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Influence of liquefaction and SFSI on structural responses

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ABSTRACT: In current practice, the numerical modelling of the dynamic behaviour of structures under seismic loads, typically do not consider the influence of soil-foundation-structure interaction (SFSI). If the influence of the soil is considered at all, substructure approach is often used, i.e. the problem is divided in smaller systems and individual results are combined to obtain the global solution. This technique required an assumption of linear behaviour for each of the subsystems. However, this approach can often lead to inaccurate results, especially when soil nonlinearity is involved. One particular way in which nonlinear behaviour of the soil can arise is through liquefaction. The objective of this work is to reveal the simultaneous influence of non-linear SFSI and soil liquefaction on the structural responses. Two approaches were considered. The first approach adopted a two-step model in which a 1D soil column was modelled, and then the acceleration obtained at free field was used to obtain the response spectra. The second method considered a 2D soil-foundation-structure system. In both cases, dry and saturated soil conditions were investigated. The impact of SFSI and soil liquefaction on structural behaviour is discussed.

1 INTRODUCTION

In past decades the destructive potential of soil liquefaction associated with strong earthquakes has been widely documented. Damage related to soil liquefaction had been observed, e.g. in the 2010 M8.8 Maule earthquake in Chile (Elnashai et al. 2010), and the 2011 M6.3 Christchurch earthquake in New Zealand (Cubrinovski, et al.2011). However, most design codes simply indicate the complexity of liquefaction and possible settlements that can affect the structural foundations. There are no specifications providing guidance on how the impact of soil liquefaction can be incorporated in the design spectra, also, no additional design parameters are provided.

Design codes worldwide take different approaches to deal with soil liquefaction. E.g., Based on Liao et al. (1988), ATC40 (1996) indicates that below M_w=6.5, soil liquefaction is considered to be unlikely (contradicting evidence presented by Cubrinovski, et al.2011). In cases where there is the likelihood of soil liquefaction, provisions indicate that a geotechnical engineer needs to determine the liquefaction susceptibility and evaluate the extent of possible settlement. Adopting a similar approach, FEMA450 (2004) requires a geotechnical report to evaluate the potential consequences of liquefaction. The main requirement is to accommodate displacements, forces, and their combination due to liquefaction. FEMA350 (2000) incorporates liquefaction as “other hazards” and only limits its implications to an estimation of permanent ground deformation.

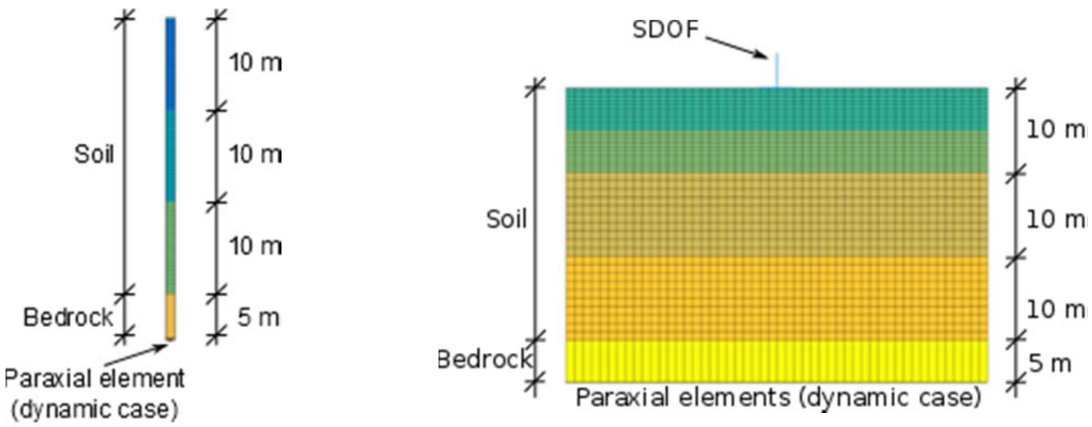
On the other hand, the influence of an existing building on the liquefaction potential was studied by Kyle and Seed (1990). They revealed that the excess pore water pressure close to a building foundation can be significantly different from values measured on free field. Their findings were supported by a numerical study developed by Lopez-Caballero and Modaressi (2008). Additionally, Youd and Carter (2005) revealed that the moment when soil liquefaction was triggered affects the acceleration recorded on the surface significantly.

All those findings support the idea that liquefaction needs to be considered as an integral part of the design process, instead of an isolated hazard represented by a fixed settlement. Based on the discrepancy between the state of art, and current design specifications, more research is needed to improve design procedures that consider the effect of liquefaction and non-linear soil behaviour.

2 METHOD

2.1 Modelling approaches

Two modelling approaches were considered. Firstly a two-step model was studied. This approach considered a 1D soil column of 30 m depth (Figure 1 a)). Then the acceleration recorded on ground surface (free field) was used to obtain the response spectra. Secondly, a 2D soil-foundation-structure system was considered. In this case, a single degree of freedom (SDOF) model was placed at the top of a 30 m deep and 50 m wide soil profile (Figure 1 b)). Four different natural frequencies for the SDOF model at the top of the soil were studied. The dimensions of the 2D model allow free field condition close to the edges. Paraxial elements (1987) were used in both cases at the base of the soil profile. Those elements were introduced to incorporate the incident wave and, at the same time, satisfy the radiation condition.



a) 1D soil column (two-step approach) b) 2D Complete model

Figure 1. FEM models

The software GEFdyn (1996) was used in this research. A plane-strain approach was considered. The soil was modelled by 4 nodes linear elements and the SDOF structure by frame elements.

Since local site effects were directly included in the numerical model, thus ground motions recorded on bedrock was used as the input motion. For all the analyses the record of Gilroy station, obtained in the 1989 Loma Prieta Earthquake was considered. The maximum amplitude was scaled to 0.18 g to avoid numerical instability problems. A total duration of 10 seconds of the main shock was considered (Figure 2).

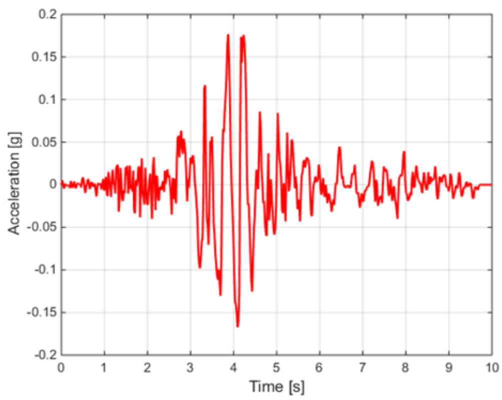


Figure 2. Ground motion (Loma Prieta, 1989)

2.2 Soil model

The ECP's (Ecole Centrale de Paris) elasto-plastic multi-mechanism model (Aubry 1982 & Hujeux 1985) was used to represent the soil behaviour. The variables considered were the displacement of the solid phase (u_s) and the pore water pressure (p) based on a couple $u_s - p$ formulation (Zienkiewicz & Shiomi, 1984). This formulation consists of neglecting the relative acceleration between the fluid and the solid phase. Soil grain compressibility and thermal effects were also neglected. The parameters considered for the soil were adopted from Lopez-Caballero and Modaressi (2008). Table 1 shows some of the selected parameters for each soil layers.

Table 1 Main soil parameters for different layers

Parameter	0 – 10 m	10 – 20 m	20 – 30 m
Bulk reference modulus, K_{ref} *(MPa)*	628.0	444.0	444.0
Shear reference modulus, G_{ref} *(MPa)*	290.0	222.4	222.4
Exponent of the elastic law (n_e)	0.50	0.40	0.40
Friction angle $(\phi'_{pp}\ °)$	30	31	31
Plasticity compression modulus (β)	33	43	43
Dilatancy angle $(\psi\ °)$	30	31	31
Reference pressure $(p'_{co}$ *MPa)*	0.04	1.80	1.80

Beneath the bottom soil layer (30 m) a 5 m thick layer of elastic bedrock was assumed.

3 RESULTS

3.1 Two-step approach

Dry sand and saturated conditions were studied. In both cases non-linear analyses were performed. Additionally a linear study was conducted. To achieve linear behaviour the input was multiplied by 0.0001. Then, the results obtained were amplified multiplying by 10000.

3.1.1 *Dry sand*

A previous static initialization of the soil profile of Table 1 was conducted considering a layer construction of the soil. The obtained stress state was assumed as the initial state of the dynamic analysis (the deformation from the static stage was neglected). The dynamic analysis corresponds to a perturbation around the static equilibrium neglecting the gravity force. Figure 3 shows a vertical stress profile during the static initialization.

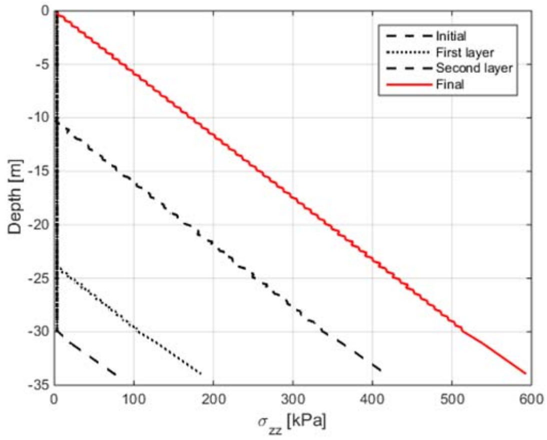


Figure 3. Static initialization

The fundamental frequency of the soil column was obtained as the ratio of the acceleration at the top of the column (a_{top}), to the acceleration at the base of the column (a_{bot}) in the frequency domain. The base of the column corresponds to the interface between bedrock and soil. A decrease in the fundamental frequency of the soil column can be seen from Figure 4 when non-linear behaviour was considered.

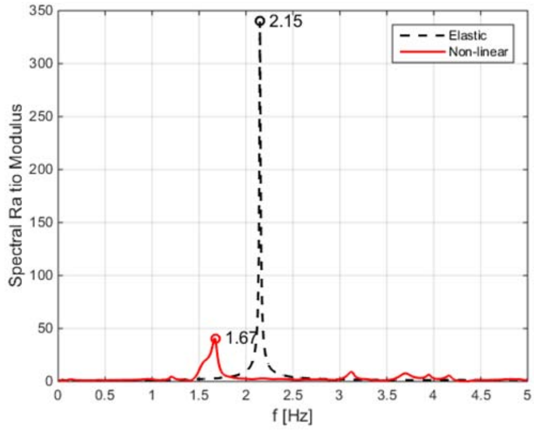
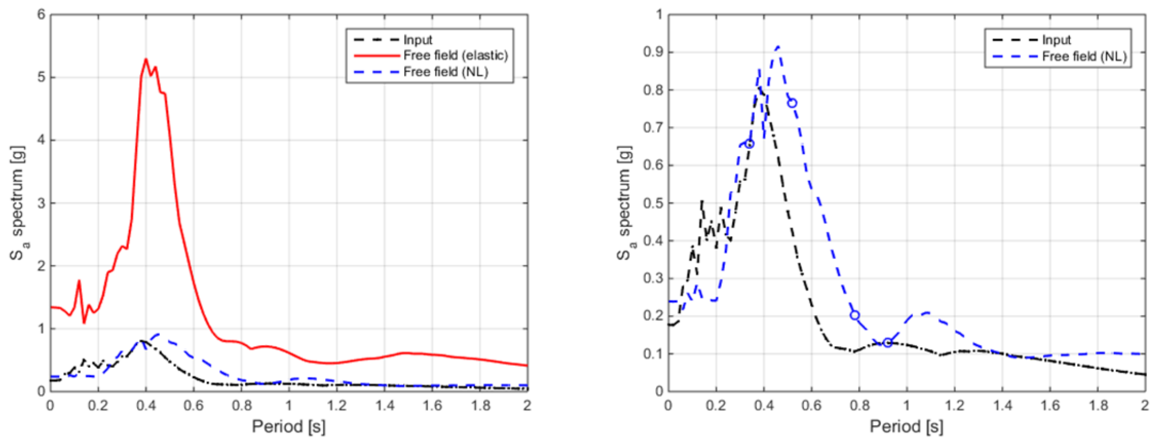


Figure 4. Fundamental frequency

Finally, the response spectrum was obtained for the acceleration at the ground surface. To compare, the response spectrum for the input was also calculated (see Figure 5).



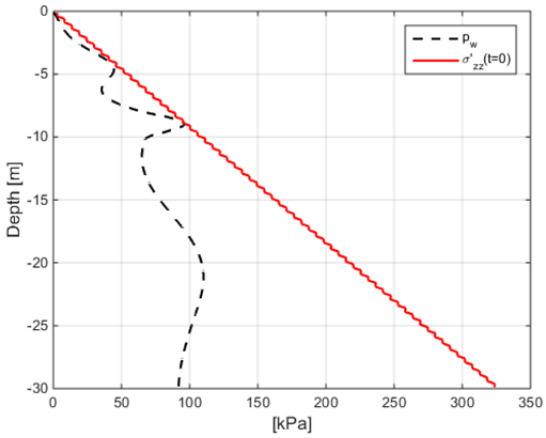
a) Elastic spectrum b) Input and non-linear spectrum

Figure 5. Response spectra for dry soil

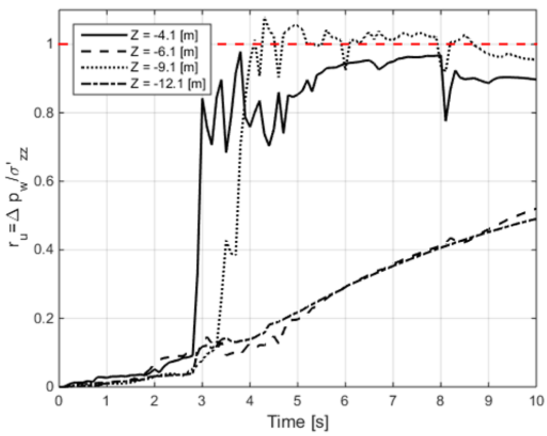
Some aspects can be highlighted from Figure 5. Firstly, The spectrum considering elastic soil behaviour shows values five times larger than those for non-linear soil (Figure 5 a)). This strongly suggests that a linear assumption is overly conservative, when compared with the values in the non-linear case. When non-linear behaviour is considered, an amplification of the response can be seen (Figure 5 b)). Also a variation in the frequency content expressed through a shift of the amplification zone can be observed (Figure 5 b)).

3.1.2 *Saturated sand*

In the case of saturated sand, the water table was considered at surface level. Figure 6 shows the pore water pressure profile obtained for the 1D soil column. Figure 6 (a) shows the increment of the pore water pressure over the hydrostatic initial state (Δp) at the end of the analysis (t=10 s). The obtained profile is compared with the initial vertical effective stress. Figure 6 (b) shows the water pressure time history at four control points of depths 4.1 m, 6.1 m, 9.1 m, and 12.1 m, respectively.



a) Increment of pore water pressure at t= 10 s



b) Ratio r_u on time for control points

Figure 6. Pore water pressure builds up

The value presented on Figure 6(b) corresponds to the ratio r_u , which is the ratio of the pore pressure build-up to the initial vertical effective stress at the same depth (1). Therefore, a value of r_u close to 1 indicates that liquefaction was triggered.

$$r_u(x,t) = \frac{\Delta p(x,y,t)}{\sigma'(x,y,t=0)} \quad (1)$$

Where:

- $\Delta p(x,y,t)$: Corresponds to the pore pressure build-up over the initial static pore water pressure on time *t*, for a given point in the model defined by the coordinates (x, y).
- $\sigma'(x,y,t=0)$: Corresponds to the initial vertical stress (*t = 0*) for a given point in the model defined by the coordinates (x, y).

Figure 7 shows the response spectrum of surface ground acceleration for saturated sand. A reduction in the response at lower periods, and amplification at periods larger than 0.6 s can be observed.

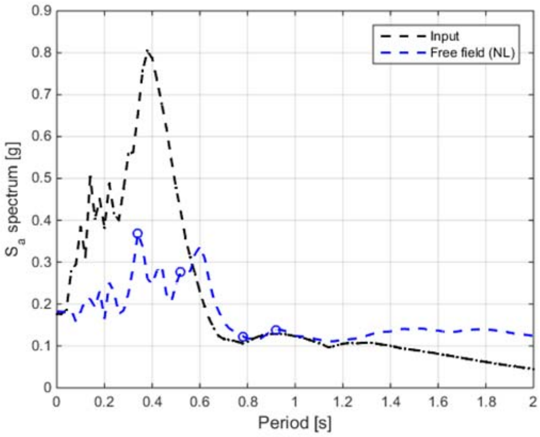


Figure 7. Response spectrum for saturated soil

3.2 2D soil-foundation-structure system

Four SDOF structures were considered. The structures are referred to as SDOF 1 to 4 with natural frequencies of 1.09 Hz, 1.28 Hz, 1.92 Hz, and 2.93 Hz respectively. Each structure had a height of 4 m and a lumped mass of 10 ton at the top. The base was 4 m long. Base and column were modelled using frame elements. No relative displacement between foundation and soil was allowed.

3.2.1 *Dry sand*

Table 2 shows the maximum acceleration for each SDOF structure studied. Values are compared with the response spectrum obtained from the two-step approach.

Table 2 Maximum acceleration at the top of the SDOF (dry sand)

		Maximum acceleration (g)		
	Fixed base f_{str}. (Hz)	2D model	1D model Non-linear Spectrum	Amplification (%)
SDOF 1	1.09	0.158	0.130	21.5
SDOF 2	1.28	0.196	0.204	-3.92
SDOF 3	1.92	0.735	0.765	-3.92
SDOF 4	2.92	0.743	0.657	13.1

The amplification is defined as the ratio of the difference between the response for the 2D case and the 1D case to the value for the 1D model (2).

$$Amplification(\%) = \frac{\text{Max. acceleration (2D)} - \text{Max. acceleration (1D)}}{\text{Max. acceleration (1D)}} * 100 \tag{2}$$

In the case of dry sand, structures with the longest and shortest natural frequencies show considerable differences between the two approaches. For the structures with intermediate frequencies, i.e. SDOF 2 and SDOF 3, the 1D approach appears to adequately reproduce the acceleration at the surface.

3.2.2 *Saturated sand*

Table 3 presents the maximum acceleration obtained for each SDOF structure for saturated sand. The maximum acceleration is compare with the corresponding value of the response spectrum obtained from the two-step model.

Table 3 Maximum acceleration at the top of the SDOF (saturated sand)

		Maximum acceleration (g)		
	Fixed base f_{str} (Hz)	2D model	1D model Non-linear Spectrum	Amplification (%)
SDOF 1	1.09	0.142	0.138	2.90
SDOF 2	1.28	0.114	0.123	-7.32
SDOF 3	1.92	0.228	0.277	-17.7
SDOF 4	2.92	0.382	0.369	3.52

Compared with dry sand, the lowest frequency structure (i.e. SDOF 1 of 1.09 Hz) shows a larger response (for 1D model) when liquefaction occurs. For SDOF structures 2 to 4, however, the response for the saturated case is lower that the dry case.

4 CONCLUSIONS

Two modelling approaches were used to analyse the simultaneous effect of SFSI and soil liquefaction. Firstly, a two-step approach that considered a 1D soil column, in which the acceleration obtained at the surface was applied to an SDOF structure with fixed base assumption. The second model considered a 2D soil-foundation-structure system. Structures with different fundamental frequencies were studied to represent a wide range of structures.

The main findings of this study are:

- A linear assumption for the soil appears to be extremely conservative when representing soil behaviour under a ground motion. The amplification with respect to the input is very large, with almost no change in the frequency content.
- In the case of dry sand, the 1D approach is able to produce a good approximation of the acceleration on surface. This is especially the case for the studied structures of an intermediate natural frequency.
- Structures with a low natural frequency seem to be more affected by liquefaction.
- An amplification zone for structures of higher periods can be observed when the 1D approach is used to study saturated sand. This amplification is not reproduced when the 2D approach was considered.

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