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DETERMINATION OF THE PEAK GROUND ACCELERATION FOR OPERATING BASIS EARTHQUAKE AND ITS INFLUENCE IN DESIGN OF STRUCTURES

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ABSTRACT
Safety-related structures, systems and components of a Nuclear Power Plant (NPP) are required to be designed for the safe shutdown earthquake (SSE) and operating basis earthquake (OBE). The OBE could reasonably be expected to affect the plant site during the operating life of the NPP. The SSE is based on the maximum earthquake potential considering the geological and tectonic settings and produces the maximum vibratory ground motion. The earthquake ground motion is characterized by its peak ground acceleration (PGA) and the response spectral shape i.e. the frequency-dependent dynamic amplification factors (DAFs). The minimum PGA for OBE to be considered in design in certain regulations is related to the PGA for SSE.

This paper presents a case study of two NPP sites - Tarapur and Kakrapar in India. Based on a combination of linear and aerial earthquake source model, the observed earthquake data over a long period of time and an attenuation relation for spectral acceleration, PGAs are evaluated for various specified values of the mean recurrence interval (i.e. return period). The return period for PGA for OBE is taken, in many countries, to be of the order of 10^2 years. From this study, it is seen that the ratio of the values of PGA for OBE and SSE is site-specific. The paper presents the distribution of the calculated values of PGA at site during past earthquakes, the computed ratio of the values of PGA for OBE and SSE and the sensitivity of this ratio to the parameters in the earthquake occurrence model. The response of structures to OBE and SSE, considering the same shape of the ground motion time-history will be dependent on the applicable values of damping. Similarly, the earthquake response of various equipment or piping supported on various floors of the structure will be dependent on damping. The paper presents a simple case study to show the response of a structure and equipment and computes the ratio of the peak responses to OBE and SSE.

The load factors for the structure and the allowable stresses for the piping or equipment are different for these two levels of earthquake. The paper presents the limiting value of the ratio of the PGA under OBE and SSE for which OBE rather than SSE would govern the design for the case considered.

Keywords: Safe Shutdown Earthquake, Operating Basis Earthquake, Peak Ground Acceleration, Service Level, Allowable Stress

1. INTRODUCTION

Safety-related structures, systems and components of a Nuclear Power Plant (NPP) are required to be designed for the safe shutdown earthquake (SSE) and operating basis earthquake (OBE). The OBE could reasonably be expected to affect the plant site during the operating life of the NPP. The SSE is based on the maximum earthquake potential considering the geological and tectonic settings and produces the maximum vibratory ground motion. The earthquake ground motion is characterized by its peal ground acceleration (PGA) and the response spectral shape i.e. the frequency-dependent dynamic amplification factors (DAFs). The minimum PGA for OBE to be considered in design in certain regulations is related to the PGA for SSE.

This paper presents a case study of two NPP sites - Tarapur and Kakrapar in India. Based on a combination of linear and aerial earthquake source models, the observed earthquake data over a long period of time and an attenuation relation for spectral acceleration, PGAs are evaluated for various specified values of the mean recurrence interval (i.e. return period). The return period for PGA for OBE is taken, in many countries, to be of the order of 10^2 years. From this study, it is seen that the ratio of the values of PGA for OBE and SSE is highly site-specific. The paper presents the distribution of the calculated values of PGA at site during past earthquakes, the computed ratio of the values of PGA for OBE and SSE and the sensitivity of this ratio to the parameters in the earthquake occurrence model. The response of structures to OBE and SSE, considering the same shape of the ground motion time-history will be dependent on the applicable values of damping. Similarly, the earthquake response of various equipment or piping supported on various floors of the structure will be dependent on damping. The paper presents a simple case study to show the response of a structure and equipment and computes the ratio of the peak responses to OBE and SSE. This ratio is significantly different from the ratio of the values of PGA under SSE and OBE.

The load factors for the structure and the allowable stresses for the piping or equipment are different for these two levels of earthquake. The paper presents the limiting value of the ratio of the PGA under OBE and SSE for which OBE rather than SSE would govern the design for the case considered.

2. PGA FOR SSE AND EVALUATION OF PGA AT A SITE FROM PAST DATA OF EARTHQUAKES.

Based on a detailed seismotectonic study, the PGA for SSE was evaluated for the two sites – Tarapur and Kakrapar (DEQ(1981), DEQ(1985), Ghosh and Banerjee(1990)). A circular area of radius 300 km was considered for each site. Using the attenuation relation (Ghosh et al. (1998)), PGA at these sites were evaluated for recorded past earthquakes (Gangrade et al. (1987,1996), AERB(1993) and USGS). The records of earthquakes from historical times to the present were obtained from various earthquake catalogues. Apart from data various global sources for the period 1504 AD to the year 2003 earthquake data were obtained from the Gauribidanur Seismic Array (GBA) (for the period 1977 to 2000) of the Bhabha Atomic Research Centre (GAngrade et al. (1987,1996)). The results from global and GBA data are presented separately. Table-1 gives a summary of the magnitude of various earthquakes (global data) in the 300 km radius circles around these sites. Earthquakes around Koyna occur in a small cluster (taken as an aerial source) in the 300 km radius circle around Tarapur. Figs. 1(a) and 1(b) present the histogram of calculated values of PGA at Site of the past earthquakes for Tarapur Site for global and GBA data respectively. Figs. 1(c) and 1(d) present the corresponding histograms for Kakrapar site.

2.1 PGA at Site with A Specified Value of Return Period or Probability of Exceedence.

Earthquake occurrence rate model given by the Gutenberg-Richter Relation (Richter (1959))

$$\log_{10} N_M = a - bM \qquad (1)$$

was developed for each of the two sites (Ghosh and Kushwaha (2003 a, b), Ghosh et al. 2003). Based on the tectonic and seismological data, the area around Tarapur Site was modeled as a combination of linear and aerial source of earthquakes while Kakrapar Site was modeled with linear sources of earthquakes.

Spectral attenuation relation was developed from a large ensemble of accelerograms of strong motion earthquakes recorded on rock sites under free field conditions (Ghosh et al. 2003).

$$S = S(M, R_h, \zeta,.T) = b_1 \exp(b_2 M) (R_h+D)^{-b_3} \qquad (2)$$

Assuming earthquakes to be occurring equally likely anywhere along the length of a fault (line source) or anywhere within as aerial source the return period (T_y) of a spectral acceleration or its probability of exceedence (P) within a specified span of time is computed (Ghosh et al. (2003)). Similarly, the spectral acceleration is evaluated for specified values the return period or the probability of exceedence. Peak ground acceleration is obtained as the spectral acceleration at zero period. Earthquakes are assumed to occur as a Poisson distribution.

Table-2 presents the computed values of the ratio of PGA for individual events to the PGA for SSE for Tarapur and Kakrapar Sites. The results are also shown graphically in Figs. 2a-2d. The ratio (R) of the value of PGA for a specified value of T_y to the value of PGA for SSE is presented for a ± 10% variation in a and b around the reference (base) values. While one parameter is varied, the other is kept constant at its base value. For T_y = 100 years, R varies between 0.41 and 0.64 around a base value of 0.54 for Tarapur Site. For Kakrapar Site, for T_y = 100 years, R varies between 0.17 and 0.28 around a base value of 0.22. There appears to be various practices of determining the peak ground acceleration for operating basis earthquake (OBE). The PGA for OBE is required to be at least a given fraction of the PGA for SSE (e.g. half) according to some Codes and Guides (USAEC(1973)/USNRC(1997), AERB(1990), for example). The return period of OBE is typically of the order of one or few hundred years (Stevenson (1975), IAEA (1989)). From the Table-2 and Figs. 2a-2d pertaining to the two sites studied, it is thus seen that the ratio R, which can be taken as the ratio of the PGA for OBE to that for SSE when T_y is of the order of 100 years can vary from about 0.17 to 0.64. This ratio is dependent on the parameters of the earthquake recurrence relation, attenuation relation and the distance of various faults from the site.

3. RESPONSE OF STRUCTURES AND COMPONENTS TO SSE AND OBE

Generally, in nuclear power plants, components and equipment are mounted on several floors of various structures. The earthquake excitation is applied at the base of the structure. The effect of soil is generally modeled through soil springs if soil-structure interaction is required to be considered, in which case the earthquake excitation is applied at the base of the soil springs. The response of the structure depends on its dynamic characteristics (e.g. natural frequencies) including damping and the characteristics of the ground motion (i.e. its amplitude and frequency contents).

A case study was carried out on a two-degree-of freedom (dof) system excited at the base. Table-3 shows the parameters of the system. The following parameters are defined.

$$R_m = \frac{m_2}{m_1} \qquad (3)$$

$$\omega_j^2 = k_j / m_j \quad \text{(j=1,2)} \qquad (4)$$

$$R_f = \frac{\omega_2}{\omega_1} \qquad (5)$$

$$\varsigma_j = c_j / 2m_j\omega_j \quad \text{(j=1,2)} \qquad (6)$$

Response of the system to two recorded earthquakes (GMD1, GMD2) was evaluated for two different sets of values of the damping ratio ζ_1 and ζ_2 corresponding to the damping values of RCC structure and equipment respectively for SSE and OBE conditions. (USNRC(1973)). The time- history GMD1 corresponds to the N21E component of Taft Lincoln School Tunnel recording of Kern County Earthquake of July 21, 1952 (04:53 PDT) whereas GMD2 corresponds to the N65W component of Temblor recording of Parkfield Earthquake of June27, 1966 (20:26 PST).

Table-4 presents the summary of the peak displacements of the two masses of the 2-dof system. The frequency ratio, R_f was varied from 0.1 to 1.0. The ratio (R1) of the peak displacements for damping value corresponding to OBE to the peak displacement for damping value corresponding to SSE was evaluated for each mass.

Table-5 shows the summary of the R1 values for the 2-dof system for the two earthquake records considered. The ratio (R1) will be dependent on the structural characteristics of the system and the amplitude and frequency contents of the earthquake. The limited study with the 2-dofs by varying the R_f and two earthquake records only is to show the variability of R1. In order to estimate an approximate bounding value of R1 for the primary structure, the structure can be modeled as a sdof system. Then the maximum peak response (acceleration/displacement) of the structure will be given by the corresponding response spectral value of earthquake ground motion. The earthquake ground motion response spectra for rock and soil sites have been presented among others by Ghosh et al.(1998).

Thus for the primary structure (Sec. Table-3), considering a critical damping ratio of 7% and 4% for SSE and OBE respectively, the value of R1 works out to be 1.261(=3.782/3.0 – the values in the numerator and denominator corresponding to the peak normalized spectral acceleration values at the aforementioned values of damping, the corresponding period is 0.2448 s). For pipings having diameter less than 300 mm, the critical damping ratio is specified as 1% and 2% respectively for OBE and SSE. Following the approach of (Stevenson(1975)), for the piping system (considered as a secondary system in the 2-dof modeling), a preliminary estimate of the ratio R1 is made from the normalized peak spectral acceleration values at critical damping ratios of 1% and 2% respectively.

$$R_1 = 1.261 * \frac{\text{Response spectral shape at 1\% damping}}{\text{Response spectral shape at 2\% damping}}$$

$$= 1.261 * 1.446 = 1.823$$

The corresponding range of period is from 0.1 s - 0.2 s. The normalized peak spectral values mentioned above are for the mean-plus-sigma level normalized ground motion response spectra derived from 144 earthquake records. The results presented in Table-5 are for two earthquake records only.

4. APPLICATION TO DESIGN

4.1Consideration for Safety Class Concrete Structures

Category 1 structures are required to be designed for the following load combinations.

$$F_{OBE} = 1.4D + 1.7L + 1.7\,E_{OBE} \tag{7}$$

$$F_{SSE} = 1.0D + 1.0L + 1.0\,E_{SSE} \tag{8}$$

where D = Dead Load, L = Live Load, E_{OBE} = Earthquake Load due to OBE and E_{SSE} = Earthquake Load due to SSE.

The base shear and the moment for a degree of freedom will be proportional to the displacement of the mass relative to the one just below it. So, neglecting D and L,

$$\frac{F_{OBE}}{F_{SSE}} = \frac{1.7\,E_{OBE}}{1.0\,E_{SSE}} = 1.7 \times R_1 * R = 2.1437\,R$$

(As seen in the previous section, the maximum value of R1 for concrete structures is 1.261). Thus it is seen that in the relevant frequency range where R_1 applies

$$\frac{F_{OBE}}{F_{SSE}} > 1 \quad \text{if} \quad R > \frac{1}{2.1437} = 0.466$$

i.e. in case only earthquake loads are considered, OBE would govern the design if R = the ratio of the PGA_{OBE} to PGA_{SSE} exceeds a value of 0.466. If the dead load and the live load are also taken into account, OBE will govern the design at still lower values of R. This limiting value of R depends, among other things, on the PGA for SSE and the ratio of the live load to dead load. Higher the fraction of the live load, lower will be the limiting value.

4.2 Consideration for Safety Class Piping

For Class I piping :

$S_{all} = 1.5\ S_m$ for upset conditions of design (corresponding to OBE) whereas S_{all} for faulted conditions(corresponding to SSE) (Stevenson (1975)) = $3.0\ S_m$

$$\text{Stress due to (Dead Load + Pressure + Earthquake}_{OBE}) \leq 1.5\ S_m \tag{9}$$
$$\text{Stress due to (Dead Load + Pressure + Earthquake}_{SSE}) \leq 3.0\ S_m \tag{10}$$

In a simplified method of uncoupled analysis of the piping, the base motion of the piping can be derived from the response of the structure without considering the stiffness of the piping. The response of the piping can be analysed as a sdof system.

The maximum peak stress in the piping will be proportional to the product of PGA, maximum DAF for the structure and the maximum DAF for the piping.

If the stresses due to dead load and pressure load are neglected, then

$$\frac{(\text{Stress})_{OBE}}{(\text{Stress})_{SSE}} = R \times R_1 \times R_2$$

$$\text{where } R_2 = \frac{\text{response of the piping at value of damping for OBE}}{\text{response of piping at the value of damping for SSE}}$$

Thus considering the highest peak spectral values

$$\frac{(\text{Stress})_{OBE}}{(\text{Stress})_{SSE}} = 1.823\ R$$

$$\text{If} \quad (\text{Stress})_{SSE} = 3\ S_m$$

$$\text{then} \quad (\text{Stress})_{OBE} = (S_m)\ (3 \times 1.823\ R) \leq 1.5\ S_m$$

$$\text{i.e. } R \leq \frac{1.5}{3 \times 1.823} = 0.274$$

If it is assumed that the sum of the stresses due to dead load and pressure is α times the stress due to SSE, and as before, it is assumed that sum of all relevant stresses combination with SSE is 3 S_m, then

$$(1 + \alpha)\ (\text{Stress})_{SSE} = 3\ S_m$$

Hence $(\text{Stress})_{OBE} + \alpha\ (\text{Stress})_{SSE} \leq 1.5\ S_m$

i.e. $\{ R R_1 R_2) + \alpha \} (\text{Stress})_{SSE} \leq 1.5 S_m$

i.e. $(1.823 R + \alpha) \frac{3S_m}{1+\alpha} \leq 1.5 S_m$

i.e. . $\frac{1.823 R + \alpha}{1+\alpha} \leq \frac{1}{2}$

If $\alpha \geq 1$, this leaves no margin to satisfy the stress limit for the OBE load combination.

for a typical example, let us consider

$\alpha = 0.1$, for which $R \leq 0.247$ and for $\alpha = 0.5$, $R \leq 0.137$

For Class 2 and Class 3 piping :

For load combination involving OBE

$S_{allowable} = 1.2 S_h$

and for load combination involving SSE

$S_{allowable} = 2.4 S_h$

If the stresses due to dead load and pressure are neglected then in this case also

$R \leq 0.274$

to satisfy the OBE's allowable stress condition when the criterion under SSE is just met. Considering non-zero values of α, in this case also,

$R \leq 0.247$ if $\alpha = 0.1$ and $R \leq 0.137$ if $\alpha = 0.5$.

5. CONCLUSIONS

The present paper considers evaluation of the PGA for OBE from probabilistic considerations for two NPP sites in India. It is seen that the ratio of the PGA in OBE to that in SSE varies quite significantly from one site to another. This is due to the variation in the parameters of the magnitude-frequency relationship and the location of the seismotectonic features. The paper next examines the influence of this ratio in the design of structures and piping through a case-study on a two-degree-of-freedom system representing a structure and a piping. Some specific lower bounds of this ratio have been worked out beyond which OBE rather than SSE would govern the design.

REFERENCES

D.E.Q. (1981), Design Basis Earthquake for Trombay and Tarapur Sites in Maharashtra, Project No. P-337, Department of Earthquake Engineering, University of Roorkee, U.P., India.

D.E.Q. (1985), Recommendations for Design Acceleration Response Spectra and Time History of Ground Motion for Kakrapar Site, Project No. P-332, Department of Earthquake Engineering, University of Roorkee, U.P., India.

Ghosh,A.K. and Banerjee, D.C. (1990), Earthquake Design Basis for Tarapur Site, Report to Nuclear Power Corporation of India Limited, Department of Atomic Energy, Government of India.

Ghosh, A.K., Rao, K.S. and Kushwaha, H.S. (1990), Development of Spectral Shapes and Attenuation Relations from Accelerograms Recorded on Rocks and Soil Sites, Report BARC/1998/E/016, Bhabha Atomic Research Centre, Government of India.

Gangrade, B.K., Prasad, A.G.V. and Sharma, R.D. (1987), Earthquakes from Peninsular India : Data From the Gauribidanur Seismic Array, Report BARC-1347, Bhabha Atomic Research Centre,

Government of India.
Gangrade, B.K. and Arora, S.K. (1996), Peninsular Seismicity : A comprehensive study from Regional Earthquake Date of Two Decades from Gauribidanur Seismic Array, Report BARC/1996/E/024, Bhabha Atomic Research Centre, Government of India.
A.E.R.B. (1993), Catalogue of Earthquakes ($m \geq 3.0$) in Peninsular India, AERB Technical Document No. AERB/TD/CSE- 1, Atomic Energy Regulatory Board, Government of India.
http://neic.usgs.gov/neis/epic/epic.htm
Ghosh, A.K. and Kushwaha, H.S. (2003a), Development of Uniform Hazard Ground Motion Response Spectra for a Site, Paper K-217, 17th International Conference on Structural Mechanics in Reactor Technology, Prague.
Ghosh, A.K. and Kushwaha, H.S. (2003b), Seismic Hazard Analysis for an NPP Site, Paper ICONE 12-49475, 12th International Conference on Nuclear Engineering, Arlington, VA.
Ghosh, A.K., Rao, K.S. and Kushwaha, H.S. (2003), Development of Uniform Hazard Response Spectra for Tarapur, Trombay and Kakrapar Sites, Report BARC/2003/E/019, Bhabha Atomic Research Centre, Government of India.
Richter, C.F. (1959) Seismic Regionalisation, Bulletin of the Seismological Society of America, 49, pp.123-162.
U.S.A.E.C./U.S.N.R.C. (1973), (Revised 1997), Seismic and Geologic Siting Criteria for Nuclear Power Plants, 10 CFR Part 100, Appendix A, U.S. Nuclear Regulatory Commission.
A.E.R.B.(1990), Seismic Studies and Design Basis Ground Motion for Nuclear Power Plant Sites, Guide No. AERB/SG/S-11, Atomic Energy Regulatory Board, Government of India.
Stevenson, J.D. (1975), Rational Determination of the Operational Basis Earthquake and Its Impact on Overall Safety and Cost of Nuclear Facilities, Nuclear Engineering and Design, 35 pp.327-333.
I.A.E.A. (1989), Earthquakes and Associated Topics in Relation to Nuclear Power Plant Siting, NUSS Safety Guide No. 50-SG-S1, Rev.1, International Atomic Energy Agency,.
U.S.N.R.C. (1973), - Values of Damping for Seismic Design, Regulatory Guide RG 1.61.

TABLE - 1

Distribution Of Earthquakes Around Tarapur (GLOBAL DATA)

Sr. No.	Magnitude (M) Range $M_0 \leq M < M_1$	No. of Earthquakes in this Range	
		Koyna	Others
1	3.0 - 3.5	0	1
2	3.5 - 4.0	14	10
3	4.0 - 4.5	32	15
4	4.5 - 5.0	23	3
5	5.0 - 5.5	19	10
6	5.5 - 6.0	7	3
7	6.0 - 6.5	3	0
8	6.5 - 7.0	2	1

Distribution Of Earthquakes Around Kakrapar (GLOBAL DATA)

Sr.No.	Magnitude (M) Range $M_0 \leq M < M_1$	No. of Earthquakes in this region
1	3.0 – 3.5	51
2	3.5 – 4.0	20
3	4.0 – 4.5	29
4	4.5 – 5.0	19
5	5.0 – 5.5	11
6	5.5 – 6.0	8
7	6.0 – 6.5	2
8	6.5 – 7.0	1
9	7.0 – 7.5	0
10	7.5 – 8.0	1

TABLE – 2

Ratio of PGA (for Individual Event) to PGA for SSE

Tarapur

Sr.No.	Range	No. of Events
1	0.01-0.05	83
2	0.05-0.1	32
3	0.1-0.15	3
4	0.15-0.2	2
5	0.2-0.25	0
6	0.25-0.3	0
7	0.3-0.35	0
8	0.35-0.4	1
9	0.4-0.45	0
10	0.45-0.5	1

Kakrapar

Sr.No.	Range	No. of Events
1	0.01-0.05	51
2	0.05-0.1	21
3	0.1-0.15	14
4	0.15-0.2	2
5	0.2-0.25	1
6	0.25-0.3	1

TABLE-3

PARAMETERS OF THE TWO DEGRE OF FREEDOM SYSTEM

m_1 (kg)	k_1 (kg/m)	R_m	ζ_1		ζ_2	
			SSE	OBE	SSE	OBE
1.24E+07	7.83E+09	0.05	0.07	0.04	0.02	0.01

TABLE-4

PEAK VALUES OF DISPLACEMENT OF THE MASSES AT DIFFERENT VALUES OF DAMPING

R_f	GMD1				GMD2			
	SSE Damping		OBE Damping		SSE Damping		OBE Damping	
	ZM(1)	ZM(2)	ZM(1)	ZM(2)	ZM(1)	ZM(2)	ZM(1)	ZM(2)
0.1	0.003	0.0602	0.004	0.0715	0.004	0.0282	0.005	0.0305
0.25	0.003	0.0364	0.003	0.0382	0.004	0.0157	0.005	0.0214
0.50	0.003	0.0282	0.003	0.0355	0.004	0.0153	0.005	0.0170
1.00	0.003	0.0119	0.004	0.0147	0.004	0.0171	0.004	0.0211

TABLE-5

RATIO OF PEAK VALUES OF DISPLACEMENT OF THE MASSES AT THE TWO VALUES OF DAMPING FOR SSE AND OBE

Sr. No.	R_f	GMD1		GMD2	
		$R_{1,1}$	$R_{1,2}$	$R_{1,1}$	$R_{1,2}$
1	0.1	1.33	1.19	1.25	1.08
2	0.25	~ 1	1.05	1.25	1.36
3	0.50	~ 1	1.26	1.25	1.11
4	1.00	1.33	1.23	~ 1	1.23

List of Symbols

a, b	Parameters in earthquake occurrence model
GMD_n	Ground Motion time-history for the nth record
k, m	Stiffness and mass of a degree of freedom
M, N	Magnitude of earthquake, No. of earthquakes
R_h	Hypocentral Distance
R_m, R_f	Frequency Ratio, Mass Ratio
$R_{1,j}$	Ratio of the peak displacement of the jth degree of freedom for OBE damping to the peak displacement of the jth degree of freedom for SSE damping with the same base excitation
S	Spectral Acceleration
S_m	Allowable Stress Intensity
T	Time Period (for Response Spectrum)
T_y	Return Period for Earthquake
ZM(j)	Peak Relative Displacement of the jth dof
ζ	Damping Ratio

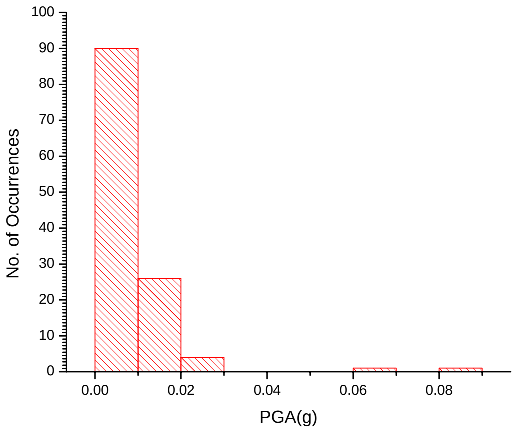

Fig.1a: Histogram of Calculated PGA for Tarapur Site: Global Data

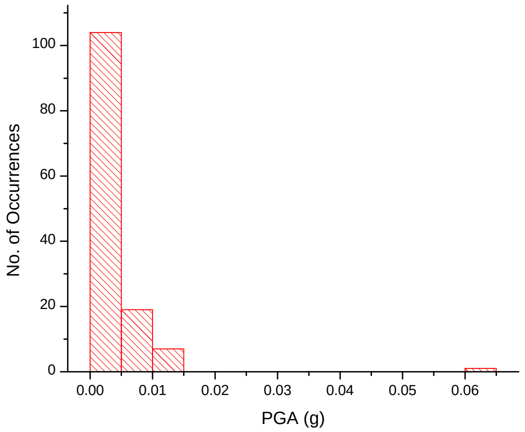

Fig.1b: Histogram of Calculated PGA for Tarapur Site: GBA Data

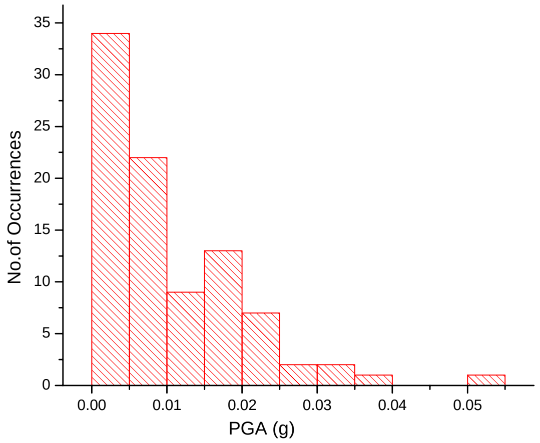

Fig.1c: Histogram of Calculated PGA for Kakrapar Site: Global Data

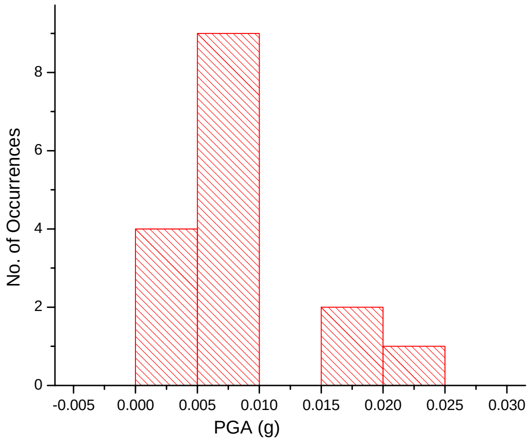

Fig.1d: Histogram of Calculated PGA for Kakrapar Site: GBA Data

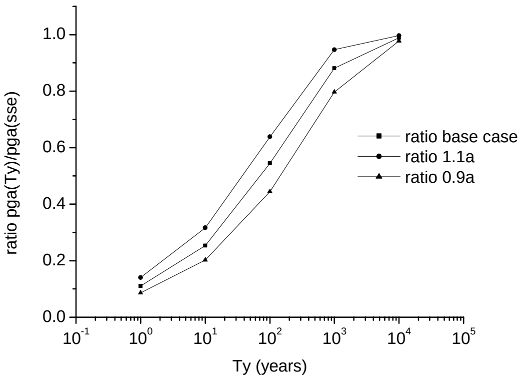

Fig.2a: Variation of Ratio (R) with Ty for Tarapur site: sensitivity to a

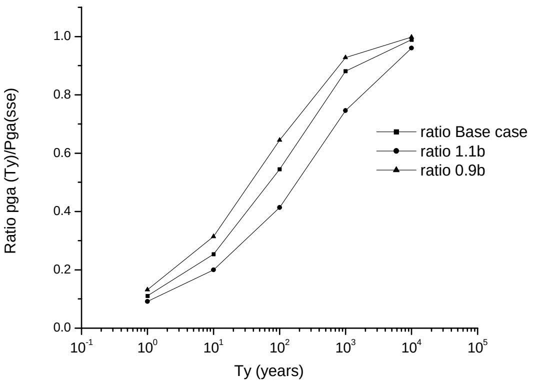

Fig.2b: Variation of Ratio (R) with Ty for Tarapur site: sensitivity to b

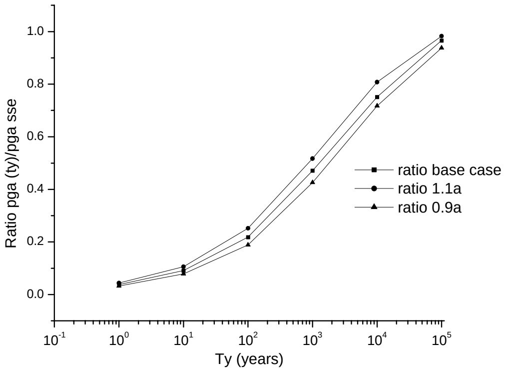

Fig.2c: Variation of Ratio (R) with Ty for Kakrapar site: sensitivity to a

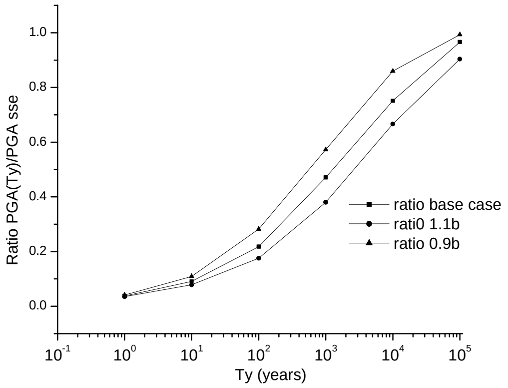

Fig.2d: Variation of Ratio (R) with Ty for Tarapur site: sensitivity to b