# INFLUENCE OF 2D AND 3D SOIL MODELING ON DYNAMIC NON-LINEAR SSI RESPONSE

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**ABSTRACT :**

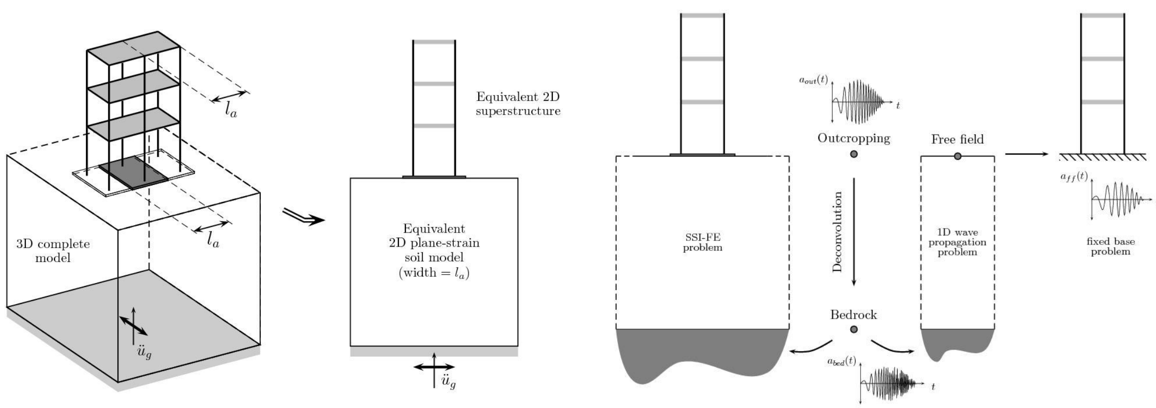
The purpose of this paper is to investigate the accuracy of 2D FE plane-strain computations compared to complete 3D FE computations for dynamic non-linear SSI problems. In particular, this study is focused on the effect of non-linear soil behavior on the dynamic response of the reinforced concrete multistory frames. An balanced 2D plane-strain approach is presented and several numerical FE simulations using this approach are carried out using a plastic-hinge column-beam model to represent the concrete elements' behavior and a realistic elastoplastic constitutive law to represent that of the soil's.

**KEYWORDS:** Seismic nonlinear response, nonlinear Soil-Structure Interaction, plane-strain modeling, nonlinear Finite Elements

## 1. INTRODUCTION

In general, under earthquake loading, the soil reaches the limit of its elastic behavior before the structural elements. Thus, an earthquake analysis approach assuming nonlinear structural behavior under fixed base condition or with linear soil-structure interaction (SSI) hypothesis is not consistent.

In practice, there are several approaches to estimate the effect of the nonlinear soil behavior on the seismic response of structures. Usually, 2D finite element computations assuming plane-strain condition for the soil can be carried out in order to asses the role of the non-linear soil behavior on the superstructure response. However, for this approach, special assumptions related to the soil condition and the superstructure must be considered.




(a) Typical 3D regular multistory building and equivalent soil plane-strain model

(b) SSI-FE and Two-Step approaches

Figure 1 Summary of proposed approaches

Assuming a standard regular multistory building as sketched in Fig.1a, an equivalent 2D model for the superstructure can be constructed straightforward for the shorter dimension. With this purpose, a simple assumption is to take a typical transverse resistant axis (equally separated of $l_a$) loaded by tributary weight/mass over the distance $l_a$ in order to preserve approximately the internal forces in structural elements. The stiffness contributions of the transverse elements (following longer direction) are neglected.

If the foundation is supposed to be infinitely rigid following the longer distance, an equivalent plane-strain foundation-soil model is valid. Nevertheless, in order to properly take into account the superstructure-foundation-soil interaction effects over the internal stress-strain state, the width $l_a$ must be included in the plane-strain formulation for the soil-foundation part of the model (Saez, 2008).

In this paper, the influence of SSI effects on the response of multistory frames is investigated. For this purpose, numerical simulations of non-linear dynamic analyses (i.e. non-linearity of the soil and the structure behavior) are performed in order to study the role of the dynamic soil behavior on the seismic response of buildings. A set of energy measures are introduced in order to highlight the role of different energy dissipation mechanisms when the non-linear SSI effects are taken into account. Thus, several 2D plane-strain finite element computations are carried out using non-linear elastoplastic models to represent both the soil and the structure behavior. Results obtained by simplified computations performed following a two-step approach (it will be described below), are compared with the ones obtained from fully SSI non-linear time-history finite element modeling analyses.

## 2. PROPOSED APPROACHES

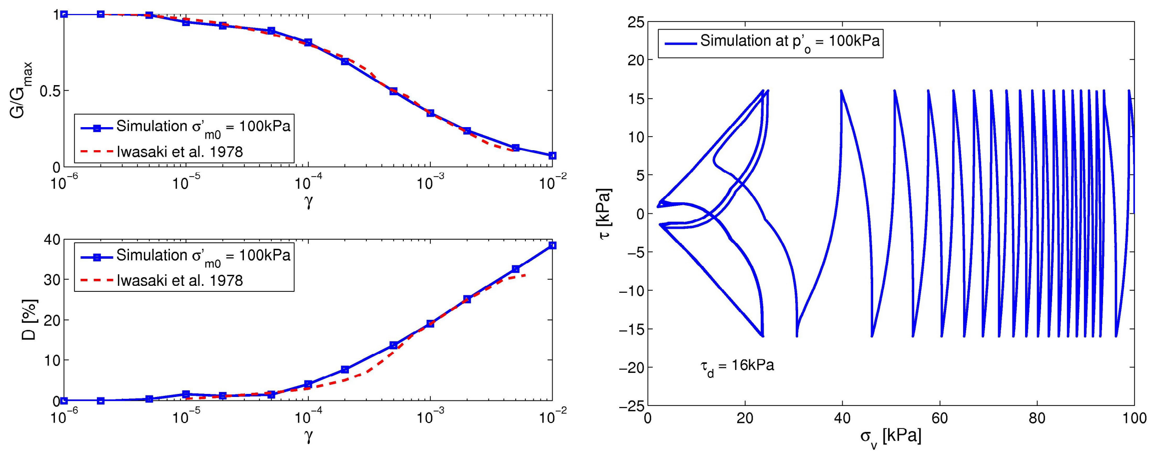
In order to investigate the effect of non-linear soil behavior on seismic demand evaluation, a comparative dynamical analysis is carried out. First, a complete finite element model including soil and structural non-linear behavior is used to asses the effect of non-linear dynamic soil-structure interaction on the structural response. Secondly, a two-step approach is carried out consisting in: first a non-linear 1D wave propagation problem is solved for a simple soil column of the foundation soil. Next, the obtained free field motion is imposed as ground motion to a fixed base structural model. The two approaches are sketched in Fig. 2b.
The analysis is conducted for two concrete multistory frames: b01 (2 levels) and b02 (7 levels) over a homogenous dense sandy soil deposit in two hydraulic conditions: dry and fully saturated. The bedrock is placed at the depth of 30m. The Table 2.1 shows the main characteristics of the used buildings. The shear wave velocity profile gives an average shear wave velocity in the upper 30m ($V_{s,30}$) of 232.8m/s for dry conditions and of 204.3m/s for saturated condition, corresponding to a site category C of Eurocode 8 (deep deposit of dense or medium dense soil) in both cases. The low-strain frequency analysis gives a fundamental period ($T_{soil}$) of 0.46s for the dry case and 0.48s for the saturated case.
The simulations were performed with the Finite Element code GEFDYN (Aubry et al. 1985; Aubry and Modaressi 1996). A numerical validation of the soil-structure interaction phenomenon assuming linear elasticity behavior for both the soil and the structure was performed comparing FE computations with a numerical BE-FE code (Saez et al. 2006; Saez, 2008).

Table 2.1. Properties of the buildings

| Building | Mean interstory height [m] | Total height [m] | Total Mass [Ton] | First fixed base period [s] ($T_0$) | Length of foundation [m] |
|---|---|---|---|---|---|
| b01 | 2.10 | 4.20 | 40 | 0.24 | 6.0 |
| b02 | 2.60 | 20.12 | 390 | 0.76 | 10.0 |

### *2.1. Material constitutive models*

The ECP's elastoplastic cyclic multi-mechanism model (Aubry et al., 1982; Hujeux, 1985) is used to represent the soil behavior. This model can take into account the soil behavior in a large range of deformations. The model is written in terms of effective stress. The representation of all irreversible phenomena is made by four coupled elementary plastic mechanisms: three plane-strain deviatoric plastic deformation mechanisms in three orthogonal planes and an isotropic one. The model uses a Coulomb type failure criterion and the critical state concept. The evolution of hardening is based on the plastic strain (deviatoric and volumetric strain for the deviatoric mechanisms and volumetric strain for the isotropic one). To take into account the cyclic behavior a kinematical hardening based on the state variables at the last load reversal is used. The soil behavior is decomposed into pseudo-elastic, hysteretic and mobilized domains.
The soil model's parameters are obtained using the methodology suggested by Lopez-Caballero et al. (Lopez-Caballero et al., 2003, 2007). In order to verify the model's parameters, the behavior of the sand must be studied by simulating drained (DCS) and undrained cyclic shear tests (UCS). The Fig. 2a shows the responses of these DCS tests obtained by the model of the sand at an effective stress of 100kPa. The tests results are compared with the reference curves given by Iwasaki (Iwasaki et al. 1986).




(a) (b)

Figure 2 Soil's test simulations

In saturated conditions, the evolution of pore pressure can be observed during the UCS. The Fig. 2b shows the pore pressure evolution for a stress controlled shear test with the same model's parameters. The increment of pore pressure reduces the effective stress inducing cyclic mobility without liquefaction.

The concrete structural elements are modeled by plastic hinge beam-column elements. The model is based on the two-component model presented by Giberson (1969) and the modifications included in DRAIN-2DX software (Prakash V. et al., 1993) to take into account axial force and bending moment interaction. The model consists of a linear component and an ideally elastoplastic component, so that each beam can have only bilinear hysteresis loops at each end. The initial slope on the moment-curvature diagram is determined from the sum of the stiffnesses of both components while the second slope is determined by the stiffness of only the linear component of the beam. Plastic hinges that yield to constant moment form an elastic-plastic component. The moments in the elastic component continue to increase, simulating strain hardening. The basic DRAIN-2DX model was extended to some features of GEFDYN.

### *2.3. Earthquake selection*

At present there are many sources of earthquake strong-motion records that could be used to provide many thousands of records as input to the structural models. However, since the studied structural models are complex and consequently take time to run, it is important that a small selection of strong-motion records be chosen in order to cut down the number of runs required but to obtain general tendencies.

In order to select an efficient set of input accelerograms some ideas from the theory of Design of Experiments (DOE) are employed. Since there is an infinite variety of possible earthquake ground motions it is useful to characterize them using a number of scalar strong-motion parameters that approximately measure different properties of the motions (amplitude, frequency content, duration, etc.).

The geographical scope of this paper is France. Metropolitan France has a seismic hazard that is thought to be characterized by earthquakes of magnitudes ($M_L$) less than or equal to 6.3 with an average focal depth less than or equal to 12km (Marin et al., 2004). In view of this, the database of strong-motion records developed by Ambraseys et al. (2004) has been chosen as the source of data for this work since it provides a large set of data mainly from moderate ($M_w$<6.5) shallow (h<30km) earthquakes that occurred within Europe and the Middle East. The selection procedure is a two-level factorial technique where for three strong-motion parameter selected records are chosen to fall within two intervals (Douglas, 2006): either high or low value bins. We choose three strong-motion parameters: significant duration $T_{SR}$ (Trifunac & Brady, 1975); Arias intensity $AI$ (Arias, 1970) and the mean period $T_m$ (Rathje et al., 2004), associated to duration, energy and frequency content, respectively. The ranges of the low and high bins were chosen in order to have sufficient numbers of records within each bin (Table 2.2).

Table 2.2. Strong-motion parameters and ranges of low and high bins used for selecting records.

| Parameter | Low bin range | High bin range |
|---|---|---|
| $T_{SR}$ | ≤ 10s | > 10s |
| $AI$ | ≤ 0.07m/s | > 0.07m/s |
| $T_m$ | ≤ 0.5s | > 0.5s |

An experiment is constituted by $2^3$=8 records (or runs). Each experiment was repeated four times (4 earthquakes selection), thus a total of 32 runs were conducted for each building on each soil type.

### *2.4. Finite element (SSI-FE) and Two-Step (T-S)approaches*

The Finite Element model is composed of: the structure, the soil foundation and a part of the bedrock. The 30m thick homogenous soil deposit is modeled by 4 node linear elements. In the bottom, a layer of 5m of elastic bedrock is added to the model. The finite element mesh used for modeling this problem is showed in Figure 3. For the bedrock's boundary condition, paraxial elements simulating a "deformable unbounded bedrock" have been used (Modaressi and Benzenati, 1994). The incident waves, defined at the outcropping bedrock are introduced into the base of the model after deconvolution. In the analysis, the lateral limits of the problem are considered to be far enough from the structure so that periodic conditions are verified on them. Thus, equivalent boundaries have been imposed on the nodes of these boundaries. The computations are carried-out in the time domain.

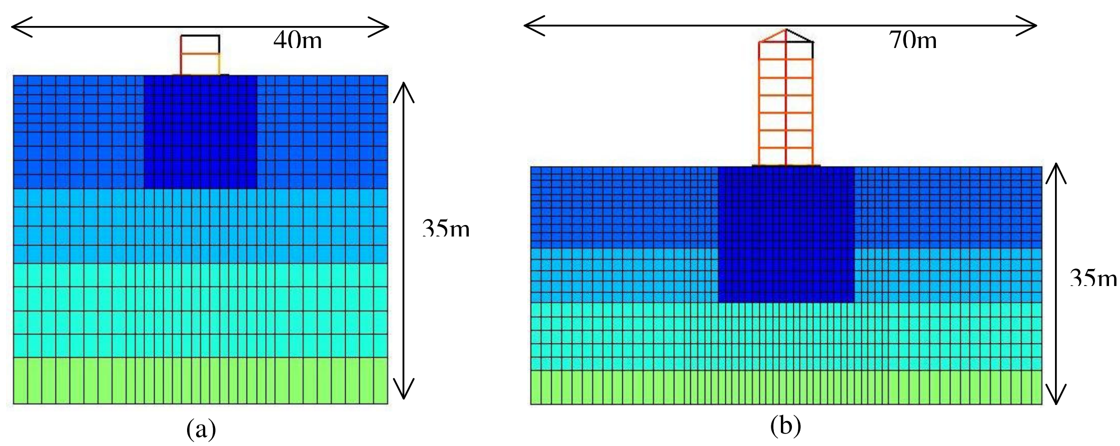



Figure 3 Finite element mesh (SSI-FE approach).

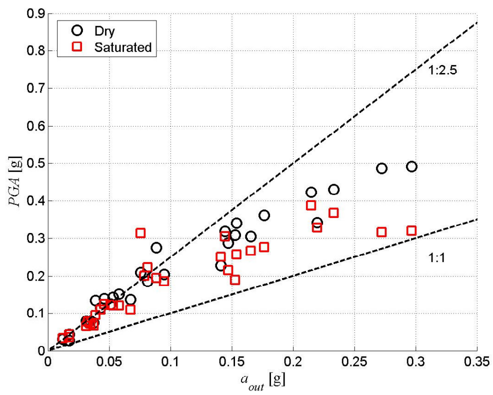
In order to prevent the apparition of tractions between the foundation and the surrounding soil under static or dynamic loading (uplift allowed), Mohr-Coulombs type interface elements have been added to the soil-foundation interface.

Concerning the Two-Step approach, the first step is to solve a non-linear one-dimensional wave propagation problem for a simple soil column. The mesh consists of one column of solid elements obeying the same constitutive model as in the SSI-FE approach. The same boundary conditions have been imposed. The incident waves, defined at the outcropping bedrock are introduced into the base of the model after deconvolution. In the second step, the obtained free field motion is imposed as ground motion to a fixed base structural model. This two-step approach neglects all SSI effects, but takes into account the effect of non-linearity in the behavior of both soil and structure.

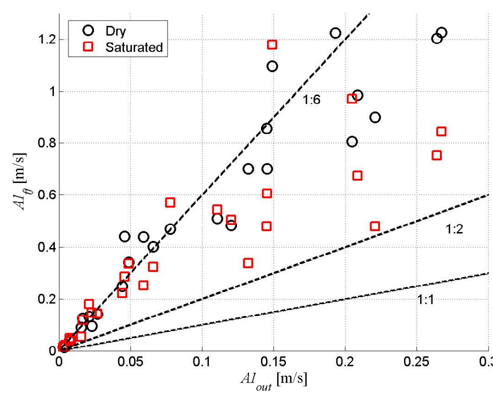
## 3. SOIL ANALYSIS AND RESULTS

In order to define the input motion for the T-S approach (corresponding to first step), a free field dynamic analysis of the soil profile, was performed. The response of the free field soil profile was analysed for the earthquake records selected as described above as outcropping input. The Figure 4a shows the simulation

values representing the *PGA* at surface with respect to maximum acceleration at outcropping ($a_{out}$). It is possible to see that for weak base acceleration the behavior of both soil deposits is similar: the amplification is near to 2.5 times the acceleration recorded at outcropping. In this range, the reduction in the effective stress due to the water has not an evident effect. It is noted that due to soil non-linearity the amplification of the ground response decays with the amplitude. In saturated conditions, the pore-pressure build-up acts as a frequency filter and the amplification of the input motion vanishes for large $a_{out}$ values (Ghosh and Madabhushi, 2003; Lopez-Caballero and Modaressi-F.-R., 2008).
The effect of the soil non-linearity over the input motion can be also studied in terms of the energy associated to each motion. The Arias intensity *AI* computed at outcropping ($AI_{out}$) and computed at free field ($AI_{ff}$) are plotted in Figure 4b for both soils. In this case, the amplification behavior is similar compared to acceleration amplitude schema. For weak *AI*, the amplification is approximately constant near to six times the outcropping value (low-strain domain). For $AI_{out}$ larger than 0.1m/s, the amplification diminishes due to hysteric energy dissipation in the soil. In this range, in general terms, the amplification of the saturated soil is lower than the obtained one for the dry case but it is more than two times the value obtained at outcropping.




(a)

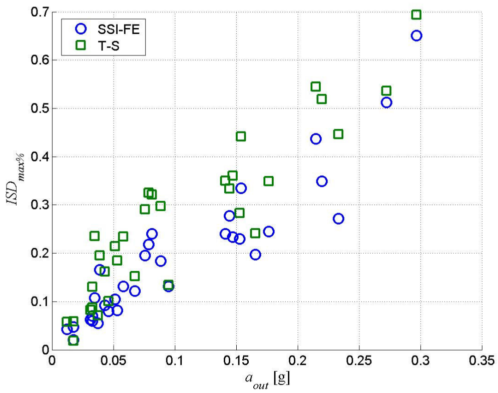



(b)

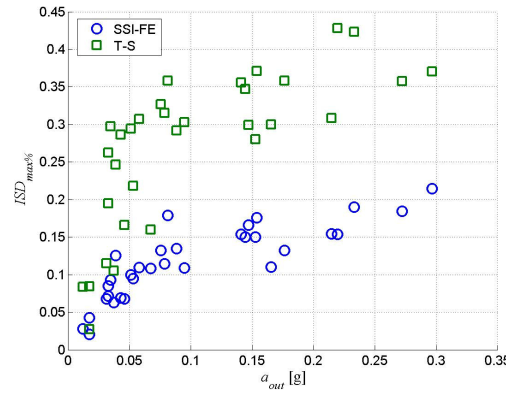
Figure 4 Non-linear 1D soil behavior for the selected earthquakes

## 4. EFFECT OF SSI ON THE DYNAMIC RESPONSE

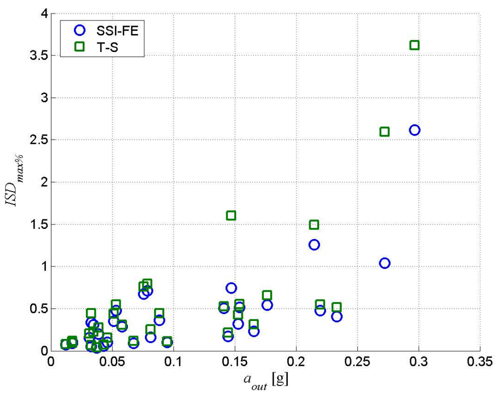
In order to assess the influence of the SSI effects on the dynamic non-linear structural response, the results of the two approaches for each building and for each soil are shown in Figures 5 and 6. The Figure 5 shows the results in terms of the maximum computed inter-story drift ($ISD_{max\%}$) normalized by the inter-story height and the outcropping acceleration amplitude for each selected motion ($a_{out}$).
According to Figure 5a for dry soil, including non-linear SSI effects in the dynamic computation reduces the obtained maximum inter-story drift. The difference between the obtained response following the SSI-FE and the T-S approach is relatively constant over the acceleration amplitude of the input motion. For the saturated case (Figure 5b), the difference between both approaches is larger. This difference can be related to deformations induced during the shaking in the soil. The imposed shear stress is approximately the same for both soils for the same motion, but the degree of confinement for the saturated case is less than the dry one, thus the induced strains are larger and the obtained damping increases. If the energy dissipation in the soil grows, the seismic demand of superstructure diminishes.
The Figures 5c and 5d show the same comparison for the b02 building. As it can be noticed, the effects of the SSI are less significant than for the b01 building. In this case, the fundamental period of the superstructure is larger than the fundamental period of the soil ($T_0$>$T_{soil}$). According to our experience, in this condition the SSI effects can be generally neglected. However, some larger variations appear for a set of particular motions. These cases correspond to a resonance between the fixed base superstructure and the motion for the T-S approach. In these particular cases, the non-linear SSI interaction modifies the fundamental frequency of the system reducing this phenomenon.



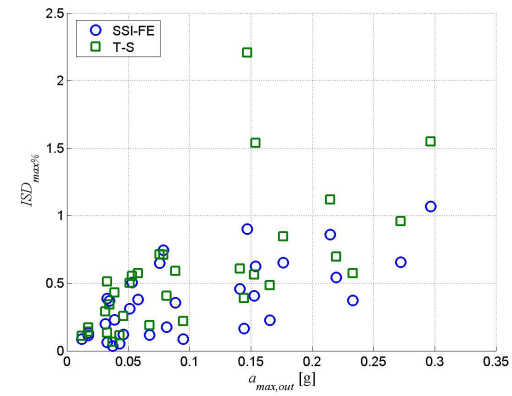

(a) b01 building on dry soil




(b) b01 building on saturated soil




(c) b02 building on dry soil




(d) b02 building on saturated soil

Figure 5 Maximum interstory drift $ISD_{max\%}$

## 4. ENERGETIC ANALYSIS OF RESULTS

In order to identify the role of the different energy dissipation mechanisms in the problem and asses the effects of the non-linear SSI, two energy dissipation indicators can be computed. For the superstructure, the material non-linearity is concentrated in plastic hinges. Thus, an indicator of the amount of energy dissipated on the superstructure $I_{str}$ can be defined as:

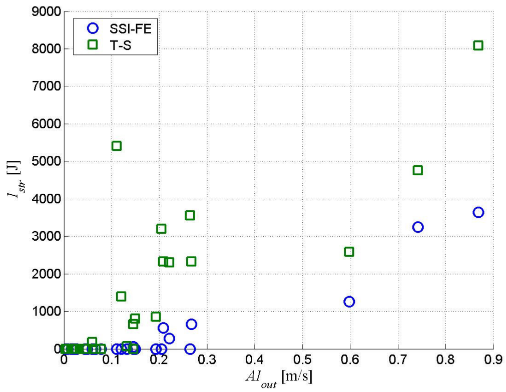
$$I_{str}=\sum_k \int_t M_p^k(t)\,d\vartheta_p^k(t) \qquad [J] \qquad (4.1)$$

where $M_p^k$ is the bending moment in plastic component and $\vartheta_p^k$ the corresponding hinge rotation. The superscript $k$ corresponds to potential plastic hinge $k$ of the superstructure. For the soil, a normalized energy dissipation index can be computed by:

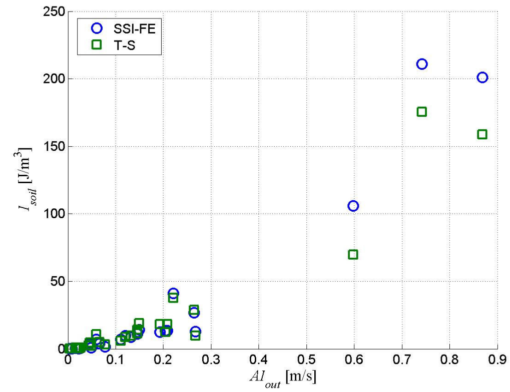
$$I_{soil}=\frac{1}{\Omega}\int_\Omega\int_t \tau(t)\,d\gamma(t)\,dV \qquad [J/m^3] \qquad (4.2)$$

where $\tau$ and $\gamma$ are the shear stress and strain induced in the soil during the dynamic loading. This integration is performed over a control volume $\Omega$ defined by two times the characteristic length $a$ of the foundation in horizontal and vertical direction, thus $\Omega = l_a\, a^2$ (darker zone in Figure 3). This portion of the surrounding soil approximately concentrates the non-linear effects due to SSI (Saez et al., 2007). Figure 7 show the obtained values for the b01 building on dry soil (a and b) and for the b02 building on saturated soil (c and d) in terms of the Arias Intensity imposed at outcropping, thus a measure of the energy of the input motion ($AI_{out}$).
As expected, the $I_{str}$ agree with the computed values of inter-story drift. Thus, a reduction of the obtained $ISD_{max\%}$ corresponds to a reduction of the dissipated energy in the structure. According to Figure 7b and 7d, when the SSI effects are included in the computation, the amount of the energy dissipated by the soil

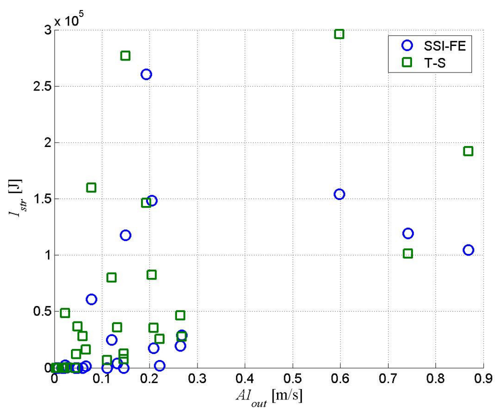
(measured by $I_{soil}$) increases compared to free field case. This increment of energy dissipated by the soil by material damping explains partially the general tendency of reduction of the structural demand when non-linear SSI effects are included. Other aspects such as radiation damping and frequency content of the motion compared to the soil and the superstructure also play a key role on the structural response.



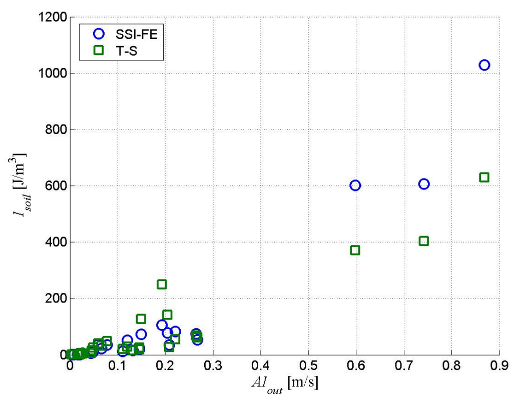

(a) $I_{str}$, b01 building on dry soil




(b) $I_{soil}$, b01 building on dry soil




(c) $I_{str}$, b02 building on saturated soil




(d) $I_{soil}$, b02 building on saturated soil

Figure 7 Energy dissipation index

## 5. CONCLUSIONS

The influence of the inelastic behaviour of soil deposit on the soil-structure interaction effects has been highlighted in this work. The main conclusion of this study is that the soil-structure interaction with a non-linear soil model can vary significantly the response of the structure compared to that obtained with fixed base condition. Then, the simple procedures specified in design codes are not sufficient to asses properly the local soil influence on the structural response.
The major challenge to quantify the non-linear SSI effects in seismic demand evaluation is to predict an accurate global damping taking into account several parameters related to SSI phenomena that can be used to improve standard fixed-base computations. The purpose of this paper was to highlight the effects of material non-linear behavior of the soil in the problem by introducing a set of energy measures. The next step is to use these measures as correction variables over traditional-fixed base computations to avoid performing costly sophisticated non-linear SSI computations.

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