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RESEARCH ARTICLE - CIVIL ENGINEERING

Seismic Demand for Low-Rise Reinforced Concrete Buildings of Islamabad–Rawalpindi Region (Pakistan)

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Abstract Determination of appropriate seismic demand is important for deriving reliable seismic forces for use in structural design and vulnerability assessment. Demand can be defined by either probabilistic or deterministic approaches depending on the use of the assessment. The use of probabilistic or deterministic approaches depends on the level of the assessment since deterministic can mostly be used for single structures whereas probabilistic for city or country level assessment. This paper presents demand characterization which is based on results of existing seismic hazard studies and local tectonic features around Islamabad–Rawalpindi region (study region) in Pakistan. Existing seismic zoning maps and recent probabilistic seismic hazard studies are reviewed, and the findings are used to quantify and compare the demand for a typical low-rise reinforced concrete building. As another option, deterministic demand is defined through spectra using suitable attenuation relationship which is assessed and validated using the Kashmir earthquake and other similar earthquakes data. Deterministic spectra for study region are generated by considering the critical local tectonic features. Federal Emergency Management Agency (FEMA 356) approach is used for smoothening of deterministic spectra, and the new spectral corner periods are calculated.

Keywords Seismic demand · Pakistan Seismic Code (PSC) · Seismic hazard · Spectra · Corner period

Abbreviations

BCP	Building Code of Pakistan
GSP	Geological Survey of Pakistan
GSHAP	Geological Seismic Hazard Assessment Program
MSSP	Micro seismic Studies Program
PAEC	Pakistan Atomic Energy Commission
PMD	Pakistan Meteorological Department
PSC	Pakistan Seismic Code
NESPAK	National Engineering Services Pakistan
UBC	Uniform Building Code

1 Introduction

Pakistan lies in an earthquake-prone region and the occurrence of major earthquakes in the northern region of Pakistan area is result of the continuing subduction of the Indian Plate under the Eurasian Plate at a rate of 40 mm/year [1]. The convergence and collision of these two plates caused folding and thrusting of the upper crustal layers, which resulted in the formation of many important thrusts and many active strike slip faults, particularly in the north and north-east region. These important thrusts include Panjal Thrust, Main Mantle

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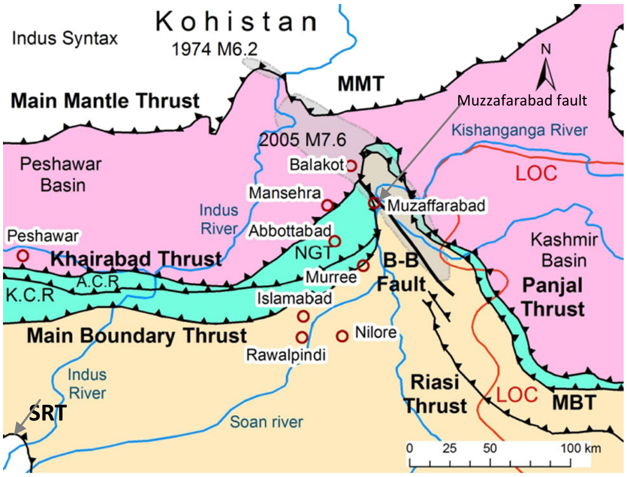


Fig. 1 Major tectonics in the northwest (NW) Himalayas. Reproduce with permission from Hussein et al. [37]

Thrust (MMT), Main Boundary Thrust (MBT), Riasi Thrust, Salt Range Thrust (SRT) as shown in Fig. 1. The seismicity along the collision boundary is relatively shallow, and most earthquakes have a depth up to 50 km [2].

More recently, the devastating Kashmir earthquake in 2005 ($M_w = 7.6$) [3] occurred due to rupture along the "Muzaffarabad fault", which is located along the northern most part of the Riasi Thrust (Fig. 1). The earthquake had a focal depth of 26 km and was located 10 km north-east of the Muzaffarabad city and 105 km north-northeast of Islamabad (Fig. 1). This earthquake caused severe damage in the Kashmir and the adjoining areas of Khyber Pakhtunkhwa (KPK) Province of Pakistan and left approximately 73,000 people dead, more than 70,000 wounded and 3.3 million displaced [4]. It is estimated that damages incurred were well over US$ 5 billion [4].

The earthquake ground motion was recorded at three different stations of Abbottabad, Murree and Nilore (Fig. 1), approximately located at 48, 64 and 100 km, respectively, from epicentre. The PGA values from these stations were 0.231, 0.078, 0.026 g, respectively [5]. The 5% damped elastic response spectrum from the Abbottabad record showed a wide range of high amplifications over the period range of 0.4–2 s with the highest amplification of 4. Since Nilore is located in Islamabad, the maximum amplification from the spectrum was found to be 3. This record is recommended to be used with care since the horizontal peak ground acceleration (PGA) is overestimated by attenuation relations, whereas vertical PGA at Nilore is well predicted (Durrani et al. [6]). The most probable reason for this overestimation of horizontal PGA was linked to raft foundation size where the instrument is anchored (Durrani et al. [6]).

In the past, a series of seismic zoning maps [7–9] were prepared for Pakistan and upgraded with the passage of time due to additional data and knowledge, resulting in variations in the seismic zonation between the maps of different times. Since no recurrence intervals of the various intensities were specified, demand in each zone could be not coupled with a probability of exceedance (POE). After the Kashmir earthquake, many probabilistic seismic hazard studies have been conducted by different organizations and researchers [10–13] which predicted higher seismic hazard as compared to previous seismic zoning maps, especially for the northern region of Pakistan.

As a result, it is now accepted that much of the building structures in the region are not adequately designed for the seismic hazard perceived today, and hence there is a need to identify appropriate seismic forces for design of new structures and to carry out an independent vulnerability assessment of the existing building stock. Moreover, seismic forces, time period, displacement, etc. can change significantly because of the soil–structure interaction (SSI) effects depending on soil stiffness, foundation type, size, method of analysis, etc. Fix base analysis of structure may be conducted if the soil shear wave velocity is $\geq$ 1100 m/s [14]. Moment-resisting frame buildings resting on relatively soft soils may significantly amplify the lateral displacements and inter-storey drifts. This amplification of lateral deformations may change the performance level of the building frames. Therefore, it is recommended to conduct SSI analysis for more realistic seismic response.

According to the results of a numerical investigation study [15], effects of dynamic soil–structure interaction for seismic design of midrise moment-resisting building frames resting on soil class C (Vs $=$ 600 m/s) are insignificant. However, dynamic soil–structure interaction has significant effects on the seismic response of midrise moment-resisting building frames resting on soil classes D (Vs $=$ 320 m/s) and E(Vs $=$ 150 m/s). Clearly, performance level of the building frames changes from life safe to near collapse in soil class E, which is dangerous and safety threatening. Inelastic seismic design of midrise moment-resisting building frames excluding soil–structure interaction is not adequate to guarantee the structural safety. As a result, considering SSI effects in seismic design of moment-resisting building frames resting on soil classes D and E is essential.

Research conducted in an other study [16] has shown that the type of foundation is a major contributor to the seismic response of buildings with SSI and should therefore be given careful consideration in order to ensure a safe and cost-effective design. The results of this study indicated that the structure supported by the pile–raft foundation and the floating pile foundation experienced more base shear than the structure supported by the shallow foundation [16]. Another study [17] shows that length of the pile foundation and load-bearing mechanism influences the way shear forces are distributed along the superstructure. Longer piles have higher contact surfaces with the surrounding soil, which enables them to absorb extra energy, and they experience less rocking than shorter piles because their resistance is stronger.

In another research study of SSI analysis of midrise moment-resisting building [18], it was concluded that the inelastic seismic response of the midrise moment-resisting building resting on soft soil is underestimated by equivalent linear method of dynamic analysis as compared to fully nonlinear dynamic analysis method.

In the current study, the Islamabad and Rawalpindi cities are selected as study region with important infrastructure and large population in the north of Pakistan. Islamabad and Rawalpindi are twin cities with Islamabad having many important building structures and Rawalpindi having a higher population density and more poorly constructed buildings. In addition, they are exposed to a number of active faults within a 150 km radius as described in NESPAK (National Engineering Services Pakistan) [11] and Monalisa et al. [19]. The significant seismic events ($M_{\mathrm{w}} = 4{-}6$) which occurred around these faults from 1919 to 2005 (NESPAK catalogue) were shallow having an average focal depth of 47 km. The soil of these cities is generally considered as very dense to stiff soil (soil class C), and SSI influence is expected be insignificant on low-rise structures. Moreover, it is also general practice of engineers to analyse and design low-rise structures considering fixed base. Hence in current study, fixed base of the structure is assumed to get a conservative estimate of base shear.

The scope of this study is to evaluate probabilistic from recent hazard studies for use it in design of new structures and for the seismic vulnerability/risk assessment of a building stock. Moreover, deterministic demand is evaluated for use in design/vulnerability assessment of structures located close to source and for specialized structures. Since a critical fault Main Boundary Thrust passes very close to Islamabad (5–10 km), deterministic spectra is evaluated to determine deterministic demand for structures in the study region.

This paper initially presents a brief review of the existing seismic zoning maps and more recent probabilistic seismic hazard studies for the study region. The demand for a typical reinforced concrete (RC) low-rise frame structure of the study region is then defined in terms of base shear-to-weight ratio. For generating deterministic spectra, suitable attenuation relationship is selected and its performance is validated using different earthquake data. Deterministic demand corresponding to the critical tectonic feature of the study region is defined. The deterministic spectra are further smoothened and corner periods are defined for near earthquake source of the study region.

2 Review of Existing Seismic Zoning Maps of Pakistan

2.1 Seismic Zoning with No Recurrence Interval Consideration

The most prominent seismic zoning maps [pre-Kashmir earthquake (2005)] by PSC-86 [7], Geological Survey of Pakistan (GSP) [8], PMD [9] were based mainly on limited instrumental data, felt intensities and historical seismicity data of different regions. UBC 1997 [20] with its world seismic risk map also defines seismic zones for some major cities of Pakistan. Geological Survey of Pakistan (GSP) 2006 [8] developed an updated map after the Kashmir earthquake in 2005. These maps give values and ranges of different seismic hazard parameters. PSC-86 and GSP (2006) seismic zoning maps are given in "Appendix". The predictions of these maps for the study area and the Kashmir earthquake affected areas are given in Tables 1 and 2, respectively.

It can be seen in Table 1 that PSC-86 [7] and PMD-99 [9] gave lower seismic hazard levels for study area, whereas GSP (1988) [8], GSP (2006) and UBC-97 [20] consider the study area to be in high seismic hazard zone with a higher intensity range $>$ 7 and PGA range between 0.1 and 0.3 g. UBC-97 placed Islamabad in zone 4, which is the most severe hazard zone. Zone 4 for Islamabad was considered rather too strict by the local engineers, and after consent, the study area was placed in zone 2B instead of zone 4 [11]. Table 2 shows that PSC-86 [7] places the Kashmir earthquake affected area in zone 2 which according to the description should have suffered moderate damage associated with MMI = VII. But the Kashmir earthquake has shown that the intensities in these cities were from VIII to X, which is a lot more than zone 2 specified intensity level. This shows the underestimation in demand prediction due to the lack of infrequent big events associated with large return periods. GSP (1988) [8] and PMD (1999) [9] also underestimate intensity/peak ground accelerations of Kashmir earthquake affected area. The GSP (2006) [8] zoning map predicts the highest intensity due to the inclusion of the recent Kashmir earthquake event and additional historical data.

As pointed out earlier, the main deficiency in all these maps is that no recurrence intervals of various intensities were given and hence, demand in each zone cannot be coupled with a probability of exceedance (POE). This can cause serious uncertainties when trying to establish demand for design or risk assessment, since the intensities can only be taken as the highest expected intensities for the design lifetime of residential dwellings.

Table 1 Hazard predictions by different seismic zoning maps for the Islamabad-Rawalpindi study area

Organization	Zone assignment to study area	Zone factor	Intensity prediction (study area), MMI	PGA range (study area) (g) Approx. PGA range from intensity*	Zones intensity description
PSC-1986 [7]	2	3/8	VII	0.146	Moderate damage
GSP-1988 [8]	3	–	$>$ VII	0.15–0.29	Major damage
PMD-1999 [9]	3	–	–	0.05–0.07	Moderate hazard
UBC-97 [20]	4	0.4	–	–	High hazard
GSP-2006 [8]	3	–	7.5–9	0.1–0.3	Moderate to severe damage

Table 2 Hazard predictions by different seismic zoning maps for Kashmir region

Organization	Zone assignment to Kashmir region	Zone factor	Intensity prediction (Kashmir region), MMI	PGA range (Kashmir region), g Approx. PGA range from intensity*	Zones intensity description
PSC-1986 [7]	2	3/8	VII	0.146	Moderate damage
GSP-1988 [8]	1	–	V–VI	0.038–0.075	Minor damage
PMD-1999 [9]	2	–	–	0.067–0.1	High hazard
GSP-2006 [8]	4	–	≥ 9	≥ 0.3	Severe damage

* Theodulidis and Papazachos [21] relationship between PGA and MMI is used

2.2 Review of Probabilistic Hazard Studies and Maps of Pakistan

The destruction caused by the 2005 Kashmir earthquake necessitated re-evaluation of seismic zoning and peak ground acceleration (PGA) levels for various regions of Pakistan, so that seismic resistant design standards of RC buildings can be revised and risk assessment studies be made possible. As a result, various probabilistic seismic hazard assessment (PSHA) studies were carried out for Pakistan by different organizations and researchers in recent years. These studies considered the recurrence intervals of various magnitude earthquake events which were not considered in the previous seismic zoning maps. The outcomes from these studies for a 500-year return period are listed in Table 3.

PMD-NORSAR (2007) [3] predicted the highest PGA for the study area, whereas Monalisa [10] and Khan [13] predicted the lower PGA values. For the Kashmir area, GSHAP and PMD-NORSAR predicted the higher PGA values. BCP [12] which is National Seismic Code suggests relatively lower PGA values for both regions as compared to GSHAP and PMD-NORSAR. BCP and PMD-NORSAR seismic zoning maps for 500-year return period are given in "Appendix". Even though this region has the potential of generating large earthquakes ($M_{\mathrm{w}} > 8$) along the Himalayan arc, the under prediction of the previous studies is mainly because the complete historical data of the region were not used. Since higher demand levels are now predicted, buildings designed according to PSC-86 [7] can be considered unsafe.

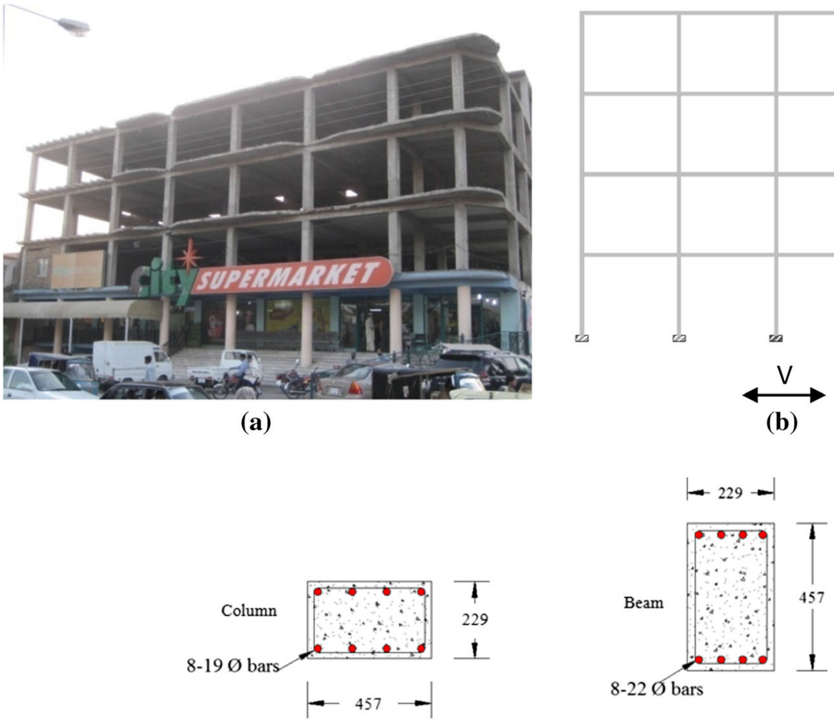
The new demand values shown in Table 3 can be used directly in the seismic re-evaluation of existing structures and design of new structures. Among the PGA values given in Table 3, the one by BCP (2007) [12] is currently used as design PGA value for ordinary structures. A comparative study on the anticipated design forces on ductile and non-ductile buildings according to the new and old hazard studies is undertaken in the following. The purpose is to quantify the underestimation in demand in the existing buildings of a particular period. This will also give an idea of the likely vulnerability of existing buildings.

3 Demand Evaluation for a Ductile and Non-Ductile Low-Rise RC Building in the Study Area

PSC-86 was introduced with an aim to achieve a better seismic performance of RC structures throughout Pakistan. Seismic zones inclusion in the PSC-86 [7] hazard map as a result introduced the concept of seismic loading and allowed earthquake-resistant design for ordinary RC buildings following provisions like those of the UBC 1985 [24]. However, the seismic design was not enforced effectively in Pakistan, and the results were quite evident from the Kashmir earthquake. After Kashmir earthquake, BCP (2007) [12] is recommended as national code for calculation of seismic forces. The number of seismic zones and their names in BCP [12] are same as UBC-97 [20], which are zone 1, 2A, 2B, 3

Table 3 500-year return period probabilistic seismic hazard predictions for the Islamabad-Rawalpindi study area and Kashmir region according to different studies

Organization/researcher	PGA range (study area) (g)	PGA range (Kashmir earthquake affected areas) (g)
GSHAP-1999 [22]	0.24–0.32	0.4–0.48
Monalisa-2005 [23]	0.15	0.13
NESPAK-2006 [11]	0.2	0.18–0.25
BCP-2007 [12]	0.16–0.24	> 0.32
PMD-NORSAR-2007 [9]	0.37	0.439
Khan-2010 [13]	0.12–0.18	0.3–0.35



(a) (b)

(c)

Fig. 2 **a**, **b** Typical four-storey RC structure in the study area, **c** Column and beam section and reinforcement details

and 4. BCP [12] assigns zone 2B to study area. Each of these zones has certain PGA range which corresponds to 500-year return period. Moreover, the spectral expressions in BCP [12] are same as in UBC-97 [20]. The expected demand for a typical low-rise ductile and non-ductile RC structures in the study area can be evaluated according to PSC-86 [7] and the UBC-97 [20] expressions for comparison between structural demands of two different design periods of these codes.

For that purpose, a typical four-storey low-rise RC building as shown in Fig. 2a, b is taken as a case structure and assumed to have a fundamental time period of 0.4 s ($T = 0.1N$ [25,26], where N is the number of storeys). Seismic forces are calculated in terms of V/W [base shear (V) normalized with respect to weight (W) of structure]. V/W is also called as base shear coefficient (Cs), which depends

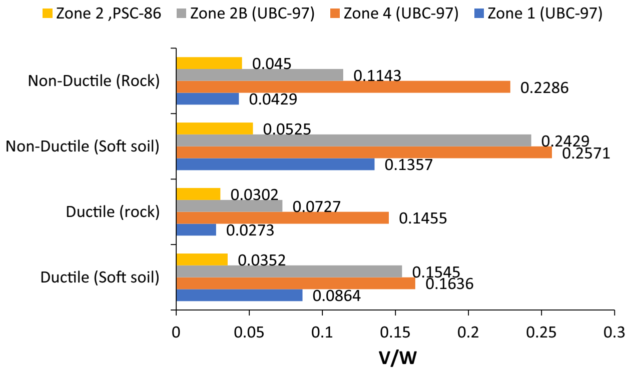
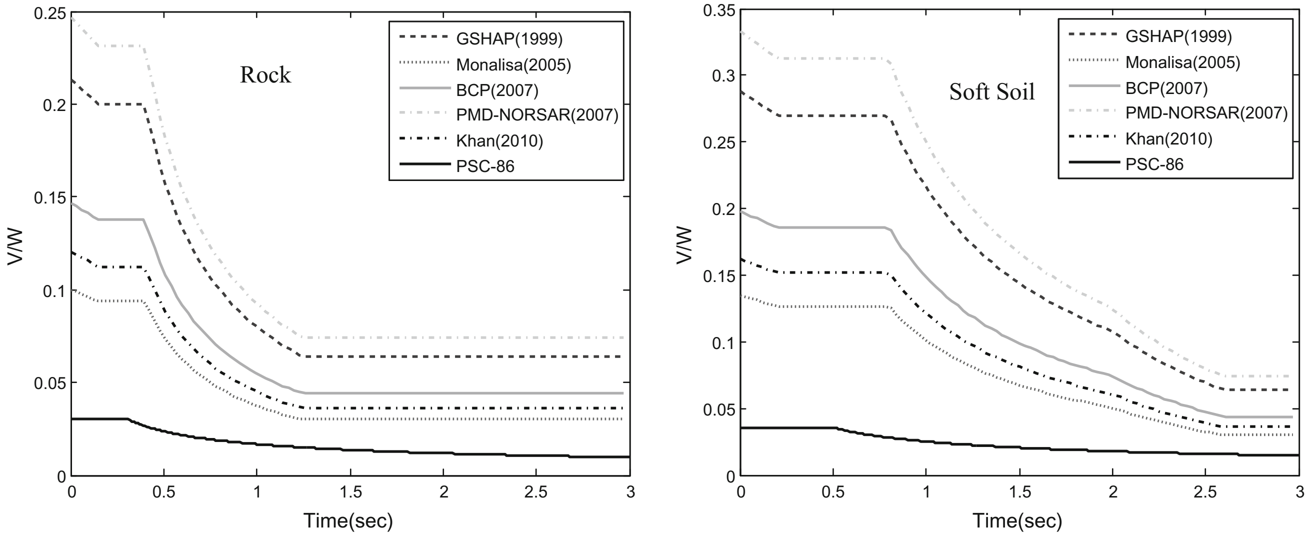
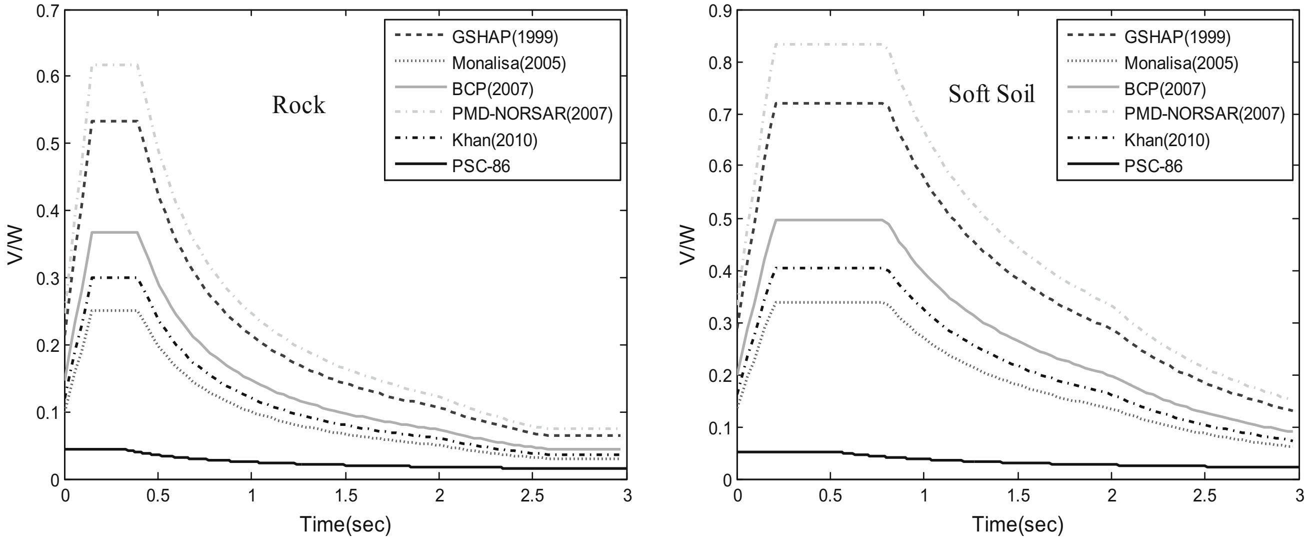


Fig. 3 Comparison between UBC-97 and PSC-86 zones seismic design forces for ductile and non-ductile RC structures



(a) **(b)**

Fig. 4 Comparison between *V*/*W* from current hazard studies (PGA values) and PSC-86 zone 2 for ductile structure on **a** rock site, **b** soft soil site



(a) **(b)**

Fig. 5 Comparison between *V*/*W* from current hazard studies (PGA values) and PSC-86 zone 2 for non-ductile structure on **a** rock site, **b** soft soil site

on many factors. *V*/*W* is representation of spectral acceleration response (Sa/g), and this coefficient value gives an easy understanding of seismic demand. The column and beam section and reinforcement details are given in Fig. 2c. The parameters values of spectral relation complying to PSC-86 [7] zone 2 and UBC-97 [20] zone 2B for rock and soft soil are used for the study area. UBC-97 [20] zone 1 and 4 are also involved in the comparison to investigate the threshold value of PSC-86 [7] demand as compared to UBC-97 [20] minimum and maximum possible demand. The *V*/*W* is further evaluated using the relations of PSC-86 [7] and UBC-97 [20], which gives comparison between existing and new demand for ductile and non-ductile structures on different site conditions (rock and soft soil). In evaluating the seismic forces on a ductile RC structure, the case structure is considered as intermediate moment-resisting frame according to UBC-97 [20] and the ductility factor 'R' is taken as 5.5. A summary of the results is shown in Fig. 3. The PSC-86 [7] demands are very low and even lower than UBC-97 [20] zone1 (soft soil) demand, but demands according to PSC-86 [7] are important for use in the design of the engineered structures when evaluating the vulnerability of buildings of this design period.

The expected PGA values from different PSHA studies (Table 3) for the study area are also used for calculating *V*/*W*

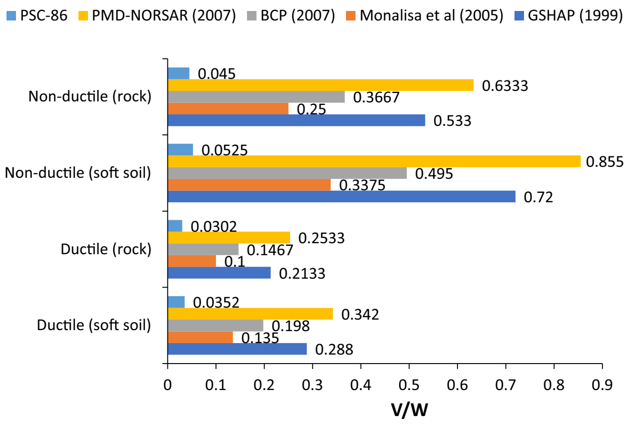


Fig. 6 Bar chart comparison of seismic force of ductile and non-ductile

by using EC8 [27] design spectrum relations. For the ductile case structure, the behaviour factor q is taken as 4 according to medium ductility class (DC'M') proposed by EC8 [27]. Figures 4 and 5 show the seismic forces calculated for ductile and non-ductile structures, respectively, on both rock and soft soil conditions using PGA outcomes from different studies. The seismic forces according to PSC-86 [7] zone 2 are also included for comparison. It is evident from the comparison bar chart in Fig. 6 that all recent PSHA studies impose significantly higher demands as compared to PSC-86 (zone 2) on the case structures. Among all, PMD-NORSAR [3] imposes the highest demand on both types of structures.

Base shear (*V*/*W*) values are also useful in making a distinction between various categories of engineered RC buildings. The base shear values of PSC-86 [7] can be used as a reference for existing post-1986 engineered structures (basic seismic design), whereas the values of PMD-NORSAR [3] can be used for modern engineered RC structures.

Deterministic spectra associated with critical tectonic features can also be generated to examine the maximum possible demand. To generate deterministic spectra for the study area, a suitable attenuation relationship having coefficients at various time periods for different soils is required. Since South Asia (particularly Pakistan) does not have specially determined attenuation relationships, the various alternatives need to be examined.

4 Selection and Validation of Attenuation Relationships for Deterministic Demand Evaluation

In the study area, shallow earthquakes are most likely to lead to the highest PGA. Suitable attenuation equations developed in different parts of the world for shallow crustal earthquakes are listed in Table 4.

In a seismic hazard assessment study by Durrani et al. [6] and Khan [13], the attenuation relationship by Ambraseys et al. [31] was adopted. Monalisa [10] adopted Boore et al. [29] attenuation relationship for seismic hazard assessment of study area. Ambraseys et al. [31] and Boore et al.'s [29] relationships are given in Eqs. (1) and (2), respectively.

$$\begin{aligned}\mathrm{In(Y)} &= a_1 + a_2 M_{\mathrm{W}} + (\mathrm{a}_3 + a_4 M_{\mathrm{W}})\log\sqrt{d^2 + a_5^2}\\ &\quad + a_6 S_{\mathrm{S}} + a_7 S_{\mathrm{A}} + a_8 F_{\mathrm{N}} + a_9 F_{\mathrm{T}} + a_{10} F_{\mathrm{O}}\end{aligned} \tag{1}$$

where Y is peak horizontal acceleration, M_{w} is moment magnitude, d is the closest distance to rupture surface. a_1, a_2, a_3, a_4, a_5, a_6, a_7 are coefficients at various time periods of soil. $F_{\mathrm{N}} = 1$ for normal fault and 0 otherwise, $S_{\mathrm{S}} = 1$ and $S_{\mathrm{A}} = 0$ for soft rock.

$$\begin{aligned}\ln(Y) &= b_1 + b_2\,(M-6) + b_3\,(M-6)^2\\ &\quad + b_s \ln r + b_v \ln V_{s,30}/V_A\end{aligned} \tag{2}$$

where M is moment magnitude, r is the closest distance to rupture surface, Vs, 30 is shear wave velocity to 30 m and b_1, b_2, b_3, b_s, b_v are coefficients at various time periods of soil.

Table 4 Characteristics of different attenuation relationships for shallow crustal earthquakes

Authors	Focal depths (km)	M_{w}	Distances (km)	Soil type	Faulting style
Campbell [28]	≤ 25	≥ 5	< 60	Soil, soft rock, hard rock	Strike slip, reverse fault
Boore et al. [29]	≤ 20	4–8.5	≤ 80	Applicable to all NEHRP site classes A–E using shear wave velocity	Strike slip, reverse fault, unspecified mechanism
Sadigh et al. [30]	20–25	4–8$^{\mathrm{a}}$	~ 100	Deep soil, rock	Strike slip, reverse fault
Ambraseys et al. [31]	20–25	≥ 5	~ 100	Stiff soil, soft soil, rock	Strike slip, normal, thrust, odd

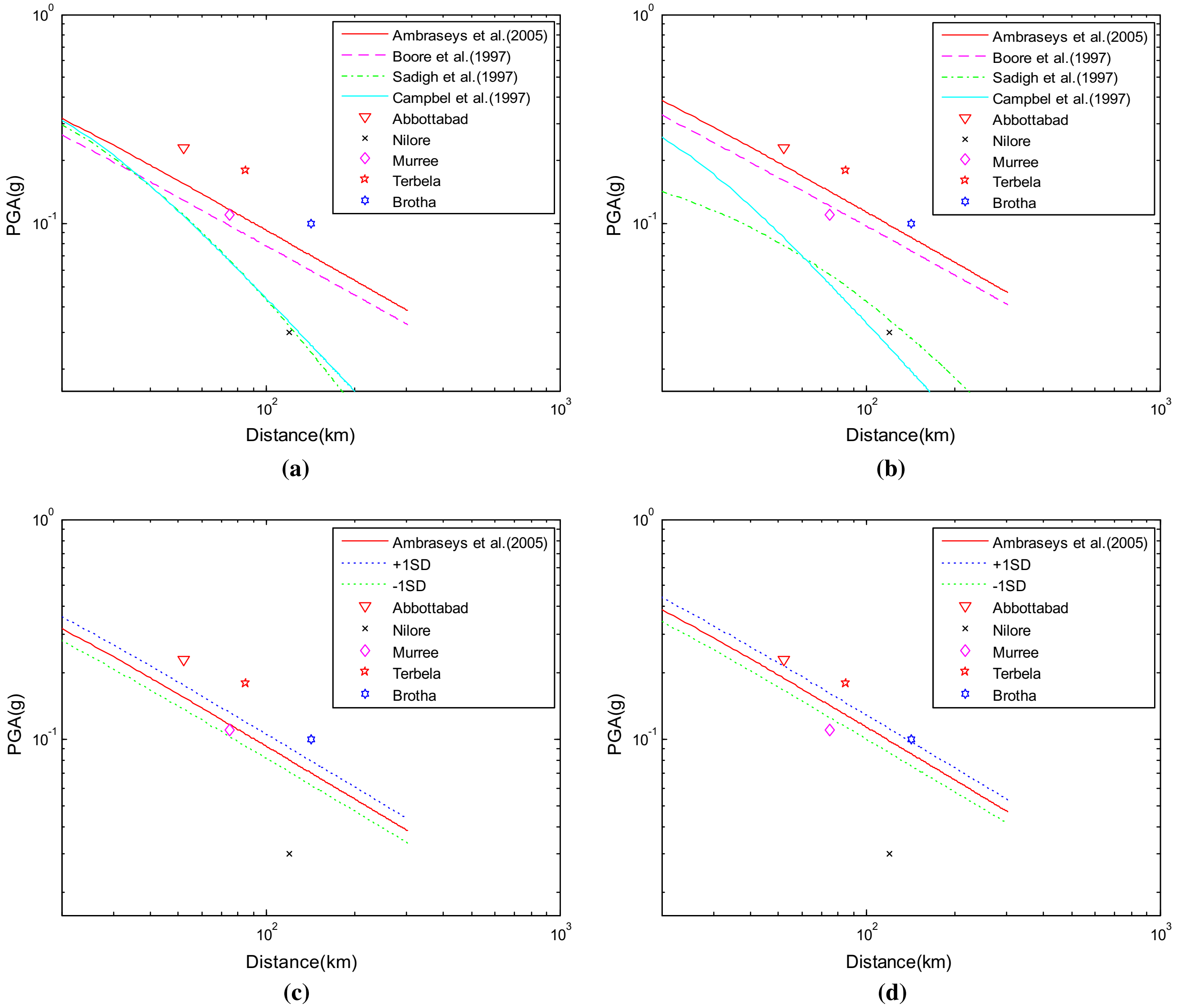
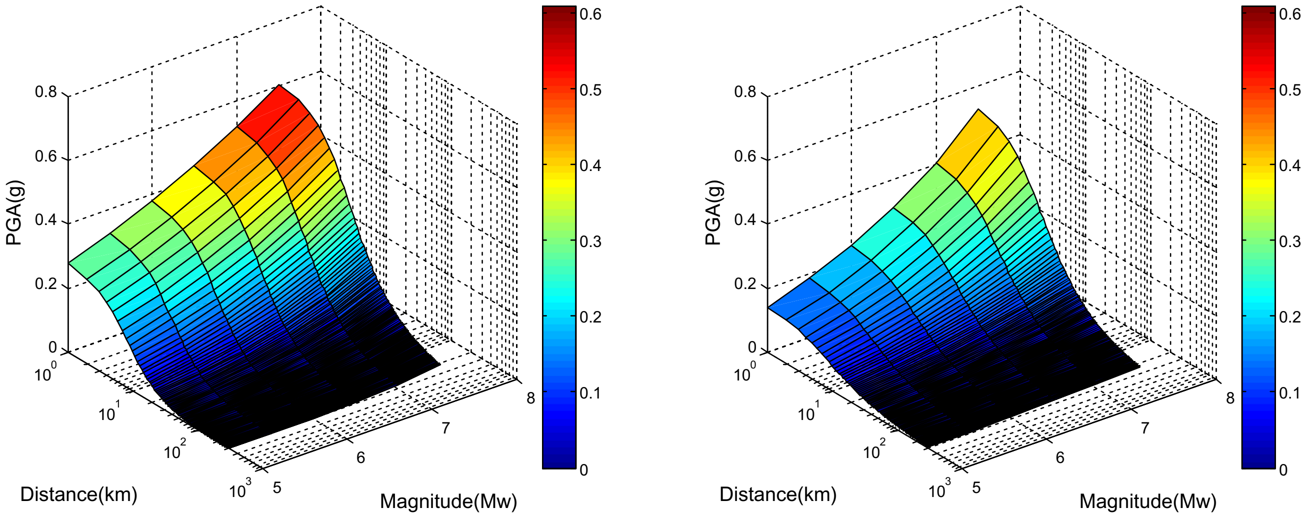


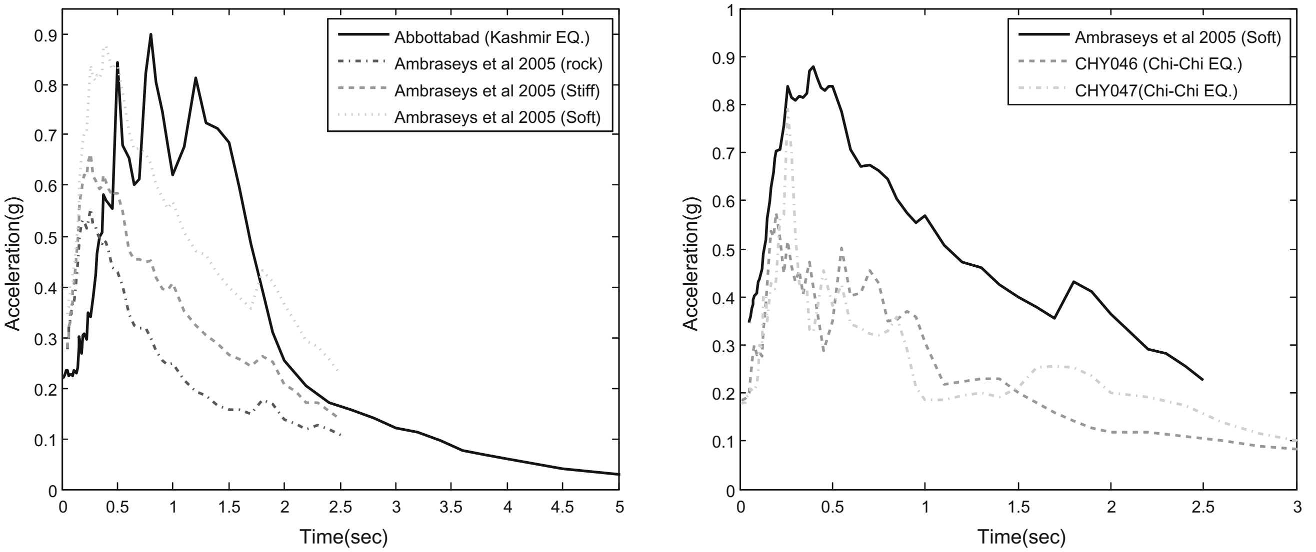
Fig. 7 Horizontal PGA prediction using attenuation relationships **a** Stiif soil, **b** soft soil, **c** stiff soil using Ambraseys et al.'s [31] relation with consideration of uncertainty, **d** soft soil using Ambraseys et al.'s [31] relation with consideration of uncertainty

To check the suitability of the different attenuation equations, PGA values at different distances are calculated for different soil site conditions and are compared with those recorded at different stations during the Kashmir earthquake, as shown in Fig. 7. The PGA predictions from Ambraseys et al. [31] and Boore et al.'s [29] relationships showed the closest agreement with the recorded PGA values (at different stations) at different distances, with exception of the Nilore station horizontal readings, which are believed to be probably affected by the size of raft where instrument was anchored. The horizontal PGA reading at Nilore is overestimated using Ambraseys et al. [31]. However, vertical PGA recorded at Nilore is well predicted and supports the effect of raft size on horizontal PGA at Nilore station (Durrani et al. [6]). Among Ambraseys et al. [31] and Boore et al.'s [29] relationships, Ambraseys et al.'s [31] gave slightly better and conservative results than Boore et al.'s [29]. Campbell et al. [28] and Sadigh et al. [30] underestimates the recorded ground motions. At large distances, the PGA prediction by Campbell et al. [28] and Sadigh et al. [30] decreases rapidly. Horizontal PGA prediction on stiff soil in Fig. 7a shows closer agreement of Ambraseys et al.'s [31] relation with PGA recorded at Murree, whereas PGA prediction on soft soil (Fig. 7b) matches well with Abbottabad recorded PGA value. This is very realistic because Murree and Abbottabad typically have stiff and soft soil, respectively. The PGA predictions for stiff and soft soil considering uncertainty (±1 standard deviation) of Ambraseys et al.'s [31] relation are shown in Fig. 7c, d, respectively. Figure 7d shows good agreement of all stations except Nilore.



(a) **(b)**

Fig. 8 Three-dimensional attenuation surface comparison **a** Ambraseys et al. [31], **b** Boore et al. [29]



(a) **(b)**

Fig. 9 **a**, **b** Comparison of the deterministic spectra with (i) Kashmir earthquake spectrum (Abbottabad station) (ii) with Chi-Chi earthquake spectra

A comparison is made between three-dimensional attenuation surfaces using Ambraseys et al. [31] and Boore et al.'s [29] relationships by considering different magnitudes (between 5 and 7.5) and stiff soil conditions as shown in Fig. 8a, b. Ambraseys et al.'s [31] equation clearly predicts higher PGA at short distances as compared to Boore et al. [29] at any magnitude level. Given the high levels of damage in the Muzaffarabad and Balakot, cities near the fault of the Kashmir earthquake, it appears that the Ambraseys et al.'s [31] equation is likely to predict better the amplification of ground motion for the study region.

4.1 Verification of Deterministic Ground Motion Spectra

To further verify the suitability of the selected attenuation relationship, the Kashmir earthquake ground motion recorded at Abbottabad station is selected since it was the nearest to the epicentre. It should be noted that Abbottabad city sustained heavy damages during the Kashmir Earthquake. Other strong ground motion records having similar characteristics to the Kashmir earthquake can also be utilized. Two records from the Chi-Chi earthquake (1999), Taiwan, are selected having focal depth of 33 km, thrust

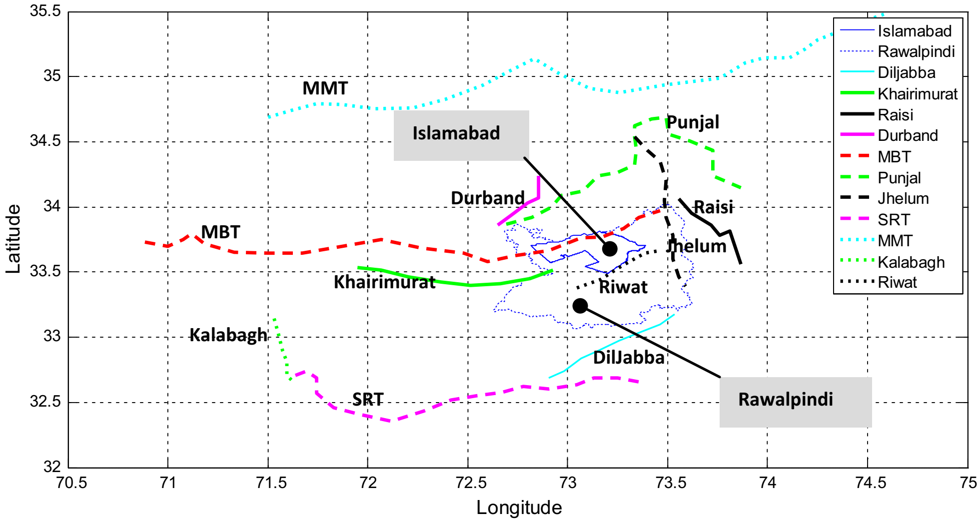


Fig. 10 Major faults in 150 km radius around Islamabad–Rawalpindi region [after Monalisa et al. [19]]

Table 5 Details of important faults around Islamabad-Rawalpindi region

Tectonic feature	Total fault length (km)	Fault rupture length (km)	Maximum potential magnitude (M_w)
Panjal Thrust	98	49	7.4
MBT	353	177	7.8
Jhelum fault	82	41	7.1
Khairabad fault	205	102	7.5
Raisi Thrust	200	100	7.5
Darband fault	47	24	6.8
MMT	339	170	7.6
Kairi-i-Murat fault	164	82	7.4
HFT	225	112	7.2
Riwat Thrust	48	24	6.8
SRT	100	50	7.2

source mechanism, $M_w = 7.6$ (very similar to the Kashmir earthquake). These earthquake records are from the Chiayi station (CHY046 and CHY047) with soft soil conditions and having duration of 43.60 and 53.53 s, respectively.

Deterministic spectra, as shown in Fig. 9a, are generated, for the Abbottabad at different soil conditions using the characteristics of the Kashmir earthquake. These spectra do not match very well the spectrum obtained from the Abbottabad strong motion record and are unable to capture the amplification over a large range of periods. This highlights that there is uncertainty in these equations in predicting PGA for events with large magnitude and long duration. However, it should be noted that the soft soil deterministic spectra give better results, since the Abbottabad site had soft soil conditions. On soft soil condition, the spectrum matches well in small period range approximately up to 0.5 s, but afterwards on longer periods, spectral acceleration prediction by Ambraseys et al. [31] is generally underestimated. From Fig 9a comparison, it can be said that the spectra generated using Ambraseys et al.'s [31] relation with soft soil conditions may be adopted for deterministic analysis of low-rise structures.

On the other hand, the spectra from the Chi-Chi earthquake ground motion are enveloped by Ambraseys et al. [31] soft soil spectrum over entire period as shown in Fig. 9b, and spectral acceleration is overestimated in this case and may be used conservatively for design and assessment purpose of low- and midrise structures. These comparisons in Fig. 9a, b highlight uncertainty in amplification and corner periods, and they should be given serious consideration particularly in vulnerability and risk assessment and studies.

4.2 Deterministic Demand for the Study Area

The important faults according to NESPAK [11] within the 150-km radius of study area are shown in Fig. 10. Fault details are provided in Table 5. The maximum earthquake potential of these faults (Fig. 11) evaluated by Monalisa et al. [10] using Wells and Coppersmith [32] relation by considering half rupture length is used. The MBT is found to be the most critical fault ($M_w = 7.8$) since it passes through the study area as shown in Fig. 12.

The spectra generated using Ambraseys et al.'s [31] relationship for all important faults within 150 km radius of the study area for stiff soil conditions are shown in Fig. 13. Again, MBT clearly shows the highest spectral acceleration values. The maximum spectral acceleration values evaluated using these spectra are 1.09, 1.33, 1.84 g for stiff, soft and rock

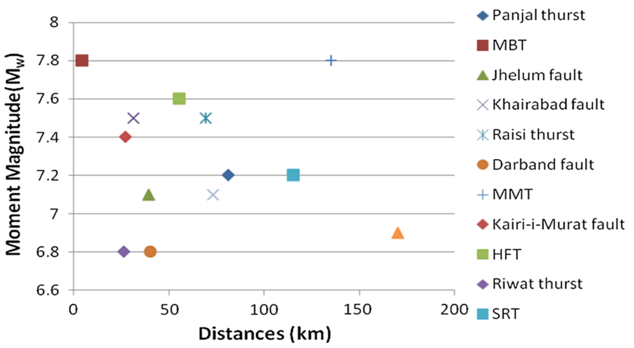


Fig. 11 Distance versus Maximum earthquake potential of different faults around Islamabad-Rawalpindi

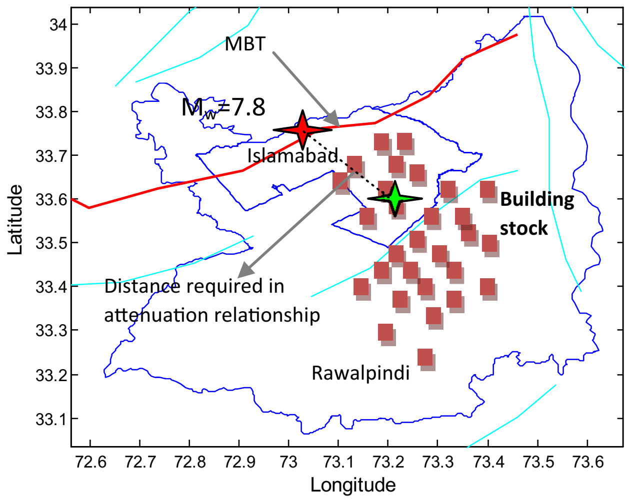


Fig. 12 Representation of the most active feature of the study area

sites, respectively. The other faults also have large earthquake potential ($M_w \geq 6.8$), but they are located relatively far from the study area and generate lower spectral acceleration values. Their relatively high magnitude range (between 6 and 7) is also believed to influence the amplification in the response spectrum [33,34]; hence, the uncertainty due to high magnitude near source earthquakes should be accounted in defining demand for design or assessment of specialized structures.

4.3 Smoothening of Deterministic Spectra and Evaluation of Corner Periods

The deterministic spectra need to be smoothened for use in either design or assessment. The FEMA356 [35] procedure was followed to determine the smooth horizontal response spectra by calibration. Researchers have used this method for evaluation of corner periods after smoothening of spectra [18].

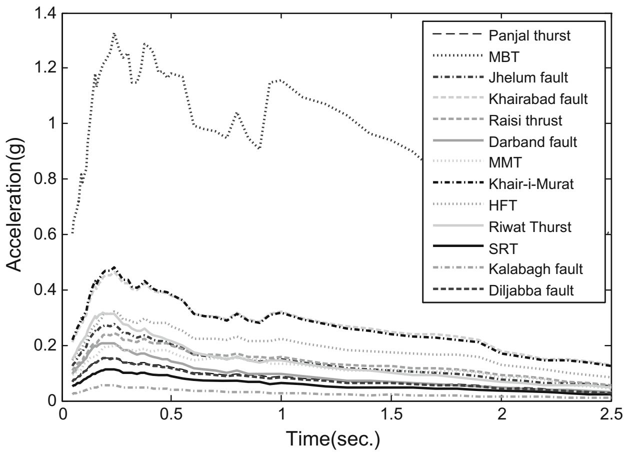


Fig. 13 Deterministic response spectra of faults within 150 km radius of study area

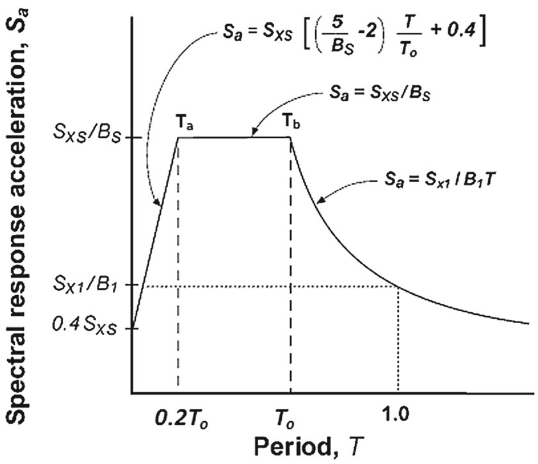


Fig. 14 General response spectrum construction based on FEMA356 (2000)

The following functions [Eqs. (2)–(4)] are plotted in the spectral acceleration and time domain to construct the smoothened response spectrum.

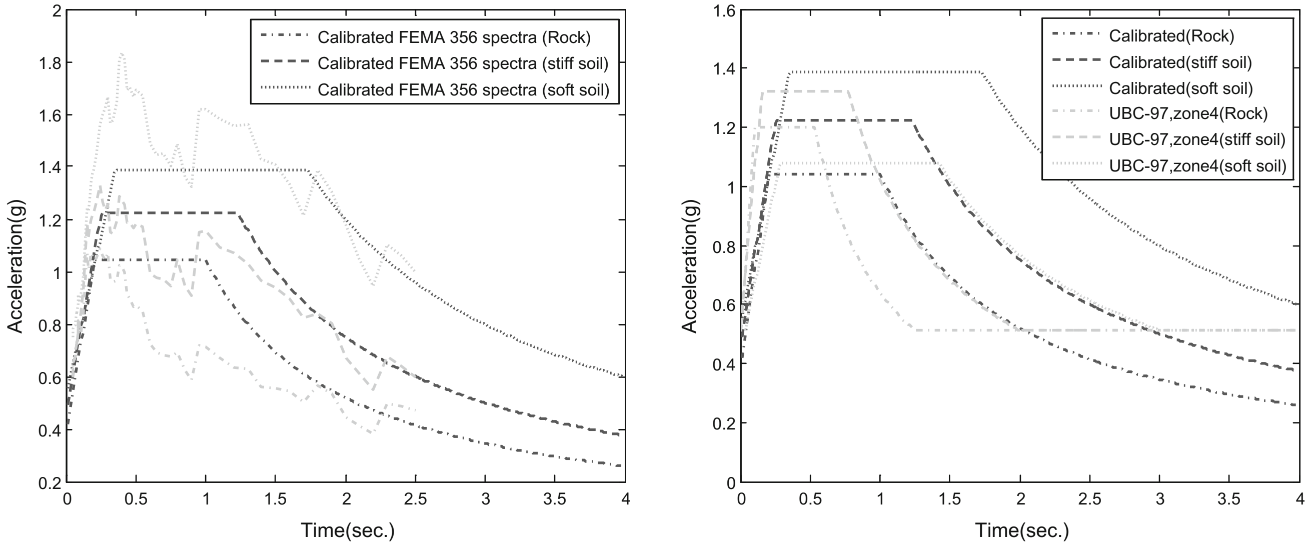
$$S_a = \left[\frac{S_{XS}}{B_S}\right]\left[0.4 + 3\frac{T}{T_O}\right] \quad 0 < T \leq 0.2T_O \tag{3}$$

$$S_a = \frac{S_{XS}}{B_S} \quad 0.2T_O < T \leq T_O \tag{4}$$

$$S_a = \frac{S_{X1}}{B_1 T} \quad T > T_O \tag{5}$$

$$T_O = \left[\frac{S_{X1}}{B_S}\right] \Big/ \left[\frac{S_{XS}}{B_1}\right] \tag{6}$$

where B_S and B_1 are taken as 1 for 5% of critical damping, S_{XS} = design short period (0.2 s) response acceleration parameter, S_{X1} = design response acceleration parameter at 1 s, S_{XS} is determined directly from the generated deterministic spectrum at the 0.2 s period.



(a) **(b)**

Fig. 15 **a** Calibrated response spectra for MBT (rock, stiff soil, soft soil), **b** comparison between calibrated FEMA 356 (2000) and UBC-97

Table 6 Corner period values in seconds obtained from FEMA (356) calibrated and UBC-97 spectrum

Spectrum	Distance (km)	Rock		Stiff soil		Soft soil	
		T_a	T_b	T_a	T_b	T_a	T_b
FEMA356 [36] (calibrated)	5	0.198	0.990	0.244	1.22	0.344	1.721
UBC-97 [20] (zone 4)	5	0.106	0.533	0.155	0.776	0.284	1.422

The part of the curve after T_o is determined in such a way that the obtained value of spectral acceleration (S_a) is not less than 90% of the spectral acceleration values of the generated spectrum. This is done by iterating for different S_{X1} values and using function $S_a = S_{\mathrm{X1}}/T$. After finding the curved portion, the corner periods T_a and T_b ($0.2T_o$ and T_o) are established. T_a and T_b provide the lower and upper limits for the period of the constant spectral acceleration branch. This approach can be used to develop 5% damped elastic spectra by utilizing the deterministic spectra evaluated from attenuation relationships. The calibrated spectra shown in Fig. 15a are developed for MBT for different soil conditions.

In Fig. 15b, a comparison is made between the calibrated FEMA 356 [35] spectra and UBC-97 [20] spectra. For UBC-97 spectra, the study area is considered in zone 4 and seismic coefficients according to this zone for rock, stiff and soft soil are selected. Near source factors are chosen according to the seismic source type ‘A’, which represents an active fault capable of generating large earthquake located at 5 km.

The comparison shows slightly lower amplitude of the calibrated spectrum for stiff soil, but with a larger plateau as compared with UBC-97 stiff soil spectrum. The same observation applies to the rock site, whereas for soft soils the amplitude of the calibrated spectrum is higher than the UBC-97 spectrum and with a longer plateau. The comparison between the corner period values of the calibrated and UBC-97 spectra (for site-source distance of 5 km) is given in Table 5. A significant increase in corner periods for calibrated spectra is observed for all three soil types. This increase is related to the fact that longer duration earthquakes have more chances of including lower frequency waves resulting in higher demand in the response spectrum in the longer period region. This has been shown by other researchers [33,34] who used longer duration earthquake records.

The maximum possible demand for the building stock of the study area can be defined using the calibrated spectra. Moreover, these spectra can also be used to define seismic demand of the specialized structures of the study region.

5 Conclusion

PSC-86 has the oldest seismic zoning map for Pakistan, and the provisions of this code can be used for calculating seismic design forces of existing engineered structures in building stock with basic seismic design considerations. This design category of RC structures must be considered for vulnerability assessment. In comparison with the latest BCP (2007) seismic zoning map, PSC-86 suggests very low seismic hazard for the study area and the Kashmir earthquake affected areas. The demand comparison shows that the existing building stock of study area is highly vulnerable.

Recent PSHA studies show significant variability in predicted PGA values for 500-year return period. For the study area, PMD-NORSAR [3] predicts the largest PGA value of 0.37 g. Hence, the seismic forces (*V*/*W*) according to PMD-NORSAR [3] for the ductile and non-ductile four-storey RC structure are found to be significantly higher as compared to the seismic forces evaluated from PSC-86 spectrum. However, BCP [12] hazard value is commonly used these days for the seismic forces calculation of modern RC structures of the study area. Monalisa et al. [10] predicted the least PGA values of 0.15 g for the study area. The variability in probabilistic hazard predictions may be used in propagating the demand uncertainty in vulnerability assessment of building stock of study area.

Ambraseys et al. [31] attenuation relationship is suitable for Pakistan and is adopted for generating deterministic demand spectra which are considered maximum for ordinary as well as specialized structures. New corner periods (for the study area) obtained through smoothening of deterministic demand spectra are even higher than those predicted using the UBC zone 4 demand. Both the probabilistic and deterministic demand findings have great significance in seismic design and assessment of the structures of study area.

Appendix

See Figs. 16, 17, 18 and 19.

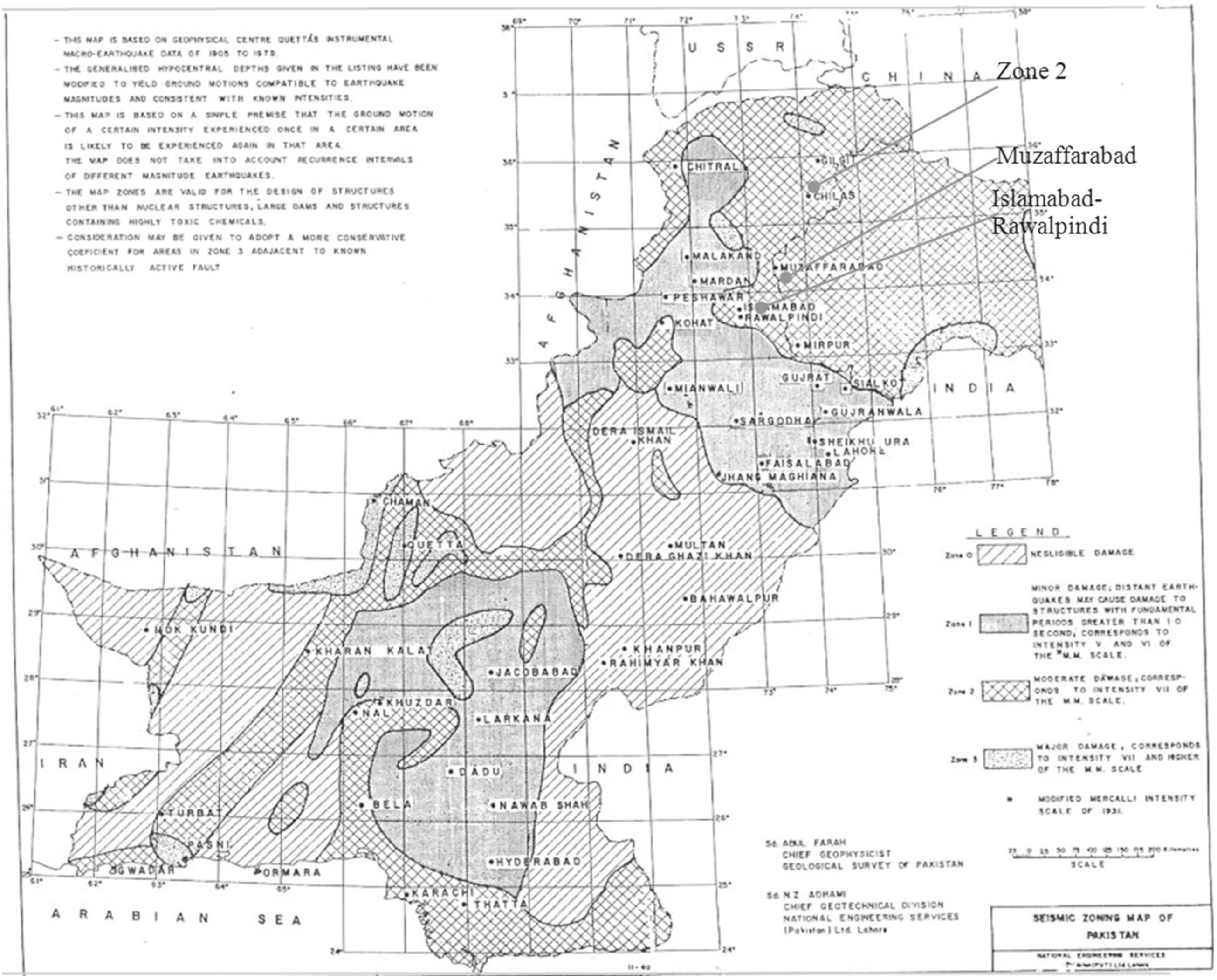


Fig. 16 Seismic zoning map from Pakistan Seismic Code 1986 (PSC-86)

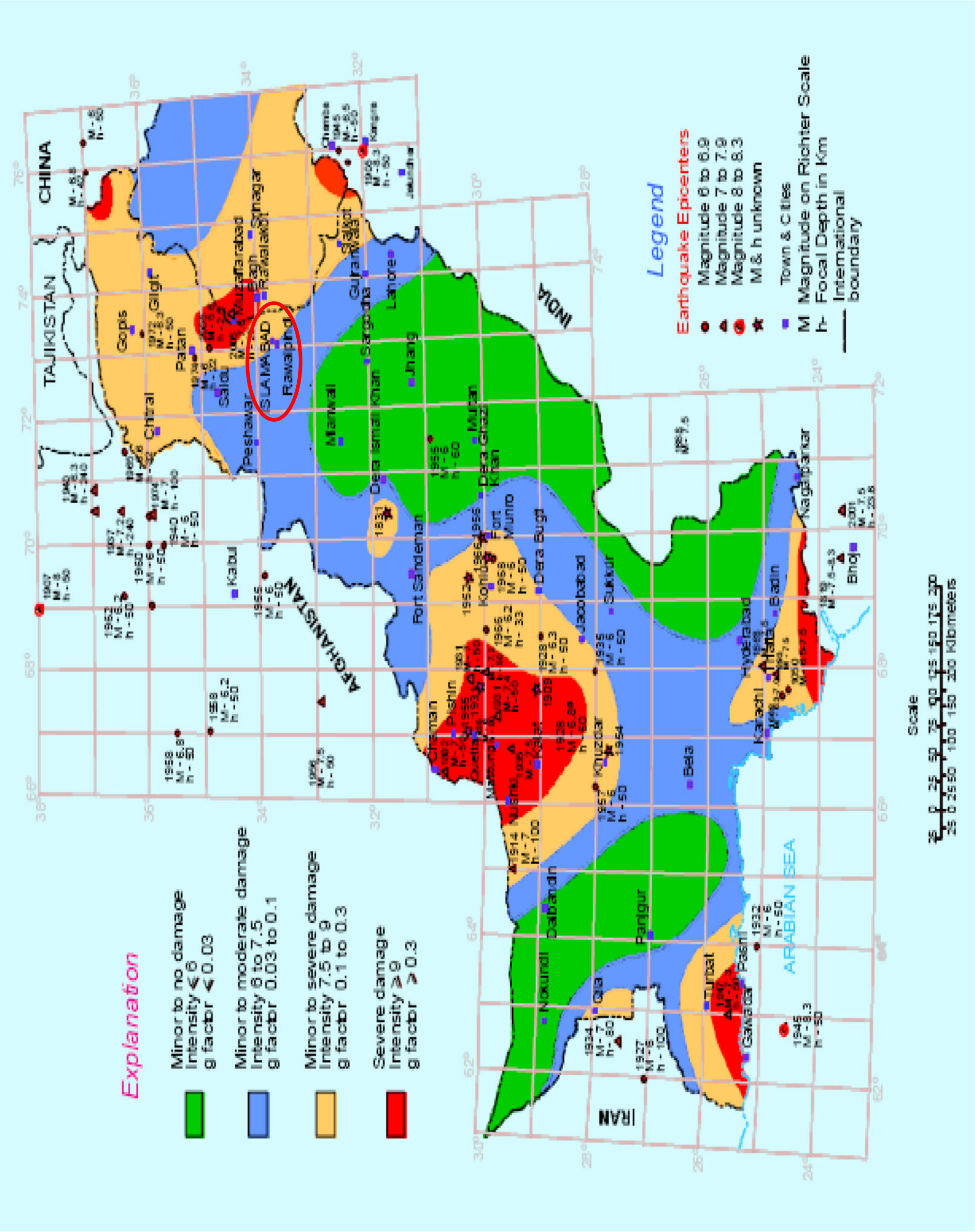


Fig. 17 Seismic hazard map of Pakistan by GSP (2005) (after Kashmir earthquake (2005))

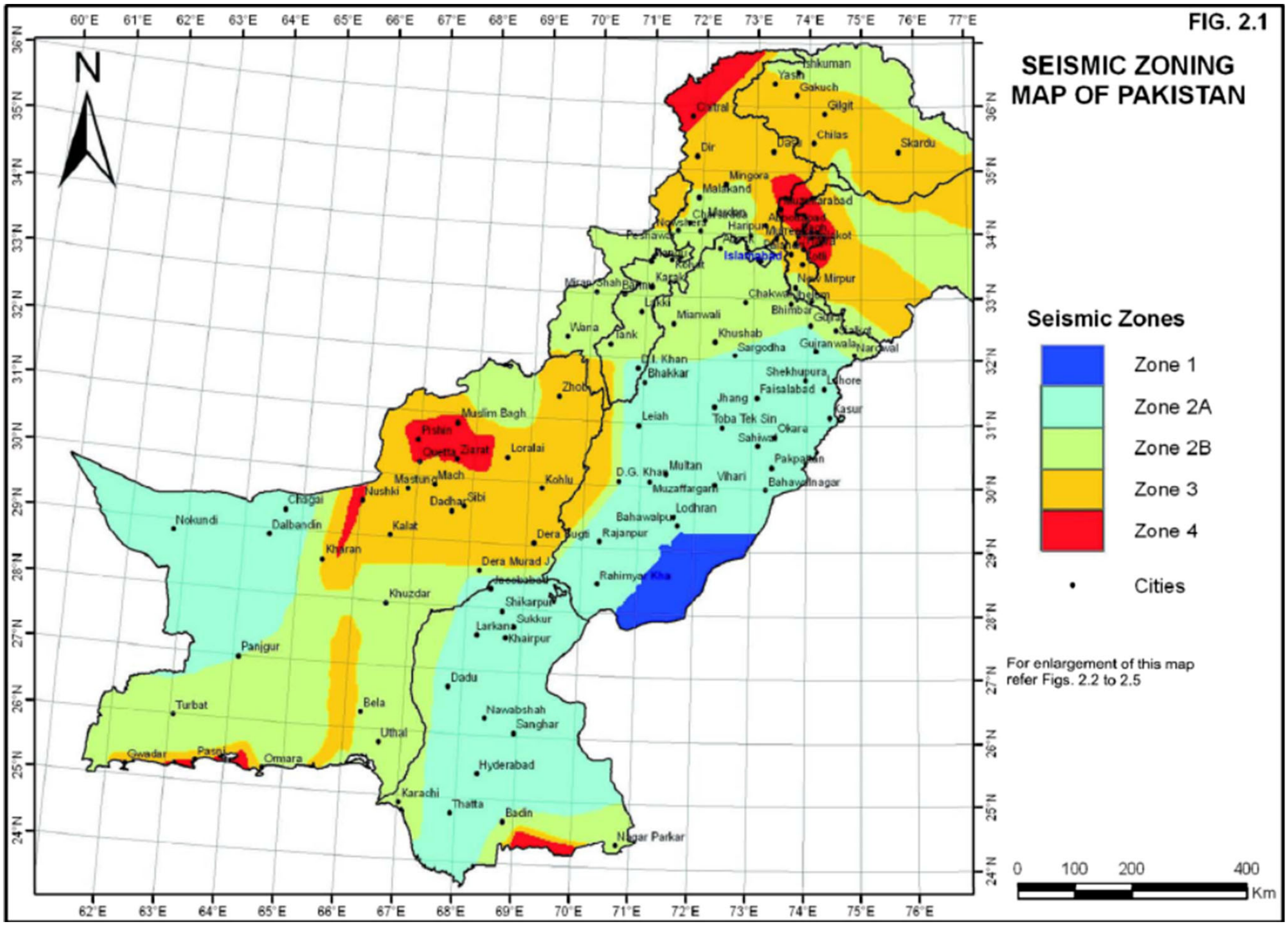


Fig. 18 Seismic hazard map of Pakistan by BCP (2007) (after Kashmir earthquake (2005))

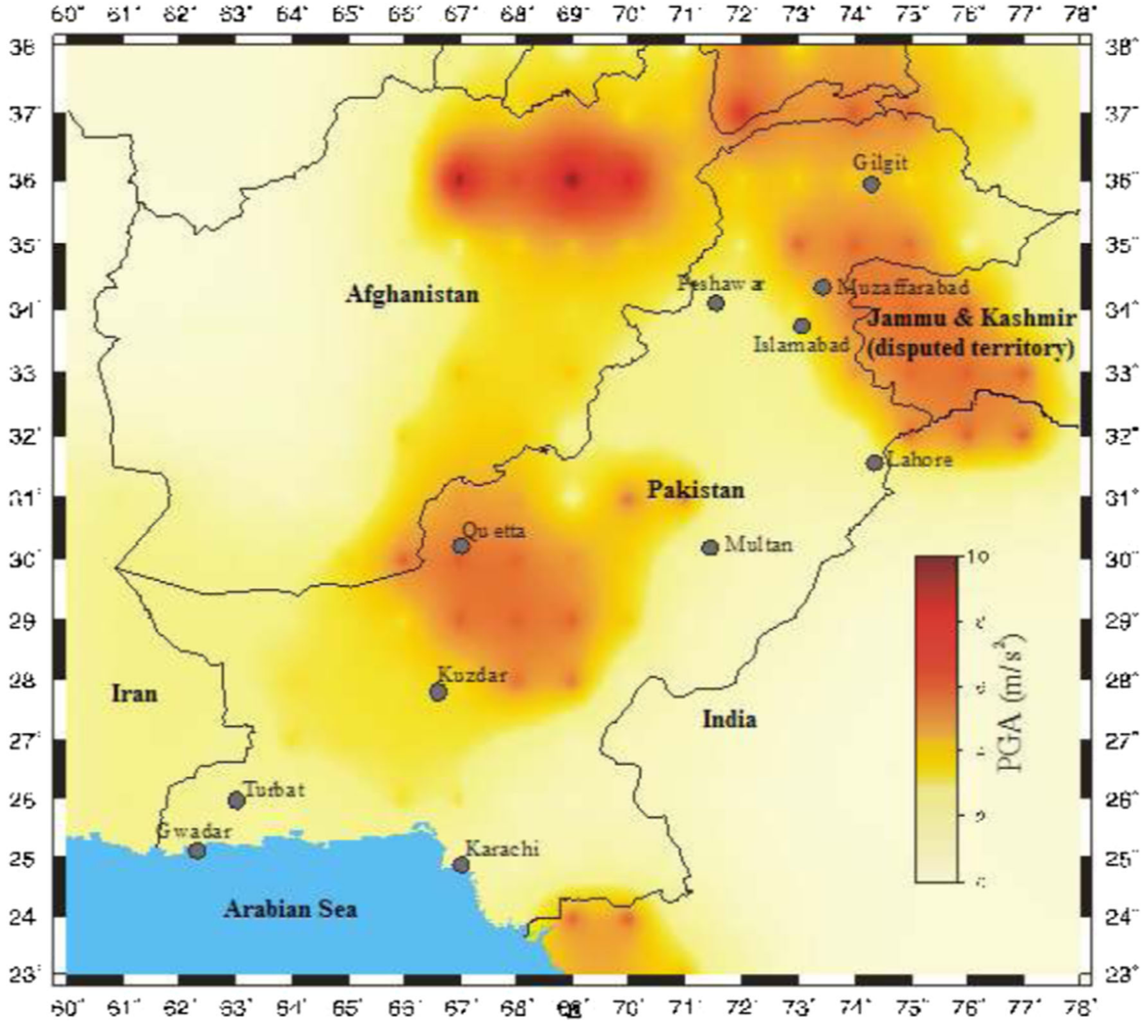


Fig. 19 Seismic hazard map of Pakistan by PMD-NORSAR (2007)

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