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Seismic performance of a RC building founded on soft stratified soils

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Abstract

The seismic performance of a structure is determined by a series of factors that are related with the earthquake source characteristics, site effects and structural capacity, among others. The state of the art of seismic vulnerability assessment methods, using probabilistic approach, are capable of incorporate the effect of some of these factors. Within the epistemic uncertainties, the soil effects have an important contribution as soil profile characteristics have a major influence on spectral amplification results. This is more evident in case of the softer soil types. Eurocode 8 specifications for seismic design of structures founded on very soft (stratified) soils, classified as S1, requires special studies to define the seismic action. In this work the seismic performance of a 9 story (7 stories and 2 underground stories) RC structure, founded on an S1 type soil, located in Lisbon, Portugal is evaluate by means of nonlinear static results (N2 method). Global performance is evaluated and damage limitation control, in terms of interstorey drifts according to Eurocode 8, is verified in the central members for the different soil profiles.

Key words: Local site effects, equivalent linear analysis, DeepSoil, seismic assessment, capacity curves

1 Introduction

Soil conditions at a location can substantially modify the intensity of soil movements caused by earthquakes, through the succession of soil layers towards the surface. Therefore, it is important to understand how soil properties modify the behaviour of the structure above. Such modifications in the ground motion that alter between the bedrock and the surface are termed as local site effects. These are responsible for the amplification of seismic waves which is greater in cases of wave propagation in soft soils (S1) [1]. This phenomenon is mainly due to the great discontinuities that these soil profiles may present, resulting in stratification and different properties that lead to high degree of seismic amplification. Eurocode 8 [2] does not establish guidelines to be followed in the seismic design of structures founded on type S1 soils (soft stratified soils), instead stipulates that a special study should be carried out to establish the dependence of the response spectrum on the thickness and shear wave velocity value, stratification of clays or soft silts and on the stiffness contrast between this layer and the underlying materials.

The response spectra so derived can be further used for the seismic design and assessment. According to EC8-3, the nonlinear pushover procedure, namely the N2 method developed by Fajfar [3], maybe used to verify the structural performance of the newly designed and of the existing buildings. When a building presents plan-asymmetry the original N2 method is unable to capture the torsional effects, thus distorting the structural response. The results can be corrected using the modified N2 method [4] by multiplying the target displacements obtained from the pushover analyses by an amplification factor that results from an elastic modal analysis. In this work preliminary results are presented for the seismic performance assessment of a 9 story (7 stories and 2 underground stories) RC structure, founded on an S1 type soil, located in Lisbon, Portugal trough the (original) N2 method. Global performance is evaluated and damage limitation control, in terms of interstorey drifts according to Eurocode 8, is verified in the central members for the different soil profiles.

2 Description of the structure and numerical modelling approach

The case study is a residential reinforced concrete nine story (7 upper stories and 2 basement) building located in Lisbon, founded on soil type S1 and class of importance. All upper story floors have the same geometry, same element dimension and same reinforcement features, except for the top floor and the roof floor, which have smaller dimensions. The height of the regular floors is 3.0 m and basement floors' height ranges from and 2.67 m to 3.85 m. The general layout of a typical floor is illustrated in Fig. 1a. The reinforced concrete superstructure consists of columns, beams, slabs, with a centrally located elevator core and additional shear walls, and lateral retaining walls in the basement. The cross sections of vertical structural elements (columns and walls) have different dimensions and reinforcement in the plan of the building. The beam sections range from 0.30x0.45m to 0.30x1.70m. The slabs are 0.22 m thick and the thickness of the shear walls is 0.20m. More detailed information is available by request from the authors. A 3D numerical model of the structures was created using Seismostruct program [5], Fig. 1 b):

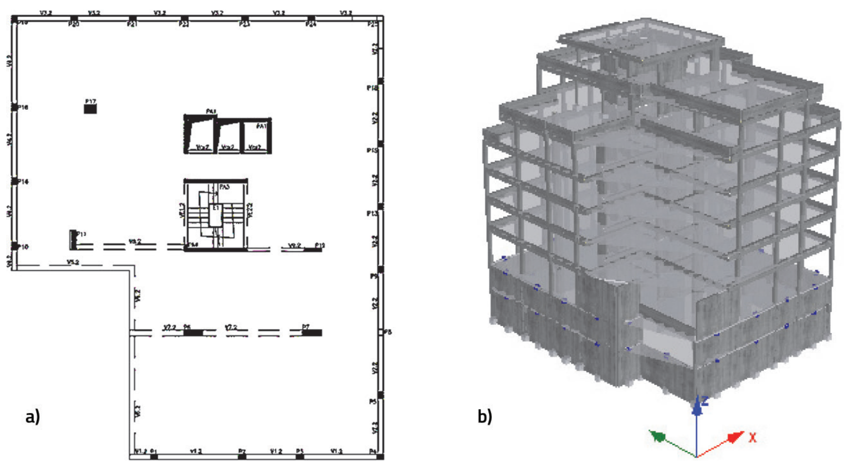

Figure 1. a) Typical floor plan; b) Seismostruct 3D model of the structure

The structural elements, namely beams, columns and shear walls, were modelled using the inelastic plastic hinge frame element, called *infrmFBPH*, with the non-linearity concentrated within a fixed length of the element (plastic hinge): The *infrmFBPH* has a force-based formulation idealized for fiber cross-sections. In total, 150 fibers were defined for the simpler sections and 200 for the more complex ones. Plastic-hinge length (Lp/L) was chosen as 16.67 %. The slabs were considered as rigid diaphragms and the weight of the slabs was distributed along the beams according to the respective tributary area.

To model the behaviour of the soil surrounding the retaining walls, distributed springs were considered. The soil springs are assumed to exhibit a linear-elastic load-displacement relationship (defined by stiffness K_H) regardless of how much they are compressed. The stiffness [kN/m] is given by equation (1) according to Bohnhoff, [6], where z [m] is the thickness of a soil layer that is represented by the soil spring, [m/s] is the maximum shear wave velocity of the soil, [ton/m^3] is the soil density and the Poisson's ratio.

$$K_H = 1(1+v)\rho z V^2_{smax} \tag{1}$$

The concrete is represented by a uniaxial model proposed by Mander et al. [7] and the cyclic rules proposed by Martinez-Rueda and Elnashai [8]. The constitutive model proposed by Menegotto and Pinto [9] was used to represent the steel behaviour coupled with the isotropic hardening rules proposed by Filippou et al. [10]. The materials used for reinforced concrete elements were C25/30 concrete and A500 NR steel, with the properties listed in Table 1.

Table 1. Steel and concrete properties

A500	f_{yd}	ε_{yd}	C25/30	f_{ck}	f_{cd}	f_{ctm}	E_{cm}
	435 MPa	2.18 10^{-3}		25 MPa	16.7 MPa	2.6 MPa	30 GPa

Linear elastic analysis was conducted to obtain the dynamic characteristics (natural frequencies and vibration modes) of the structure. Three modes of vibration with a more relevant modal participation were identified, the first corresponding to a mode in the Y direction and two corresponding to modes with significant participation in the X direction, with periods of 1.01s , 0.68s and 0.55s, respectively. A significant influence of the torsion, due to the asymmetry property of the structure affecting mainly the 1st mode in the X direction is verified. As a consequence, according to EC8 [2] classification criterion the structure is classified as a torsionally flexible system in X direction.

3 Seismic assessment

The global seismic capacity of the structure is assessed by the N2 method prescribed by the EC8 [2]. The ground type S1, used in this study, can produce anomalous effects of amplification of the local seismic movement and interaction between the soil and the structure. Being so, EC 8 does not propose a design response spectra and states that particular attention should be paid in this case. Taking into account these particular soil conditions, a special study is carried out to define the seismic action.

3.1 Seismic action

To conduct the analysis, two soil profiles were chosen for comparison using ground types D and S1 that matches the required specifications of EC8 [2]. Profile 1 is a homogenous soil layer of ground type D of medium cohesionless soil and profile 2 consists of two layers: the top layer of the same type of soil of profile 1 and a second layer of ground type S1 of a soft clay with high plasticity index. A profile 2b) is taken into consideration in this analysis, quite similar to profile 2, being the only difference a new value of the plasticity index of the clay layer. In both examples it was specified a value for the bedrock damping ratio but has negligible effect on the results. Both profiles along with the properties and parameters used in the analysys for each layer can be seen in Fig. 2. The shear wave velocities (Vs) and plastic indexes (PI) are taken according to EC 8 values for each soil type. The other soil parameters are average values taken from literature and experience.

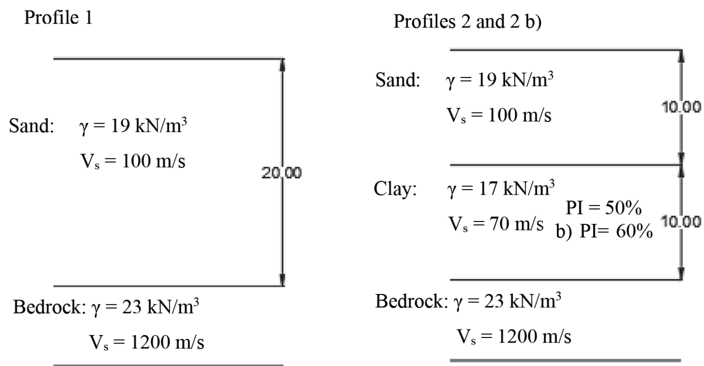

Figure 2. Soil profiles

The soils are modelled in DeepSoil [11] that uses an equivalent linear methodology in the frequency domain. The bedrock is defined as an elastic half space and the sand and clay layers are modelled with an equivalent linear model that employs an iterative procedure in the selection of the shear modulus, (G/Gmax) and damping ratio (%) soil properties defined as functions of shear strain (%): These properties are defined by discrete points model of the curves suggested by Seed & Idriss (1970) [12], for the sand, and Vucetic & Dobry (1991) [13], for the clay to take into account the PI index. The iterative procedure quantifies the stiffness degradation and damping increase of the soils with the increase of the soil deformation.

For the ground motion at the bedrock, a set of 8 ground motions was selected from ESM - (PEER' s) [14], and NGA – West 2 [15] databases website, using SeismoSelect [16] software. The following criteria was used: magnitude between around 5.5, source (epicenter distance < 40 km) and shear wave velocities higher than 800 m/s (bedrock): As each ground motion has accelerograms in two directions, a total of 16 records were used. This is in accordance with type 2 earthquake ground motion of EC 8.

All selected ground motions were fitted to the Eurocode 8 elastic design spectrum with the Portuguese code feature for type 2 soil A (η = 1, S = 1, β_0 = 0.05, T_B = 0.1 sec, T_C = 0.25 sec, T_D = 2 sec) using the software SeismoMatch [17]. It was considered the site in Lisbon with the value of the reference peak ground acceleration, a_{gR} = 1.7g . Fig. 3 presents the original ground motions spectrum and the adjusted ones. The red line is the EC 8 elastic design spectrum type 2 for ground type A.

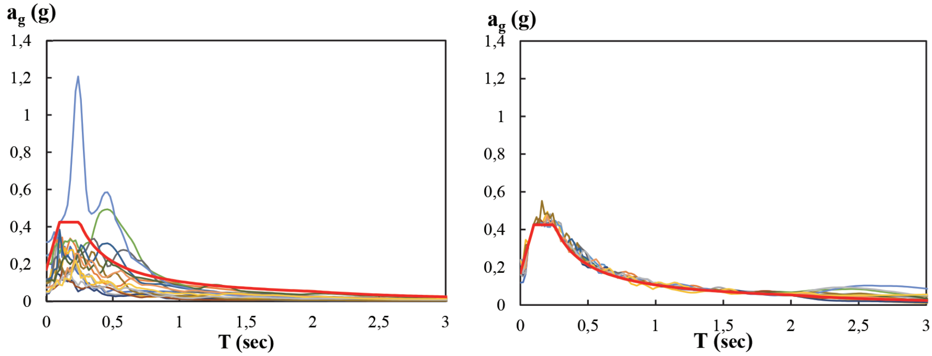

Figure 3. Average response spectrum of the original (left) and adjusted ground motions (right)

Then it is performed the site response analysis with DeepSoil [11] software where the ground input motions are filtered by each layer of each profile in order to obtain the average response spectrum at the surface. In Fig. 4 are presented the average response spectrum for each profile, as well as the red line response spectrum reference A that represents the average unfiltered response spectrum of the ground motions at the bed-rock.

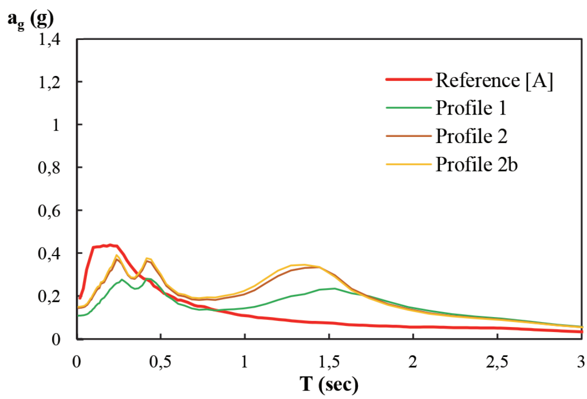

Figure 4. Response spectra of each soil profile at the surface and reference spectrum at the bedrock

It can be seen that profile 2 with the S1 soil type induces higher values of spectral accelerations at the than profile 1. Nevertheless, both profiles mitigate accelerations for low periods but for periods above 0,8 s there is considerable amplification. The average response spectrum of each set of ground motions is used to compute the nonlinear static procedures response.

3.2 Structural analysis

3D pushover analyses of the structure were carried out to evaluate the seismic capacity of the structure considering the variation in the soil profiles. According to N2 method at least two different load pattern should be considered. Here the proportional to fundamental mode shape and proportional to mass distribution over the height was considered. The loads were applied independently in the two horizontal directions. The resulting capacity curves are shown in Fig. 5 for modal and mass proportional load distributions for the X and Y directions.

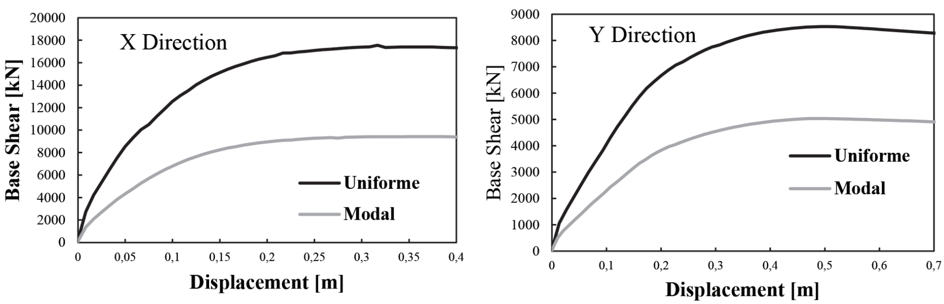

Figure 5. Capacity Curves of the structure

In both directions the two curves for the modal proportional load pattern presents lower values of base shear than the uniform load pattern curve for the same top displacement. However in the elastic range are equal. The structure presents more strength along X direction due to the presence of the shear walls and elongated columns developing in along this direction.

4 Analysis results

The seismic assessment of the building is done by comparing the capacity and the corresponding demand. For the calculation of the target displacements the earthquake spectra presented in Fig. 4 were used. The results obtained for the considered soil profiles are compared with ones obtained for soil type A in order to better understand how site effects condition the structural response. The seismic demand of the equivalent SDOF system is calculated using the EC8 graphical procedure for each considered soil profile. The target displacement for each response spectrum and direction is given on Table 2, where "Reference (A)" represents the results obtained for the mean response spectrum of the ground motions scaled to match the response spectrum for a soil type A.

Table 2. Roof target displacement for each response spectrum for the X and Y directions

	Direction X			
	Reference (A)	**Profile 1**	**Profile 2**	**Profile 2b**
Target Displacement [m]	0.034	0.031	0.041	0.043
	Direction Y			
	Reference (A)	**Profile 1**	**Profile 2**	**Profile 2b**
	0.070	0.226	0.261	0.251

The target displacements obtained for the two soil profiles show that the site effect have a significant influence on the capacity of the structure with more expression in the Y direction with the lower strength.

Additionally, damage limitation control in terms of interstorey drifts according to Eurocode 8 is verified in the central members, where the torsional effects are not significant. Fig. 6 represents the lateral displacement profiles of the mass center of the structure for the three soil profiles and for the soil type A, Reference (A): It can be observed that the results in the X direction are less sensitive to the variation of the soil type, while direction Y show a great dependence on the site conditions and consequently feature considerable amplification of the displacements. In both directions soil Profile 2 is more demanding.

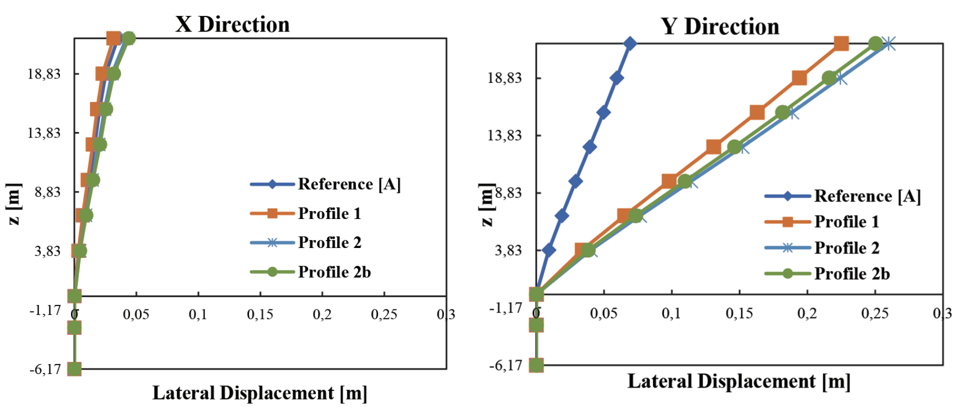

Figure 6. Lateral displacement profiles of mass center of the building

In Fig. 7 the interstory drifts are presented along with the EC8 drift limit (in red): One can be observed that in both directions the structure complies with damage limitation defined according to EC8, at the DL. However, further study is required in order to better understand the behaviour of the structure.

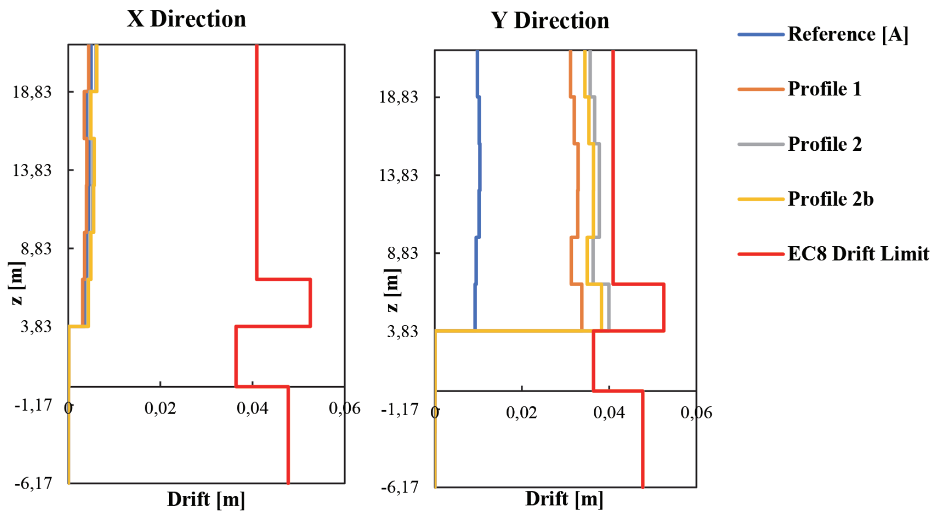

Figure 7. Drifts of each storey mass center

5 Conclusions

This study showed the importance of considering soil conditions in the seismic assessment of buildings. Especially soft soils and stratified can modify the intensity of ground motions through the succession of soil layers towards the surface. Soils classified as S1 in EC 8 were studied in this work as they meet those characteristics. The stiffness degradation and damping increase with the increase of the soil deformation were quantified by using a linear equivalent method in the frequency domain. For the soil profile adopted and for the ground motion type considered it was clear that, for periods above 0,8s, soil type S1 induced higher values of spectral accelerations.
The original N2 method was used for the seismic assessment of the structure, in terms of the global and interstorey drift limit checks. The results show that they comply with the global and damage limitation levels established in the Portuguese National Annex of the EC8-3, but much higher values of story drifts and target displacements are obtained for the studied soft soil types.

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