regions, a long wait is required to build a meaningful database. Till such time, there is bound to be controversy in the choice of earthquake parameters. Evaluation of earthquake potential of a site should be independent of the structure to be built. The choice of design earthquake parameters should be based on engineering experience gained worldwide.

1. *Earthquake-Resistant Regulation, A World List*, International Association of Earthquake Engineering, Tokyo, 1992.
2. *Seismic and Geologic Siting Criteria for Nuclear Power Plant*, 10 CFR Part 100, USNRC, US Atomic Energy Commission, Washington, DC, 1973.
3. *Design Response Spectra for Seismic Design of Nuclear Power Plants*, Regulatory Guide 1.60, USNRC, US Atomic Energy Commission, Washington, DC, 1973
4. *Earthquake and Associated Topics in Relation to Nuclear Power Plant Siting*, Safety Series No. 50–SG-S1, IAEA, Vienna, 1979.
5. *Seismic Analysis and Testing of Nuclear Power Plant*, Safety Series No. 50–SG-S2, IAEA, Vienna, 1979
6. *Seismic Studies and Design Basis Ground Motion For Nuclear Power Plant Sites*, Guide No. AERB/SG/S-11, Atomic Energy Regulatory Board, India, 1990.
7. Chandrasekaran, A. R., Sixth World Conference on Earthquake Engineering, New Delhi, 1977.
8. Kennedy, R. P. and Short, S. A., SMIRT, K4/1 9, 1984.
9. International Commission on Large Dams Bulletin 72, 1989.
10. *Draft Recommendations for Determination of Seismic Parameters*, Central Water Commission, India, 1990
11. IS 1893–1984, *Indian Standard Criteria for Earthquake-Resistant Design of Structures*, Indian Standard Institution, New Delhi, 1986
12. Chandrasekaran, A. R. and Das, J., 'Strong Motion Arrays in India and Characteristics of Recent Recorded Events', ISET Annual Lecture, 1988, Bulletin, ISET, 1990, vol 27, no. 1.
13. Chandrasekaran, A. R. and Das, J., *Curr. Sci.*, 1990, **62**, 233–250.
14. Chandrasekaran A. R. and Das, J., Fourth International Conference in Seismic Zonation, Stanford, USA, 1991.
15. Campbell, K. W., *Earthquake Spectra*, 1985, vol. 1.

Specifying aseismic design inputs for critical structures

R. D. Sharma

Seismotectonics Group, Nuclear Power Corporation of India Limited, Vikram Sarabhai Bhavan, Anushaktinagar, Bombay 400 094, India

It is possible to design engineering structures, which would withstand the impact of future earthquakes, if this impact is specified in terms of the vibratory ground motion, which the structures are expected to experience during future earthquakes. Such specifications of the vibratory ground motions form the basis of aseismic design. While the procedures for specifying aseismic design inputs for conventional structures which came into existence earlier, were based on an approach of minimizing the losses by preventing collapse, more elaborate procedures are now adopted for aseismic design of structures of critical facilities, e.g. dams and nuclear power plants, where the acceptable limits of damage are much lower. The approach to specifying aseismic design inputs for such structures is discussed in some detail with a view to identifying issues, that need to be addressed from the standpoint of adequacy of design.

State-of-the-art techniques in engineering design have made it possible to design engineering structures to withstand earthquakes. The Bureau of Indian Standards has specified criteria for designing structures to withstand ground vibrations during earthquakes (IS-1893)[1]. Here, the design inputs are specified in terms of a seismic coefficient (or zone factor) and a set of response spectra. The seismic coefficient at any place is equivalent to the maximum peak ground acceleration (PGA), which can be expected on the basis of the maximum earthquake intensity at that place from past earthquakes. The prediction of intensities assumes that earthquakes will follow the observed patterns. Some recent experiences have, however, shown that occurrence of earthquakes stronger than those occurred in a region during historical times, cannot be precluded. The IS-1893 specifications aim at safety of the engineered structures as long as ground motion remains within these levels. If these levels are exceeded, the structures may be damaged, but will not collapse, thereby saving lives and property. Protecting the structures against moderate earthquake intensities and limiting the damage to acceptable limits during the most severe event is, thus, the intent of the IS-1893. It is believed that beyond the levels of these specifications it would be more economical to repair the structures, or even reconstruct them. This works out well for most structures, except for critical ones like dams, nuclear power plants and lifelines, where safety requirements are more stringent. The approaches to aseismic design of such structures were developed during the past twenty five years, particularly in the context of nuclear power plants[2,3]. Salient features of these approaches are discussed here.

Levels of severity in aseismic design

The aseismic design criteria of nuclear power plants have been so chosen that damage to any component, equipment, system or structure, if it occurs, does not lead to any unacceptable consequences (i.e. release of radioactivity), and interruptions in power generation on account of earthquakes are limited to a minimum. This is achieved by using two levels of severity of earthquake ground motion for design. These two levels are based on the 'maximum probable' and 'maximum possible' events inferred from geological, geophysical and seismological information from the region. These two levels constitute the earthquake design basis. The less severe event (the maximum probable earthquake) corresponds to the Operating Basis Earthquake (OBE) in the USNRC terminology or to the S1 level earthquake ground motion in the IAEA terminology. The more severe event (the maximum possible earthquake) corresponds to the Safe Shutdown Earthquake (SSE), or the S2 level ground motion. The OBE (S1 level event) may affect the structures just about once during their useful life. The SSE (S2 level event) is the maximum possible event for the known seismotectonic set up of the region. During operation of the power plant the ground motion at the plant site is continuously monitored. In case the ground motion exceeds the S1 level motion, the power plant is shut down and inspected in accordance with the laid down procedures. Only after certificate of fitness for operation is issued, the plant resumes operation. The S2 level motion determines the ultimate safety of the structures against earthquakes. After occurrence of an S2 level earthquake the power plant is expected to remain in a safe shutdown condition, even if the damage goes beyond repair.

Aseismic design inputs

Among the various effects of earthquakes such as vibratory ground motion, surface faulting and ground failure phenomena, only the vibratory effects can be effectively mitigated through design. If the extent of maximum surface faulting and its direction are known, it may also be possible to mitigate its effects for certain special type of structures. However, sites with potential of surface faulting are not considered acceptable for nuclear power plants.

The basic inputs to aseismic design are the maximum amplitudes of ground motion (displacement, velocity and acceleration) at different frequencies of vibrations, which the engineering structures could be subjected to during future earthquakes. It is, generally, agreed that acceleration is a measure of the force exerted on rigid (high frequency) structures like those in nuclear power plants. At lower frequencies the earthquake forces can be expressed in terms of amplification factors over the PGA – called Dynamic Amplification Factors (DAF), which are different for different natural frequencies. A plot of the DAF versus natural frequency (or period) is the response spectrum. The maximum responses of a set of single degree-of-freedom systems (SDOF) of different frequencies to the earthquake ground motion, is the response spectrum. The response spectrum normalized to PGA of 1.0 g, is the response spectral shape.

Apart from the maximum response of the structures (characterized by the PGA and the response spectrum) during an earthquake, a ground motion time history is used for investigating the temporal behaviour of the structures under earthquake-induced vibrations. No real accelerogram will contain all the characteristics of the design basis earthquake, which is represented by the specified design response spectrum (SDRS), because of the maximization of several parameters. This necessitates the use of a spectrum compatible accelerogram (SCA). The SCA is also used to determine the response spectra at various floor levels. Thus the basic inputs to aseismic design are:

(a) PGA i.e., Peak ground acceleration, at the site corresponding to the design basis earthquake,
(b) A response spectral shape, and
(c) A spectrum compatible acceleration time history.

These three inputs quantify the vibratory effects of the design basis earthquake at the site.

Design inputs for nuclear power plants

Earthquakes result from release of strain energy accumulated along geological faults, which have moved during geological times. The earthquake history of a region and state-of-the-art techniques in field geology, geophysics and seismology can be used to locate faults capable of generating earthquakes, and to estimate their magnitudes and frequencies of occurrence. The OBE and SSE are specified in terms of the magnitude of the earthquake and its location parameters (latitude, longitude and depth of focus).

Design basis earthquakes

The documents 10-CFR-100:APPENDIX-A and the IAEA guide 50-SG-S1 list out the procedures of investigating the region around a proposed nuclear power plant site for evaluating the seismotectonic set up, estimating the maximum earthquake potential and fixing the earthquake design basis. In a geologically mapable and investigatable region the basic entity to be evaluated is a fault, a geological structure having the potential of releasing strain energy. Such a fault has been referred to as capable fault (or a seismogenic fault). From the viewpoint of investigations, a capable fault has been defined as one which:

(i) shows evidence of a recurring nature at or near the surface within about one and a half million years before present, such that the possibility of further movements can be inferred, and/or
(ii) has demonstrated structural relationship to a known capable fault such that movement on one may cause movements on the other at or near the surface.

In an area where the geological conditions do not permit detailed mapping, and seismological data are sparse, the basic unit for evaluation is a tectonic province: a contiguous region characterized by a relative consistency of geologic, structural and seismological features. On the basis of the available geological information and earthquake data, the maximum earthquake potential of each geological fault (or tectonic province), in which the site is located (and the adjoining tectonic provinces), is to be estimated. For such evaluation an area of 300 km radius around the site is considered. Epicentres and magnitudes of past earthquakes in the area are superimposed on lineaments/faults plotted on a geological map (1 : 1,000,000). Using this map distinct tectonic provinces are delineated. Association between faults and earthquakes is studied. Earthquakes, which cannot be associated with any known fault are identified. Satellite imageries, aerial photographs and field observations are studied to locate new faults, with which these earthquakes could be associated. Any earthquake which cannot be associated with any known fault has been called a 'floating earthquake'. The maximum earthquake potential, specified in terms of magnitude, which can be associated with each fault (or tectonic province) is estimated using the available information, and is assumed to occur on the fault (or within the tectonic province) at the point which is nearest to the site. The floating earthquake potential of the tectonic province, in which the site lies, is assumed to occur at the nearest point on the closest geological fault. If any reservoir exists (or is likely to come up in future) in the area, the possibility of a reservoir-induced earthquake is also to be included. For each earthquake source the maximum earthquake magnitude, location and depth of the earthquake source is determined. The design basis events for the S2 level motion (SSE) are then fixed according to their definitions stated above.

Design basis vibratory ground motions

Estimating the ground motions at a site from the magnitude(s) and location(s) of the design basis earthquake(s) – the SSE and the OBE – is the next step in specifying aseismic design inputs. A large amount of recorded data are now available for estimating the values of the ground motion parameters at a site from magnitude and locations of earthquakes. Empirical relations between PGA, magnitude and source to site distance under different geological conditions at the site are available for several regions of the world[4, 5]. A well accepted empirical relationship is that of McGuire[6], according to which the PGA is given by:

$$a = \{\, 0.0306 \exp(0.89\,M)/R_{\mathrm{H}}^{1.17} \} \times \exp(-0.2\beta) \tag{1}$$

$$\sigma(\log_e a) = 0.62\,, \tag{2}$$

where M is the magnitude of the earthquake on Richter scale. R_{H} is the hypocentral distance and β accounts for site geology – 0 for rock and 1 for soil sites. σ quantifies the dispersion of the data assuming a lognormal distribution for PGA. For a given source site combination, a more suitable empirical relation if available may be chosen.

Response spectral shape

In the earlier days of aseismic design, standard response spectral shapes based on a limited number of accelerograms, from western United States, were used[7, 8]. With increase in the number of accelerograms recorded under different site conditions it was found that soil conditions affect the spectral shapes considerably[9–12]. Studies demonstrated that, on an average, for frequencies below 2–3 Hz the spectral amplifications for rock sites were substantially lower than those for soft sites. For higher frequencies the spectral amplifications for rock sites were found to be higher than those for soft site conditions. These observations favoured the use of site-dependent spectral shapes.

In an ideal situation, the site-dependent spectral shape should be obtained from an accelerogram of an earthquake having magnitude and source location close to that of the design basis earthquake, recorded in close proximity to the site under consideration. In practice, this is rarely possible. As an alternative, a number of accelerograms recorded under similar site conditions and having the desired ground motion characteristics are selected[13], and their spectral shapes are combined and smoothened to arrive at design spectral shapes. Methods based on the recorded data and analytical modelling of earthquake sources, signal transmission path and site characteristics have also been developed[14, 15]. Application of these methods, which call for detailed information in each of these areas, does not always make this approach more acceptable because of the uncertainties in the underlying assumptions.

In the estimation of spectral shapes two issues are important. The first relates to the selection of the database (accelerograms) and the other to the methods used for combining and smoothening the shapes. If a very large data set is used, it does not remain homogeneous, and adds some features to the spectral shapes which are not relevant to design in the particular case. Selection of the dataset on the basis of some preferences for a specific situation restricts the database. On account of these contradictions, it is necessary that the constraints on the spectral shapes are laid down explicitly, and are used to guide the entire exercise including the review. Site-dependent spectral shapes for use where the earthquake magnitude is between 6 and 6.5 and the epicentral distance is around 25 kilometers, were derived by Nuclear Power Corporation (NPC) for rock and soil sites[16]. These shapes are shown in Figure 1.

Smoothening of the spectral shapes is carried out using averaging and interpolation methods. For a conservative shape, the dominant peaks in the spectral shape are identified, the ordinates at the intermediate frequencies are interpolated so as to give a smooth response spectral shape. Such spectral shapes tend to be over conservative. Other issues, such as multiple peaks in the spectral shape, peak broadening and flattening need to be examined in some detail. If the spectral shape has been so derived that it does not contain too many peaks and valleys, and conservatism has been ensured by enveloping the ordinates of the unsmoothened shape, these issues become unimportant. Observations have shown that the peak of the spectral shapes for rock sites are sharper than those for soil sites. For an acceleration time history ad/v^2 (a = PGA, v = peak velocity, d = peak displacement) is considered as a measure of the sharpness of the spectral shape, and its value has been found to be considerably lower for accelerograms from soil sites, compared to those from rock sites. The requirement of flatness in the spectral shape at a certain frequency should be determined only from the smoothening requirements, which arise from the uncertainty in the frequencies under consideration.

Spectrum compatible accelerograms

For generating spectrum compatible accelerograms, either an existing accelerogram is modified to include the requirements laid down by the design basis earthquake or the ground motion is modelled as a stationary Gaussian random process having characteristics dictated by the design response spectrum. Generation of artificial accelerograms and criteria for their acceptance in design also come under critical examination during a review.

Whereas PGA is a measure of the force exerted by the earthquake vibrations on high frequency structures, and is used for normalizing the accelerogram as well as the response spectral shape, acceptability of the accelerogram for use in design calls for additional factors, most important of these being the frequency content of the accelerogram. The amplitude and frequency distribution in the ground motion time history should also have the characteristic values of the ground velocity, displacement and duration of strong motion. The most commonly used method of generating artificial accelerograms is to synthesize the ground motion as a combination of sinusoids with random amplitudes and phase angles, so that the acceleration at any time t is given by:

$$a(t) = \Sigma A_n \sin(\omega_{nt} + \Phi_n), \tag{3}$$

where A_n, ω_n and Φ_n are the amplitudes, the angular frequencies and the phase angles, respectively. The amplitudes A_n are controlled by the power spectral density (PSD) function, $G(\omega)$, defined as:

$$\int_0^{\omega_{max}} G(\omega)\, d\omega = \sum_n A_n^2/2. \tag{4}$$

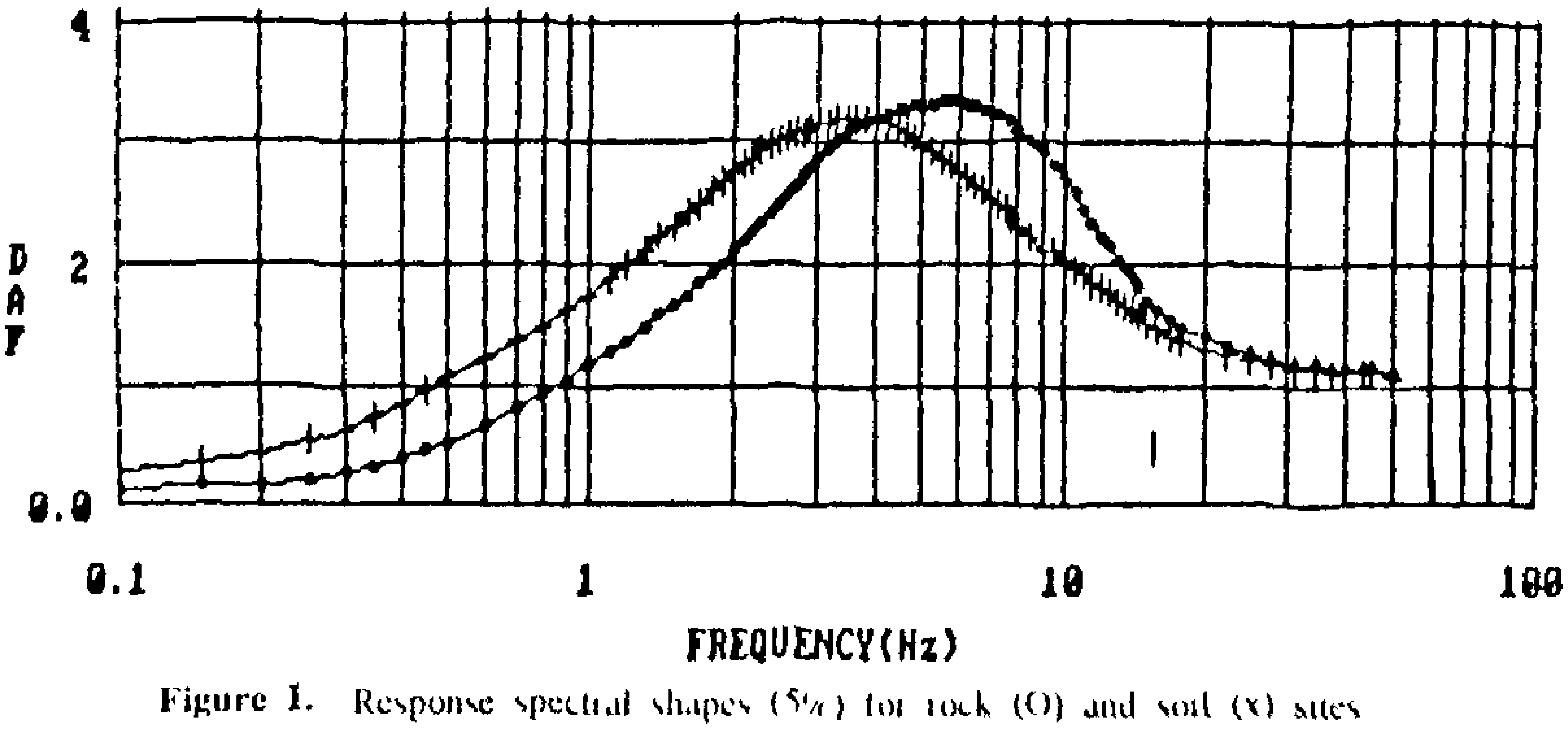

Figure 1. Response spectral shapes (5%) for rock (O) and soil (x) sites

This formulation, based on the Fourier series representation of the time series, allows maximization of the velocity response by adjusting the number and values of A_n, and provides unique values of $G(\omega)$ when the accelerogram is subjected to Fourier analysis. An artificial accelerogram compatible with the rock site spectrum of *Figure 1 is shown in Figure 2a, and the matching* of its spectrum is shown in Figure 2*b*. The time history has the following characteristics:

Maximum peak acceleration	:	1 0 g
Maximum peak velocity	:	111 cm/sec
Maximum peak displacement	:	67 cm
Zero crossing rate	:	10/sec
ad/v^2	:	5 5.

These are consistent with those observed for rock sites. The corresponding velocity and displacement time histories are shown in Figures 2*c* and *d*, respectively, the power spectral density is shown in Figure 2*e*.

An alternative normalization parameter

The PSD is related to the Fourier transform, $F(\omega)$, by the relation:

$$G(\omega) = (1/\pi T) \cdot [F(\omega)]^2 \tag{5}$$

where T is the duration of the time series in seconds. The total power of the accelerogram is related to the Fourier transform as

$$(1/T) \int_0^T a^2(t)\, \mathrm{d}t = (1/\pi T) \int_0^{\omega_n} F^2(\omega)\, \mathrm{d}\omega\,. \tag{6}$$

The left hand side is the mean square acceleration in the record and the right hand side the area under the PSD curve. The PSD at a frequency ω_n may, thus, be represented by the product of the mean square acceleration in the record and the ordinate of a normalized PSD curve with unit area. It is, therefore, the mean square acceleration in the accelerogram, which influences the PSD, rather than the peak value. The root mean square (RMS) acceleration can, therefore, serve as a suitable parameter to normalize accelerograms. If the accelerogram has long rise and decay times, a conservative estimate of the mean square acceleration, and hence the PSD, will be obtained when the most intense

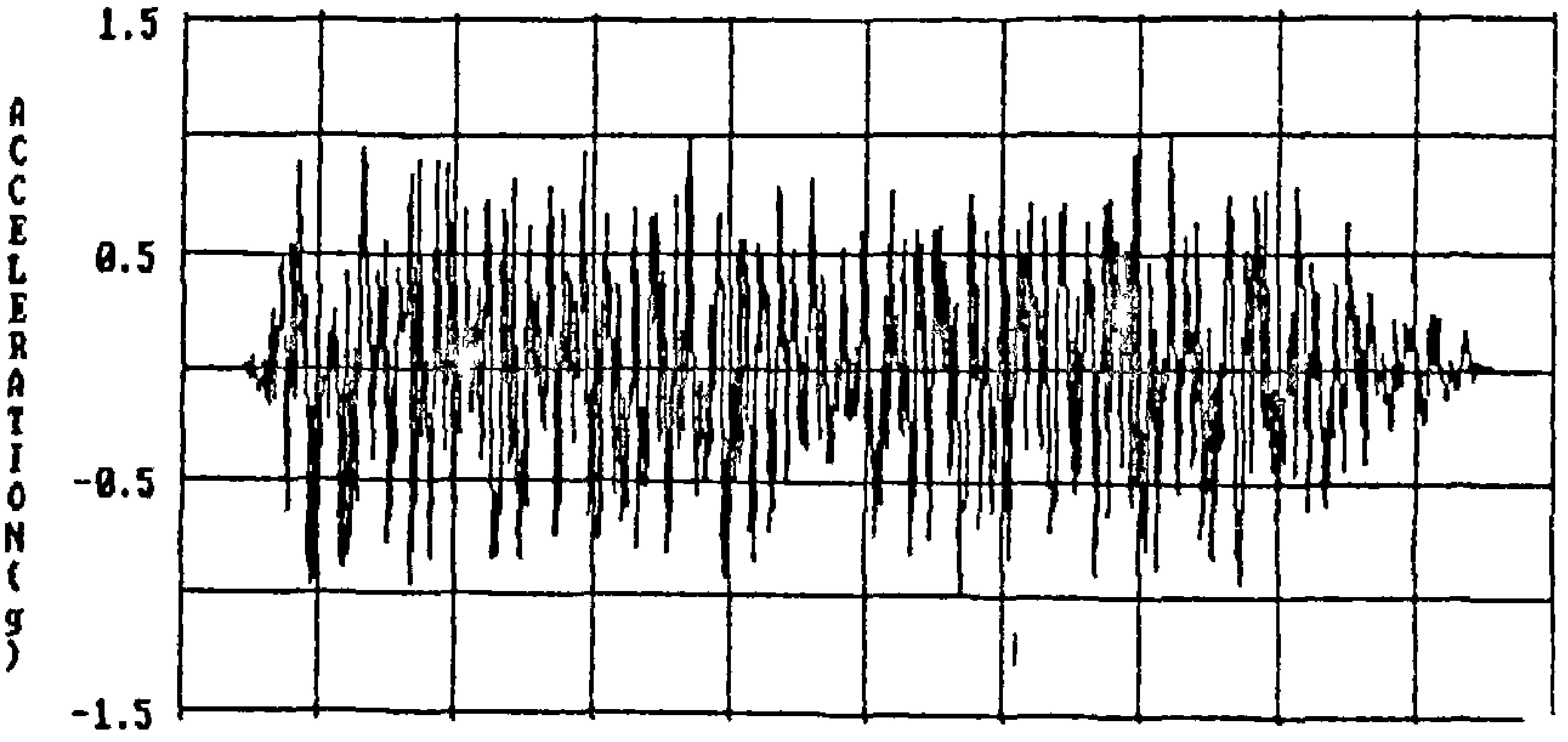

Figure 2*a*. Spectrum compatible acceleration time history – rock sites

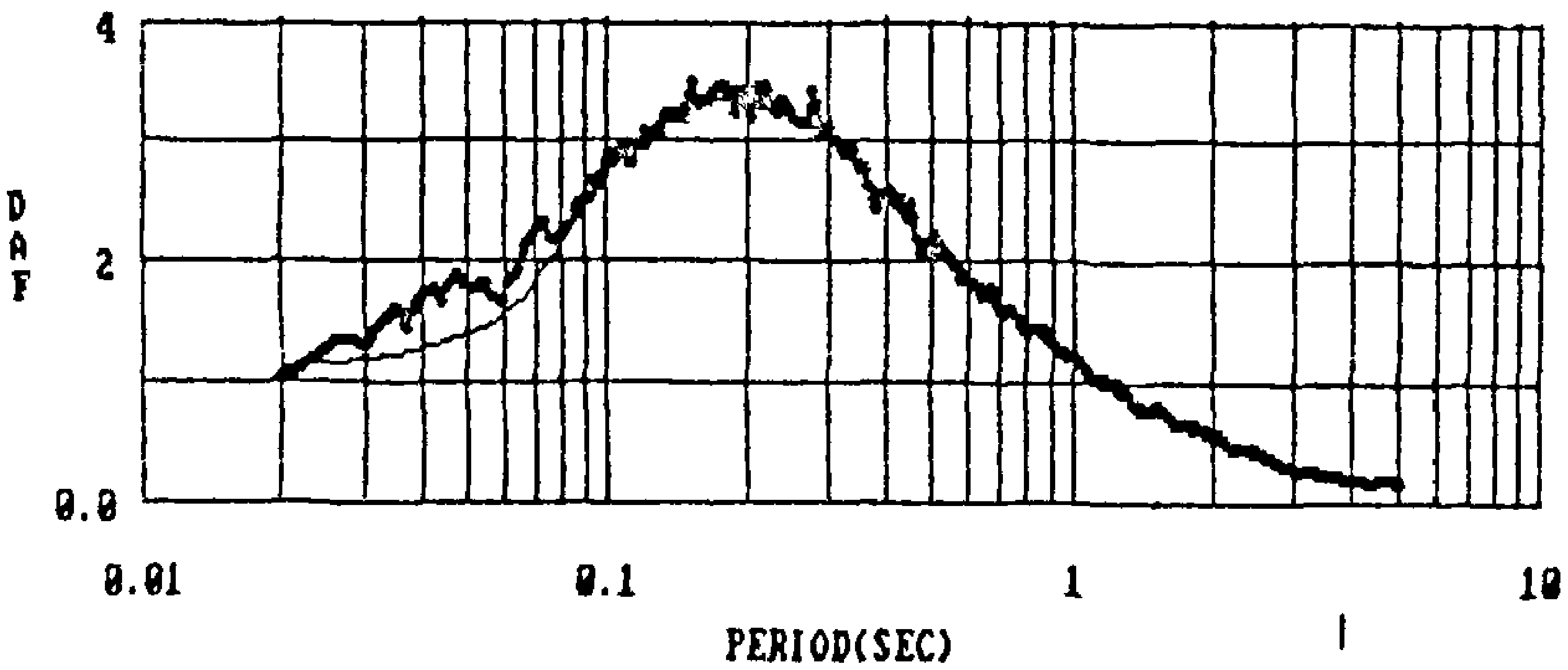

Figure 2*b*. Comparison of IHRS and SDRS – rock sites (5% damping).

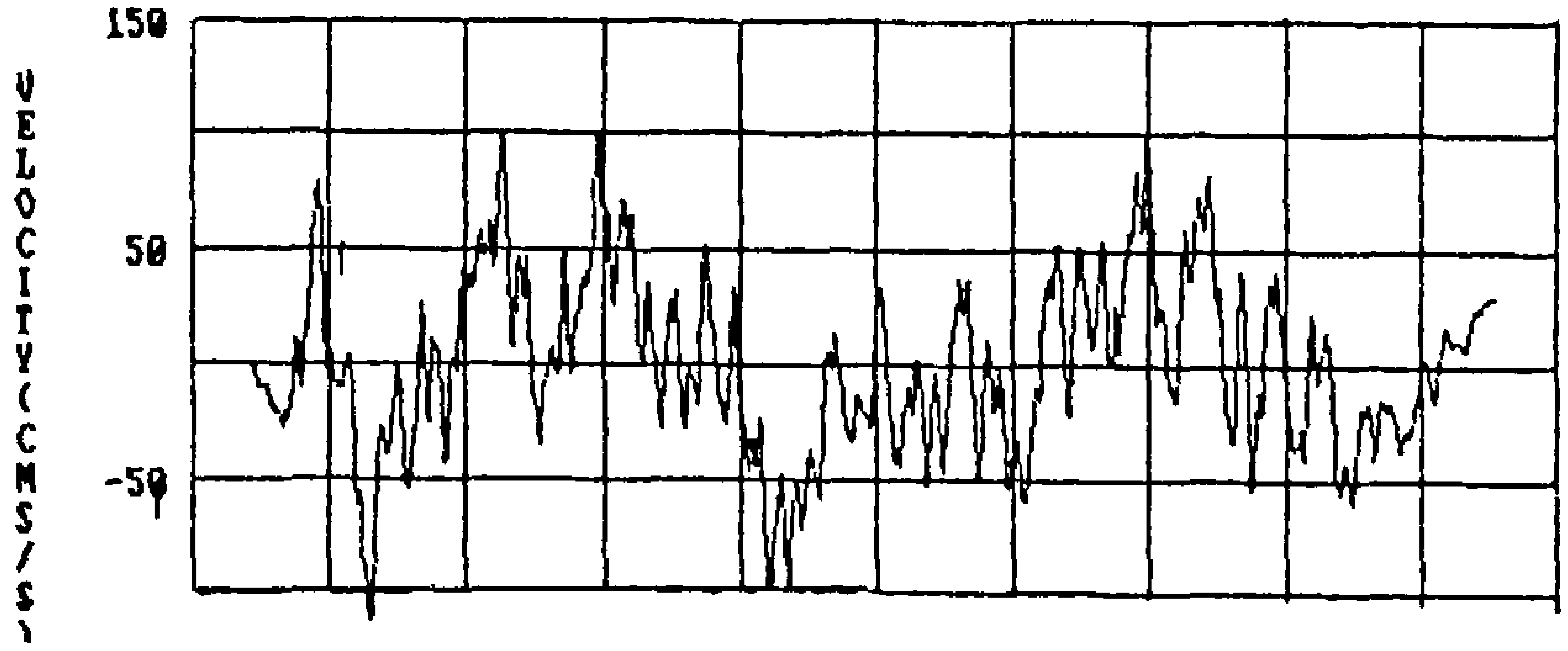

Figure 2*c*. Velocity time history corresponding to Figure 2*a*

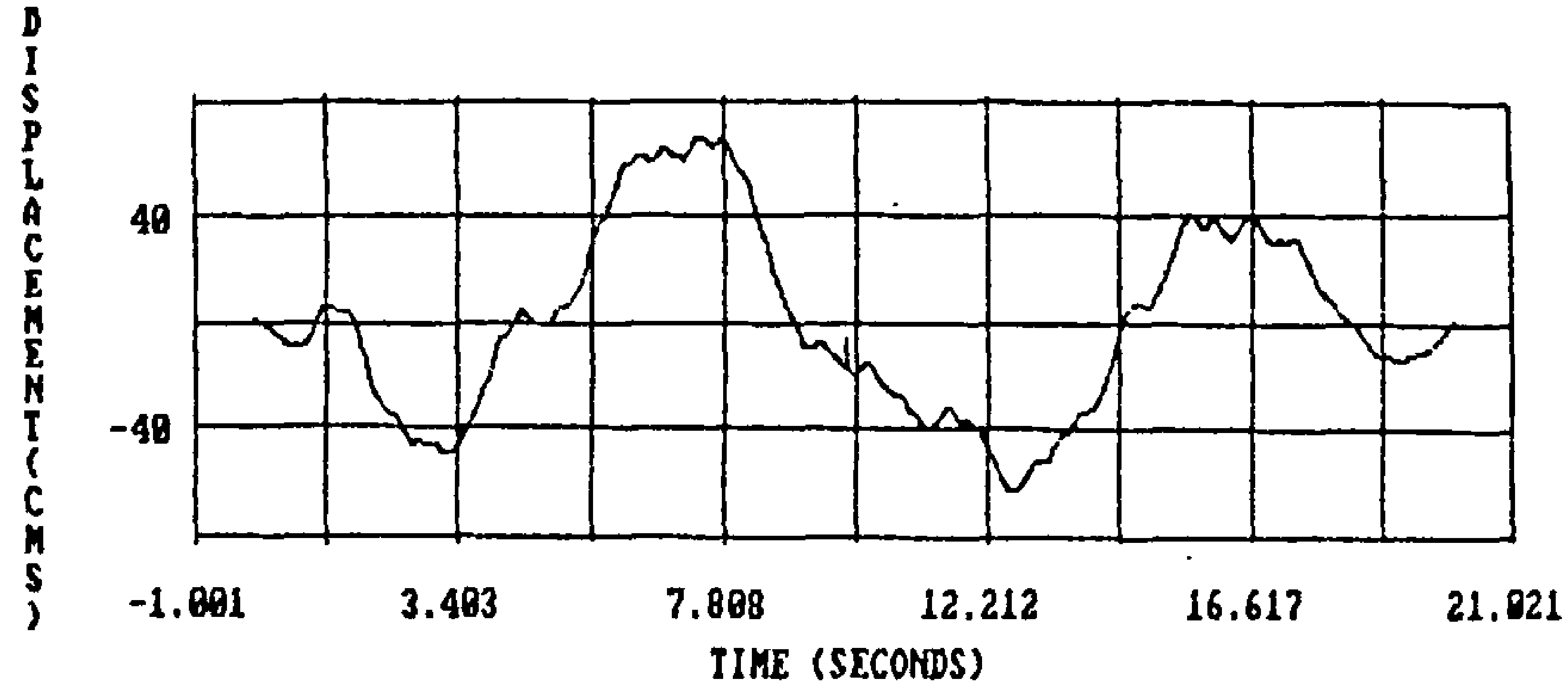

Figure 2*d*. Displacement time history corresponding to Figure 2*a*

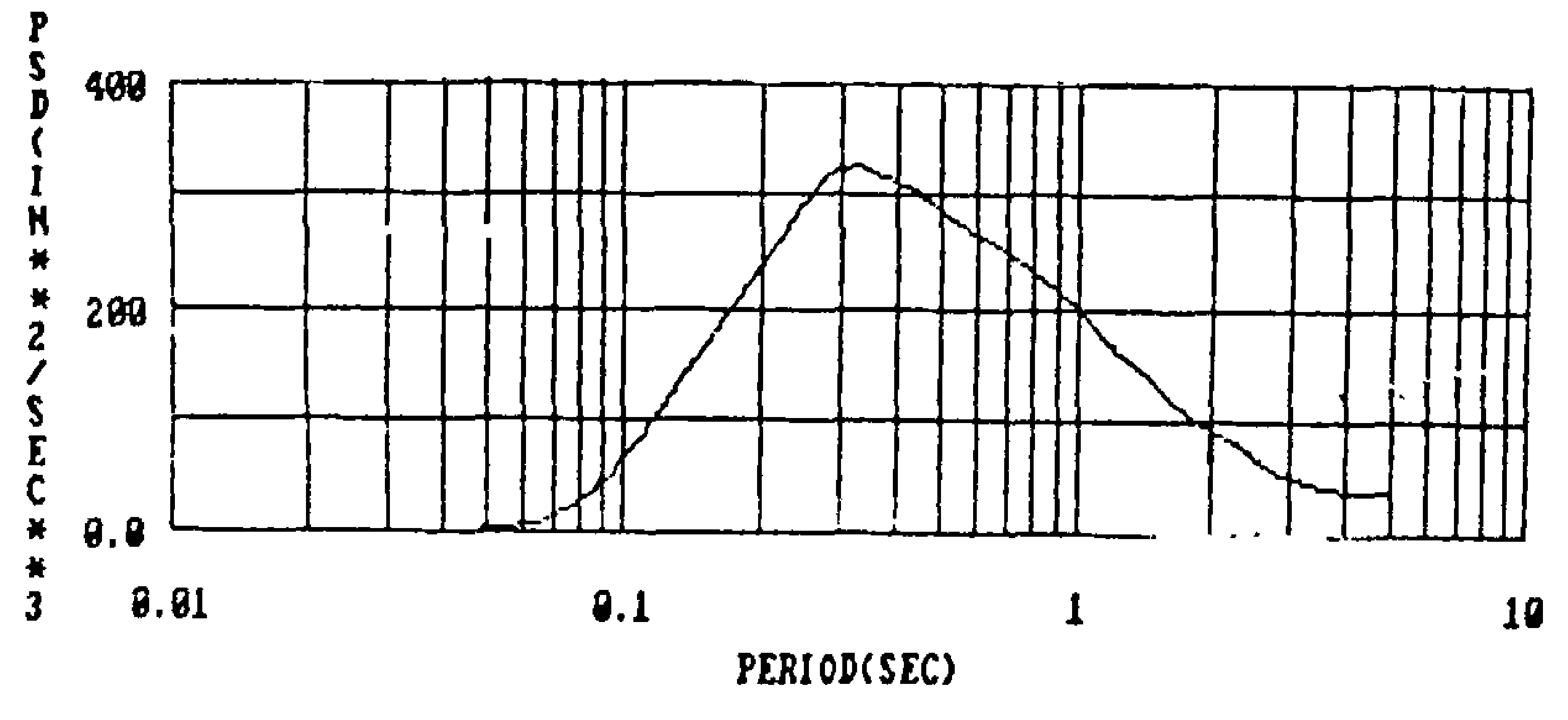

Figure 2*e*. Power spectral density of the ground motion

part of the vibrations is included in the computations. PSD of ground motion can also be estimated from response spectra. The relationship, however, is not exact because of its dependence on factors like exceedance probability, damping, signal duration, and sampling frequencies[17]. A plot of RMS acceleration values against PGA for the accelerograms used in the derivation of response spectral shapes shown in Figure 1 is shown in Figure 3. The correlation between the RMS acceleration and PGA is apparent. Hence the practice of constraining the PGA while generating synthetic accelerograms based on the PSD is consistent with the requirements of aseismic design, and so is the use of PGA as a normalization parameter for accelerograms.

Issues in aseismic design

The issues, which are to be resolved in fixing the aseismic design at a project site are:

(a) Where will the future earthquakes occur?
(b) What will be their magnitudes and frequencies?, and
(c) How much is the maximum ground motion experienced by different structures at the site as a result of these earthquakes?

The probability, $P(a)$, of the PGA exceeding a specified value during the life of the engineering structures (the operating life of the nuclear power plant in our case), may be written as:

$$P(a) = \int_{M=M_0}^{M_{\max}} \int_{S=a}^{a_{\max}} P_1(M) \cdot P_2(M,S) \cdot \mathrm{d}M \cdot dS, \qquad (7)$$

where $P_1(M)$ is the probability of an earthquake of magnitude M occurring in the site region, and $P_2(M, S)$ is the probability that the PGA at the site due to an earthquake of magnitude M will exceed a value S, M_0 is a limit on the magnitude, below which earthquakes are of no engineering consequence, $M_{\max}$ is the upper limit on the earthquake magnitude and $a_{\max}$ is that on the PGA. For easy comprehension the probability, $P(a)$, may be considered in terms of the return period of the event T, which causes the specified value of PGA to be exceeded, given by:

$$T = -L/\log_e [1 - P(a)], \qquad (8)$$

where L is the life time of the structures in years. The $S2$ level motion is expected to have a return period of not less than 10,000 years. (It may be noted here that in the SSE formulation the interpretation of the event in terms of probability is not encouraged.) For the S1 level motion an event with a return period of over a few hundred years is considered acceptable. Evaluation of the design basis PGA, thus, requires the following:

(i) Knowledge of all geological faults in the area: their locations with respect to the site, dimensions, and history of tectonic movements to estimate the magnitude of the strongest earthquake, which could be generated by each fault.
(ii) A magnitude frequency relationship for earthquakes on each fault.
(iii) An analytical relationship to estimate the values of PGA at the site due to earthquakes of given magnitude and location.

As there are no unique and unambiguous answers to any of these questions some decisions have to be made. Whether such decisions would lead to conservative designs is the issue which is decided during the review, which is based on a critical examination of the quality and quantity of the collected information and the applicability of the evaluation procedures. The PGA for each postulated event is estimated using an empirical relation. It has been agreed that the S1 level motion may be either estimated from probabilistic considerations, or fixed at half of that for the S2 level.

Use of PGA as a design parameter

Though use of PGA as a design parameter has been widely accepted, and hardly any substitute is yet found more agreeable, it is, sometimes, argued that:

(a) PGA is only one point in the accelerogram (and the response spectrum), and hence is not fit for use as a design parameter,
(b) Observed values of PGA, when plotted against earthquake magnitude or source to site distance, and for different source mechanisms show large scatter,
(c) Any other ground motion parameter could be used as a normalizing parameter instead of the PGA.

In view of these objections more controversial parameters have been put forward in spite of comparatively lower levels of acceptance. Use of effective peak ground acceleration (EPGA) is one such example. This situation rose on account of accelerograms of some events, in which recorded PGA values were much higher than normally observed. In India, the Koyna earthquake of 10 December 1967 was one such event. For one of the horizontal component ground motion of this earthquake the PGA was 0.63 *g*. Some explanations were attempted to explain the excessively large value of PGA. It was argued that being an epicentral shock PGA was not highly correlated with earthquake magnitude. In almost every review the question why the Koyna accelerograms have not been used in arriving at the design specifications, is to be faced. Figure 4*a* and *b* show the response spectral shapes for the two orthogonal horizontal components of ground motion for this earthquake. The rock site response spectral shape of Figure 1 has also been superimposed in this figure. It is apparent that the Koyna response spectral shapes are not characteristic of those for rock sites, and are not suitable for use in design. Why the Koyna accelerograms have such characteristics is outside the scope of this paper. Here, it is adequate to state that the accelerograms of this earthquake are not to be included in the aseismic design database on account of the inadequacy in the spectral shapes.

It is desirable to appreciate the context in which PGA is referred to. Fixing the aseismic design inputs, namely the PGA and response spectral shapes, requires a characteristic data set. The very basis of the empirical relationships between PGA and earthquake magnitude and focal parameters is the existence of a correlation. There are instances where the observed values do not strongly follow this correlation. But the deviations cannot be used to undermine the utility of PGA. To say that PGA is only one point in the accelerogram is also an out of the context truth. It is true that damage to structures cannot be explained in terms of PGA alone. But aseismic designs are not based on PGA alone, even when PGA is used as a design parameter. Use of response spectra and a spectrum compatible time history implies that the frequency content of the ground motion vibrations has been specified in terms of a power spectral density (PSD), which is dependent on the mean square acceleration, and not just on the PGA. This issue has already been discussed above. It is therefore clear that PGA, though is not always an index of observed damage, is still suitable for use as a design parameter.

Applicability of the specifications

A complete specification of the aseismic design inputs for protection against vibratory ground motions consists of: (i) Maximum amplitudes of ground acceleration, velocity and displacement, (ii) a response spectral shape, and (iii) a spectrum compatible time history.

The efficacy of the specifications lies in their adequacy in producing a design, which would provide protection against future earthquakes without being over conservative. This depends a great deal on the information available for generating the aseismic design inputs. Whereas, the quality and quantity of this information varies from one region to another, there is no way to prove the applicability of the design specifications, except through the use of some accepted analytical procedures. Acceptability of the aseismic design inputs is as important as the design itself. In absence of acceptability, the design may not take off and if it does, it will always be under the clouds of controversies and fear. It is therefore desirable that the factors determining the adequacy of the design inputs and their acceptability are analysed carefully.

Dependence on seismological data

Specification of the maximum values of ground motion parameters at a construction site is based on the magnitude of the maximum credible earthquake (MCE) and its location with respect to the site. Hence, the accuracy of the specified ground motion parameters, PGA for example, depends on that of the MCE. Estimates of the MCE are dependent on the available seismological magnitude and location data, and the geological and tectonic information on history of fault movements in the region.

Information on occurrence of earthquakes in different parts of the country is available for a period of two to three hundred years. This information has been used in the preparation of the seismic zoning map of India, in which every location on the map has been assigned a maximum earthquake intensity[1]. Though, the data on past earthquakes are not considered adequate to accurately estimate the magnitudes of the MCE in different parts of the country, it is possible to identify a given area as of high, moderate or low seismicity. An important point, which should be noted here, concerns the comparison of the seismic hazard at two locations on the seismic zoning map. Between the two locations in areas of the same zone number, the one whose neighboring zone is associated with a higher MM intensity implies higher earthquake hazard. For low seismicity areas, there are no records of earthquakes in historical times. If no earthquakes have been recorded in these areas during the instrumental period the aseismic design problem is not difficult to handle. Operation of a microearthquake network for a period of six to twelve months and examination of satellite imageries and aerial photographs to rule out the existence of seismogenic faults in the area will provide adequate assurance against a severe impending earthquake hazard. In nuclear power plant design, a PGA of 0.1 *g* has been recommended for such areas, where the estimated design PGA is smaller than this value. In moderately seismic areas, the most important issue to be decided on is the location of the design basis earthquake. In such areas, the information on faults and fault movements is not likely to be adequate to enable estimation of the MCE on the basis of existing databases. The magnitude of the earthquake will, therefore, be based on that of the strongest earthquake in the past. For nuclear power plants in such areas, the MCE is fixed by increasing the magnitude (or epicentral intensity) of the strongest historical earthquake by an equivalent of one unit of MM intensity for the epicentral region (0.7 unit of Richter magnitude). This magnitude is assumed to occur at a certain distance, say *R* km, from the site, which is decided on the basis of investigations, so that there is no active geological fault in the site area within this distance. A reasonable source depth, which is necessary for an earthquake of the specified magnitude to occur, is also assumed. Examination of the magnitudes of past earthquakes assumes considerable significance in this case. In the case of highly seismic areas, fixing the magnitude and location of the design basis earthquake may be comparatively more straightforward, because of better availability of isoseismal maps, and locations of epicentres. In a geologically well-mapped area, the magnitude and location of the MCE could be fixed on the basis of the available data. In areas where good tectonic maps are not available, the tectonic province approach will have to be used. In such areas, however, selection of the empirical relationship to convert the earthquake magnitude and hypocentral distance into values of ground motion parameters will be more critical. This is because, the site may lie in the nearfield of an earthquake, where PGA and earthquake magnitude do not show a good correlation, and local factors, at the site and at the earthquake source, become more important. Apart from detailed and critical examination of the available earthquake data, design specifications in such areas require that the margins available for alternative interpretations are established, before the specifications are finally accepted.

Local issues

The first and foremost issue to be addressed in specifying the inputs for aseismic design relates to identification of faults with seismogenic potential. Geological mapping in the country is within the purview of the Geological Survey of India (GSI). A large part of the country has been mapped at 1 : 50,000 scale but there is a time delay between the field data collection and publication of maps. On our requests GSI may provide unpublished maps. Inclusion of these maps enlarges the geological database, but the investigations and debates on important geological features of these maps, which should have been completed by this time, actually begin at this stage. The required investigations being time consuming, expensive and resource intensive often get replaced by 'engineering judgements' which are debated over endlessly. The only alternative is to augment the resources at GSI on the one hand and encouraging geological research within the country on the other.

The methodologies of specifying aseismic design inputs developed in different parts of the world can be transported only to a limited extent. Basing the aseismic design on PGA and a response spectral shape, for example, has become almost a universally used practice. Selection of the empirical relationship to convert the magnitude and location of the design basis earthquake (SSE or OBE) into PGA, the response spectral shape and acceleration time history may, to a certain extent, be handled on the basis of the global databases. However, the basic inputs to all these are the magnitude, location of the MCE, and the local geological conditions. In a typical case of an earthquake occurring at a depth of 20 kilometers and epicentral distance of 20 kilometres from a site, the PGA may change from $0.2\,g$ to $0.3\,g$ if its magnitude is taken as 7.0 instead of 6.5 (using equation (1)). The magnitude of the Koyna earthquake of 10 December 1967 was initially placed at 7.5 by the IMD. Subsequently, this figure was revised to 6.5, which is frequently quoted in literature. In most earthquake engineering applications, the magnitudes of the earthquakes in the post instrumental period are taken from the bulletins of the International Seismological Centre (ISC), or from the Earthquake Data Reports (EDR) published by the USGS. The magnitude of the Koyna earthquake is given as 6.0 in the EDR, and as 5.9 in the ISC bulletin. If the magnitude of the MCE in this region is based on that of the Koyna earthquake, it is necessary to distinguish between the figures based on data from different sources. Another example, which came to our notice concerns an earthquake which had occurred in 1684 near Surat. Based on the catalog of Oldham, seismologists had assigned an epicentral MM intensity of IV to this earthquake. Recently the magnitude of this earthquake has appeared as 6.5 in the catalogue of the Indian Society of Earthquake Technology (ISET). This revision could not be substantiated on the basis of the enquiries, which we made from the IMD, and several seismologists in the country. Taken on the face value, this revision has the potential of changing the aseismic design scenario in this region completely.

Examination of satellite imageries and aerial photographs, and even the field checks for ground truth verification are not sufficient to decide, with definiteness, on the seismotectonic status of a lineament, which is crucial for fixing the earthquake design basis for a particular site. In such cases a microearthquake survey of the area has the potential for providing additional information. It is believed that one section, or the other, of an active fault would generate microearthquakes. This requires establishing a network of telemetered microearthquake stations in the site area within a short time, operated for several months and locating earthquakes with epicentral accuracies of as much as a kilometer before arriving at any decision on such lineaments[18]. Because of the unmanageable cable lengths required in the telemetry of the data, wireless telemetry becomes a necessity. However, the licensing procedures for wireless data transmission are so elaborate and time consuming that realization of a microearthquake network within the available time becomes almost impossible. Examination of aerial photographs which are available in the depository of the Survey of India requires defence clearances and these can be seen only by those one or two individuals for whom defence clearances have been obtained. The rules are very stringent in this regard. For example, even taking notes of the observations on the photographs falls outside the rules of consulting aerial photographs. It is absolutely necessary that the requirements of earthquake hazard mitigation in general, and aseismic design in particular, are recognized at the national level, and steps are taken to simplify rules and regulations to enable speedy data collection.

Immediate priorities

An overview of the problem of specifying aseismic design inputs reveals that insight into the various aspects of the problem, is very important. To arrive at the specifications for a project site, and to approve these as adequate (or reject them as inadequate) are two equally important steps in solving the problem. It is, generally, observed that at both stages the inadequacy of the database is emphasized, but sometimes out of proportion. In certain situations lack of data itself is information. Irrespective of the gaps between the available and the desired information, some decisions have to be taken before arriving at the specifications for aseismic design. The situation during review is, somewhat, different. The reviewer finds it difficult to decide either way, because if he disapproves the specifications outright, the roles get reversed. He is called upon to justify his decision. He also does not like to approve, because this may put him out of the enviable position by aligning with the applicant and against that section of specialists who do not still accept it. Prolonged debates are the result of the present scenario, where information generation for sorting out outstanding issues is difficult and time consuming, if not actually impossible. The following steps may help in correcting the situation.

Preparation of acceptable earthquake catalogues

Examination of the existing earthquake data by a body of experts on regional basis, and publication of a catalogue of earthquakes in India and its neighbourhood is needed. Examination of these data can be handled in a phased manner. For example first all major and moderately sized earthquakes (magnitude 6 and above) may be taken up. Support from different national agencies and institutions will have to be enlisted by a coordinating central agency.

Installation of seismological stations

Installing a sufficient number of seismographs in the country to achieve a detection capability for earthquakes of magnitude 3 for moderately and highly seismic areas, and magnitude 2 in less seismic areas is suggested. Only when seismographs have been operating for sufficiently long intervals in an area, and proof exists that no earthquakes were recorded, the absence of earthquakes in a region becomes information for design.

Publication of detailed geological maps

Augmenting the resources of the Geological Survey of India to enable speedy publication of the detailed geological maps (1 : 25,000/1 : 50,000) is necessary.

Making data generation easier and effective

Revision of existing rules and procedures for grant of permits for examination of data, and generating new data is needed. In particular, it should be possible to obtain licences for wireless telemetry of microearthquake data within a short time interval of say, not more than two months. For temporary networks which may be required for short period surveys, such as those required in undertaking post earthquake surveys, clearance should be available within a few days.

Organized effort

Recognizing that generation of aseismic design inputs and their use in design is a multidisciplinary subject, taking up earthquake-related studies and aseismic design in an organized manner by institutions with governmental support on a continuing basis is essential. For handling the aseismic design of major projects, availability of data, data processing facilities and a body of specialists in these areas is highly desirable.

Conclusions

Identification of the seismogenic faults in the region around the construction site, estimating their maximum earthquake potential are the most important issues to be resolved while specifying the aseismic design inputs for a project. While certain issues, like choice of design parameters and data processing techniques, can be chosen according to international practices, availability of the data to achieve the desired standards is dependent on the national state of the art and data generation practices. Whenever the issues of magnitude and distance of the earthquake from the site are not resolved beyond doubt, the uncertainty gets compensated by overconservatism and licencing delays. It is therefore desirable that a national earthquake hazard mitigation policy is framed and rules in various areas are simplified to encourage data generation. Compared to the costs of aseismic design based on overconservative aseismic design inputs, the costs for data generation will be much less. If suitable steps to generate earthquake related information in a systematic manner are taken on a national level, the problem of aseismic design can be handled more effectively.

---

1. IS 1893, Criteria for Earthquake-Resistant Design of Structures (Fourth Revision), Indian Standards Institution, New Delhi, 1984
2 USNRC, Reactor Site Criteria, Rules and Regulations, Title 10 Chapter 1 Code of Federal Regulation Energy, Part 100—Appendix-A United States Nuclear Regulatory Commission, 1980
3. Earthquakes and Associated Topics in Relation to Nuclear Power Plant Siting—Safety Guide 50 SG S1, International Atomic Agency, Vienna, 1979

4 McGuire, R K, EQRISK a computer programme to evaluate risk to site. Open file report 76–67, 1976

5 Campbell, K W, *Earthquake Spectra*, 1985, **1**, No 4

6 McGuire, R K, *J Geotech. Engg Divn*, *ASCE*, 1978, **104**, 481–490

7 Housner, G W, *Proc ASCE*, 1959, **85**, October

8 Newmark, N M and Consulting Engineers, A Study of Vertical and Horizontal Earthquake Spectra, Report WASH-1255, Directorate of Licensing, United States Atomic Energy Commission, 1973

9 Hayashi, S, Tsuchida, H and Kurata, E., Average Response Spectra for Various Subsoil Conditions, Third Joint Meeting U S-Japan Panel on Wind and Seismic Effects, UJNR, Tokyo., 1971

10 Kuribayashi, E, Iwasaki, T, Ida, Y and Tuji, K, Proc. International Conf Microzonation, Seattle, WA, 1972, pp. 499–512.

11\. Seed, H B, Ugas, C. and Lysmer, J, *Bull. Seismol Soc. Am*, 1976, 66, 221–243

12 Mohraz, B, *Bull Seismol. Soc Am*, 1976, **46**, 915–933.

13\. Iyengar, R. N and Prodhan, K. C., *Earthquake Engg Struct Dyn.*, 1983, **11**, 415–426

14 Trifunac, M D., and Lee, V W., Frequency Dependent Attenuation of Strong Earthquake Ground Motion, Report No. CE 85-02, Department of Civil Engg, Univ Southern California, 1985.

15 Lee, V, W, Influence of Local Soil and Geologic Site Conditions on Pseudo Relative Velocity Spectrum Amplitudes of Recorded Strong Motion Acceleration, Report No CE 87-06, Department of Civil Engg, Univ South California, 1987

16 Kumar, S, Gupta, S and Sharma, R D, Site Dependent Response Spectra for Rock and Soil Sites 2nd International Conference on Vibration Problems of Mathematical Elasticity and Physics, A C College, Jalpaiguri, West Bengal, Nov 1–2, 1993.

17\. Gasparini, D A and Vanmarcke, E. H, Simulated Earthquake Motions Compatible with Prescribed Response Spectra, Publication No R76-4, MIT, Department of Civil Engineering, Cambridge, Massachusetts, 1976.

18 IAEA, *Application of Microearthquake Surveys in Nuclear Power Plant Siting*, TECDOC-343, 1985

ACKNOWLEDGEMENTS The correlation between peak ground acceleration and root mean square acceleration was made available by Sanjeev Kumar and B K Bansal Computer program for computing velocity and displacement time histories from the accelerogram was made available by A. G. Chhatre. Computer program SIMQKE of Gasparini and Vanmarcke obtained from NISEE was used for generating synthetic accelerograms Kamakshi Srinivas assisted in preparation of the manuscript

---

Field investigation of the 30 September 1993 earthquake in Maharashtra

R. N. Iyengar*, C. S. Manohar and O. R. Jaiswal

Department of Civil Engineering, Indian Institute of Science, Bangalore 560 012, India

*Present address Central Building Research Institute, Roorkee 247 667, India

An engineering field study of the 30 September Marathwada earthquake is reported. The study covered Osmanabad, Latur, Sholapur, Bijapur, Gulbarga and Bidar districts. The level of ground acceleration was estimated based on tilting of free standing objects. The study shows that the epicentral intensity has been VIII on the UNESCO scale. The horizontal ground acceleration near the epicentre has been estimated to be about 0.2 *g*.

THE 6.4 magnitude earthquake of 30 September 1993 with its epicentre in the Marathwada region of Maharashtra has been a reminder that peninsular India is also seismically active. This earthquake which was felt in the distant cities of Bangalore and Madras has resulted in complete destruction of several villages located in the epicentral tract.

From past data available for about 300 years it can only be said that earthquakes are more frequent in the Himalayan region than in other parts of India. Since the available historical data are scanty, other regions wherein seismic activity is less frequent, are not well delineated. This may be the reason for the popular opinion that peninsular India in general and Deccan plateau in particular, is aseismic. On the contrary, from well documented historical data it is known[1] that during the period 1340–1984, nearly 300 earthquakes have occurred in the peninsular region (8° N–25° N latitude). Of these, 70 events have been of magnitude 5 and above. Still considering the vastness of peninsular land mass, for any given subregion, the average recurrence interval of a strong earthquake is perhaps of the order of centuries. This is in contrast to the frequent occurrence of earthquakes in the northeastern parts of India, which in turn, has influenced construction practices in rural areas of Assam and neighbouring states. This observation explains to some extent the large number of house collapses and subsequent loss of life in the Marathwada region.

Objectives

The field study was undertaken (i) to understand the extent and severity of structural damage from an