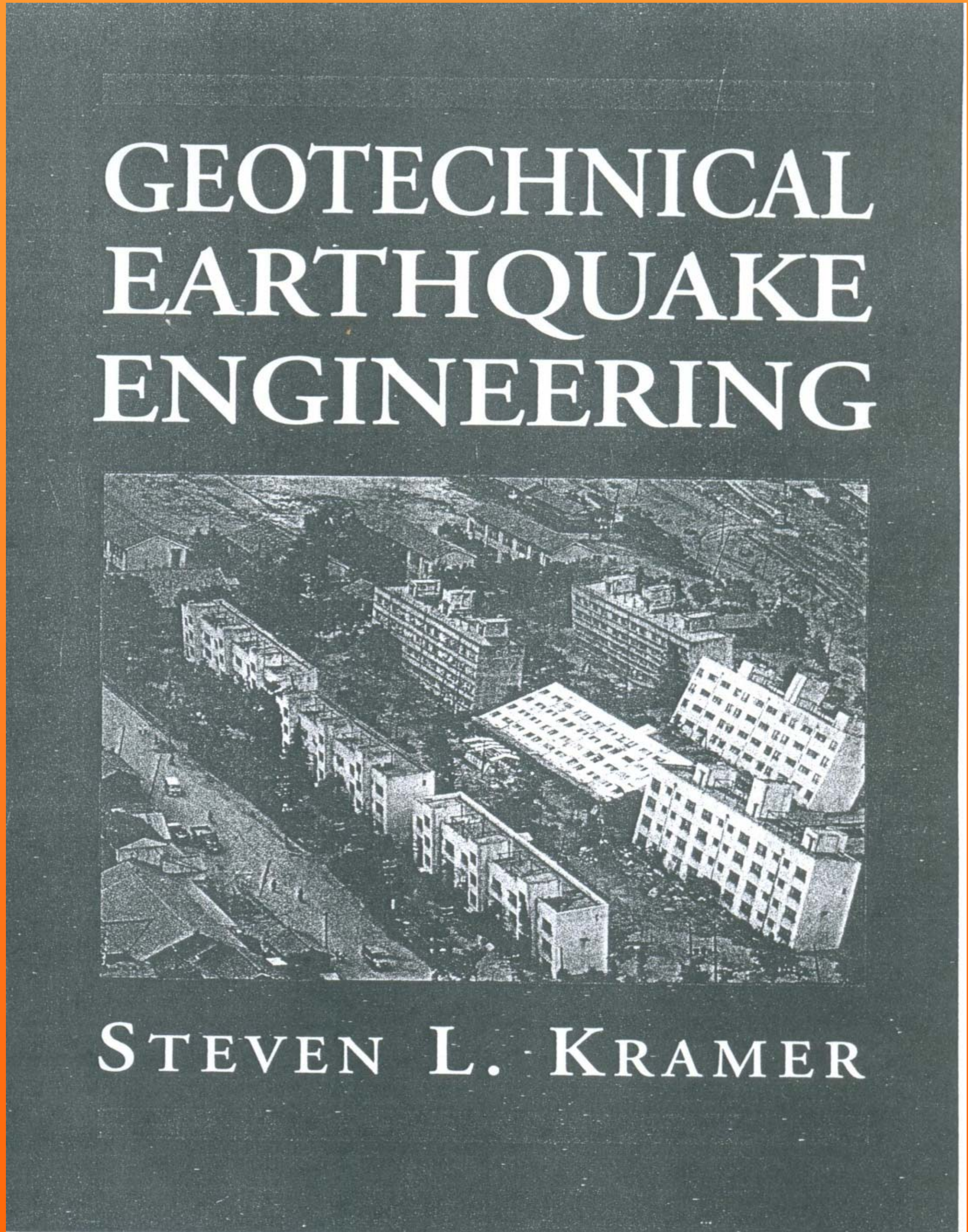
Reference:
ธนู หาญพัฒนพาณิชย์. 2554. การสำรวจทางธรณีวิทยาเพื่อประเมินตัวแปรแผ่นดินไหว. เอกสารประกอบการอบรม "การวิเคราะห์เพื่อออกแบบและประเมินความปลอดภัยเขื่อน", ระหว่างวันที่ 5,7 และ 8 เมษายน 2554, จัดโดย ศูนย์วิจัยและพัฒนาวิศวกรรมปฐพีและฐานราก มหาวิทยาลัยเกษตรศาสตร์ ร่วมกับ Thai Geotechnical Society (TGS), ณ โรงแรมมิราเคิล แกรนด์ คอนเวนชั่น, กรุงเทพฯ.

Seismic Hazard Analysis

General Knowledge


GEOTECHNICAL
EARTHQUAKE
ENGINEERING
STEVEN L. KRAMER

3.3.1.1 Peak Acceleration

The most commonly used measure of the amplitude of a particular ground motion is the peak horizontal acceleration (PHA). The PHA for a given component of motion is simply the largest (absolute) value of horizontal acceleration obtained from the accelerogram of that component. By taking the vector sum of two orthogonal components, the maximum resultant PHA (the direction of which will usually not coincide with either of the measured components) can be obtained.

Horizontal accelerations have commonly been used to describe ground motions because of their natural relationship to inertial forces; indeed, the largest dynamic forces induced in certain types of structures (i.e., very stiff structures) are closely related to the PHA. The PHA can also be correlated to earthquake intensity (e.g., Trifunac and Brady, 1975a; Murphy and O'Brien, 1977; Krinitzsky and Chang, 1987). Although this correlation is far from precise, it can be very useful for estimation of PHA when only intensity information is available, as in the cases of earthquakes that occurred before strong motion instruments were available (preinstrumental earthquakes). A number of intensity–acceleration relationships have been proposed, several of which are shown in Figure 3.11. The use of intensity–attenuation relationships also allows estimation of the spatial variability of peak acceleration from the isoseismal maps of historical earthquakes.

Vertical accelerations have received less attention in earthquake engineering than horizontal accelerations, primarily because the margins of safety against gravity-induced static vertical forces in constructed works usually provide adequate resistance to dynamic forces induced by vertical accelerations during earthquakes. For engineering purposes, the peak vertical acceleration (PVA) is often assumed to be two-thirds of the PHA (Newmark and Hall, 1982). The ratio of PVA to PHA, however, has more recently been observed to be quite variable but generally to be greater than two-thirds near the source of moderate to large earthquakes and less than two-thirds at large distances (Campbell, 1985; Abrahamson and Litehiser, 1989). Peak vertical accelerations can be quite large; a PVA of 1.74g was measured between the Imperial and Brawley faults in the 1979 Imperial Valley earthquake.

Ground motions with high peak accelerations are usually, but not always, more destructive than motions with lower peak accelerations. Very high peak accelerations that last for only a very short period of time may cause little damage to many types of structures. A number of earthquakes have produced peak accelerations in excess of 0.5g but caused no significant damage to structures because the peak accelerations occurred at very high frequencies and the duration of the earthquake was not long. Although peak acceleration is a very useful parameter, it provides no information on the frequency content or duration of the motion; consequently, it must be supplemented by additional information to characterize a ground motion accurately.

3.3.1.2 Peak Velocity

The peak horizontal velocity (PHV) is another useful parameter for characterization of ground motion amplitude. Since the velocity is less sensitive to the higher-frequency components of the ground motion, as illustrated in Figure 3.10, the PHV is more likely than the PHA to characterize ground motion amplitude accurately at intermediate frequencies. For structures or facilities that are sensitive to loading in this intermediate-frequency range (e.g., tall or flexible buildings, bridges, etc.), the PHV may provide a much more accurate indication of the potential for damage than the PHA. PHV has also been correlated to earthquake intensity (e.g., Trifunac and Brady, 1975a; Krinitzsky and Chang, 1987).

3.3.1.3 Peak Displacement

Peak displacements are generally associated with the lower-frequency components of an earthquake motion. They are, however, often difficult to determine accurately (Campbell, 1985; Joyner and Boore, 1988), due to signal processing errors in the filtering and integration of accelerograms and due to long-period noise. As a result, peak displacement is less commonly used as a measure of ground motion than is peak acceleration or peak velocity.

Example 3.1

PHA

PVA

PHV

PHD

B.7 RESPONSE SPECTRA

For earthquake-resistant design, the entire time history of response may not be required. Instead, earthquake-resistant design may be based on the maximum (absolute) value of the response of a structure to a particular base motion. Obviously, the response will depend on the mass, stiffness, and damping characteristics of the structure and on the characteristics of the base motion.

The *response spectrum* describes the maximum response of a single-degree-of-freedom (SDOF) system to a particular input motion as a function of the natural frequency (or natural period) and damping ratio of the SDOF system (Figure B.19). The response may be expressed in terms of acceleration, velocity, or displacement. The maximum values of each of these parameters depend only on the natural frequency and damping ratio of the SDOF

SA

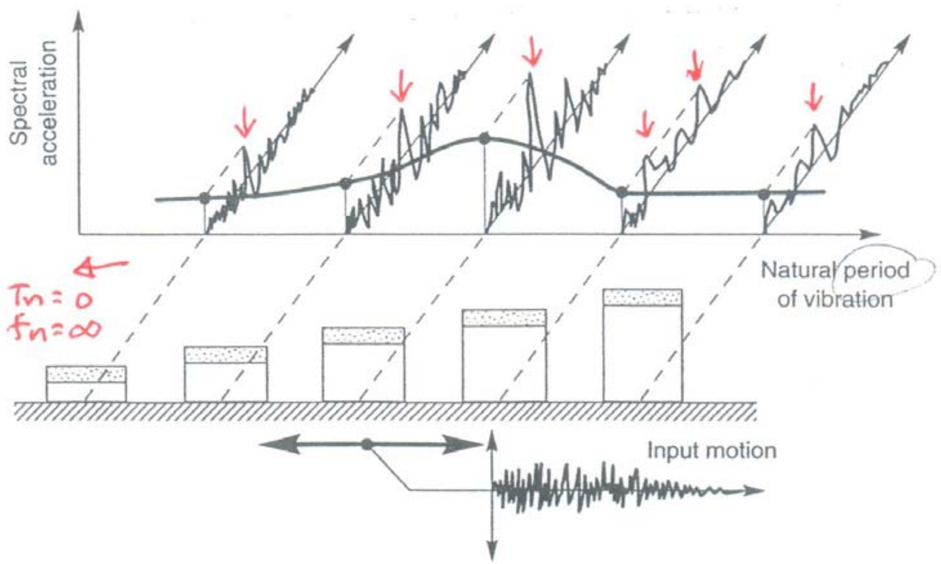


Figure B.19 Response specrum. Spectral accelerations are the maximum acceleration amplitudes of SDOF systems in response to the same input motion. The response system is obtained by plotting the spectral accelerations against the periods of vibrations of the SDOF systems.

system (for a particular input motion). The maximum values of acceleration, velocity, and displacement are referred to as the spectral acceleration (S_a), spectral velocity (S_v), and spectral displacement (S_d), respectively. Note that a SDOF system of zero natural period (infinite natural frequency) would be rigid, and its spectral acceleration would be equal to the peak ground acceleration. ✗ ก $T_n = 0$ หรือ $f_n = \infty$

Sa(T=0) = PGA

B.4 EQUATION OF MOTION FOR SDOF SYSTEM

Many SDOF systems are acted upon by externally applied loads. In earthquake engineering, dynamic loading often results from another source—movement of the supports of the system. The dynamic response of a SDOF system such as that shown in Figure B.3 is governed by an *equation of motion*. The equation of motion can be derived in a number of ways; a simple, force equilibrium approach will be used here.

B.4.1 Equation of Motion: External Loading

When a dynamic load is applied to the mass of a SDOF system (Figure B.3), the tendency for motion is resisted by the inertia of the mass and by forces that develop in the dashpot and spring. Thus the external load, $Q(t)$, acting in the positive x-direction is opposed by three forces (Figure B.4) that act in the negative x-direction: the *inertial force*, f_I, the *viscous damping force*, f_D, and the *elastic spring force*, f_S. The equation of motion can be expressed in terms of the dynamic equilibrium of these forces:

$$f_I(t) + f_D(t) + f_S(t) = Q(t) \tag{B.1}$$

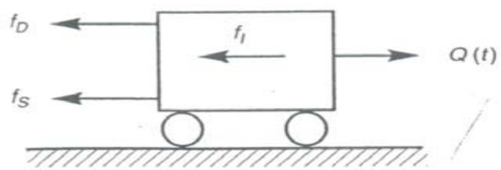


Figure B.4 Dynamic forces acting on mass from Figure B.3.

These forces can also be expressed in terms of the motion of the mass. Newton's second law states that the inertial force acting on a mass is equal to its rate of change of momentum, which for a system of constant mass produces

$$f_I(t) = \frac{d}{dt}\left(m\frac{du(t)}{dt}\right) = m\frac{d^2u(t)}{dt^2} = m\ddot{u}(t) \tag{B.2a}$$

For a viscous dashpot, the damping force is proportional to the velocity of the mass:

$$f_D(t) = c\frac{du(t)}{dt} = c\dot{u}(t) \tag{B.2b}$$

and the force provided by the spring is simply the product of its stiffness and the amount by which it is displaced

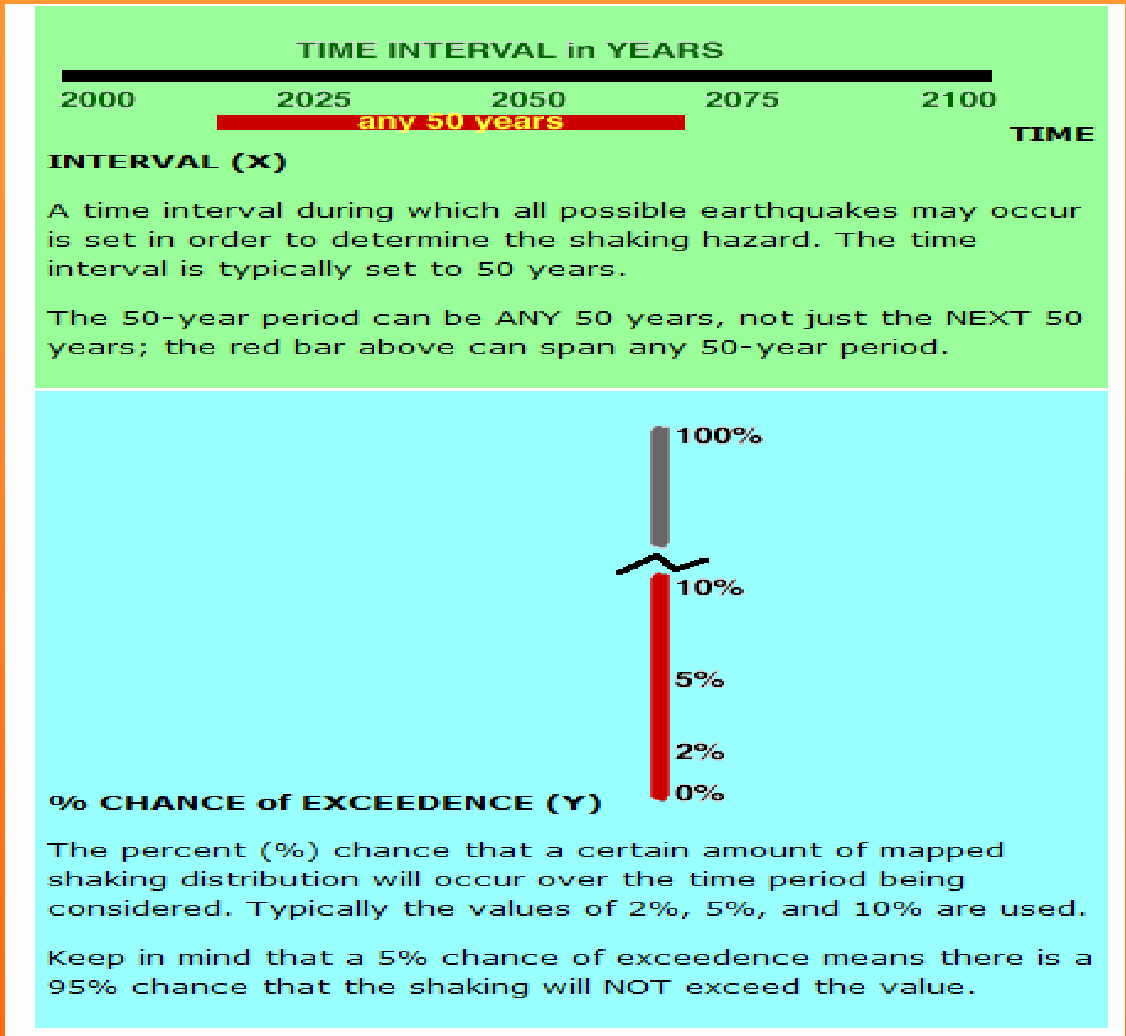
$$f_S(t) = ku(t) \tag{B.2c}$$

The behavior of these forces is illustrated graphically in Figure B.5. The inertial force is proportional to the acceleration and the constant of proportionality is the mass. Similarly, the viscous damping force and the elastic spring force are proportional to the velocity and displacement with the damping and spring coefficients serving as the respective constants of proportionality.

Substituting equations (B.2) into equation (B.1), the equation of motion for the SDOF system can be written as

$$m\ddot{u}(t) + c\dot{u}(t) + ku(t) = Q(t) \tag{B.3}$$

"With 10% Probability of Exceedance in 50 years"



http://earthquake.usgs.gov/researsh/hazmaps/haz101

If :Earthquake is Poisson Process

$PE = 1 - e^{(-rate \times time)}$ **OR** $rate = \dfrac{-(\ln(1-PE))}{time}$

eg. PE=10%, time=50

$rate = \dfrac{-(\ln(1.0 - 0.1))}{50}$

$= 0.0021$ per year

Return period = 1/rate

RT = 1/0.0021 = 476.2 year USED = **500** year

กรมชลประทาน
กระทรวงเกษตรและสหกรณ์

บริษัท ปัญญา คอนซัลแตนท์ จำกัด

EM 1110-2-6050
30 June 1999

US Army Corps
of Engineers
ENGINEERING AND DESIGN

Response Spectra and Seismic Analysis for Concrete Hydraulic Structures

EM 1110-2-6050
30 Jun 99

Table 3-4
Relationship Between Return Period and Probability of Exceedance for Different Time Periods

Probability of Exceedance, %	Return Period, Years, for Different Design Time Periods *t*					
	t = 10 years	*t* = 20 years	*t* = 30 years	*t* = 40 years	*t* = 50 years	*t* = 100 years
1	995	1,990	2,985	3,980	4,975	9,950
2	495	990	1,485	1,980	2,475	4,950
5	195	390	585	780	975	1,950
10	95	190	285	380	475	950
20	45	90	135	180	225	450
30	28	56	84	112	140	280
40	20	39	59	78	98	195
50	14	29	43	58	72	145
60	11	22	33	44	55	110
70	8.3	17	25	33	42	83
80	6.2	12	19	25	31	62
90	4.3	8.7	13	17	22	43
95	3.3	6.7	10	13	17	33
99	2.2	4.3	6.5	8.7	11	22
99.5	1.9	3.8	5.7	7.5	9.4	19

Hazard Level Selection Guide

Building Research Establishment Report

CI/SfB 187(H16)

1991

An engineering guide to seismic risk to dams in the United Kingdom

bre

4.2 Risk classification

It is proposed that risk classification should follow ICOLD Bulletin 72[7] as indicated in Tables 2 and 3. Terminology has been amended for clarity when considered appropriate for this Guide.

Table 2 Classification factors

	Classification factor			
Capacity ($10^6 m^3$)	> 120 (6)	120–1 (4)	1–0.1 (2)	<0.1 (0)
Height (m)	> 45 (6)	45–30 (4)	30–15 (2)	<15 (0)
Evacuation requirements (No of persons)	> 1000 (12)	1000–100 (8)	100–1 (4)	None (0)
Potential downstream damage	High (12)	Moderate (8)	Low (4)	None (0)

The weighting points of each of the four classification factors, shown in parentheses in Table 2, are summed to provide the total classification factor; ie:

Total classification factor =
classification factor (capacity)
+ classification factor (height)
+ classification factor (evacuation requirements)
+ classification factor (potential downstream damage)

Table 3 Dam category

Total classification factor	Dam category	
(0–6)	I	Low
(7–18)	II	Moderate
(19–30)	III	High
(31–36)	IV	Extreme

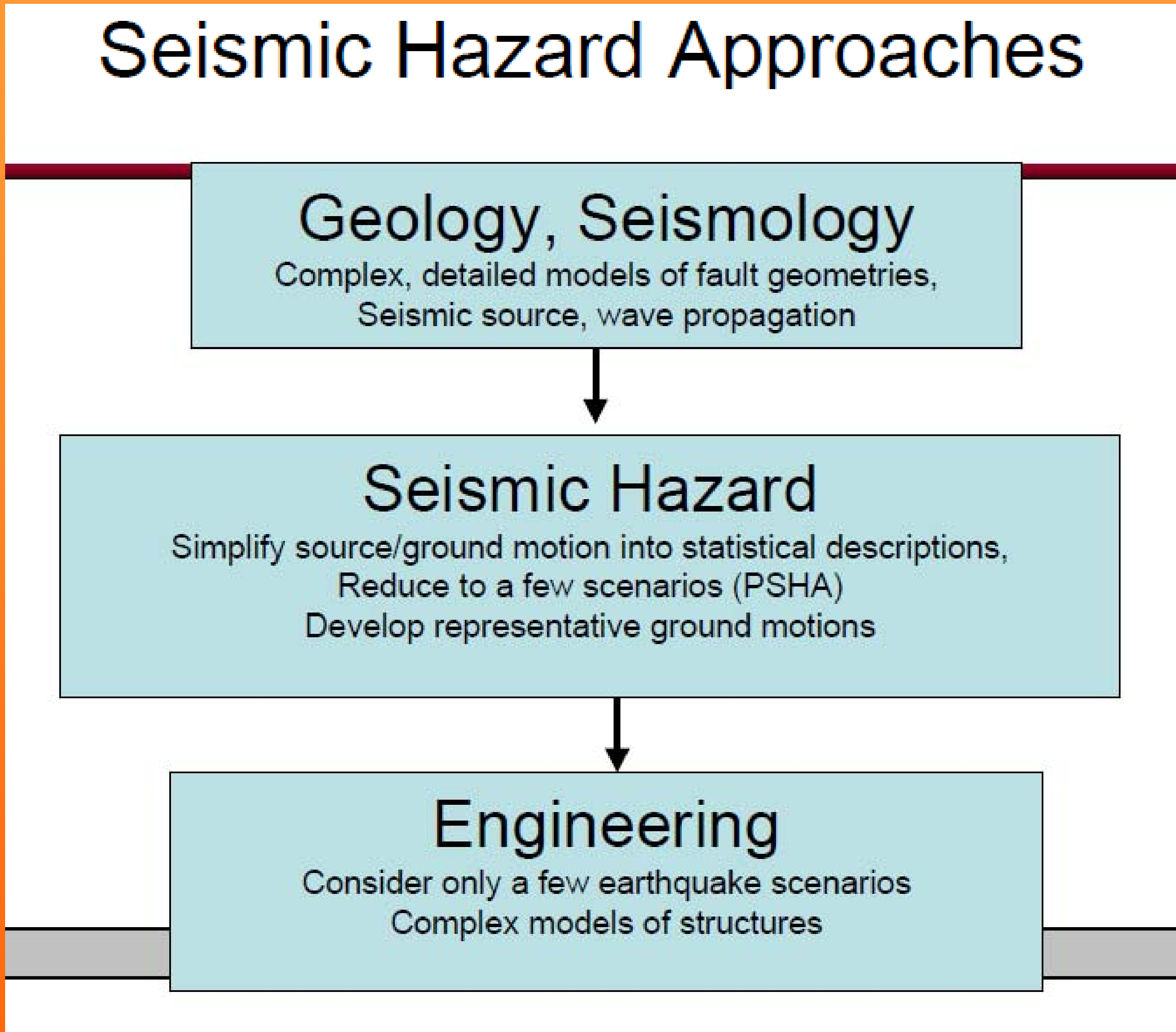
Table 4 Return period (years) and peak ground accelerations for SEE

Dam category	Return period	PGA		
		Zone A	Zone B	Zone C
IV*	30 000	0.375 *g*	0.30 *g*	0.25 *g*
III	10 000	0.25 *g*	0.20 *g*	0.15 *g*
II	3000	0.15 *g*	0.125 *g*	0.10 *g*
I†	1000	0.10 *g*	0.075 *g*	0.05 *g*

* In many category IV situations it may be considered desirable to use the maximum credible earthquake calculated from a regional geological and seismological survey.

† For category I situations seismic safety evaluation is not generally considered to be necessary (Sections 7.7, 7.10, 10.7 and 10.10).

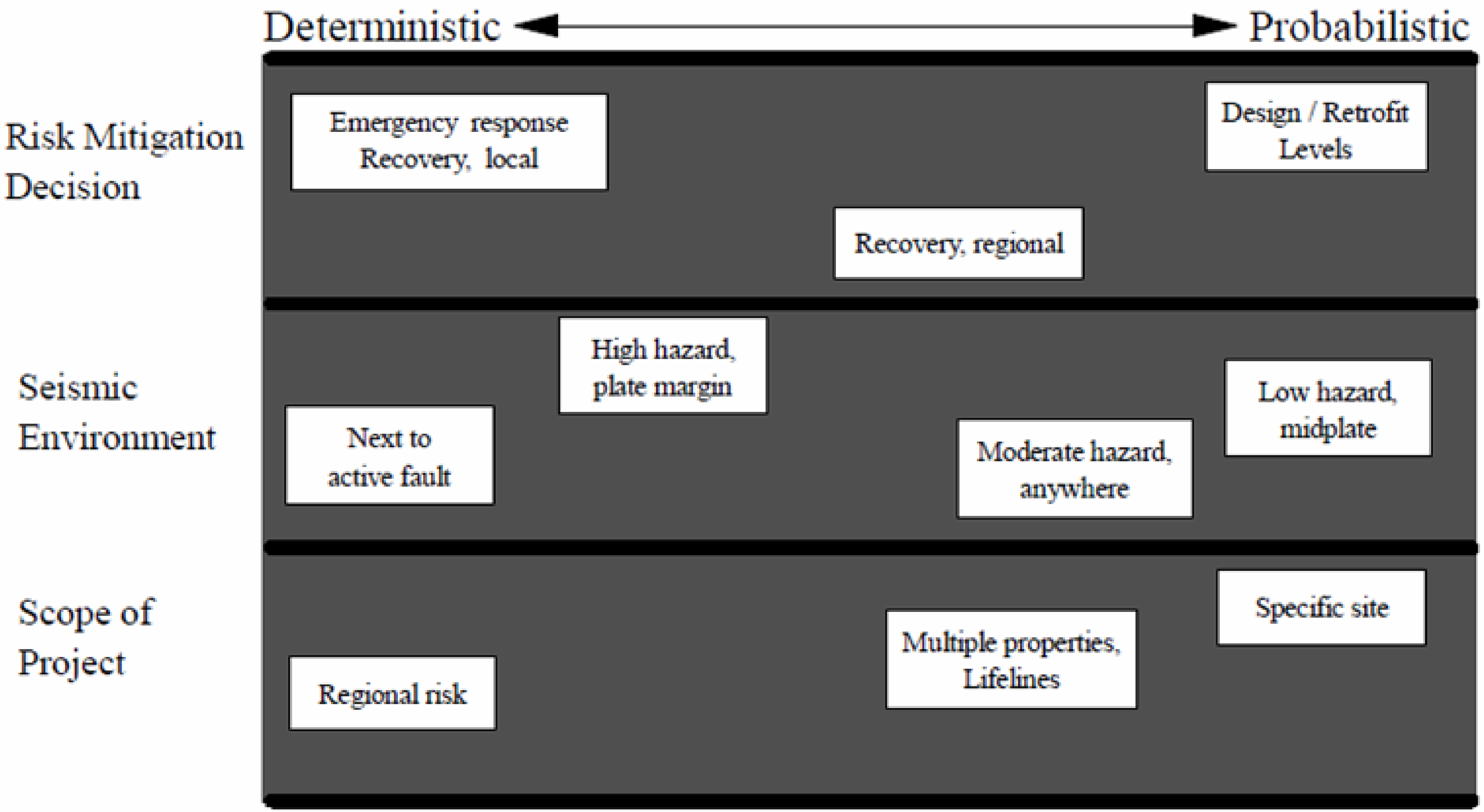
- **Zone A** — relatively high number of events, especially larger ones (ie $M_L > 4.5$)
- **Zone B** — moderate chance of local earthquakes, but larger events rare
- **Zone C** — few or no recorded earthquakes but some events possible (eg Colchester 1884).


Seismic Hazard Approaches
Geology, Seismology
Complex, detailed models of fault geometries,
Seismic source, wave propagation
Seismic Hazard
Simplify source/ground motion into statistical descriptions,
Reduce to a few scenarios (PSHA)
Develop representative ground motions
Engineering
Consider only a few earthquake scenarios
Complex models of structures

Seismic Hazard Approaches

- Deterministic approach
  - Rare earthquake selected
  - Median or 84th percentile ground motion
- Probabilistic approach
  - Probability of ground motion selected
    - Return period defines rare
- Performance approach
  - Probability of damage states of structure
    - Structural fragility needed
- Risk approach
  - Probability of consequence
    - Loss of life
    - Dollars

Dominance of deterministic/probabilistic analysis



R.K. McGuire, Risk Engineering Inc.,2002

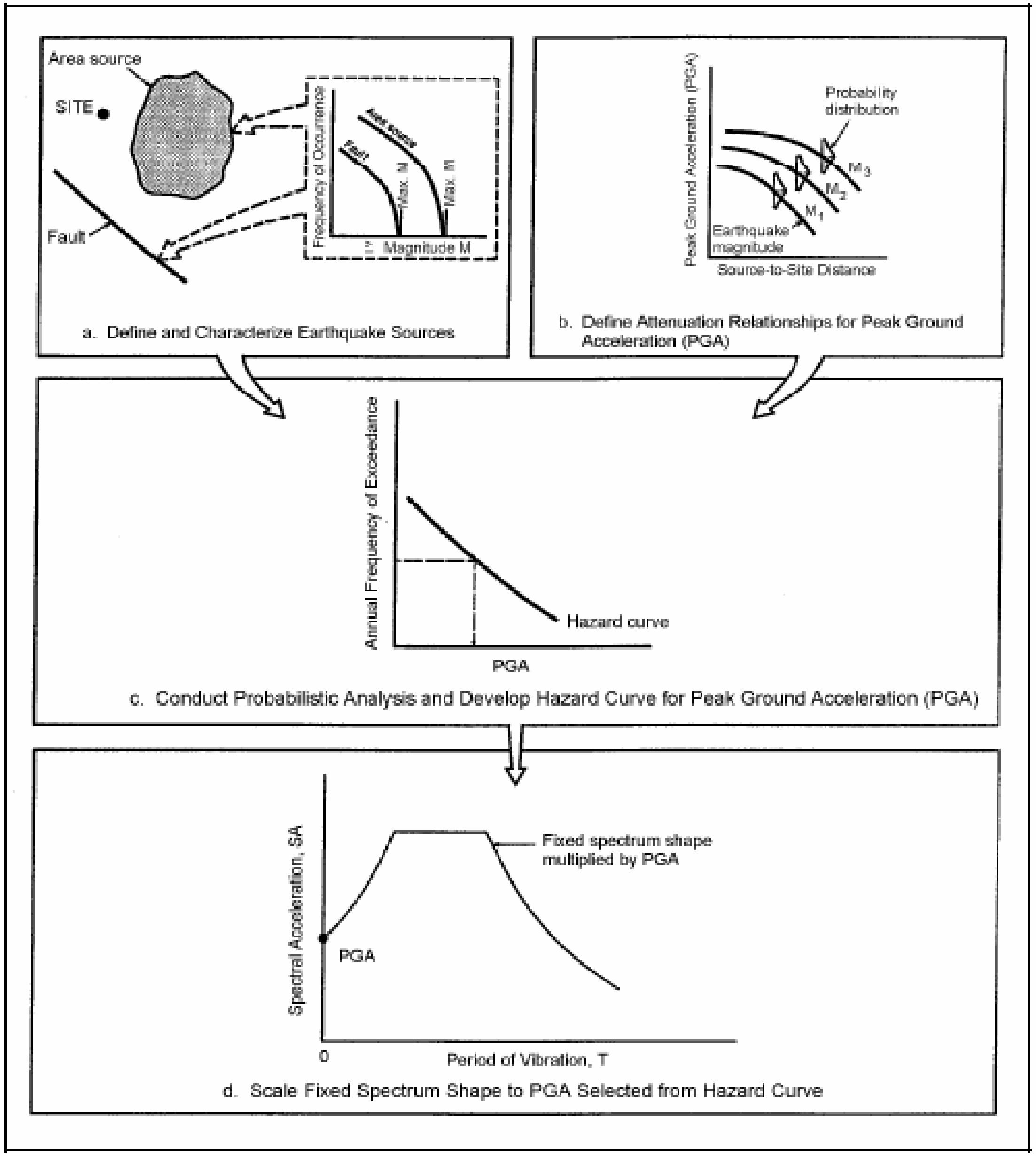


Figure 3-21. Development of response spectrum based on a fixed spectrum shape and a probabilistic seismic hazard analysis for peak ground acceleration

USACE- EM1110-2-6050, 1999

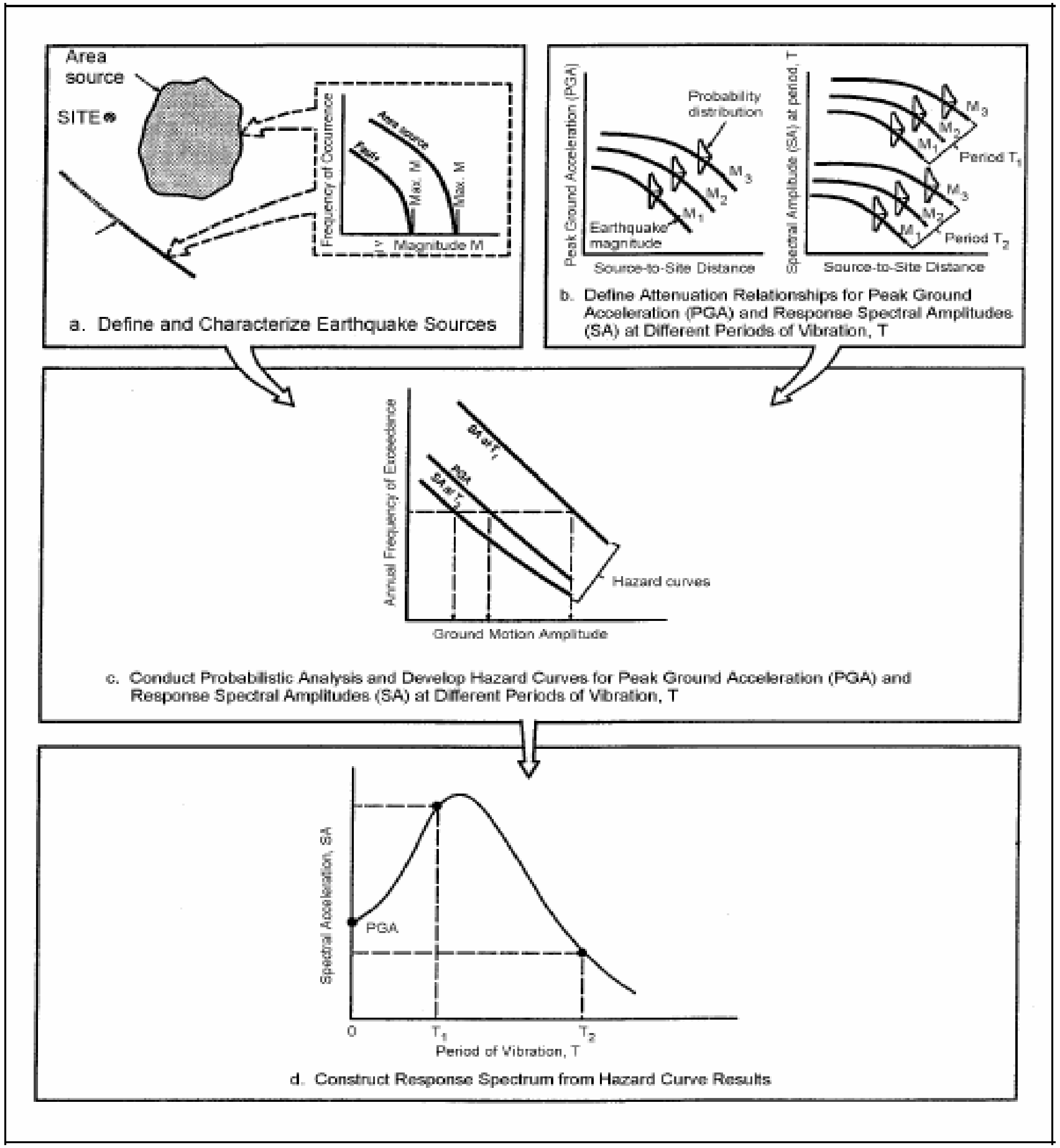


Figure 3-22. Development of equal-hazard response spectrum from probabilistic seismic hazard analysis for response spectral values

USACE- EM1110-2-6050, 1999

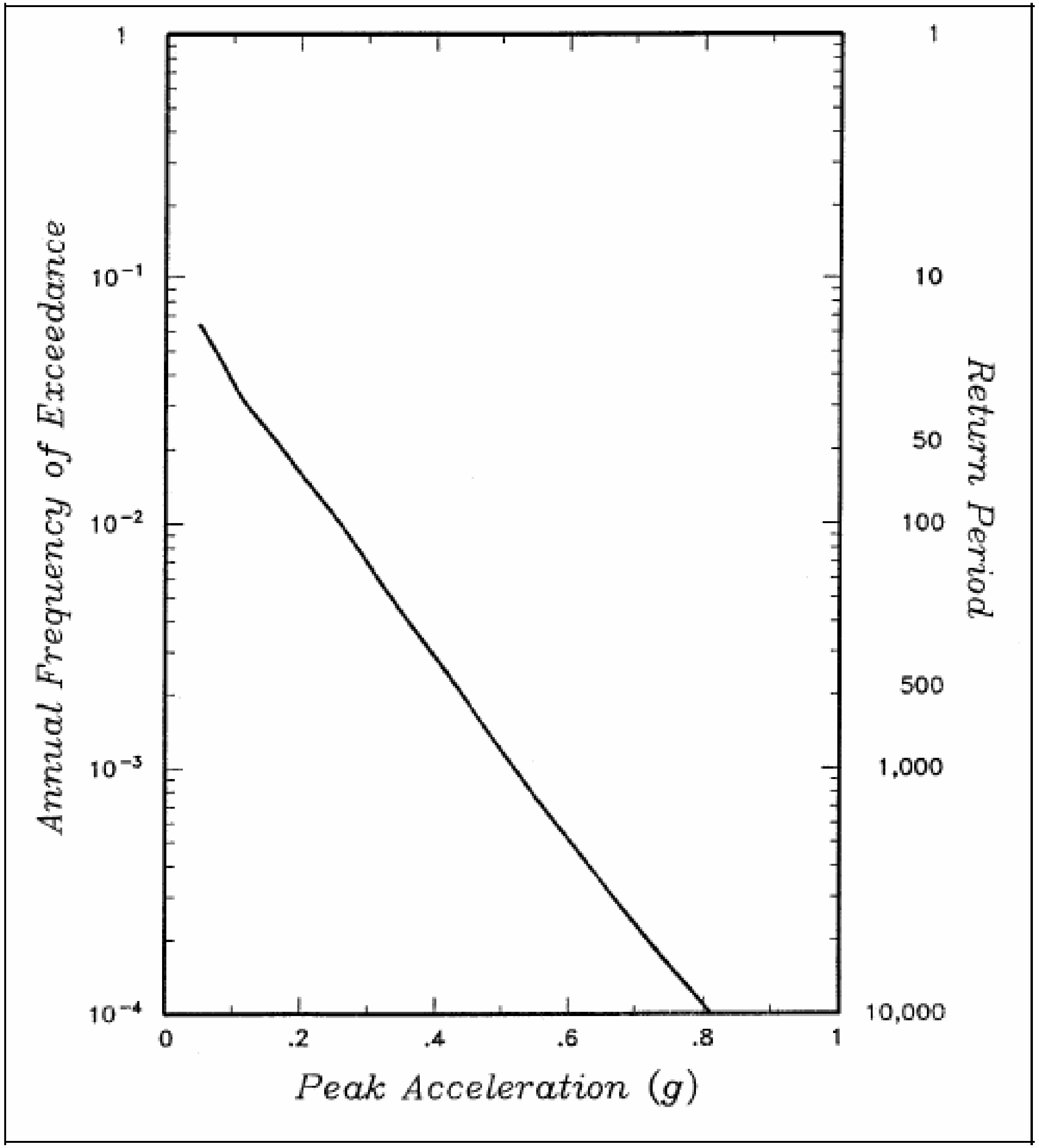
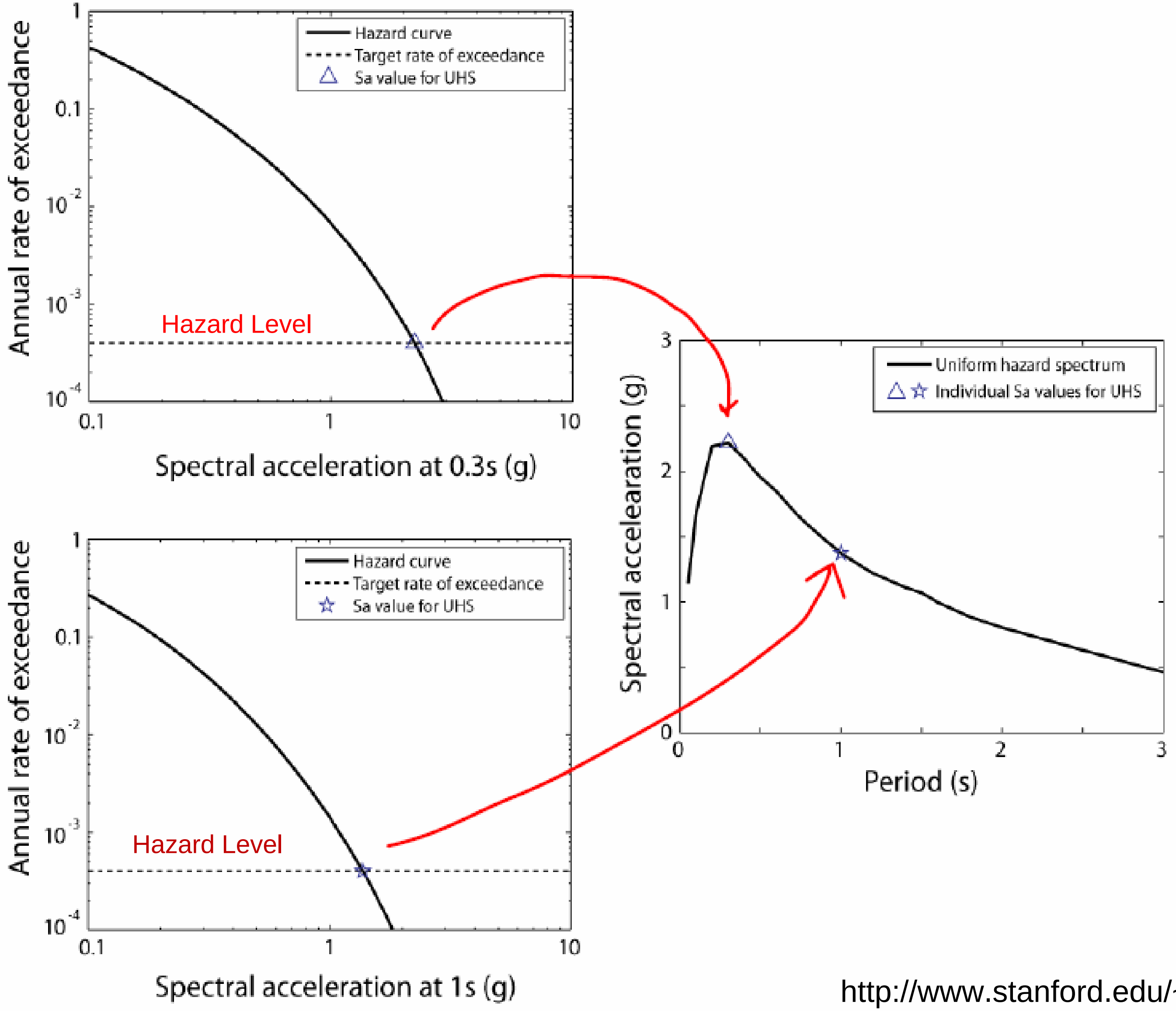


Figure 3-20. Example seismic hazard curve showing relationship between peak ground acceleration and probability (annual frequency) of exceedance

USACE- EM1110-2-6050, 1999

The Uniform Hazard Spectrum (UHS)



http://www.stanford.edu/~bakerjw

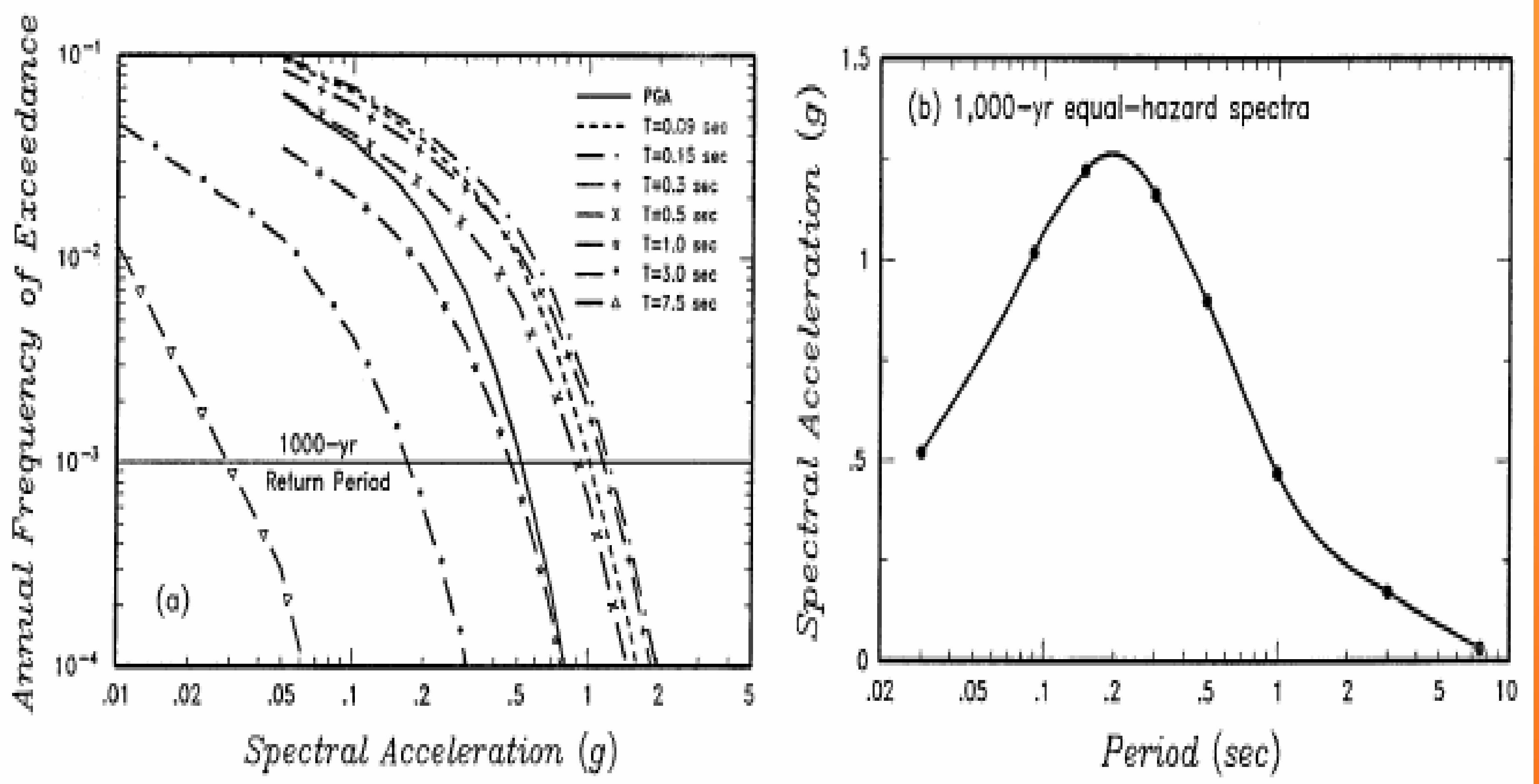


Figure 3-31. Construction of equal-hazard spectra. Top plot (a) shows hazard curves for a range of spectral periods. Bottom plot (b) shows equal hazard spectrum for a period of 1,000 years

USACE EM-1110-2-6050 , 1999

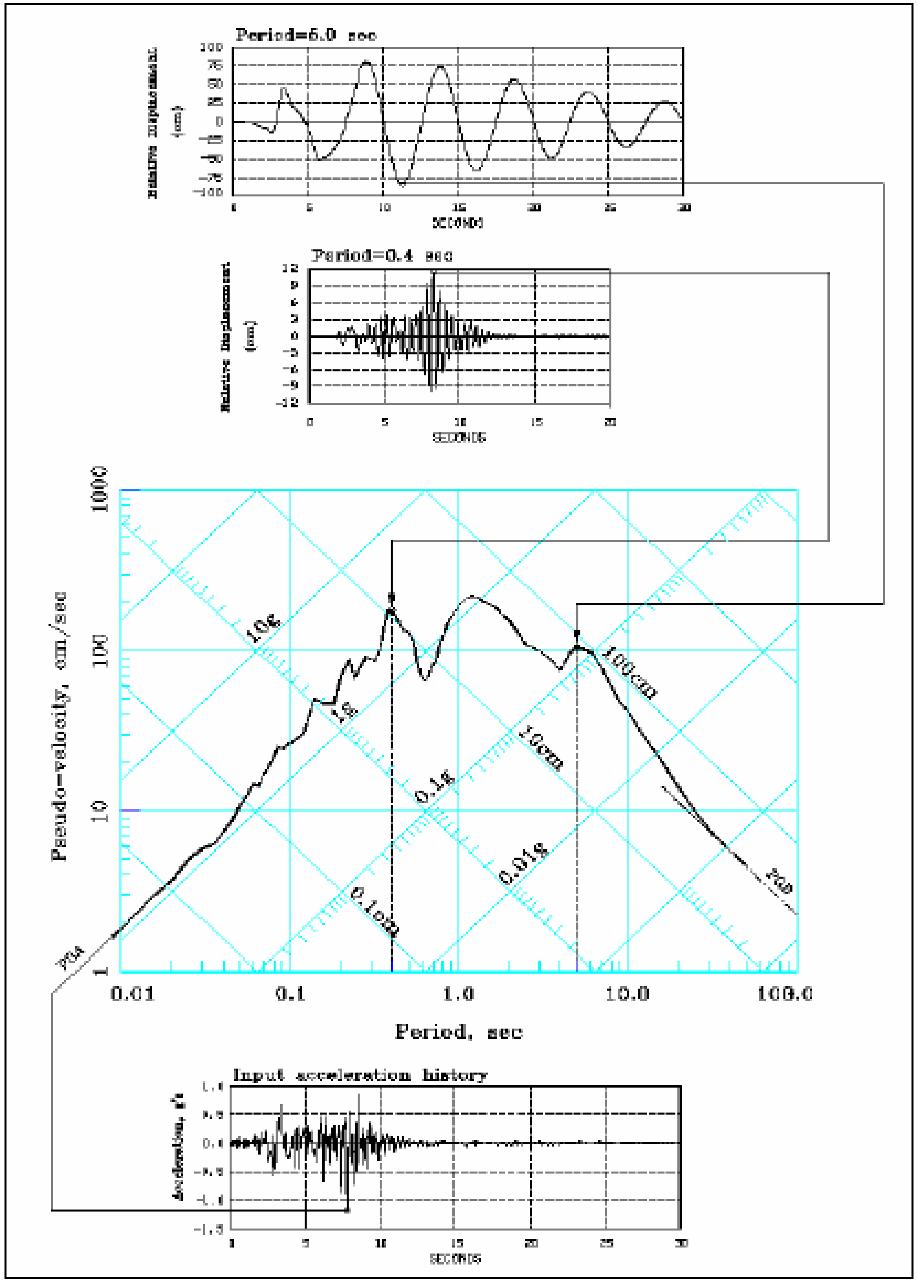


Figure 2-2. Construction of tripartite elastic design response spectrum

USACE- EM1110-2-6050, 1999

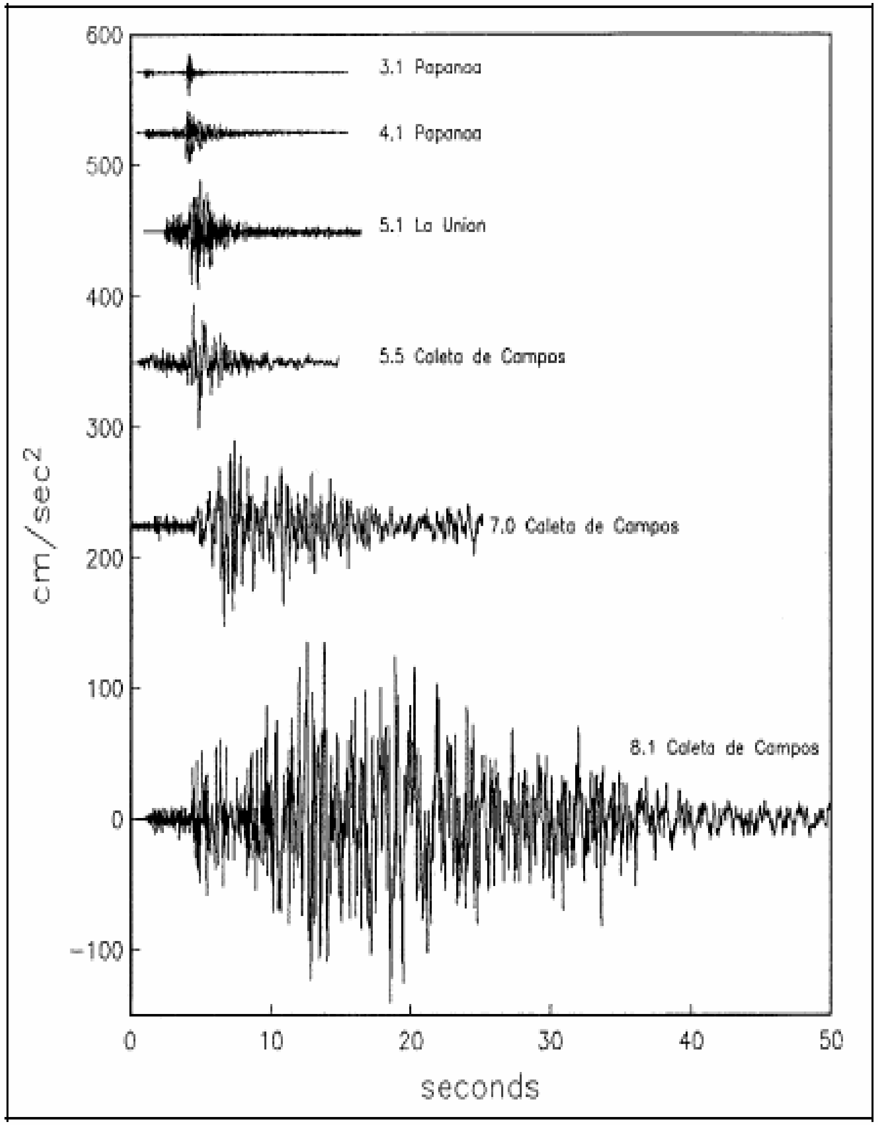


Figure 3-1. An example of accelerograms recorded in 1985 and 1986 on the Guerrero accelerograph array (Anderson and Quaas (1988), courtesy of Earthquake Engineering Research Institute, Oakland, CA)

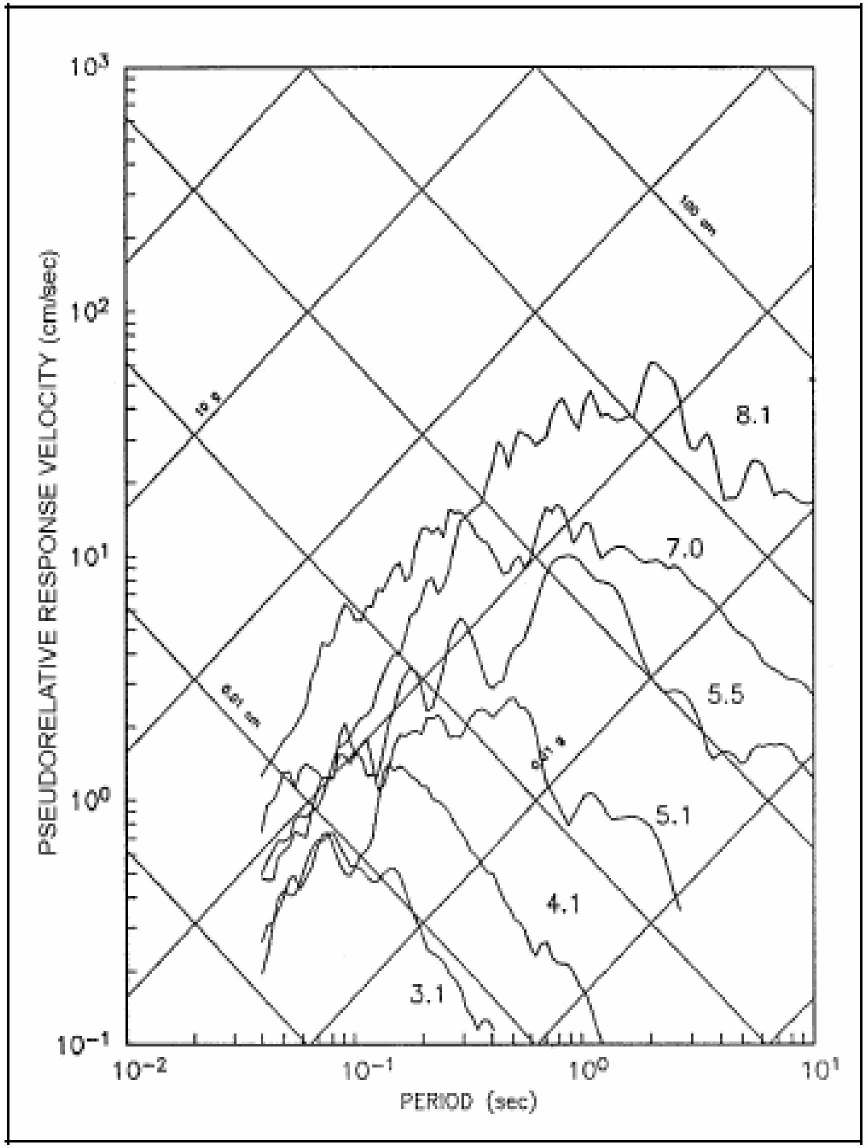
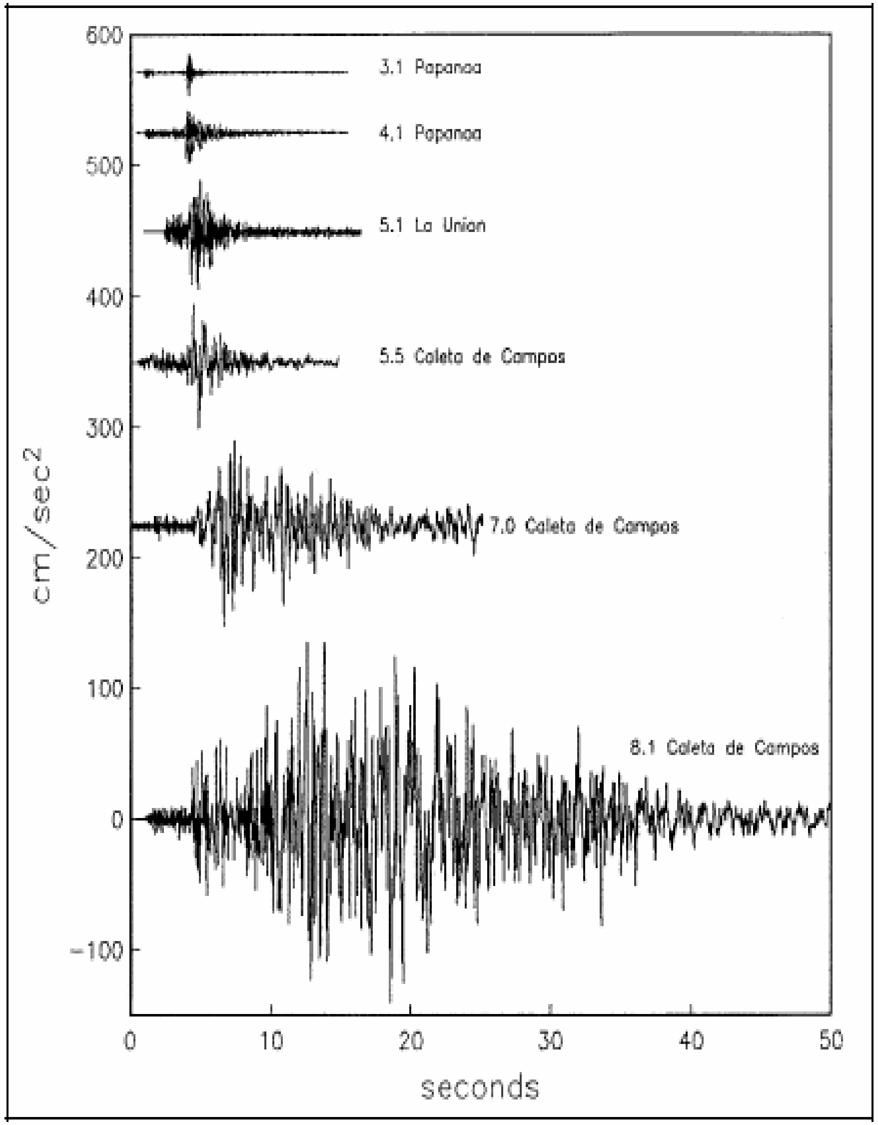


Figure 3-2. Response spectra (5 percent damped, pseudo-relative velocity) corresponding to the acceleration traces in Figure 3-1 (Anderson and Quaas (1988), courtesy of Earthquake Engineering Research Institute, Oakland, CA)



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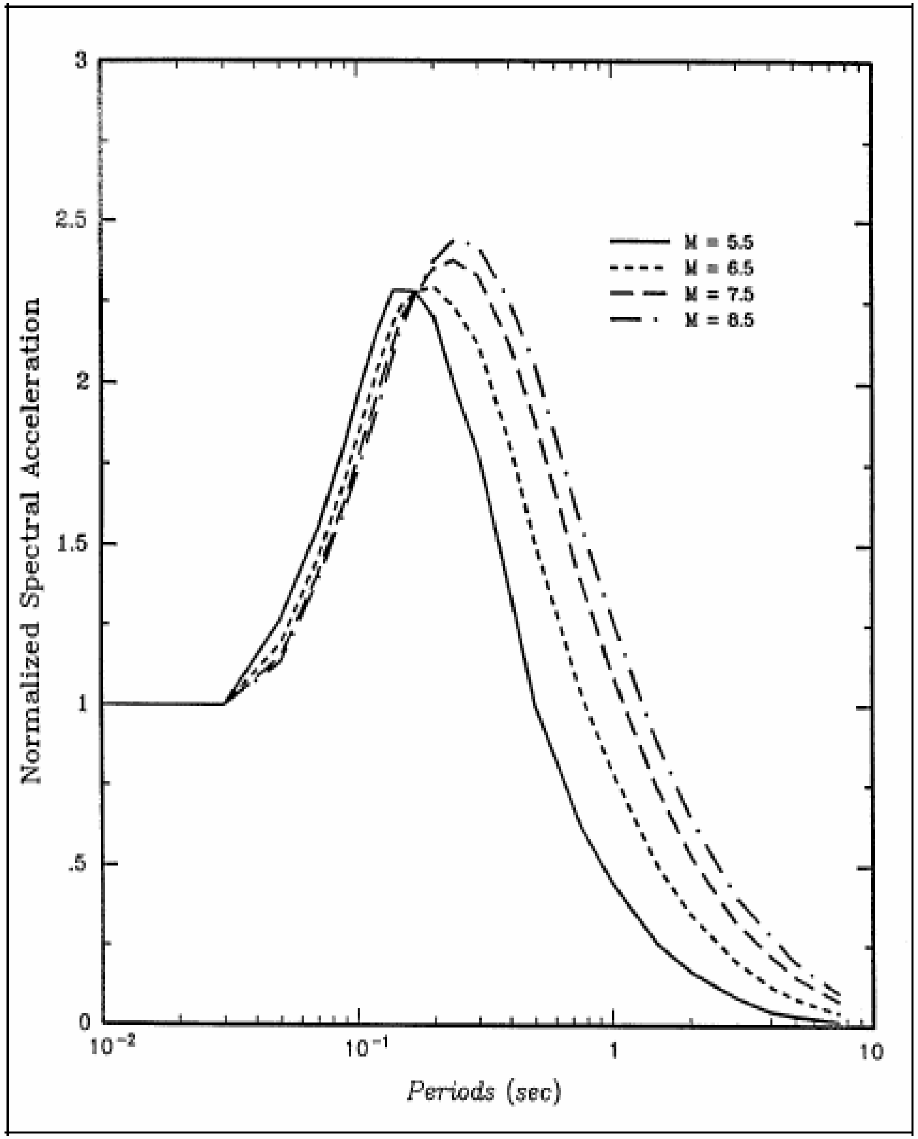

Figure 3-3. Effect of magnitude *M* on response spectral shape of rock motions based on attenuation relationships of Sadigh et al. (1993), 30-km distance from source to site, 5 percent damping

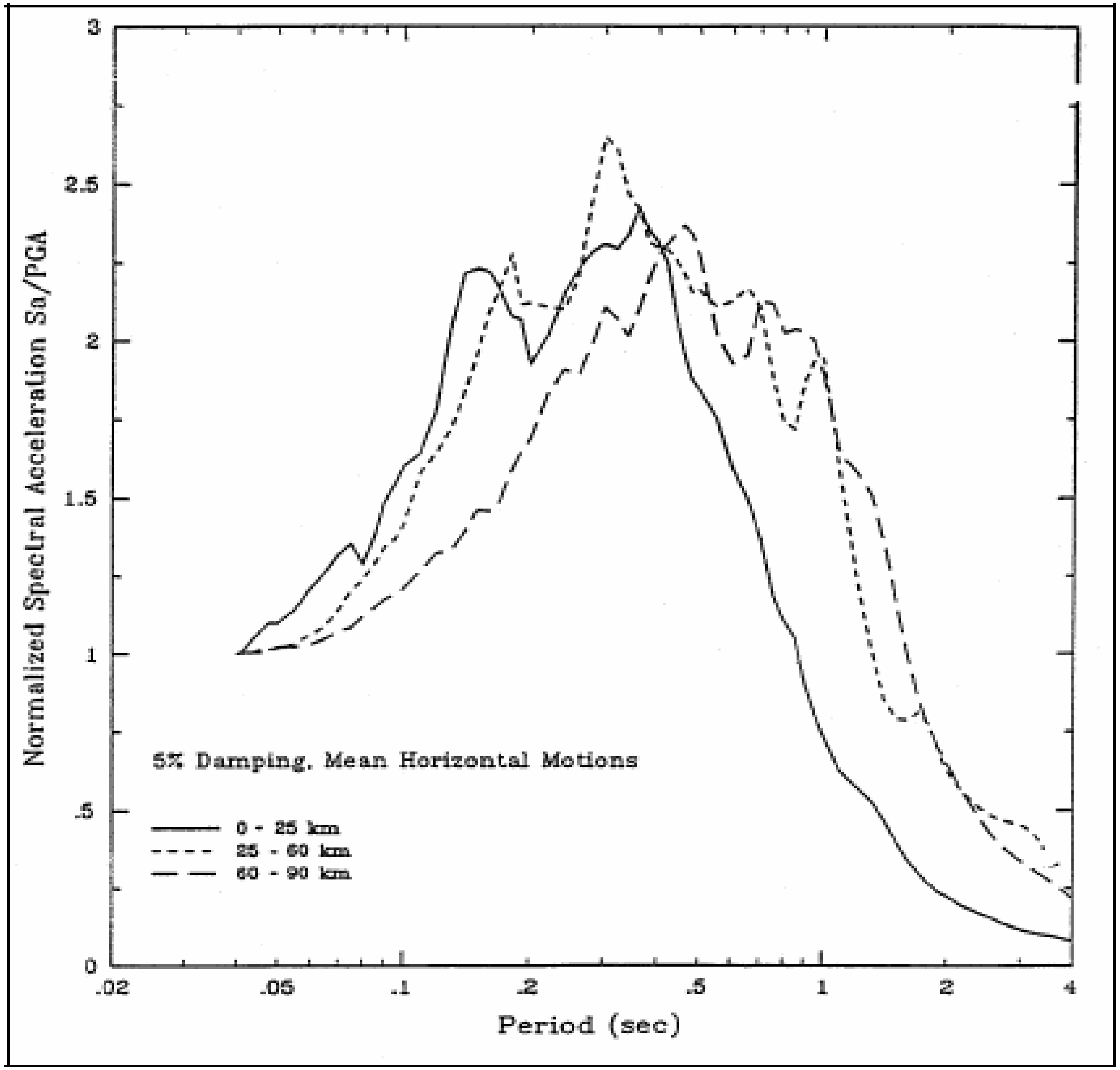

Figure 3-4. Variation of spectral shape with distance for rock recordings of the October 17, 1989, Loma Prieta earthquake

USACE- EM1110-2-6050, 1999

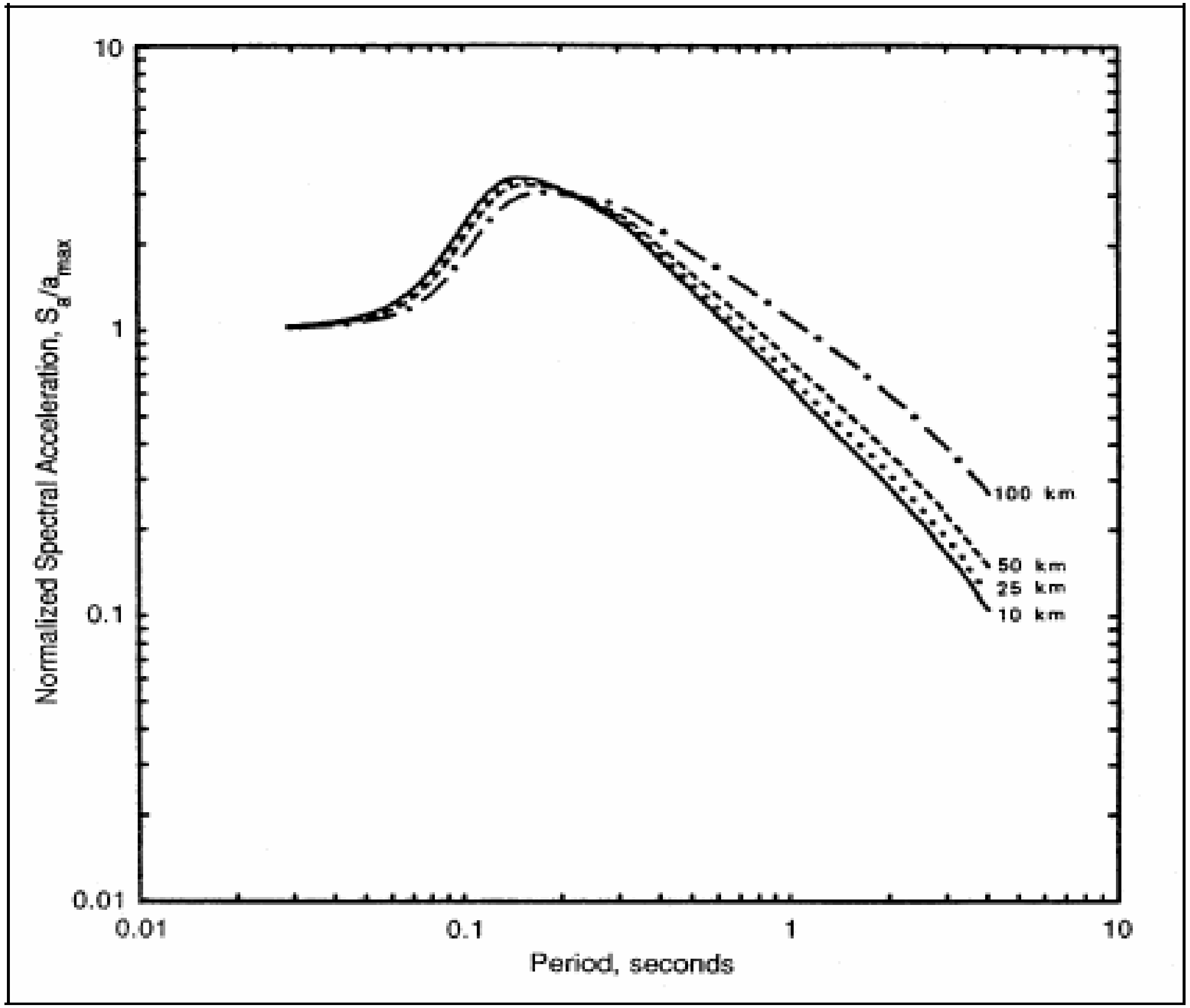

Figure 3-5. Effect of distance on response spectral shapes for a moment magnitude M_w 6.5 earthquake using western North American parameters (Silva and Green 1989, courtesy of Earthquake Engineering Research Institute, Oakland, CA)

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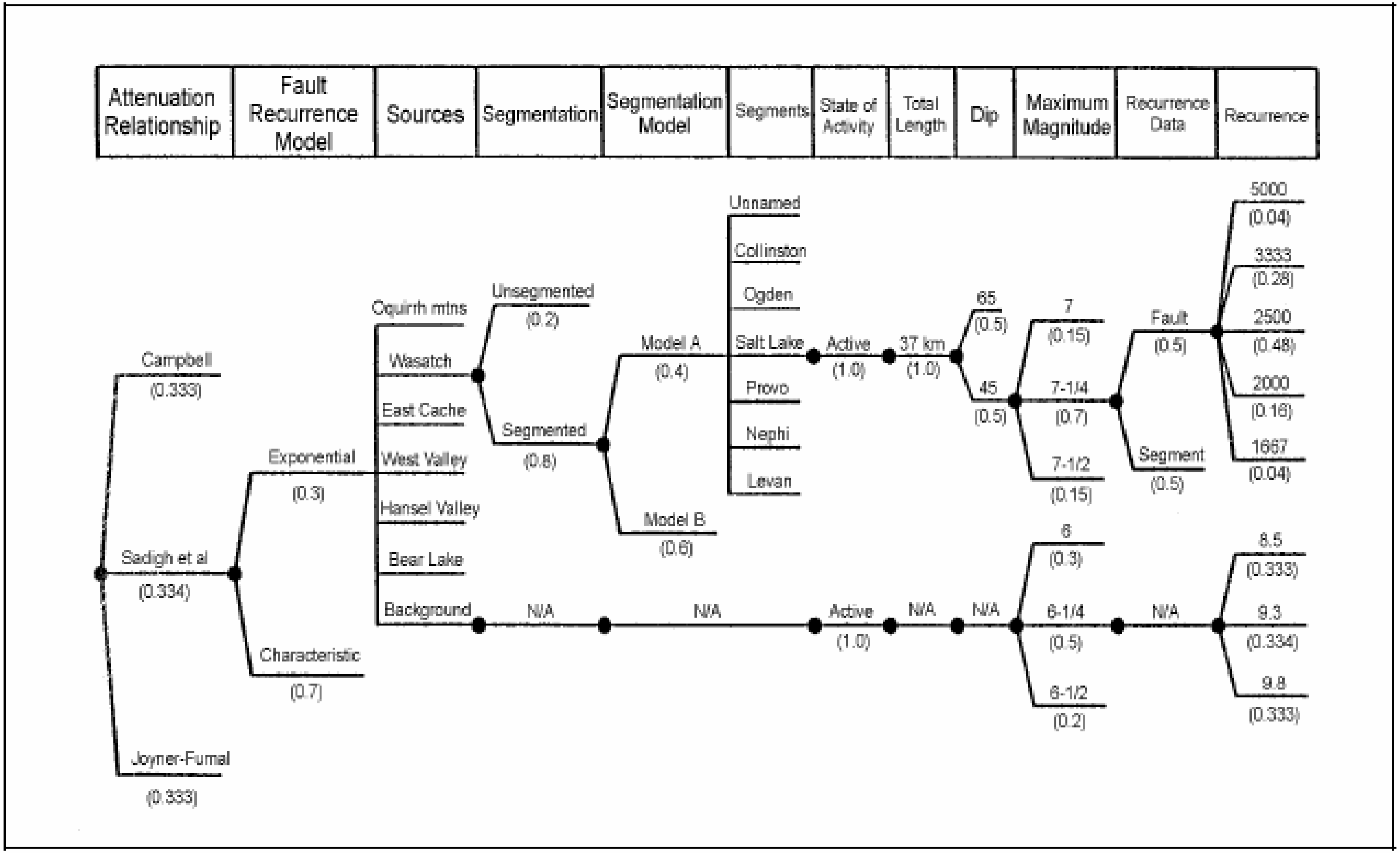

Figure 3-29. Example of logic tree for characterizing uncertainty in seismic hazard input (Youngs, Swan, and Power 1988, reprinted by permission of ASCE)

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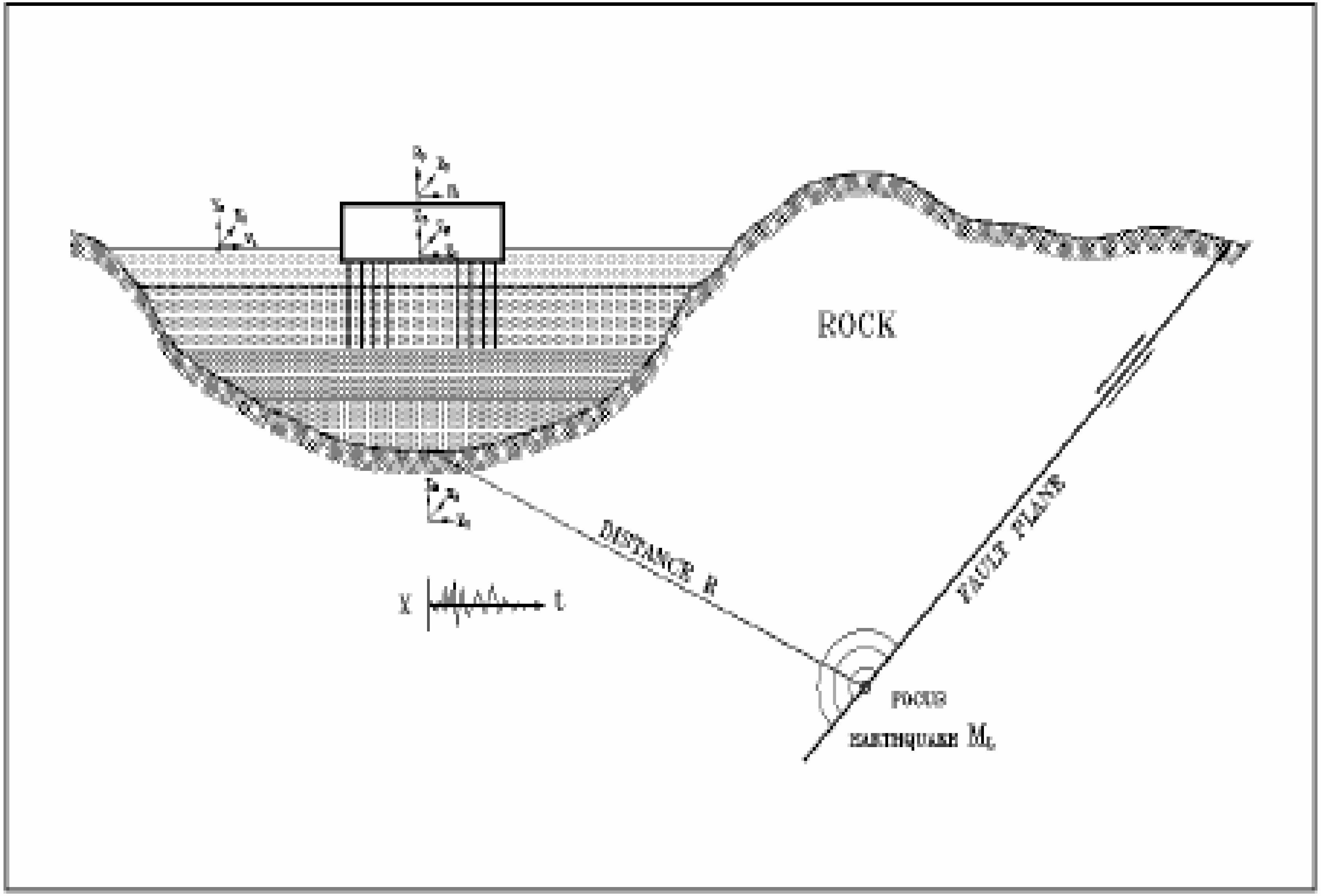
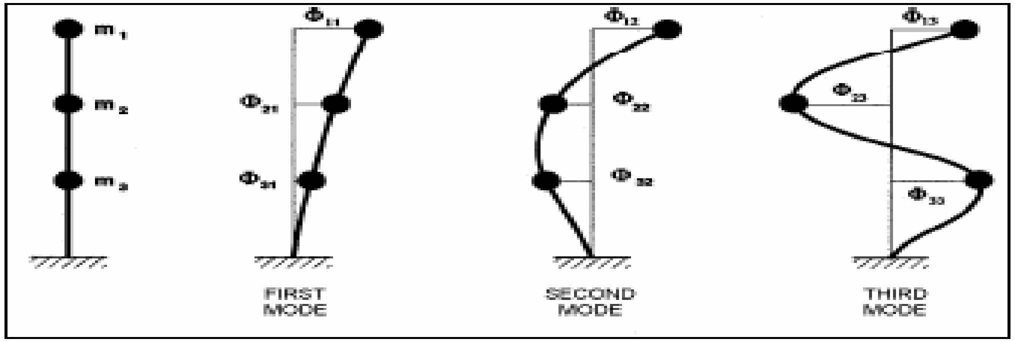


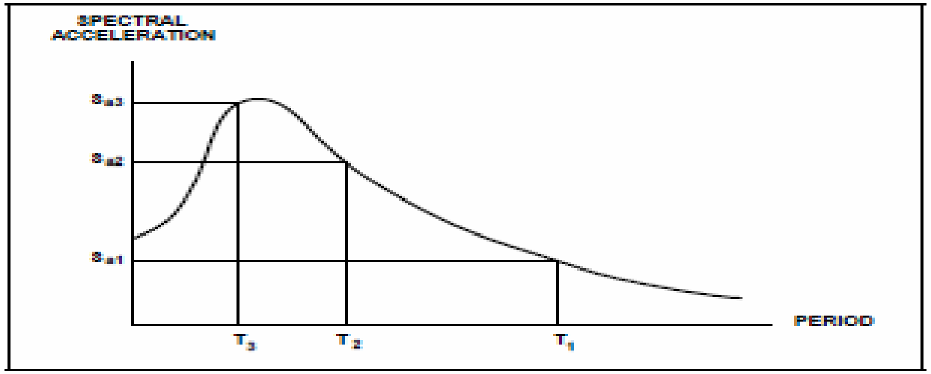
Figure 2-1. Factors affecting seismic input motion for a structure founded on soil-pile foundation

USACE- EM1110-2-6050, 1999

RESPONSE SPECTRUM ANALYSIS



(1) Compute mode shapes [ϕ_{j1}, ϕ_{j2}, ϕ_{j3}] and natural periods [T_1, T_2, T_3]



(2) Obtain spectral accelerations [S_{a1}, S_{a2}, S_{a3}] for all modes

$$L_n = \sum_{j=1}^{3} \phi_{jn} m_j \quad ; \quad M_n = \sum_{j=1}^{3} \phi_{jn}^2 m_j$$

$$PF_n = L_n / M_n$$

$$Y_n = \frac{PF_n}{\omega_n^2} \cdot S_{an} \qquad where \quad n = 1,2,3$$

(3) Compute modal participation factor PF_n
and maximum modal response Y_n

Figure 2-9. Illustration of response spectrum mode-superposition analysis (Continued)

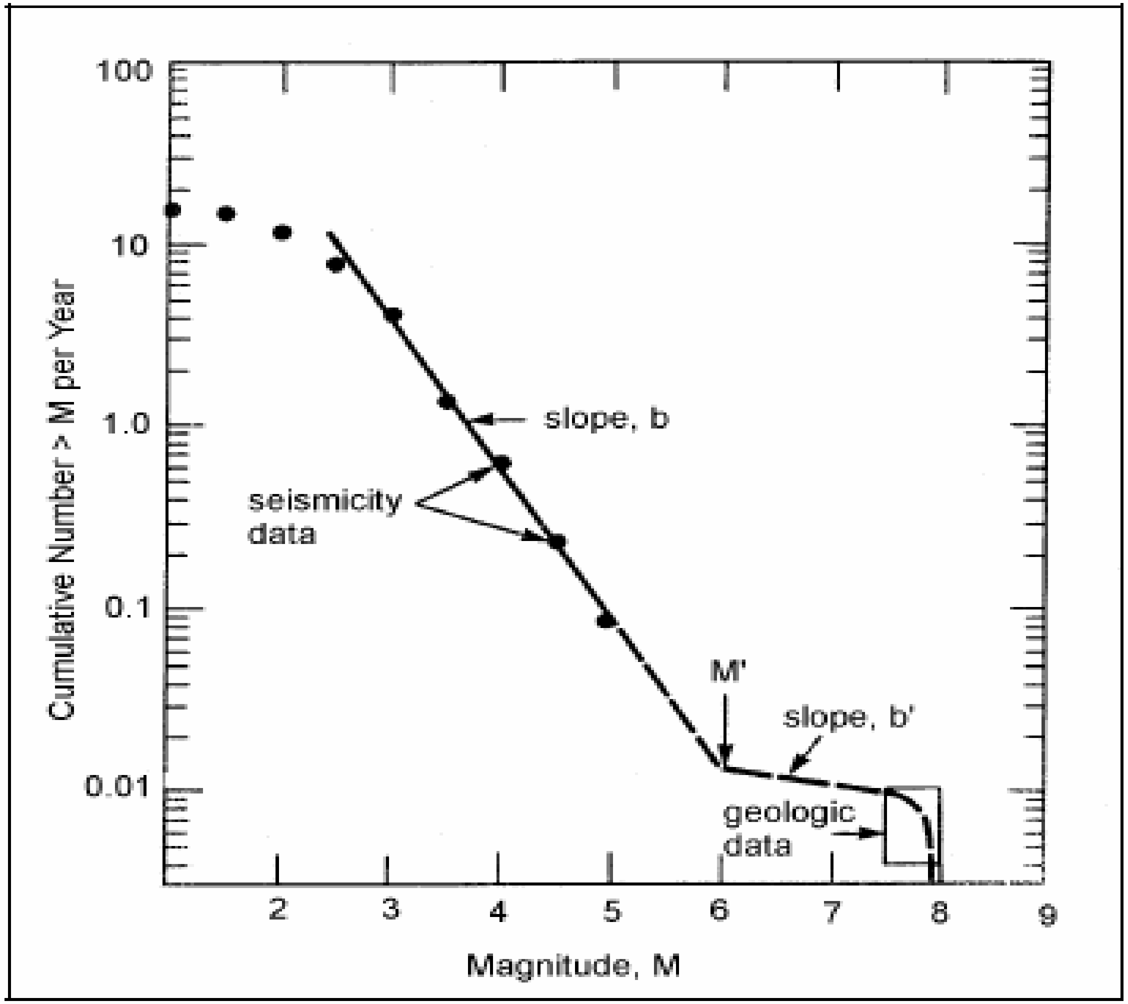


Figure 3-27. Diagrammatic characteristic earthquake recurrence relationship for an individual fault or fault segment. Above magnitude *M'* a low *b* value (*b'*) is required to reconcile the small-magnitude recurrence with geologic recurrence, which is represented by the box (Schwartz and Coppersmith 1984; National Research Council 1988)

USACE- EM1110-2-6050, 1999

? Q/A