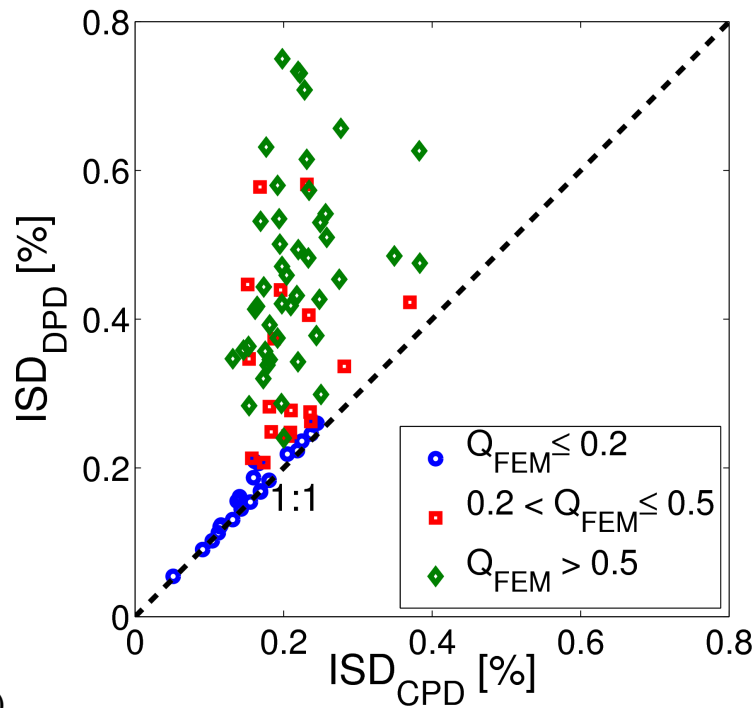
584 Recall that the SSI effect for the quasi-elastic behavior is more important for B01, as shown
 585 in Figure 4. Hence, with this structure and for low levels of liquefaction the SSI effect is
 586 greater than the coupling effect and ISD is higher with CPD analysis. But when the
 587 liquefaction level increases, the coupling effect is more important as the soil will attenuate
 588 the motion and the structure drift with CPD will be reduced. Concerning T040, the values
 589 with the DPD analysis are more than doubled when Q_{FEM} is above 0.5, even four times
 590 greater for some cases. Additionally, a high dispersion is evidenced for these values. For
 591 models of liquefiable soil where structure nonlinearity is taken into account but there is no
 592 coupling of Δp_w with soil deformation, the response could be largely overestimated. This
 593 would lead to a conservative design. In contrast, when SSI effects are important, the lack of
 594 coupling of Δp_w will lead to an unconservative or prejudicial conception.

595

596



a)



b)

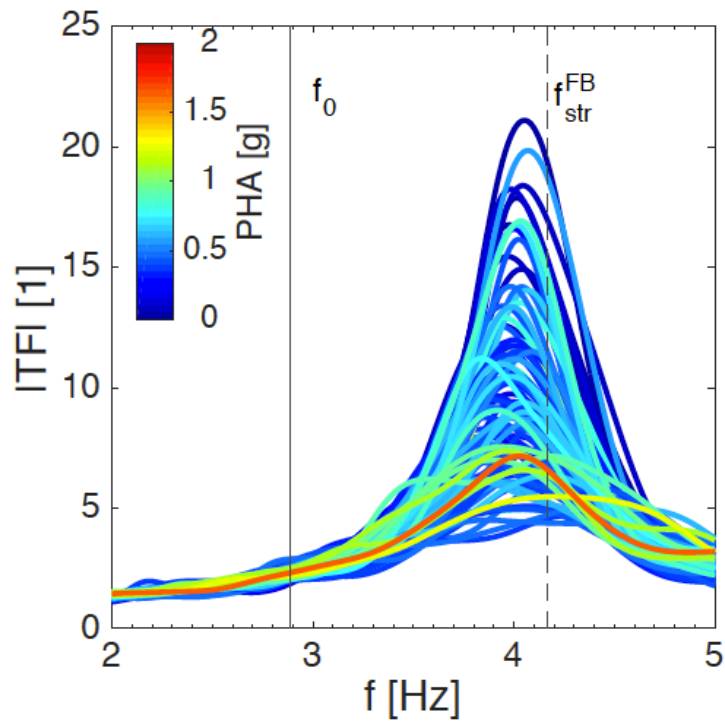
597

598 Figure 13: Comparison of CPD and DPD results for the maximum inter story drift: a) B01
599 and b) T040

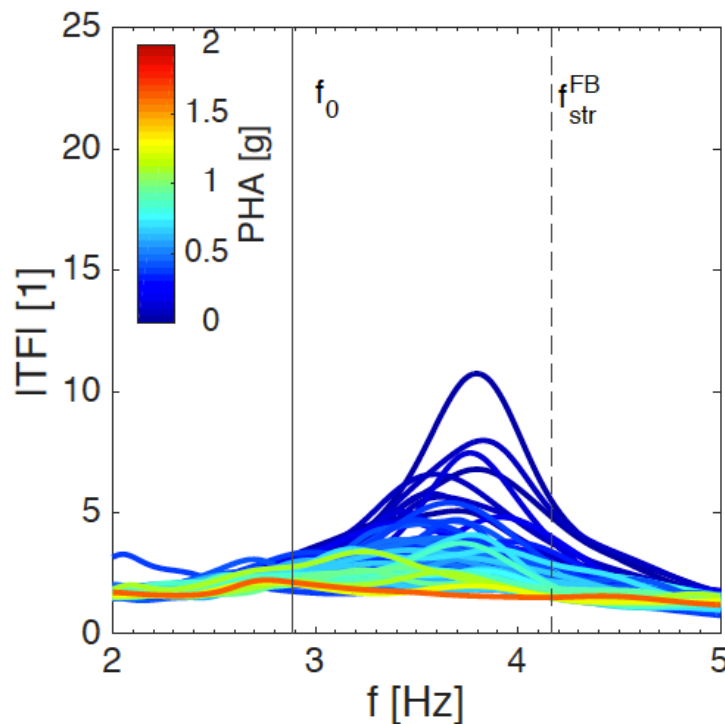
600

601 Figure 14 shows the nonlinearity evolution of the soil - structure system by analyzing the
 602 transfer function ($|TF|$) between the top floor and the free field (FF) for B01. This function
 603 takes into account the structure's performance as well as the SSI effects. Results are ranged
 604 by the peak horizontal acceleration (PHA) recorded at the outcropping rock. Additionally, the
 605 fundamental elastic frequency of the soil deposit (f_0) and that of the structure in fixed
 606 base condition (f_{str}^{FB}) are shown as reference. With DPD analysis, a lower amplitude of the
 607 predominant frequency even for low PHA values. This deamplification drops drastically after
 608 about 0.7g and the frequency is reduced by more than 1Hz for the highest PHA. In contrast,

609 with CPD analysis the deamplification is smaller and almost no frequency shift is evidenced.
 610 This can be explained by the liquefaction presented in the deposit that can isolate the SSI
 611 effect between the surficial soil and the structure.
 612



613 a)



614 b)

615 Figure 14: Nonlinearity evolution at the structure's $|TF|$ (Top/FF) for B01 with a) CPD and b)
 616 DPD analyses

617
 618 3.4 Comparison with 1D models

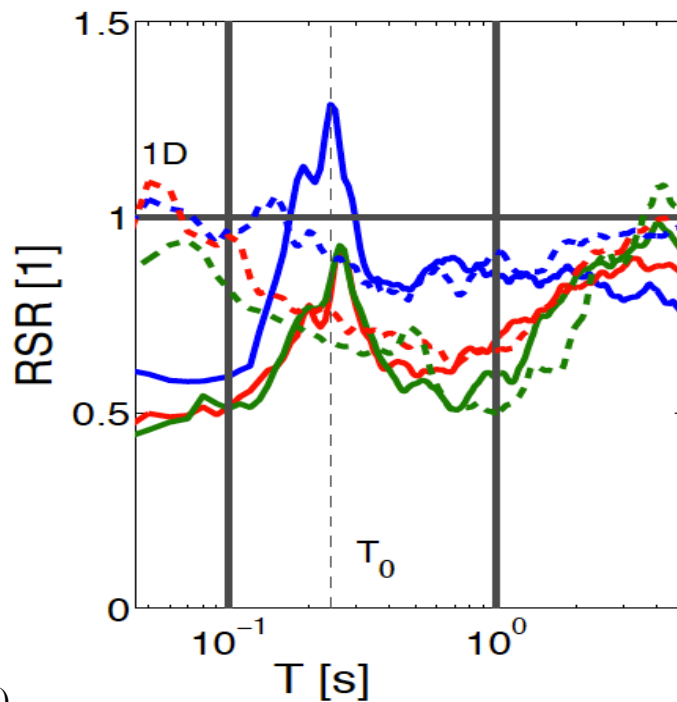
619 Finally the effect of pore pressure generation on the surface acceleration is compared
 620 between a 1D FF model and the 2D soil-structure model. When no soil-structure interaction

621 (SSI) is present, the wave propagation is evaluated on a soil column and the altered spectra of
 622 acceleration is used on the structure's model. In Figure 15 the response spectra ratio (RSR)
 623 evaluated at the surface of a 1D model is compared to that of the base of each structure. In
 624 this figure the effects of both the SSI and the coupling of Δp_w and soil deformation can be
 625 analyzed. RSR is defined by:

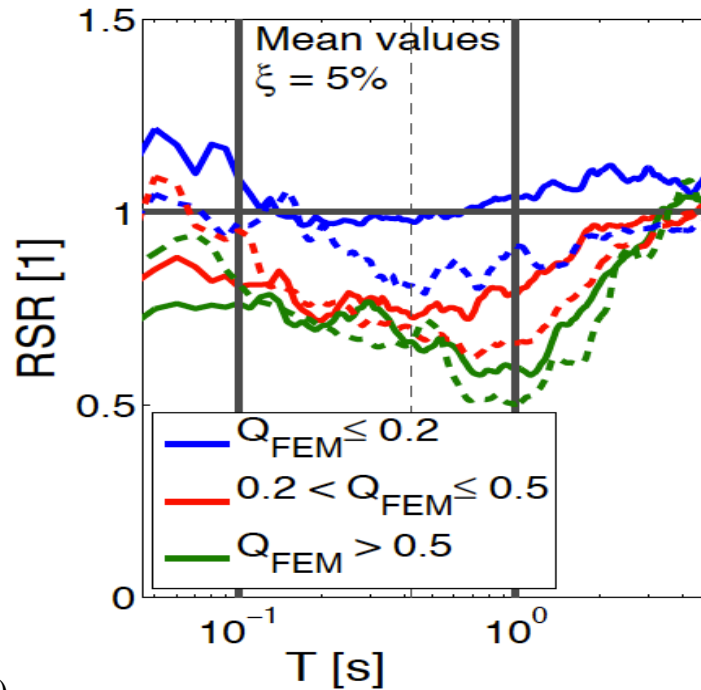
$$RSR(T) = PSA_{CPD}(T)/PSA_{DPD}(T) \quad (5)$$

626
 627
 628
 629 The mean values for each liquefaction index level are shown in Figure 15. The results for the
 630 1D model are shown in dashed lines and are the same in both figures, as no structure is
 631 present. However, the results with the structures vary greatly. It seems that the SSI effect is
 632 important for B01 while it is almost negligible for T040 - as 1D and 2D results are roughly
 633 similar. Specially, at the structure's predominant period, RSR is affected for B01 ($T_0=0.24s$)
 634 where with low liquefaction levels the CPD amplification is even 25% higher than the DPD
 635 one; while for T040 no change is visible at this period ($T_0=0.4s$). Differences between 1D and
 636 2D results are greater for short periods, below 0.2s and more important for B01. Which
 637 means that the coupling effects are greater when SSI effects are more important. However,
 638 even if RSR appears to be related to the liquefaction level, the mean values for high and
 639 intermediate levels are similar in all the period range used and specially for B01 results. This
 640 aspect is worth mentioning as it means that although liquefaction is not triggered, the pore
 641 pressure migration due to the structure's effect on the soil modifies the acceleration response
 642 at the structure's base. Hence, a CPD analysis is preferred when modeling 2D SSI even if
 643 liquefaction potential is low.

644



645 a)



646 b)
 647 Figure 15: RSR at 1D model and at structure's base for: a) B01 and b) T040
 648

649 4 CONCLUSIONS

650 A finite element analysis was used to investigate the effect of coupling excess pore pressure
 651 and soil deformation on a soil-structure model. Two mechanically-equivalent analyses were
 652 performed with 90 unscaled earthquake motions: one taking into account coupling (CPD) and
 653 one fully-drained (DPD). The same effective-stress model was used for the calculations and
 654 the initial elastic behavior was proved to be the same. The present study aimed to highlight
 655 the importance of accurately model liquefiable soils in order to improve performance-based
 656 earthquake engineering (PBEE). One of the main findings of this study is that when models
 657 are fully drained, less shear strains are developed hence the relative settlement of the
 658 structure is underestimated for most motions and both structures tested. Even if in free-field
 659 (FF), DPD analysis presented higher settlement, the structure's weight affects the soil
 660 behavior and hence the structure's settlement with respect to FF is in general underestimated
 661 with DPD models. Additionally, if the SSI effects are significant, i.e. when the predominant
 662 period of the structure is near to that of the soil, the maximum ISD is consistently
 663 underestimated by the DPD analysis. In this regard, the use of DPD models will not be
 664 recommended for a PBEE design. Two main effects are involved in this analysis: the
 665 coupling of excess pore pressure and soil deformation and the interactions between the soil
 666 deposit and the structure. These can be beneficial or detrimental for different EDPs but the
 667 analysis should include both as it seems that the complex relation between them will vary for
 668 each motion, soil and structure tested.

669
 670 The relation of both SSI and CPD effects was shown to be highly complex. Thus, the effects
 671 of CPD could be further evaluated for other soil deposits and other structures in order to
 672 increase the reliability of the results. In addition to the nonlinearity of the soil, the structure's
 673 nonlinearity could be further enriched comprehending the evaluation of damage and failure.
 674 The present study focused on the comparison between drained and coupled analyses,
 675 however, it would be interesting to also compare the response of perfectly undrained analysis
 676 available on several commercial softwares and performed by practitioners. More on this topic
 677 will be analyzed in a future work Furthermore, some challenges are still to be acknowledged.

678 Comparisons of CPD and DPD models with vertical array data in liquefiable soils could be
679 helpful to better understand the importance of coupling excess pore pressure and
680 deformations. Additionally, physical modeling of structures founded on liquefiable soil could
681 be performed in shaking tables and centrifuge tests to further validate the relation between
682 both SSI and CPD effects.

683

684 5 ACKNOWLEDGMENT

685 The work described in this paper was partly supported by the SEISM Paris Saclay Research
686 Institute. This support is gratefully acknowledged.

687

688 References

- 689 V.P. Drenevich and F.E. Richart. Dynamic prestraining of dry sand. *Soil Mechanics and*
690 *Foundations Division*, 96 (SM2):453–469, March 1970. PROC PAPER 7160.
- 691 R. Dobry and W.F. Swiger. Threshold strain and cyclic behavior of cohesionless soils. In *3rd*
692 *ASCE/EMDE Specialty Conference*, pages 521–525, Austin, Texas, September 1979.
- 693 R. Dobry, R.S. Ladd, F.Y. Yokel, R.M. Chung, and D. Powell. Prediction of pore water
694 pressure buildup and liquefaction of sands during earthquakes by the cyclic strain method.
695 *NBS Building Science Series 138*, page 150, 1982.
- 696 N. Yoshida and S. Iai. Nonlinear site response and its evaluation and prediction. In *2nd*
697 *International Symposium on the Effect of Surface Geology on Seismic Motion*, pages 71–90,
698 Yokosuka, Japan, 1998.
- 699 A. Hartvigsen. Influence of pore pressures in liquefiable soils on elastic response spectra.
700 Master’s thesis, University of Washington, Seattle, 2007.
- 701 N. Yoshida. Applicability of total stress seismic ground response analysis under large
702 earthquakes. In *COMPDYN2013: 4th ECCOMAS Thematic Conference on Computational*
703 *methods in structural Dynamics and earthquake engineering*, page 13, Kos Island, Greece,
704 June 2013.
- 705 S. Montoya-Noguera and F. Lopez-Caballero. Effect of coupling excess pore pressure and
706 deformation on nonlinear seismic soil response. *Acta Geotechnica*, 11(1):191–207, February
707 2016. 10.1007/s11440-014-0355-7.
- 708 S.L. Kramer, A.J. Hartvigsen, S.S. Sideras, and P.T. Ozener. Site response modeling in
709 liquefiable soil deposits. In *4th IASPEI / IAEE International Symposium: Effects of Surface*
710 *Geology on Seismic Motion*, page 13, University of California Santa Barbara, august 2011.
- 711 H.B. Seed and I.M. Idriss. Simplified procedure for evaluating soil liquefaction potential.
712 *Journal of the Soil Mechanics and Foundations Division*, 107(SM9):1249–1274, 1971.
- 713 I.A. Beresnev and K.L. Wen. Nonlinear site response: a reality? *Bulletin of the Seismological*
714 *Society of America*, 86 (6):1964–1978, 1996.
- 715 J.R. Gingery, A. Elgamal, and J.D. Bray. Response Spectra at Liquefaction Sites during
716 Shallow Crustal Earthquakes. *Earthquake Spectra*, 2014. 10.1193/101813EQS272M. In
717 press.
- 718 B.S. Chiou and R.R. Youngs. An NGA model for the average horizontal component of peak
719 ground motion and response spectra. *Earthquake Spectra*, 24(1):173–215, 2008.
- 720 D. Aubry and A. Modaressi. *GEFDyn - manuel scientifique*. Ecole Centrale Paris, France:
721 LMSSMat, July 1996.

722 E. Saez, F. Lopez-Caballero, and A. Modaressi-Farahmand-Razavi. Inelastic dynamic soil-
723 structure interaction effects on moment-resisting frame buildings. *Engineering structures*,
724 51(1):166–177, 2013. 10.1016/j.engstruct.2013.01.020.

725 E. Foerster and H. Modaressi. A diagonal consistent mass matrix for earthquake site response
726 simulations. In *4th International Conference on Earthquake Geotechnical Engineering*
727 (*ICEGE*), Thessaloniki, Greece, June 2007a. Paper: 1242.

728 O.C. Zienkiewicz and T. Shiomi. Dynamic behavior of saturated porous media: the
729 generalised biot formulation and its numerical solution. *International Journal for Numerical*
730 *and Analytical Methods in Geomechanics*, 8(1):71–96, 1984.

731 O.C. Zienkiewicz and R.L. Taylor. *The Finite element method, solid and fluid mechanics,*
732 *dynamics and non-linearity*, volume 2. McGraw-Hill Book Company, London, 4th edition,
733 1991.

734 H. Modaressi. *Modélisation numérique de la propagation des ondes dans les milieux poreux*
735 *anelastiques*. PhD thesis, Laboratoire MSSMat, Ecole Centrale Paris, Châtenay-Malabry,
736 France, 1987. Advisor: Denis Aubry.

737 K. Terzaghi. *Theoretical Soil Mechanics*. John Wiley & Sons, 1943.

738 F. Lopez-Caballero and A. Modaressi. Numerical analysis: Specification and validation of
739 used numerical methods. FP7-SME-2010-1-262161, PREMISERI project, Paris, France,
740 2011.

741 H. Modaressi and I. Benzenati. Paraxial approximation for poroelastic media. *Soil Dynamics*
742 *and Earthquake Engineering*, 13 (2):117–129, 1994.

743 D. Aubry, D. Chouvet, A. Modaressi, and H. Modaressi. GEFDyn 5, logiciel d’analyse du
744 comportement statique et dynamique des sols par elements finis avec prise en compte du
745 couplage sol-eau-air, 1985.

746 E. Foerster and H. Modaressi. Nonlinear numerical methods for earthquake site response
747 analysis II- case studies. *Bulletin of Earthquake Engineering*, 5(3): 325–345, 2007b.
748 10.1007/s10518-007-9034-5.

749 J. Régnier, L. F. Bonilla, P. Y. Bard, D. Assimaki, Y. Hashash, and et al. International
750 benchmark on numerical simulations for 1D, non-linear site response (PRENOLIN):
751 verification phase based on canonical cases. *Bulletin of the Seismological Society of America*,
752 2015. In review.

753 S. Sica, L. Pagano, and A. Modaressi. Influence of past loading history on the seismic
754 response of earth dams. *Computers and Geotechnics*, 35(1):61–85, 2008.
755 10.1016/j.compgeo.2007.03.004.

756 L.A. Berenguer Todo-Bom. *Numerical modeling of soil-pile interaction considering grain*
757 *breakage in finite deformations*. PhD thesis, Ecole Centrale Paris, Paris, France, 2014.

758 D. Aubry, J.C. Hujeux, F. Lassoudiere, and Y. Meimon. A double memory model with
759 multiple mechanisms for cyclic soil behavior. In *International Symposium on Numerical*
760 *Modeling in Geomechanics*, pages 3–13, Balkema, 1982.

761 J.C. Hujeux. Une loi de comportement pour le chargement cyclique des sols. In *Génie*
762 *Parasismique*, pages 278–302, France, 1985. Presses ENPC. Edited by V. Davidovici.

763 F. Lopez-Caballero and A. Modaressi-Farahmand-Razavi. Assessment of variability and
764 uncertainties effects on the seismic response of a liquefiable soil profile. *Soil Dynamics and*
765 *Earthquake Engineering*, 30 (7):600–613, 2010.

766 F. Lopez-Caballero, A. Modaressi-Farahmand-Razavi, and H. Modaressi. Nonlinear
767 numerical method for earthquake site response analysis i — elastoplastic cyclic model and
768 parameter identification strategy. *Bulletin of Earthquake Engineering*, 5(3): 303–323, jun
769 2007. 10.1007/s10518-007-9032-7.

770 V. Prakash, G. Powel, and S. Campbell. *DRAIN-2DX Base program description and user*
771 *guide: version 1.10*. Dept. of Civil Engineering, University of California, Berkeley, 1993.
772 UCB/SEMM-1993/17.

773 F.J. Vecchio and M.B. Emara. Shear deformation in reinforced concrete frames. *ACI*
774 *Structural journal*, 89(1):46–56, 1992.

775 HAZUS-MH MR3. *Multi-hazard Loss Estimation Methodology*. Federal Emergency
776 Management Agency, Washington, DC, 2003.

777 G. Mylonakis and G. Gazetas. Seismic soil–structure interaction : beneficial or detrimental?
778 *Journal of Earthquake Engineering*, 4(3): 277–301, jul 2000. 10.1080/13632460009350372.

779 S. Koutsourelakis, J.H. Prévost, and G. Deodatis. Risk assessment of an interacting structure-
780 soil system due to liquefaction. *Earthquake Engineering and Structural Dynamics*, 31
781 (4):851–879, 2002. 10.1002/eqe.125.

782 R. Popescu, J. H. Prévost, G. Deodatis, and P. Chakraborty. Dynamics of nonlinear porous
783 media with applications to soil liquefaction. *Soil dynamics and earthquake engineering*, 26
784 (6):648–665, 2006.

785 M.D. Trifunac and A.G. Brady. A study on the duration of strong earthquake ground motion.
786 *Bulletin of the Seismological Society of America*, 65(3):581–626, 1975.

787 H.B. Seed, J. Murarka, J. Lysmer, and I.M. Idriss. Relationships between maximum
788 acceleration, maximum velocity, distance from source and local site conditions for
789 moderately strong earthquakes. *Bulletin Seismological Society of America*, 66 (4):1323–1342,
790 1976.

791 I.M. Idriss. Influence of local site conditions on earthquake ground motions. In *Proc. of IV*
792 *U.S. Nat. Conf. on Earthquake Engineering*, volume 1, pages 55–57, Palm Springs,
793 California, 1990.

794 S.E. Dickenson and R.B. Seed. Nonlinear dynamic response of soft and deep cohesive soil
795 deposits. In *Proceedings of the international workshop on site response subjected to strong*
796 *earthquake motions*, volume 2, pages 67–81, Yokosuka, Japan, 1996.

797 M. Shinozuka and K. Ohtomo. Spatial severity of liquefaction. In *Proceedings of the second*
798 *US-Japan workshop in liquefaction, large ground deformation and their effects on lifelines*,
799 pages 193–206, New York, 1989. NCEER.

800 S. Montoya-Noguera. *Assessment and mitigation of liquefaction seismic risk: Numerical*
801 *modeling of their effects on SSI*. PhD thesis, CentraleSupélec, Paris-Saclay University,
802 Châtenay-Malabry, France, 2016.

803 J.F. Bird, J.J. Bommer, H. Crowley, and R. Pinho. Modelling liquefaction-induced building
804 damage in earthquake loss estimation. *Soil Dynamics and Earthquake Engineering*, 26
805 (1):15–30, 2006. 10.1016/j.soildyn.2005.10.002.