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# SEISMIC MICROZONATION OF URMIA, IRAN

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## ABSTRACT

Iran is in a high seismic area and almost all of the country is subject to earthquake hazard. In recent decades, Iran has suffered drastic damage and fatalities caused by earthquakes. The main cause of damage is due to seismic design code deficiency. This has triggered many seismic microzonation studies in several large cities. Urmia city located in the north west of country is one of the Iran's most populated and strategic cities with a relatively high seismic hazard that makes it necessary to conduct comprehensive hazard studies such as seismic microzonation. In this paper, results of seismic microzonation of Urmia City are presented. One dimensional nonlinear dynamic analyse using synthetic acceleration time histories synthesized according to results of probabilistic seismic hazard analysis (PSHA), is performed. Geological, geotechnical and geophysical settings of Urmia basin have been described according to information gathered from more than 100 boreholes and field tests. The results show that the amplification factor in the central and eastern part of city is high due to relatively deep sediment. A liquefaction assessment, based on Geotechnical and SPT values, was performed. Soil liquefaction hazard is seen in eastern part of city under SL2 level of earthquake with return period of 2475 years.

## INTRODUCTION

Iran is founded in a seismic prone area and almost all of the counytry is subjected to earthquake hazards. Be that as it may, it is clear that earthquake magnitude, intensity, and seismic risk as a function of these parameters, are not equal in all of Iran's regions. Even the most important characteristics of an earthquake like duration and frequency content that depends on faulting mechanism, return period of activity, and site soil condition are vary widely from city to city.

Regarding the difference between seismic hazard and seismic risk, even if two cities have the same seismic hazard, they may have different seismic risk, because of the parameters involved in the assessment of seismic risk. With this point of view these parameters are divided into two categories:

- Social parameters like population, cultural condition, economic condition, and geographic condition.
- Parameters like local soil condition and urban planning fall into the second category.

Seismic microzonation studies focus on second category parameters as input data. The output of study procedure gives us precise maps that picture the seismic condition of the city in each block. Depending on the level of microzonation the size of blocks can change from meters to kilometres. In

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the next phase, microzonation studies output data can be used as input data in order to analysis several engineering demands and social events both for strategic crisis management and structural design or local seismic design criteria. For instance, one can perform fragility analysis for lifeline systems and structures embedded on liquefiable soil with assassin liquefaction probability using accurate models like MFS and an artificial acceleration time history synthesized according to the local site uniform hazard spectrum. One can deduct criteria for that region or propose a retrofit plan for existing structures. In this paper a brief description of Urmia city and a short review of literature on microzonation has been provided. Seismicity and seismotectonical information for Urmia city, like faults mechanism, faults length, and historical earthquakes pertinenet to each fault have been presented. As seen in Fig.1 there are many active faults surrounding the city so one can expect a high seismicity level in this area. Finally, by conducting a probabilistic seismic hazard analysis (PSHA) using attenuation relations that are most pertinent to the seismotectonical condition of the study area, a series of contour maps for peak ground acceleration, peak ground velocity and peak ground displacement for various return periods have been prepared. The procedure used for PSHA analysis is similar to those demonstrated in literature (McGuire, 1995).

## SEISMIC MICROZONATION

In recent years, due to globalization, economic downturn, and environmental impacts of development, attention to the concepts of sustainable development has grown all around the world. Sustainable development consists of many aspects such as urban design, environmental aspects, as well as political and social development. Making systems, including infrastructure, social systems and economic systems, resilient is one of the most important goals in implementation of sustainable development policies. On other hand GIS based microzonation is a potent tool to develop strategic plans that can account for these policies.

Seismic microzonation studies begin by conducting a seismic hazard analysis (SHA) for to chosen region. If results have been calculated for the seismic bedrock level, then one must perform a site response analysis using one of the following methods:

A) Comparing acceleration time histories recorded at soil sites with those recorded at rocky sites.
B) Conducting numerical modelling and solving these models for a given hazard (Cid et al.; 2001).

Knowing the site effect, one can produce high-resolution hazard maps. Then the corresponding damage to structures and infrastructure is calculated using existing procedures (Milutinovic and Trendafiloski; 2003). Japanese and American engineers in early 70's used the procedure that is known as microzonation (Kandpal et al.; 2009). Since then, many other countries have conducted similar studies. More recently the functionality of microzonation studies has increased by using GIS based software (Ganapathy; 2011)

## SEISMIC HAZARD NALYSIS

Determining seismic sources and events, then making a seismotectonic map is the first step in seismic hazard analysis. This map contains the location of these sources and previous earthquakes with certain magnitudes. Then, according to the number of earthquakes with a certain magnitude, a regional recurrence model is developed. Attenuation relationships pertinent to the tectonic settings of the study region are selected. Finally a uniform hazard design spectrum as an output of study is calculated (Cornell; 1968).

*Serow* fault is the most important seismic sours of Urmia city's area. It has a normal mechanism and is located at a distance of 23 km from city Fig.1. A maximum magnitude that it can generate is 7 in surface wave magnitude scale. The type of past earthquakes that have occurred in this area are shallow crustal events. For instance, *Dehkharghan* earthquake in February 1641 had a surface wave magnitude of 6.8 and the *Marand* earthquake in October 1786 had a surface wave magnitude of 6.3.

In this study, three attenuation relationships (Akkar and Bommer; 2010, Abrahamson and Silva; 2008, Campbell and Bozorgnia; 2008) have been used. This type of relationship has developed for shallow crustal zones all around the world. According to uniform hazard spectrums, synthetic acceleration time histories have been generated. In addition, a logic tree approach is adopted in order to reduce uncertainty in attenuation relationships.

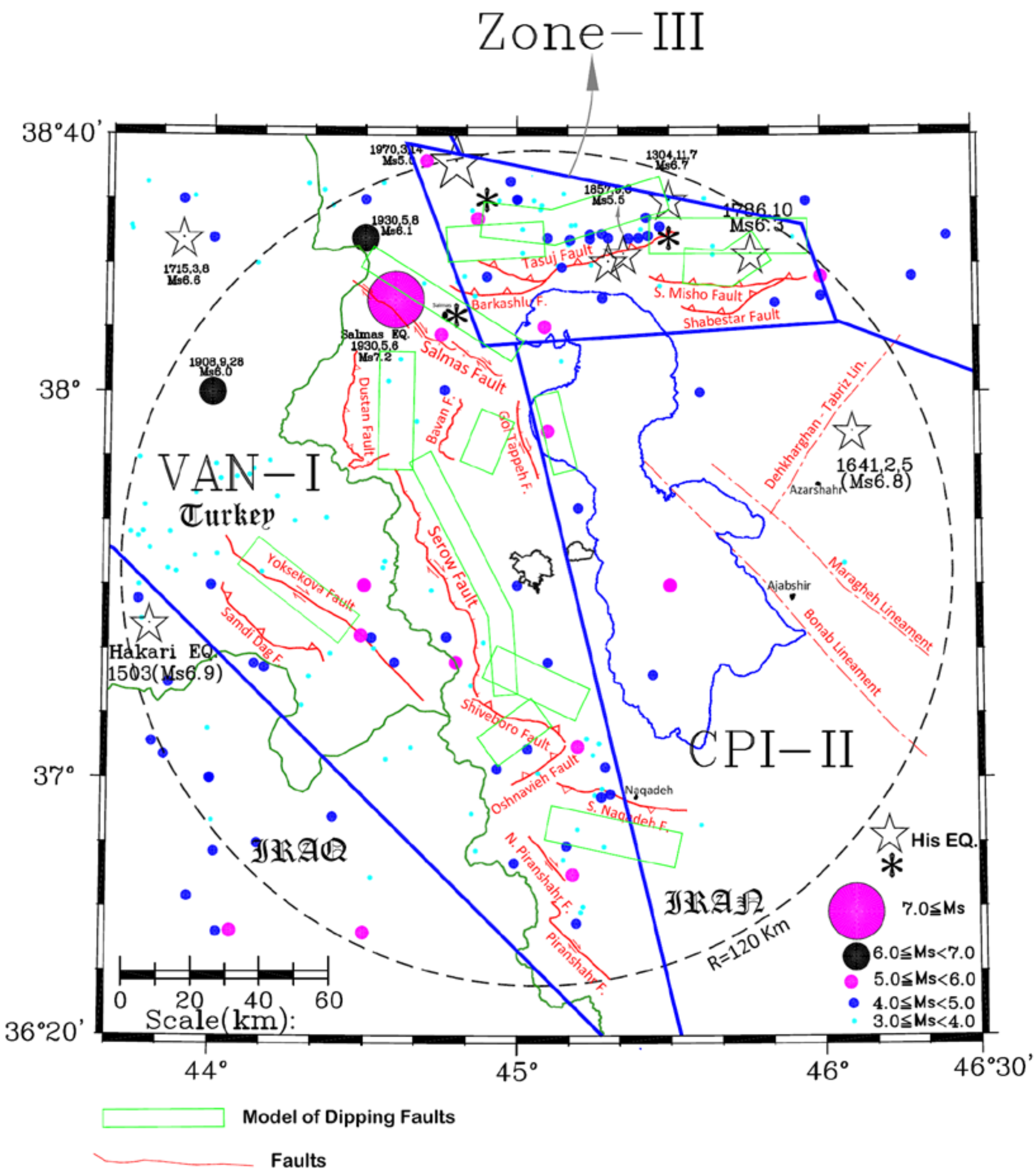


Figure 1. Seismotectonic Map of Urmia city including faults and past earthquakes location

The results of seismic hazard analysis are provided for three return periods, namely, 75 years, 475 years, and 2475 year events in terms of peak ground acceleration, velocity, and displacement in seismic bed rock level. An example of these results – peak ground acceleration in seismic bedrock level with a 475 years return period – is shown in Fig.2.

In the next step, according to the uniform hazard spectra for each part of city, synthetic acceleration time histories are generated. Following, a brief description of acceleration time history synthesizing is provided. The process of simulating a synthetic time history starts from a real event. Then adapting its frequency content, using the Fourier Transformation Method to match the target spectrum, called the Uniform Hazard Response Spectrum, is obtained from the PSHA. The important factor in choosing real acceleration is the similarity of real time history spectrum with the calculated target spectrum. Because this method can lead to an unrealistic artificial time history when the target response spectrum differs from the real time history spectrum the procedure developed by Deodatis

(1996) and coded by Papageorgiou et al. (2000) was utilized. In this study five simulated acceleration time histories for each region were used for site response analyses and the average of the calculated acceleration response spectra were used.

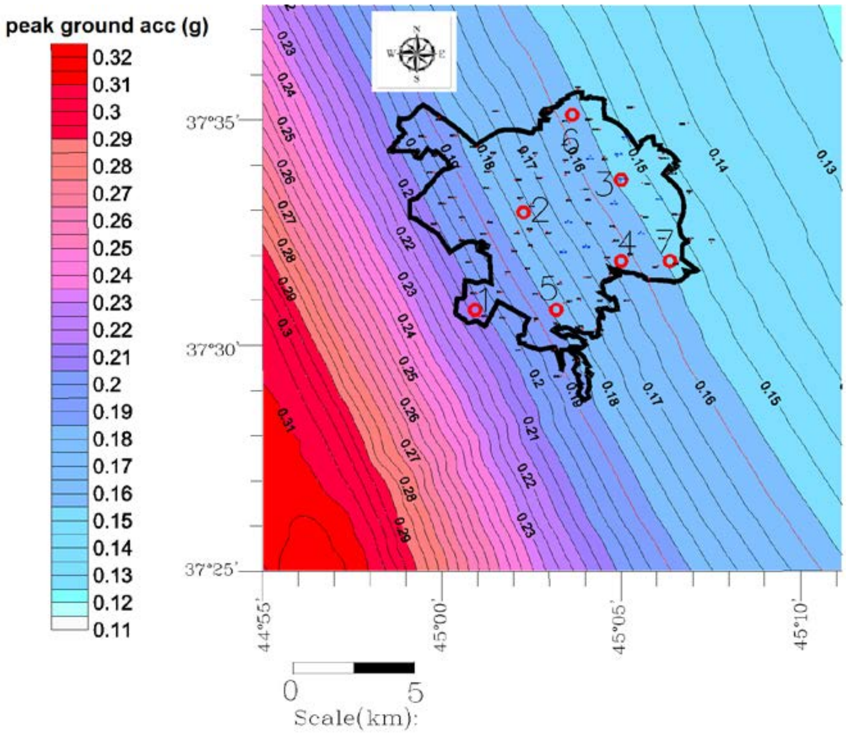


Figure 2. Seismic hazard analysis results for seismic bedrock (peak ground acceleration for a return period of 475 years)

## GEOLOGICAL SETTING

In this study, digging more than 100 boreholes, an exhaustive underground investigation including SPT measurements, geophysics related parameters measurements, soil layer mechanics and dynamic properties of each layer are performed. Locations of boreholes are illustrated in Fig. 3.

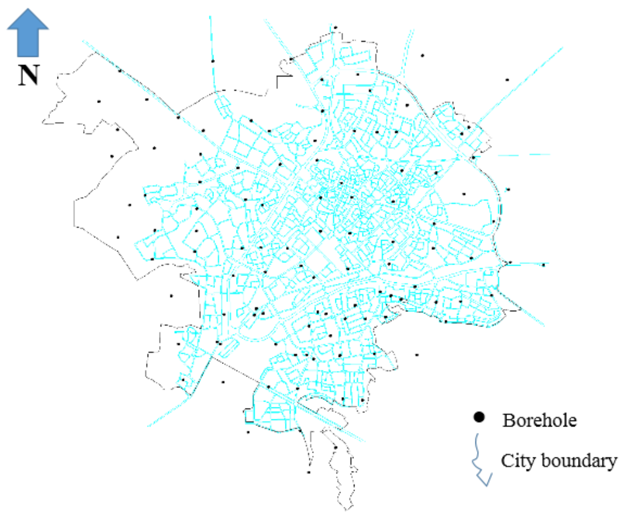


Figure 3. Location of drilled boreholes within the Urmia city

In addition to subsurface soil layers properties, surface geology plays a crucial role in site effect Frankel et al. (2002) computed soil-to-rock spectral ratio for 35 locations using recordings of the Nisqually earthquake (M=6.8) and its aftershock (M_L=3.4) to study site response and basin effects in the region. They found that site amplification was correlated to surface geology and the nonlinearity of soft soils. Whit studying Miyagi-Oki earthquake Tsuda et al. (2006) observed that nonlinear soil behaviour occurred at the stations with a softer near-surface velocity structures. Average shear wave velocity of the top 5 meter soil profile within the study area is shown in Fig. 4.

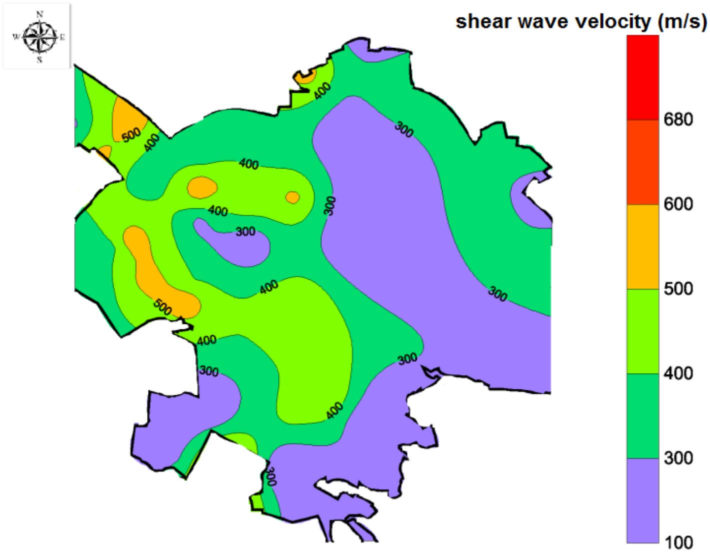


Figure 4. Average shear wave velocity of the top 5 meter soil profile within the study area

As it can be inferred from Fig.4, there are soft surface soil regions in the central and southern part of city that may cause high permanent ground deformation in these regions and associated lifeline damage.

## LIQUEFACTION ASSESSMENT

Liquefaction potential is one of the most important aspects in seismic microzonation. There are two methods that have been used for assessing liquefaction potential of soils:

A) Laboratory tests on intact samples
B) Experimental methods.

Unlike experimental methods, the former method is very difficult to perform especially because of the difficulty in providing intact samples. There are four approaches to experimental methods:

1) SPT based assessment
2) CPT based assessment
3) BPT based assessment
4) Shear wave velocity based assessment.

Among these approaches, SPT based assessment has been widely using in liquefaction studies under SL2 level of earthquake with return period of 2475 year. As can see in Fig.5, the eastern and northern part of city are susceptible to liquefaction (red points) and in the southern and western part of city due to very low underground water table level and soil type, liquefaction will not occur (blue and green points).

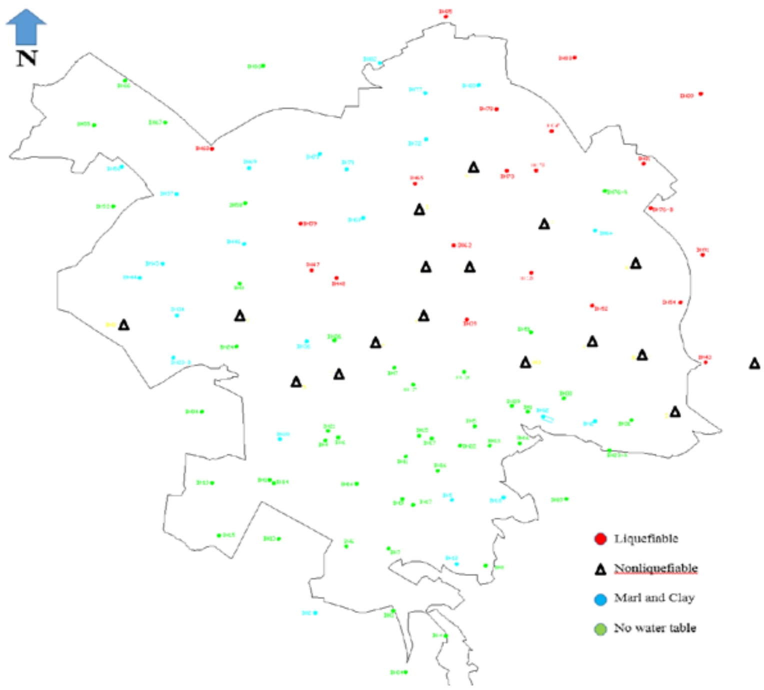


Figure 5. Susceptibility of different locations of city to liquefaction (red points: liquefiable)

## SITE EFFECT

According to the studies down by Streeter et al. (1974) and Fin et al. (1978) the seismic response of soils is overestimated when using equivalent linear approach. This happens due to pseudo-resonance at periods corresponding to the strain-compatible stiffness used in the final elastic iteration analysis. Additionally, in the equivalent linear analysis material behavior remains elastic so it cannot produce permanent deformations. In addition, soil nonlinearities seem to be more significant in the frequency band between 0.5 to 4 Hz (Darragh and shakal, 1991; Field et al, 1997; Beresnev, 2002) and have a significant effect on motions with a PGA value greater than 0.1 – 0.2g and shear strain larger than $10^{-5}$ to $10^{-4}$ (Chang et al, 1989; Chin and Aki, 1991; Beresnev and Wen, 1996; Trifunac and Todorovska, 1996).

With respect to the condition that we have in Urmia city (peak ground acceleration is greater than 0.2 g) it is necessary to conduct a nonlinear dynamic approach. This approach will be explained in following subsections. This section can be divided into three parts. Namely, soil model, one dimensional site response and numerical formulation of solution.

- Soil model

In the early 1960s a soil model proposed by Konder and Zelasko (1963) is presented in Eq. (1):

$$\tau = \frac{G_{mo}\gamma}{1 + \left(\frac{G_{mo}}{\tau_{mo}}\gamma\right)} \tag{1}$$

Where τ is the shear stress; ϒ the shear strain; $G_{mo}$ the initial shear modul; $\tau_{mo}$ the shear stress at approximately $1\%$ shear strain. After that Matasovic (1993) modified this model by adding parameters $\beta$ and $s$ and proposed a new model that can represent a wider range of maser soil behaviour as shown in Eq.(2) as following:

$$\tau = \frac{G_{mo}\,\gamma}{1+\beta\left(\frac{G_{mo}}{\tau_{mo}}\gamma\right)^{s}} = \frac{G_{mo}\,\gamma}{1+\beta\left(\frac{\gamma}{\gamma_r}\right)^{s}}, \qquad (2)$$

In this model $\gamma_r = \tau_{mo} / G_{mo}$ is the reference shear strain and considered as a soil constant parameters (Hardin and Drnevich, 1972). After that Hashash and Park (2001) introduce a new formulation for the reference strain, $\gamma_r$, to capture the influence of confining pressure on modulus degradation and damping ratio by the Eq.(3):

$$\gamma_r = a\left(\frac{\sigma'}{\sigma_{ref}}\right)^{b} \qquad (3)$$

Where a and b are curve fitting parameters and $\sigma_{ref}$ is reference confining presser of $0.18\ Mpa$.

- One dimensional site response

The main assumption in one dimensional site response analysis is the propagation of SH waves (Horizontal Shear Waves), an approximation of earthquake ground motion, through a horizontally layered sediment. In this situation, the behaviour of soil can be modelled as a Kelvin-Voigt model. In early 1970s an equivalent linear method was proposed to model nonlinear cyclic response of soil for a certain ground motion acceleration time history and initial values and curves for module reduction and damping for each layer (Schabel et al., 1972). This approach needs an iteration scheme to solve the problem. After that studies done to involve dependency of soil parameters to pressure and frequency (Sugito, 1995; Assimaki et al., 2000). The advantage of these new models was the capture of some high frequency components of strong ground motions that were filtered by older models. However, these models do not address the full range of soil nonlinear behaviour like module degradation due to the number of loading cycles, residual strain of soils and pore pressure generation.

- Numerical formulation and time domain solution

The equation of motion for one-dimensional propagating shear waves through an unbounded medium can be written as Eq. (4):

$$\rho\,\frac{\partial^2 u}{\partial z^2} = \frac{\partial \tau}{\partial z} \qquad (4)$$

Where ρ = density, τ = shear stress, u= displacement and z= depth below the ground surface.
Soil behaviour is approximated as a Kelvin – voigt solid. The shear stress – shear strain relationship is expressed as Eq.(5):

$$\tau = G\gamma + \eta\,\frac{\partial \gamma}{\partial t} \qquad (5)$$

Where G= shear moguls, $\gamma$ = shear strain and $\eta$= viscosity.
Substituting Eq.4 to Eq.5 results in Eq.(6):

$$\rho\,\frac{\partial^2 u}{\partial t^2} = G\,\frac{\partial^2 u}{\partial z^2} + \eta\,\frac{\partial^3 u}{\partial z^2\,\partial t} \qquad (6)$$

In time domain analysis the soil column is idealized as discrete lumped mass system as sown in Fig. 6 (Matasovic, 1993). The wave propagation equation is written as Eq.(7):

$$[M]\{\ddot{u}\} + [C]\,\{\dot{u}\} + [K]\,\{u\} = -\,[M]\,\{I\}\,\ddot{u}_g \qquad (7)$$

Where $[M]$ = mass matrix; [C] = damping matrix; [K] = stiffness matrix; $\{\ddot{U}\}$ = vector of nodal relative acceleration; $\{\dot{U}\}$ = vector of nodal relative velocity and $\{U\}$ = vector of nodal relative displacement. $\ddot{U}_g$ Is the acceleration at the base of sediment and {I} is the unit vector. Eq 4, is solved numerically at each time step using the Newmark $\beta$ method (Newmark, 1959). The Newmark $\beta$ (average) method is unconditionally stable and dose not introduces algorithmic damping.

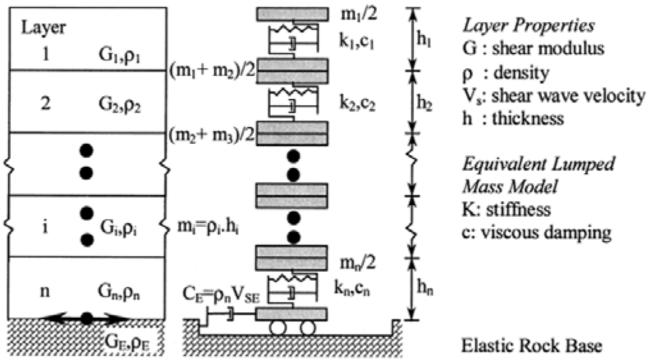


Figure 6. Idealized soil column as discrete lumped mass system (Matasovic, 1993)

Each individual layer $i$ is represented by a corresponding mass, a nonlinear spring and a dashpot for viscos damping. The [K], [M] and $[C]$ matrices are assembled using incremental properties of the soil layers. This procedure needs a constitutive model that describes the cyclic behaviour of soil. The mass matrix forms by Lumping half mass of each of two consecutive layers at their common boundary and in order to consider nonlinearity of soil, the stiffness matrix is updated at each time increment and for each layer $i$, is defined as Eq.(8):

$$K_i = \frac{G_i}{h_i} = \frac{\Delta\tau_i \gamma_i}{h_i \Delta\gamma_i} \tag{8}$$

Where $G_i$ is the shear modulus and $h_i$ is the thickness of layer $i$.
Matrix [C] represents additional viscos damping at very small strains where the soil behaves linear. This matrix is frequency dependent. In the original damping formulation proposed by Rayleigh and Lindsay (1945), this matrix is a combination of mass and stiffness matrices that can represented as Eq.(9):

$$[C] = a_0[M] + a_1[K]. \tag{9}$$

But here the damping matrix [C] is dependent on natural mods of soil column in addition to mass and stiffness. Scalar values of $a_0$ and $a_1$ can be computed using tow significant mods $m$ and $n$ by Eq.(10) as following:

$$\begin{bmatrix}\varepsilon_m \\ \varepsilon_n\end{bmatrix} = \frac{1}{4\pi}\begin{bmatrix}1/f_m & f_m \\ 1/f_n & f_n\end{bmatrix}\begin{Bmatrix}a_0 \\ a_1\end{Bmatrix} \tag{10}$$

Usually in site response analysis the natural frequency of selected mode calculated using Eq.(11) (keramer, 1996)

$$f_n = \frac{V_s}{4H}(2n-1) \tag{11}$$

Where $n$ is mode number and $f_n$ is the natural frequency of the corresponding mode
Equation 11 results in a frequency dependent damping ratio even if the specified damping ratios in selected modes are equal. Since soil behaviour in medium to strong earthquakes like those predicted for Urmia region is inelastic, the linear and equivalent linear solutions are no longer valid. In this study in order to capture important parameters mentioned earlier, a nonlinear approach that solves dynamic equation of motion in discreet time increments has been used. In this approach the dynamic equation of motion is integrated in time domain and soil nonlinearity can accurately modelled.
According to guidelines proposed by Park and Hashash (2004), to performing nonlinear site response layer thickness must calculate by using Eq.(12) to avoid filtering of relevant frequencies.

$$f_{max} = \frac{(V_s)_i}{4h_i} \tag{12}$$

Where $f_{max}$ = maximum frequency that layer $i$ can propagate, $(V_s)_i$ = shear velocity, $h_i$ = thickness of each layer.

It is a common practice to set the maximum frequency to $25\ Hz$ in a non-linear site response analysis (Hashash et al., 2011). The frequency used here is equal to $f = 60Hz$.

Since the conventional guideline of using the first and higher mod of soil column or predominant period of the input motion will not always result in a good match whit the linear frequency domain solution. Appropriate frequencies for the viscos damping formulation are chosen based on the linear analysis for each soil profile and each input motion.

## RESULTS

Typically seismic microzonation results are presented by counter maps showing different hazards such as peak ground parameters (i.e. PGA, PGV, and PGD), spectral acceleration, and susceptibility of soil to liquefaction, within the study area. In this paper peak ground acceleration for three return period is presented.

- **Peak ground acceleration**

Peak ground acceleration in the ground surface level is one of the important parameters in field of earthquake engineering. Many engineering design parameters (EDPs) are related to PGA and many correlations between damage index in different structures and earthquake intensity in terms of PGA have represented. Peak ground acceleration in ground surface within the study area in three return periods, are presented in Fig.7, 8, 9.

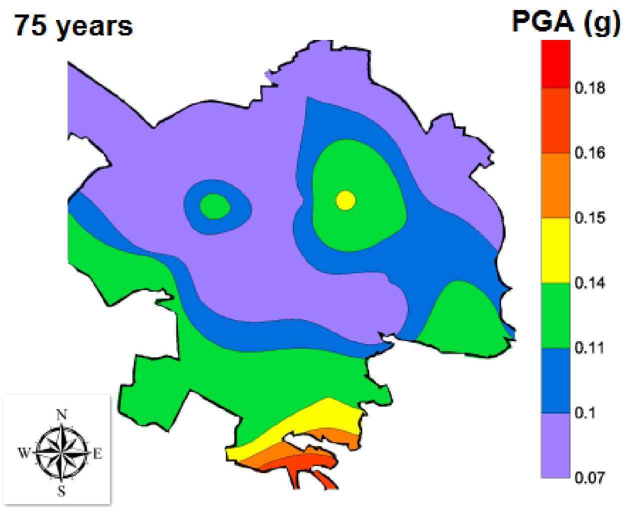


Figure 7. Peak ground acceleration in ground surface level for 75 years return period

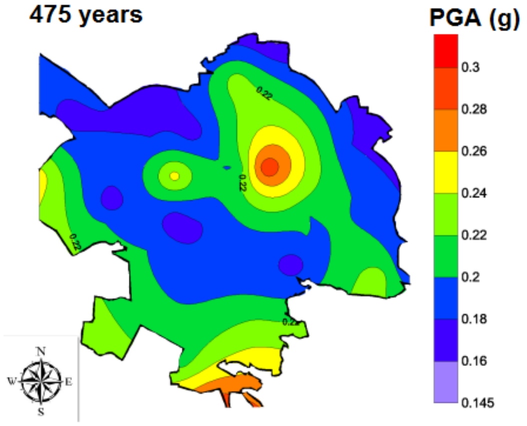


Figure 8. Peak ground acceleration in ground surface level for 475 years return period

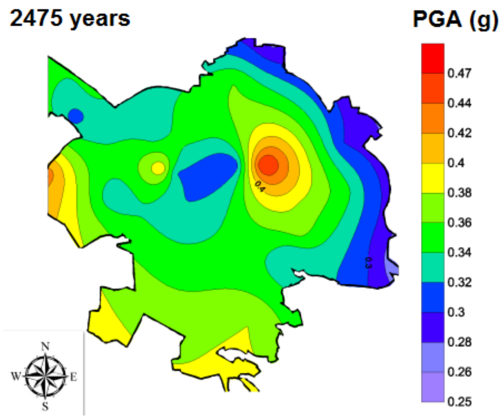


Figure 9. Peak ground acceleration in ground surface level for 2475 years return period

- **Spectral acceleration**

Peak ground acceleration cannot describe soil amplification carefully. A more precise way to illustrate microzonation results is by the spectral acceleration. The spectral acceleration for a period of 0.5 second for 475 years return period is shown in Fig.10.

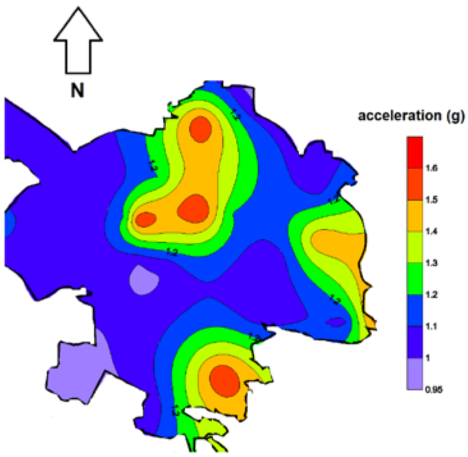


Figure 10. spectral acceleration for 475 year return period

## CONCLUSIONS

Seismic microzonation of Urmia city has been presented in this paper. Geological and geotechnical settings of Urmia basin have been described according to information gathered from digging more than 100 boreholes in Urmia city and conducting field tests such as SPT and down hole test. A one dimensional nonlinear soil column analysis has been performed using a modified hyperbolic model. The use of a nonlinear approach provides an opportunity to obtain more accurate results and account for soil properties that cannot be captured by means of an equivalent linear approach. Results indicate a high hazard zone in central part of Urmia city, due to deep alluvia, and a liquefaction susceptible area in eastern part of city. This may cause severe damage to infrastructures such as bridges and lifelines in these regions, and calls for more strategic planning before the next earthquake.

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