First European Conference on Earthquake Engineering and Seismology
(a joint event of the 13th ECEE & 30th General Assembly of the ESC)
Geneva, Switzerland, 3-8 September 2006
Paper Number:1105

INVESTIGATION OF THE SEISMIC SOIL-STRUCTURE INTERACTION ON A CONCRETE INSTRUMENTED BUILDING

Teresa CRESPELLANI[1], Johann FACCIORUSSO[2] and Claudia MADIAI[3]

SUMMARY

Experience has shown that soil-structure interaction can play an important role on structural behaviour during earthquakes. A complete soil-structure interaction analysis consists of two parts: a site response analysis for free field motions and an interaction analysis for structural response. In the present study the effects of soil-structure interaction are evaluated on a two-storey concrete building about 10 m high with a rectangular horizontal section of about 11m x 28 m. The building, of public interest, is equipped with several accelerometers. The structure and the foundation system were idealised by a 3-D finite elements model, while the underlying soil is represented by a semi-infinite visco-elastic 1-D model. The soil-structure interaction analysis was performed by using the substructuring method implemented by the SASSI2000 numerical code. The accelerograms recorded during a low magnitude earthquake (ML = 4) by a free field seismic station, about 5m away from the building, were adopted as the seismic input motion. In this paper the results of the numerical analyses obtained in some significant nodes of the structure are presented both in time and frequency domain, and compared with the seismic recordings; moreover the influence of the structure on the ground motion is evaluated by comparing the free field actual motion with the numerical modelling results at the boundary between the soil and the foundation.

1. INTRODUCTION

Due to the numerous uncertainties affecting seismic motion and soil behaviour, a precise prediction of soil-structure interaction (SSI) during earthquakes is generally difficult. Therefore, in Italy, current design practices and seismic provisions disregard the influence of structure on local seismic motion and take into account only the free-field soil response. But, a large amount of experimental and analytical data show that the consideration of SSI effects can lead either to seismic actions lower than the free field ones, or to more conservative actions. For this reason, as it is a controversial matter, mainly for monuments and other remarkable buildings or critical facilities, taking SSI in consideration may be essential for assuring safety and reliability or reducing repair costs. Even recently, in Central Italy many important historical monuments have suffered damage during earthquakes, with a large number of them requiring to be restored (Crespellani et al., 2003).
In this light, a preliminary condition in order to perform a safe design is to verify how the current seismic SSI computer programs are able to model appropriately the most important aspects of SSI at the site, as SSI is largely dependent on soil deformability and the softer the soil is, the larger the differences between free-field displacements and the displacements at the base of the structure are. It is evident that, for a true validation of these programs, the possibility of comparing numerical results to real motions recorded at the basis of a structure, may represent a great chance.

[1] Dipartimento di Ingegneria Civile dell'Università di Firenze, 50139, Via di S. Marta, Firenze, Italia
Email : teresa.crespellani@dicea.unifi.it
[2] Dipartimento di Ingegneria Civile dell'Università di Firenze, 50139, Via di S. Marta, Firenze, Italia
Email : johannf@dicea.unifi.it
[3] Dipartimento di Ingegneria Civile dell'Università di Firenze, 50139, Via di S. Marta, Firenze, Italia
Email: clau@dicea.unifi.it

As is well known, SSI has a long 30-year history. Since the pioneering studies of Parmelee (1967) and Lysmer et al. (1974), the complicate dynamic processes of SSI have received increasing attention. Nowadays, the essential features of SSI are clear enough and many procedures, both rigorous and approximate, allow the designer to make predictions.
Over the last decade one of the computer programme most used for SSI evaluations has been FLUSH (Lysmer et al., 1990). Recently, a new finite-element-based programme, SASSI2000 (Lysmer et al., 1999), that uses the substructure method, has attracted the attention of researchers and designers.
In order to validate its use for SSI studies on a site in Central Italy, representative of many other sites in the region, the Authors had the opportunity of using the accelerograms recorded by a monitoring permanent network of the National Seismic Service during a recent weak earthquake ($M_L = 4$) in a two-storey concrete building about 10 m high and with a rectangular horizontal section of about 11m x 28m.
The site, where the building is located, is characterised by a frequent, though prevalently non-destructive, seismicity. Because SASSI2000 relies on superposition principle, and its use is limited to linear or equivalent linear systems, its use appeared to be congruent with the site seismicity and soil conditions.
The present paper provides a synthetic report of the soil-structure interaction analyses performed for SASSI2000 validation.

2. GEOTECHNICAL CHARACTERISATION AND MODELLING OF SUBSOIL

The subsoil underlying the examined building consists of an alluvial deposit, with a thickness of about 170m, lying on marine clays containing fossil remains of Quaternary. The alluvial deposit is prevalently constituted by silty (locally sandy) clay with, at times, local levels of peat. At various depth, layers of gravels and sand, reaching a thickness of ten metres or more, constitute about 26% of the entire stratigraphical sequence.
The stratigraphical section can be summarised as shown in Figure 1, where soil layers are grouped on the basis of their geological origin, lithological properties and mechanical behaviour in: Unit A, mostly formed by clay and silt with local intercalation or enrichment of fine silty sand; Unit B, constituted by a sequence of levels and lenses of fine sand alternated with clay or silty sediments of various thicknesses; Unit C, made up of gravel or gravel with sand and silt. The main average geotechnical properties obtained from samples collected in Unit A and B are summarised in Table 1 (no meaningful data can be obtained for the gravely soils of Unit C).

Table 1: Main average geotechnical properties of Unit A and B

	Gravel [%]	Sand [%]	Silt [%]	Clay [%]	γ_{sat} [kN/m^3]	w_L [%]	w_P [%]	I_P [%]	w [%]	I_c	c' [kPa]	φ' [°]
Unit A	0	49.4	21.0	29.6	19.1	37	23	14	29	0.6	-	-
Unit B	0	76.9	18.9	4.2	19.4	32	23	9	27	0.6	25	25.2

To perform the dynamic SSI analysis with SASSI2000 code, the foundation soil below the building needs to be modelled as an horizontally layered deposit. Besides the unit weight γ, at each layer the following must be assigned: the damping ratio, D, shear and compression waves velocity, Vs and Vp respectively. SASSI2000 does not need the initial value of these dynamic soil parameters, but the value consistent with the induced effective shear strain amplitude, γ_{eff}, calculated by means of a numerical 1-D local seismic response analysis performed with the computer program SHAKE (Schnabel et al., 1972). The initial values of shear and the compression waves velocity with depth, utilised as input to SHAKE code, were estimated from down-hole tests previously performed in the surrounding area on soils characterised by physical properties and mechanical behaviour very similar to those of the subsoil underlying the building in question. The Vs and Vp profile at depths greater than those explored with down-hole tests, the bedrock depth (at about 115 m from the ground level) and the corresponding Vs and Vp values were estimated on the basis of specific local geological studies. For example, the Vs profile with depth is given in Figure 1. The normalised shear modulus, $G(\gamma)/Go$, and the damping ratio, $D(\gamma)$, values, varying with shear strain amplitude, were obtained for soils of Unit A and B from resonant column and cyclic torsional shear tests performed on undisturbed specimens with physical and mechanical properties which can be compared with those of the foundation soils. The regression curves and experimental data are shown in Figure 2 together with the corresponding curves assigned to soils of Unit C, proposed by Rollins et al. (1998) for gravely soils (undisturbed samples not being available).

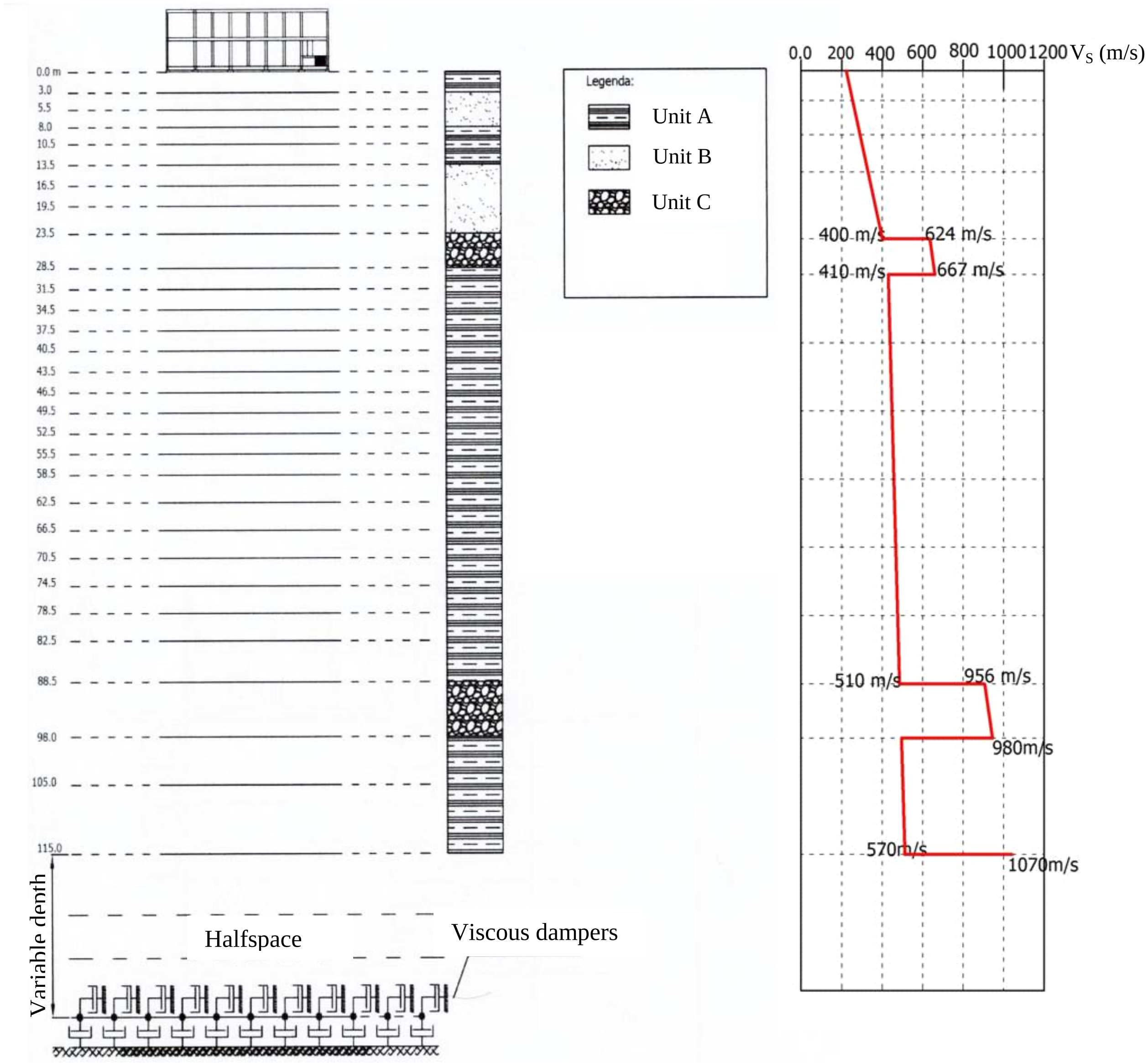

Figure 1: Model of soil below the building and V_S profile with depth (in metres)

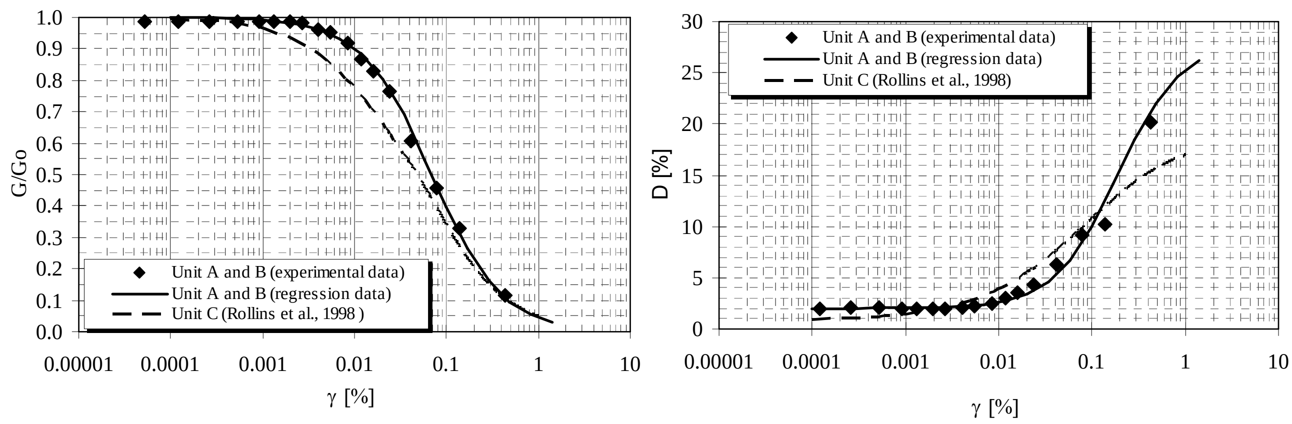

Figure 2: Normalised shear modulus, G/G_0, and damping ratio, D, with shear strain amplitude from experimental data, regression model and literature model (Rollins et al., 1998) for Unit A, B and C

3. STRUCTURAL CHARACTERISATION AND MODELLING OF SUPERSTRUCTURE AND FOUNDATION SYSTEM

The building examined, about 10 m high, has a regular compact shape with a rectangular horizontal section of about 11m x 28m and a flat roof. The structure is composed of a concrete frame with two masonry cement floors respectively at 4.25 m and 8.35 m from ground level with a total thickness of 55 cm of. A horizontal structural section of the building corresponding to the first floor is given in Figure 3.

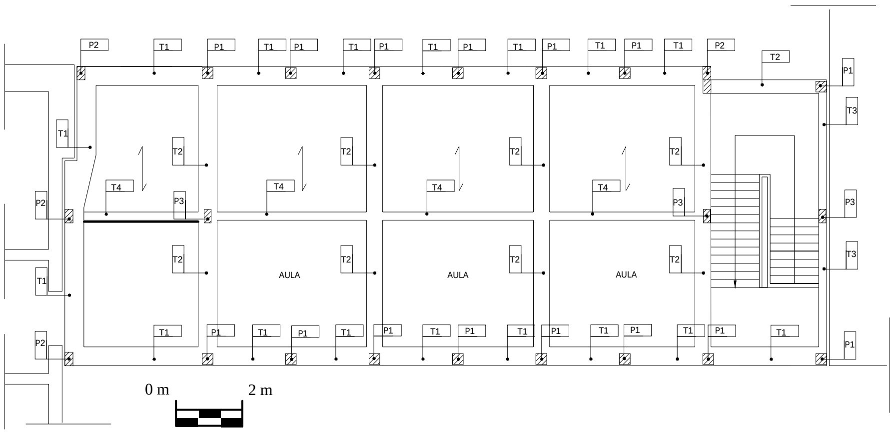

Figure 3: First floor horizontal structural section

The foundation system, made up of a system of transversal and longitudinal ground beams, is placed at a depth of 80 cm below ground level and lies on soils belonging to Unit A. The water level is around 3 meters below ground level. The shape of the transversal section of the ground beams varies according to the loading forces as shown in Figure 4.

The excavated soil volume, the foundation system and superstructure were modelled with finite elements. The maximum size of the elements was chosen between $1/5\lambda$ and $1/8\lambda$, where λ is the minimum wavelength of the input seismic excitation (calculated as the shear waves propagation velocity of the medium divided by the maximum frequency contained in the input motion). The shape of the elements adopted is "beam" (for the superstructure) and "solid" (for the foundation system). The mechanical properties of each elements were assigned as unit weight, γ_m (25 kN/m^3), elastic modulus, E_m (0.285 10^8 kN/m^2 for solid elements; 0.317 10^8 kN/m^2 for beam elements), Poisson coefficient, υ (0.2), damping ratio, D (3%) and a linear-elastic constitutive model was adopted for the material.

The whole superstructure was modelled by means of monodimensional elements (beams). The floor systems were assumed infinitely rigid on their own plane and the subsequent mechanical behaviour of each floor (a type not considered in SASSSI code) was reproduced by introducing diagonal infinitely rigid and weightless rods (as highlighted in Figure 5) The influence of the infill systems on the behaviour of the structure was considered only for thickness greater or equal to 12 cm and simulated by considering in the vertical frame two diagonal rods in the place of each infill (see Figure 5). The infill system weight was divided by the adjacent beams by considering the corresponding influence area and added to the actual beam weight. The floor system masses were divided by the floor system nodes relating to the respective influence area. The foundation system was modelled with 8 nodes prismatic elements (solid) and represented in Figure 4.

4. RECORDING SYSTEM AND SEISMIC INPUT

The building under study is continuously monitored by an accelerometric network composed of 5 recording instruments placed on the structure and one in free field (as shown in Figure 6). The acceleration time histories considered in this study were obtained from the horizontal components recorded during a low magnitude earthquake, ML = 4.0 (Greenwich Date: 2003/12/07; Greenwich Time 10:20), whereas the vertical components were considered of less significance. The location and the orientation of the recording channels are indicated in Figure 6. The acceleration time histories adopted as input to SASSI2000 code and obtained from the recordings of the horizontal channels (1 and 2) at the free field station are represented in Figure 7 and the corresponding seismic motion parameters are synthesised in Table 2.

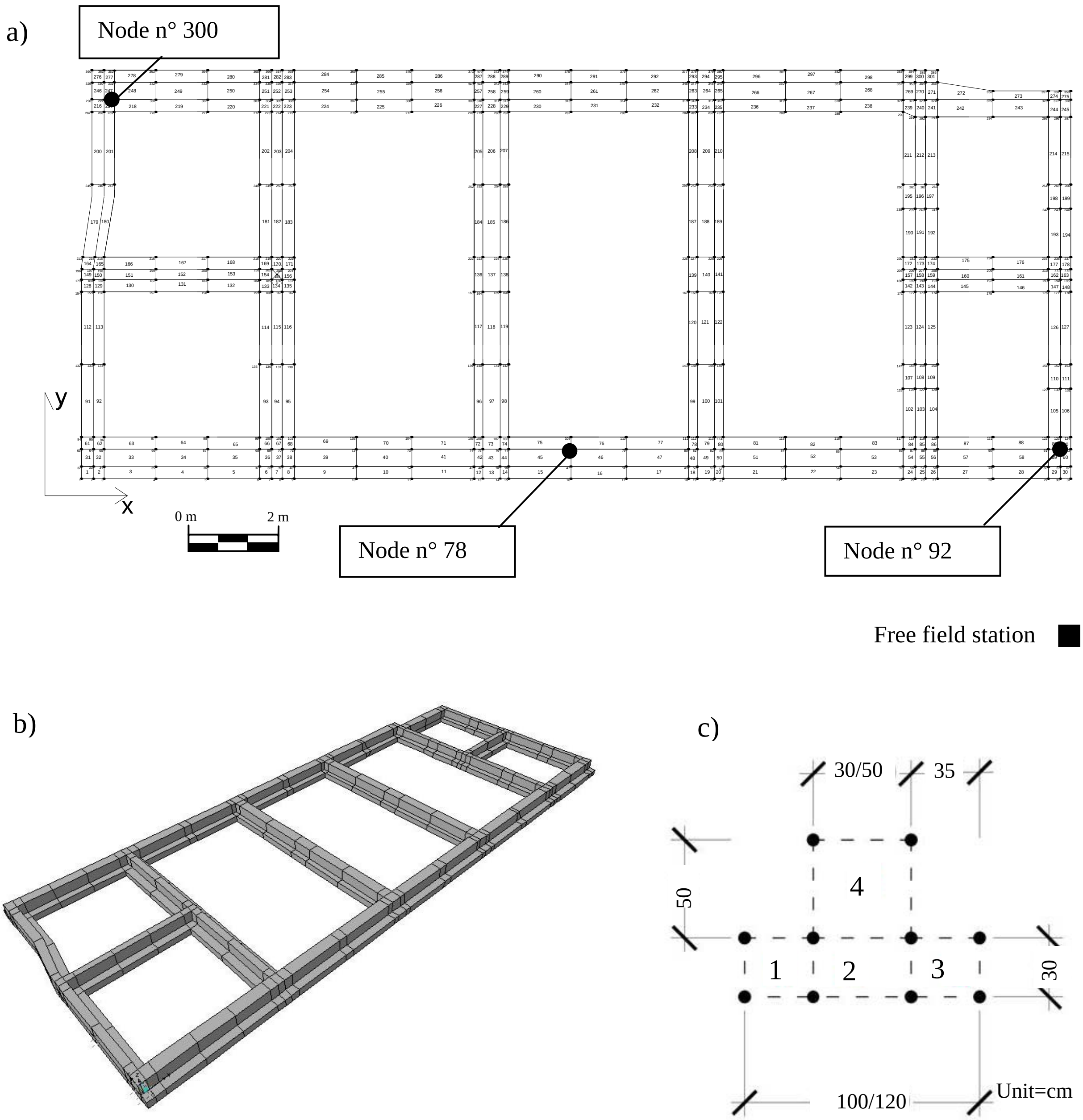

Figure 4: Planar section (a), 3-D view (b) and type-section (c) of the input foundation model (after Palermo, 2005)

The range of frequencies within to perform the SSI analysis with SASSI2000 code, were selected on the basis of the elastic response spectra of the input accelerograms and the predominant frequencies of the structure calculated from a modal analysis conducted by the he National Seismic Service.Finally a range between 2 and 20 Hz was selected and 14 frequencies were considered within this interval.

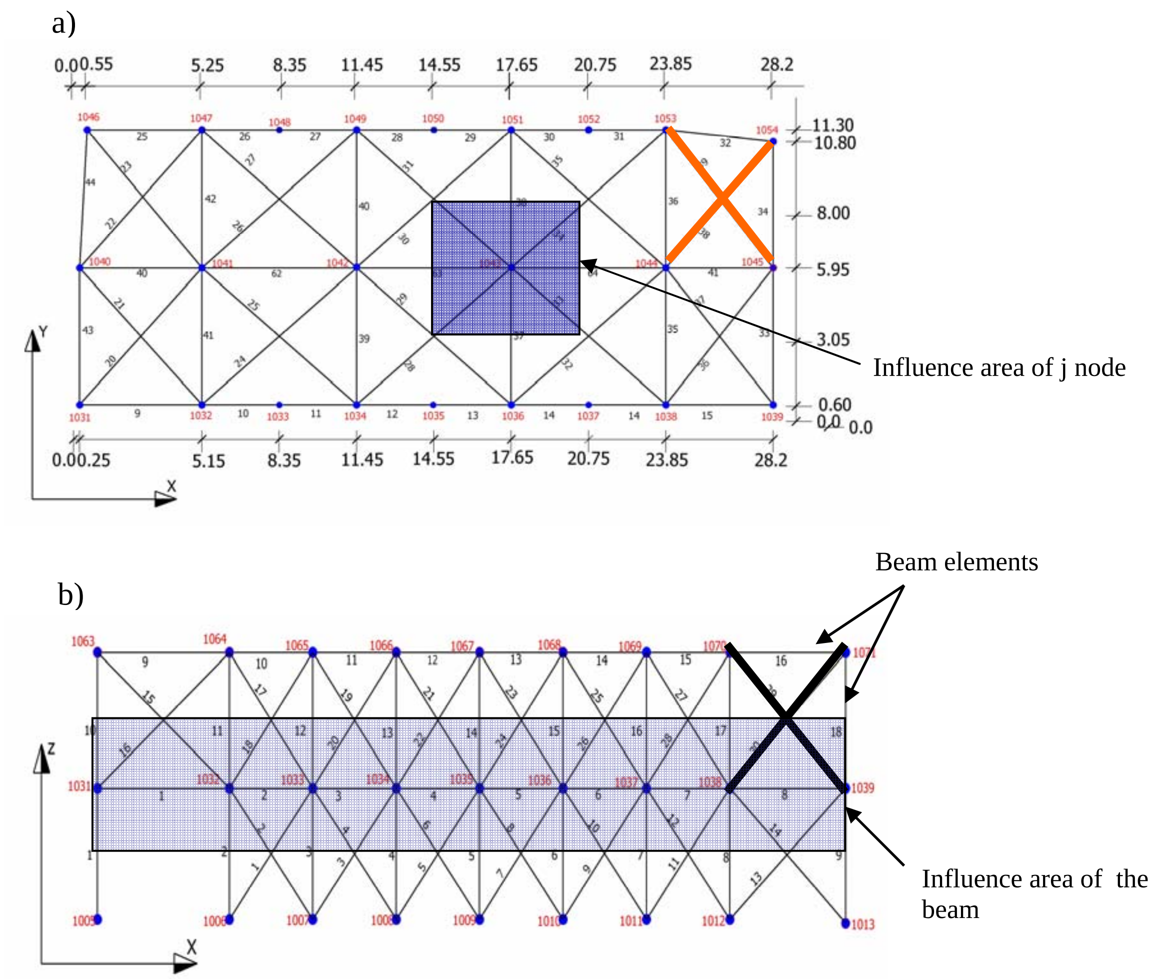

Figure 5: Input model of the type floor system (a) and type vertical frame (b) (after Palermo, 2005)

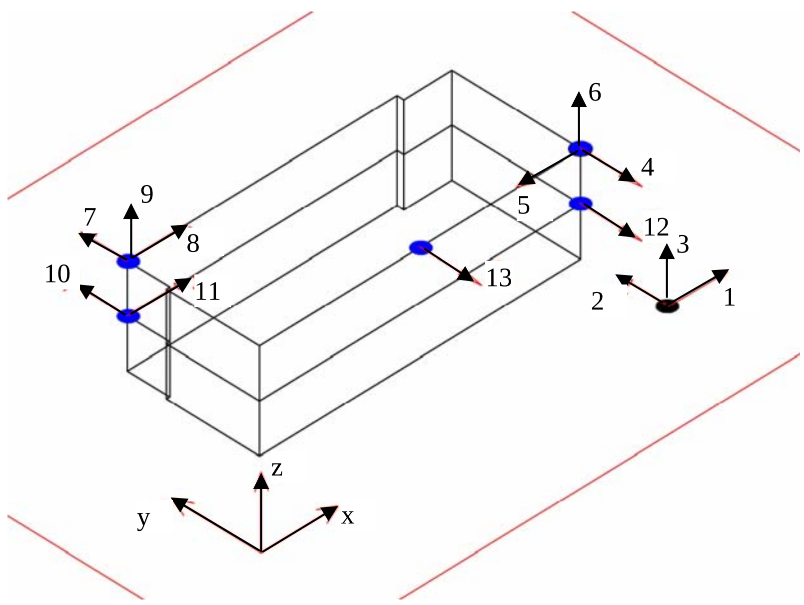

Figure 6: Location of accelerometers at free field and on the structure, code and orientation of the recording channels

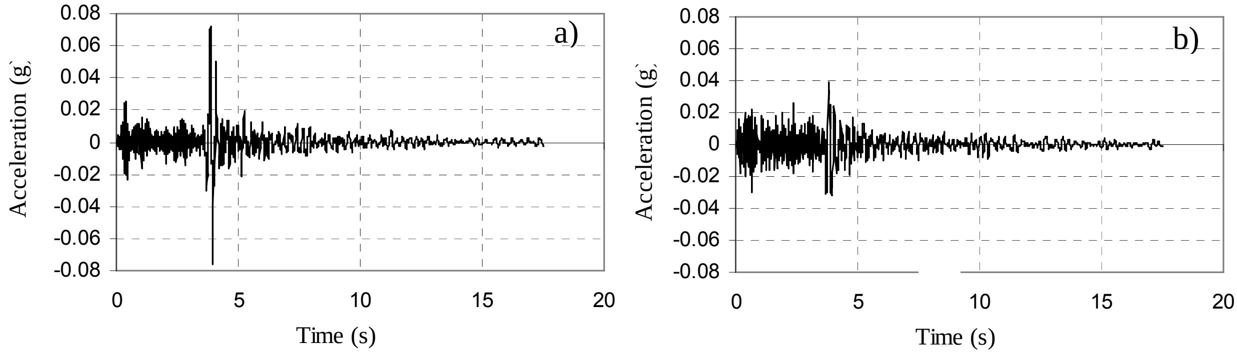

Figure 7: Input acceleration time histories obtained from recordings at free field station: a) channel 1 (X-axis); b) channel 2 (Y-axis)

Table 2: Input seismic motion parameters at free field

	Channel 1	Channel 2
PGA (g)	0.0756	0.0388
PGV (mm/s)	27.98	14.36
PGD (mm)	1.86	2.01
Arias Intensity (mm/s)	14.1	7.92
Trifunac duration (s)	7.48	10.31
Predominant period (s)	0.2731	0.2467
Maximum spectral acceleration (g)	0.187	0.134

5. RESULTS: DISCUSSION AND COMPARISONS

5.1 Experimental and numerical response of the structure to the selected seismic input

The results of the numerical analyses were compared with the available seismic recordings of the earthquake that occurred on 2003.12.07 from the accelerometric network installed by the National Seismic Service. The comparison was made in the nodes corresponding to the points where the accelerometers were located (Figure 4). For a deeper insight on SSI analysis, the parameters used to compare numerical and recorded data were chosen both in frequency and time domain. The results of the comparison for the eight considered channels are summarised in Figure 8, in terms of peak ground acceleration (PGA), Arias Intensity (I_a), Trifunac duration (Td) and maximum spectral acceleration (S_{Amax}). In particular, for each parameter, the per cent difference between the value from the numerical analysis (MD3) and the value from the recording (SSR), defined as: Δparameter = [parameter(MD3)-parameter (SSR)]/parameter(SSR) is shown.

The comparison evidences a good agreement between numerical results and recorded data, with maximum differences of about 8% in peak ground acceleration (channel 4) and 33% in Arias intensity (channel 8). With regard to Trifunac duration, values from the numerical analysis are always greater than those obtained from recorded data, with remarkable differences given for channel 4 (32%) and 8 (72%). The comparison of the acceleration time histories indicates that the model values are higher than those recorded during the initial phase of the earthquake (0-3s). This fact is also evidenced by Arias intensity time histories, which indicate that energy content of numerical signals during the initial phase (3-3.5sec) is generally higher than recording energy content. By way of example, in Figure 9 the recorded accelerograms (RRS) and the corresponding acceleration time histories of the model (MD3) at channels 5 (X-axis) and 10 (Y-axis) are represented. Figures 10 shows the values of Arias Intensity versus time obtained for the instrumental data and model at the same channels. To compare recording and numerical frequency contents, Fourier and elastic spectra in terms of acceleration were used. The comparison between the elastic spectra (critical damping ratio ζ=5%) evidences that the spectra obtained from the numerical analysis are very similar to those obtained from recordings. As Figure 8 shows, maximum spectral acceleration differences are very small, with a maximum value of about 22% for channel 11.

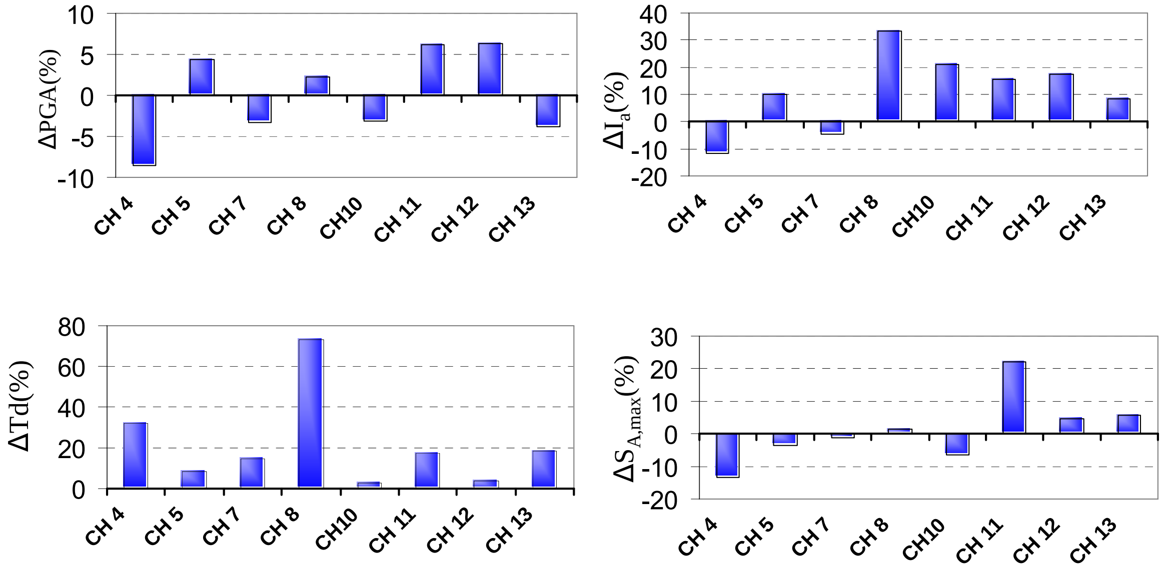

Figure 8: Comparisons between experimental and numerical rsults in terms of seismic parameters

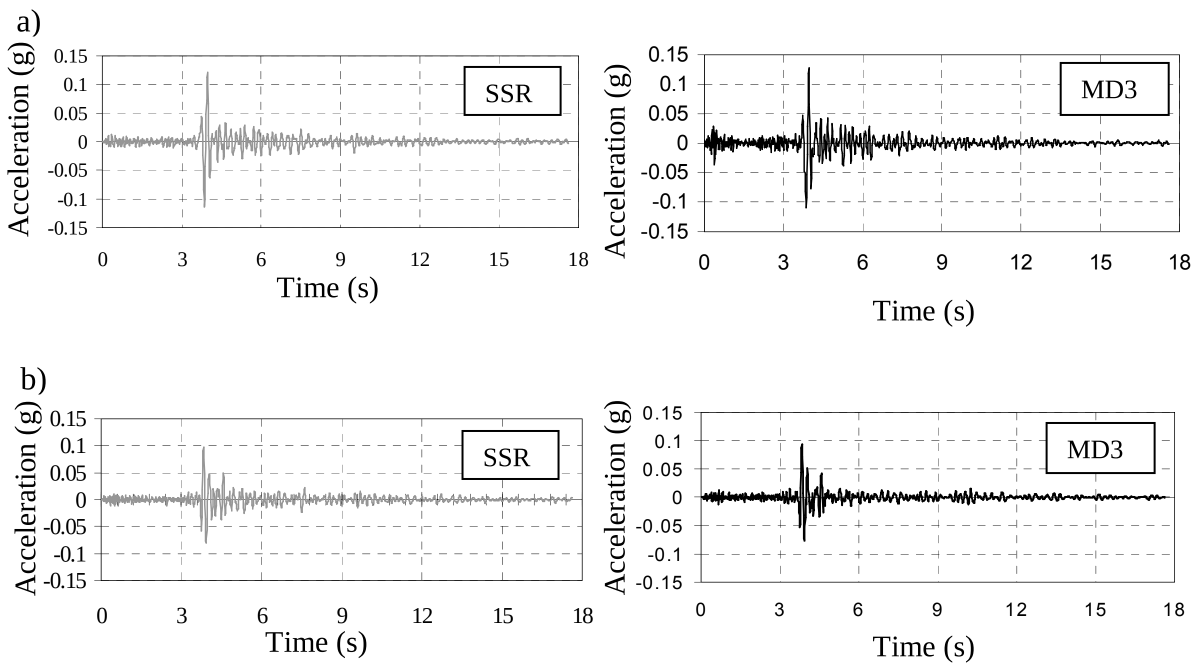

Figure 9: Experimental data (SSR) from channel 5(a) and 10 (b) and numerical results (MD3) from the corresponding nodes (b) in the time domain

Comparing the Fourier spectra evidences that maximum amplitudes range from 3 to 8 Hz, both for recorded and calculated acceleration time histories. The spectral shape obtained from numerical analysis is in good agreement with that of the recordings to frequency values of about 12 Hz; since the analysis frequencies are defined in detail only to a frequency of 12.55 Hz, the spectral functions greatly differ at high frequencies. By way of example, Figure 11 shows the elastic response spectra and the Fourier spectra obtained from recordings and numerical analysis for channel 5 (X-axis) and 10 (Y-axis).

5.2 Free field and far field motion comparison

With the aim of evaluating the SSI effects, several seismic parameters obtained in free-field were compared with those obtained from numerical analysis in some significant nodes (shown in Figure 4 as 92, 78 and 300) at the foundation level. Seismic motion was considered in both X and Y directions.

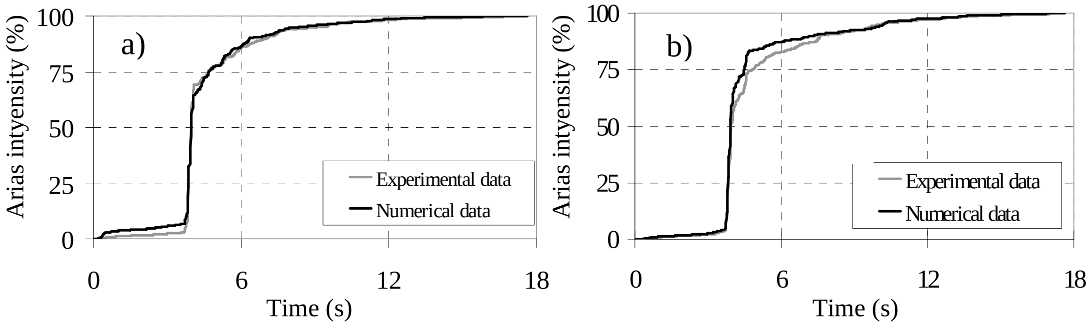

Figure 10: Comparison of Arias intensity time histories obtained form experimental and numerical data in correspondence of channel 5(a) and channel 10 (b)

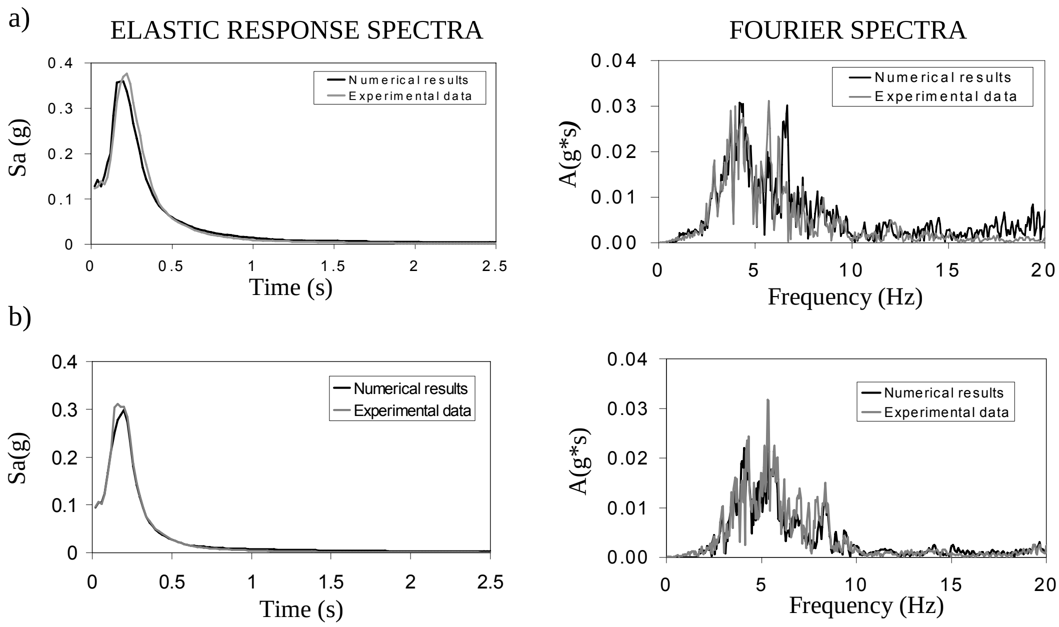

Figure 11: Comparisons between experimental data from channel 5 (a) and 10(b) and numerical results from the corresponding nodes in the frequency domain

In order to analyse the effects of the presence of the structure, SSI factors, $K_{parameter}$, were defined for each considered parameter (PGA , I_A , T_d) as the ratio between the value of the parameter obtained from numerical modelling in the selected node and the corresponding value from motion recorded at free-field. Table 3 summarises the parameters of the motion from numerical analysis and the values of the SSI factors previously defined. The results suggest that the presence of the structure has a damping effect. SSI factors are always smaller than 1 in all of the three nodes considered; damping is about 15% in terms of maximum peak ground acceleration and 30-40% in terms of Arias intensity and Trifunac duration. Damping effect is especially pronounced during the first phase of the time history, between t=0 e t=3.5 sec. In the frequency domain, the spectral accelerations of the free-field elastic spectra are greater than those of elastic spectra in the analysed nodes for the period ranging from T=0 and T=0.14sec and are substantially equal for T>0.14 sec. The maximum spectral accelerations and the corresponding periods obtained in the base nodes considering the reference input motion in X and Y direction are given In Table 4 with the per cent difference defined as $\Delta S_{Amax}=[S_{Amax}(\text{node})-S_{Amax}(\text{free-field})]/S_{Amax}(\text{free-field})$.

Table 3: Main parameters of the motion from numerical analysis and values of the SSI factors in the selected -nodes

	PGA [g]	I_A *[m/s]*	T_d *[s]*	K_{PGA}	K_{Ia}	K_{Td}
Node 92-X	0.0651	0.1	4.88	0.87	0.76	0.65
Node 78-X	0.0608	0.0093	4.53	0.82	0.70	0.60
Node 300-X	0.0633	0.0095	4.62	0.85	0.72	0.62
Node 92-Y	0.0328	0.0046	7.84	0.88	0.63	0.78
Node 78-Y	0.03125	0.0043	7.84	0.84	0.59	0.78
Node 300-Y	0.0315	0.0042	7.84	0.85	0.58	0.78

Table 4: Maximum spectral accelerations, corresponding periods and percent difference in the considered nodes

	Node 78-X	Node 92-X	Node 300-X	Node 78-Y	Node 92-Y	Node 300-Y
S_{Amax} [g]	0.182	0.192	0.184	0.127	0.131	0.128
T [s]	0.24	0.24	0.24	0.22	0.22	0.20
ΔS_{Amax} [%]	-2.67	+2.67	-1.60	-5.22	-2.23	-4.48

6. CONCLUDING REMARKS

Comparisons are made between analytical results obtained by applying the numerical program SASSI2000 and instrumental data, recorded in a building in Central Italy during a recent earthquake of M_L=4. The building is part of a set of buildings of public interest, in which the Italian National Seismic Service installed a network of accelerometric instruments. The main objectives of the analyses carried out were essentially to test the reliability of the computer program SASSI2000 for SSI evaluation on the site and, secondly, to verify by comparing free-field response and seismic response at the basis of the construction, the importance of SSI for seismic design of buildings with similar structural characteristics in the area. For such comparisons, different characteristics and parameters of motion were used both in time and frequency domain, such as acceleration time histories, PGA, Arias Intensity, Trifunac duration, Fourier spectra, acceleration elastic response spectra and maximum spectral acceleration. The research carried out indicates that:

1) in general the numerical predictions are a googd match for the experimental data from measurements;
2) the free-field response and response at the base of the building are very similar.

Thus, the following conclusion can be drawn: the computer program used seems to be reliable enough for SSI analyses on the site, but SSI does not seem to have a significant role in the case of buildings having the structural characteristics of that monitored. For such buildings, SSI can be disregarded in current seismic design.

7. REFERENCES

Crespellani, T., Madiai, C. and Uzielli, M. (2003), FEM Geotechnical Analysis of the Seismic Damage to the Church of S. Filippo in Nocera Umbra during the September 26, 1997 Seismic Sequence, *in: Maugeri e Nova (a cura di): Geotechnical analysis of seismic vulnerability of monuments and historical sites, Cap. 8, pp. 187-208*

Lysmer, J., Udaka, T., Seed, H.B. and Hwang, R. (1974), LUSH - A computer program for complex response analysis of soil-structure system, *Rep. EERC 74-4, EERC, University of California, Berkeley*

Lysmer, J., Udaka, T., Tsai, C., Seed, H.B. (1990). *FLUSH - A computer program for approximate 3-D analysis of soil-structure interaction problems.* Rep. EERC 75-30, University of California, Berkeley.

Lysmer, J., Ostadan, F., and Cheng Chin, C. (1999), SASSI2000 - A system for analysis of soil-structure interaction, T*heoretical manual, University of California, Berkeley*

Palermo, M. (2005), Analisi con un modello 3D dell'interazione terreno struttura in condizioni sismiche . Applicazione al caso dell'Istituto Tecnico Industriale G. Marconi, Forlì, *Master Degree Thesis, Firenze*

Parmelee, R.A.(1967), Building-foundation interacion effects, *J. Eng. Mech. Div. ASCE, 93 (EM 2), pp.131-152.*

Rollins, M.K, Evans, M.D., Diehl, N.B. and Daily III, W.D. (1998), Shear modulus and damping relationships for gravels, *ASCE Journal, 396-405*

Schnabel, P. B., Lysmer, J. and Seed H.B. (1972), SHAKE – A computer Program for Earthquake Response Analysis of a horizontally layered site, *Report N° EERC72-12, University of California, Berkeley*