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Seismic response and soil-structure dynamic interaction for a large building in Florence

Réponse sismique et interaction dynamique sol-structure pour un grand bâtiment à Florence

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ABSTRACT: The New Law Court complex of Florence, that is at present under construction, consists of some buildings having different heights, maximum 77 meters from g.l., covers a surface of about 30,000 m^2 extended, and has a total volume of 750,000 m^3, about 550,000 m^3 of which is in elevation. Given the importance of the work, numerous geotechnical investigations, both in situ and in laboratory, have been carried out, and studies on specific topics have been developed within the framework of a research program developed through a collaboration with the Municipality of Florence. Among these the study of local seismic response and an analysis of the soil-structure dynamic interaction were carried out, the results of which will be presented in this paper.

RÉSUMÉ: Le nouveau Palais de Justice de Florence, qui est en train d'être construit, est composé de plusieurs bâtiments de différentes hauteurs, jusqu'à 77 m de la surface du sol. Il s'étend sur une surface d'environ 30,000 m^2, il a un volume total de 750,000 m^3, dont environ 550,000 m^3 sont hors de terre. Compte-tenu de l'importance de l'oeuvre, pour son projet définitif beaucoup d'essais géotechniques en place et de laboratoire ont eté exécutées, et des études spécifiques ont été conduites en collaboration avec la Municipalité de Florence. Parmi celles-ci il y a l'étude de la réponse sismique locale et l'analyse de l'interaction dynamique sol-structure. Les résultats de telles études son l'object de cette communication.

1 INTRODUCTION

The New Law Court in Florence, the construction of which is under way, consists of buildings of different heights, up to a maximum of 77 m from g.l. It covers an area of about 30,000 m^2, and has a total volume of 750,000 m^3, approximately 550,000 m^3 of which are above ground level. Figure 1 reproduces a photograph of the plastic model of the building in progress.

The foundation soil is part of a very thick recent alluvial deposit (more than 80 m) consisting mostly of consistent and overconsolidated silts and clays, with local inclusions of pebbles, gravel and calcareous nodules with dimensions that are even centimetric, more frequently in the first 20 m of depth.

Thorough geotechnical studies were carried out (Vannucchi, 1997; 1999) in view of the importance of the work. In particular, the local seismic response in a free field was evaluated, and an analysis of the soil structure dynamic interaction was made. The results of these analyses are discussed synthetically here as follows.

Figure 1. Plastic model photograph of the New Law Court of Florence

2 CHARACTERISTICS OF THE DESIGN MOTION

On the basis of seismic hazard maps for Italy (Peruzza et al., 1996; Rebez et al., 1996), the maximum values for peak acceleration and macroseismic intensity expected in Florence with a return period T=475 years (that is 10% probability of exceedance in fifty years) are the following: PGA=0.2 g and I=VII MCS. The maximum expected value for the peak acceleration refers to the surface of the deposit, since the data base from which it was obtained is macroseismic.

As no accelerometric recordings exist in the examined area, the response spectrum of the expected motion on rock (Figure 2a) was obtained by analysing the recordings of the strongest seismic events that occurred in the past in central Italy. According to the response spectrum thus obtained, an artificial accelerogram was generated using the SIMQKE computer program (Gasparini & Vanmarcke, 1976) and assumed as a design motion on rock in the subsequent local seismic response analysis.

The signal envelope function, that is necessary to simulate the actual form of the expected earthquake, was chosen in a trapezoidal shape, so as to obtain an artificial earthquake that was similar to most of the recorded ones utilised for determining the reference spectrum. The duration of the design motion was assumed at 20 seconds, as suggested in Eurocode 8 (1988) and as result from the mean duration of the accelerograms examined. Figures 2b and 2c show the synthetic time history and the Fourier spectrum of the design motion at the bedrock, chosen so as to have the greatest number of peaks with high acceleration, and scaled so as to produce the peak acceleration PGA=0.2 g at the surface of the deposit.

3 STRATIGRAPHIC SUCCESSION AND GEOTECHNICAL PROPERTIES

Under a shallow layer of filling soil, two main geological formations can be identified, overlying a deep bedrock. The soils of the upper formation, A, have been deposited in a mainly fluvial environment, while the soils of the lower formation, B, were deposited in a lacustrine environment. Formation A generally ex-

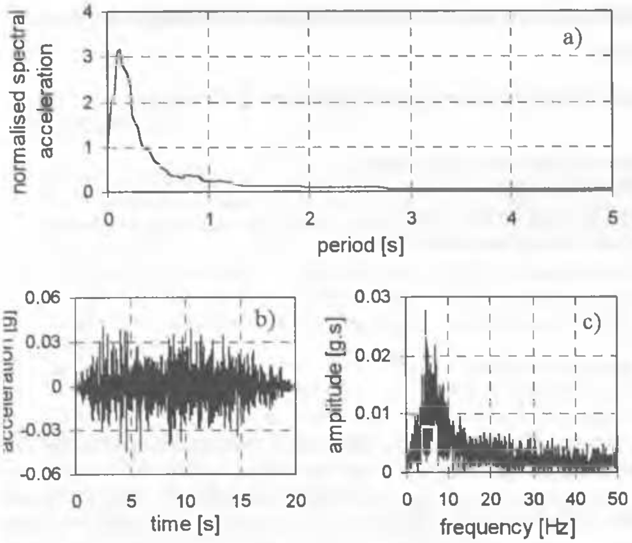

Figure 2. Design motion on the bedrock: a) normalised response spectrum; b) synthetic time history; c) Fourier spectrum

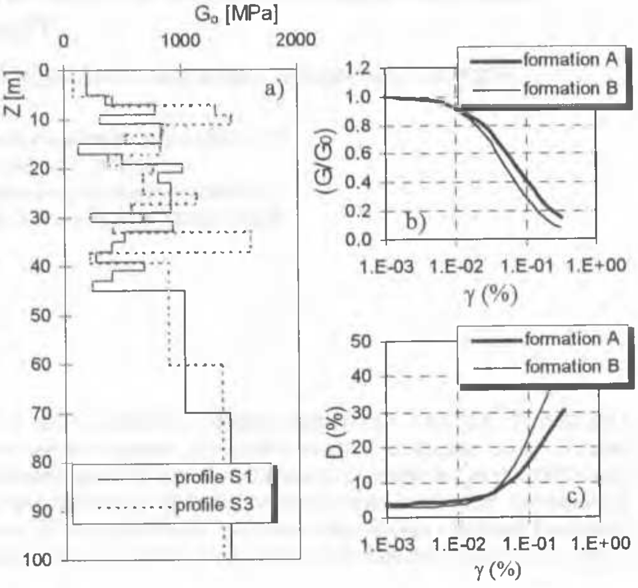

Figure 3. Maximum shear modulus profiles (a), modulus reduction (b) and damping ratio (c) curves for the two formations

tends to a depth of 20-25 m, but the two formations are often in succession. Thus, levels that can be attributed to formation A have been found at a depth of 40 m and levels belonging to formation B have been found at a depth of 10 m.

The geotechnical, physical and mechanical properties of the fine fraction, that are prevalent in any case in both formations, show no differences that can be attributed to the formation to which they belong. Thus, the two formations have not been distinguished for the purposes of the geotechnical modelling. More than 90% in volume consists of overconsolidated silts and clays that are cemented, due to the presence of limestone. Pebbles and gravel are present diffusely and randomly inside the deposit, and in several zones they become the prevailing granulometric fraction. Because of the lithological irregularity, both horizontal and vertical, it is not possible to define stratigraphic sections into details. Moreover, the stratigraphic profiles corresponding to the 26 verticals explored in the area of the New Law Court building (6 geotechnical boreholes, 10 CPT tests, 5 CPTU tests, and 5 DMT tests), are substantially equivalent, even if they are different from each other and cannot be correlated.

The soil is almost fully saturated. The groundwater level has depths ranging from 5.3 m to 3.2 m from ground level.

The following model for the foundation geotechnical design was used: horizontal ground level at altitude 0.00 of the local reference, depth of the groundwater: Z_w = -5 m from ground level; foundation soil that is cohesive, overconsolidated and homogeneous in a horizontal direction, having the following mean geotechnical properties: I_p = 26.5%, w_L = 51.5%, w = 24.2%, c' = 26.6 kPa, ϕ' = 20.7°.

Two down-hole tests at the depths of 47.5 and 39.5 m from g.l. were carried out to measure soil initial properties. The average velocity trend of the shear waves in the two boreholes (S1 and S3) were quite similar, with means values of 552 m/s and 588 m/s, respectively (Figure 3a). In both cases, no marked tendency to increase with the depth was observed. From the maximum depth of the down-hole tests at the bedrock, the values of the shear modulus were estimated by extrapolation by means of a correlation between shear wave velocity and N_{SPT} index (a growth trend was observed in the last 8 metres of depth explored by means the down hole tests).

G/G_0 and D vs. γ average curves were obtained by applying a modified hyperbolic model (Yokota et al., 1981) to the results of resonant column tests performed on 7 undisturbed samples belonging to formation A and formation B (Figures 3b and 3c). The mean values and the standard deviations of the elastic and volumetric threshold strains for all samples were, respectively: $M(\gamma_l) = 2.4\ 10^{-3}$ %, $SD(\gamma_l) = 9.6\ 10^{-4}$ %, and $M(\gamma_v) = 2.0\ 10^{-2}$ %, $SD(\gamma_v) = 1.0\ 10^{-2}$ %.

4 LOCAL SEISMIC RESPONSE

At the examined site, geomorphological, geotechnical and seismic conditions were in good agreement with the assumptions implied in the SHAKE computer program. Therefore it was used for local seismic response analysis. One-dimensional local seismic response analysis in a free field was performed at the two verticals explored by the S1 and S3 boreholes. The results of the down-hole tests were used as far as G_0 (z) was concerned, and those of the resonant-column tests, relative to the $G(\gamma)$ and $D(\gamma)$ laws.

The main ground motion parameters for the two tested profiles obtained at the deposit surface (z = 0.0 m), at the foundation design depth (z = -7.0 m), and on the bedrock (z = -100.0 m) are synthesised in Tables 1 and 2.

Table 1. Representative ground motion parameters, inferred from a local seismic response analysis in a free field on the S1 profile

LEVEL	PGA [g]	PGV [cm/s]	PGS [cm]	v_0 [1/s]	T_0 [s]	I_a [cm/s]	P_D [10^{-4} g s^3]
Surface	0.18	6.86	1.81	12.9	0.22	67.6	4.0
Footing	0.13	5.78	1.34	10.9	0.22	39.2	3.3
Bedrock	0.045	1.61	0.44	25.3	0.20	3.98	0.06

Table 2. Representative ground motion parameters, inferred from a local seismic response analysis in a free field on the S3 profile

LEVEL	PGA [g]	PGV [cm/s]	PGS [cm]	v_0 [1/s]	T_0 [s]	I_a [cm/s]	P_D [10^{-4} g s^3]
Surface	0.20	6.73	1.10	16.6	0.20	88.2	3.2
Footing	0.11	4.61	1.16	12.3	0.20	25.5	1.7
Bedrock	0.045	1.61	0.44	25.2	0.20	3.98	0.06

The absolute values of the peak ground acceleration (PGA) are indicated in the first column: the analysis was performed using deconvolution of the accelerogram, so that the design value PGA=0.2 g, determined using hazard analysis, resulted for the S3 reference profile to which the greatest amplification at the surface is associated. The amplification factor of the peak acceleration at the surface was equal to 4.0 for the S1 profile and to 4.5 for the S3 profile. The absolute values of peak ground velocity (PGV) and displacement (PGS), the "zero crossing" (ν_0), the fundamental period (T_0), the Arias Intensity (Ia) and the earthquake destructiveness potential factor (P_D) values of the accelerogram are indicated in the subsequent columns.

In Figure 4, the elastic response spectrum for a damping of 5% at the foundation depth (-7 m) and at the deposit surface of the S1 profile is shown. It can be observed that the amplification processes involve two maxima in the response spectra: the absolute maximum (which, on the surface, consists of two very close peaks) is set at around 0.2 s, while the local maximum at 0.5 s, and involves the less rigid structures. The structure of the New Law Court building consists of a series of interacting elementary buildings that are characterised by a varying number of floors and, therefore, by different stiffness and mass. If the simplified equation T = 0.1 n, that links the number of floors, n, of a multi-floor, framework building and its fundamental period, is considered valid, it can be affirmed that the lower-lying buildings of the Courthouse complex are involved in this second relative maximum of the response spectrum.

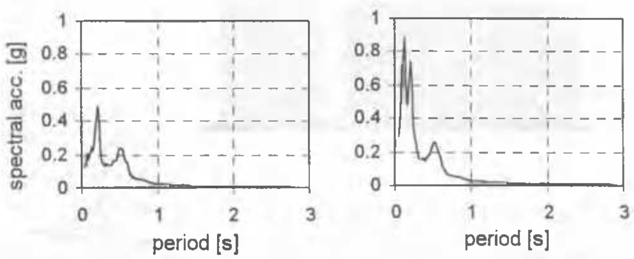

Figure 4. Elastic response spectra at the ground level (right) and at the foundation depth (left) from one-dimensional analysis at S1 profile

The elastic response spectrum was compared with those suggested in EC8. Figure 5 contains a comparison of the response spectrum normalised at the surface for the S1 profile and the spectrum recommended by Eurocode 8 for overconsolidated cohesive soils characterised by stiffness that increases with the depth, and by $V_s > 400$ m/s at a depth of 10 m (A type soil).

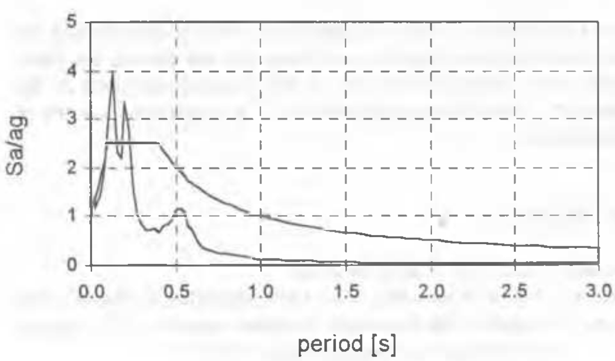

Figure 5. Comparison between the normalised elastic response spectrum at the surface for the S1 profile and the EC8 spectrum for A soil type profile

By applying a 15% reduction, that is generally applied to the maximum of spectral acceleration, the maximum amplification ratio relative to the mean spectrum is equal to 3.5, that is, relatively higher (about 40%) than the value of 2.5 suggested by EC8.

5 SOIL-STRUCTURE DYNAMIC INTERACTION

The soil structure dynamic interaction for the New Law Court building was carried out using the FLUSH-PLUS computer program. The New Law Court is a complex structure which is developed over a relatively extended trapezoidal area. It consists of a complex of buildings differing as to stiffness and load transmitted to the foundation structures. The zones most heavily loaded ($q \approx 320$ kPa) are located in correspondence with the verticals S1 and S3 on which the down-hole tests were performed (Figure 6). To calculate the seismic local response using FLUSH-PLUS, that makes a pseudo three-dimensional analysis, the three significant sections indicated in Figure 6 were selected.

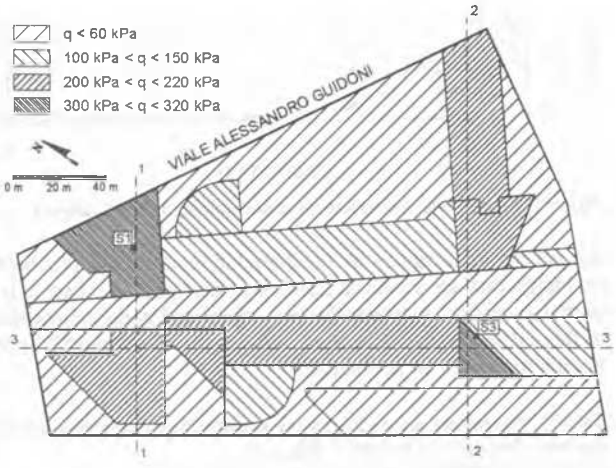

Figure 6. Loading areas, boreholes and examined sections

The sections were chosen so as to include within them the most greatly loaded and significant zones. Four kinds of elements were used in the model: solid soil elements, solid elements of a homogeneous section of concrete, solid elements consisting

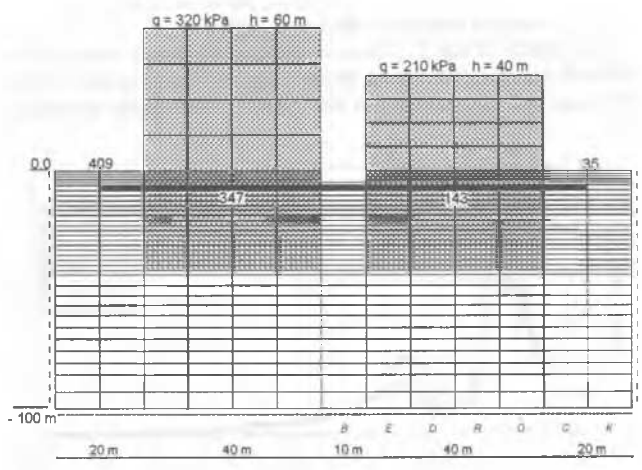

Figure 7. Section 1: scheme used in FLUSH modelling

of soil stiffened with foundation piles, solid elements equivalent to the building structure. As an example, the finite element mesh utilised in the analysis for Section 1 is shown in Figure 7.

The elastic response spectrum for a damping of 5% obtained from the interaction analysis at node 347 of Section 1, are illustrated in Figure 8; the significant parameters of the motion at the bedrock and in correspondence of 4 nodal point of Section 1 have been reported in Table 3.

Starting from a ground surface acceleration in a free field with a peak of 0.198 g and of 0.058 g at the bedrock, taking into account the soil-structure interaction, lesser values on the surface were obtained, with reductions of 23% for Section 1, 13% for Section 2, and 14% for Section 3. Then, within each section, in general no great differences exist between the values of surface acceleration close to the right and left boundaries and between

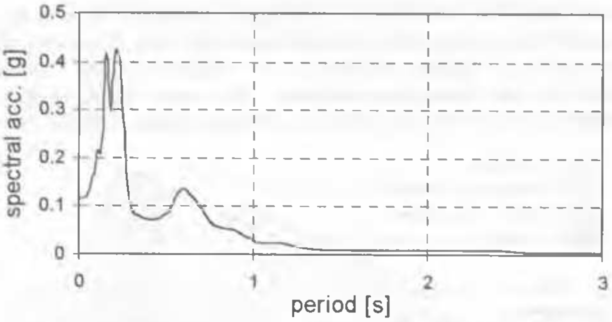

Figure 8. Elastic response spectrum at nodal point 347 of Section 1

the acceleration values at the foundation design depth below buildings that are more or less heavy and stiff. Section 2 is a partial exception: between the two superficial nodal points close to the left and right boundaries, differences of about 15% in the acceleration peak were obtained.

Table 3. Parameters inferred from the soil-structure interaction analysis at some nodal point of Section 1 (Figure 7)

FLUSH	PGA [g]	PGV [cm/s]	PGS [cm]	ν_0 [1/s]	T_0 [s]	I_a [cm/s]	P_D [10^{-4} g s^3]
Bedrock	0.058	2.09	0.57	25.25	0.203	6.73	0.106
Free field	0.198	7.68	1.73	11.38	0.200	61.88	4.781
nodal point 35	0.150	5.32	1.18	12.89	0.223	36.30	2.184
nodal point 409	0.152	5.60	1.22	12.31	0.223	37.45	2.473
nodal point 143	0.110	4.43	1.07	10.79	0.223	21.01	1.804
nodal point 347	0.113	4.47	1.00	10.86	0.223	21.77	1.845

A comparison between the response spectra at the surface and at the depth of the foundations for the three sections considered in the interaction analysis is shown in Figure 9.

In Figures 10 and 11, the amplification functions between the bedrock and the foundation design depth in the presence of the structure and the variation of the strains versus depth are represented for Section 1 and 2. In particular, we can observe that the maximum strain amplitudes calculated did not exceed the volumetric strain threshold for any of the samples analysed in the laboratory, thus upholding the choice of an approach in terms of total stresses.

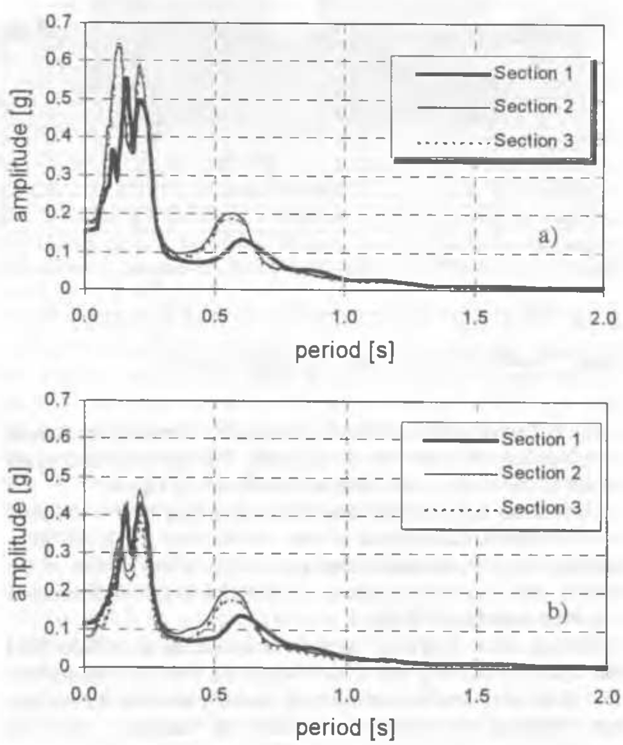

Figure 9. Elastic response spectra at the ground surface (a) and at the foundation design depth (b) for the three examined sections

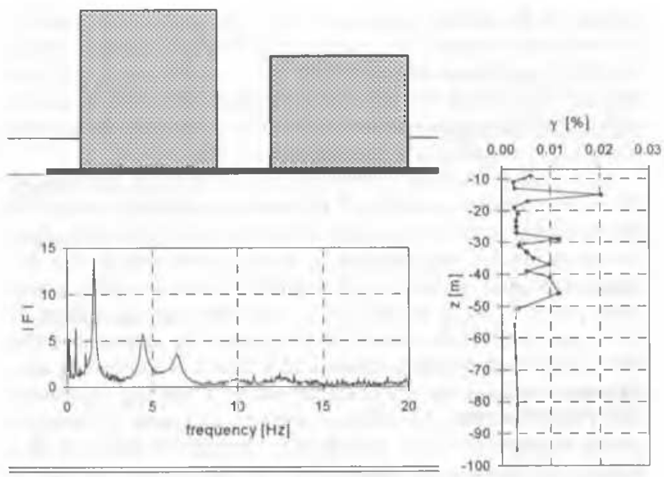

Figure 10. Section 1: amplification function and shear strains versus depth

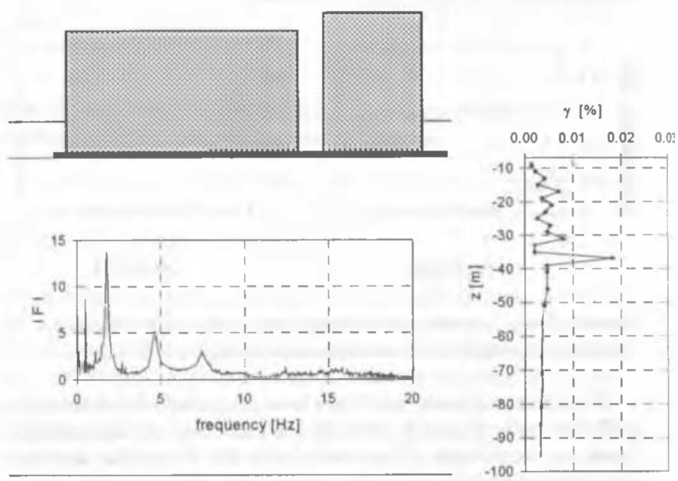

Figure 11. Section 2: amplification function and shear strains versus depth

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