Abstract— Although in Central Italy seismicity is generally moderate and the maximum expected local Richter magnitudes are about 6-6.5, in the past many urban settlements and villages have experienced, even during low intensity earthquakes, very severe damage, which can be attributed to the sharp topography or the presence of thick deposits or the particular dynamic properties of the soils. Therefore, in order to provide new updated guidelines for seismic building design, great research effort has been aimed at the quantification of site responses in various Italian historical centres over the latest years. Moreover, since Eurocode 8 (EC8) will be adopted within a short period in countries of the European Union, these studies have also been aimed at controlling reliability of EC8 spectra for Italian sites. This paper describes the research developed for site effects evaluation at Offida, a historical centre in The Marche with important monuments and environmental heritage. A specific program of seismological investigations, consisting in measuring low amplitude earthquakes and microtremors, and geotechnical testing, including boreholes, dynamic field and laboratory tests, was carried out. A variety of methods were utilised and compared, considering different earthquake scenarios and ground motions. The obtained spectra were finally compared with those proposed by Eurocode 8.

Keywords—Eurocode 8; SHAKE; site coefficient; site effects; soil dynamics; response spectrum

INTRODUCTION

In most international seismic codes for building design, site effects on seismic actions are included by specifying several site categories (soil profile types) and assigning one or more spectra or site coefficients for each soil profile. In Eurocode 8-Part 1 (EC8) soils are classified into five different soil profiles depending on average shear wave velocities and other geotechnical parameters in the upper 30 meters. Two elastic spectra are suggested for each soil profile with regard to the expected magnitude value at the site [1].

As is well known, EC8 will soon become a European harmonised rule (EN). Therefore, with a view to conforming the national building codes to EC8, the proposed spectra reliability control is a key problem for all European Union countries. Especially for historical settlements in Central Italy, as these are characterised by particular geomorphologic, geotechnical and architectural features, the assessment of protection levels implicit in EC8 spectra is of primary importance.

Although seismicity is generally moderate and expected maximum local Richter magnitudes are about 6-6.5, in the past many urban settlements and villages have experienced very severe damage, which can be attributed to the sharp topography or the presence of thick soil deposits or the dynamic properties of the soils. After the latest seismic events in September-October 1997, which affected one of the most important Italian historical zones, great research effort has been directed by regional governments into the assessment of ground responses in various sites of Emilia-Romagna, The Marche, Umbria and Tuscany [2], [3], [4], [5], [6], [7], [8].

Recently, a few centres located in a seismically active region in the eastern part of The Marche have been selected as test-sites for site effects evaluation.

The present paper describes the methodology and the results obtained at Offida, one of the centres examined in the region, rich in suggestive environmental and historic heritage (churches, palaces, bell-towers, etc.).

Two main approaches, seismological and geotechnical, were followed.

The first approach was based on the acquisition, statistical treatment and comparison of recordings of microseismes (low frequencies) and microtremors (high frequencies), from mobile seismic stations installed temporarily on sites with different soil conditions. Being a relatively low-cost approach, it enabled the investigation of a large number of sites.

The geotechnical approach, which will be described in detail in this paper, was based on the evaluation of ground responses in the free field in more restricted target areas, by means of numerical models, using, as input motions, different accelerograms representative of the possible types of ground motions which can be expected to occur at the site on outcropping rock. In order to define the dynamic properties of the rock and soil formations present in the area examined, extended geotechnical investigations, including dynamic field and laboratory testing, were carried out.

The researches performed enabled two main sets of comparisons, that is:

a) the resulting elastic acceleration spectra from numerical analyses, obtained at the various sites, were compared with those proposed by EC8 for the same soil profiles;

b) the results of calculations, being essentially elastic, were compared with the results of the instrumental recordings.
The results of these comparisons provided useful indications for assigning a design spectrum for building design and site coefficients to the various soil formations underlying the ancient part and the new development areas of Offida.

SITE DESCRIPTION

The town of Offida (Fig. 1) is situated on two perpendicular ridge-lines separating the rivers Tesino and Tronto, on the first of which the historical centre is located in an anti-Apennine direction, whereas on the second, the more recent part of the town has been built.

The site has been inhabited since prehistoric times. The urban configuration preserves Medieval features and ancient XIV century walls, but mainly a heritage from the period of the Papal States. The historical centre hosts important monuments and items of environmental value.

Local geology is characterised by a diffuse presence of the so-called Umbria-Marche serie, composed of limestones, marls and flysch, alternated with Schlier, Bisciario, Scaglia (Rossa, Bianca and Cinerea) units, which have very similar mechanical properties (shear wave velocity ranging from 1000 to 1200 m/sec). These units are outcropping or covered by alluvial, colluvial, eluvial or debris soils.

The area is crossed by numerous faults oriented NW-SE and NS, some of which are seismologically active. As can be seen from Fig. 1, showing the surface geology of the area, the more superficial formations are Plio-Pleistocene soils, consisting of grey-blue clays, silty sands, and conglomerates overlying “bluish grey” marls. Most of the ancient town lies on Plio-Pleistocene marine deposits, which are often covered by thin layers of filling and sometimes by thicker strata of eluvium and colluvium up to 13-15 m in depth.

EXISTING DATA AND NEW INVESTIGATIONS

A general framework of geological and seismological features of the area under examination was obtained from aerial photos, geological maps at 1:5000 scale and acceleration time histories from the national accelerograph network.

With a view to achieving more detailed information on the seismic responses of different soils, an extended experimental multidisciplinary program, including seismometric and microtremors measurements, boreholes and geotechnical testing, was studied.

The following seismological instrumental data were purchased [5]:
- velocimetric measurements of microtremors on 13 sites by using mobile apparatus (Lennartz 3D-Lite sensor, with a natural frequency of 1 Hz);
- recordings of weak earthquakes and quarry blasts from 5 seismometers (MARS 88 – FD and Lennartz model “LE-3D Classic”, at short period) that recorded on average more than 30 seismic events.

For instrumental data interpretation, two main techniques were adopted by using both two recording sites and only one site. In the first case, the amplification function at the site was calculated from the spectral ratio between the horizontal component recorded at the site examined and the horizontal component recorded at a reference site on outcropping rock (Standard Spectral Ratio method, hereinafter referred to as SSR). In the second case, since it has been demonstrated [9] that the spectral ratio between horizontal and vertical components (H/V) of the ground mainly depends on soil stratigraphy and properties, and its maximum is correlated with the fundamental period of soil deposit [10], soil amplification was evaluated in terms of amplification function as the H/V ratio of the recording (Horizontal to Vertical Spectral Ratio method, referred to by the acronym HVSR in this paper).

To define a subsoil model for ground response one-dimensional numerical analyses at the different sites the following geotechnical data were acquired:
- soil profiles from 10 boreholes ranging from 15.5 to 32 m in depth;
- records from 4 Down-Hole tests;
- data from geotechnical laboratory tests, including resonant column and cyclic shear torsional testing.
The seismic risk in the various zones of the inhabited nucleus was also assessed by examining the structural typology of the existing buildings.

SEISMICITY AND INPUT MOTIONS

The test site of Offida is located in a seismically active region, characterised by a frequent and diffuse seismicity that even recently, in the years 1972, 1979, 1984 and 1997, produced many sequences of several months of shocks with local Richter magnitudes ranging from 5 to 6. In the past, Offida experienced various destructive earthquakes; the most recent of which an VIII MCS seismic event which struck in 1943.

Offida falls in the second seismic category. From the hazard maps of the National Group for Defense from Earthquakes (GNDT), values of macroseismic intensity I = 7.5 MCS and PGA = 0.173 g are expected for a return period of 475 years (probability of exceedance of 10% in 50 years) [11], [12].

Although the probability of strong earthquakes is relatively low, given the presence of suggestive historic heritage (churches, palaces, bell-towers, etc.) and the possibility of high economic impact even during weak earthquakes, various seismic scenarios and ground motion levels with different energy content were considered [5] in order to examine site effects on seismic actions:

1. Level 1 (design motion): this level is associated with an event conventionally defined according to EC8 having a probability of exceedance of 10% in 50 years (return period of 475 years);
2. Level 2 (more likely motion): this level is representative of a small magnitude event likely to occur during a life-span of 50 years with a probability of exceedance of 50%;
3. Level 3 (very low motion): this level is associated to events similar to the aftershocks of the seismic sequence of September-October 1997.

For the definition of the three levels of motion, several accelerograms were selected. For Level 1, the NS and WE horizontal components of the Pollino earthquake of 09.09.1998 were adopted. For Level 2, a historical earthquake which occurred on October 3, 1943 of Magnitude 5.5 was taken as reference, and a synthetic accelerogram was generated by using a 2-D model [13].

For Level 3, the horizontal acceleration components recorded at a nearby station (located at Cagli) of the Italian accelerograph network on May 2, 1984, were assumed. Given the very low energy content these were scaled by multiplying acceleration values by 5, in order to make them comparable to the aftershocks acceleration values of the 1997 Umbria-Marche earthquake.

For the purposes of this paper, which focuses on calculated response spectra, EC8 and instrumental data comparisons, only the results regarding Level 1 and Level 3 will be presented in the following. The main seismic parameters of the selected input motions for 1-D numerical analyses are summarised in Table 1.

### Table 1: Main seismic parameters of seismic ground motions

<table>
<thead>
<tr>
<th>Recording</th>
<th>PGA</th>
<th>$T_a$</th>
<th>D</th>
<th>$D_T$</th>
<th>$I_a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cagli-NS</td>
<td>0.0285</td>
<td>0.0570</td>
<td>20.02</td>
<td>3.45</td>
<td>0.195</td>
</tr>
<tr>
<td>Cagli-EW</td>
<td>0.0246</td>
<td>0.0514</td>
<td>20.02</td>
<td>3.98</td>
<td>0.311</td>
</tr>
<tr>
<td>Pollino-NS</td>
<td>0.1730</td>
<td>0.6400</td>
<td>22.11</td>
<td>3.56</td>
<td>16.72</td>
</tr>
<tr>
<td>Pollino-EW</td>
<td>0.1649</td>
<td>0.3562</td>
<td>22.11</td>
<td>4.26</td>
<td>11.89</td>
</tr>
</tbody>
</table>

The soil profiles obtained from borings and the S-wave velocities from down-hole tests, adopted in the model, as well as the locations of undisturbed samples, are shown in Fig. 2.

The physical and mechanical properties of the identified soil layers, referable to the two main formations explored with soundings, Plio-Pleistocene marine and eluvio-colluvial deposits, were obtained from:

- classification laboratory tests carried out on 54 disturbed samples from both formations; oedometric tests, resonant column coupled with shearing torsional tests performed on 3 undisturbed samples from the eluvio-colluvial formation;
- SPT and Down-Hole tests.

The soil stratigraphy; the existence of a geotechnical borehole defining the soil stratigraphy;

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- SPT and Down-Hole tests.

The minimum and maximum values of the main geotechnical properties (unit weight, $\gamma$, solid unit weight, $\gamma_s$, water content, $w$, liquid limit, $w_l$, plasticity index, $I_p$, void ratio, $e$, overconsolidation ratio, OCR) are synthesised in Table 2.

For the strain-dependent shear moduli, G, and damping ratios, D, different relationships were adopted for each soil type:

- for eluvio-colluvial (and debris) soil layers, the following empirical models [15], fitted to the experimental measurements from resonant column and torsional shear tests, were utilised (Fig. 3):
The bedrock depths were defined as follows:
- at site 1, where marl formation was not encountered, the depth was set to correspond to a $V_s$ value greater than 800 m/s (at a depth of 21 m);
- at sites 2 and 3, where marl formation was not found and $V_s$ is generally lower than 700 m/s, with no significant impedance ratio values, a conventional bedrock depth was estimated by extending the linear trend observed in $V_s$-profiles to a depth corresponding to $V_s$-values equal to 800 m/s (at depths of 28 m and 27 m respectively);
- at site 4, where marl formation is found, the depth was set at 16 m.

Level 1 and level 3 ground motions were utilised as input motions for numerical modelling.

PROSHAKE analyses results were processed in a time and frequency domain to obtain:
- maximum acceleration and maximum shear strain with depth for each soil column examined;
- elastic response spectra at the bedrock (outcropping) and the ground surface;
- amplification functions and amplification factors of the deposit

Maximum peak ground acceleration values, computed for level 1 input motions (Fig.4a), range between 0.18g and 0.26g at the soil deposit surface of the four examined sites; the highest value (0.26g) was obtained at site 1, the lowest value at site 2. Consequently, the maximum ratio

![Fig.2: Geotechnical boreholes profiles with sample location and Vs values from in situ (down-hole) and laboratory (resonant column and shear torsional) tests](image)

\[
\frac{G}{G_0} = \frac{l}{1 + 20 \cdot \gamma^{0.9 \times 8}} \\
D = 33 \cdot e
\]  

For Plio-Pleistocene marine soil layers, the experimental values obtained by the Authors for other deposits of Central Italy having the same geologic origin were adopted (referred to as (2) in Fig. 3);
- for marl, the experimental shear moduli and damping ratio variation laws proposed by CNR-GNDT [4]

**TABLE 2: Average geotechnical properties of the main geological formations (EC = eluvium and colluvium, MAR = Plio-Pleistocene marine deposit)**

<table>
<thead>
<tr>
<th>Formation</th>
<th>$\gamma$ [kN/m$^3$]</th>
<th>$\gamma_s$ [kN/m$^3$]</th>
<th>$W$ [%]</th>
<th>$W_1$ [%]</th>
<th>$I_F$ [%]</th>
<th>$e$ [-]</th>
<th>OCR [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>EC</td>
<td>min 20.7</td>
<td>23.3</td>
<td>20.5</td>
<td>37</td>
<td>67</td>
<td>38</td>
<td>0.55</td>
</tr>
<tr>
<td></td>
<td>max 20.8</td>
<td>27.0</td>
<td>24.1</td>
<td>67</td>
<td>38</td>
<td>57</td>
<td>0.57</td>
</tr>
<tr>
<td>MAR</td>
<td>min 20.4</td>
<td>26.2</td>
<td>30</td>
<td>10</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>max 20.7</td>
<td>26.9</td>
<td>16.9</td>
<td>46</td>
<td>23</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Fig. 3: Shear modulus (a) and damping ratio (b) versus shear strain for the eluvium and colluvium (1), Plio-Pleistocene marine deposits (2) and marl bedrock (3)
between the PGA value at the soil surface and the PGA value on outcropping rock (0.173g) is about 1.5 at site 1, while the minimum ratio is about 1 at site 2.

Maximum shear strain with depth ranges from 0.009% to 0.024% for level 1 (Fig. 5a) input motions, and from 0.002% to 0.005% for level 3 at the four sites (Fig. 5b). Since maximum values are reached within soil type (2) at sites 2 and 3, it can be observed (Fig. 3) that the corresponding reduction in shear modulus is lower than 10%. Consequently, soil behaviour is approximately linear at each depth of the examined profiles and the amplification functions are almost independent from the input motion at each of the four sites considered.

On the basis of stratigraphy and V_s-values, one or more EC8 soil types were attributed to each soil profile of Fig. 2. At Site 1, soil profile was of type E; at Sites 2, 3, 4, soil profiles could be considered to be of type C or type E. Therefore, both C and E type spectra were superimposed on those calculated.

EC8 suggests two sets of different shapes for elastic acceleration response spectra, in relation to surface wave magnitude, M_s. As Offida is located in a medium-seismicity environment, both Type 1 (for M_s > 5.5) and Type 2 spectra (for M_s ≤ 5.5) were compared with numerical results. The comparison, shown in Fig. 6, 7, 8 and 9, was limited to level 1 of the seismic motion, since the energy content of the level 3 input motion was considered too weak for engineering purposes.

As is shown in Fig. 6, at site 1 for periods below 0.4 sec, Type 2 spectrum agrees more with numerical results relative to EW Pollino earthquake than Type 1 one, and agrees less with NS ones. At periods in the range 0.4 – 0.8 sec the situation is reversed, and Type 1 spectra fit numerical predictions with NS component better than Type 2 ones.
At periods above 0.8 sec, the results obtained with both motions are practically the same, and inferior to Type 1 and Type 2 spectra. As can be seen in Fig. 7, 8 and 9, very similar results are obtained at the other sites. The EC8 spectra for C and E soil profiles are also comparable. As far as concerns the influence of time histories on numerical responses, at low periods the highest amplifications were observed for the EW component; at periods between 0.4 -0.8 sec the higher amplifications are obtained for NS time history; at higher periods (> 0.8 sec) the responses are practically the same for the two components. It must be outlined that Pollino horizontal acceleration components have very different Fourier spectra (Fig.10). EW spectrum shows a broader frequency band, with two pronounced peaks in the ranges of 2-3 Hz and 6-7 Hz; NS Fourier spectrum has a well defined resonance frequency at 6 Hz, with a typical noise pattern at the other frequencies. Therefore, EW acceleration time history appears to be more representative of seismic ground motion for engineering purposes.

The comparison between numerical predictions and observed ground motions was made in terms of amplification function and amplification factor. It must be noted that the amplification functions from 1-D numerical analyses were computed as ratios between deposit surface and outcropping rock responses by considering Level 1 and Level 3 ground motions, whereas those from field data were calculated by means of the above mentioned SSR and HVSR techniques.

The results of the comparison are shown in Fig. 11, 12, 13 and 14. It can be observed that the amplification functions from microtremors and microseism recordings in terms of fundamental frequencies are quite similar to each other, and present generally similar patterns. On the contrary, surprisingly, the amplification functions from numerical analyses and from field instrumental data are quite different, both in terms of fundamental frequencies and amplification ratio peak values.

This disagreement could be due either to three-dimensional effects or, more probably, to the overconsolidated nature of upper layers of the soil deposits (i.e. overconsolidation ratio values, OCR ≅ 3.5 have been measured at 4.5 m of depth in the eluvium and colluvium). Several Authors [16] have found that geotechnical and seismological approach can provide amplification functions which are substantially different in terms of fundamental frequencies and amplification values for overconsolidated residual soil deposits, which, on the contrary, were found comparable, at least in terms...
of fundamental frequencies, for normal-consolidated soils [8].

To estimate soil amplification also by means of an integral parameter of the motion, an amplification factor or site coefficient, $F_a$, for each of the investigated sites was defined as the ratio between the area under the pseudo-velocity response spectrum at the ground surface and at the outcropping bedrock, computed within a certain frequency interval. Since most structures have a fundamental period of between 0.1 and 2 seconds, corresponding to a natural frequency of 10 and 0.5 Hz, amplification factors were computed within this interval of frequency. The values calculated on the results of numerical analyses were compared with those obtained from the instrumental data.

The results obtained are summarized in Table 3, where maximum and minimum $F_a$ values from the instrumental data and from both input motion levels are reported for each site. It can be observed that the lowest values of maximum $F_a$ are associated to level 1 of the input motion, the lowest values of minimum $F_a$ correspond to the instrumental data, the values obtained from numerical analyses with level 3 of the input motion are very high and the corresponding ranges are wide. The average highest values of $F_a$ were found at site 1 in all three cases, while the average lowest value is different for each case. Thus, based on the geological features and on the results of seismological and geotechnical approaches, either in terms of a single parameter or an integral parameter of the motion, the microzonation map of Offida shown in Fig. 15 was obtained as a support for the regional guidelines for building design.

**TABLE 3:** Minimum and maximum values of the amplification factors from 1-D modeling and from instrumental data

<table>
<thead>
<tr>
<th>Site</th>
<th>Numerical analyses</th>
<th>Instrumental data</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Level 1</td>
<td>Level 3</td>
</tr>
<tr>
<td>Site 1</td>
<td>1.12</td>
<td>1.19</td>
</tr>
<tr>
<td>Site 2</td>
<td>1.05</td>
<td>1.11</td>
</tr>
<tr>
<td>Site 3</td>
<td>1.09</td>
<td>1.15</td>
</tr>
<tr>
<td>Site 4</td>
<td>1.07</td>
<td>1.14</td>
</tr>
</tbody>
</table>

**CONCLUSIONS**

A quantification of ground responses on an urban scale in a historical centre of the Marche region, Italy, was carried out in order to update the seismic building code and to draw up a microzonation map, by using microseismes and microtremor recordings and site-response models. The reliability of Type 1 and Type 2 EC8 spectra for the site was also analysed. The main conclusions of the research are:

1. the comparison in terms of amplification functions of numerical and instrumental results does not show consistent results; this disagreement, however, could

![Fig. 11 – Site 1: amplification functions from PROSHAKE analyses and from instrumental recordings](image1)

![Fig. 12 – Site 2: amplification functions from PROSHAKE analyses and from instrumental recordings](image2)

![Fig. 13 – Site 3: amplification functions from PROSHAKE analyses and from instrumental recordings](image3)

![Fig. 14 – Site 4: amplification functions from PROSHAKE analyses and from instrumental recordings](image4)
be due either to three-dimensional effects or, as observed by other Authors, to the particular stress history of the overconsolidated Offida deposits;

2. numerical predictions and instrumental results are on the contrary quite comparable (with only one insignificant exception, when level 3 input motion is considered) in terms of amplification factors (that is, of site coefficients $F_a$); therefore, as the seismological investigations covered a broader area than that investigated by means of one-dimensional models, instrumental data were used for assigning site coefficients to the various zones of Offida and drawing up a microzonation map;

3. comparison of input motions shows that soil behaviour is essentially linear also for level 1 ground motions; since EW Pollino horizontal acceleration time history is more representative and significant for engineering decision making than the NS component, in order to define a design site spectrum, reference was made mainly to the modelling results obtained by using its acceleration history.

4. the comparison among EC8 elastic acceleration spectra and numerical predictions at the various sites suggested that in order to study a design spectrum for building design at Offida, the spectrum Type 2 of EC8 best interprets the numerical predictions and ensure adequate protection at limited costs. In fact:
- for seismic reinforcement and repair of the constructions in the Offida historical centre, where buildings generally have two or three floors, EC8 Type 2 spectrum is more conservative than Type 1 spectrum in the range of periods $T<0.4$ sec;
- for new constructions, located in the new development area, Type 1 spectrum is certainly more conservative for periods higher than 0.4 sec. However, it could become too conservative, thus implying excessively high building costs.

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