RISK CRITERIA AS A RATIONAL BASIS FOR SEISMIC RESISTANCE OF STRUCTURES OF DIFFERENT GRADES[1]

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Introduction

Economic growth of a country involves continuous rapid development in the various sectors of the construction industry. The housing sector creates particular problems in developing countries, as the demand for mass housing is very great, the majority who are in need of this housing are the low-income people and their budgets are very limited. To meet the demand for low-cost mass housing in the shortest possible time, it may be a matter of government policy to implement housing schemes in which various grades of housing are considered based on their durability, thus on their building cost. The useful lifetime of those buildings may vary from as little as 10 years to perhaps 30 years, depending on the type and grade of building material used. When these buildings are located in seismic areas, they should be seismic resistant. The problem, therefore, is how to determine the seismic design loads for these buildings, because the normal seismic loading stipulated in building codes is not intended for the design of buildings of different grades. A rational basis is to provide the building appropriate strength and stiffness, so that no matter what its lifetime (whether short or long) it will have a uniform risk with respect to onset of structural damage and with respect to incipient failure. This paper will discuss these two different risks and how to implement them in design.

Risk Criteria

Seismic activities are natural events which recur from time to time. Like a few other natural phenomena such as river floods, seismic events are stochastic processes which can be represented by a mathematical model of a given physical system that changes in accordance with the laws of probability. Therefore, by knowing the seismic history of a region obtained from observations or records made over a sufficiently long period of time in the past, the return periods of the various seismic intensities in that region can be assessed by applying the laws of mathematical statistics. It is obvious that low seismic intensities will reappear more frequently than high ones.

In connection with the recurring nature of seismic intensities, it is only logical that it is not economic to design a structure to remain undamaged when subjected to a seismic intensity with a return period much longer than the useful lifetime of the structure. A rational basis for seismic resistant design of structures is therefore a two stage process, the objective of which is first, to provide the structure sufficient strength and stiffness to resist moderate earthquakes so that the risk of occurrence of unrepairable structural damage in its useful lifetime is acceptably low, and second, to ensure that the risk of collapse leading to loss of human lives in its useful lifetime in a severe earthquake is as well acceptably low. A moderate earthquake can be defined as one which has a high probability of occurrence in the useful lifetime of the structure, while a severe earthquake is one having a low probability of occurrence. The first of these objectives can be achieved by setting the seismic design load at an appropriate level, the second objective can be achieved by providing the structure proper detailing so as to ensure proper ductile behavior in the post-elastic range.

Suppose that for a certain seismic region the mean return period of the various seismic intensitites have been assessed properly, based on the available seismic historic data. Mean return period and annual risk are reciprocally related, so that

$$R_A = \frac{1}{T} \qquad (1)$$

where R_A is the annual risk and T is the mean return period of the considered intensity. The annual risk can thus be defined as the probability that, in any given year, that intensity will be equaled or exceeded. Risks in certain time periods corresponding to various lifetimes of structures can be derived from the annual risks. Assuming that risks in successive years are independent, the following relationship applies:

$$R_N = 1 - (1 - R_A)^N \qquad (2)$$

where R_N is the risk in a time period of N years. Thus, within the context of this paper risk is a mathematically defined quantity, which implies the probability of occurrence of an undesirable event in a certain period of time.

A philosophical, economical, and political question to be answered now is how much risk is allowable for the occurrence of unrepairable structural damage in the useful lifetime of a structure, and how much risk is tolerable for the occurrence of structural collapse causing loss of human lives in the same period of time. One may argue about the various factors governing the selection of the above risk levels, but the author is of the opinion that for the occurrence of unrepairable structural damage, the probability must not be more than once in the useful lifetime. This means that the return period of the onset of structural damage must at least be equal to the useful lifetime itself, and based on equations (1) and (2) this implies a risk of about 60%. With regard to incipient collapse of the structure in its useful lifetime, the risk must certainly be smaller, and in this connection the author is of the opinion that it is not unreasonable to accept 20% for this risk. Based on equations (1) and (2) the return periods of seismic intensities associated with 60% risk for the onset of structural damage and 20% risk for incipient failure in any seismic region are shown on Table 1 for various lifetimes of structures.

Table 1

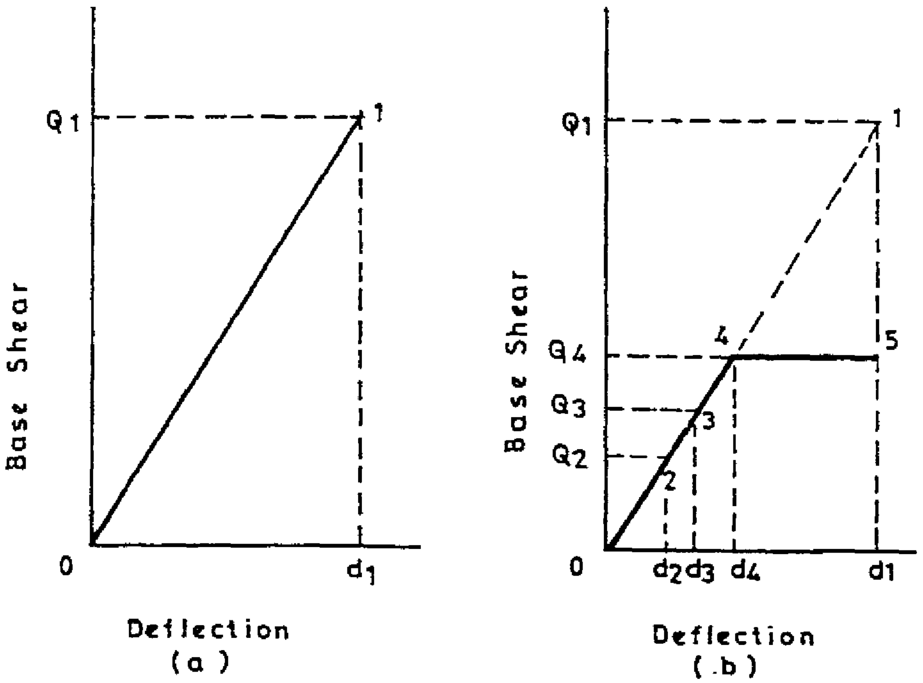
Return Periods of Seismic Intensities to be Considered in Structural Design

Structural lifetime (year)	Return period of seismic intensities (year)	
	60% risk for onset of structural damage	20% risk for incipient failure
10	10	40
15	15	60
20	20	80
25	25	100
30	30	120
40	40	160
50	50	200

Seismic Behavior of Structures

Load-Deflection Diagram

The seismic behavior of a structure can be represented by a load-deflection diagram, indicating the relationship between the seismic base shear acting on the structure and the corresponding deflection. If a structure is so strongly designed that it behaves fully elastic and thus remains undamaged up to the point of near collapse, its load-deflection diagram will be a straight line as shown in Figure 1(a). In this figure Q_1 is the base shear acting on the structure at the state of near collapse, and d_1 is the corresponding maximum deflection. As mentioned previously it is not economically feasible to design a structure that way, and a rational basis for design is to set the seismic design load at an appropriate level, so as to ensure that the structure will not be damaged in small to moderate earthquakes, and reliance is further placed on the structure performing in a ductile manner in a severe but infrequent earthquake, dissipating the earthquake energy and limiting the base shear that acts on the structure. The load-deflection diagram of such a structure can be represented by a bilinear flat top diagram as shown in Figure 1(b). At the state of near collapse the maximum deflection d_1 is approximately the same as for the full elastic system, however the base shear induced in the structure is limited to only Q_4. Points 2, 3, and 4 are successive points on the diagram corresponding to the base shear Q_2 considered in design, base shear Q_3 at first yield, and base shear Q_4 at the onset of structural damage, at which significant yielding in the structure starts to develop. Up to point 4 the structure behaves practically elastic, so that deflections d_2, d_3, and d_4 are directly proportional to their corresponding base shear.



$$f_1 = \frac{d_1}{d_4} = \frac{Q_1}{Q_4}$$

$$f_2 = \frac{d_4}{d_2} = \frac{Q_4}{Q_2}$$

Figure 1

Load-deflection Diagram
of Structures

These base shears are in turn directly proportional to the corresponding earthquake intensities. Beyond point 4 deflections by increased earthquake intensities will continue to occur at a constant base shear Q_4 until the maximum deflection d_1 is reached at a state of near collapse. The ratio $f_1 = d_1/d_4$ is a measure of the ductility of the structure. From the known value of this ratio the intensity of the earthquake that causes the state of near collapse of the ductile structure can be calculated by proportionality.

For a specific type of structure designed according to a specific code, points 1 up to 5 on the load-deflection diagram shown on Figure 1(b) are known, assessed analytically or experimentally, so that the earthquake intensity at the onset of structural damage and at the state of near collapse can be calculated by proportionality from the base shears Q_4 and Q_1. The seismic risk criteria then require that the risk for the occurrence of base shear Q_4 in the useful lifetime of the structure must not exceed 60%, and the risk for the occurrence of base shear Q_1 must not exceed 20%.

The ratio f_1 depends on the type of structure and the material used. From the study of all possible combinations of types of structure and material used, it had been found that a minimum value of $f_1 = 3.5$ could be expected to be present in a properly designed and constructed building structure, regardless of its type and the material used.

From the known value of $f_2 = d_4 / d_2$ the level of the design seismic base shear Q_2 can readily be calculated. This ratio depends on the over design of the structure due to code requirements, and on the type of structure in terms of capability to absorb increases in base shear and in lateral deflection beyond field yield, to cause plastic deformations in a significant number of structural elements. From the study of several possible combinations of recognized building codes and structural types in the determination of the strength and stiffness of a structure, it had been found that the average value of $f_2 = 3.0$ could be expected to be present in a structure, regardless of its type and the material used.

Response Spectra

The strict meaning of a response spectrum in earthquake engineering is a plot of the maximum elastic response to a specified earthquake excitation for all possible single degree-of-freedom systems. The abscissa of the spectrum is the undamped natural period of the system and the ordinate the maximum elastic response. Knowing the accelerogram of the specified earthquake excitation, spectra can be obtained by solving the differential equation of motion of a damped single degree-of-freedom system using the well-known Duhamel's integral. Of particular interest is the maximum acceleration response spectrum, because it is directly related to the maximum inertial forces induced in the system. It is obvious that for natural periods approaching zero, the maximum response acceleration is approaching the maximum ground acceleration.

It is apparent that in a certain seismic zone the response spectra of different earthquakes recorded in that zone are fairly similar to each other, so that it is possible to plot average response spectra for that zone. By further modification and simplification, those average response spectra can be transformed into several plots indicating the maximum base shear coefficients for all possible elastically responding normally damped building structures in that zone.

Each plot corresponds to a selected maximum ground acceleration expectable in that zone, or to a selected mean return period of ground acceleration in that zone. The maximum base shear coefficient is the ratio of the maximum base shear acting on the elastic structure to the total weight of the structure. Normally damped structures are those having fractions of critical damping in the order of 5%. The abscissa of such design response spectra is the undamped fundamental natural period of the structure and the ordinate the maximum base shear coefficient. For natural periods approaching zero, the maximum base shear coefficient approaches the maximum seismic coefficient, which is the maximum ground acceleration expressed in 'g' (= gravity acceleration). To take into account the influence of local subsoil conditions on the response spectra, such design response spectra may be plotted for various subsoil classifications, for example for hard and for soft subsoils. Design response spectra are sometimes included in seismic codes, so that they are suitable for use in seismic resistant design of structures based on risk criteria proposed in this paper. Examples of such design spectra typically defined for each of the seismic zones of Indonesia are shown on Figure 2, in which each spectrum is idealized as a trilinear diagram.

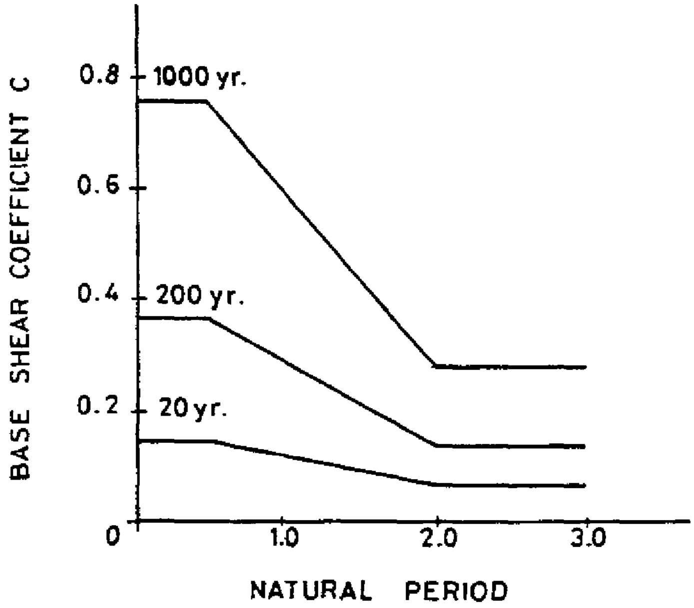


Figure 2

Typical Design Response Spectra for
a Seismic Zone in Indonesia

The application of design response spectra as shown on Figure 2 in seismic resistant design based on risk criteria will be explained with the following example. Suppose a structure with a useful lifetime of 30 years is to be designed. From Table 1 it can be seen that to meet the two levels of risk, the return period of seismic intensity producing onset of structural damage may not be less than 30 years, and that producing incipient collapse may not be less than 120 years. Calculate or estimate first the undamped fundamental natural period of the structure. Find from the design response spectra the base shear coefficient C_{30} corresponding to the seismic intensity having 30 years return period, if necessary applying proper interpolation. Estimate an appropriate value for the f_1 factor (e.g. $f_1 = 3.5$), so that the base shear coefficient at incipient failure in an elastic structure is $f_1 C_{30}$. Find from the design response spectra the return period of this base shear coefficient, if necessary again applying proper interpolation. If it is found that the return period of $f_1 C_{30}$ is at least equal to 120 years, the minimum required risk level for incipient collapse is satisfied. If it is not satisfied, find from the design response spectra first the base shear coefficient C_{120} in an elastic structure corresponding to the seismic intensity having a 120 year return period. The base shear coefficient at the onset of structural damage which automatically will satisfy the minimum risk level of 20% is then $C_{30} = C_{120} / f_1$. Having found the proper C_{30} value, estimate further an appropriate value for the f_2 factor (e.g. $f_2 = 3.0$), so that the design base shear coefficient $C_2 = C_{30} / f_2$ is readily available. From the base shear coefficient the design base shear can be calculated directly, and following standard code procedures the distribution of the design base shear to obtain the design horizontal seismic loads acting at each of the floor levels can be determined.

From the above discussion it is apparent that it is very important to establish proper seismic zoning, each zone characterized by the various return periods of seismic intensities expressed in the form of design response spectra.

Seismic Zoning

Determination of Return Periods of Seismic Intensities

As previously mentioned, the return period of seismic intensities can be assessed analytically from the historic seismic data by applying the laws of mathematical statistics. An empirical expression for the annual cumulative frequency of earthquakes in a source area having magnitude M equal or greater than a certain lower bound magnitude m_0 as found by Gutenberg and Richter, is of an exponential form as follows:

$$(3) \qquad N(M \geq m_0) = 10^{a' - b\, m_0}$$

where a' and b are constants statistically evaluated from the historic seismic data of the source area. Assuming earthquake magnitude M at the source and distance R from the source to the considered site are continuous random variables which influence intensity I of the site, and considering that sources may occur anywhere within a source area, the total probability that intensity i is equaled or exceeded at the site can be expressed by the following double integral:

$$(4) \qquad P(I \geq i) = \int_{r_o}^{r_1} \int_{m_o}^{m_1} P(I \geq i | m \text{ and } r|)\ f_M(m) f_R(r) dm\, dr$$

in which: $P\ I \geq i$ = the total probability that intensity i is equaled or exceeded at the site;

$P\ I \geq i | m \text{ and } r|$ = the conditional probability that intensity i is equaled or exceeded at the site given m and r;

$f_M(m)$ = density function on magnitude;

$f_R(r)$ = density function on source distance;

m_o = lower bound magnitude below which it is not of engineering importance;

m_1 = upper bound magnitude, which is the maximum magnitude that can be expected to occur in the source area;

r_o = distance to the nearest boundary of the source area;

r_1 = distance to the farthest boundary of the source area.

The above functions are all derived from earthquake statistics, except the density function on source distance, which is the spatial relation between the source and the considered site. It is beyond the scope of this paper to discuss these functions in more detail, therefore the reader is further referred to the related references listed at the end of this paper. It can merely be stated here, that the evaluation of the above double integral requires a very complicated mathematical calculation, for which the aid of computers is a requisite.

Having obtained the total probability that intensity i is equaled or exceeded at the site, the total annual number of that intensity that is being equaled or exceeded at the site is thus:

$$(5) \qquad N_A = N\ (M \geq m_o)\ \ P\ [I \geq i]$$

Assuming further that earthquake events occur as Poisson arrivals, the annual risk that intensity i is equaled or exceeded at the site can then be expressed:

$$(6) \qquad R_A = 1 - \exp\ (\ -\ N_A)$$

and the corresponding return period can be obtained from equation (1).

Determination of Seismic Zones

Applying the previously discussed theory, seismic intensities in the form of ground accelerations can be calculated at any site for which the return period is given. If for a given return period ground accelerations are calculated at a sufficient number of sites in a region, an isoseismal map of the region can be plotted, showing contours of equal ground acceleration. Such isoseismal maps can be prepared for various selected return periods. From those isoseismal maps it is apparent that for a certain return period expectable ground accelerations vary from site to site, which means that seismic risk is not uniformly distributed over the region. By selecting appropriate contour intervals, distinct zones of differing seismic risk can be identified. Those intervals are then the various seismic zones. It should be noted, that it does not matter very much, whether for the determination of seismic zones contour maps with short or with long return periods are considered. Whichever map is used to delineate the boundaries of the seismic zones, the relative risks of the zones remain approximately the same.

Determination of Design Response Spectra

In regions where practically no instrumental records of earthquake motions are available, so that no response spectra are known, it is not easy to derive average response spectra. Comparative studies in seismicity and local geology with other regions of known spectra are required to arrive at the most appropriate design response spectra for such regions.

The simplification of a design response spectrum into a trilinear diagram as shown in Figure 2 is just a matter of choice. Other simplifications may also be considered, for example a simplification into a continuous S-curve (as defined in the latest Japanese seismic code) or a hyperbola with a horizontal part for short periods (as defined in ATC-3). Because of the averaging of so many parameters, any such simplification seems to be equally justified.

Application in Indonesia

Isoseismal Maps

Return periods of earthquakes have been calculated for many sites throughout Indonesia, applying the previously discussed theory. Those sites were mostly the big cities, where important industrial, utility and other public buildings were being constructed, or locations of important projects which required strict seismic resistance such as dams, power stations, etc. In regions where the construction industry is in rapid development, isoseismal maps have been prepared. A typical example is shown in Figures 3, 4, and 5, which show isoseismal maps for the West Java region prepared by the author. The maps have been plotted for ground accelerations with return periods of successively 20 years, 100 years and 500 years.

Isoseismal maps for the whole Indonesian territory had been prepared by the Directorate of Hydraulic Engineering, Directorate of Water Resources Development, Ministry of Public Works, based on the same principles. Also other agencies have prepared similar maps, with slightly different approaches.

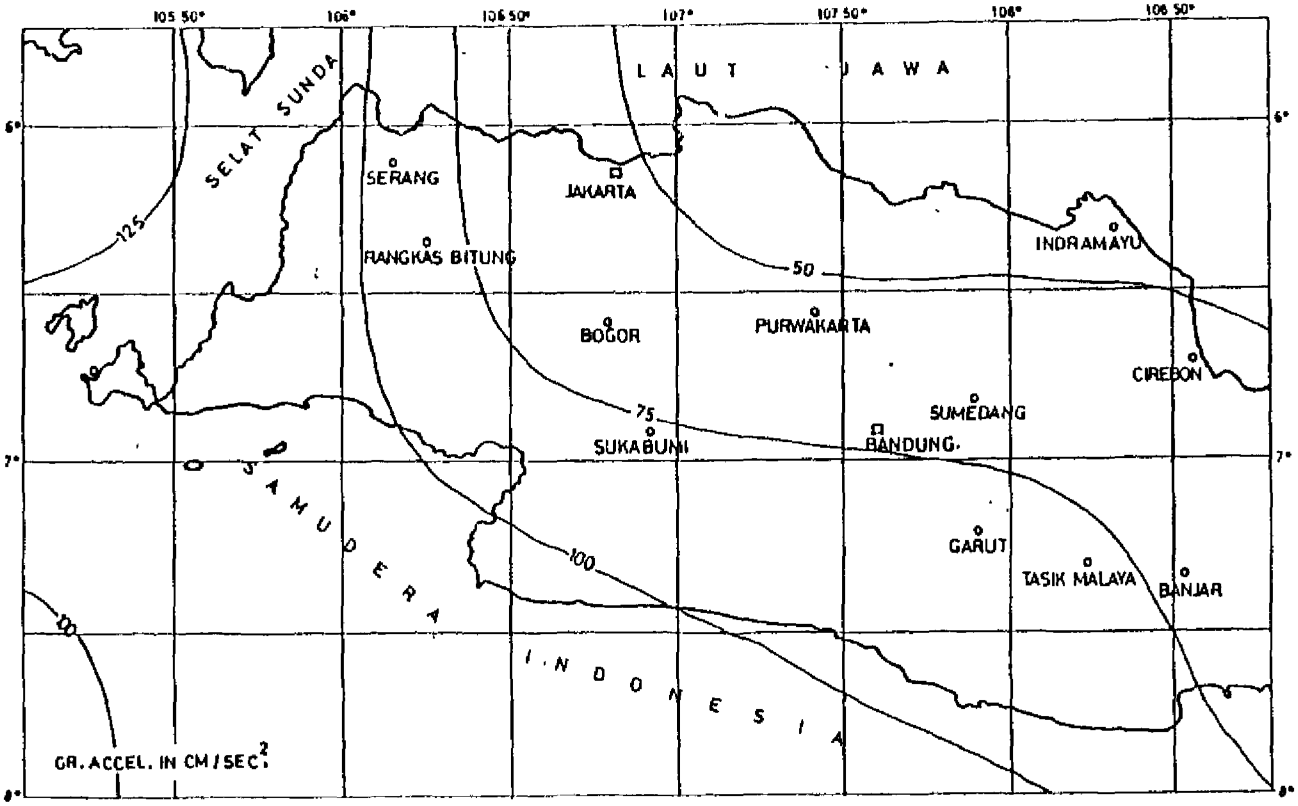


Figure 3

Isoseismal Map of West Java for Ground Acceleration Having 20 Year Return Period

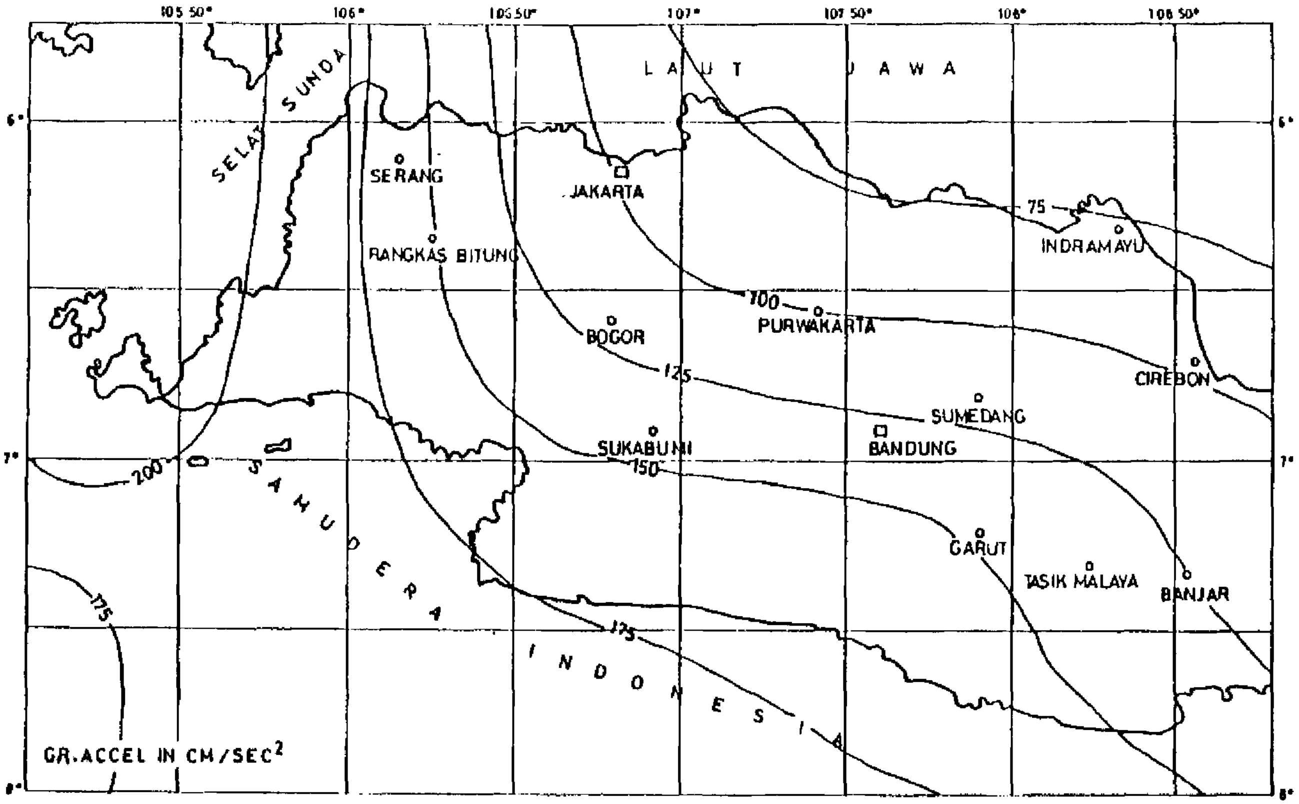


Figure 4

Isoseismal Map of West Java for Ground Acceleration Having 100 Year Return Period

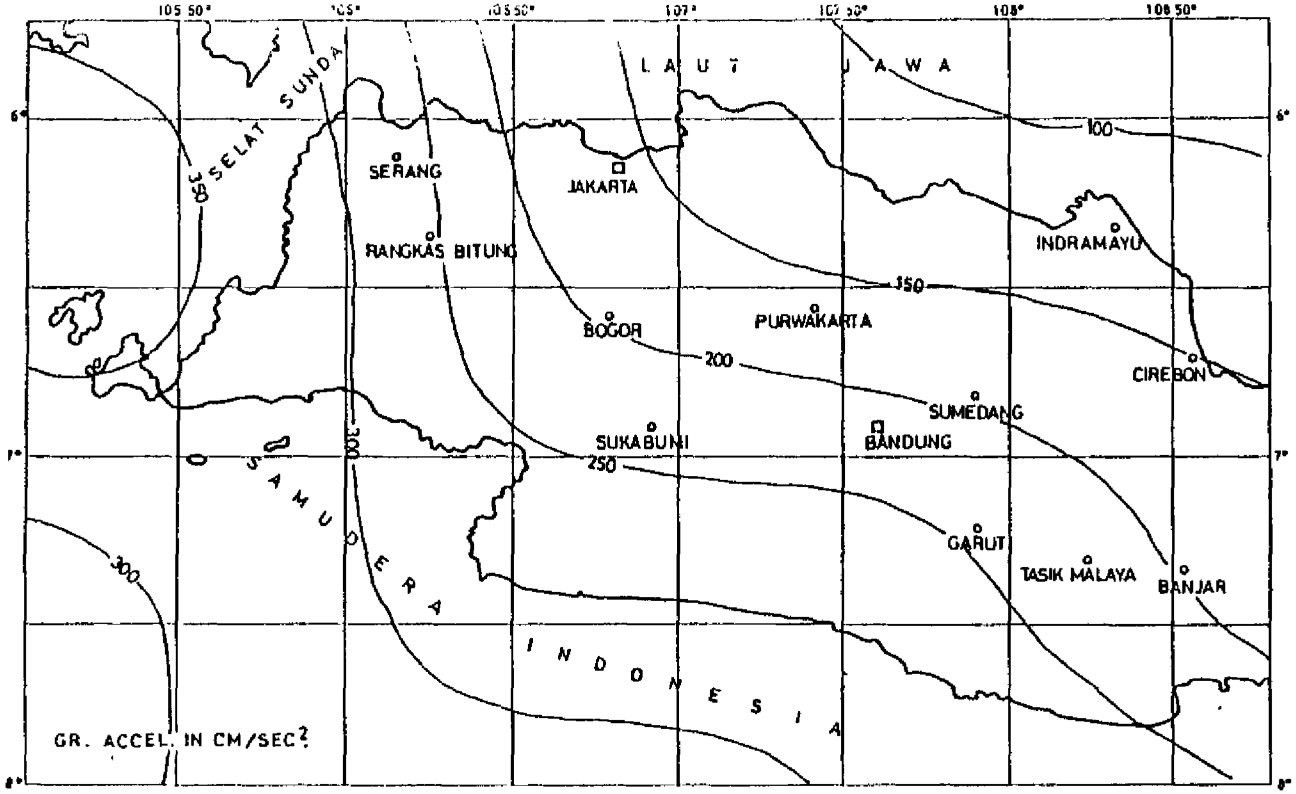


Figure 5

Isoseismal Map of West Java for Ground Acceleration
Having 500 Year Return Period

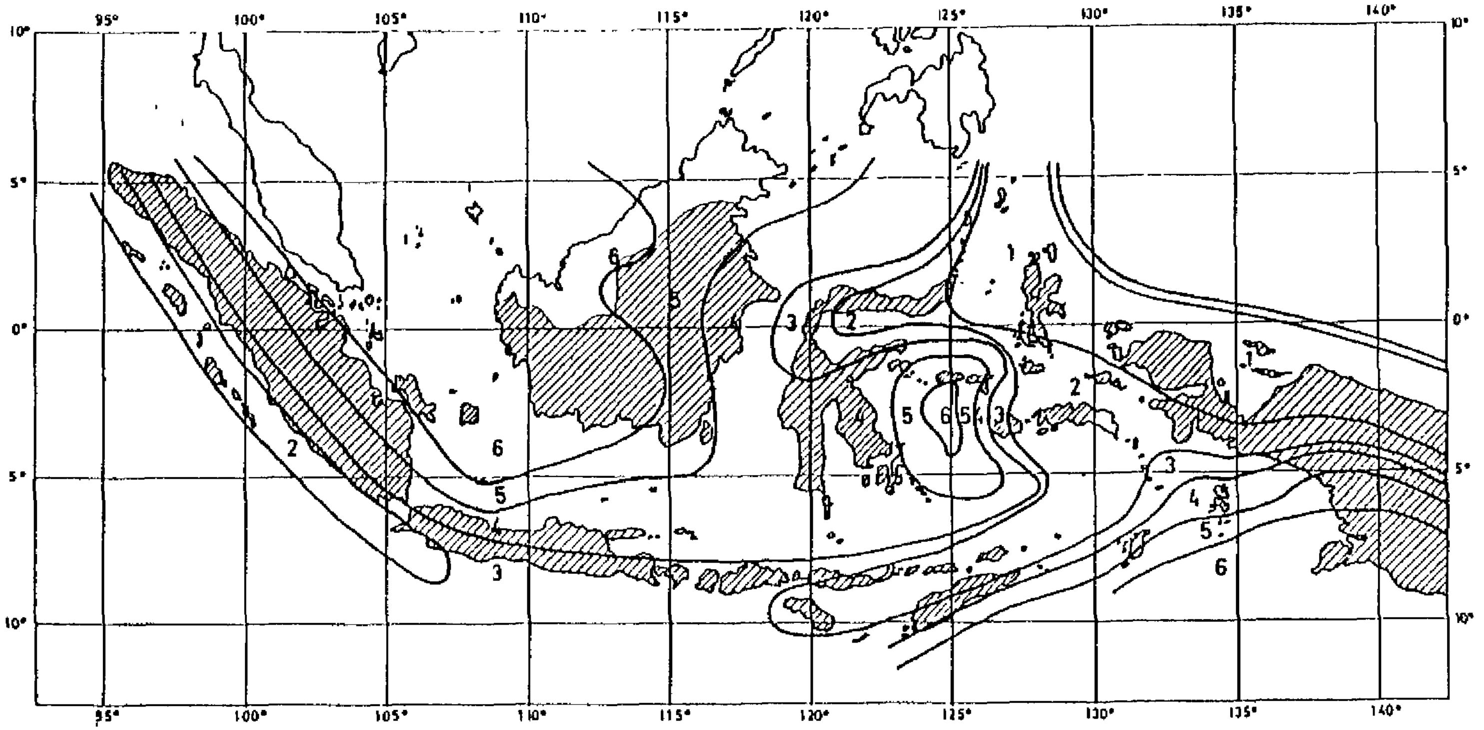


Figure 6

Seismic Zoning of Indonesia

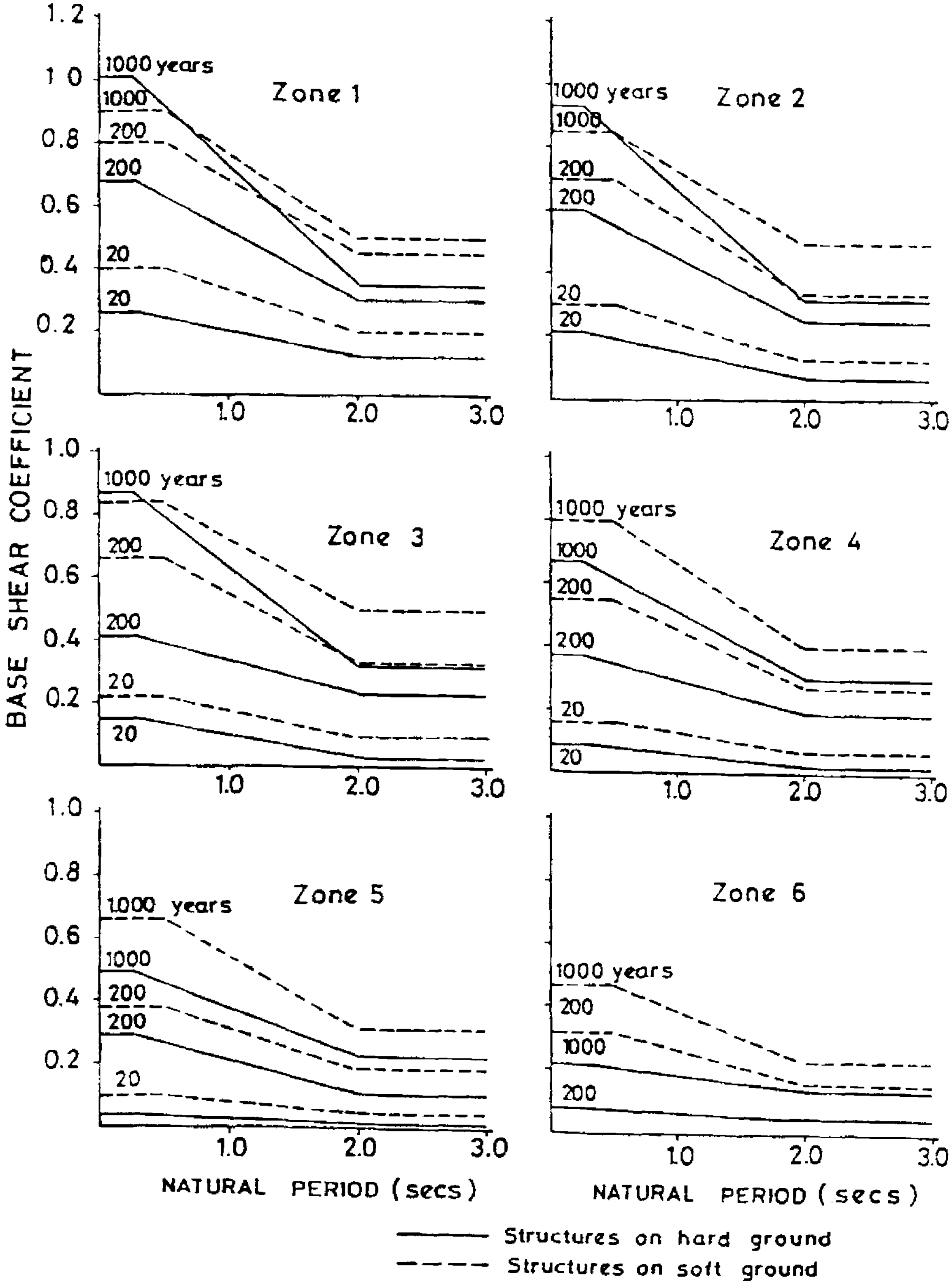


Figure 7

Design Response Spectra for the Six Seismic Zones of Indonesia

Seismic Zoning

Based on the study of isoseismal maps, distinct seismic zones in the Indonesian territory have been identified. By selecting appropriate contour lines for the zone boundaries, 6 seismic zones have been determined in which the probability of occurrence of seismic events is approximately uniform, as shown in Figure 6. This seismic zoning map of Indonesia has been prepared as part of a project called "Indonesian Earthquake Study", conducted under a bilateral aid agreement between the Government of Indonesia and that of New Zealand. This project has been going on since 1976 with a final aim of establishing a comprehensive seismic code for Indonesia. A joint team of Indonesian and New Zealand experts has been involved in this project, resulting in the formulation of the above seismic zoning map of Indonesia. The 6 seismic zones are numbered in decreasing order of seismicity. Zone 1 where Central Irian Jaya and North Maluku are situated is the most severe seismic area of Indonesia, while Zone 6 where West Kalimantan is located is the most stable part.

Table 2

Soft Soil Designation

Description	Depth exceeding (m)
Cohesive soil with an average undrained shear strength not exceeding 50 kPa	6
Any site where the overlying soils are either cohesive with an average undrained shear strength not exceeding 100 kPa or a very dense granular material	9
Cohesive soil with an average undrained shear strength not exceeding 200 kPa	12
Very dense cemented granular soil	20

Design Response Spectra

Another result of the Indonesia-New Zealand project has been the formulation of design response spectra for each of the 6 seismic zones of Indonesia. These are shown together on Figure 7, where they are shown for seismic intensities with return periods of successively 20, 200 and 1000 years, each plotted for hard and for soft subsoils. A structure is considered as being on a soft subsoil, if it is located on soil deposits exceeding the depths indicated in Table 2.

The establishment of the above design response spectra has been the result of an extensive study of all available response spectra of earthquakes recorded all over the world. Studies have been conducted on the influence of the magnitude, focal distance and site geology on the shape of the response spectra. Based on these studies it has been found that for each seismic zone it was not unreasonable to simplify the design response spectra into the trilinear diagrams as shown.

FOOTNOTES

1. The term "seismic risk" is used in this paper as originally proposed by Cornell, McGuire and others, which implies the probability of exceedence of a seismic quantity. Following the UNDRO definition, this should be termed "seismic hazard," and accordingly the associated map should be termed "Seismic Hazard Maps" instead of "Isoseismal Maps".

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