# EARTHQUAKE GROUND MOTION AND ENGINEERING PROCEDURES FOR IMPORTANT INSTALLATIONS NEAR ACTIVE FAULTS

by
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## ABSTRACT

Costly or sensitive buildings, structures, and facilities are being constructed in active earthquake areas throughout the world. Building codes and normal design practices are not always compatible with the inherent risks. This paper outlines procedures for site studies and site selection, evaluation of seismic probability and risk, a new approach to ground motion based upon soil and rock properties, response spectra, inelastic design criteria, and related matters, in the light of current (limited) knowledge. Hypothetical examples are based upon experience with nuclear power plants and major research facilities close to active faults. Analytical, empirical, philosophical, and practical considerations are reconciled.

## NOMENCLATURE

$a$ = peak ground acceleration; cm/sec$^2$ (or g if noted)

$a_o$ = peak ground acceleration at epicenter; cm/sec$^2$ (or g if noted)

$\overline{b}$ = the site, or ground, factor as shown in Figures 2 and 3.

$C$ = static, elastic lateral force design coefficient; dimensionless

$F$ = a factor to express earthquake spectral response values

$g$ = acceleration of gravity

$h$ = focal depth, miles

$M$ = earthquake magnitude; per reference (4)

$R$ = reserve energy reduction coefficient; per reference (18) or (27)

$r$ = hypocentral distance, miles

$T$ = period of vibration, seconds; $T_o$ = period at or near epicenter

$S_v$ = elastic spectral response velocity; ft/sec

$V_s$ = shear velocity of soil or rock; ft/sec

$\alpha$ = elastic spectral response acceleration; g units

$\beta$ = ratio of unit stress at C determination to yield unit stress

$\Delta$ = epicentral distance; miles (See Fig. 8 for $\Delta_i$ and $\Delta_y$)

$\mu$ = ductility factor, or ratio of total deformation to yield deformation

$\rho$ = specific density of soil or rock in place; lb./62.4 ft$^3$ = gram/cm$^3$

log = common logarithm (base 10)

(A few other notations are defined in the text or the figures.)

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## INTRODUCTION

Much has been learned about earthquake activity and seismological engineering in recent decades. However, it must be recognized by all concerned that a great deal more experience and knowledge are necessary before scientific status is attained or the art of aseismic design becomes mature. In the meantime, economic and other pressures demand the construction of sensitive, vital, costly or perhaps potentially explosive installations in earthquake areas, sometimes in close proximity to active faults and to population centers. Examples would include chemical plants, refineries, nuclear power plants, buildings for storing vital records or works of art, military installations, water or fuel storage facilities, tall buildings, and complex scientific research facilities.

Such important installations should be designed not merely to earthquake code design criteria or to broad generalizations or assumptions, but with the full benefits of special site studies; of currently available knowledge and procedures in structural dynamics and earthquake engineering; and with specific consideration of the consequences of damage and the cost of earthquake risk reduction or elimination.

These matters will be discussed herein, with full recognition of the limited state of the art, and with attention directed to more factors than the popular criterion of the epicentral distance,and static lateral force design coefficients. Experience is drawn from earthquake studies for several important installations including nuclear power plants and the $114,000,000 Stanford University electron accelerator facility with its nearest point about 0.7 mile to the rift zone of the active San Andreas Fault. Because of the confidential nature of certain phases of these projects, hypothetical situations and data will be presented in lieu of specific project information. Any similarity to actual projects is unintentional and coincidental.

## PROBABILITY, RISK AND DESIGN PHILOSOPHY

A comprehensive earthquake engineering approach to the design of important installations or structures in active seismic areas requires the development of a special design philosophy. One must first realize the limitations in the state of the art and in empirical data, of apparently rigorous methods which none-the-less depend upon the use of sometimes doubtful assumptions or numerical values in design, and of the very short history of reliably-measured seismic data as compared to the vast ages of geologic periods and processes. It is essential to have humility in the face of what one does not know of the past or of the future, and in the light of his heavy responsibilities today. Above all, it is imperative that one recognize the limitations of the most modern earthquake codes, and those codes in the probable distant future, for special structures, for special risks, and for important installations.

Probability, economics, risk evaluation and good judgment are essential elements in earthquake engineering design procedures. Those responsible for a facility or for the results of a failure, or perhaps a disaster resulting from the sudden release of earthquake energy want to know the probabilities of occurrence. It is useless to say that there are inadequate historical data today to properly assess these probabilities and risks because construction would proceed regardless and probably under the concept that the risks can't be too serious, or they could be evaluated.

A basic philosophy of the recommended approach to the problem is to explore what is known in depth, and to do so specifically for each important project, with full knowledge of the earthquake problem and all developments toward its ultimate solution. A basic objective should be to provide the maximum possible earthquake resistance at no increased cost of construction (over that required for the non-seismic requirements); to evaluate insofar as possible the nature and extent of any residual risks and the means and costs of reducing these where indicated; and to identify and reduce at minimum cost any risks that appear to be untenable. One should also refrain from seeking refuge in the excuse for using doubtful or old assumptions, numerical values, or broad generalizations, that such are within the limits of accuracy of the end product. It should be recognized today that the whole is the sum, if not the product, of its constituent parts.

It is often very helpful for owners and authorities, as well as for engineers, to consider degrees of earthquake risk and of probability rather than to select one earthquake level and to make that criterion a case of "all or nothing". The writer prefers to assign limits or bounds - a lower limit below which the installation would be an unacceptable risk, and an upper limit which is the maximum possible or credible event in the present state of knowledge. Two or three intermediate levels are then established and probabilities of occurrence (in years of frequency) and of results (in extent and consequences of damage) are estimated along with the initial incremental costs to improve the risk and reduce the damage potential in advancing to each level of probability. It is remarkable how much more earthquake resistance can be obtained at little or no extra cost with this "CARE" program ("Compatibility Analysis of Risk and Economics") for special problems, especially when advantage is taken in design of the reserve potentials in the inelastic range of response.

A basic criterion is to gather all possible facts, general and specific, including the characteristics of the specific site and installation,and to avoid over-simplified rules or practices that tend to compound error and risk.

## SITE CONDITIONS AND EARTHQUAKE EXPOSURE

Some have held that proximity to an active fault is so potentially dangerous as to make such installations unwise or impractical. Others feel the problem is essentially regional and that proximity to a fault is not in itself a controlling parameter [1]. Regardless of the basic causes of fault or block movement, it is generally recognized that earthquake epicenters occur somewhere along fault systems and constitute the immediate sources of energy potentially destructive to man's structures and facilities. A study of iso-intensity maps based upon the Modified-Mercalli[2] [3] [4] [5][6] or similar rating systems, reveals (in spite of the many problems and inaccuracies inherent in any such rating system) useful relationships of epicenter, magnitude, focal depth and damage. However, the use of such data in the development of design criteria requires judgment based upon much experience to allow for the changes (with time) in buildings and structures, variations in exposure or population density, soil and geological conditions, extrapolation into future exposures and many other factors. As an example, it would be meaningless to use such data (without adjustment) in the prediction of damage to tall modern buildings if no such buildings existed at the time the intensity maps were prepared.

Parameters that have not always been considered sufficiently include the geological and soil conditions at and about the site of a proposed installation, specific (local) probabilities and types of ground motion, dynamic

characteristics of the installation, the probabilities and consequences of possible response phenomena, and evaluations of the interaction of structure and ground.

Site Selection. Selection of the site may be limited or it may extend over a considerable area. There are many factors to be considered besides that of earthquake history. These factors are often ignored until it is too late for the earthquake engineer to participate effectively, if at all, in the site selection. However, even when the property is limited by conditions beyond control, there may be a choice of two or more sites, or a selection may be possible from specific locations within a large area. At many locations there are diversified geological and soil conditions, sometimes with wide variations in a relatively small area. In any event, geological, seismological, soil and rock mechanics studies are essential initial steps to select or generally evaluate the site or to properly locate the installation within a large area.

Geological Procedures and Factors. One should obtain copies of all existing maps and geological classifications of the general area in which the site is located. The data should be compared and evaluated in view of the time, the extent, the then current knowledge, and the reliability and adequacy of each effort. Usually, additional on-site studies are indicated, with borings, detailed sample studies and a search for possibly unknown faults. If major faults are on site or close to the site, detailed field work is indicated to locate the extent of the rift zone and any fault branches. Geophysical procedures together with aerial photography and mapping are valuable aids in determining where rock is located, its type, age, faulting and the geological history of the area. The final decisions on site suitability and plant location on the site should be made only in the light of complete geological as well as other essential data.

Important installations near active faults are generally feasible but they should be placed on and into competent and stable rock with no faults or rift zones under the installations per se. This is not always possible or practical but it is a basic matter to explore this possibility. Sometimes inactive faults must be considered and accepted. The definition of an inactive fault is sometimes a difficult problem involving probability considerations and judgment. Any indication of differential movement along a fault within recent decades or centuries, or perhaps within several thousand years, should be considered a sign of possible future activity to be avoided by relocation of the installation away from this fault. If a known active fault is adjacent to the site, a search should be made for any offshoot faults that might be potentially active. If there should be indications of fault or crack movement some time ago, tests can be conducted to estimate the time of these movements. It is generally very difficult to find completely uncracked bedrock but most of these cracks are harmless and are not faults per se. Even long-inactive minor faults have been tolerated if unavoidable and when competent experts agree future differential movement is highly improbable. Some structures have been built over inactive faults with provisions for non-destructive differential movement.

No structure should be built across a major, active, or potentially active fault or over a fault rift zone since permanent base distortion is irresistible. However, mere proximity to an active fault can be a lesser risk than other parameters and site conditions [(1) (7)]. Competent rock

bearing is very desirable but if a rock base can not be obtained practically, the site may still be satisfactory for many installations, but with more design and construction problems, and more cost. Saturated, soft or poorly consolidated alluvial materials and wind deposited materials are generally undesirable for important installations near active faults and, in many cases, at considerable distances from major faults.

Seismological Studies. The complete earthquake history of the general area of proposed construction and also of the major geological structure over a large contiguous area should be obtained or developed. The history of seismic damage may be poorly recorded or be misleading, especially in formerly non-populated areas. Experts can, however, evaluate the reliability and the scope of the reports. A lack of reports about damage should not be considered, necessarily, as an indication of no past major earthquake activity. There may have been no structures to be damaged, perhaps no one recorded the damage, the types of structures may not have been responsive to the particular earthquake motions, or the epicenters may not have been close enough to the area in recent decades or centuries even though a nearby fault or faults were, and are, still active. Seismologists can make detailed studies of specific areas and prepare maps showing known epicenters, magnitudes, faults and possible faults, intensities, and provide predictions based on the past and also on the geological conditions. The basic problem is that the probabilities of future events must be based upon what is usually inadequate knowledge or history of past events. However, there are procedures to improve statistical data.

Detailed knowledge of events in similar localities plus careful study of the area with stochastic processes can, to some degree at least, improve the confidence level over that based solely upon inadequate history of the specific area under consideration. For example, projections of possible close epicenters can be made, as illustrated by the hypothetical situation in Figure 1. The nearby active faults are "A" and "B" with known earthquakes as shown. The distances in miles to the epicenters of interest, and also normal to the two faults are shown on the radial lines. Table I provides local earthquake data and Table II a particular probability study based upon the relative exposure to active faults. A problem here is whether or not a major earthquake could occur 8 mi. from the proposed installation. Some major earthquakes apparently tear or spread so rapidly along a fault that for all practical purposes the energy release is from a line and not a point(8)(9). If the effective line were taken as 30 miles, any "point" within 30 miles on either side of point "a" would thus still constitute a minimum distance exposure.

A design based only upon the earthquakes indicated in Figure 1 would hardly be an adequate consideration of the potential risks to an important installation at Area X. Tables I and II indicate that both faults are active, but that Fault A is more active for the major earthquakes. One should consider the possibility of the fault's greatest known magnitude of 8.3 occurring as a line-release at the minimum distance. The average frequency of this particular event happening (with a 30-mile "line") is estimated as once (average) per 8333 years based upon a re-evaluation of the frequency shown, for 60 miles of fault within 30 miles of the site, as compared to 250 miles within the 60 mile radius. If one should decide to consider in design the frequency of once per 8333 years, he would no doubt do so with considerable allowable damage, short of catastrophic consequences. In view of world

history, an event greater than M8.3 could occur here also, but the probability would be extremely small. This is one of the great earthquake problems - one does not know when the frequency clock started to run. Obviously, various levels of risk and probability must be considered.

Future focal depths can generally be assumed to be the same as for the shocks recorded on the corresponding faults. For example, in Figure 1, 15 miles would be a logical figure. However, for a very sensitive and close installation, shallower releases should also be considered possible. Such, even at low magnitude, could be critical for rigid elements, whereas longer epicentral-distance shocks of greater magnitude could be critical for long period and lightly damped structures. Complete evaluation should include all possible situations and not an assumption, a priori, that only certain magnitudes, depths, or epicenters might control the response and design of all elements.

In the writer's opinion, the use of seismic history for areas[9] of states or countries such as California, New Zealand, or Japan, is not sufficient as an exposure basis for assigning probability factors for a specific local area. The historical record is all too short and an important "proximity" parameter is not included. Other procedures have also been followed[10] to estimate ground motion for important installations near active faults. One is to obtain all Modified Mercalli, or similar ratings, for a given site and to adjust these by calculation and judgment for the changes (with time) of building types, population densities, and revisions in the MM ratings, all as compared to the times of the intensity evaluations. With a tabulation of the frequencies in years of each intensity rating at the site and finally, by analogy to intensity frequencies in the better known seismic areas, the probabilities and yearly frequencies for intensity values of interest can be estimated. A third and new procedure based upon ground motion for various magnitudes as indicated by site properties will be submitted in detail subsequently.

Soil and Layer Conditions. Even where the main unit or structure of an installation is to be founded on rock, the superimposed soil conditions may be very important, from several considerations, and must be studied. Moreover, there may be other plant structures of lesser importance which could be founded on alluvium. The relative motion between rock surface and alluvium above the rock, and also at various levels in the alluvium including its surface, must be estimated to provide in design for the movements or movement tendencies and the resulting forces - for pipelines, for connections of various structures, and for the action of alluvial materials or backfill on the portion of a rock bearing structure between the rock and the ground surface. In some cases, the main structure on rock should be so strong and rigid as to cause adjoining soil to rupture before the structure is damaged.

Dominant ground periods may exist with natural modes which would be excited by earthquakes. This motion would in turn be especially troublesome to any structures having natural periods in the same spectral range as that of the dominant ground motion. Kanai and others[11] [12] [13] have done much work in this subject and have used instruments to record microtremors and to detect dominant ground periods. It is often desirable to use geophysical exploration methods to determine the depth and number of alluvial layers and the seismic properties of density and velocity of the materials. Such information is needed in the specific calculation of any dominant ground periods and of

amplifications of motion in soil above rock due to refraction and reflection from layer to layer and from the surface. Very useful compilations of soil data have been developed for certain USC&GS California strong motion recording stations (4) (14) (15).

It is fundamental to make borings to determine ground water levels, to obtain soil samples, and to test these samples for density, moisture content, grain size, cohesion if any, and their appropriate strength, deformation and other properties. The subgrade modulus, or coefficient of subgrade reaction, is also of basic importance in period calculations and in interaction seismic studies. Where important soil-structures or embankments are involved, special studies or tests of the dynamic stability of the soil or of slopes can be made(10) (16). In some cases, it is desirable to estimate the amount of embankment sliding, if any, under the probable periods, amplitudes and durations of ground motion. Shaking table tests(17) are informative as are calculation procedures based upon the balance of energy with work done under "elastic" and permanent deformations. The possibility of dynamic liquefaction is not to be ignored.

## EARTHQUAKE GROUND MOTION

One of the greatest problems is to reliably estimate the ground motions to which an installation will be subjected. Code lateral force coefficients do not represent the maximum probable response accelerations but much less so long as the structure remains elastic(20) (28). Ground motion often exceeds these values and is amplified in the alluvium, and the response of the structure leads to even greater values. Although some structures may have reserves of strength or of energy absorption value adequate to withstand accelerations much greater than normal code design levels, it would be an inadequate procedure in today's state of knowledge to design an important installation solely under this assumption. Seismic coefficients, possibly increased or with extended safety factors for special risks (22) may be used for initial (trial) design, but the structure should still be analyzed for the motion it will probably receive.

Available Data. Many have worked on this problem and much has been learned. Unfortunately, however, most strong motion records have been made on soils of variable and generally little known characteristics. Gutenberg(1) has provided data on the motion of various alluvial deposits as compared to motion in rock at the Pasadena Seismological Laboratory and he and Richter have provided useful relationships of magnitude, energy, intensity and acceleration and certain corrections for alluvium characteristics(4)(19) Neumann worked diligently to correlate intensity in alluvium with that on granitic rock(5). Kanai has worked in not only layering and site periodicity but in rock motion (7) (11) (12). Wiggins has considered site conditions as related to ground motion spectra (23). For special projects it seems essential to develop motion characteristics based upon specific site conditions and materials rather than to depend upon general or averaged - and perhaps inapplicable - data. Unfortunately, very few strong motion recordings have been made on rock. The U. S. Coast & Geodetic Survey has recorded Helena, Montana shocks on limestone (1935, M 6.0) and a San Francisco shock (1957, M 5.3) on weathered Jurassic sandstone(2). More rock data are needed. However, seismologists have recorded a considerable amount of rock motion and have issued reports which can be very useful to earthquake engineers (4).

Gutenberg and Richter provide the equation:

$$\log a_o = -2.1 + 0.81M - 0.027M^2 \qquad \text{-----(1)}$$

in which the constant 2.1 applies to the average base conditions of the Pasadena Seismological Laboratory stations which are generally situated on or close to weathered granitic rock. For the purpose at hand, let us replace the constant in equation (1) with a symbol $\bar{b}$. Suggested average values[4] to reconcile M and $a_o$ with recorded data from other sites are 1.7 for USC & GS stations at cities in Southern California, generally on alluvium; and 1.4 for relatively unstable ground such as at El Centro or Bishop, California. Gutenberg provides similar results[1] with amplitudes for stations on alluvium compared to those on rock with identical instruments. Ratios of the alluvium to rock motion range up to 10, but with typical values of 2.5 to 6 for alluvium from 300 to 1200 feet thick. Gutenberg suggests the following rough relationship for this ratio, designated m, for the Los Angeles area: $m = 1 + d/300$, with other parameters ignored. However, there is considerable scatter of plotted data and it is noted that "relatively large amplitudes----may be observed in areas with only a thin layer of alluvium covering crystalline rock."[1] The duration of shaking, the number of cycles and also the periods, tend to increase with the thickness of alluvium. There are thus many reasons why important installations near active faults should be situated on and be keyed into competent rock.

Evaluating Site Motion. There have been no major earthquakes recorded on rock close to an epicenter. An exhaustive study was made[10] to correlate soil and rock properties and other parameters for the application of available seismic data to the problem. In order to "reach" the rock it was first necessary to estimate the alluvium magnification over rock. Part two of the UCLA station program[15] was not available when the studies were made and Part one[14] became available very late in the program. Approximate calculations had been made of layering effects for specific USC&GS stations and these data were compared to the data from the Seismological Laboratory[1] and, subsequently, from reference (14). The comparisons were acceptable. A method was developed[10] for evaluating site motion in the light of the specific soil or rock properties under a proposed structure. Alluvium depth, d, and layering are not included as direct parameters but the procedure utilizes as average material much empirical data in which the actual soil response is measured. It is hoped that with time enough station site data will be available to use these parameters and perhaps dominant site periods more directly in a relationship $\bar{b} = f(\rho, Vs, d, T)$.

A procedure is offered herein to evaluate site acceleration as a function of $\rho$ and Vs with empirical relationships (partially extrapolated) to recorded earthquake data in Southern California[4]. The results are considered more reliable for rock or other firm material (say $\log \rho Vs > 3.5$) than for soft, deep or layered alluvium for which the accelerations obtained should be reviewed and perhaps modified for any unusual site conditions. The site and general area material is tested or reliable data procured otherwise, and values of specific density $\rho$ and shear velocity Vs are obtained as indicated in Figure 3. The site factor $\bar{b}$ is then selected. Figure 3 indicates the average, maximum, and minimum values of $\bar{b}$ necessary to reconcile the Gutenberg-Richter data [4] with what was known or could be estimated about USC&GS station sites. The constant 2.1 in equation (1) represents the average (curve B) at a $\log (\rho Vs)$ value of 3.93. The average value of $\bar{b}$, or values falling between curves B and C would be appropriate in most cases. In example 1 shown on Figure 3, the

average curve B is used to get $\bar{b}$ = 1.9. The values of $\rho$ and Vs are test results for an Eocene sandstone. Entering Figure 2 with values of M and $\bar{b}$, one obtains the corresponding values of $a_o$. The epicentral acceleration for example 1 is 0.7g.

Figure 4 indicates the sensitivity of $\bar{b}$ and the magnification between two hypothetical sites. The example 2 difference in $\bar{b}$ of 0.7 indicates 5 times more amplitude at site 2 than at site 1, as shown in Figure 4. Site 1 is rock and site 2 rather poor alluvium. As can be seen also in Figure 2, the accelerations increase rapidly as $\bar{b}$ decreases.

Figure 2 provides epicentral acceleration. If the site is not at the epicentral area (and it usually is not), the site acceleration, a, must be determined. A study of the attenuation of acceleration with epicentral distance included the consideration of several procedures: the application of a great deal of empirical data[4]; energy scaling; Kanai's equations[7]; Wiggins' approach[23]; and finally the developed procedure[10]. Typical curves are shown in Figure 5 for M = 8 and for rock sites. In the case of curves (3), (4) and (6) the $\bar{b}$ values are from Figure 3. The Vs product is shown for curves (5) and (7), Wiggins' procedure[23]. For Kanai's curve (1), and curve (2), the only description is "bedrock"[7]. Where $\bar{b}$ or $\rho$Vs varies, there is, of course, no comparison except by relative curvature. However, curves (3) and (5) are for the same site material, and curves (6) and (7) are for another identical material; thus these two pairs of curves can be compared directly. For curves (3) and (6) the $\bar{b}$ values are from the B curve of Figure 3. Curve (4) is according to reference (4) but with a $\bar{b}$ value of 2.0. The sharp average attenuation above 12 miles was of concern so the writer employed the following relationship for curves (3) and (6), with $a_o$ in gravity units as per Figure 2:

$$a = \frac{a_o}{1 + \left(\frac{\Delta}{h}\right)^2} \text{ ; in g units} \qquad \text{-----(2)}$$

Kanai's bedrock formula (7) was apparently not intended for short epicentral distances where it tends toward infinity. Curve (1) is shown for T = 0.5 seconds, a possible dominant mode[4] for M = 8 near the epicenter. The writer took the liberty of substituting the hypocentral distance for the epicentral distance in Kanai's formula with the results shown as curve (2), a satisfactory agreement with the shape of curve (3), which is a continuation of example 1 from Figure 2 and 3.

The conversion of acceleration and epicentral distance relationships to ground motion spectra is the next step. Much more raw data are needed in this subject. A study of rock motion recorded at Helena and at San Francisco, and also of the rock signals received by seismologists finally led to the spectra shown in Figure 6. Although there are dominant period tendencies as indicated, it must be assumed that there is a complete random energy supply containing all frequencies. It was decided, more or less arbitrarily, to assign a magnification of 1.25 times $a_o$ and to peak this motion as shown. Some rock stations show such peaking[1]. Microtremor studies and forced vibration tests are indicated for sensitive installations. Velocities were calculated at

the $T_o$ period, from the original non-amplified accelerations, $a_o$. Accelerations at longer periods were computed for harmonic relationships with assumed constant velocity.

Variable Motion with Height. In addition to dominant periods and layering phenomena in alluvium, amplification of motion with height above the rock surface must be considered. Although these matters are related, one could be concerned in design with one and not the others. If a major structure such as a nuclear reactor containment vessel is founded on firm rock well below the ground surface, its subterranean portion would have to resist local backfill motion that would be much greater, especially at its surface, than the rock base motion. Underground connections to the main structure would be subject to differential movement, the amount of which has to be provided for. Relative motion in alluvium has been investigated (7) (12) (13). Surface displacement due to natural processes is at least twice the motion at the interface of alluvium and rock, and this ratio is probably much more than 2 under many conditions. Where massive elements extend from rock through a soil layer,the differential soil motion must be considered up to rupture level of the soil. In some cases, a compressible buffer is provided outside of, and to protect, the main structure.

## RESPONSE TO GROUND MOTION

Ground or rock motion and ground motion spectra are not to be confused with response spectra which represent the response of idealized structures to the ground motion. Spectral response diagrams (24) are available for several earthquakes and various damping values, usually in terms of acceleration or velocity. Most are for elastic systems (8) (24) (25) although response of inelastic systems is also available in spectral form (26) (27) (28)(21). However, these spectral diagrams are not for close epicenters or rock sites under severe earthquake motion. In 1958 the writer developed "smoothed" diagrams without peaks and valleys and with allowances for near and far shocks not yet recorded. The equations provided for spectral response are readily applied in design or analysis (18) (20) (27) (29):

$$\text{Spectral response velocity} = S_v = FT^{1/4} \qquad \text{----(3)}$$

$$\text{Spectral response acceleration} = \alpha = 0.194FT^{-3/4} \qquad \text{----(4)}$$

F varies with damping, magnitude, and the site; F 1.83 agrees very closely with the El Centro 1940 N-S spectrum for 5% damping (29). There is considerable evidence that an assumption of constant spectral velocity or of averages of a few spectra can be an inadequate basis for special projects.

The short period portions of the 5% damping response spectra in Figure 7 were developed for the rock spectra of Figure 6. At zero period the response is the basic rock acceleration $a_o$; this would be so for any damping (18). The peaks were taken at 2 $T_o/3$ with a dynamic magnification of 1.87 times, averaged from calculations of various pulse, random, and sinusoidal responses and by analogy to many actual response spectra. The 1.6g cutoff is arbitrary.The longer period portions of the curves are by equation (4) with F values as shown. The M 8.3 - F 3.08 curve represents an unlikely event at any particular site. Response would be even greater if the site was not rock. All of these "elastic" values can be reduced significantly by designing for inelastic response (18) (27). (See "Structural-Dynamic Design Considerations".)

The use of equivalent masses and other adjustments if actual systems can not be idealized by single-mass shear type systems, modal combinations in spectral response where multi-mass or distributed mass systems are involved,

and period computations, are beyond the scope and space of this paper but are treated in the proceedings of the two prior world conferences and in other literature. In some design situations it is possible to apply a forcing function to a simplified dynamic system in order to obtain response directly. The forcing function could have the motion characteristics of the ground (such as in Figure 6), or it could be taken arbitrarily at a "tuning" period of say 0.85 or 0.90 of the structure period in order to approximate transient response by steady state computations.

## STRUCTURAL-DYNAMIC DESIGN CONSIDERATIONS

The determination of applicable dynamic systems and their characteristics is an art which embraces both structural engineering and dynamics, and much experience. From trial designs one determines degrees of freedom, periods of vibration, damping, ductility and work capacity in the inelastic range. It is important to consider all effective mass and various possible types of motion such as from shear, flexural, axial, torsional, and ground stresses, and possible coupling phenomena. Although multi-mass systems can often be analyzed conveniently by computers, many plant or industrial facilities are more suited for manual calculations, well seasoned with good judgment.

It is customary to assume a ratio of viscous damping to critical damping. The writer's earlier suggestions (18) (20) (29) of 5% damping for normal buildings or composite structures seem to be confirmed in view of recent test data. Damping values of bare metal structures are much less, however, particularly if the connections are welded. Where there is interaction of the structure and ground and severe motion is involved, 10% of critical damping seems reasonable. Damping per se is not to be confused with other forms of energy absorption. For design, damping represents internal attenuation without residual damage, cracking or distortion. As structures respond in the inelastic range, there is much greater attenuation. The writer prefers to consider this inelastic work as a separate parameter and to add energy and deformation to strength to create three basic design considerations. The severe response accelerations to which a structure tends to be subjected if it remains elastic in major earthquakes are indicated in Figure 7 for rocky material ($\bar{b} = 1.9$). If the material is less competent and/or if the damping should be less, the response is greater. These values, which are far greater than code coefficients, generally can not, but need not, be met with elastic unit stresses or traditional static procedures. Where distortion beyond the yield point can be tolerated under a severe and rare emergency, economy and security can be achieved by utilizing the great energy absorption characteristics of ductile materials in the inelastic range. The procedure is to design the structure on a trial basis according to code (or perhaps slightly higher) coefficients and then to check for the energy absorption capacity necessary to resist the earthquake motions that have been selected for the particular site and risk. The reserve energy technique (18) (29) is a useful tool to perform this analysis with terms and procedures familiar to the structural engineer.

As a simple illustration of the value of inelastic response to "reduce" the accelerations, an elastoplastic single-mass system is shown in Figure 8 together with curves showing the relationships of ductility $\mu$, the ratio of $C/\beta$, and spectral response acceleration $\alpha$. $\mu$ values up to 1 are elastic and thus at 1, the $C/\beta$ ratio conforms to the spectral demand at yield level. For example $3\rho.5g$ is met elastically at $C/\beta = 0.50$ and $\mu = 1$. But if a $\mu$ factor of 4 could be tolerated, a $C/\beta$ ratio (interpolated) of 0.19 would also satisfy the earthquake demand. If C were developed at a unit stress of 2/3 yield stress, the required value of C would be 2/3 x 0.19 or about 0.13. The reserve

energy procedure can also be used for complex or composite systems regardless of the force-deformation relationships.

Materials which fail at "brittle" $\mu$ values, from 1 to 3, are to be avoided or the structures must be equal to or greater than the elastic spectral demand. If the geometry of a structure causes great resistance in the elastic range, it is only necessary that the strength be adequate for the response acceleration. In such cases, however, the consequences of failure must be evaluated and appropriate safety factors be provided for emergencies or the cumulative effects of error and approximations in the earthquake design procedure. Materials have been blamed for many failures and they have been lauded for survivals. In view of the nature of the seismic problem and the facts that there are 15 to 20 parameters involved and material characteristics per se may account for only 2 or 3 of these, the materials deserve neither all the blame or all the credit. It is the writer's opinion that any consistent structural material can be used successfully, and that any material can be used poorly. Materials that fail abruptly in shear or compression can be, and should be, so designed where ductility is essential that they cannot so fail prior to the stretching of the reinforcing steel beyond its yield point, in order to insure ductility. Reference (27) provides data for this important concept which should be observed for all important installations. Where deformations beyond the yield point are utilized for resistance to severe ground motion, such must be sustainable without critical buckling.(30) Overall adequacy is the proper criterion rather than the properties of test samples. $\mu$ values of the structure per se must be reliable.

## CONCLUSION

Probability, risk, and design philosophy have been considered with site investigation and a procedure to evaluate earthquake motion for specific sites. Design spectra are developed, and special analysis proceeds with attention to energy capacity and strength. Risk may be considered in 2 or 3 stages. How much can the structure sustain in each stage with no increase in construction cost over that required for normal purposes and code design? An installation should withstand severe earthquakes with little or no damage and with only minor excursions into the inelastic range. However, for a catastrophic earthquake which might severely damage a neighboring city, it seems permissible to allow excursions into the inelastic range, to $\mu$ factors of 4 to 6, or more. For the devastating occurrence, which would essentially destroy the city, the installation should not collapse or be damaged to the point of requiring replacement. However, deeper excursions into the inelastic range with more damage could be tolerated if reserve ductility can be mobilized.

In many cases capacity to resist very severe earthquakes can be provided in design at little or no increase in cost. There is everything to be gained and nothing to be lost by this process. In other cases, it may be found that to develop the initially-desired earthquake value without damage may require considerable extra money. A reevaluation may then reveal that a lesser value can be used without dire consequences, and the extra cost not be expended since it would be an added investment with inadequate returns. The term "CARE", for "Compatibility Analysis of Risk and Economics" is appropriate. In spite of the amount yet to be learned or to be confirmed, a great deal more can be done today with "CARE" and special effort, than ever before in seismic history. The gains could be enormous.

There are also other basic factors - good construction according to well-detailed drawings, good testing and inspection, and engineering supervision during construction. In addition, permanent, built-in instrumentation is needed to obtain data for future improvements in the art.

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## TABLE I - EARTHQUAKE EXPOSURE FOR AREA X

| Earthquake data for Area X on Figure 1 | Fault A | Fault B |
|---|---|---|
| Closest > M5: | | |
| Magnitude; year | M6.2; 1890 | M6.7; 1929 |
| Epicentral distance | 18 miles | 40 miles |
| Maximum within 60 miles: | | |
| Magnitude; year | M7.6; 1915 | M7.0; 1902 |
| Epicentral distance | 29 miles | 58 miles |
| Maximum within 200 miles: | | |
| Magnitude; year | M8.0; 1860 | M7.2; 1875 |
| Epicentral distance | 90 miles | 140 miles |
| Maximum anywhere on fault: | | |
| Magnitude; year | M8.3; 1830 | M7.6; 1885 |
| Epicentral distance | 250 miles | 300 miles |
| Minimum distance to fault: | 8 miles | 35 miles |

## TABLE II - EARTHQUAKE PROBABILITY CONSIDERATIONS

| (1) | (2) | | (3) | | (4) |
|---|---|---|---|---|---|
| | Historically, entire state or country | | Historically, with epicenters within 60 miles of Area X | | *Estimated average years between earthquakes for the area within a 60-mile radial sweep of Area X; from seismic history of entire state or country, weighted for fault proximity |
| Earthquakes in a 100 year period of magnitudes shown | No. | Av. yrs. between earthquakes | No. | Av. yrs. between earthquakes | |
| 8.4 or more | 0 | Infinite | 0 | Infinite | Infinite |
| 8.3 or more | 1 | 100 | 0 | " | 2000 |
| 8.2 or more | 1 | 100 | 0 | " | 2000 |
| 8.1 or more | 2 | 50 | 0 | " | 1000 |
| 8.0 or more | 5 | 20 | 0 | " | 400 |
| 7.5 or more | 15 | 6.7 | 1 | 100 | 134 |
| 7.0 or more | 65 | 1.5 | 6 | 17 | 30 |
| 6.5 or more | 190 | 0.5 | 17 | 6 | 10 |
| *Total length of active faults, miles | 5,000 | | 250 | | |
| *Proximity factor | | 250/5000 = 0.05 | | | |

Column (4) data indicate that the probabilities of earthquakes at Area X are greater, especially for major shocks, than shown by the historical data in column (3).

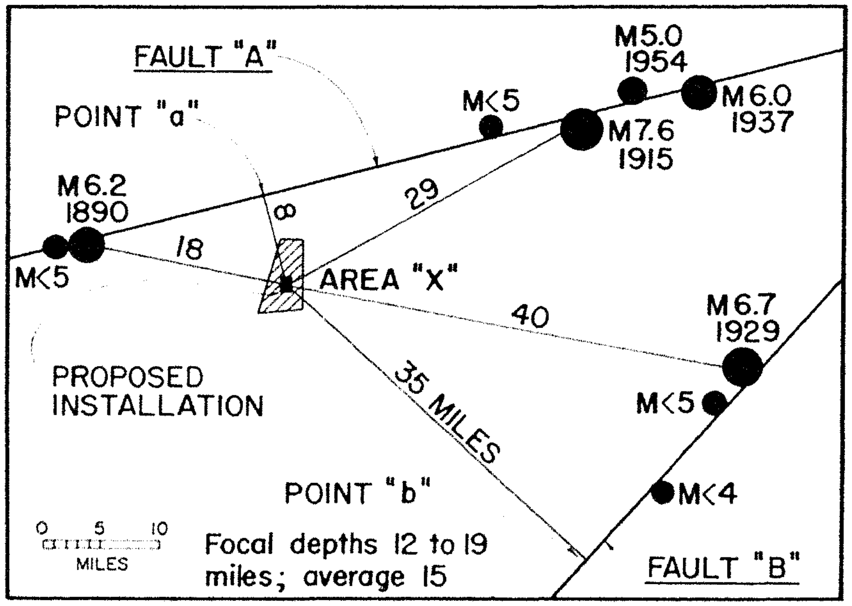


FIG. 1 - A HYPOTHETICAL FAULT PROXIMITY MAP

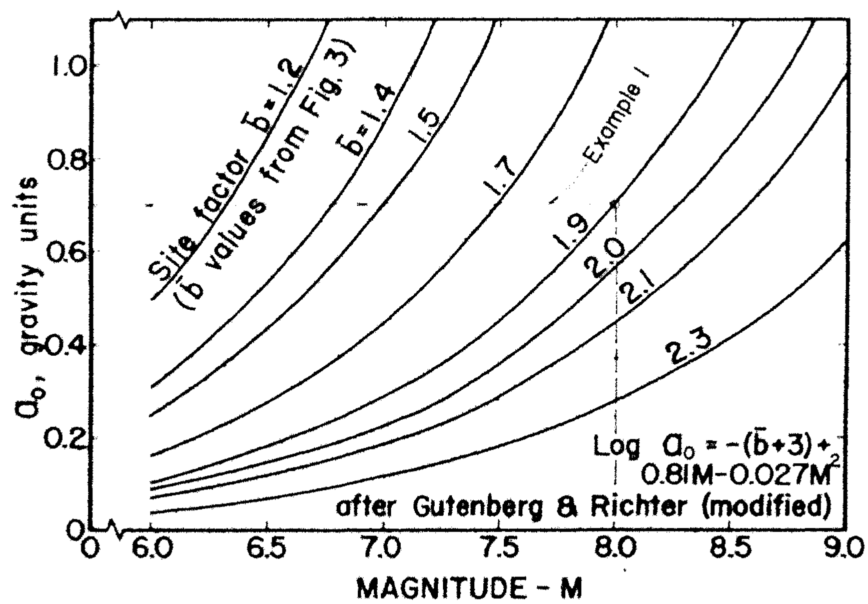


FIG. 2 - DETERMINATION OF EPICENTRAL ACCELERATION

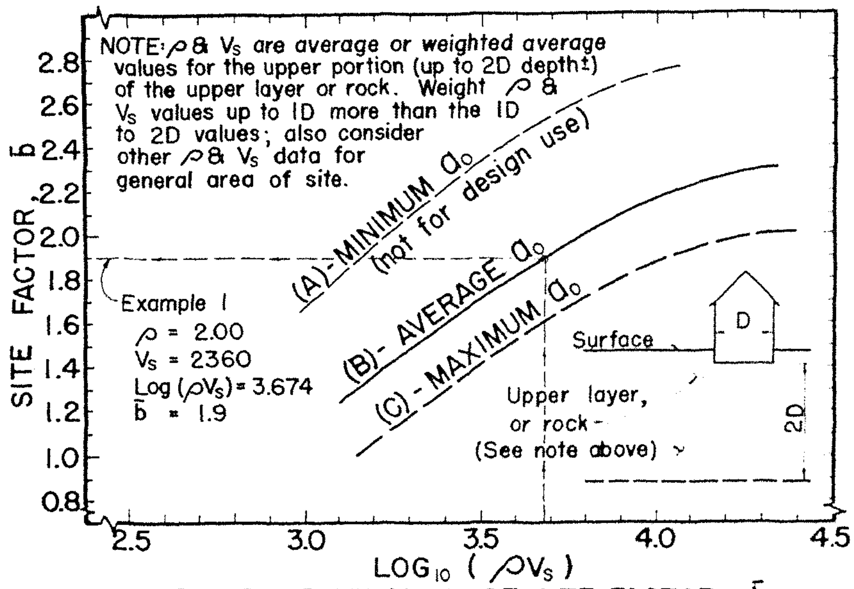


FIG. 3 - DETERMINATION OF SITE FACTOR, $\bar{b}$

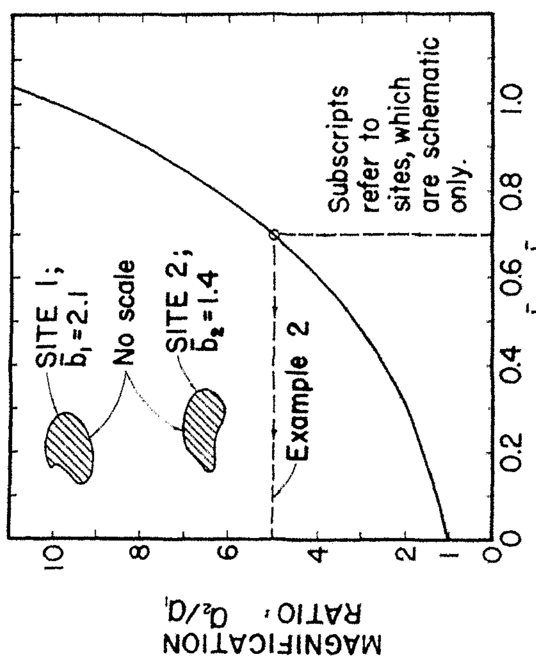


FIG. 4 - MAGNIFICATION BETWEEN TWO SITES

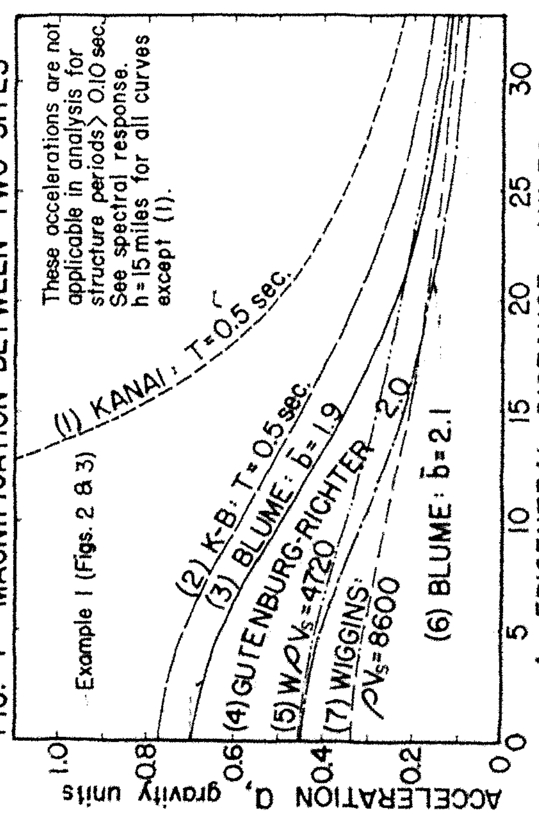


FIG. 5 - ROCK ACCELERATIONS FOR M = 8

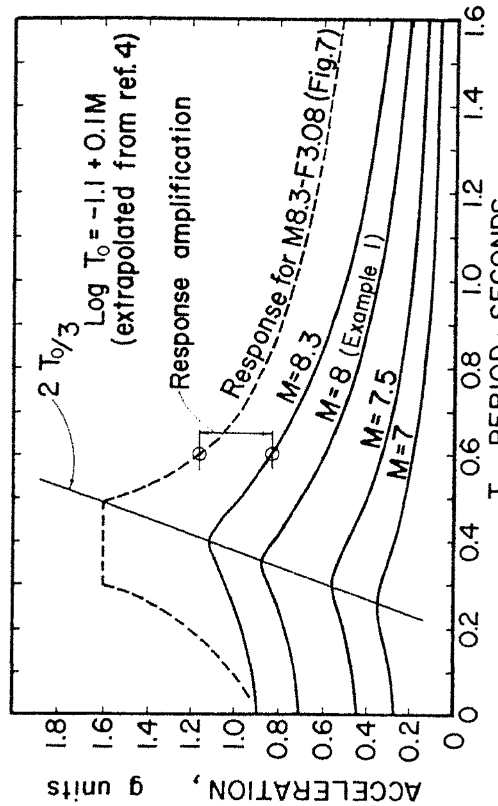


FIG. 6 - GROUND MOTION SPECTRA FOR $\bar{b}$ = 1.9

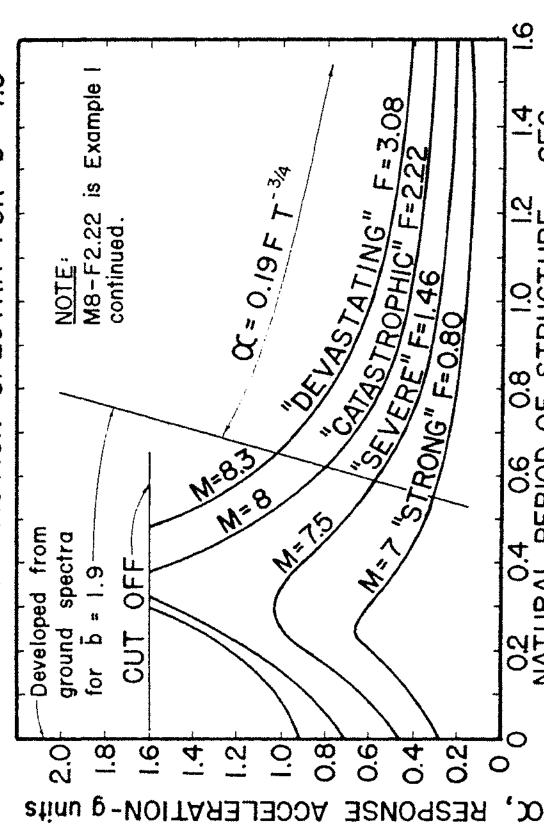


FIG.7 - ELASTIC RESPONSE SPECTRA FOR DAMPING 5% & $\bar{b}$ = 1.9

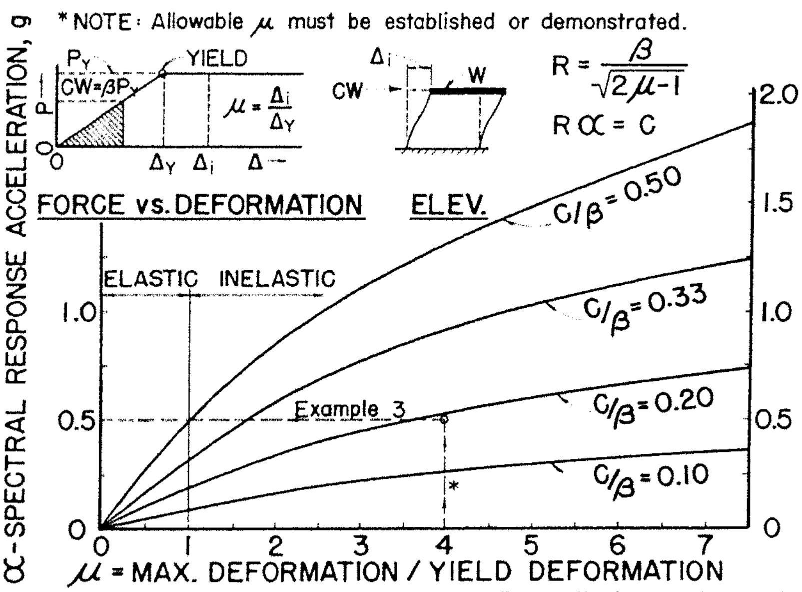


FIG. 8 - RESERVE ENERGY DATA FOR ELASTOPLASTIC SYSTEM

EARTHQUAKE GROUND-MOTION AND ENGINEERING PROCEDURES FOR IMPORTANT INSTALLATIONS NEAR ACTIVE FAULTS

BY J.A. BLUME

QUESTION BY: J.A. FISCHER - U.S.A.

In order to eliminate any possible future confusion I would like to comment upon Mr. Blume's references numbered 14 and 15. It might be possible to assume that these reports by Duke, Leeds, Matthiesen & Frazer could be used directly in design. The authors state the presented amplification spectra are predicated upon many assumptions as to site soil conditions and should not be used for design. In addition, no damping is included in the prepared spectra. Thus, additional investigation and analysis would be necessary before actual site design criteria are available.

AUTHOR'S REPLY: It was not intended to imply that the reports by Duke, Leeds, Matthiesen & Fraser could be used directly in design and it is appreciated that Mr. Fischer has emphasized this point. The peak response values obtained by these investigators are, of course, greater than indicated by the recordings of ground motion in Southern California. However, there are certain qualitative similarities between the recorded motion and the analytical response that are quite interesting. The corresponding material in the writer's paper is empirical and does not rely upon any idealized response calculations. It is hoped that as more numerical information becomes available on layering phenomena and on the effects of alluvial depth, additional parameters can be included in the determination of the site factor, $\bar{b}$. Although such additional parameters are not expected to necessarily or appreciably alter the numerical values in most cases, they would tend to increase reliability.

QUESTION BY: B.W. SPOONER - NEW ZEALAND

What distance has the author in mind when he uses the word "near" active faults? This question has importance in regard to existing cities and buildings and also in regard to the future use of areas which are "near" active faults. A definition of "active" faults is also necessary of course.

AUTHOR'S REPLY: The definition of "near" and "active fault" must necessarily be somewhat subjective and perhaps vary

under different situations, risks, sensitivity of installations, and other factors. The meaning of the work "near" in a specific case is also involved with the magnitude of the earthquake under consideration, and that in turn is a function of the design probability. Generally speaking, any distance less than 30 or 40 miles should be considered "near" for exposure to a major earthquake and in the context of this paper (See Figure 5). An "inactive fault" should be a fault that can be shown not to have moved differentially between its two faces or in its rift zone in a significant geological period. For example, if it could be proved by seismological and geological investigations that a fault had not moved for say 20,000 or more years, it could probably be classified "inactive". If this can not be proved, the fault should be considered as potentially "active". Recorded history is so short in this matter that it is almost meaningless as a criteria in determining activity or inactivity. Detailed field investigations are essential.