



Seismic Microzonation of Islamabad–Rawalpindi Metropolitan Area, Pakistan

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Abstract—Microzonation deals with classifying seismic hazards in terms of ground motions resulting from amplification of seismic waves by nature of soil profiles underlying a site, town or city. This paper presents the results of microzonation study for Islamabad metropolitan, the capital of Pakistan. Cumulative SPT-*N* values from geophysical borehole and microtremor (Tromino Engy Plus) data were used to classify the soils into classes C (very dense soil profile and soft rock) and D (stiff soil profile) as devised by the National Earthquake Hazard Reduction Program (NEHRP). Soil response analyses were carried out based on scaled time histories of Kashmir earthquake (2005, 0.02 g), Mangla earthquake (2006, 0.031 g) and Haripur earthquake (2010, 0.13 g) corresponding to return periods of 150, 475, 975 and 2475 years. Spectral accelerations on the ground surface are calculated by two different approaches (1) soil response analysis performed using one dimensional shear wave propagation method (equivalent linear approach); and (2) NEHRP and Borchardt amplification factors. Microzonation maps are produced with respect to ground shaking intensity for the return periods of 150, 475, 975 and 2475 years taking into account the variation of the spectral accelerations calculated based on these two procedures. The results show that the accelerations at the ground surface in the Islamabad–Rawalpindi metropolitan are in the range of 0.40–0.48 g (for 150 years), 0.59–0.65 g (for 475 years), 0.71–0.77 g (for 975 years), and 0.92–0.94 g (for 2475 years). The amplification factors for these four hazard levels range from 0.96 to 1.38 (150 years), 0.90–1.14 (475 years), 0.85–1.04 (975 years) and 0.84–1.00 (2475 years).

Key words: Peak ground acceleration, ground shaking intensity, microzonation, Islamabad, Pakistan.

1. Introduction

The ground accelerations at a site primarily depend upon three factors: the earthquake magnitude, source to site distance and crustal property of the region as defined by the attenuation relations. A

fourth factor is the local site effect that primarily refers to various geological and geotechnical characteristics of the site including soil/rock types and geomorphology. These parameters make a fundamental component of the detailed microzonation maps at the scale of a site, town or city. With enhanced computational capabilities in evaluating seismic risks in the last two decades, this approach is being increasingly used in constructing probabilistic microzonation maps at town/city levels or for sites hosting critical infrastructures like airports, power plants, dams, etc. (Ozcebe et al. 2004; Sextos et al. 2008).

The Islamabad–Rawalpindi area lies in a tectonically active zone (Seeber and Armbruster 1981). In A.D. 25, the Buddhist monasteries at Taxila, 25 km west-northwest of Islamabad, were destroyed by an earthquake estimated at Modified Mercalli Intensity (MMI) of IX (Sheikh et al. 2007). The 2005 (Ms 7.6) Kashmir earthquake epicenter at 110 km NW of Islamabad at Balakot–Muzaffarabad area caused considerable damages at Islamabad. The local earthquakes in the northern Potwar within or close to the Islamabad metropolitan included an M 5.8 earthquake on 14 February 1977, centered 7 km northeast of Rawalpindi that produced Modified Mercalli Intensity VII near the epicenter (Adhami et al. 1980).

2. Geological Setting

Geologically, the Islamabad–Rawalpindi is part of the Sub-Himalayas (Gansser 1964), which in Pakistan is defined by the Potwar plateau and the Salt Ranges (Gee 1989). The Sub-Himalayas, as opposed to medium to high altitude mountains of the Lesser and Greater Himalayas occurring to the north of the Islamabad, represent the southern outskirts of the

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Himalayas characterized by narrow low-altitude hills and intervening broad valleys. Being part of the Himalayas and located at the northern fringes of the Potwar plateau, the Islamabad-Rawalpindi metropolitan area is underlain by important Himalayan-age fold and fault structures. The Islamabad capital territory (ICT) occupies the immediate foot-wall of the main boundary thrust (MBT), one of the principal Himalayan boundary faults. A segment of this regional boundary fault in the Kashmir region NE of the Islamabad produced the major (Ms 7.6) earthquake in 2005 (Bendick et al. 2007; Kaneda et al. 2008; Sayab and Khan 2010). Another major fault, termed Riwat Thrust, runs close to the SW edge of the studied area (Jadoon and Frisch 1997) that has also been associated with some historical earthquakes in the past. It is worth mentioning that in the 2005 earthquake, Islamabad–Rawalpindi city experienced considerable damages to buildings including collapse of a major 16-storey apartment tower (the Margala Tower) that caused a life loss of over seventy people (Express Tribune 2011).

Until 2007, no seismic hazard assessment was available specific to the Islamabad despite its importance as being the capital of Pakistan and headquarters of the defense forces. The seismic hazard zonation map of Pakistan in 1986 Pakistan Building code (PBC) classified Islamabad–Rawalpindi region as zone 2 (moderate damage corresponds to intensity VII of Modified Mercalli Intensity scale) (Naseer et al. 2010). Building Codes of Pakistan Seismic Provisions 2007 (BCPSP 2007) indicated that a realistic seismic factor for building design should probably be higher than that indicated on the seismic zoning map of Pakistan. For sensitive structures, the BCPSP (2007) recommended micro-level site-specific designs taking into account the strength of the underlying soil, bedrock and the distance from probable earthquake sources.

This study is focused on microzonation of the Islamabad–Rawalpindi city. The existing seismic hazard assessments for Islamabad and surroundings are mostly based on regional macrozonation (e.g., MonaLisa and Khawaja 2002; Ali and Khan, 2004; NORSAR and PMD 2006; BCPSP 2007; Bhatti et al. 2011; Hashash et al. 2012; Rafi et al. 2012; Qadri

et al. 2015). None of these can be classified as microzonation studies as these do not take into account the soil/subsoil conditions thus do not provide a realistic appraisal of seismic conditions to be incorporated in engineering practices.

3. Real and Scaled Time Histories

Time histories (accelerograms) are recorded during natural earthquake. Accelerogram is a treasure of information and characterizes ground motion in terms of many parameters including peak amplitude (Housner 1975), duration (Bolt 1973; Trifunac and Brady 1975) and Arias Intensity (Arias 1970). We selected three real-time records including 2005 Kashmir earthquake (PGA 0.02), 2006 Mangla earthquake (PGA 0.031) and 2010 Haripur earthquake (PGA 0.13) based on the criteria of Yagci and Ansal (2009), i.e., the site conditions (soil or rock), magnitude and epicentral distance ($20 \text{ km} \leq R \leq 60 \text{ km}$). Statistical analyses (deaggregation results) were conducted by assuming that all epicenters of instrumentally recorded earthquakes with magnitude $M \geq 4.0$ around Islamabad–Rawalpindi are possible epicenters for future earthquakes. By this approach, the epicenter distance (R) for $M_w = 5.98\text{--}6.75$ earthquake corresponding to return period of 475 years was determined as 5–21 km is likely scenario for hazard expected (Fig. 1). The transverse components of the three records are selected according to engineering approach (Baker and Cornell 2006). These three acceleration time histories are shown in Fig. 2 and the parameters of the records are given in Table 1.

Generally, design earthquake characteristics are determined based on a probabilistic approach taking into account the existing data related to the seismic history and previous earthquakes around study area. The uniform hazard response spectrum (Fig. 3) at the bedrock level was calculated in CRISIS 2007 Ver. 7.6 (a software module for seismic hazard assessment developed by CAPRA) using Boore and Atkinson (2008) Next Generation Attenuation (NGA) Eq. (1) compatible with regional seismic settings.

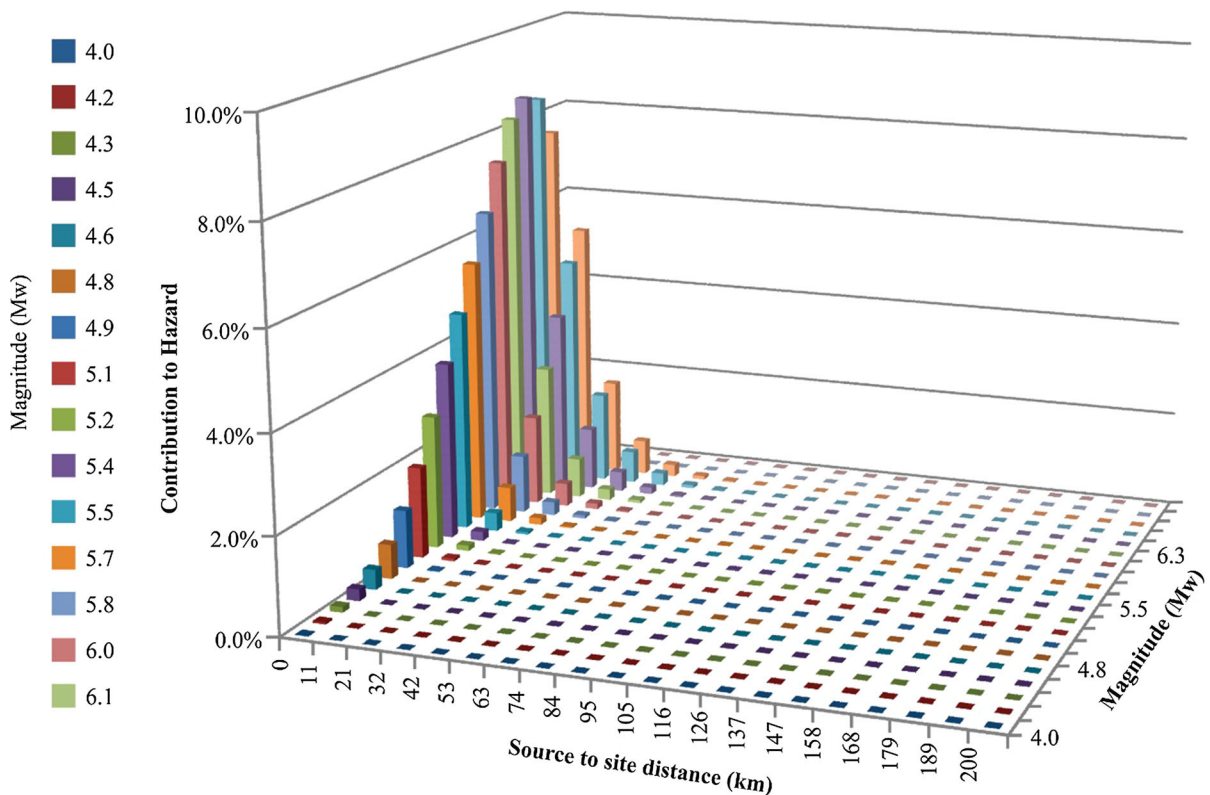


Figure 1
Disaggregation of probabilistic seismic hazard for Islamabad–Rawalpindi city at return period of 475 years

$$\ln y = M \times F_M + F_D(R_{jb}, M) + F_S(V_{s30}, R_{jb}, M) + \varepsilon\sigma_T, \quad (1)$$

where F_M is the magnitude scaling factor, F_D is the distance function, F_S is the site amplification, M is the moment magnitude, R_{jb} is the Joyner–Boore distance, V_{s30} is the average shear wave velocity (top 30 m depth), ε is the fractional number of standard deviations for predicted values of higher and lower $\ln y$ from the mean value, and period-dependent coefficient σ_T is computed as $\sigma_T = \sqrt{(\sigma^2 + \tau^2)}$, where σ is the intra-event aleatory uncertainty and τ is the inter-event aleatory uncertainty.

From the probabilistic seismic hazard analysis (PSHA), the maximum values of PGA (0.21, 0.32, 0.41 and 0.53 g) corresponding to return periods of 150, 475, 975 and 2475 years are computed. The selected real accelerograms are scaled to these PGAs

using scale factors (Table 2) computed using Eq. (2). These scaled accelerograms (Fig. 4) were used as outcrop motions in site response analysis in computer program EERA. EERA is an equivalent-linear analysis computer program developed for site response analysis in FORTRAN 90 starting from the same basic concepts as SHAKE (Seed and Idriss 1970; Idriss and Sun 1992; Kramer 1996).

$$\text{Scale factor} = \frac{\text{PGA}_{(\text{Real time histories})}}{\text{PGA}_{(\text{Hazard Analysis})}}. \quad (2)$$

4. Site Characterization

In the study area, site class specifications are employed to characterize generic subsurface conditions (e.g., Qadri et al. 2015; Khan and Khan 2016) towards seismic response of the soil. Various in situ

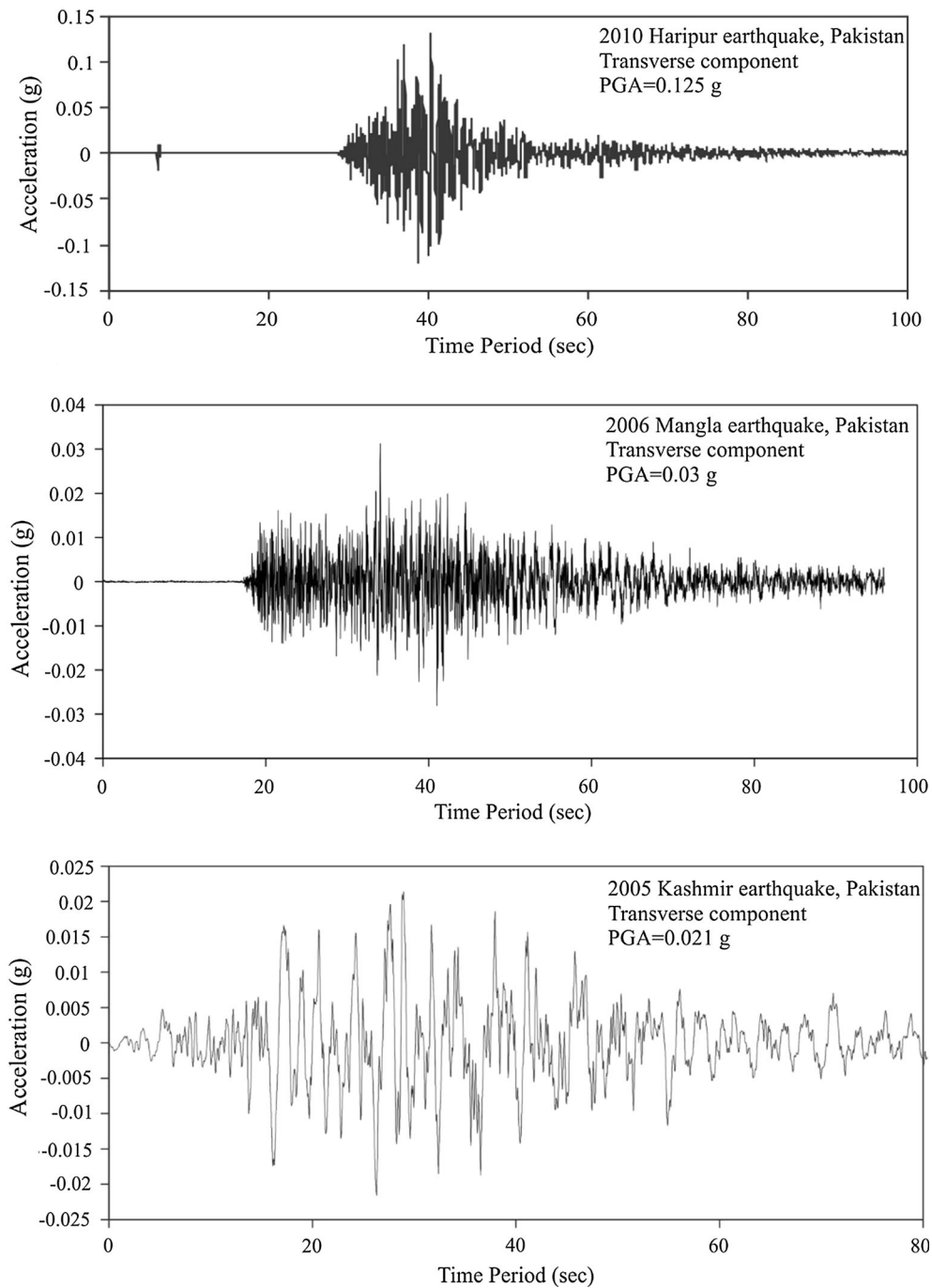


Figure 2
Input real time history records used in the ground response analysis

field tests are available for site characterization. The most common field tests conducted for site characterization are multichannel analysis of surface wave

(MASW), cone penetration test (CPT), standard penetration test (SPT), spectral analysis of surface wave (SASW), and Nakamura method of horizontal

Table 1
Selected real accelerograms and their parameters

Event name	Station	Location		Component	Magnitude	Depth (km)	Maximum acceleration (g)
		Latitude	Longitude				
1. 2005 Kashmir earthquake	Nilore	33.10	73.76	Transverse	Ms 7.6	28	0.02
2. 2010 Haripur earthquake	Nilore	33.10	73.76	Transverse	ML 5.2	33	0.13
3. 2006 Mangla earthquake	Murree	33.86	72.87	Transverse	ML 5.0	10	0.031

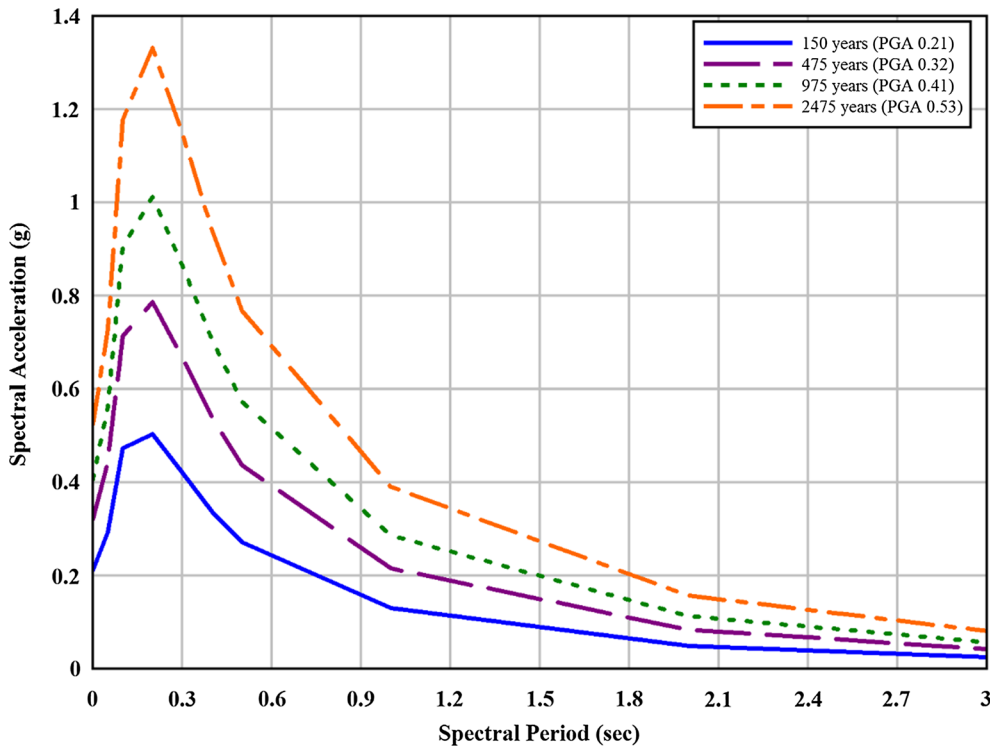


Figure 3

Uniform hazard spectra for Islamabad–Rawalpindi city at return periods of 150, 475, 975 and 2475 years using median value of Boore and Atkinson (2008)

to vertical spectral ratio (H/V). In this study, 47 numbers of SPT borehole data out of 250 boreholes (acquired using a split-spoon sampler in a borehole/ auger hole) were selected and 75 numbers of H/V spectral ratio data (Fig. 5) were acquired using Tromino Engy Plus developed by Micromed, Italy.

Table 2
Scale factors used to scale time histories calculated using Eq. (2) for return periods of 150, 475, 975 and 2475 years

Event name	Scale factor			
	150 year	475 year	975 year	2475 year
1. 2005 Kashmir earthquake	0.0952	0.0625	0.0488	0.0377
2. 2010 Haripur earthquake	0.6190	0.4063	0.3171	0.2453
3. 2006 Mangla earthquake	0.1476	0.0969	0.0756	0.0585

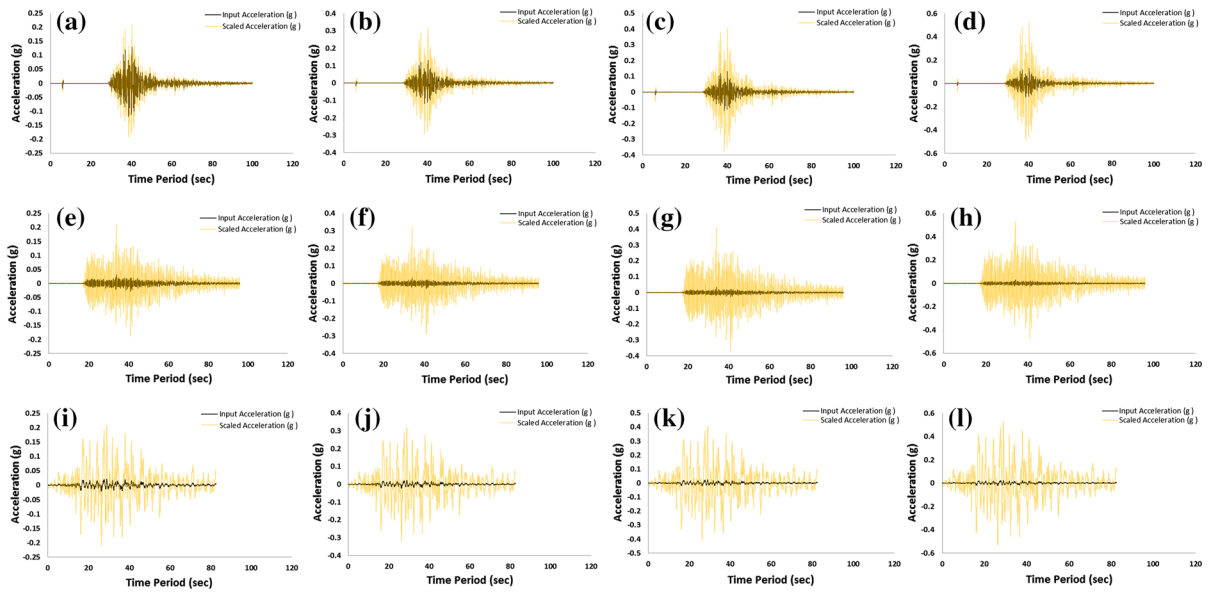


Figure 4

Scaled real time history records of Haripur, Mangla and Kashmir earthquake (top to bottom) for return periods of 150 years (a, e, i), 475 years (b, f, j), 975 years (c, g, k) and 2475 years (d, h, l)

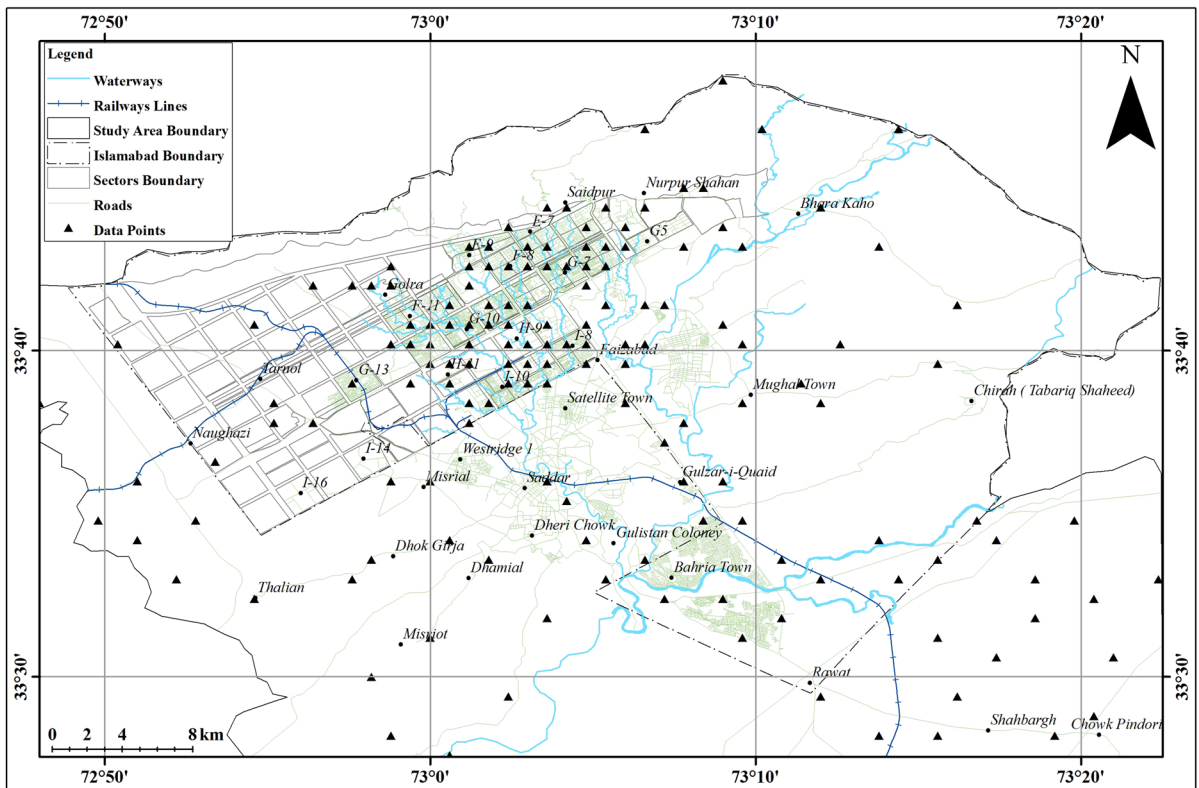


Figure 5

Location of the points (triangle symbol) for acquisition of data using Tromino Engy Plus and SPT in the Islamabad-Rawalpindi metropolitan area

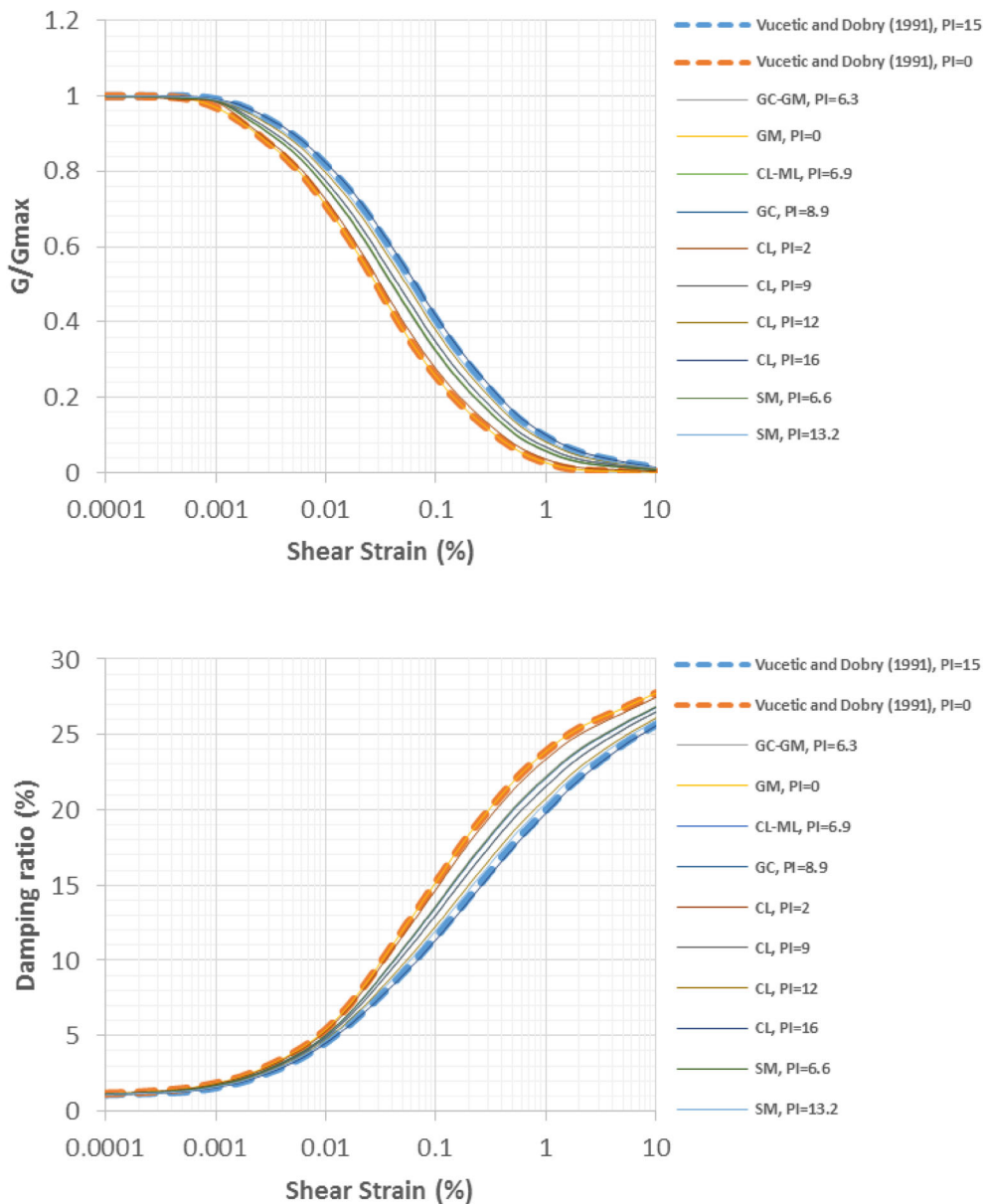


Figure 6

Damping ratio and normalized shear modulus of the soils on the basis of Plasticity Index, interpolated between PI = 0–15 of Vucetic and Dobry (1991)

5. Soil Investigation

Soil investigations in the study area include 250 borehole laboratory tests that are mostly limited to the upper layers. All bore holes from the studied area range in depth from 2 to 25 feet, of which 47 bore holes that penetrated to maximum depth were selected for further analysis. According to the result of the

sieve analysis, hydrometer test and density test, the soils in the region are found mainly of clay, sand and gravely clay. Soil is further classified based on Plasticity Index (PI) as: CL, GC, GM, GC-GM, SM, and CL-M for calculation of shear modulus and damping reduction curves.

In this study, a site-specific relationship has been developed ($R^2 = 0.84$) between HVSR estimated

Table 3

NEHRP (1997) site classification standards followed for different soils in the studied area

Site class	Definitions
A	Hard Rock with shear wave velocity > 1500 m/s
B	Rock with shear wave velocity to (760–1500 m/s)
C	Very dense soil and soft rock with shear wave velocity (360–760 m/s or $N > 50$ or $S_u > 100$ kPa)
D	Stiff soil with shear wave velocity (180–360 m/s or $15 \leq N \leq 50$ or $S_u 50 \leq 100$ kPa)
E	Soil with shear wave velocity < 180 m/s
F	Site specific evaluations

Table 4

Coefficients of Eq. (6) in text, relating $\log V_{S30}$ and $\log V_{S(z)}$, for KiK-Net Data, without regard to site class

Depth (z) (m)	C_0	C_1	C_2	Standard deviation of residuals, σ_{RES}
5	0.20	1.318	-0.12	0.119
6	-0.06	1.482	-0.14	0.111
7	-0.27	1.607	-0.16	0.103
8	-0.37	1.649	-0.16	0.097
9	-0.49	1.707	-0.17	0.09
10	-0.54	1.715	-0.17	0.084
11	-0.60	1.727	-0.16	0.078
12	-0.61	1.707	-0.16	0.072
13	-0.63	1.698	-0.15	0.067
14	-0.61	1.659	-0.14	0.062
15	-0.58	1.611	-0.13	0.056
16	-0.54	1.565	-0.12	0.052
17	-0.53	1.535	-0.11	0.047
18	-0.50	1.494	-0.10	0.043
19	-0.46	1.447	-0.09	0.038
20	-0.41	1.396	-0.08	0.035
21	-0.38	1.365	-0.07	0.03
22	-0.35	1.331	-0.07	0.027
23	-0.32	1.291	-0.06	0.023
24	-0.27	1.25	-0.05	0.019
25	-0.22	1.202	-0.04	0.016
26	-0.18	1.159	-0.03	0.013
27	-0.13	1.12	-0.02	0.009
28	-0.09	1.08	-0.02	0.006
29	-0.05	1.04	-0.01	0.003

shear wave velocity (V_S) and SPT data for the Islamabad–Rawalpindi site, although shear wave velocity is a stiffness parameter and SPT- N is a strength parameter determined at different strain

values. This relationship (3) when used with caution allows for an initial estimate of the SPT- N value using shear wave velocity measurements at location which are not covered by the mechanical SPT.

$$V_s = 73.77 \times N^{0.484}, \quad (3)$$

$$G_{\max} = 14070 \times N^{0.68}. \quad (4)$$

Unit weight is referred to weight of a unit volume of soil. This important parameter is calculated from shear modulus relation of Imai and Tonouchi (1982) (Eq. 4) and shear wave velocity (Eq. 3) in Eq. (5).

$$\text{Unit weight} = \frac{(g \times G_{\max})}{V_s^2}. \quad (5)$$

In the present study, the gravitational acceleration ‘ g ’ is taken as 9.81 m/s^2 . The values fall within conventionally accepted ranges of unit weight for all soils.

6. Modulus Reduction Curve

Soil response analysis includes soil stiffness and material damping information for each soil layer. At moderate to high strains, the shear modulus (G_{\max}) is used to represent the average soil stiffness. The material damping ratio (ξ) represents the energy dissipated by the soil. Mechanisms that contribute to material damping are friction between soil particles, strain rate effect, and nonlinear soil behavior (Zhang et al. 2005).

In this study, the normalized shear modulus and damping curves of Vucetic and Dobry (1991) were used. Vucetic and Dobry (1991) proposed a model for the shear modulus reduction and damping curves as a function of plasticity index. Schneider et al. (1999) conducted another study to investigate the dynamic behavior of four different soil groups (i.e., SM-ML, SM, MH and ML); the results obtained during the study were compatible with the curves proposed by Vucetic and Dobry (1991).

The available data for plasticity index are averaged for 0–5, 5–10, 10–15 and 15–20 m depths to find the modulus curves for soils. The curves for CL,

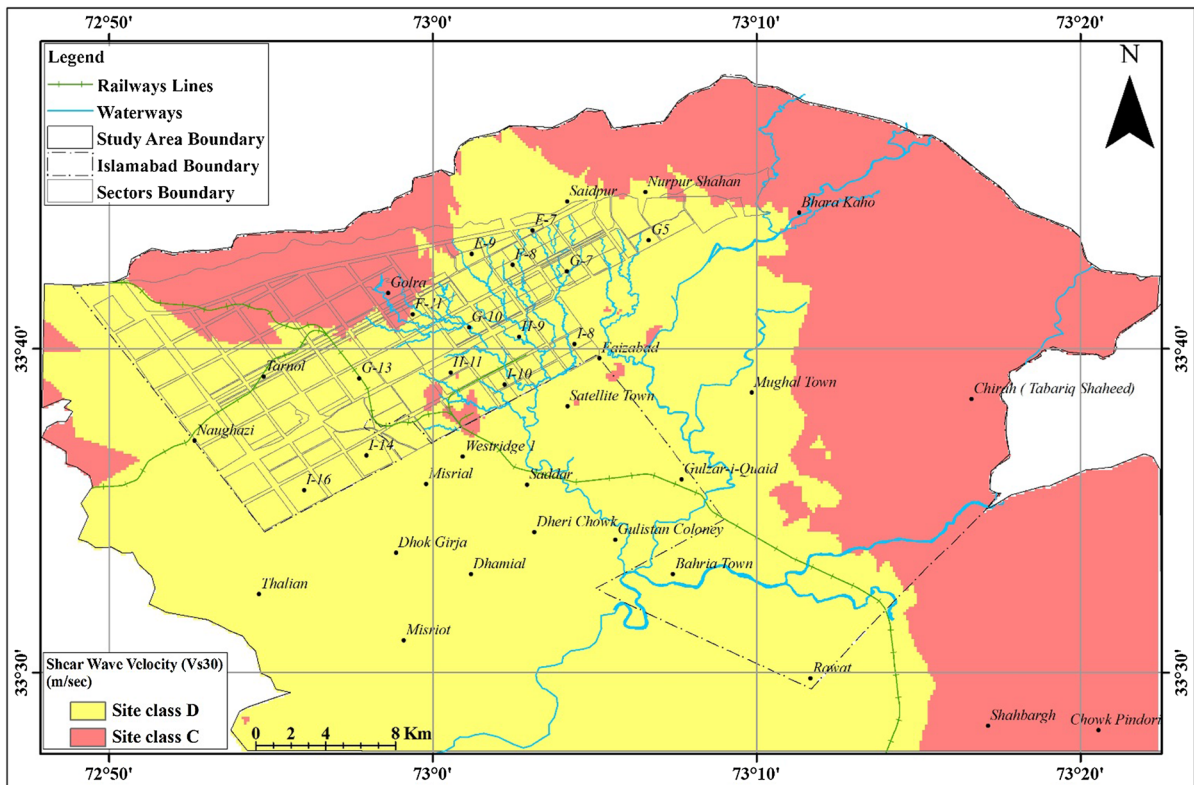


Figure 7

Spatial distribution of V_{S30} in the area, categorized into soil site C and D according to NEHRP (1997) code

SM, GC, GC-GM, CL-ML and GM group of soils were interpolated between $PI = 0-15$ of Vucetic and Dobry (1991) curves (Fig. 6).

7. Site Classification and Calculation of V_{S30}

A site classification scheme based on V_{S30} values was proposed by Borcherdt (1994) and a similar scheme was adopted by the NEHRP (Table 3). Eurocode-8 has also classified the site based on V_{S30} standard penetration test (SPT) values. Even though both the schemes use similar methods to identify site classes, the range of V_{S30} values specified for each site class is different in both the methods.

In many locations, the rock depth was shallow (less than 30 m) and hence the evaluation of V_{S30} value was not possible. In such cases, extrapolation of available shear wave velocity was done to evaluate the V_{S30} values. The method of Boore (2004) for

different models to extrapolate the shear wave velocities for depths less than 30 m (to get the V_{S30} value) was used for this purpose. In the first method, it was assumed that the shear wave velocity remains constant from the deepest velocity measurement to the 30 m level. Even though this method is simple, it is found to underestimate the V_{S30} values, since in most of the soils, the shear wave velocity is found to increase with depth. Therefore, another Eq. (6) based on a second-order polynomial relation was proposed by Boore et al. (2011) as:

$$\log V_{S30} = C_0 + C_1 \log V_{s(z)} + C_2 (\log V_{s(z)})^2 \quad (6)$$

The coefficients and the standard deviation of the residuals to the fit (σ_{RES}) are given in Table 4.

The available SPT borehole data give shear wave velocity up to 20 m depth of soils. The average shear wave velocity of 47 boreholes was extrapolated to 30 m depth to find the V_{S30} using Eq. (6). The instrument Tromino Engy Plus gives V_{S30} value

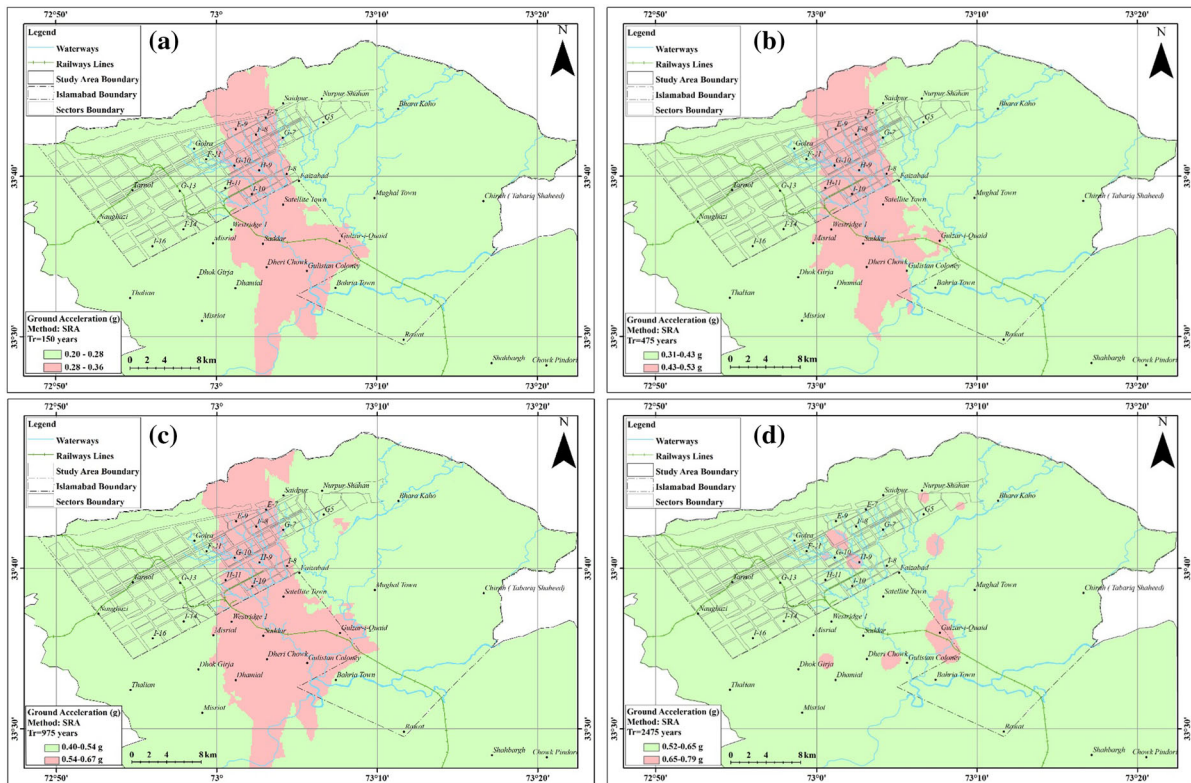


Figure 8

Microzonation map 1 of the study area, acceleration values are calculated using ground response analysis for return periods of **a** 150 years, **b** 475 years, **c** 975 years and **d** 2475 years

obtained by curve matching technique in Grilla software, with first layer thickness as constraint (Khan and Khan 2016) and Eq. (6) was not used in this case for extrapolation of shear wave velocity. After compiling the results of V_{S30} from SPT boreholes and Tromino Engy Plus, the study area was then classified according to NEHRP codes into soil site C and D (Fig. 7).

8. Ground Response Analysis

After analyzing the local soil conditions, shear wave velocity, thickness, unit weight, material curves and site classifications, the next step is to perform equivalent linear site response analysis using EERA software developed by Bardet et al. (2000).

In this study, the ground response analysis was carried out using 1-D shear wave propagation theory. The equivalent linear site response of soil profile is

evaluated for 1-D ground response analysis. In EERA, an equivalent linear procedure (Idriss and Seed 1968; Seed and Idriss 1970) is used to account for the non-linearity of the soil using an iterative procedure to obtain values for modulus and damping that are compatible with the equivalent uniform strain induced in each sublayer. Thus, at the outset, a set of properties (shear modulus, damping and total unit weight) is assigned to each sublayer of the soil deposit.

The geotechnical parameters like local soil type, thickness of the layer in meters, unit weight of the material, shear wave velocity (V_S) of the material and peak ground acceleration data were provided as inputs. The available SPT borehole data were drilled maximum up to 25 feet where bedrock was not encountered; therefore, layered elastic half space (Tsai and Housner 1970) is selected for the analysis.

Three scaled acceleration records were used in the site response analysis for 47 representative soil profiles in the Islamabad–Rawalpindi metropolitan area.

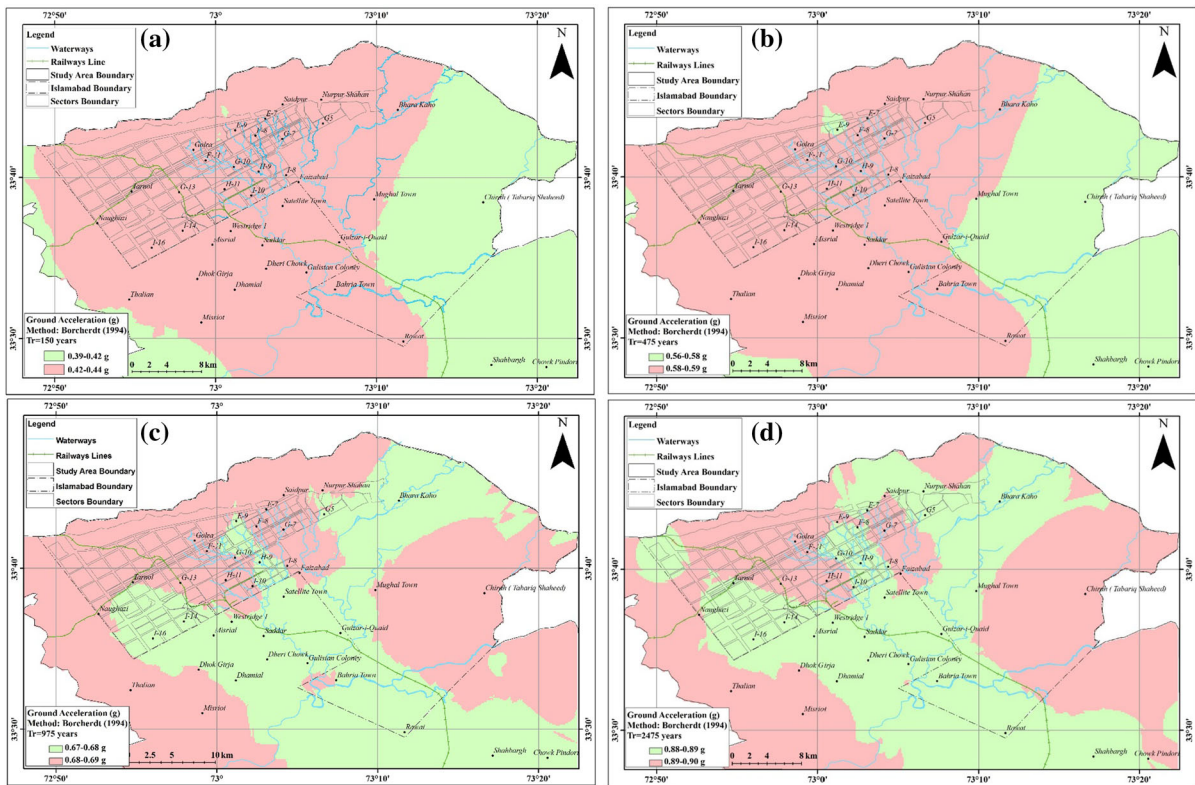


Figure 9

Microzonation map 2 of the study area, acceleration values are calculated using Borchardt (1994) relation for return periods of **a** 150 years, **b** 475 years, **c** 975 years and **d** 2475 years

As a result of these analyses, the geometric means of the three response spectra (for each return period), obtained on the ground surface for each borehole, were determined and the arithmetic means of these average spectra between 0.1 and 1 s periods were calculated. This calculated average spectral acceleration is one of the microzonation parameters (Fig. 8) suggested for microzonation applications (DRM-World 2004; Yagci and Ansal 2009).

In general, the results of site response analyses are evaluated together with the parameters calculated based on empirical amplification equations. In accordance with this approach, the maximum spectral accelerations calculated using the Eq. (7) proposed by Borchardt (1994) based on average shear wave velocities are defined as the second microzonation parameter. The maximum spectral accelerations in the range of short periods are calculated based on the amplification factor F_a defined as:

$$F_a = \left(v/v_o \right)^{m_a}, \quad (7)$$

where v_o is the reference bedrock shear wave velocity (760 m/s) and v is the average (equivalent) shear wave velocity (V_{S30}) for the top 30 ms of soil profile. Exponential coefficient “ m_a ” is determined on the basis of ground motion intensity level on the bedrock.

In this study, the peak ground acceleration on the bedrock level for return periods of 150, 475, 975 and 2475 years was determined by taking the maximum value calculated based on the selected attenuation relationships of Boore and Atkinson (2008). The resultant acceleration values were 0.21 g (150 years), 0.32 g (475 years), 0.41 g (975 years) and 0.53 g (2475 years). The exponential coefficient (m_a) proposed by Borchardt (1994) was determined as 0.24, 0.07, -0.05 and 0 for the return periods of 150, 475, 975 and 2475 years, respectively. These amplification

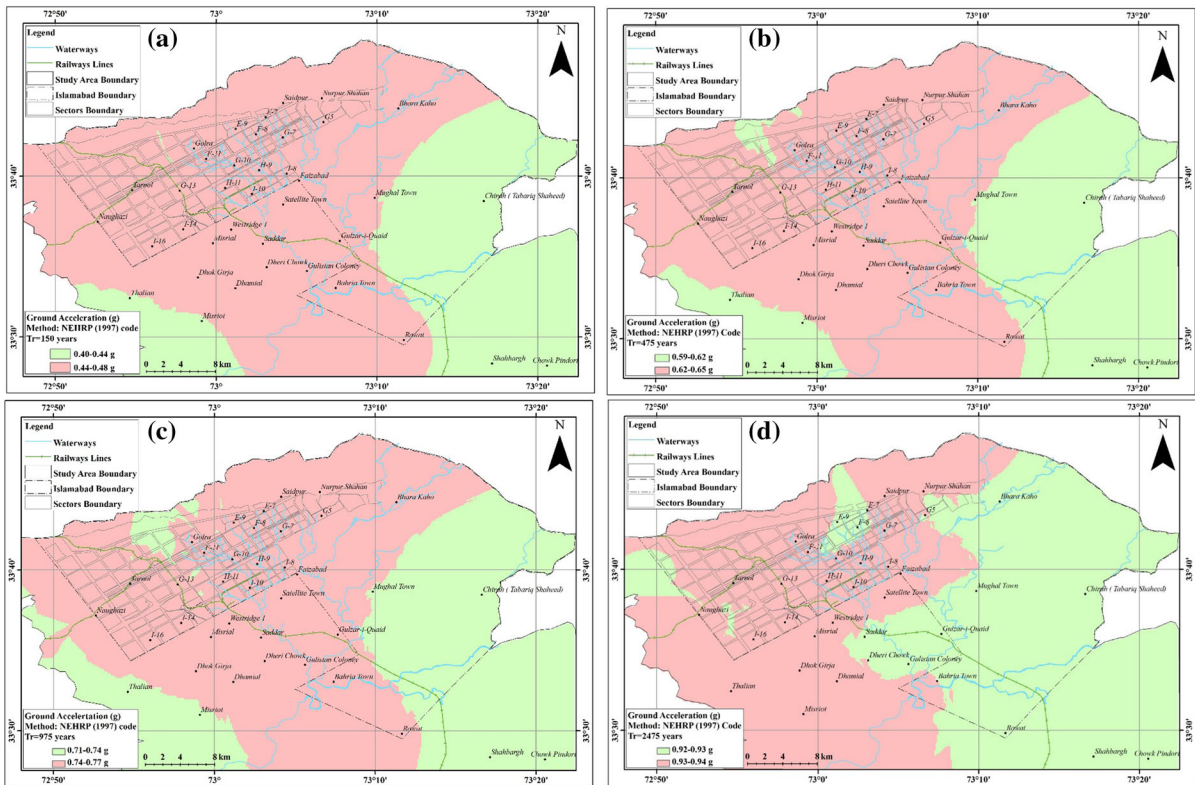


Figure 10

Microzonation maps with respect to ground shaking intensity for return periods of **a** 150 years, **b** 475 years, **c** 975 years and **d** 2475 years. The study area is divided into two zones based on average acceleration value

factors were multiplied with the average bedrock spectral acceleration between 0.1 and 1 s to estimate the spectral accelerations on the ground surface, this gives the second microzonation parameter (Fig. 9).

9. Microzonation with Respect to Ground Shaking Intensity

In microzonation with respect to ground shaking intensity, the preferred approach is to compare and utilize multiple parameters (Ansal et al. 2003, 2004, 2005). There are two primary reasons for such an approach. Firstly, site response analysis is a numerical procedure and the results are significantly affected by the soil and earthquake properties used in analysis. The reliability of the results obtained from site response analysis is highly dependent on the selected soil properties and on the characteristics of

acceleration time records used in analysis. Secondly, in contrast to empirical method, the Borchardt (1994) method may yield consistent results due to its empirical nature that was formulated based on the observational data.

The average spectral accelerations between 0.1 and 1 s periods calculated based on site response analysis and evaluated relatively with respect to the frequency distribution in the study area were proposed as one of the microzonation parameters in the literature (DRM-World 2004; Ansal et al. 2004, 2005).

However, in this study, an approach was preferred that would be applicable for the entire residential area in Islamabad–Rawalpindi. For this purpose, in accordance with the proposed approach; first, the regions within Islamabad–Rawalpindi residential area but outside of the areas with site response analyses were classified according to NEHRP (BSSC 2001) based on the soil

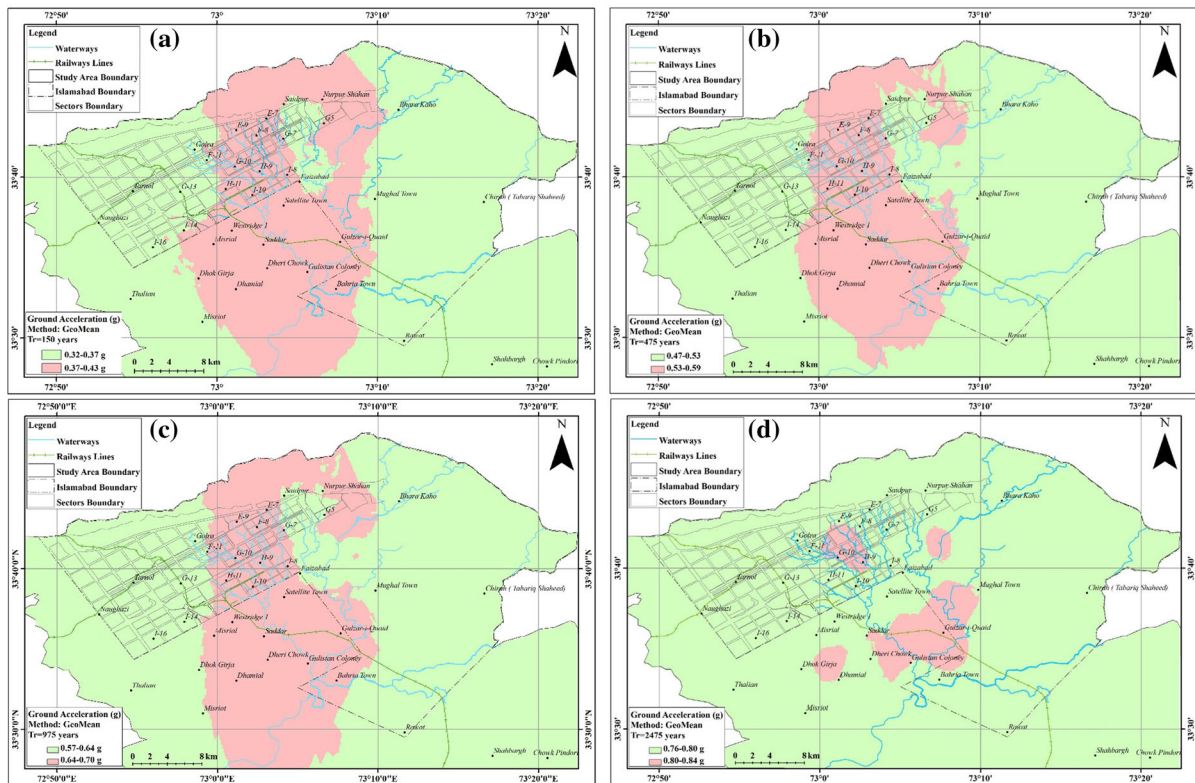


Figure 11

Final microzonation map generating by superimposing three maps generated for return periods of **a** 150 years, **b** 475 years, **c** 975 years and **d** 2475 years. The basin part in the study area has high values of acceleration as compared to surrounding parts for the hazard levels of 150, 475 and 975 years

investigations. Even though these soil investigations do not contain detailed geotechnical data, they reflect the variations in site conditions for all residential areas. According to the NEHRP site classification, areas within Islamabad–Rawalpindi residential boundaries may be classified as C and D.

Afterwards, based on the site classes according to NEHRP, spectral accelerations on the ground surface were calculated using spectral amplification factors proposed in NEHRP code. At this stage, for the determination of spectral acceleration at 0.2 s period at the bedrock level in study area corresponding to the return periods of 150, 475, 975 and 2475 years, similar to the previous approach, the maximum calculated value was selected. For this spectral acceleration determined at the bedrock level, amplification factor varies based on the site class. Accordingly, the spectral accelerations

calculated on the ground surface within study area vary between 0.40–0.48 g for 150 years, 0.59–0.65 g for 475 years, 0.71–0.77 g for 975 years and 0.92–0.94 g for 2475 years (Fig. 10). The average value of 0.44, 0.62, 0.74 and 0.93 g (for return periods of 150, 475, 975 and 2475 years, respectively) was defined as the boundary value for microzonation and accordingly two zones have been identified at this stage for microzonation of the study area. Thus, cells with spectral accelerations higher than 0.44, 0.62, 0.74 and 0.93 g were identified as zone A (where higher ground shaking is expected) and cells with spectral accelerations less than 0.44, 0.62, 0.74 and 0.93 g were identified as zone B (where relatively lower ground shaking is expected).

When the microzonation maps generated (1) with respect to the average spectral accelerations between

Table 5

Acceleration and amplification values calculated for microzonation map for return periods of 150, 475, 975 and 2475 years

Hazard level (year)	Hazard Zone		Amplification
	High (g)	Moderate (g)	
150	0.33–0.36	0.26–0.30	0.96–1.38
475	0.46–0.50	0.38–0.42	0.90–1.14
975	0.55–0.60	0.46–0.51	0.85–1.04
2475	0.72–0.78	0.61–0.66	0.84–1.00

0.1 and 1 s periods based on site response analysis; (2) with respect to spectral accelerations calculated using the empirical relationships proposed by Borchardt (1994) and (3) with respect to ground shaking are compared, it is observed that the distribution of zones has some differences as well as similarities. Finally, microzonation maps were generated (Fig. 11) by the superposition of the microzonation maps obtained based on the spectral accelerations calculated by three different approaches.

10. Conclusions

1. An approach compatible with the recent advances in microzonation methodology was employed for this study. Two different data sets were used in this study. The geotechnical borehole data were used for modeling the representative soil profiles.
2. Soil classification and dynamic soil models in the area were represented with a gradual transition based on the plasticity index for soils. *H/V* spectral ratio data were used to evaluate the shear wave velocity for 30 m depth.
3. The relation developed for shear wave velocity estimation from SPT data for Islamabad–Rawalpindi area depicts same trend as previously calculated from other regions by different workers. This newly developed equation can only be applied to soils in northern Pakistan, as soil profiles in southern may not be compatible with those from northern Pakistan due to variation in soil depths and other parameters.
4. Based on the results of the site response analysis and shear wave velocity, it was observed that most of the studied area is characterized by moderate to high

acceleration values (Table 5). In the basin part of the studied area, high hazard is observed due to relatively greater sediment thickness. The microzonation maps prepared for the return periods of 150, 475, 975 and 2475 years represent same zonation with difference in the acceleration values.

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