RESPONSE OF ROCK TUNNELS TO EARTHQUAKE SHAKING

by

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ABSTRACT

The potential for damage resulting from earthquakes is a major factor to be considered in the siting of any structure whose failure would result in substantial loss of life and/or property. The thesis focuses on the evaluation of damage which may be caused in tunnels (larger than 3 meters, or 10 feet, in diameter) by ground shaking. The reason for this narrow focus is twofold: Firstly, it is assumed that other damage sources, such as ground failure or fault movement, may be minimized by careful siting; and secondly, it is of interest to compare the damage in tunnels to that which occurs in aboveground structures at the same intensity of ground shaking.

It was felt that these questions must be answered by locating, studying, and summarizing all known case histories in which some level of damage to tunnels is reported. This feeling was based on the assumption that any analytical model must be checked with real field data.

More than seventy cases were studied, and a correlation was made between the reported damage and the associated level of shaking. Several site factors, such as tunnel cross-section, depth of cover, ground conditions, etc. were studied.

It was found that no damage occurred in tunnels associated with accelerations up to 0.19 g or MM-VIII. Only a few cases of minor damage, such as falling of loose stones and/or cracking in brick or concrete linings, were observed in tunnels associated with accelerations up to 0.25 g (MM-VIII-IX). It was also found that most of the damage cases, still "minor damage" as described above,

were associated with accelerations above 0.4 g (MM-IX); and no case of collapse was associated with accelerations up to 0.7 g (almost MM-X). Local collapses or large distortions in tunnels were observed when a tunnel was sheared by fault displacement or where ground failure caused landslides near the portals.

Based on these case histories, it is clear that tunnels suffered only "minor" structural damage while aboveground structures suffer from "considerable" damage at the same MM-intensity levels. However, tunnels experiencing accelerations greater than 0.85 g should be inspected to eliminate possible accidents associated with small debris.

It is found by analysis of the case studies that (a) Tunnels in sound rock are safer than those in jointed rock or soil; (b) Stabilizing and improving ground conditions improves the ability of a tunnel to withstand earthquake shaking more than strengthening the linings; and (c) Deep tunnels are safer than shallow tunnels.

Finally, deep tunnels in competent rock are safe during earthquake shaking up to MM-IX, and may be recommended for use as components in large-scale installations such as nuclear power plants.

Thesis Supervisor: Professor Charles H. Dowding

Title: Assistant Professor

ACKNOWLEDGEMENTS

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LIST OF SYMBOLS

A - Amplitude

a - Acceleration

a_o - Inner radius of a circular tunnel

b - Outer radius of a circular tunnel (including the liner's thickness)

c - Wave velocity

c_h - Wave velocity in homogeneous rock

c_j - Wave velocity in jointed rock

c_p - Velocity of compression wave

c_s - Velocity of shear wave

D - Diameter

d - Displacement

E - Modulus of Elasticity (Young's Modulus)

E_F - Modulus of Elasticity determined in the field

E_L - Modulus of Elasticity determined in the laboratory

f - Frequency

G - Shear Modulus

H - Depth of overburden

h - Thickness of tunnel's liner

I - Moment of inertia

I_o - Modified Mercalli intensity at the causative fault

List of Symbols (Continued)

$I(R)$	- Modified Mercalli intensity at the distance R from the causative fault
i	- Angle of irregularities (in joints)
K	- Spring constant for ground
K_d	- Dynamic stress concentration factor
K_s	- Static stress concentration factos
M	- Richter's magnitude
MM	- Modified Mercalli
N	- Number of joints along wave-path (Eq. 2.1)
R	- Epicentral distance
r	- Radius of curvature
RQD	- Rock Quality Designation
s	- Duration
T	- Period
t	- Time
t_h	- Travel time of shock wave in homogeneous rock
t_j	- Travel time of shock wave in jointed rock
Δt	- Time delay per joint (Eq. 2.1)
U	- Ground displacement
U_t	- Tunnel's liner displacement
V	- Particle velocity
V_c	- Critical particle velocity
x	- axis

List of Symbols (Continued)

γ - Ratio of dilatational phase velocities in medium and liner

ε - Strain

η - Ratio of outer radius to inner radius of the liner

θ - Angle of shearing distortion

λ - Lame constant $= \dfrac{\nu E}{(1+\nu)(1-2\nu)}$

λ_o - Wave length

μ - Wave interaction factor $= \dfrac{2\pi a}{\lambda_o}$

ν - Poisson's Ratio

ν_o - Ratio of shear moduli of the medium and the liner

π - 3.14...

ρ - Mass density

σ - Normal stress

σ_c - Critical normal stress (usually tensile strength of the rock mass)

τ - Shear stress or shear strength

τ_θ - Ground-liner interaction factor defined by Eq. 2.13

ϕ - Friction angle

ϕ_i - Friction angle of intact rock

ϕ_j - Friction angle of joint

ψ - Joint density

List of Symbols (Continued)

$\bar{\psi}$ - Mean joint density or fracture frequency (Eq. 2.2)

ω - Circular frequency

CHAPTER 1

INTRODUCTION

1.1 OBJECTIVES

This study is carried out to correlate earthquake-induced ground motions and their engineering consequences in underground tunnels and cavities. This correlation may help to determine the extent of damage to be expected in tunnels under dynamic loading in the form of earthquake disturbances, and hence will contribute to the understanding of the ability of underground structures to withstand earthquake loading.

Furthermore, using known attenuation laws, it will be possible to define the minimum distance from an expected earthquake with a given magnitude for no cavity damage or what the degree of damage expected is if the underground structure will be closed to the caucasive fault. The details of the damage data and of the geological conditions in the surveyed case histories are journalistic in nature, but correlation of available facts appears to warrant useful generalizationss and preliminary conclusions which are summarized in Chapters 5 and 6.

1.2 METHODOLOGY OF THE STUDY

This paragraph will summarize the main steps that were followed in the research reported in this thesis.

The first step in the study was a thorough literature search for material bearing on the problem. This search was based on the opinion that the problem may be approached by one or more of the following methods:

(a) Analytical solutions using either close-form solution or a numerical analysis technique, including the finite-element method;

(b) An experimental approach using physical modelling or photoelastic techniques;

(c) Field-study approach analyzing field data from known case histories.

The third approach was adopted in this thesis.

The literature search was divided into the following areas:

(a) Behavior of rock masses in general and the influence of natural discontinuities on this behavior, both under simple static and more complicated dynamic conditions. This is summarized briefly in Chapter 2.2.

(b) Dynamics of rocks under various time loading histories, including both continuums and discontinuities (Chapters 2.3 - 2.6).

(c) Damage criteria for rocks and rock-like material such as concrete (Chapters 2.6 - 2.7).

(d) Analytical and numerical solutions to similar problems, especially those developed for hardened tunnels designed to withstand nuclear explosions (Chapter 2.3).

(e) Determination of earthquake "input" data by application of attenuation laws (Chapter 3).

The references that have direct applicability to the work of this thesis were thoroughly examined, and those which have a permanent value to this and related works have been included in the bibliography at the end of the pertinent chapter. This bibliography should serve as a useful aid to other researchers in this general field. Amplification of this methodology will be found in later paragraphs where pertinent results will be reported in detail.

1.3 CONTRIBUTIONS OF THIS THESIS

The main contribution of this thesis is the location, collection, summary, and analysis of the available published information on earthquake damage -- <u>or the lack of real damage</u> -- to tunnels. These field data are necessary for comparison with either analytical and numerical solutions or experimental results of small-scale physical models. This comparison to real life data is a basic step to estimate, and maybe to define, the validity of any analytical solution or experimental conclusion.

1.4 PRESENTATION OF THE RESULTS

This study is divided into four parts. Chapters 2 and 3 summarize very briefly those basic ideas and concepts which are necessary for the understanding of the behavior of cavities in ground under earthquake shaking. Chapter 4 introduces the case histories upon which the study is based. Chapter 5 summarizes in a comparative way the characteristics of earthquake-induced ground motions and their engineering consequences, and Chapter 6 outlines the major conclusions resulting from the study and recommendations for future research.

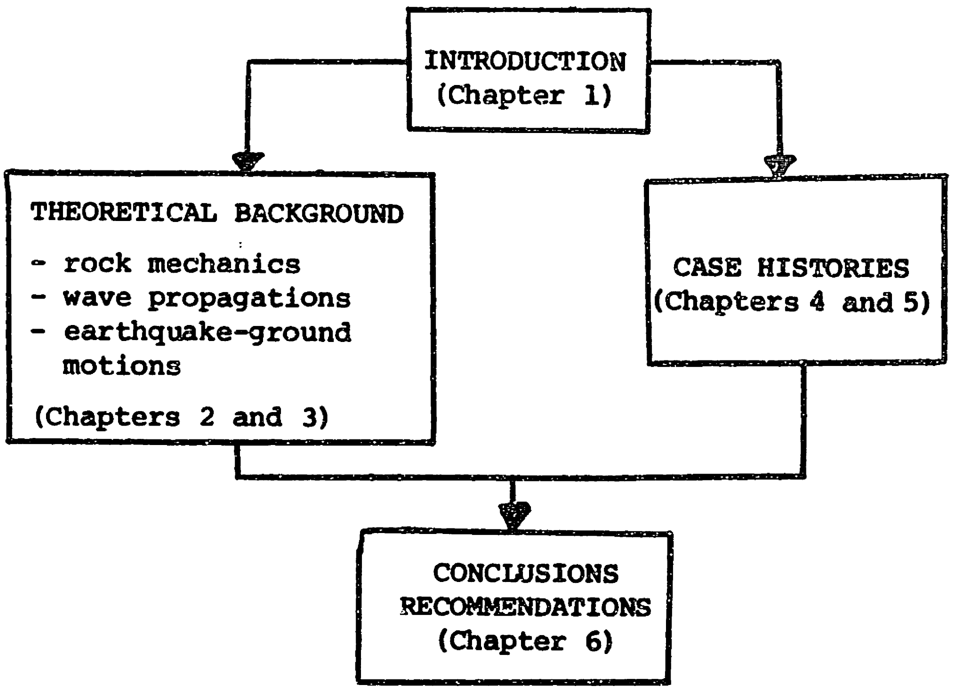


FIGURE 1.1 Framework and Presentation of the Results

CHAPTER 2

BACKGROUND

2.1 INTRODUCTION

The potential of cavity damage from earthquakes is a major factor to be considered in the siting of any large-scale project whose failure would result in substantial loss of life and/or property. Damage in underground structures due to earthquakes is generally manifested in one or a combination of the following forms:

(a) Damage from ground failure, such as liquifaction or landslide;

(b) Damage from ground "shaking" or ground vibration;

(c) Damage from fault movement;

(d) Damage from tsunamis.

The potential of cavity damage from ground failure may be evaluated by ordinary soil mechanics and geological explorations and testings. Prudent siting can avoid this problem.

Fault displacement is usually considered as very serious. Similar to ground failure, siting to avoid intersection with active faults capable of movement can minimize this problem for new tunnels. It may be proven, as will

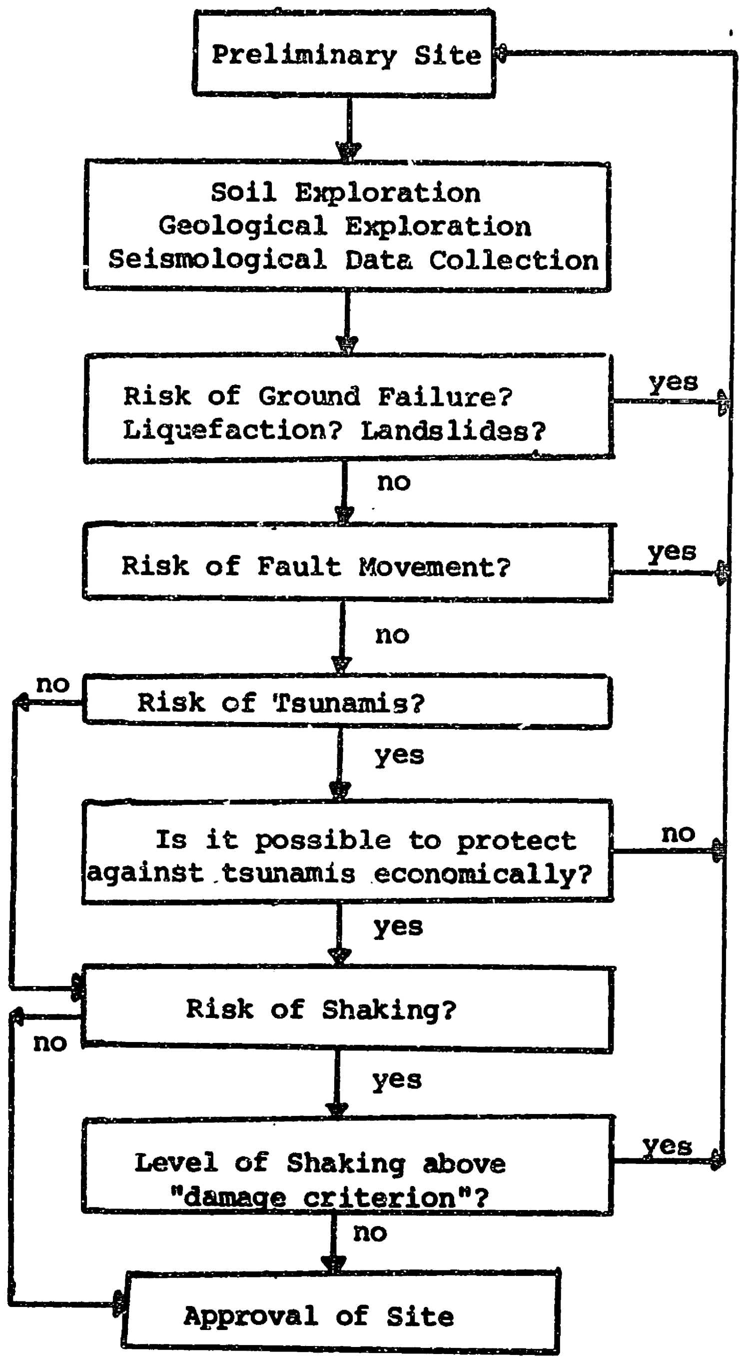


FIGURE 2.1 <u>Steps in Selection of Site</u>

be done in Chapter 4, that most of the damage occured to tunnels due to fault movement was caused firstly by already known faults, and secondly by siting the tunnels across the faults without trying to circumnavigate them.

Damage from tsunamis is limited to structures near the seashore and may be eliminated by proper design of protective measures around the entrances to the tunnels.

Assuming that these three problems have been evaluated, there remains the problem of the ground shaking, which is the main subject of this study.

The "check list" of siting considerations is given in Figure 2.1.

2.2 THE ROCK MASS

In general, rock is a nonhomogeneous, anisotropic, non-elastic, discontinuous media. The degree of departure from the "ideal" homogeneous, isotropic, elastic continuum depends on the rock type, composition, depth, and several geological factors. These non-ideal characteristics made the analysis of rock masses and structures in rock a most complicated subject in which the unknown is much more than the known. The last twenty years contributed a great deal to the understanding of rock behavior; but unfortunately, most areas of research and hence most advancements are concentrated on static stress analysis and rock properties determination.

Before analysis of a problem can begin, knowledge of the mechanical problem of the material -- how it deforms and fails under the action of applied forces -- must be gained. For rock, it is a sad fact that a rational analysis cannot always be made, especially in the field of dynamic loading caused by ground motions since much of the pertinent needed background is still obscure.

Commonly accepted practice in rock mechanics regarding both static and dynamic stress analysis is to assume the rock as an ideal homogeneous, isotropic, elastic continuum. In static analysis of unlined tunnels, the papers by Mindlin (1940) and Terzaghi and Richart (1952) are used, and both give an exceptionally complete treatment of the entire subject from the theory of elasticity point of view. For lined cylinders, Burns and Richart (1964) and Hoeg (1968) gave solutions for displacements and stresses in an elastic media.

In dynamic analysis, there are several closed form and numerical, especially finite element, solutions which were developed during the 60's for various time-varying waves simulating the interaction between ground-induced shock waves caused by nuclear detonations and cavities in soils and rock. A thorough review of many analytical and numerical solutions was introduced by Mow (1964).

Both solutions just referred to (for static and dynamic problems) make assumptions of a homogeneous and isotropic media for the purpose of analysis; and thus, certain phenomena of jointed rock behavior are not accommodated in these solutions. The last ten years saw a tremendous advance in understanding of the great influence that all kinds of discontinuities have on rock behavior. It is now clear that in analysis of rock behavior, both in static and dynamic stress analysis, joints, cracks, faults, and other kinds of planes of weakness of geological origin have to be taken into consideration. As a matter of convenience, all the planes of weakness will later be called "joints" or "discontinuities."

The behavior of joints is summarized in an updated book by Goodman (1976). Specific subjects such as joint stiffness, shear, dilatancy, influence of joint-filler, confining stress, etc. were subject for many researches which are too numerous to be counted here. Nevertheless, some papers will be mentioned here, giving the possibility to those wishing to study them in detail.

Joint strength as influenced by the roughness of the rock surface (Figure 2.2) was studied by Patton (1966), and the joint's shear movement was further studied by

Goodman and Dubois (1972).

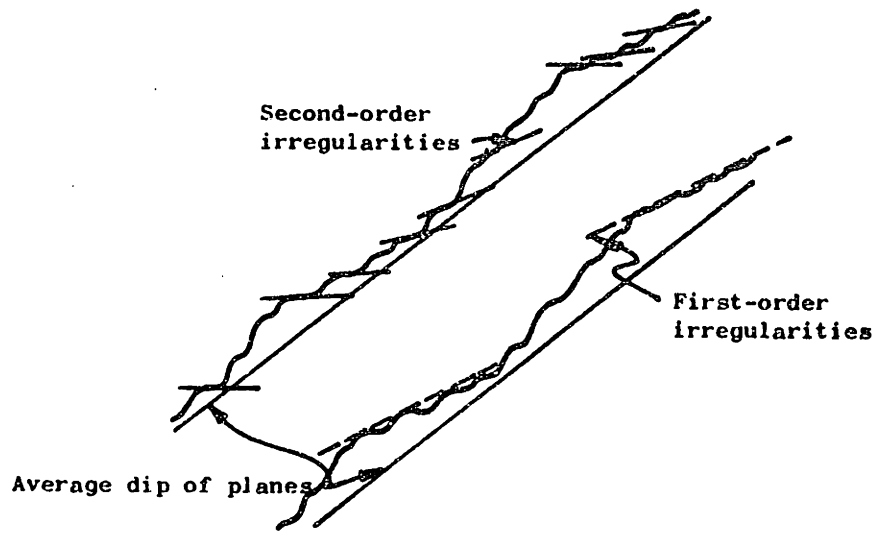


FIGURE 2.2 Rough Rock Surface With First- and Second-Order Irregularities (after Patton 1966)

The mechanism of dilatancy was discussed by Patton (1966), Rengers (1970), Barton (1972), and its role in the relation between shear strength and normal stress is explained qualitatively in Figure 2.3. Frictional behavior was analyzed by Byerlee (1967), assuming elastic mechanism, and Bowden and Tabor (1967), assuming plastic mechanism.

The influence of the joint filler on joint behavior was studied by Coulson (1970, 1972) and Brekke and Howard (1973), as described in Figure 2.4. Water under pressure,

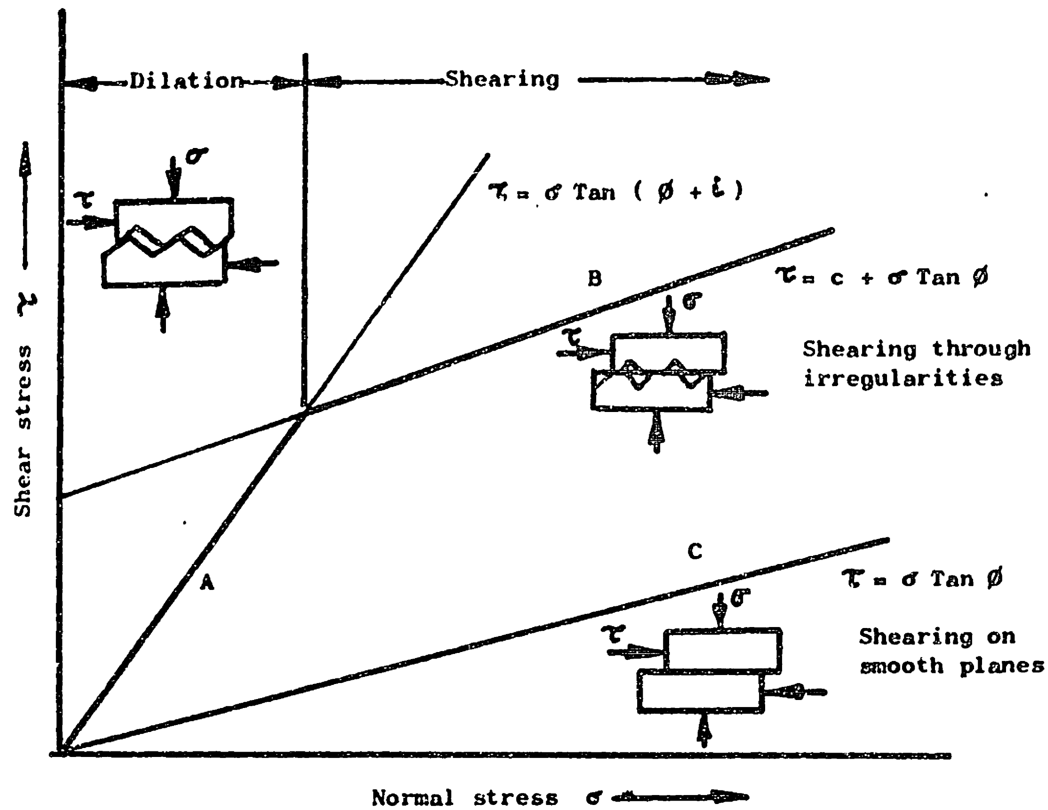


FIGURE 2.3 Simplified Relationships Between Shear Strength and Normal Stress for Rough Surfaces (after Hoek & Bray, 1974)

which reduces the effective normal stress, also reduces the shear resistance of a joint. In addition, water alters the deformation behavior and properties of the filling material. The influence of water pressure in reducing effective stress is similar to that measured by Wiid and Colback (1965) shown in Figure 2.5.

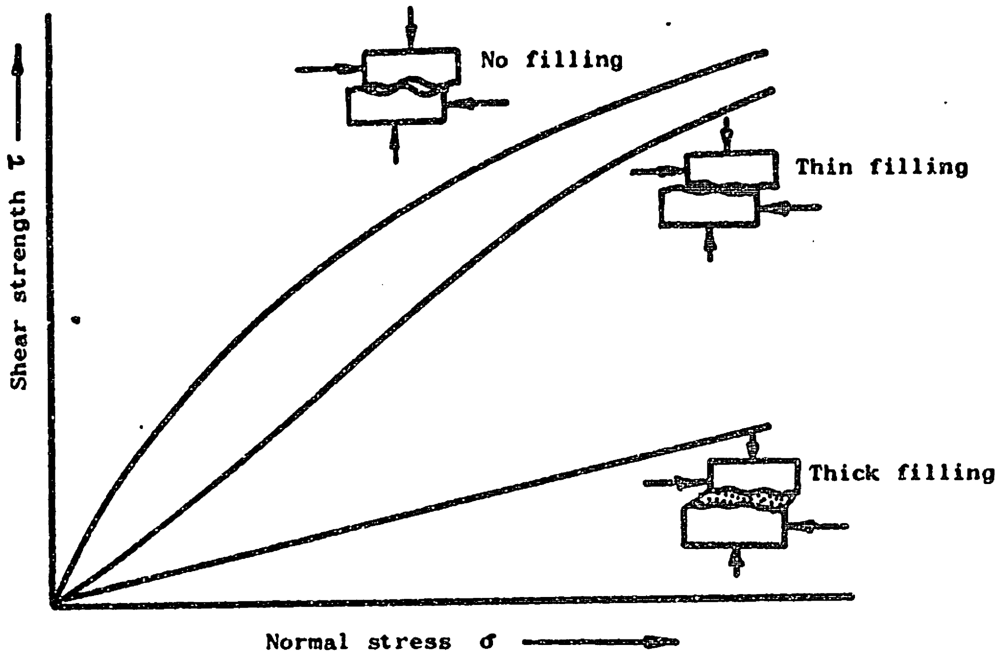


FIGURE 2.4 Relationship Between Shear Strength and Normal Stress for Discontinuities With Different Thicknesses of Gouge Infilling (after Hoek and Bray, 1974)

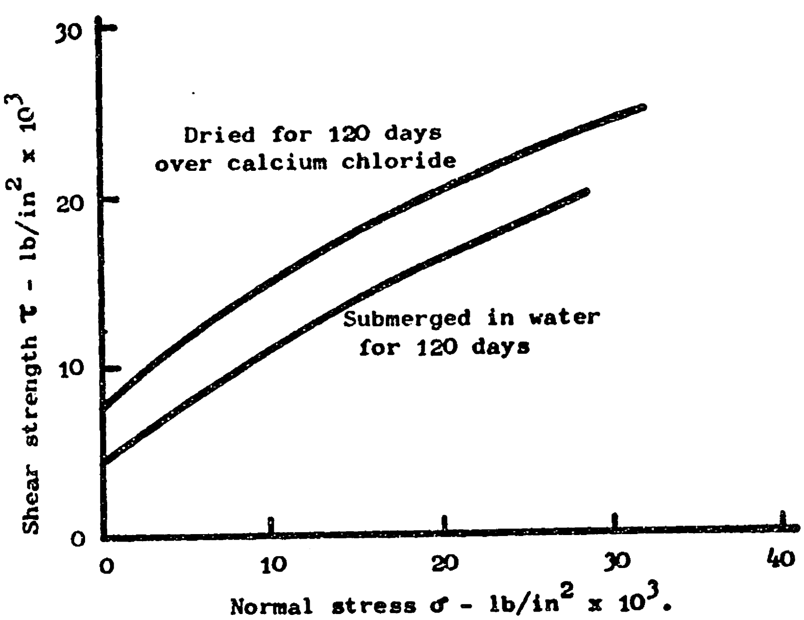


FIGURE 2.5 Influence of Water Upon the Shear Strength of Intact Specimens of Quatrzitic Shale (after Wiid & Colback, 1965)

The jointed mass, as a whole, is influenced by the characteristics of its individual joints, which were mentioned above; and as a conclusion, there are different failure envelopes for the intact rock and the jointed rock (Figure 2.6).

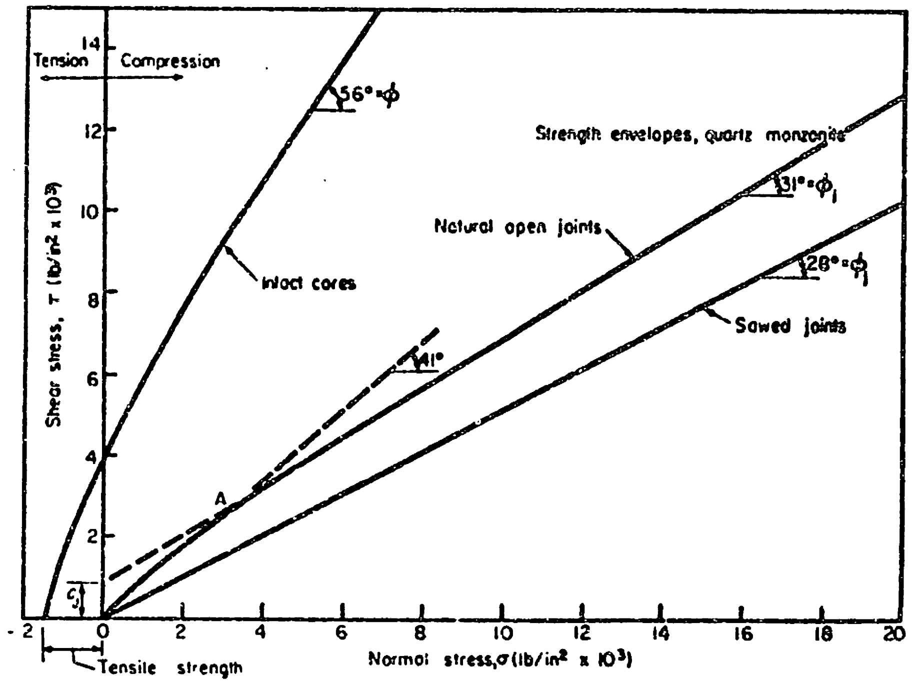


FIGURE 2.6 <u>Strength of Intact and Jointed Specimens of Quartz Monzonite</u> (after Hendron, 1968)

It is of interest to note here two points:

(1) While the intact rock does have tensile strength, even if it is approximately only 10% of its comprehensive strength, the joints have no tensile strength (Figure 2.6).

(2) Jointed rock which have failed by shear displace-

ment still have residual strength which is less than its peak strength, but it is still of engineering value and is usually the reason why local failure in tunnels does not cause immediately a total collapse (Figure 2.7).

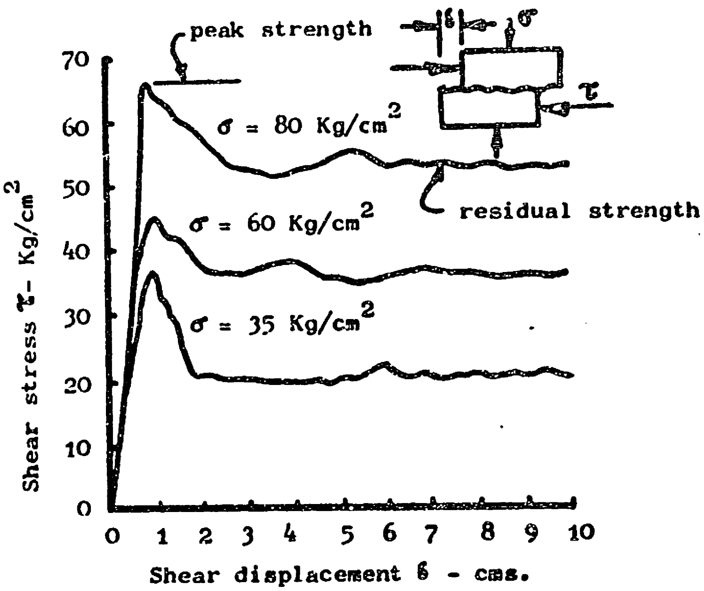


FIGURE 2.7 Shear Stress Versus Shear Displacement Results for Tests on Porphyry Joints (after Hoek and Bray, 1974).

A thorough literature search by Brekke and Howard (1973) and Brekke et. al. (1965) proved the important role of the discontinuities on the failure of tunnels in jointed rock. Summaries of the behavior of jointed rock may be found in works by Einstein et. al. (1970) and Lew (1973).

Hence, the influence of the discontinuities must be taken into consideration when analyzing the stability of tunnel in jointed rock both for static and dynamic loading.

The main question which arises now before the designer is, "What is the 'right', or maybe 'practical', way to take the discontinuity, inhomogeneity, and anisotropy of the jointed mass into account?".

Some attempts are now made to solve it by using Finite Element techniques, but to the writer's opinion, this approach suffers from two distinct disadvantages.

(1) A detailed description of all the significant structural features may require storage capacity in excess of that available in the present generation of computers. By reducing the level of required details, the advantage of this approach is lost.

(2) Even if there is a good and accurate analytical description of the discontinuities (which at the present time does not exist), and even if the needed storage capacity of computers was available, the geological field information is not available in sufficient detail to permit accurate computer modelling of the rock mass (Dowding, 1975).

Furthermore, it is impractical in most civil engineering works to consider in detail every structural feature even if these were known. It is especially true for a very long structure such as a tunnel which passes through varied rock conditions. The design must be based, from the practical and economical point of view, on average rock conditions while still being capable of accommodating moderate variations from the average conditions. It is, then, the writer's opinion that the "Elastic" approach, assuming the rock to be an elastic, homogeneous, isotropic continuum, is the most practical one at present; but the average behavior of the discontinuities must be taken into account by using appropriate reduced values for the elastic characteristics of the jointed rock. Several attempts to use this approach were made for shock wave propagation in jointed rock media. All the suggested solutions are far from being reliable as they are based on very simple models; but, nevertheless, it is of interest to describe them. This will be done in paragraph 2.5.

A practical engineering method to estimate the reduced elastic constants of jointed rock mass may be based on the Rock Quality Designation (RQD) suggested by Deere (1964) and Deere et. al. (1969).

The RQD is based on a modified core recovery procedure taking into account rock cores of 4" length or longer. It is based, therefore, indirectly on the number of fractures or their average frequency per lineal feet. Figure 2.8 summarized the relation between fracture (or joint) frequency and RQD, while Figures 2.9 and 2.10 are correlation of RQD and the reduction of the in-situ moduli of elasticity for a wide range of rock types and locations.

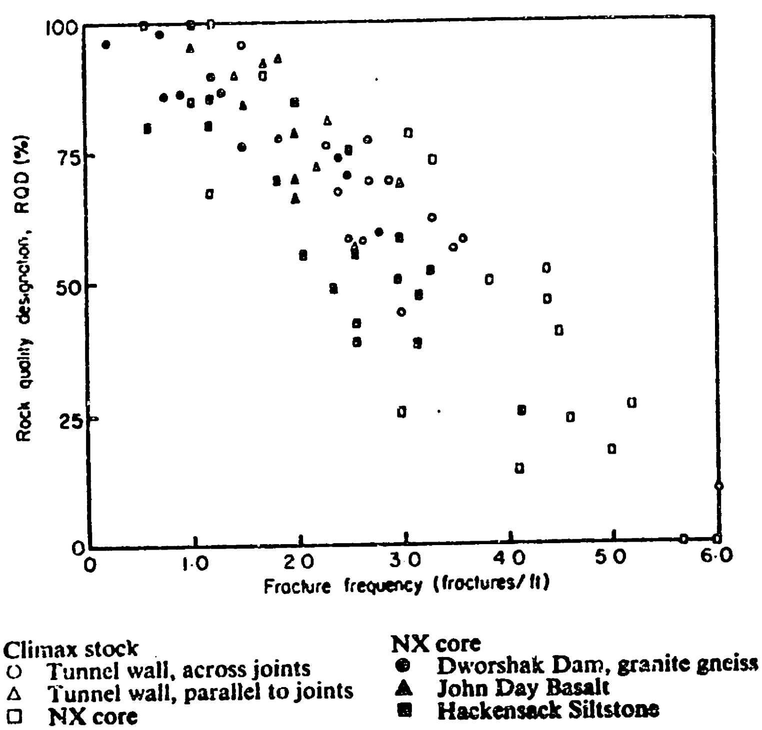


FIGURE 2.8 Correlation of Rock Mass Quality Indices: Fracture Frequency and RQD (after Deere, 1968)

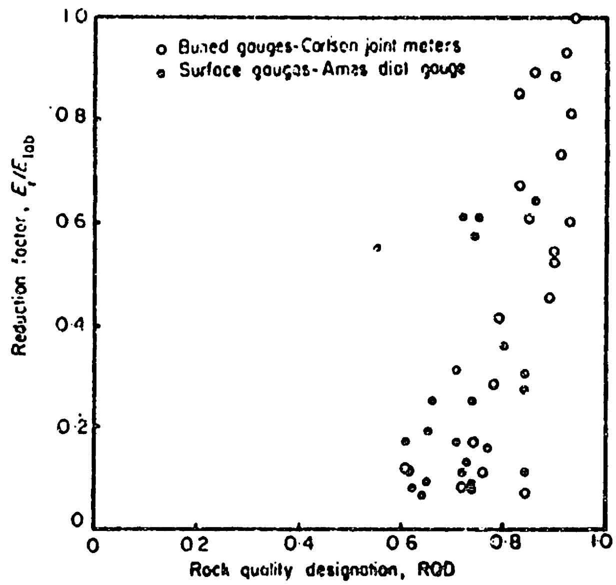
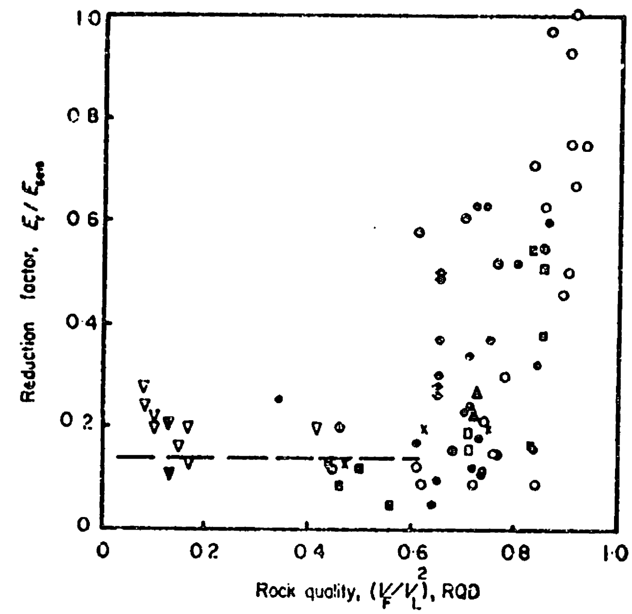


FIGURE 2.9 Variation of Reduction Factor With Rock Quality From Plate Jacking Tests, Dworshak Dam (after Hendron, 1968)

By this approach, it is possible to use the theory of elasticity even in jointed rock by modifying the elastic constants according to the RQD.



Dworshak Dam, pressure chamber test (F), buried gauges[17]
Dworshak Dam, pressure chamber test (F), surface gauges[17]
Dworshak Dam, pressure chamber test (E), buried gauges[17]
Dworshak Dam, pressure chamber test (E), surface gauges[17]
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Onodera[26]
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FIGURE 2.10 <u>Variation of Reduction Factor With Rock Quality (after Hendron, 1968)</u>

2.3 DYNAMIC RESPONSE OF OPENINGS IN AN ELASTIC HALF SPACE

2.3.1 General Background on Diffraction

The problem of elastic wave propagation is now a classical subject with many known aspects (e.g. waves in rods, plates, shells, semi-infinite and infinite bodies). This is a vast subject which is far beyond our discussion. Of interest in this research is the diffraction of elastic waves around cavities, which is a rather limited aspect of the general wave problem.

The elastic wave diffraction, or scattering problem, has received considerable attention from geophysicsts and seismologists, as well as elasticians. However, due to the complexity of the mathematical expression, numerical results are still few.

Sesawa (1927) has treated the diffraction of elastic waves by cylindrical, spherical, and elliptical obstacles, but without any numerical results. Nishimura (1937) and Nishimura and Takyama (1938) discussed the problem of elastic wave at the surface of a spherical cavity, and Sesawa and Kanai (1937) discussed the stationary vibration of a spherical cavity. These works might be of qualitative value for large underground chambers, but they are not recommended for long, cylindrical tunnels.

A study of the dynamic loading caused by standing waves was presented by Nishimura and Jimbo (1955) with numerical results for vacuous, elastic, and rigid spherical inclusions. The problem of scattering of elastic waves by cylindrical and spherical obstacles was also studied by acousticians and physicists such as Junder (1952) and Ying and Truell (1956), White (1958), Knopoff (1959), and Gilbert and Knopoff (1959). These papers are concerned with "scattering cross-section" (defined as the ratio of the total energy scattered per unit time by the obstacle to the energy per unit time of the incident wave) and the "far field displacements", and less attention was given to the stress field around the inclusion. Mindlin and Bleich (1953) have treated the response of a cylindrical shell to transverse step-shock wave.

Gilbert (1959) solved the problem of elastic-wave interaction with a cylindrical cavity using the "high-frequency" approximation which limits the solution to the early-time stress phenomena. This solution was limited to the incident side of the cavity.

2.3.2 Diffraction Around Unlined Cavities

In the early 60's, the increase of emphasis on hardened underground installations gave a real "push" to studies of dynamic loading of tunnels. The dynamic response of an unlined cavity within an elastic medium was calculated usually by three different methods: (1) integral transform technique, (2) numerical integration, and (3) steady-state response. The mathematical details will not be treated here, and the reader is referred to the original papers which are mentioned next.

The Integral Transform Technique was used by Baron and Matthews (1961) and Baron and Parnes (1961, 1962) for pressure waves. They got numerical results and proved that the dynamic-stress concentration factor was about 10 percent greater than the static stress concentration factor for $\lambda \leq 15D$. Figure 2.11 summarizes data from Baron and Mathesus (1961) and Pao (1962) and shows the maximum dynamic and static-stress concentration factors at the cavity as a function of the Poisson's ratio for $\theta = 90°$ (side-on to the incident wave) and $\lambda \gtrapprox 4\pi D$. Baron and Matthews (1962) used the same approach for the diffraction of shear waves and also found that the dynamic stress-concentration factor was 10 percent greater than in the static case. Their solution was refined by Logcher (1962), who extended the earlier work of Baron et. al. (1960, 1963) by including two additional modes in his solution.

Numerical integration was used by Ho (1963). The governing partial differential equations and the relevant boundary and initial conditions were converted into a

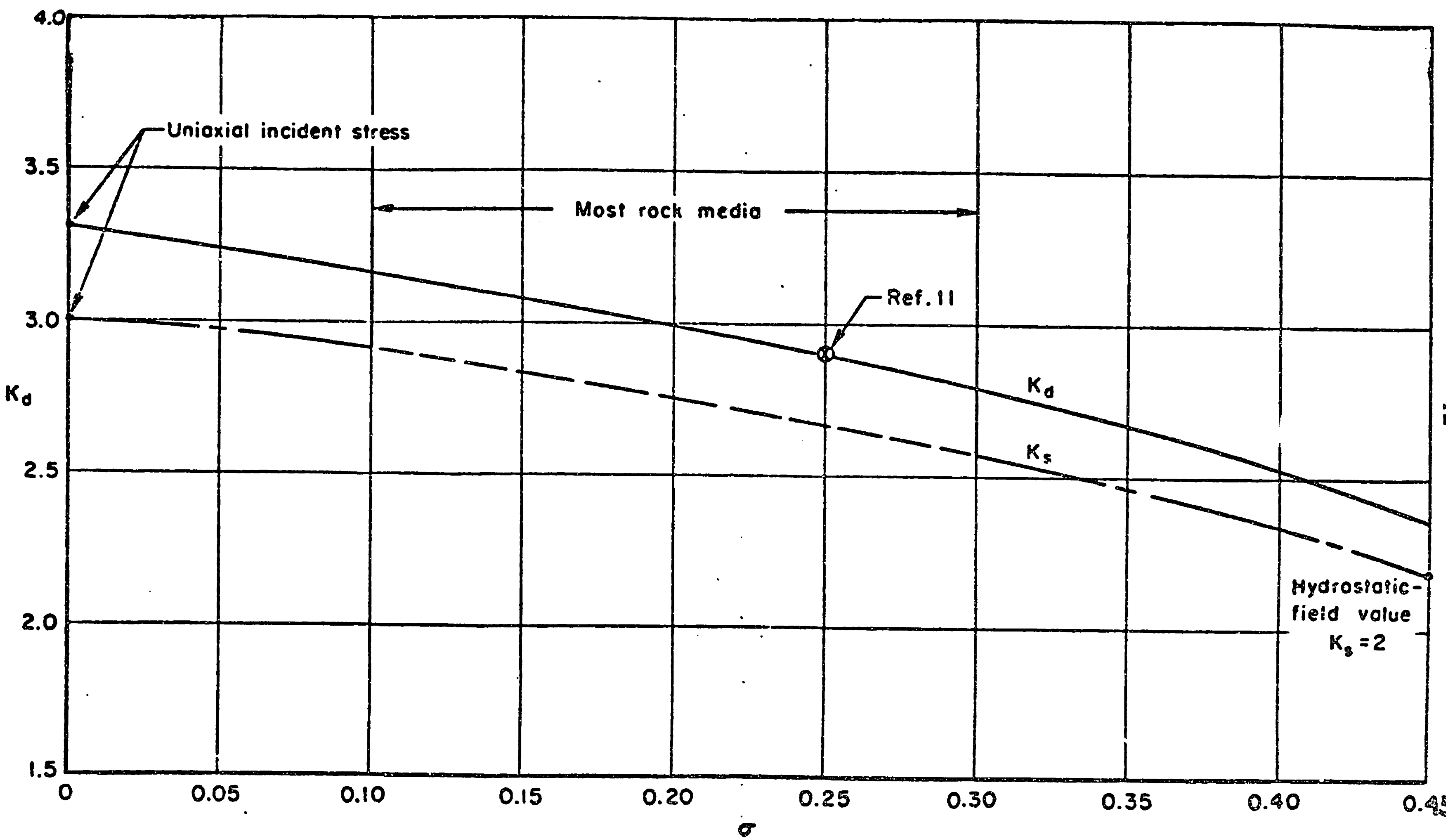

FIGURE 2.11 Dynamic- and Static-Stress Concentration Factors For Cylindrical Cavity Versus Poisson Ratio (compressional wave) (after Mow, 1964)

special system of ordinary differential equations through the Finite Difference Technique. The solution of the latter was obtained by a recurrence formula, and the results were interpreted for the original continuous system.

The steady-state response was studied for sinusoidal pressure waves. By the method of separation, the partial differential wave equations were reduced to ordinary differential equations. Solution for these ordinary differential equations were then sought to satisfy the prescribed boundary conditions. The transient response was obtained from the steady-state case by Fourier Transform technique. This approach was used by Pao and Mow (1961), Mow and McCabe (1962, 1963), Pao (1962), Pao and Mow (1962, 1963), and Mow and Mente (1962). Their results show that the critical stresses of the steady-state response are a very good approximation for the ultimate dynamic stresses due to transient-type loading. Furthermore, for travelling harmonic waves, Pao (1962) found a maximum stress concentration factor which was identical to that in Baron's results. Pao and Mow solved the problem of dynamic stresses in the vicinity of rigid circular inclusions and cylindrical discontinuities (both rigid and vacuous) for compression and shear waves.

Experimental work using dynamic photoelastic techniques was conducted in the late 50's and early 60's and resumed by Riley et. al. (1961). In their study, they found that the dynamic stress concentration factors for plates with holes of various shapes (including elliptical and square holes with round corners) are not appreciably changed, except that the maximum value is about 15 percent higher than the corresponding static value.

2.3.3 Diffraction Around Lined Tunnels

A lining can be used for two main reasons: (1) to strengthen and stabilize the tunnel walls during or after construction as a result of poor rock conditions, or (2) to enhance the survivability of an underground cavity against an exploson-induced wave. During dynamic loading, the lining may reduce the stresses around the opening, or by using a backpacking material, it may even attenaute the incident wave to such an extent that the liner itself will survive.

Some work on the stress wave-structure interaction problem was made by Baron and Parnes (1962) on elastic medium and thin-shell, Mow and McCabe (1963) on linings of arbitrary thickness, Baron et. al. (1963) on lined cavities, and Soldate and Hook (1961, 1962) for a step wave.

All the studies of structure-medium-interaction show that the dynamic stresses are about 10 percent higher than those of corresponding static problems, similar to the observed trend in the unlined cavity studies. This conclusion was confirmed by Assini, Hawly, and Mow (1961), and is recommended as a first approximation.

The main mathematical approach in all these studies was to use the unlined-cavity displacements as influence coefficients to obtain the corresponding displacements and stresses in the lined cavity. Another approach, used by Mow (1964), is to use the theory of elasticity for a travelling harmonic wave. The results obtained by this method do not give the time history of the stress in the liner and medium.

Some results, summarized by Mow (1964), are illustrated in Figure 2.12. The maximum dynamic stress concentration factor (K_{DA}) is mainly a function of three parameters:

ν_0 - The most important factor for a given geometrical configuration;

η - Predominant factor for a given ratio of shear moduli;

γ - The least effective of the three.

It is apparent that the stresses in the liner and the

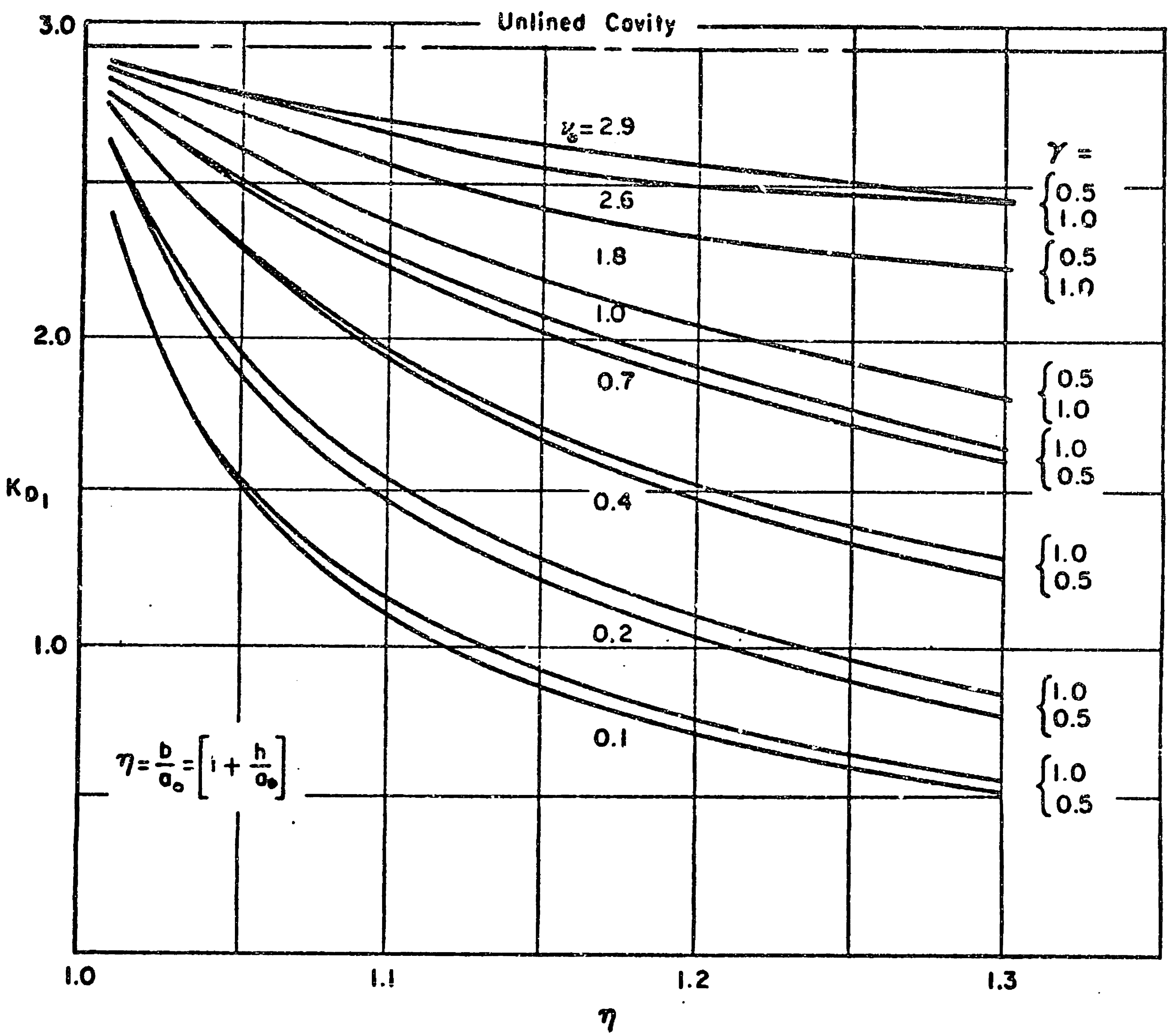

FIGURE 2.12 Maximum Medium Dynamic-Stress Concentration Factor K_{D1} Versus Liner Thickness Paramter η For Various ν_0 and γ (after Mow, 1964)

medium depend primarily on the relative properties of the liner and the medium, hence it is essential that efforts should be made to determine the properties of the site before any decisions are made on the type of liners to be used.

2.4 CONCLUSIONS FOR INTERACTION OF LONG WAVES

When an elastic wave (whether acoustic, seismic, or optical in nature) encounters an obstacle, some of the wave is deflected from its original course. It is usual to define the difference between the actual wave after incidence and the undisturbed (or "free field") wave, which would be present if the obstacles were not there, as a scattered wave.

Some of the many studies about this subject were mentioned briefly in paragraph 2.3. This paragraph is concerned with the scattering of very long waves (much longer than the diameter of the obstacle) from a cylindrical cavity (Morse, 1948).

When a plane wave strikes a tunnel in its path, in addition to the "free field" plane wave there is a scattered wave spreading out from the tunnel's walls in all directions distorting and interfering with the original plane wave. If the tunnel diameter is very large compared to the wave length (which usually is the case for high-frequency blasting waves), half of the intensity of the scattered wave spreads out more or less uniformly in all directions from the tunnel's walls. The other half is concentrated behind the tunnel in such a manner as to interfere destructively with the unchanged plane wave behind the tunnel creating a sharp-edged "shadow" there. (See Figure 2.13 for $\mu \geq 5$ or $\lambda_0 = \frac{2}{5}\pi a$).

If the tunnel is very small compared with the wave-length (as it often is for low-frequency seismic waves), then all the scattered wave is sent out uniformly in all directions and there is no sharp-edged "shadow." (See Figure 2.13a for $\mu=1$ or $\lambda_0=2\pi a$).

It is interesting to note that for a 20-foot diameter tunnel (a=10') in a medium rock with seismic velocity of 10000 ft/sec, the μ value for 1 Hz earthquake's wave will be 0.062, and for construction blasting with 100 Hz will be 0.62. This difference has a very large effect in the sharp inclined zone of the curve. (Figure 2.13b). Similar conclusions had been made by Rayleigh (1945) for light and sound waves, and a similar tendency was found by Engineering Research Associates (1953) for blasting waves.

We may conclude that the scattering of earthquake waves (in frequency range up to 25 Hz) is quite negligible for tunnels.

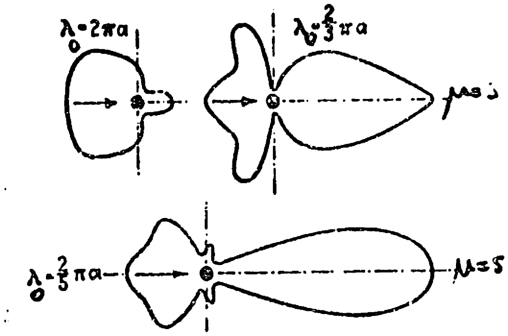

FIGURE 2.13a Distribution of the Intensity of the Scattered Wave for Different μ (or λ_0) Values. (After Morse, 1948)

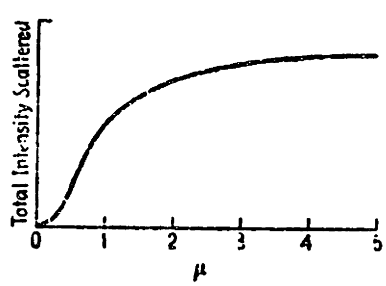

FIGURE 2.13b The Scattering of Elastic Waves From a Rigid Cylinder of Radius a: Dependence of the Total Scattered Intensity on $\mu = 2\pi a/\lambda_0$ (After Morse, 1948)

An analytical study using the theory of elasticity has been done by Okamoto et. al. (1963) and summarized briefly by Okamoto (1973). The analysis is based on the assumption that a seismic force is built in the tunnel lining during the reflection of shear and longitudinal waves. For sinusoidal shear waves propagating perpendicular to the longitudinal axis of the tunnel, it was found that when tunnel's diameter was small in comparison to the earthquake wavelength, the seismic load acting on the tunnel's lining and the bending stress produced in the lining is increased with increased lining thickness. This theoretical analysis may explain the in-situ findings reported in the Kwanto Earthquake (Chapter 4.13).

Several Japanese researchers studied the strains in large diameter pipelines (or small diameter prefabricated installation tunnels) and found that the strains due to deformation in the axial direction exceed the strain due to bending. The strain was found to be inversely proportional to the seismic velocity of the ground; and the earthquake will produce greater strain in prefabricated tunnels buried in soft ground than in hard ground or soft rock.

2.5 INFLUENCE OF DISCONTINUITIES ON WAVE PROPAGATION

The influence of discontinuities on wave propagation was studied by (a) seismologists for long wave reflection and refraction and (b) military-oriented engineers for shock waves induced by nuclear bursts or close high explosive detonations.

Abbott and Davis (1969) developed a one-dimensional code to solve wave propagation in elastic or visco-elastic medium with cracks transverse to the direction of propagation of the dilatational wave. They found that decreasing the crack size causes less impulse to be included under the spikes caused by the cracks, and increasing the crack spacing induces more loading and unloading in the stress pulses and may cause higher spikes to be induced. According to Abbott and Davis, the large amplitude, short duration, stress spikes induced by cracks may not have sufficient impulse to cause significant structural motions in buried structures; but they could cause severe spallation damage.

Wyatt (1964) studied the influence of joints and joint patterns on rock failures, landslides, and the cratering phenomena caused by explosive-initiated stress waves. Wyatt found that jointing reduced the intensity and increased the length of a stress wave, and that the stress wave tends to lengthen and widen a joint in a rock mass.

An attempt was made by Abbott (1970) to develop a one-dimensional constitutive law for a continuous material which would have the same macroscopic response as a transversely cracked material, and which would have comparable results to those obtained earlier (Abbott et. al., 1969). This model was based on identical elastic plates separated by equalized gaps before loading. This approach was based on a plate-gap model developed by Heyda (1967) and Hoffman, et. al. (1968).

Static studies on anisotropic continuum model were done by Smart (1970) for mass containing three orthogonal equally-spaced joint systems, each joint containing filler (gouge) which was assumed to be elastic and isotropic; and another solution was derived by Salamon (1968) for rock masses containing horizontally filled joints with arbitrarily distributed elastic constraints.

Hashin and Shtrickmann (1963) tried to determine some range of properties by proving the existence of upper and lower limits for the modulus of deformation for a heterogeneous material. This approach may be used to study the elastic properties of a rock mass which could be assumed to consist of isotropic, intact rock containing randomly-oriented joints.

There are still some contradictory statements, such as: Kroner (1967) who found that the effective shear modulus of a perfectly disordered material may be significantly less than the mean value experimentally determined; and Su et. al. (1970) who found that the mean stress in a stochastic body is not significantly different from the stresses in a classical elastic body.

Another approach to the "statistical" model was done by Singh (1973), who developed "joint stress concentration factors", which are defined as the ratio of stresses along the joints to the overall stresses in the rock. Singh also found that tensile stresses are developed inside the block of a rock embraced by staggered joints and may be as high as twice the overall shear stress or the overall compressive stress. In a second article, Singh (1973) found that stresses are transmitted to a considerably greater depth along joints and to some extent even across joints. This tendency is more pronounced in a joint set of low shear stiffness.

The mechanism of attenuation caused by discontinuities is affected by many factors which complicate the phenomenon. The transmitted wave is affected by the relative properties of the competent rock and the joint's filler, the diffraction around the joint causing both reduction of amplitude and a time delay, redistribution of propagating energy, closing of open joints, crushing contact points along the joint, joint slippage, and compaction of soft gouge materials.

Most of these mechanisms are important only for high-frequency, short-length waves common in HE blasting or even in nuclear detonation, but their influence and importance are somewhat obscure for low-frequency long waves encountered in earthquakes.

Seismic surveys indicate a significant decrease in the velocity propagation within a jointed (fractured) zone. Correlations were obtained by Deere et. al. (1967) and Onodera (1963), which relate the joint frequency of the rock mass to the ratio of wave speeds in the jointed rock to that measured on intact laboratory specimens (Figure 2.14). This correlation, mentioned earlier in Chapter 2.2, was adapted by Drake (1973), who studied an explosion-induced wave propagation in jointed rock.

Drake (1973) considered only joints that are much smaller than the wavelengths of the ground motions. He considered a medium with a random distribution of joints and suggested a simple joint model in which the arrival time derivation due to the joints is:

$$t_j - t_h = N\Delta t \qquad 2.1$$

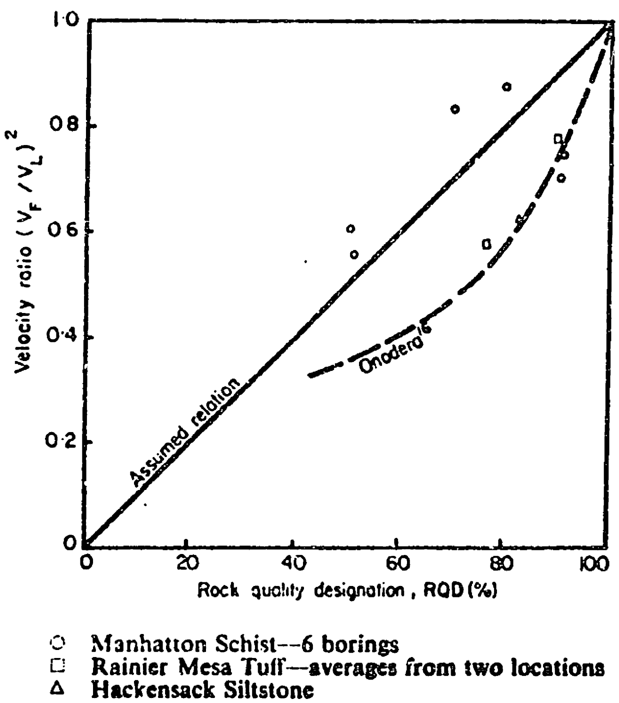

FIGURE 2.14 Correlation of Rock Quality as Determined By Velocity Ratio and RQD (after Deere, 1968)

where N and t_j are random functions. Using this model, he found that the average velocity in the jointed rock may be calculated by:

$$\frac{1}{C_j} = \frac{1}{C_h} + \Delta t \cdot \Psi . \qquad 2.2$$

The results of Drake's work are summarized in Figure 2.15 for various values of the parameter $C\Delta t$, based on the assumption that Ψ is linearly related to the RQD.

$$RQD \cong 100\left(1 - \frac{\Psi}{6}\right) \text{ for } \Psi < 6. \qquad 2.3$$

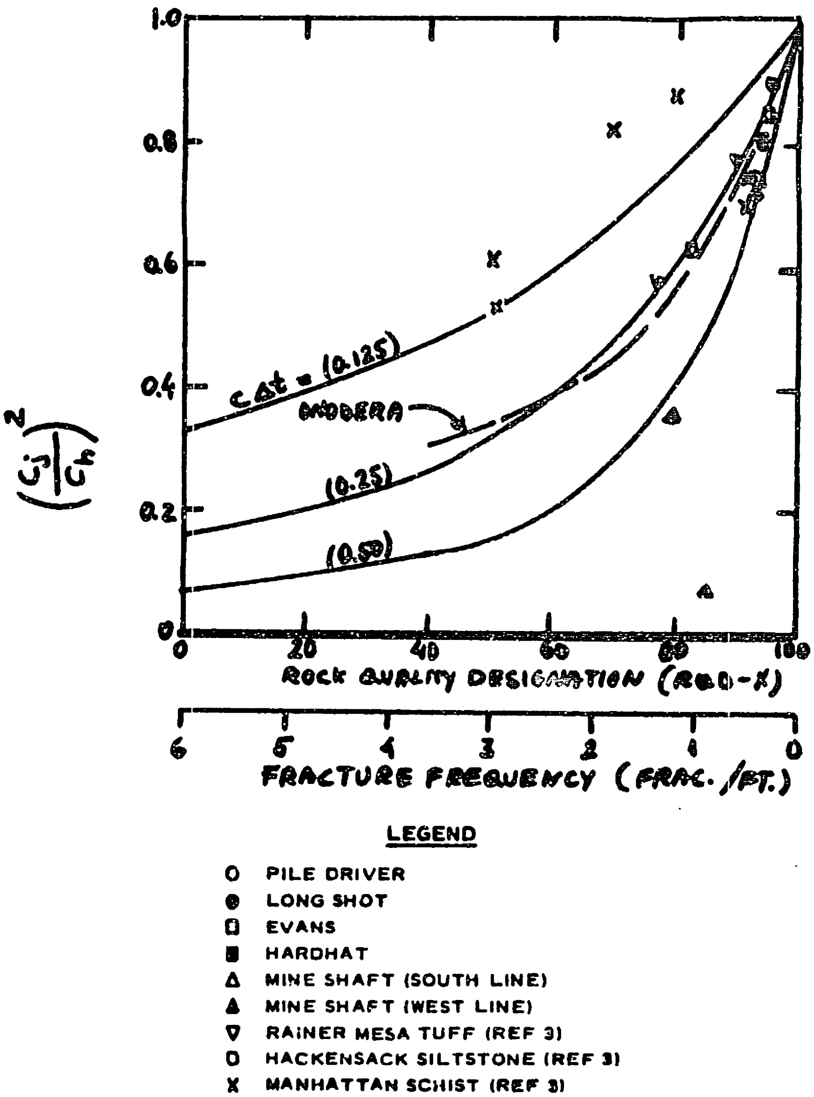

FIGURE 2.15 The Effect of Joints on the Average Propagation Velocity (after Drake, 1973)

Small values of CΔt indicate tight joints which transmit competently the wave energy. Large values of CΔt indicate wide and open joints which disperse the wave energy. The

Drake model was not tested enough in field conditions, and hence it is only a theoretical suggestion which might be used with caution.

The writer was unable to find any analytical expression or experimental measurements concerning the influence of discontinuities on low-frequency, long-wave propagation. It is the writer's intuitive opinion that cracks and other small-scale discontinuities have no influence on the propagation of low frequency (earthquake-type) waves. On the other hand, faults, joints, layers' interface, and other large-scale discontinuities influence the propagation of long waves in a similar way, qualitatively, as was described in this chapter for shock waves.

2.6 DYNAMIC FAILURE MECHANISMS

Introduction: The mechanisms of fracture and failure in rock are markedly different from those connected with failure under static load. Hence, it seems appropriate to lay down several basic rules and accepted principles which may help us to understand, at least conceptually, the reaction of the rock media around a cavity.

Brittle failure of intact rock will not be discussed. This is a vast subject by itself which is well-documented in reviews by Jaeger (1967) and Hoek (1975). One has to accept the idea that rock is a brittle material which contains and is influenced by a large number of randomly-oriented, microscopic cracks and macroscopic weakness planes.

The usual approach to rock failure is made using the Griffith's theory which states that crack propagation occurs when very high tensile stresses are induced at the tips of pre-existing cracks or flaws in the material (Griffith, 1921). This theory is further modified by McClintock and Walsh (1962) to account for the closing of cracks in a compressive stress field and verified experimentally by Hoek (1964), who used it to explain failure around openings in rock.

Further propagation is stable as long as there is a definite relationship between the applied (uniaxial) tensile stress and the half-length of the pre-existing crack; but then the relationship ceases to exist, the crack growth becomes important. Crack propagation is, therfore, a dynamic phenomenon on an interparticle scale and, as such, has been treated by Yoffe (1951), Roberts and Wells (1954), and Baker (1962).

A dynamic stress may be applied to the rock by earthquake, detonation, high velocity impact, or even sharp mechanical blows. Regardless of the method of application, the dynamic stress disturbance has very similar, almost identical, properties. In our discussion, we will distinguish between compressive wave with a particle motion parallel to the direction of wave propagation and shear wave with a particle motion perpendicular to the direction of propagation. In each case, one has to consider two different velocities: (1) the wave velocity, which is the velocity by which the stress is transmitted to other parts of the medium; and (2) particle velocity, which is the velocity with which a point in the material moves as the stress moves across it. Since the velocity of propagation of compressional waves is greater than that of shear waves, it is expected that the compressional wave will reach the free face of a cavity and hence cause fracturing in tension before the arrival of the shear wave. Hence, we may neglect the influence of the shear wave and concentrate on a first approximate analysis on the compressional wave only.

The velocity of propagation may be expressed in terms of the period (or frequency) and wave-length of the wave as shown in the following equations:

$$C = \frac{\lambda}{T} \quad \text{or} \quad C = \lambda f \qquad 2.4$$

It is important to stress here that the wave propagation is a property of the medium in which the wave is propagated, and it is directly related to other elastic constants of the material. For example, for wave propagation in an infinite, homogeneous, isotropic, elastic medium, the wave propagation velocities are defined by:

$$C_p = \sqrt{\frac{\lambda+2G}{\rho}} \qquad 2.5a$$

$$C_S = \sqrt{\frac{G}{\rho}} \qquad 2.5b$$

The wave propagation velocity is independent of the source of dynamic excitation. It is exactly the same, being caused by either earthquake, nuclear burst, HE detonation, or mechanical impact. The differences between the various sources of excitation are expressed in both the frequency and wavelength; but this dependency is cancelled out in their product, which remains constant for a given rock.

The particle velocity is a function of many parameters such as type and shape of the wave, travel distance to origin of excitation, attenuation properties of the medium, etc. But, for a specific given rock, there is an upper bound to the particle velocity which is a characteristic property of the rock. This point will be discussed later in this paragraph, as the behavior of rock mass depends upon this value of the particle velocity, as well as on other input features (Ambraseys and Hendron, 1968; Puchkov, 1965).

The most significant mechanism of damage under dynamic loading is the spalling, sometimes called "scabbing". Brobberg, 1955, 1960; Rinehart, 1952). The spalling is caused when a compressive wave is reflected in a free surface causing a returning tensile wave. This wave may exceed the tensile strength of the rock, thereby forming a fracture and creating a spall. Some of the dynamic impulse's energy will give the spall an initial velocity throwing it into the cavity, while a portion of the remaining wave energy in the original material will further initiate cracking and spalling until the incident stress is reduced to some value less than the tensile strength. (See for example, Rinehart and Pearson, 1954; Rinehart, 1960.) This mechanism is the most important in short duration, sharp fronted, shock waves caused by explosives; but its relative importance decreases when the wavelength becomes much longer. Nevertheless, observation of damage and crack patterns in tunnels proves that this phenomenon is common even in earthquakes which have a predominant long wavelength.

Rinehart (1960b) noted that the factors which are important in describing spalling are: (1) the shape of the stress wave, and (2) the critical normal fracture stress which is related to the critical particle velocity. For a given wave shape, the critical stress and critical particle velocity are correlated by:

$$\sigma_c = \rho V_c \quad \text{or} \quad V_c = \frac{\sigma_c}{\rho} C \qquad 2.6$$

He also gave the following critical velocities (Table 2-1):

ROCK TYPE	COMPRESSION WAVE (ft/sec)	SPECIFIC GRAVITY	TENSILE STRESS (lb/in^2)	CRITICAL VELOCITY (ft/sec)
Granite	10,400	2.7	590	1.6
Limestone	16,200	2.6	350	0.6
Sandstone	9,600	2.5	280	0.9

TABLE 2-1 CRITICAL PARTICLE VELOCITIES FOR ROCKS (after Rinehart, 1960)

It is important to note that the relative weakness of rocks in tension as compared with their strength in compression is the main reason for the easy, brittle fracturing of rock under impact loading. Hence, the fracture criteria may be defined as the tensile stress which exceeds a critical minimum value and may be correlated by Equation 2.6 to the critical particle velocity which develops within the rock medium.

Puchkov (1965) suggested a relationship for the intensity of a wave travelling in a medium:

$$V^2 \cdot c \cdot \rho = \text{constant} \qquad 2.7$$

By using Equation 2.6, it is seen that

$$V = \frac{\sigma}{c \cdot \rho} = \frac{\sigma \cdot g}{c \cdot G_m \cdot \gamma_w} \qquad 2.8$$

It is then possible to establish an upper, vertical particle velocity which is correlated to the critical stress at failure. Obviously, a safe velocity may be determined from the critical velocity by the use of a suitable safety factor.

To obtain some feeling about the order of magnitude of the critical velocity, Table 2-2 summarizes some data from D'Andrea, et. al. (1965).

These data were obtained in laboratory tests on intact cores, and obviously the strength and longitudinal velocity are higher than those expected in-situ. Hence, the critical velocity in the jointed rock mass must be reduced.

ROCK TYPE	TENSILE STRENGTH psi	SPECIFIC GRAVITY	LONGITUDINAL VELOCITY fps	CRITICAL VELOCITY in/sec
Granite gneiss	2054	2.65	18367	37.7
Slate	1518	2.71	18100	27.7
Limestone	1314	2.74	20928	20.5
Granite	1308	2.70	15888	27.2
Limestone	1408	2.71	20808	22.3
Taconite	2474	2.95	20140	37.2
Limestone	911	2.65	16242	18.9
Sandstone	60	1.88	5532	5.2
Basalt	1502	2.88	17150	28.1
Dolomite	506	2.51	16392	11.0
Sandstone	165	2.18	6907	9.8
Schist	1273	2.74	18478	22.5
Chalk	132	1.68	8205	8.6
Limestone	510	2.30	13100	15.1

TABLE 2-2 ROCK PROPERTIES AND CRITICAL PARTICLE VELOCITY

A practical first estimate may be obtained from the work of Deere, et. al. (1967). It is known that the Rock Quality Designation (RQD) may be correlated to the velocity ratio $(V_t/V_L)^2$. Hence, for an excellent rock of RQD 90-100, the critical velocity in-situ will be 0.95 of the velocity in lab.

On the other hand, a rock mass with three joints per foot (ten joints per meter) with a RQD of 60-65 will have approximately an in-situ velocity of 81% of the lab velocity.

In these cases, the critical in-situ velocity will be reduced as shown in Table 2-3.

ROCK TYPE	INTACT	RQD-90	RQD-60	RQD-40
		Excellent	Fair	Poor
Granite gneiss	37.7	35.8	29.2	23.8
Slate	27.7	26.3	20.1	12.9
Limestone	20.5	19.4	15.1	9.5
Granite	27.2	25.8	20.0	12.6
Limestone	22.3	21.2	16.4	10.4
Taconite	37.2	35.3	27.3	17.3
Limestone	18.9	17.9	13.9	8.8
Sandstone	5.2	4.9	3.8	2.4
Basalt	28.1	26.7	20.6	13.1
Dolomite	11.0	10.4	8.1	5.1
Sandstone	9.8	9.3	7.2	4.6
Schist	22.5	21.3	16.5	10.5
Chalk	8.6	8.2	6.3	4.0
Limestone	15.1	14.3	11.0	7.0

TABLE 2-3 CRITICAL PARTICLE VELOCITIES FOR VARIOUS ROCKS AND RQD VALUES

The failure and spalling phenomena become complicated by boundary stresses. If only one free face exists and this face is irregular in shape, boundary stresses caused by the irregularities of the surface have a strong influence on the manner of failure of the rock. This complex phenomenon is treated by many books on blasting and theoretical wave diffraction; but, as explained in paragraph 2.4, it is of minor influence in quasistatic cases of low frequency, long waves encoutered in earthquakes.

Boundary conditions such as the shape of tunnels and existence of sharp, irregular edges and corners have a large influence on the damage caused by blasting. In field experiments by the Colorado School of Mines (1948), two different types of failure occurred in the rock around the cavity. For a relatively short unsupported span of the tunnel, the failure was characterized by tension scabbing at the tunnel's side walls. The "horizontal compression" phenomenon, as it was called, was confirmed by Hasselbacher (1951) in photoelastic experiments. When the unsupported span was increased with all other variables remaining unchanged, the failure started first in the top of the opening rather than in the ribs. Duvall (1948) tried to explain this phenomenon by saying that the compressive stresses which were far below the ultimate compressive strength of the rock had induced perpendicular tensile stresses in the ribs exceeding the relatively small tensile strength of the rock. When the span was small enough, the tension scabbing at the ribs took place before the tensile stresses in the rock over the top of the tunnel reached the tensile strength of the rock. But, when the span is large and the unsupported rock above the opening may be assumed to act somewhat similar to a continuous beam, a relatively small applied stress could produce at the midpoint of the tunnel's crown a tensile stress exceeding the strength of the rock, and the second type of failure would occur.

Leavey (1951) wrote about several attempts to determine the natural frequency of a mine roof and the possibility of resonance between the natural period of the mine roof and vibrations produced by blasting. With the mechanical methods used during the late 40's to early 50's, the roof stratum did not vibrate as a single unit over the entire roof area. Instead, it vibrated as a small local unit in which the frequencies varied to a minor degree. Hence, loose pieces might fall, but the roof as a whole did not react dangerously to imposed vibrations.

The resonance phenomenon is of utmost importance in failure of aboveground structures during long-duration dynamic excitation. Intuitively, it was thought to be important to check its influence in underground construction.

During the Rio Blanko Event (Munson, 1975) in Colorado, measurements were made in three underground mines: the Colony Mine with 70'-50' span, 58'x58' pillars, 90' height, and overburden of 600'-800'; the Anvil Points Mine with 60'-70' span, 60'x60' pillars, 30'-60' height, and overburden of 300'-700'; and the Dutch Creek Mine on which no data was available. During the explosion, two to five seconds of low frequency (2-10 Hz) waves failed to cause any measurable resonance. No free field measurements were made.

Similar findings were reported during the Ruliston Event (Merrill et. al., 1970) which failed to cause any resonance in the Rifle Mine. Measurements in tunnels during nuclear detonation out of the "close-in" region may be an important source of useful information to earthquake studies because of the similarity between the ground movements. Figure 2.16 gives the amplitudes of the velocity, displacement, and acceleration versus time, and the frequency content of the Rulison Event-induced ground motions. The qualitative similarity to earthquakes is striking, but the amplitudes are too small compared to

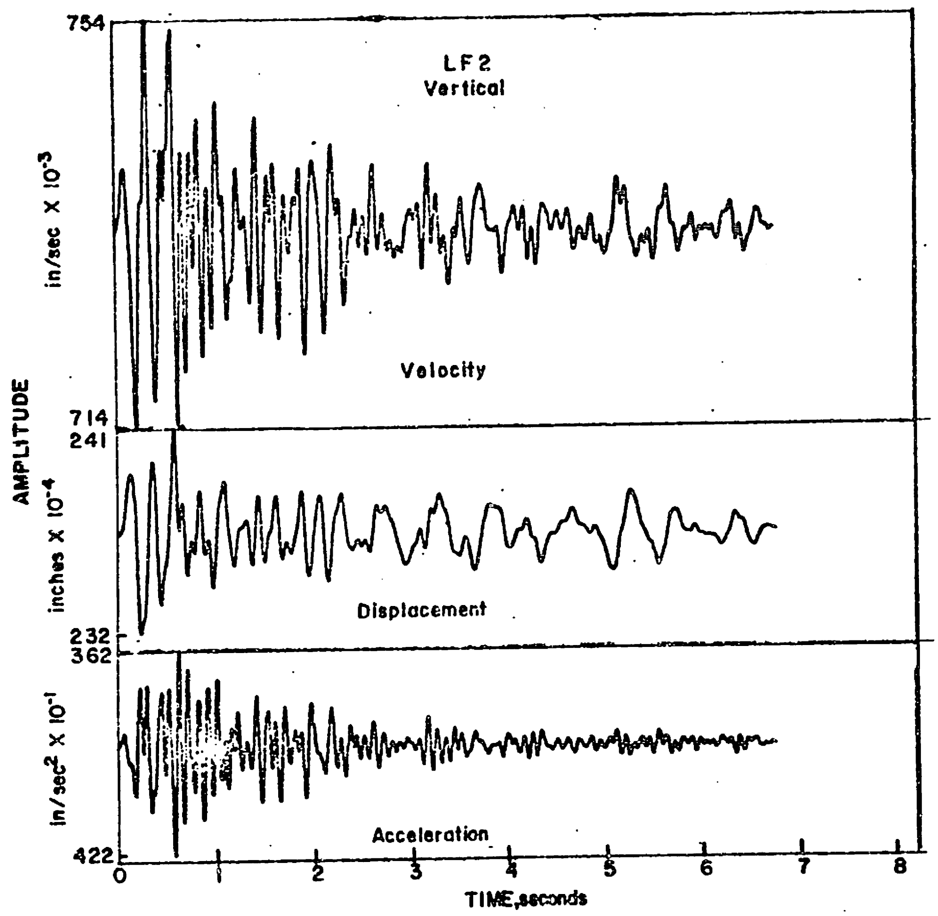

FIGURE 2.16 Amplitude Versus Time and Frequency, LF2 Rifle Mine, Colorado. Rulison Event (after Merrill et. al., 1970)

those expected during a magnitude 6 earthquake.

A numerical analysis of cavities in elastic continuum was made by Glass (1973), who found that "no resonant-type behavior should be expected to occur in underground cavity systems in rock." His results are restricted to excitation frequencies between 1 and 100 Hz which covers both earthquakes (lower range) and blasting (upper range). His results are valid regardless of cavity geometry or orientation.

Hence, resonance problems may be neglected in seismic design of tunnels.

2.7 DAMAGE CRITERIA

2.7.1 Background

This section summarizes various thresholds of damage to tunnels caused by either quarry blasting, nuclear detonations, or earthquakes while an attempt is made to suggest some damage criteria based on this data.

Before establishing these criteria, it is important to remember that damage levels are not a purely technical value defined by precise calculations, but rather their definition and recognition are highly dependent on people. Jackson (1967) has said that "minor damage may be tolerated if caused by...superhuman causes... However, an owner who sees any possible human cause for the smallest... damage may have an associated emotional factor that can magnify any claimed damage by several orders of magnitude... Minor damage may include such items as cracked plaster."

Furthermore, it is of utmost importance to distinguish structural and architectural damage and to define which are the type and level of damage which must be considered in underground construction.

The damage level is dependent not only on the source of damage, "emotional reactions," and distinction between "cosmetic" and structural damage, but it is also a function of the importance of occupancy of the structure. The problem of legal liability for the consequences of an explosion may force the party responsible for it to choose a very low "safe" threshold of damage, eliminating the danger of "damage claims." This approach is influenced by insurance companies and may be demonstrated in the work of Crandall (1949). Similar approaches were used by many other researchers who tried to correlate blasting-induced ground motions near buildings to plaster cracks in the buildings' walls. (See Dowding, 1971, for a summary.)

In public projects, either city, state, or federal, the "acceptable" thresholds of damage are influenced mainly by the importance of occupancy of the facility. There are public facilities where the nature of function is such that damage from earthquakes, in this case, may cause a danger for the surrounding community, a significant loss of strategic or disaster-response capability, an extensive safety hazard, or a large economic loss beyond the facility itself. This is true not only for aboveground structures (as is now defined in earthquake-resistant building codes), but also for underground structures: rapid transit systems, intercities' transportation systems, water supply, water power systems, water cooling systems for power plants, gas and other small installations, not to mention various types of underground chambers. These underground cavities may be divided into groups, depending on "high-loss" or "low-loss" potential, and the acceptable thresholds of damage must be determined accordingly.

Alford (1960) classified the various criteria which were developed by practicing engineers into three groups: (1) Direct Criteria, (2) Indirect Criteria, and (3) Response Spectra Criteria. (1) The Direct Criteria are used for blasting vibrations <u>only</u>, as they are concerned in correlating the damage levels with variables such as weight of explosive, distance from blast to structure, and geological conditions. They are not suitable for earthquake-oriented problems. (2) The Indirect Criteria are based on correlations between the damage level and some property of the ground motion, such as ground displacement, particle velocity, peak acceleration, maximum strain, and strength of the medium. (3) The Response Spectra Criteria are widely applied in earthquake resistant design of aboveground structures, where it is based on the response of a single degree of freedom linearly elastic damped oscillator under the influence of a specific record of transient earth motion. The writer was unable to find any references concerning response spectra analysis for rock tunnels, but some new ideas concerning the use of response spectra for lined tunnels in soft ground will be described later.

The next paragraph (2.7.2) will summarize briefly the potential use of the response spectra and strain criteria which are now used for prefabricated tunnels in soft ground. In paragraph 2.7.3., the various indirect criteria will be described and discussed.

2.7.2 Strain and Displacement Response Spectrum

The use of strains and displacement as failure criteria for lined tunnels was suggested by Kuesel (1969), but strains were already used to establish damage zones in unlined rock tunnels (ERA, 1952 and Hendron, 1970), as will be discussed in paragraph 2.7.3. Kuesel was interested in the sinusoidal displacments produced by seismic waves transmitted to tunnels. The seismic waves may enforce different types of distortions on the tunnel walls depending on the direction and relative orientation of the waves. These deformations may be grouped into two types: Bending and shearing. The damage or failure criteria depend on the elastic or plastic distortions capacity of the lining material. Kuesel recommended that subway structures shall be designed to withstand a strain of:

$$E = \frac{5.2A}{\lambda} \qquad 2.9$$

where the amplitude A is that corresponding to the proper λ taken from a "ground displacement spectrum" based on the ground properties. For prefabricated concrete sections as used in BART, a strain less than 100 millionths of an inch per inch (0.0001 in/in) was considered elastic, and no special provisions were needed to be made in the structure.

Shearing distortions were calculated by:

$$\theta = \frac{5}{2}\frac{H}{C^2} \qquad 2.10$$

The values of C were ranging in BART from 100 ft/sec in soft clay to 1000 ft/sec in compact granular soil. As θ

is inversely proportional to the C^2, it will be negligible in rock mass.

Aoki and Hayashi (1973) improved this approach and introduced a response spectra analysis based on the following assumptions:

(a) The tunnel will not resonate with the earthquake;

(b) The displacement of the tunnel's liner during the quake is, at most, equal to the displacement of the surrounding ground;

(c) The displacement transfer ratio from soil to the tunnel's liner depends on the ratio between the ground and liner rigidities. These assumptions are logical and acceptable, even for tunnels in a rock mass. Differences between tunnels in rock and soil lie mainly in the differences of relative earth and liner stiffnesses.

The Aoki-Hayashi Method is based on a design spectrum for bending of the tunnel:

$$H_\rho(\tau_o) = \left|\rho V_s^2\right|_{max} = \left|\sum G(\tau_o, T_n)\, a_n \sin\left(\frac{2\pi t}{T_n} + \phi_n\right)\right|_{max} \qquad 2.11$$

where:

$H_\rho(\tau_o)$ - is the curvature response spectra shown in Figure 2.18.

$G(\tau_o, T_n)$ - is the transferred function of displacement from soil to tunnel's liner and given by Equation 2.12.

τ_o - is a relative rigidity factor which depends upon the relative rigidity between soil and tunnel, and given by

$$G(\tau_o, T) = \frac{1}{\left(\frac{\tau}{T}\right)^4 - 1} \qquad 2.12$$

$$\tau_o = \frac{2\pi}{V_s}\sqrt[4]{\frac{EI}{E}} \qquad 2.13$$

The influence of τ_o on computing the design spectrum is demonstrated in Figure 2.17, where for stiffer linings (τ_o greater), the displacement response is only for periods greater than 1.5 seconds smaller in amplitude and of lower frequency. The curvature induced in the tunnel by a single sinusoidal wave is:

$$\rho = G(\tau_o, T)\,U\left(\frac{2\pi}{T}\right)^2 \frac{1}{V_s^2} \sin\frac{2\pi}{T}t \qquad 2.14$$

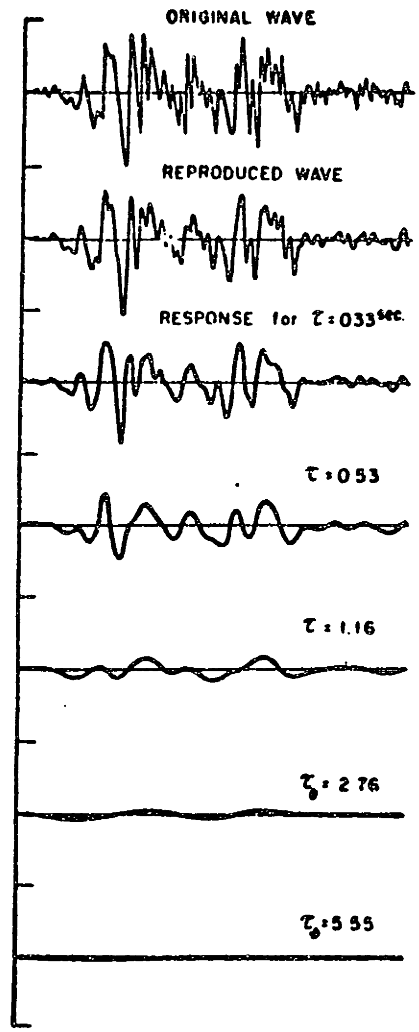

FIGURE 2.17 Computing Process of Design Spectrum (after Aoki and Hayashi, 1973)

Knowing the curvature, one can derive a group of curves (Figure 2.19) which relate the displacement-amplitude and the period for a constant, expected, maximum curvature

$$U = \frac{H_\rho(\tau_0)}{G(\tau_0, T)} \left(\frac{T}{2\pi}\right)^2 \qquad 2.15$$

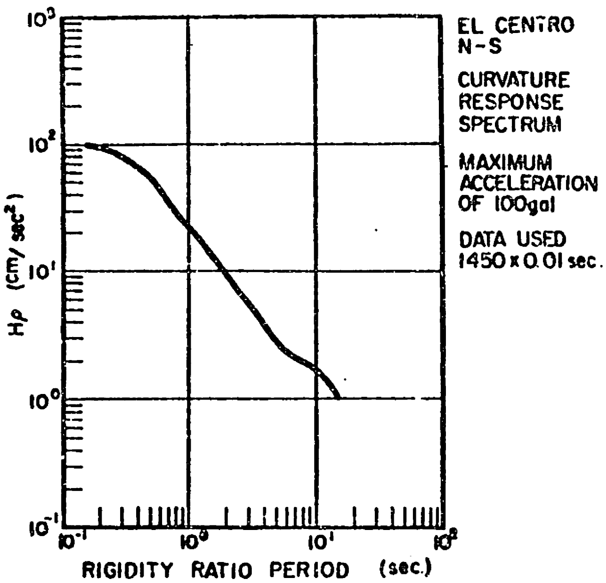

FIGURE 2.18 Curvature Response Spectrum (after Aoki and Hayashi, 1973)

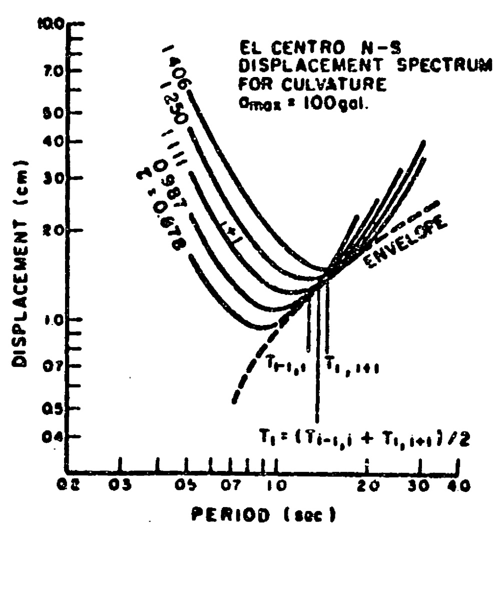

FIGURE 2.19 Relation Between Displacement and Period (after Aoki and Hayashi, 1973)

The curvature-frequency relationship was then used by Aoki and Hayashi to derive similar spectral curves for shear and axial strains and forces. This response spectrum approach may be useful for lined tunnels in which safe levels of dynamic stresses and strains may be defined for the liner structural material. The analytical results were modelled for a trench-type underwater concrete tunnel and were validated, but were never tested for rock medium.

2.7.3 The Choice of Indirect Measure

The main problem in both aboveground structures and underground tunnels was, and still is: "What is the measure, or measures, of vibration that can be related in a meaningful and readily applicable manner to assess their disturbing or damaging potential to man and his property?".

Three measures are commonly used in blasting vibrations phenomena: acceleration, particle velocity, and ground displacement. Two other measures which may be of interest will be discussed later. In earthquake engineering, the peak ground acceleration is, by far, the most widely accepted measure of the ground shaking intensity. The use of acceleration as an index of damage does not mean that acceleration is the cause of damage, but simply that the use of acceleration as an index will give a workable method for determining the imminence of damage. The acceleration approach is especially useful in earthquake engineering where the actual ground motion measurements are made with accelerometers which measure the acceleration-history of the quake and furnishes the engineer with a directly measured peak acceleration.

There are known correlations, will be shown in paragraph 3.6, between peak acceleration and the MM Intensity Scale and hence a correlation between accelerations levels and rate of damage to aboveground structures. A parallel approach may be used to correlate those peak accelerations and their pertinent intensity levels to the level of damage underground.

The main advantage for the use of the peak ground acceleration as a damage index is the availability of its records from earthquake and the familiarity of the earthquake engineers with its use. But, on the other hand, most of the given damage levels in rock mechanics and tunnel engineering, as shown in paragraph 2.7.4, are based on peak particle velocity; and there is no history of the use of peak acceleration as damage index. For earthquakes, velocity and displacement are calculated from the original acceleration records by integrations.

Hence, they are not based on direct measurements and cannot furnish any new information.

Nevertheless, the particle velocity was used by many researchers working on blasting-induced damage; and it is of interest to compare their velocity-oriented damage levels with earthquake ground motions. Paragraph 2.7.4 summarizes the use of velocity as a damage index in rock mechanics and tunnel engineering; and Chapters 4 and 5 compare these blasting-oriented indices to earthquake damage as reported in the case histories. The use of the particle velocity as damage criteria is widely used to assess the damage potential of ground motions that result from blasting operations when one- and two-story residences are endangered. This was already mentioned in paragraph 2.7.1, and a thorough comparison between various studies on this approach may be found in Dowding (1970). The damage potential was found to be related to: (a) Energy Ratio, defined as the square of the ratio of maximum measured acceleration to its pertinent frequency. For a sinusoidal motion, this Energy Ratio is proportional to the square of the maximum particle velocity. (b) The product of maximum measured displacement and its pertinent frequency which is equivalent to a maximum particle velocity. These experimental findings seem reasonable because the maximum radial particle velocity is proportional to the maximum radial strain, as shown by Ambraseys and Hendron (1968).

$$\varepsilon_r = V_r/C \qquad 2.16$$

This relation and its use to define damage zones in unlined tunnels will be discussed later. Even if the dynamic response of aboveground structures is not similar to the dynamic response of a rock tunnel, it is of interest to see if a velocity index may serve as a better criterion than the acceleration.

The use of displacement and strain was already discussed in paragraph 2.7.2, which introduced the displacement response spectra approach; but it is important to note that a strain index was used in establishing damage zones in underground blasting near tunnels.

In the early 50's, the US Corps of Engineers conducted a research which was summarized in several reports by the Eng. Res. Assoc. (1952, 1953). In these tests, high explosive charges were detonated near unlined tunnels in granite, limestone, and sandstone. The damage caused to the tunnels was divided into four zones: 1,2,3, and 4 as shown in Figure 2.20. Zone 1 was characterized by complete collapse of the rock, and Zone 2 by continuous heavy rock breakage. They are of military interest only and will not be treated here. The accelerations and strain levels in these zones are much above those expected in earthquake engineering. Zone 3 is characterized by continuous rock breakage of relatively uniform thickness, and Zone 4 is characterized by irregular spalling of rock which may have been loosened by the driving of the tunnel. The outer limit of Zone 4 is that point beyond which no significant rock breakage occurs. This limit, determined by either the strain or the peak acceleration related to it, may serve as an upper threshold of damage. Most engineered structures will not permit or sustain damage even to the lower extent of Zone 4, and will preferable be even safer by some margin.

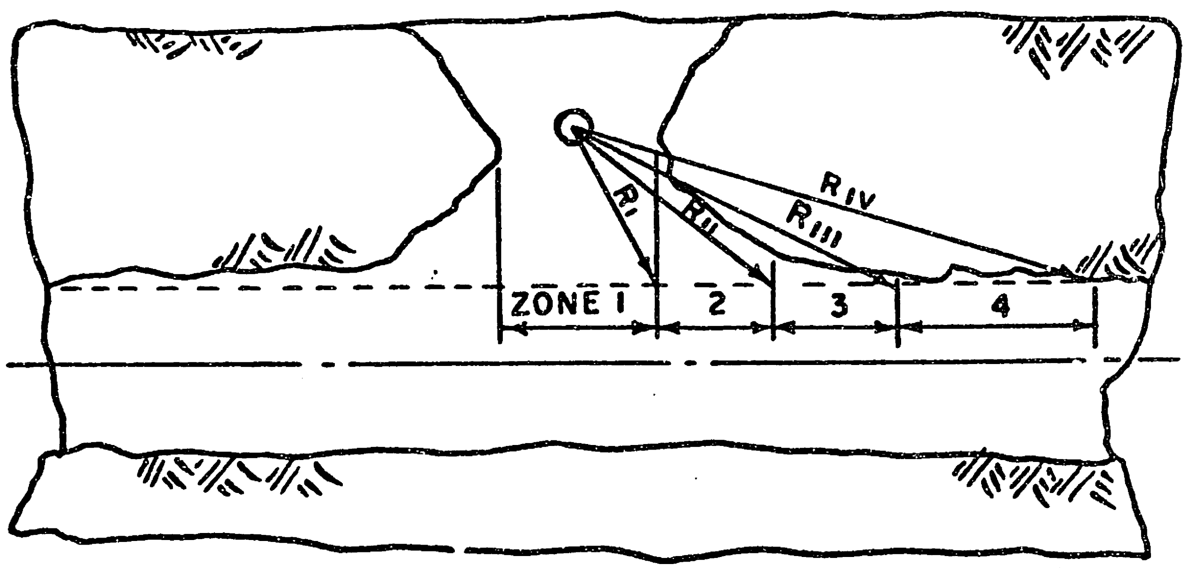

FIGURE 2.20 Empirically Determined Tunnel Damage Zones (after ERA, 1952)

Hendron (1970) interpreted the ERA data by giving the damage criteria as a function of the strain:

Zone	Strain
1	0.01
2	0.004
3	0.0015
4	0.0004

TABLE 2-4 DAMAGE ZONE BOUNDARIES

The strain criterion may be translated into a particle velocity criterion by using the relationship of:

$$\varepsilon_r = \frac{V_r}{C} \qquad 2.17$$

Assuming wave velocities for intact rock as given by D'Andrea et. al. (1965), it is possible to establish the outer limit of Zone 4:

ROCK	C(fps)	V(in/sec)	Earthquake Magnitude at Distance
Granite gneiss	18000	86.4	
Limestone	21000	100.8	
Granite	16000	76.8	>M=8 at fault
Sandstone	6000	28.8	M=6.5 at 12 km

TABLE 2-5 PARTICLE VELOCITIES AT ZONE 4 FOR INTACT ROCKS

Except for soft, weak rocks, the critical velocity, as defined by Zone 4, is not to be expected during earthquakes with the exception of tunnels located very close to source.

But, the data of D'Andrea were obtained of intact rock measured in labs. Using Deere et. al. (1967), correlaing the reduction of in-situ seismic velocity and the RQD of the rock, we may calculate lower values of critical (Zone 4) velocities:

ROCK	INTACT	RQD 60		RQD 40	
		C	V	C	V
Granite gneiss	18000	10800	51.8	7200	34.6
Limestone	21000	12600	60.5	8400	40.3
Granite	16000	9600	46.1	6400	30.7
Sandstone	6000	3600	17.3	2400	11.5

TABLE 2-6 PARTICLE VELOCITIES AT ZONE 4 FOR JOINTED ROCK

It is clear that for fair rock (RQD-60%) or poor rock (RQD-40%), the critical velocities are at the order of magnitude of the velocities expected during earthquakes. Discussion of this point will follow in Chapters 4 and 5.

2.7.4 Survey of Damage Criteria in Tunnels and Rock Structures

Langefores and Kihlstrom (1963) defined particle velocity as a damage criteria for unlined rock tunnels. A particle velocity of 12 in/sec (30cm/sec) is given as a criterion for the fall of stones, and 24 in/sec (60 cm/sec) is correlated with formation of new cracks in the rock. Such peak velocities may be expected at 11 miles (18 kms) from the causative fault and above the causative fault respectively for a magnitude 6 earthquake. (Seed et. al., 1975).

Oriard (1972) collected several case histories about the criteria for rock slope stability:

1. In the case of man-made slopes, a particle velocity under 2-4 in/sec (5-10 cm/sec) does not constitute a primary influence on the slope; but if the slope was already unstable, then the addition of the vibration may change the time-displacement history of the movement.

2. At 2-4 in/sec (5-10 cm/sec), one may expect occasional falling of loose stones on slopes.

3. At 5-10 in/sec (12.5-25 cm/sec), one may expect falling of partly loosened sections of rock underground and on aboveground slopes--sections of rock that might otherwise remain in place.

4. Above 25 in/sec (62.5 cm/sec), one may expect

some damage to occur in the relatively unsound rock types that are found in most open pit mine slopes. Usually in civil engineering works done in higher quality rock, one may expect that it will remain undamaged.

5. It is important to note that all mentioned criteria and especially the last one are not only dependent on the inherent strength of the rock, but also are influenced by the geometric configuration of the slope (or the cavity) in relation to the wavelength and the surface of reflection. The tensile failure by scabbing is well known in blasting; and similarly, a seismic wave may pass through a section of confined rock without causing any damage, but will cause local failure when it will be reflected at a free boundary of the rock surface.

Some other case histories which were described by Oriard (1972) may illustrate the subject a little more:

1. At a large underground chamber at Dworshak Dam, Idaho, two small sections of unreinforced rock, which were loosened by blasting in the access tunnel, fell at 5 in/sec (12.5 cm/sec), but additional blasting at the same intensity did not cause any more rock to fall.

2. In an intake structure leading to underground power house in Manapouri, New Zealand, a particle velocity of 25 in/sec (62.5 cm/sec) did not cause any visible damage

to intact rock or reinforced concrete. One small section of granite was dislodged from one of the pen-stock throats.

3. During the San Fernando Earthquake, 1971, 12 seconds of strong shaking with peak particle velocity of 45-55 in/sec (112.5-137.5 cm/sec) caused damage to steep, weathered slopes and changed their elastic moduli. No observable damage or measured changes in physical proper-ties were found in the competent rock.

Bauer and Calder (1971) suggested several damage levels for blasting vibrations using particle velocity as the criterion:

1. 10 in/sec (25 cm/sec) - no fracturing of intact rock;

2. 10-25 in/sec (25-62.5 cm/sec) - minor tensile slabbing will occur;

3. 25-100 in/sec (62.5-250 cm/sec) - strong tensile and some radial cracking;

4. 100 in/sec (250 cm/sec) - complete breakup of rock mass will occur.

Figure 2.21 summarizes the findings of the research-ers referred to in this paragraph. It is apparent that, in spite of the scatter of the data and the wide dif-erence bewtween the rocks, some average levels can be introduced as shown.

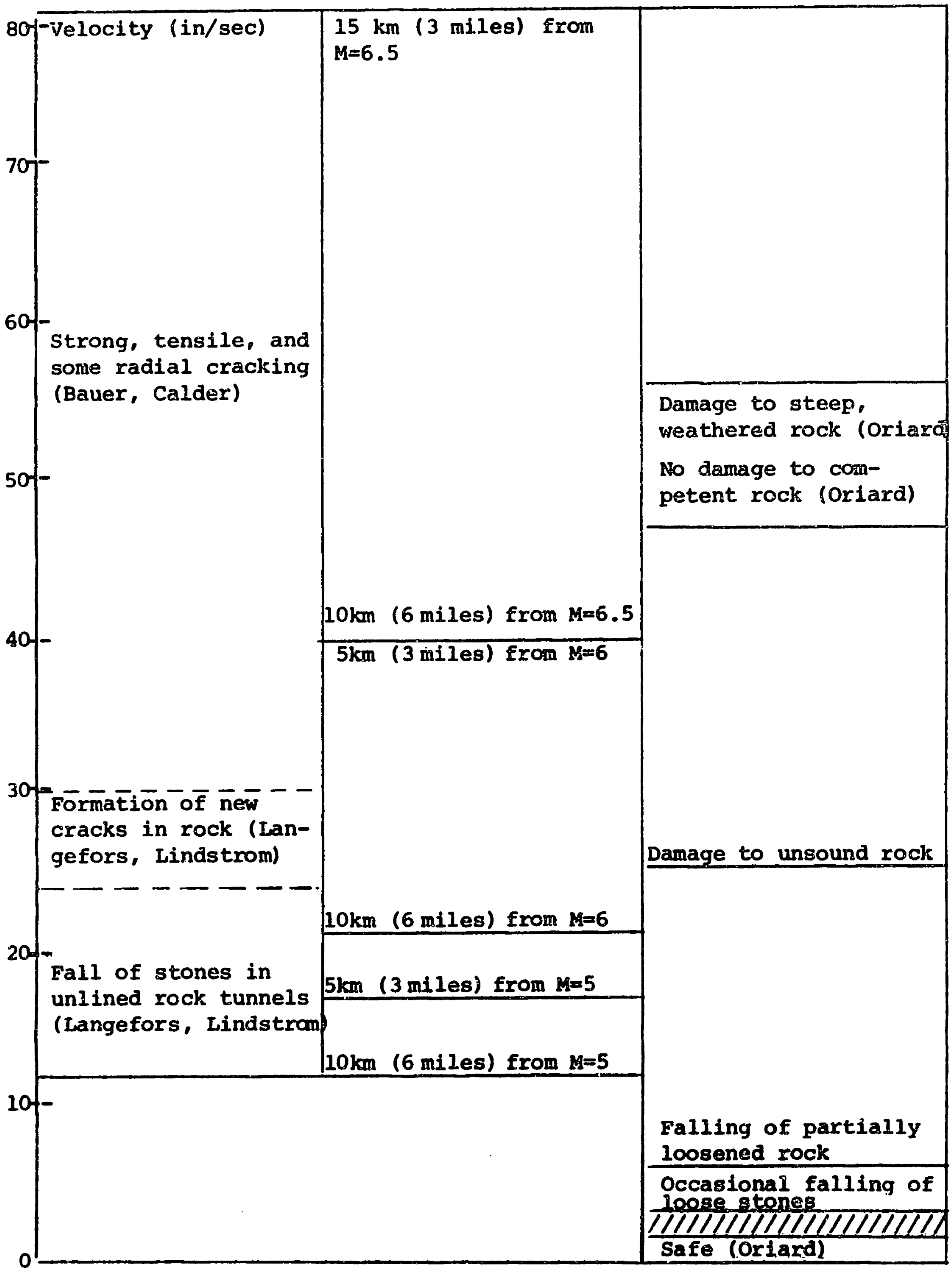

FIGURE 2.21 Summary of Damage Levels

CHAPTER 2

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CHAPTER 3

EARTHQUAKE GROUND MOTIONS

3.1 GENERAL APPROACH

The basic data of earthquake engineering are the recording of ground accelerations during the strong motion period of the earthquakes. A knowledge of the ground motion is essential to an understanding of the "earthquake behavior" of structures. Ground motions are especially necessary for aboveground structures, where this data serves as "input" to determine the design response specturm and hence to determine the ability of the structure to withstand earthquakes.

Even if the response spectrum approach is not widely used for underground structures, the same strong-motion recordings of ground acceleration still serve as the basic "input" data to analyze and determine the destructiveness of the ground shaking.

In Chapter 2.7, the damage potential of blasting-induced ground motion was defined as the function of the peak values of ground acceleration, velocity, and displacement. The same approach may be introduced for earthquake-induced ground motion. Hence, this chapter will define the method of estimating the peak amplitude of the ground motions, which will be correlated in Chapter 5 to the damage described in Chapter 4.

As may be seen from the description of case histories in Chapter 4, many of the earthquakes analyzed occurred before the 1950's, when strong motion instruments were quite scarce, and records near the sites under consideration were not available to serve as input. Furthermore, it is not practical to base an analysis on measured-at-site recordings, since even today it is not practical to put accelographs in all sites. Therefore, one must use a technique which will allow him to use actual recordings from another site and to get a "best estimate" for the level of shaking which caused the reported damage.

It is obvious that recordings are accurate only for the specific earthquake and location at which they have been recorded, and that in another site they are different. Hence, the "best estimate" that one may calculate using the technique, which will be described later, is based on statistical analysis, taking into account many earthquakes from many locations and within wide ranges of distances from a given site.

The problem is, then, to correlate the expected ground motion as defined by peak values of ground acceleration, velocity, and displacement at a specific site for a "design" earthquake, defined by its magnitude and epicentral distance to the site (FIgure 3.1).

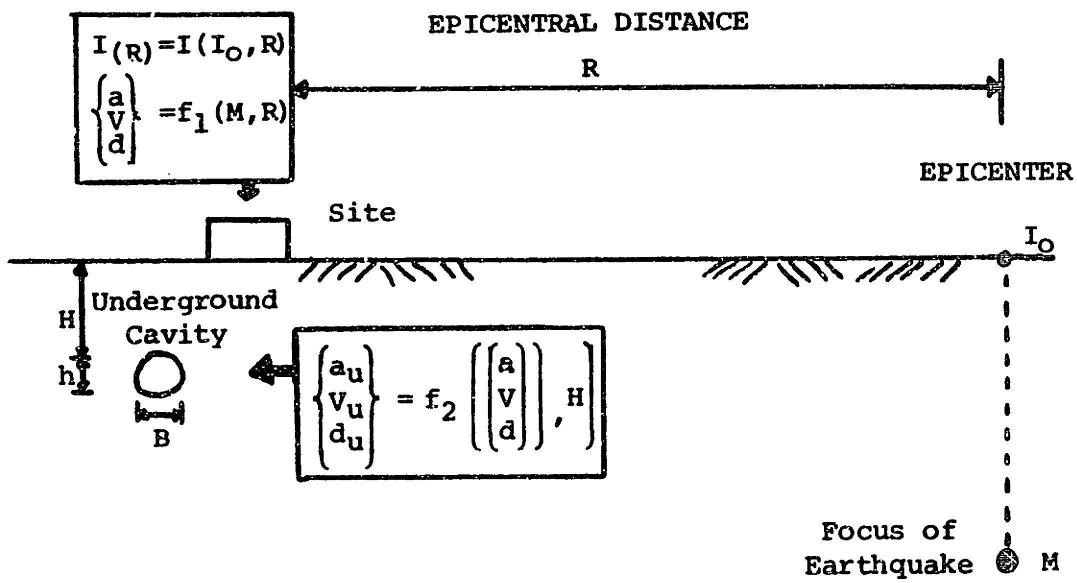


FIGURE 3.1a <u>Sketch of The Use of Attenuation Laws</u>

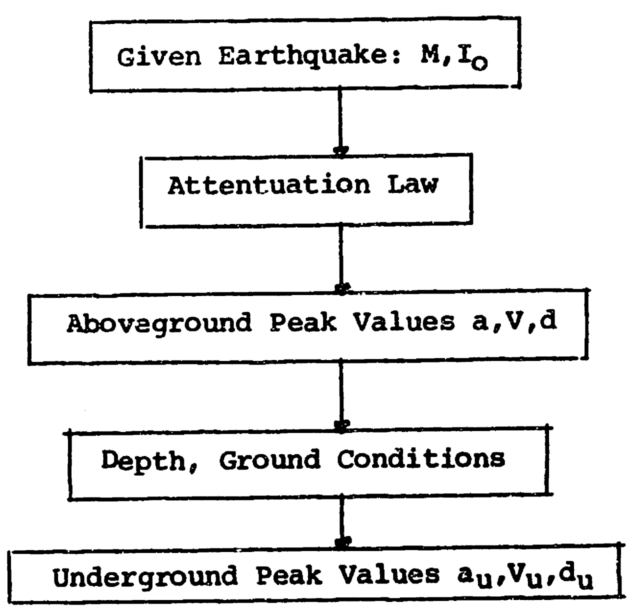


FIGURE 3.1b <u>Flow Chart of Use of Attenuation Laws</u>

Such correlations, which are known as "Attenuation Laws," were developed by many researchers during the last two decades when the accumulation of measured records permitted a stastical comparison. It is common practice to characterize the ground motion by one or several parameters. These might be the peak ground acceleration, velocity, or displacement (Newmark et. al., 1973), some scale of intensity such as the Modified-Mercalli, or other parameters. This characterization provides the analyst some objective means for evaluating the level of shaking or degree of destruction implied by different records.

The ability of the chosen parameter to reflect the intensity of shaking is obviously a function of the parameter chosen. In aboveground structures, it is known that this ability is also a function of the properties of the structure under consideration. For example, acceleration is a better parameter for high frequency structures, while displacement is preferred for low frequency structures.

The author failed to find any material concerning the preferred characterization for underground structures. However, it seems that a parameter which might be useful for unlined tunnels is not necessarily suitable to lined tunnels where the liner/rock interaction must be taken into consideration.

The various difficulties encountered and the assumptions to be made to effectively choose a characterizing parameter were discussed elsewhere (Cornell, 1968; Housner, 1969; McGuire, 1974) and will not be repeated herein. For the purpose of this study, it is sufficient to say that knowing the local seismicity and geology and having a means of estimating the intensity of motion produced by an earthquake originating from these faults, it is possible either to choose a set of peak values for design purposes or to "reconstruct" a set of probably values which might be expected to have occurred at the site.

All the input data used in the comparative table of Chapter 5 are estimated by using attenuation laws, which are described in the next section.

3.2 ATTENUATION LAWS FOR PEAK VALUES

There are usually three peak values which are of interest to engineers: peak acceleration, peak velocity, and peak displacement. In earthquake engineering, most of the studies of attenuation with increasing distance were made with accelerations. Figure 3.2 (from Seed et. al., 1975) summarizes some of the studies during the last two decades. It is clear from this figure that the scatter of the suggested curves is high. This scatter is attributed to the wide range of fault movement mechanisms, site geology, instrumentation, etc.

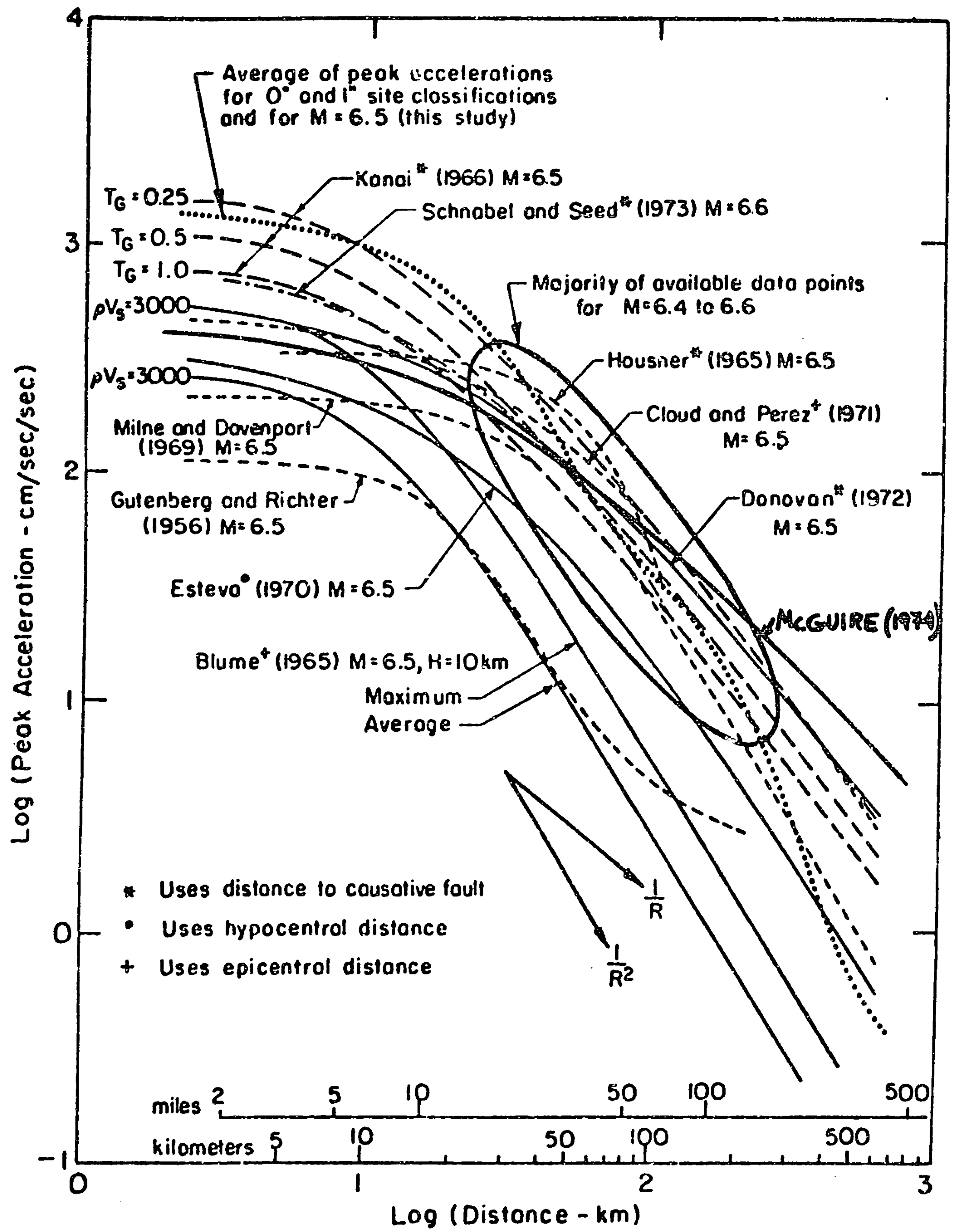


FIGURE 3.2 Relationships Between Peak Acceleration And Distance From Source For Magnitude 6.5 Earthquakes (after Seed et. al., 1975)

Comparatively few studies have been made on the attenuation of maximum ground velocity with increasing distance from the source of energy release. This lack of study may be explained by the fact that more difficulty is involved in computing maximum velocity values from records. A relationship suggested by Ambraseys (1973) is shown in Figure 3.3, but there is no available data concerning its scatter around the average lines.

As already discussed, from the damage criteria point of view, it is of more interest to know the peak velocity than the peak accelration. However, the relative scarcity of the data may lead to much more uncertainty concerning the form of the relationship involved.

One way to overcome the absence of these data is by correlating the peak acceleration and the peak velocity. There is much data on recorded peak acceleration on one hand, and correlation between peak velocity and damage potential on the other hand. Such correlation is not yet well defined, and it is clearly influenced by local geological conditions. Nevertheless, some rough correlations have been suggested by Newmark et. al. (1973) and Seed et. al. (1975), based on the ratio between the mean peak velocity over the mean peak acceleration:

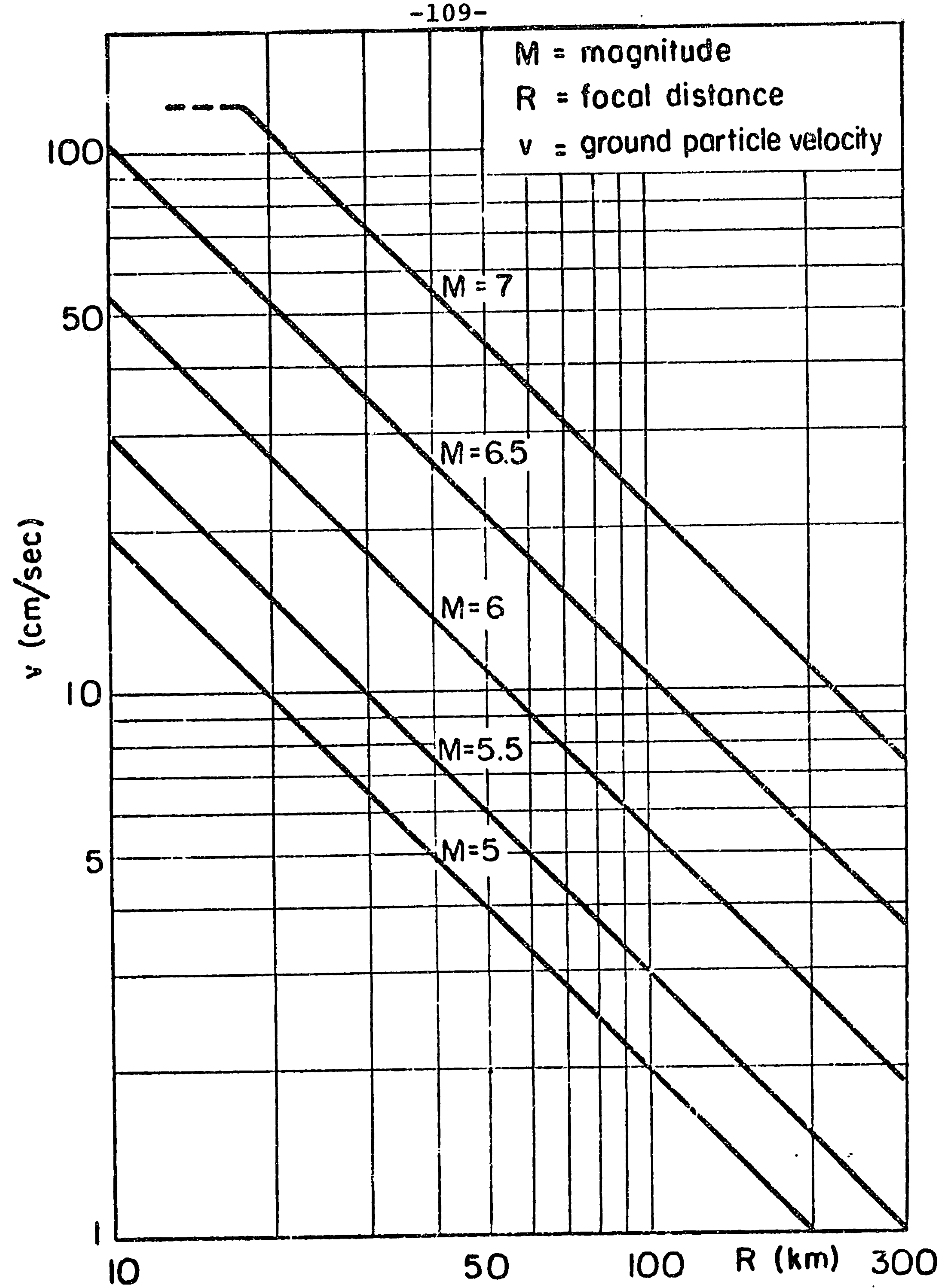


FIGURE 3.3 Maximum Probable Ground Velocities (after Ambraseys, 1973)

a. For rock V_{max}/a_{max} = 26 in/sec/g.

b. For "stiff" soil " = 45 in/sec/g.

c. For "deep" soil " = 55 in/sec/g.

There is no specific reason to accept one of the attenuation laws, described in Figure 3.2, as superior to the others. On the other hand, it is of interest to use a set of attenuation laws which gives information on acceleration, velocity, and displacement. Therefore, the writer chose the work by McGuire (1974) for this study, since he derives attenuation relationships for all three.

McGuire (1974) made a thorough statistical analysis of the uncertainty involved in the determination of his attenuation laws, and hence no attempt will be made here to discuss this point and his equations will be used without comment.

$$a = 472 \times 10^{0.278M} (R+25)^{-1.301} \qquad 3.1$$

$$V = 5.64 \times 10^{0.401M} (R+25)^{-1.202} \qquad 3.2$$

$$d = 0.393 \times 10^{0.434M} (R+25)^{-0.885} \qquad 3.3$$

R, in this case, is the hypocentral (or focal) distance and not the epicentral distance.

Most of the attenuation laws, described in Figure 3.2

were derived from stations located on a very wide range of ground conditions, both soils and rocks, without differentiating them according to the different geological conditions. This is, of course, a serious disadvantage while dealing iwth tunnels; but at the present, very limited data is available to overcome it. Some of the updated limited information concerning the influence of ground conditions will be summarized in Section 3.3.

3.3 INFLUENCE OF LOCAL GEOLOGICAL CONDITIONS

It is known that local soil conditions have a significant effect on damage caused by earthquake shaking. There are many observations on excessive damage caused to aboveground structures built on soft layers of soils compared to lesser amounts of damage caused to similar structures on stiffer soils or rock. The same tendency was observed in damage experienced by small diameter pipes, box-culverts and other types of underground structures in soils with different stiffnesses.

The interest in rock motions is small compared to the studies conducted on soft ground motions. The data collected about rock motions is quite limited, and most of the data collected in the United States (Seed et. al., 1968, Seed and Idriss, 1969, Schnabel and Seed, 1972, 1973) is used as input data to soil amplification studies. No attempt had been made in the United States, to the author's knowledge, to learn about the changes of rock motion in the rock mass itself. Furthermore, the rock motions collected in the US are measured usually on outcrops of rocks, and there are no US measurements in the rock mass itself. Some important Japanese contributions to this subject will be discussed in Section 3.4.

Trifunac and Brady (1975), as quoted by Seed et. al. (1975), compared the results obtained for hard and soft rock and alluvial deposits and concluded that: (a) For magnitudes larger than 6, it seems that the peak acceleration on the three different site types are all the same; and (b) For magnitudes greater than 5, the peak velocity and displacement in alluviam sites are greater than in rock sites.

Duke et. al. (1972) and Donovan (1973) found that for epicentral distances of 25 to 38 miles (40 to 60 kms), the peak accelerations on rock are somewhat higher than those recorded on soil, while the reverse is true for distances above 38 miles (60 km). (Figure 3.4) From the figure, which is based on records obtained during the San Fernando Earthquake of 1971, it is clear that the attenuation of peak acceleration with distance on alluvial soils is slower than those on rock.

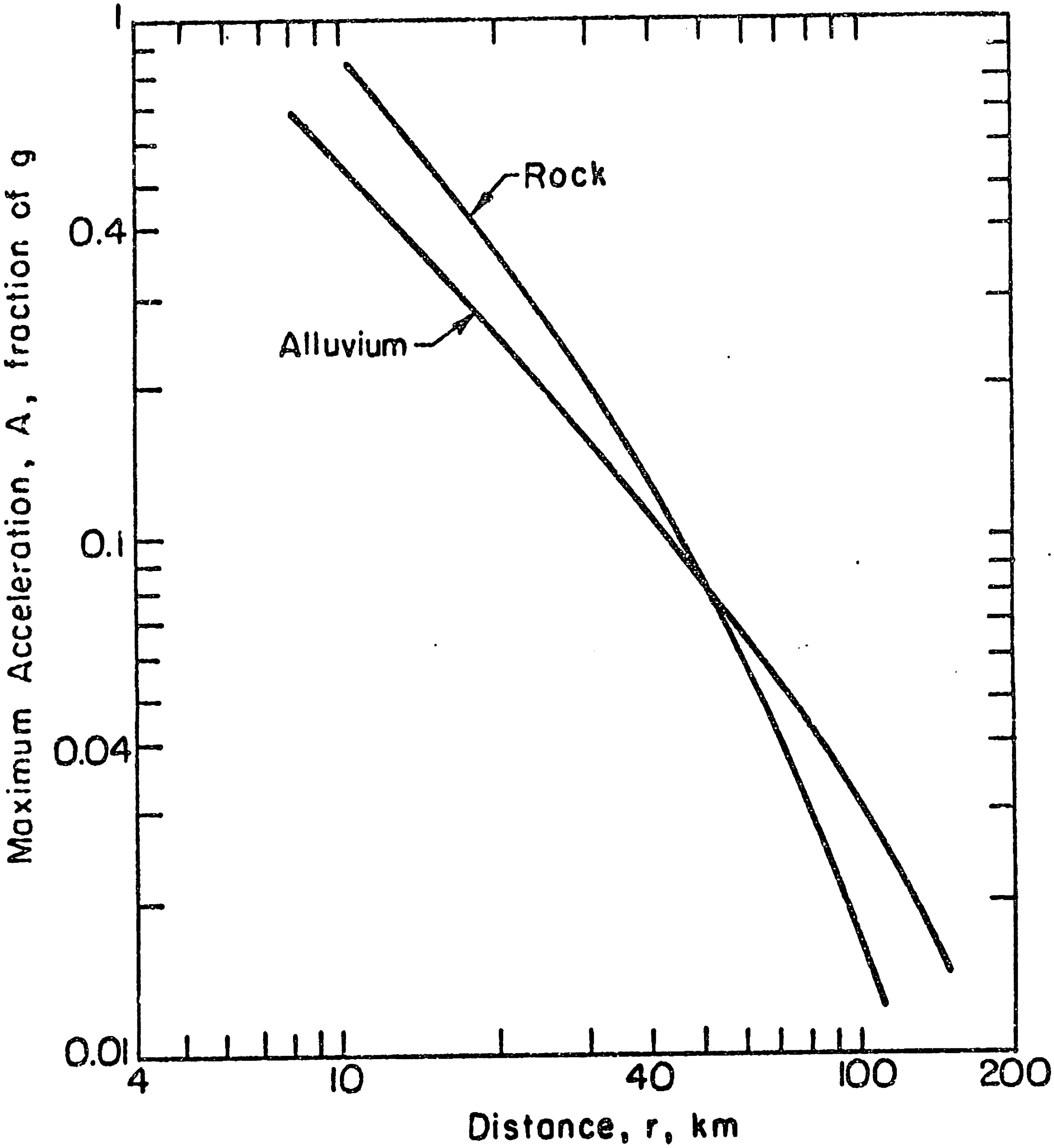


FIGURE 3.4 Fitted Attenuation of Maximum Acceleration For Different Site Classifications (after Duke et. al.)

Seed et. al. (1975) extended the work of the recent researchers and suggested a relationship between local ground conditions and the peak velocity (Figure 3.5). In this work, rock was a shale-like material with shear wave velocity of 2500 ft/sec (762.5 m/sec), and the alluvium sites were defined as "stiff" for an alluvium layer up to 150 ft (45.7 m) depth and "deep" where the layer's depth was greater than 250 ft(76.2m). Because of the scarcity of data (27 points for rock, 28 points for deep alluvium, and 35 points for stiff alluvium), and especially because most of the data was "bunched" together in a limited range of distances from the source, no statistical studies have been made on this relationship. A study on the influence of local site conditions on peak acceleration has the same qualitative tendency as found by Duke et. al. (1975).

Blume (1965) counted several attempts to analyze the influence of local geological conditions. Of interest may be the works by Gutenberg (1957) comparing alluvium and rock and Newman (1965) correlating the alluvium with granite rock. Unfortunately, most strong motion records have been made on soils with generally little known characteristics, and only very few strong motion recordings have been made on rock. According to Blume (1965), only

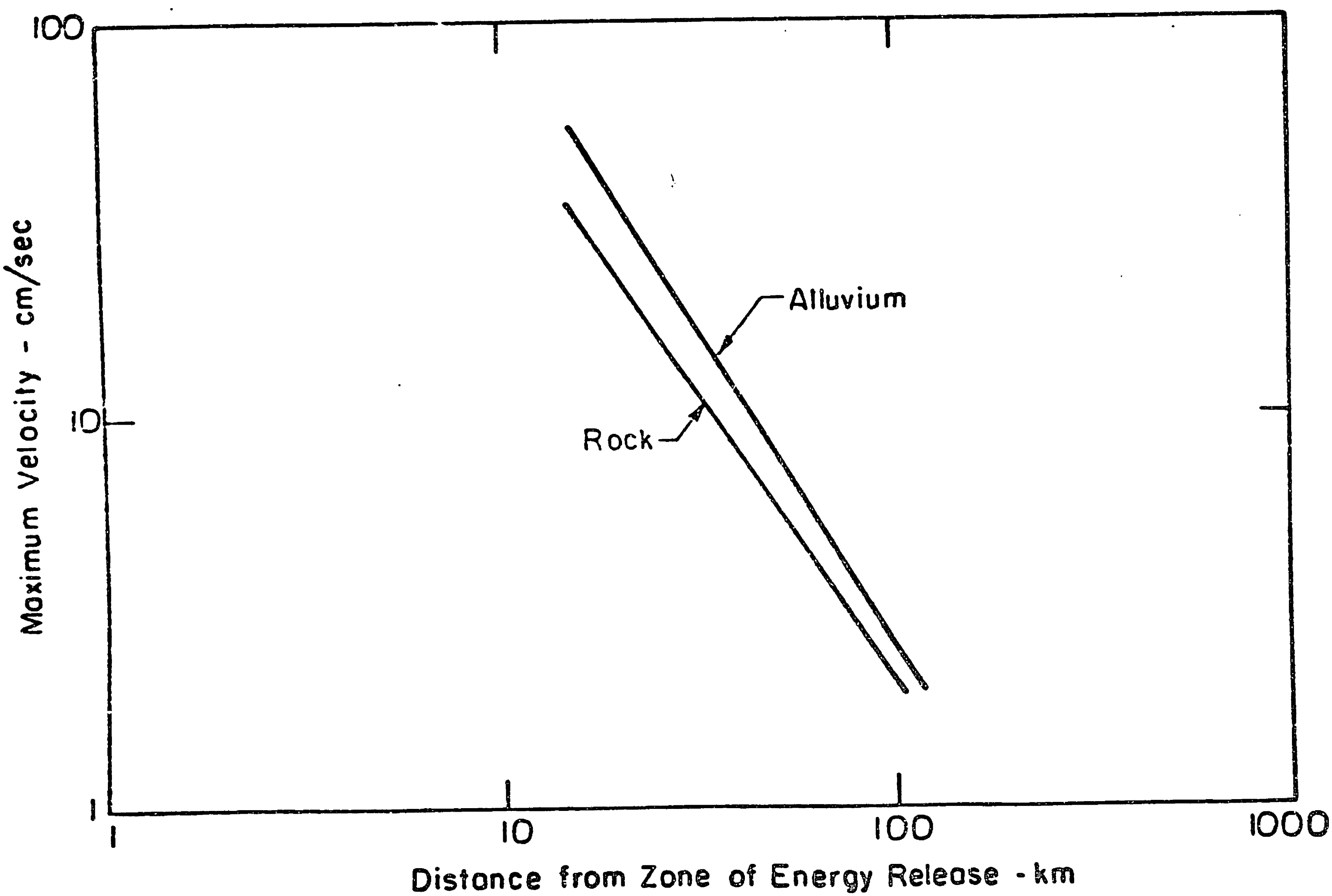


FIGURE 3.5 Influence of Local Soil Conditions on Maximum Ground Velocities For Magnitude 6.5 Earthquakes (after Seed et. al., 1975)

two cases were recorded on rock: In the Helena, Montana Quake of 1935, Magnitude 6 was recorded on limestone; and a Magnitude 5.3 quake in 1957 was recorded at San Francisco on weathered Jurassic sandstone. Gutenberg and Richter (1956) suggested an attenuation law which takes site conditions into account by using an "average" value for b.

$$\log a = -b + 0.81M - 0.027M^2 \qquad 3.4$$

where:

b = 2.1 for weathered granite rock at Pasadena, California;

b = 1.7 for alluvium at Southern California;

b = 1.4 for "relatively unstable ground" such as at El Centro, California.

For a given earthquake, the peak accelerations in rock are only 40% of those in alluvium and 20% of those in unstable ground. The last ten years did not change this unfortunate situation; and in an updated work by Seed et. al. (1975), they point out the fact that "extremely limited knowledge is available concerning the soil conditions at most of the recording stations." For example, from 38 strong-motion stations in Northern California in 1971, the nature of soil conditions underlying these stations was known for only ten (20%), and the data on only 4 (11%) of these sites was available in the general literature.

It is, then, the writer's opinion that it is still premature to conclude any meaningful quantitative relationship between attenuation and local geological conditions. Therefore, the data given here must be used with care!

3.4 INFLUENCE OF DEPTH IN ROCK MASS

Sections 3.1 - 3.2 discussed the ground motion in what may be called "typical average conditions," while in Section 3.3 some data have been given on the influence of "gross"local geology. There still remains a question about the influence of geological conditions at depth, physical properties of the rock, discontinuities, layering of different rocks, vertical and horizontal dimensions of the rock body, etc.

For many researchers who are interested in soil amplification problems, the baserock serves as a "black box" of which only the "output" motion at the upper surface is of importance. For a tunnel in rock, the changes, if any, of the motion in the rock layer are of utmost importance: Does a shallow tunnel in rock suffer from stronger movement than a deep tunnel?

There are very few comparative measurements both in the surface and in depth; and hence, some data will be given here for measurements in both soils and rocks so that the reader will be able to get some engineering feeling of what was really measured.

Most of the measured evidence of depth influence in various types of soils and rocks is based on findings by Japanese researchers. In his updated book, Okamoto (1973) counts several cases which follow:

a. Omori measured simultaneously the ground motion at surface and at the bottom of a 5.4 m (17 ft) well in hard loam. The values of peak displacement, velocity, and acceleration at the well's bottom were only 0.45, 0.36, and 0.21 of their respective values at the surface.

b. Okamoto measured simultaneously the ground motions at surface and at the bottom of a 10 m (32 ft) deep borehole. The surrounding soil was a 30 m (100 ft) layer of silt underlain by a hard sand stratum. Figure 3.6 demonstrates his findings. The peak acceleration at depth is reduced by a factor of 2, and the quake duration becomes much shorter. Several other measurements at the same vicinity have given a reduction factor with depth of 2 to 4.

c. Saita and Suzuki (1934) made similar observations in alluvial ground at surface 10 m (30 ft) and 20.4 m

(60 ft) below ground. For ground motions of predominant period in the range 0.2 - 0.7 seconds (or frequencies of 5 to 1.4), the periods remain unchanged; but the acceleration at 20 m depth was 33% to 50% of that at surface.

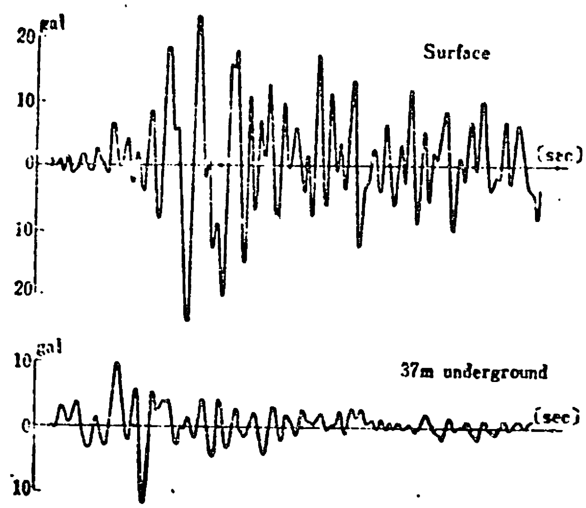


FIGURE 3.6 Surface and Underground Acceleration Records Obtained At Urayasu, Chiba Prefecture (after Okamoto, 1973)

Extensive comparative research was conducted by Kanai et. al. (1966) at four stations located in a circle of 300 km (187 miles) around Yokohama. The ground profiles were different from site to site ranging from sand and clay to mudstone and granite. In all cases, the amplitudes at depth were smaller and the spectral velocity versus period was flatter. Of special interest are the recordings obtained at Tsuruga B (Figure 3.7a) and Futaba A (Figure 3.7b). At the first point measurements were made at 19.4 m (63.6 ft) depth and 36.8 m (120.7 ft) depth corresponding to the top and bottom surfaces of a weathered granite layer. At the second point, similar measurements were made at the top and bottom of a mudstone layer ranging from 20.2 m (66.3 ft) to 52.4 m (171.9 ft).

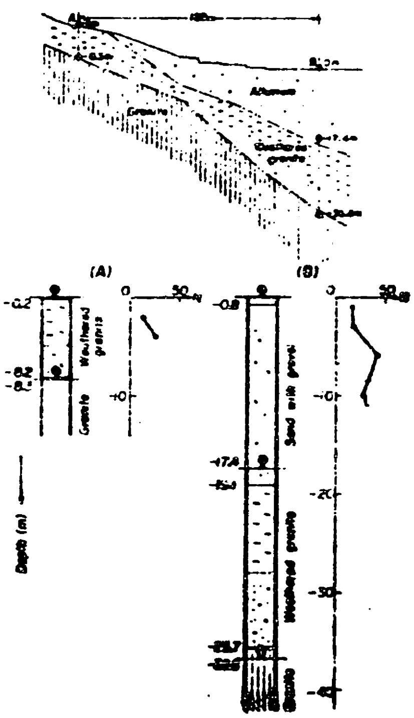


FIGURE 3.7a Station Tsuruga. Geological Formations and Subsoil Conditions (after Kanai, 1966)

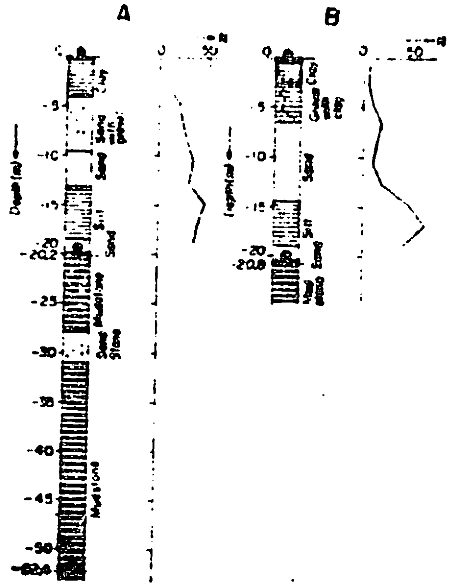
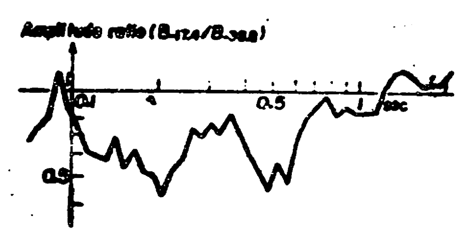


FIGURE 3.7b Station Futaba. Subsoil Conditions. (after Kanai, 1966)

In Tsuruga B, deamplification existed in the weathered granite layer with the amplitude at the bottom of the granite layer 0.5 to 0.9 of the amplitude at the top. At Futaba A, the amplitude at the top of the mudstone layer was fluctuating between 1 and 4 of the amplitude at the bottom (Figure 8a,b,c). In both stations the top to bottom amplification ratios were a function of the quake period.

FIGURE 3.8a Station Tsuruga B. Average of the Amplitude Ratios Between the 17.4 m Depth and 36.8 m Depth. (after Kanai, 1966)



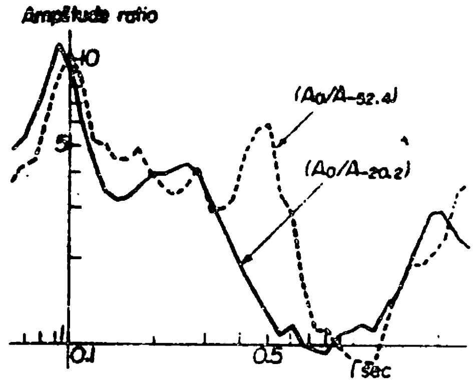


FIGURE 3.8b <u>Station Futaba A. Average of the Amplitude Ratios Between the Surface and 52.4 m Depth, and the Surface and 20.2 m Depth, Respectively</u> (after Kanai, 1966)

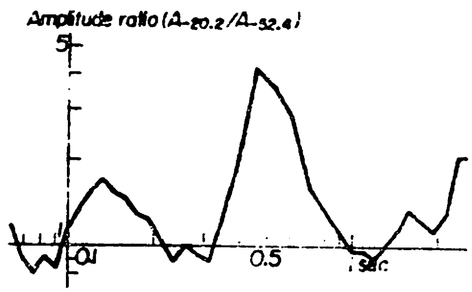


FIGURE 3.8c <u>Station Futaba A. Average of the Amplitude Ratios Between 20.2 m Depth and 52.4 m Depth.</u> (after Kanai, 1966)

Figures 3.9a,b show a somewhat different tendency. The large amplification in the soft soil layer above the rock is very clear, but the change in rock layers is quite negligible at Futaba A and a little more pronounced at Tsuruga B. We may say that qualitatively the tendency of deamplification with depth is as in soil, but quantitatively it is in much smaller scale.

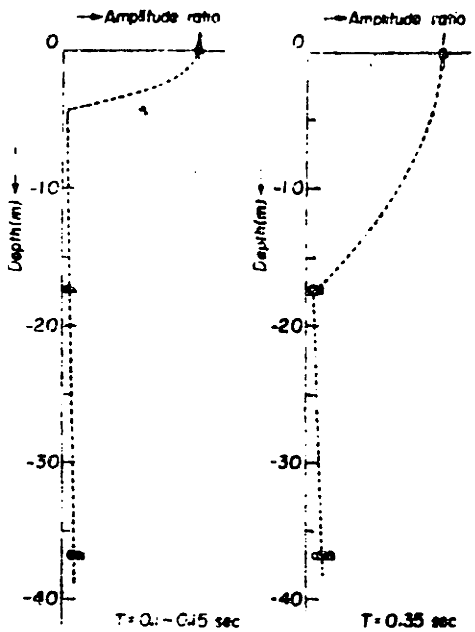


FIGURE 3.9a Station Tsuruga B. Spectral Amplitude of Earthquake Motions Versus Depth. (after Kanai, 1966)

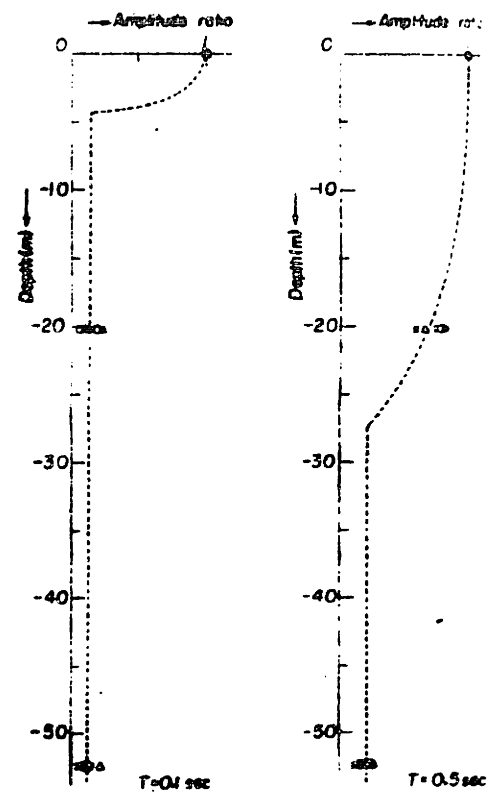


FIGURE 3.9b Station Futaba A. Spectral Amplitude of Earthquake Motions Versus Depth. (after Kanai, 1966)

Similar comparisons of the acceleration spectra show that the spectra at the bottom of a rock layer becomes somewhat smaller and flatter than the spectra at the top of the layer, but this change is very small compared to the deamplification in the soil layer which overlays the rock. Okamoto (1973) conducted observations in a vertical shaft at Kinugawa Power Station. His measurements were made simultaneously at five stations of various depths ranging from ground surface to -67 meters (220 ft). The geology of the site was mainly hard, coarse grained tuff. From a depth of 42 to 50 m (138-164 ft), the tuff is fine-grained, and from 50 to 54 m (164-177 ft), it is highly cracked. Three thin layers of clay, with thicknesses of 30, 40, and 20 cm (12, 16, and 8 in.) were found at depth of 15, 22, and 30 m (49, 72, and 118 ft) respectively. The longitudinal wave propagation was 3900 m/sec (12800 ft/sec) at the upper layers and 3500 m/sec (11500 ft/sec) at the lower layers. The locations of the five stations are shown in Figure 3.10, and the attenuation of acceleration records with depth is shown in Figure 3.11.

While the attenuation of acceleration is clear, it is difficult to observe any change in the displacements between the recorded at surface and at 67.2 m (220 ft) underground.

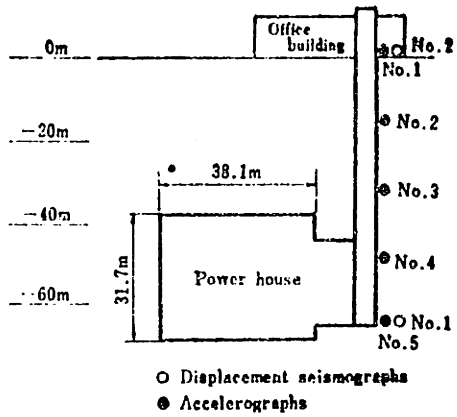


FIGURE 3.10 Location of Instruments at Kinugawa Power Station. (after Okamoto, 1973)

Okamoto summarized his findings in Figure 3.12 and suggested an attenuation law for a tuff site at 67.2 m (220 ft) below ground:

$$\log_{10} \frac{a_{max}}{640} = \frac{R+40}{100} (-7.604 + 1.7244M - 0.1036M^2) \qquad 3.5$$

He also compared his measurements with calculated amplitudes based on Tsuboi (1957) for surface movement and found that his underground observed values were considerably smaller, being only 6% to 20% of those predicted by Tsuboi's formula (Figure 3.13).

a.

b.

c.

d.

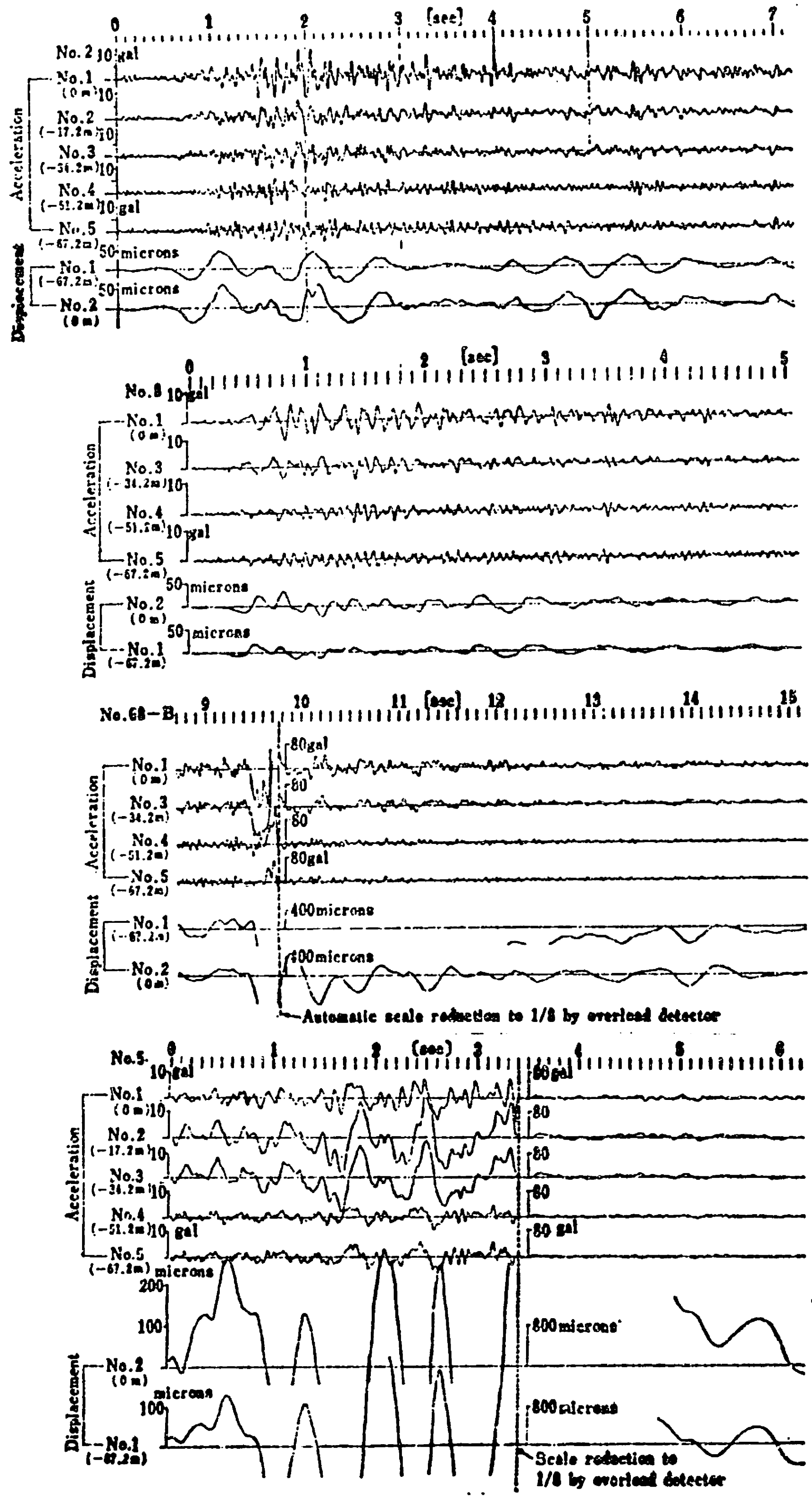


FIGURE 3.11 Records of Earthquake Acceleration and Displacement at Kinugawa Power Station (a) Earthquake on Dec. 24, 1962; (b) Earthquake on Dec. 20, 1964; (c) Earthquake on Apr. 6, 1965, and (d) Niigata Earthquake of June 16, 1964. (after Okamoto, 1973)

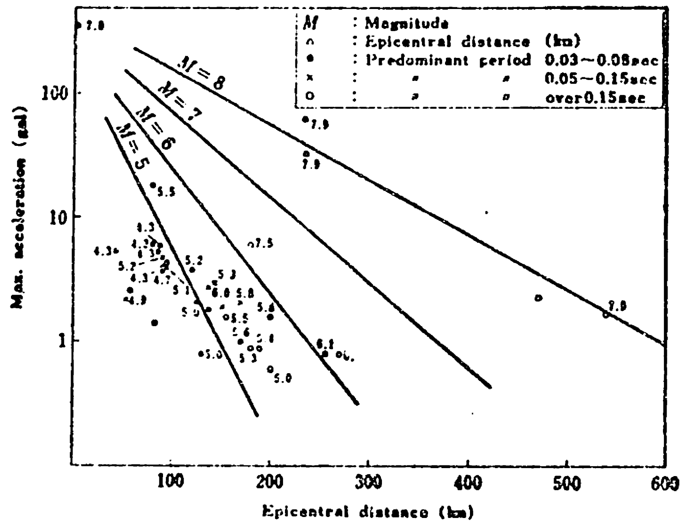


FIGURE 3.12 Relationship Between Magnitude, Epicentral Distance, and Maximum Acceleration 67.2 m (220 ft) Underground at the Kinugawa Power Station (Rocky Ground). The Figure gives the magnitude of each Earthquake. (after Okamoto, 1973)

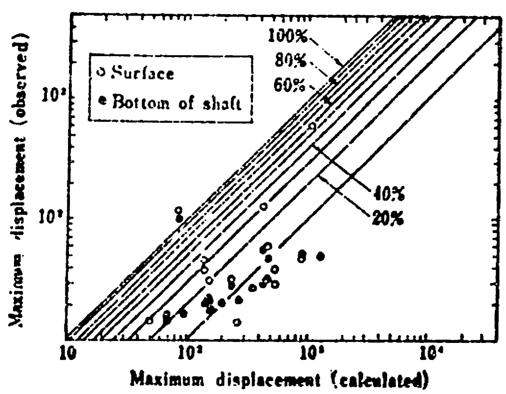


FIGURE 3.13 Relationship of Maximum Displacement as Calculated From Tsuboi (aboveground) and as Actually Observed at Kinugawa Power Station. (after Okamoto, 1973)

Kanai (1951, 1953) measured accelerations at depths of 150, 300, and 400 m in the Hitachi Mine. He found that the amplitude generally decreased with increasing depth, but he also found that the rate of attenuation became lower at depth, and a minimum was reached at a certain depth, 150 m in his observation. The rock at the Hitachi Mine was amphibolite with some inclusions of sericite-schist and chlorite-schist at 150 m (492 ft) and epi-diabase at 350 to 400 m (1148 to 1312 ft). Some of his findings are shown in Figure 3.14; but no explanations were given by him, and no relationship of displacement versus depth was established.

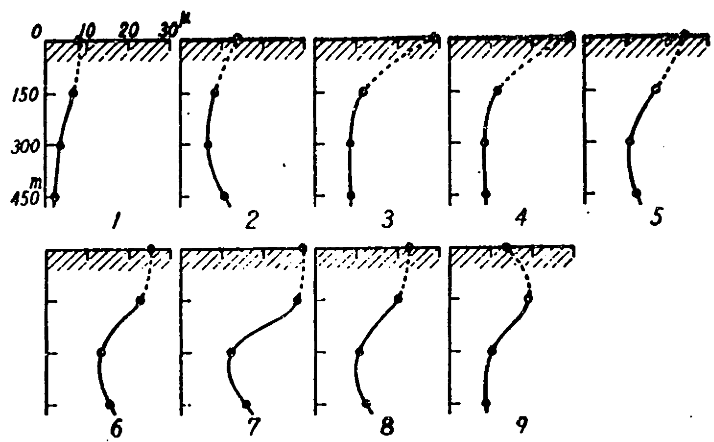


FIGURE 3.14 <u>The Distributions of Horizontal Displacements at Different Depths of Several Earthquake Cases</u>. (after Kanai, 1951)

Based on his findings in the Hitachi Mine, Kanai (1960) suggested a set of attenuation laws for 300 m (984 ft) below ground in rock:

$$d = T \times 10^{0.61M - 1.73 \log R - 1.47} \qquad 3.6$$

$$V = 2\pi \times 10^{0.61M - 1.73 \log R - 1.47} \qquad 3.7$$

$$a = \frac{(2\pi)^2}{T} 10^{0.61M - 1.73 \log R - 1.47} \qquad 3.8$$

The relationship between the three peak values is based on the assumption that the earth motion may be simplified into a sinusoidal motion where $v = \frac{2\pi}{T}d$ and $a = \left(\frac{2\pi}{T}\right)^2 d$.

Contrary to the attenuation laws used in the US and Mexico (Section 3.2), Kanai considered the peak values as a function of the predominant period of the quake motion. The velocity, on the other hand, is period-independent. Kanai (1960) stressed that in general for long period waves, the peak amplitude at the bedrock is approximately the same as that at the ground surface. This conclusion is similar to Nasu's (1931). He also explained that his formula, which was suggested originally by Tsuboi for Japan, is valid also for the US quakes.

The attenuation laws suggested by Kanai for acceleration and displacement will not be used herein, as there is no data available on the predominant frequency of the earthquakes which caused the damage described in the next chapter.

Okamoto (1975) told about measurements at Sudagai in liparite rock, both at surface and 38 m (125 ft) below ground. For periods of 0.3 to 0.5 seconds (frequencies of 3.3-2 Hz), the peak accelerations below ground were reduced to 45-50% of those at surface (Figure 3.15). Okamoto summarized this case by saying that "the diminution with depth is similar to that in alluvial stratum, but the point which differs from the records of alluvial layers is that the waveforms above and below ground are very similar."

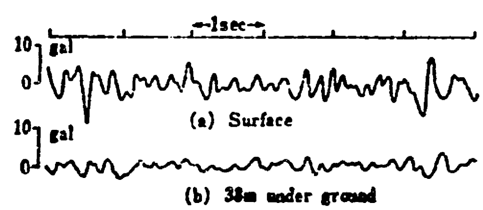


FIGURE 3.15 <u>Acceleration Records Taken on the Surface And 38 Meters Underground at Sudagai, Northern Gumma Prefecture</u>. (after Okamoto, 1973)

Nakamura (1925), as quoted by Nasu (1931), installed two seismographs of the same type at the entrance to a tunnel and in a side adit 89.2 m (292.5 ft) below the surface and 451 m (1479 ft) from the entrance. He found that the amplitudes of the quake's motions inside the tunnel were in general less than that at the portals. The ratio of the motion inside to that outside the tunnel was 0.43 - 0.80 (average of 0.62) when measured parallel to the tunnel axis and 0.60 - 1.12 (average of 0.86) perpendicular to that axis. He found also that the reduction of amplitude with depth (or deamplification, as it may be called today) was not constant but dependent on the predominant frequency of the quake.

Nasu (1931) conducted measurements in the Tanna Tunnel and on the ground surface just above the tunnel's station. The ground layers at the site were: 30 m (100 ft) of lake deposit including sandy clay and boulders, 10 m (33 ft) of lava and pyroxene andesite, 100 m (328 ft) of agglomerate, and 20 m (66 ft) of olivine-pyroxene-andesite. The tunnel itself passes 60 m (525 ft) below the surface in the olivine-pyroxene-andesite stratum. Nasu made his measurement during six months following the Idu Earthquake which caused some damage due to fault movement in the tunnel (Chapter 4.1.4), and hence was able to measure many after shocks with predominant frequencies in the range of 4 - 0.22 Hz (or periods of 0.25 - 4.5 seconds). His studies made it clear that for high frequencies the motion on the surface is much greater (on the order of 2-4) than below ground, and for low frequencies the aboveground motions are still larger, but only 1.0 - 2.0 of those below ground. For frequencies below 0.25 Hz, or periods above 4 seconds, the ratio of above to below ground motions tends to unity (Figure 3.16).

Nasu (1931) made clear that in his study both stations were on agglomerate rock, hence the difference in the intensity of the motion in the tunnel and outside must be accounted for by the deamplification with depth.

Vanmarcke (1975) gave some data about the predominant frequencies and frequency band width of US earthquakes, which are summarized in columns 1 to 5 in Table 3-1.

By using Nasu's curve, it is clear that in these cases the expected underground rock motion may be four times less than the surface motions measured on the same rock material. Nasu's conclusion was that it is highly probable that earthquake motions in rock at depth are less intense than on the surface of the earth.

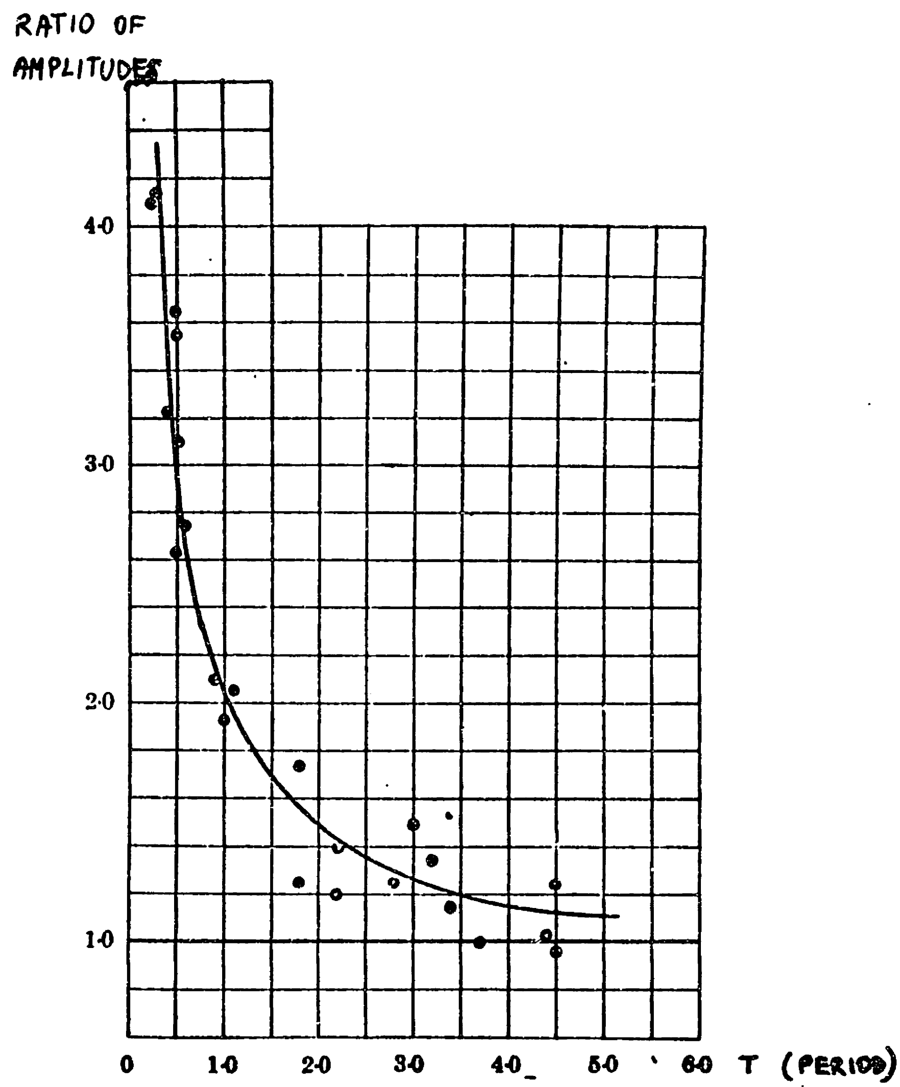


FIGURE 3.16 The Relation Between the Ratio of Earthquake Motions Aboveground to That in the Tunnel and the Period of Earthquake.

Earthquake	Predominant Frequency Hz	Predominant Period Sec.	Width ± Frequency	Width ± Period	Ratio of Above to Below Ground Motion
El Centro 1940 NS	4.99	0.20	0.12	0.01	>4
El Centro 1940 EW	4.06	0.25	0.10	0.01	>4
Olympia N10W	5.74	0.17	0.10	0.01	>4
Olympia N80E	4.91	0.20	0.10	0.01	>4
Taft N69W	4.33	0.23	0.11	0.01	>4
Taft S21W	4.41	0.23	0.10	0.01	>4
Average	4.70	0.21	0.11	0.01	>4

TABLE 3-1 DEAMPLIFICATION RATIO FOR SEVERAL EARTHQUAKES

As explained in paragraph 3.1, the attenuation laws are used in this study to estimate the level of shaking that caused the damage in the tunnels. It seems desirable to adapt attenuation laws (which are based on aboveground measurements) so as to take also into consideration the attenuation with depth. This may improve the estimate of the level of shaking which caused the damage in the tunnels. But, as may be seen from this section, the data is too scarce, somewhat conflicting, and is not suitable to be summarized into a general analytical expression.

A more detailed discussion on the Kanai equation compared to the aboveground attenuation laws will be presented in Chapter 3.7.

3.5 OTHER CHARACTERISTICS OF EARTHQUAKES

In addition to the three peak amplitudes already mentioned -- displacement, velocity, and acceleration -- there are two other characteristics of the earthquake shaking which may influence the damage caused by it. Those are: (a) the period or the frequency, and (b) the duration of shaking.

The period or the frequency was discussed briefly in Section 3.4. It was found by the Japanese researchers that the deamplification with depth is influenced by the predominant frequency, and it is usually higher for higher frequencies. For earthquake-resistant design of aboveground structures, period of 1 to 3 seconds (or frequencies of 1 to 0.3 cps.) are of interest. For tunnels, these waves are too long to cause a dynamic response in the tunnel. Seismic waves may cause dynamic effects around a tunnel if, and only if, they are the same order of magnitude with the tunnel's diameter, ($\lambda \approx D$), or at the most, up to four times the diameter ($\lambda \leq 4D$).

Assuming a large tunnel of 15 m (50 ft) diameter, the wavelength of interest should be less than 200 ft ($\lambda \leq 4 \times 50 = 200$). The periods/frequencies of interest should be as summarized in Table 3-2, using the definition $C = \lambda \cdot f$.

ROCK TYPE	INTACT		RQD 60%		RQD 40%	
	V(fps)	f(cps)	V	f	V	f
Hard Limestone	21000	52.5	12600	31.5	8400	21
Granite Gneiss	18000	45.0	10800	27.0	7200	18
Granite	1600	40.0	9600	24.0	6400	16
Sandstone	6000	15.0	3600	9.0	2400	6

TABLE 3-2 LIMITING FREQUENCIES ($\lambda = 4D$) FOR VARIOUS ROCKS

From the table, one may conclude that in excellent rock (RQD = 90 to 100%), the periods are too short or frequencies too high to be expected in earthquakes, except for weak material like sandstone of 6000 ft/sec seismic velocity. For very jointed rock defined as poor (RQD = 40%), the expected frequencies are at the range of 6 - 18 Hz, which may be found even in earthquakes.

Usually in earthquake design of aboveground structures, the high frequency range of the ground shaking is neglected as it excites only the higher modes, which do not contribute significantly to the total response of the aboveground structure. It seems that in the response of tunnels the situation is different, and the high frequency content is much more important. Usually, it is assumed that high frequencies do not contribute significantly to the acceleration; but while discussing the possibility of local failure of small-size blocks or stones, the high frequency content may contribute to relative displacement between blocks, hence triggering local failure along planes of weakness. This phenomenon may explain the local spalling of stones or concrete which were reported in several cases after earthquakes.

It is known (Housner, 1970) that the higher frequency components attenuate more rapidly than the lower frequency components, so that the destructive (from the tunnel point of view) frequencies may be expected mainly at short distances from the causative fault. The present knowledge of the ground motions near the causative fault is very limited, as very few, if any, measurements were made there.

Seed and Idriss (1969) suggested, based on limited data, a relationship between the predominant period in rock and the distance from the causative fault. (Figure 3.17). At short distancesup to 40 km (25 miles) from the epicenter the predominant period is almost magnitude-independent, varying only in the range of 0.22 - 0.4 seconds (4.55 - 2.5 Hz), with an average of 0.31 seconds (3.23 Hz). At larger distances, the period is magnitude dependent; and, for a given distance, longer periods (lower frequencies) may be expected from larger magnitudes.

It may be concluded that small magnitude earthquakes at shorter distances from the fault are more dangerous to rock tunnels than large magnitude quakes at greater distances from the fault.

Duration of the strong-motion part of the quake may be of utmost importance as it may cause a failure through fatigue. This mode of failure is dependent on the total number of cycles induced by the ground shaking. Hence, the longer the duration, the higher the number of cycles. It is known that the quake duration or the repetitive loading cause liquefaction failure in sand by building up pore pressure and decreasing shear resistance, causing a failure at a stress less than strength during a single loading.

Haimson and Kim (1972) found that long duration cycling loading may cause fatigue failure in intact rock,

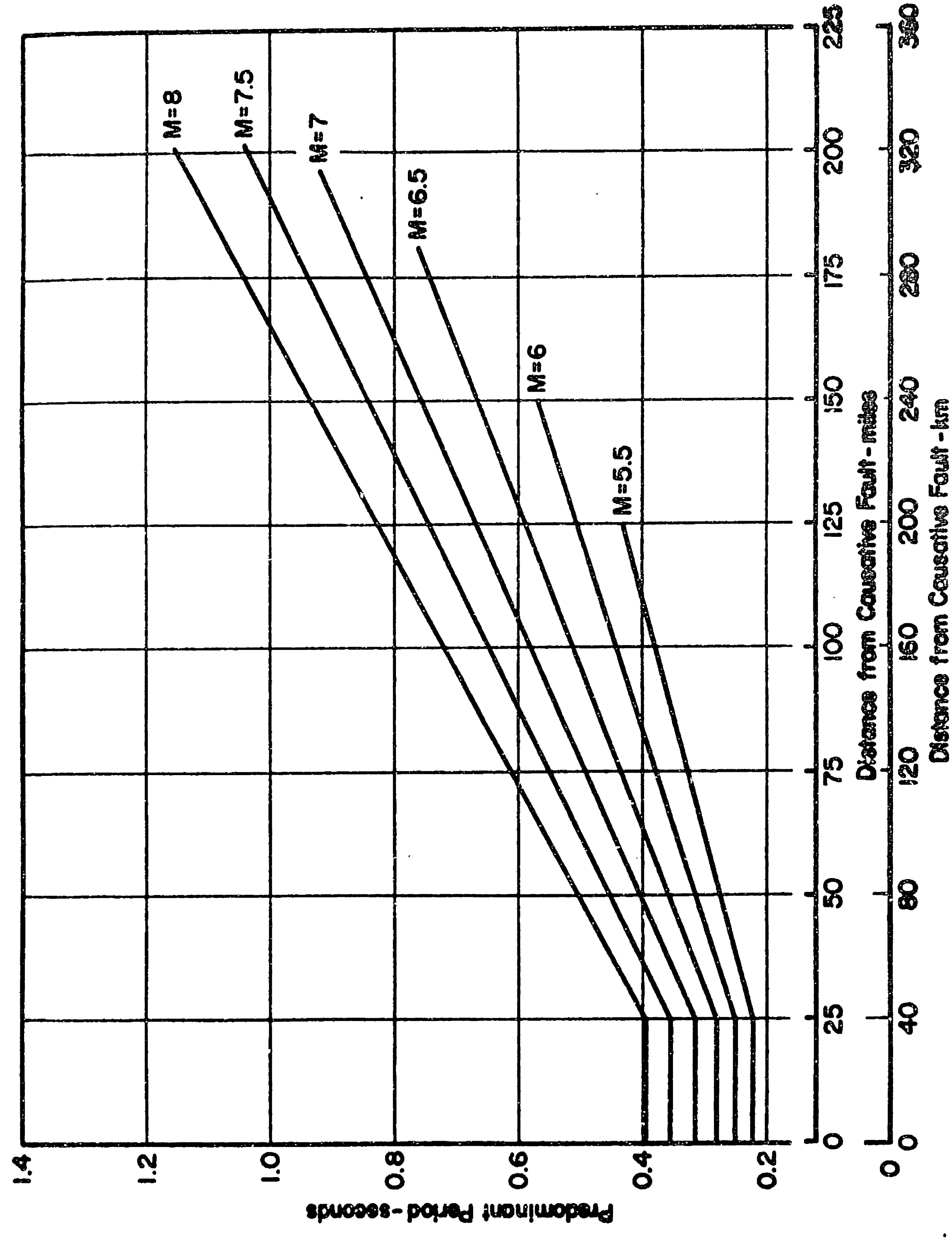


FIGURE 3.17 Predominant Periods for Maximum Acceleration in Rock

and Brown and Hudson (1974) proved it experimentally for jointed media. The large number of cycles required to cause fatigue failure, as found by Brown and Hudson, is too large to be of importance in a single earthquake; but it seems to be of importance in a series of after-shocks in which the accumulated number of cycles may be large enough to cause failure.

In general, the duration of the quake is longer at greater distances from the epicenter; but, on the other hand, the amplitude of the shaking is less.

Housner (1970) gave a relationship between magnitude and duration. Assuming an "average" frequency range of 1 - 5 Hz, Table 3-3 gives an estimate of the number of cycles that the tunnel and its rock walls may suffer.

MAGNITUDE	DURATION (seconds)	CYCLES (range)
5.0	2	2- 10
5.5	6	6- 30
6.0	12	12- 60
6.5	18	18- 90
7.0	24	24-120
7.5	30	30-150
8.0	34	34-170
8.5	37	37-185

TABLE 3-3 NUMBER OF CYCLES AS A FUNCTION OF MAGNITUDE

Esteva and Rosenblueth (1964), as quoted by Bolt (1973), suggested a relationship of the form:

$$s = 0.02 \exp(0.74M) + 0.3R. \qquad 3.9$$

Bolt (1973) gave the duration as a function of the predominant frequency, taking into account the fact that high frequency waves attenuate more strongly than low frequency waves. (Figure 3.18).

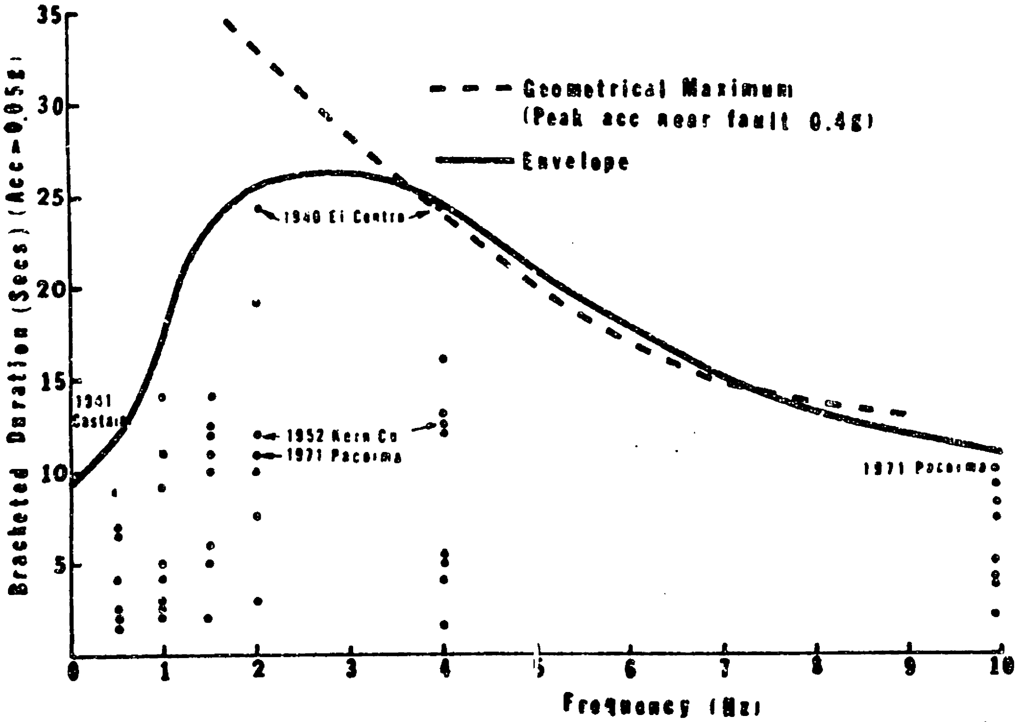


FIGURE 3.18 Duration of Acceleration Above 0.05g Versus Frequency (after Bolt, 1973)

According to Bolt, the longest durations may be expected for frequencies in the 1 - 5 Hz (1-0.2 sec) range while the duration decreases rapidly below 1 Hz and somewhat slower above 5 Hz (Figure 3.18). Figure 3.19 shows the relationships between magnitude and durations for (a) duration above 2.05 g level and (b) duration above 0.1 g level. The term "bracketed duration", as appeared in Figure 3.19, was used by Bolt to indicate the part of the total duration that is above a certain threshold.

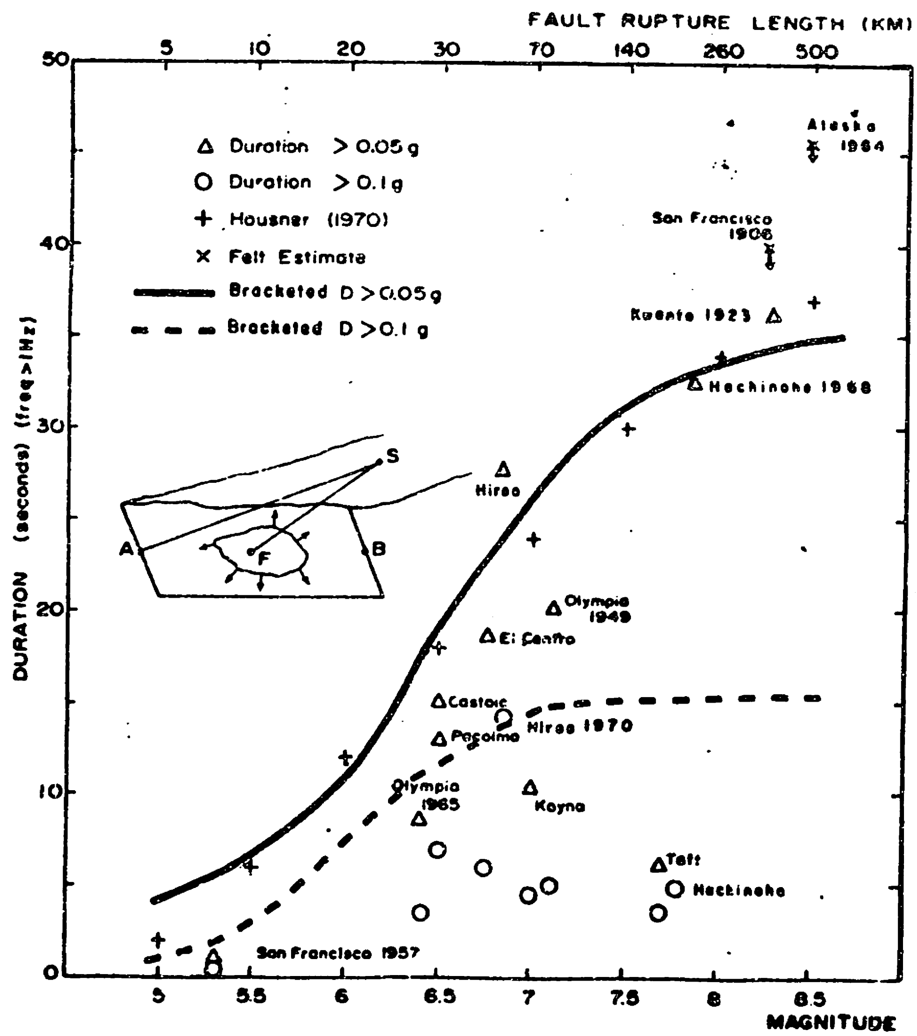


FIGURE 3.19 Duration Versus Magnitude (after Bolt, 1973)

3.6 INTENSITY AND GROUND MOTIONS RELATIONSHIPS

Over most of the world, excluding California, Japan, and some other highly-seismic regions, the effects of earthquakes are known only through intensities. Thus, attempts have been made to relate strong-motion recordings to intensity and to use this relationship where only intensities were available. The main work has been done with accelerations. Velocities and displacements have been compared on a much more limited basis.

The most commonly used intensity scales are Modified Mercalli (MM), Rossi-Forel (RF), the Russian Scale (MKS), and the Japanese Scale. The different scales are described elsewhere. (See for example, Barosh, 1969). In this investigation, the use of the Modified Mercalli Scale will be preferred as it is, by far, the most popular in the United States.

Again, there are many correlations which may be used: Newmann, (1954), Gutenberg and Richter (1956), Hershberger (1956), Medvedev, Sponheuer, and Karnick (quoted by Barosh, 1968). Krinitzsky and Chang (1975) made a comparison between various relationships (Figure 3.20) and found a large scatter. A set of equations given by Trifunac and Brady (1975) give correlations not only for acceleration, but also for velocity and displacement:

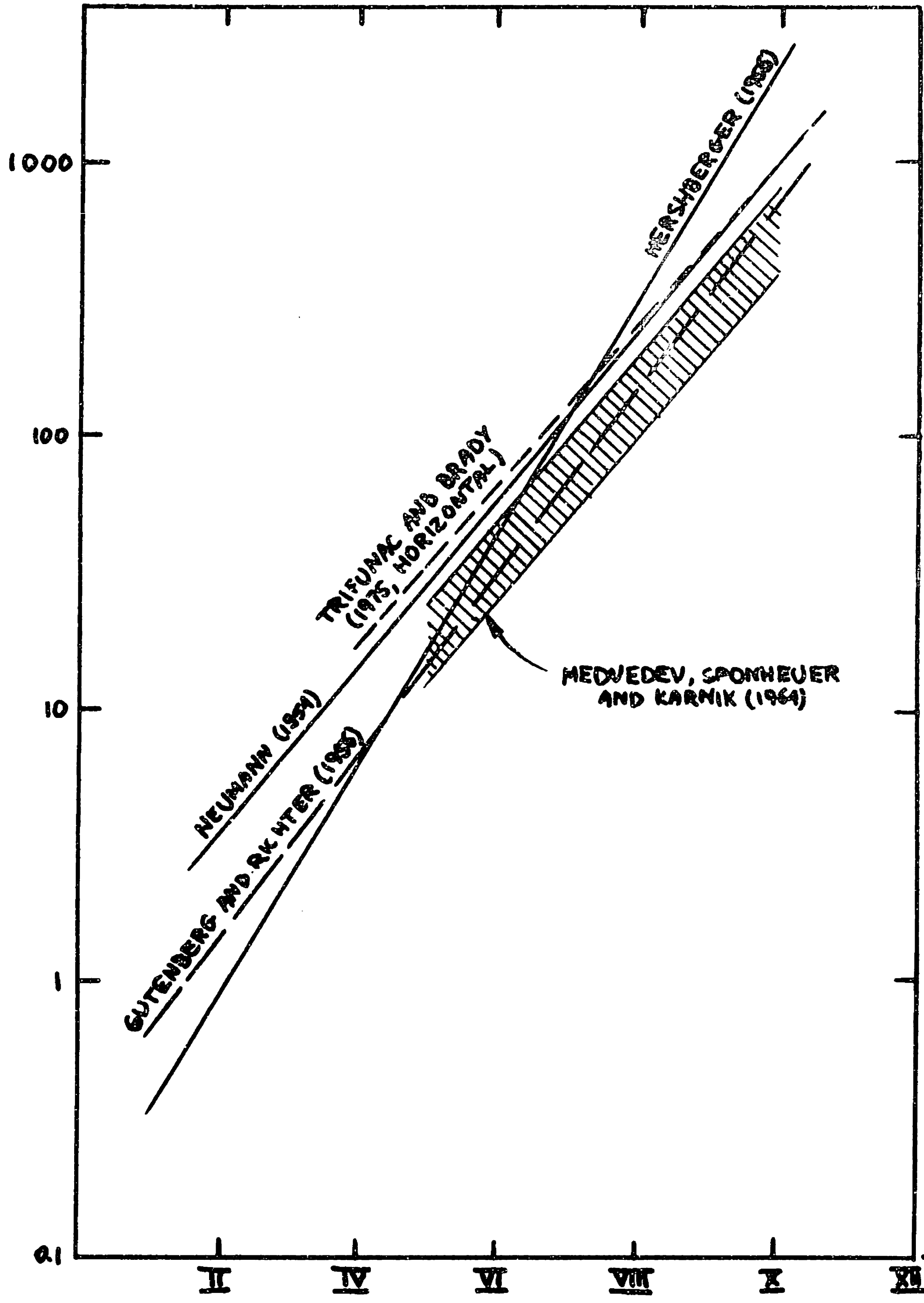


FIGURE 3.20 Comparison of Selected Curves for MM Intensity Versus Acceleration (after Krinitzsky, 1975)

$$\log a_H = 0.014 + 0.30I$$

$$\log V_H = -0.63 + 0.25I$$

$$\log d_H = -0.53 + 0.19I$$

where the subscript H is for horizontal motion.

3.7 DISCUSSION AND PRELIMINARY CONCLUSIONS

As already discussed in Chapter 3.1, a technique is needed to estimate the level of shaking which caused the damage reported in the case histories (Chapter 4). In Section 3.2, Figure 3.2, several attenuation laws were demonstrated; and in Sections 3.3 and 3.4 some material representing the influence of geological conditions and depth in ground was summarized.

Figure 3.21 shows the "envelope" of all the attenuation laws of Figure 3.2. An "average" curve, passing between the midpoints of the upper and lower boundaries, was sketched. This curve has no statistical meaning, as the information on which the various attenuation laws are based is unknown. Nevertheless, this curve provides a visual tool to get a better feeling of the general tendency of the attenuation laws. McGuire's Law coincides with the "average" curve up to 10 kms (6.3 miles) and with the upper boundary of the envelope from 200 km (125 miles) and above. At the range of interest from

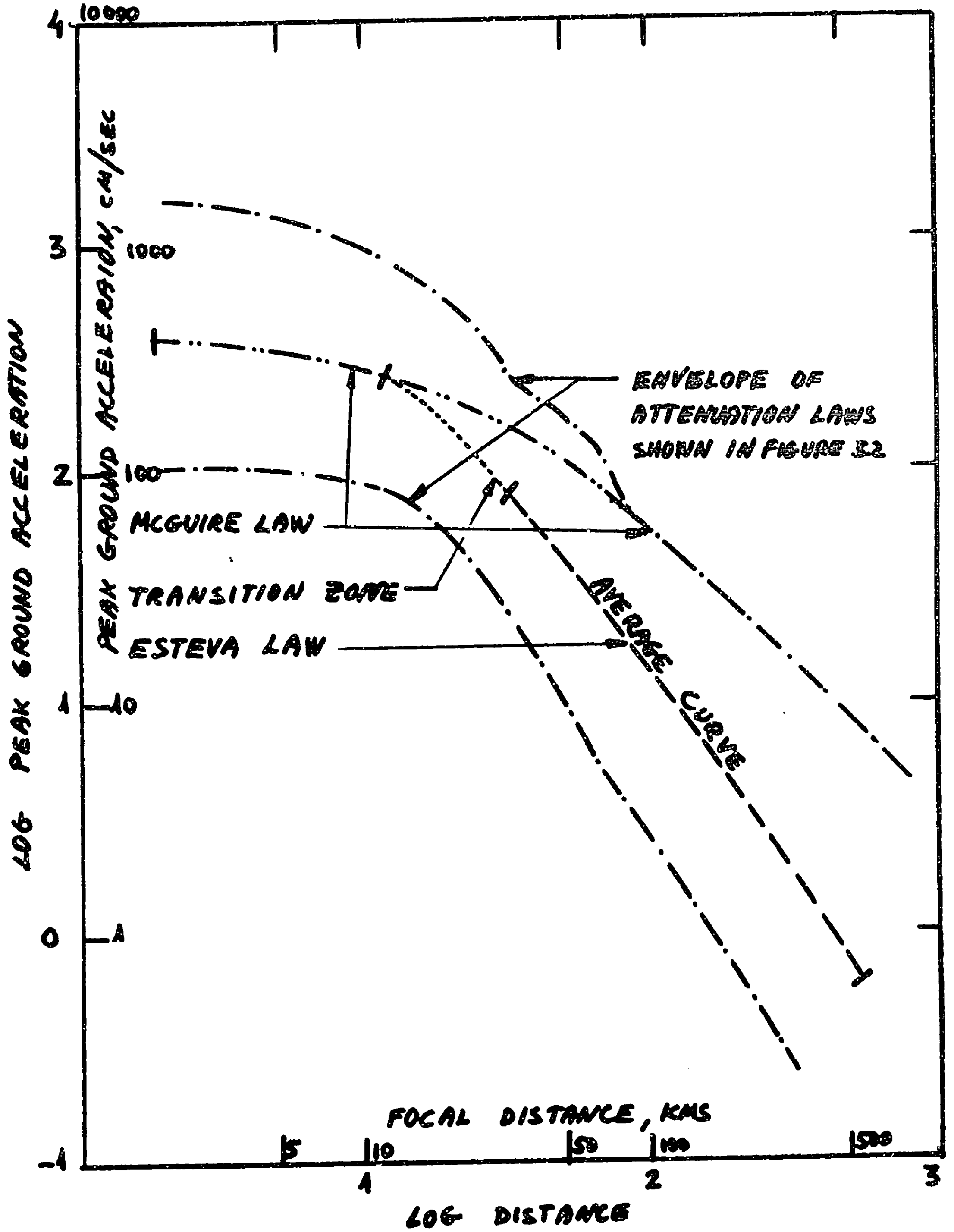


FIGURE 3.21 Comparison of Aboveground Attenuation Laws For Accelerations and McGuire's Laws (For M=6.5)

10 to 200 kms (6.2 to 125 miles), it was found that the Esteva (1970) Attenuation Law for peak acceleration is the best estimate of the "average." The use of McGuire's Laws in the comparative table of Chapter 4 will result in peak values which are higher than expected from most of the other attenuation laws.

Figure 3.22 provides a comparison between the aboveground peak acceleration attenuation laws, represented by their envelope, and the Kanai Equation (Equation 3-8), established on measurements 300 m below ground level. As the peak acceleration is a function of the earthquake predominant period, three curves are given for 0.25, 1, and 3 seconds (4 to 0.33 Hz). It is clear from this figure that Kanai curves remain in the range defined by the scatter of the aboveground data. There is no information concerning the frequency pertinent to the aboveground attenuation laws, hence it is impossible to define an analytical expression for attenuation with depth.

Figure 3.23 shows a comparison between four curves for surface velocities as given by Ambraseys (1973), McGuire (1974), and Seed et. al. (1975) and the equation developed by Kanai based on data measured 300 m (984 ft) below ground. Neglecting Seed's data, which are, as already mentioned, based on limited data and according

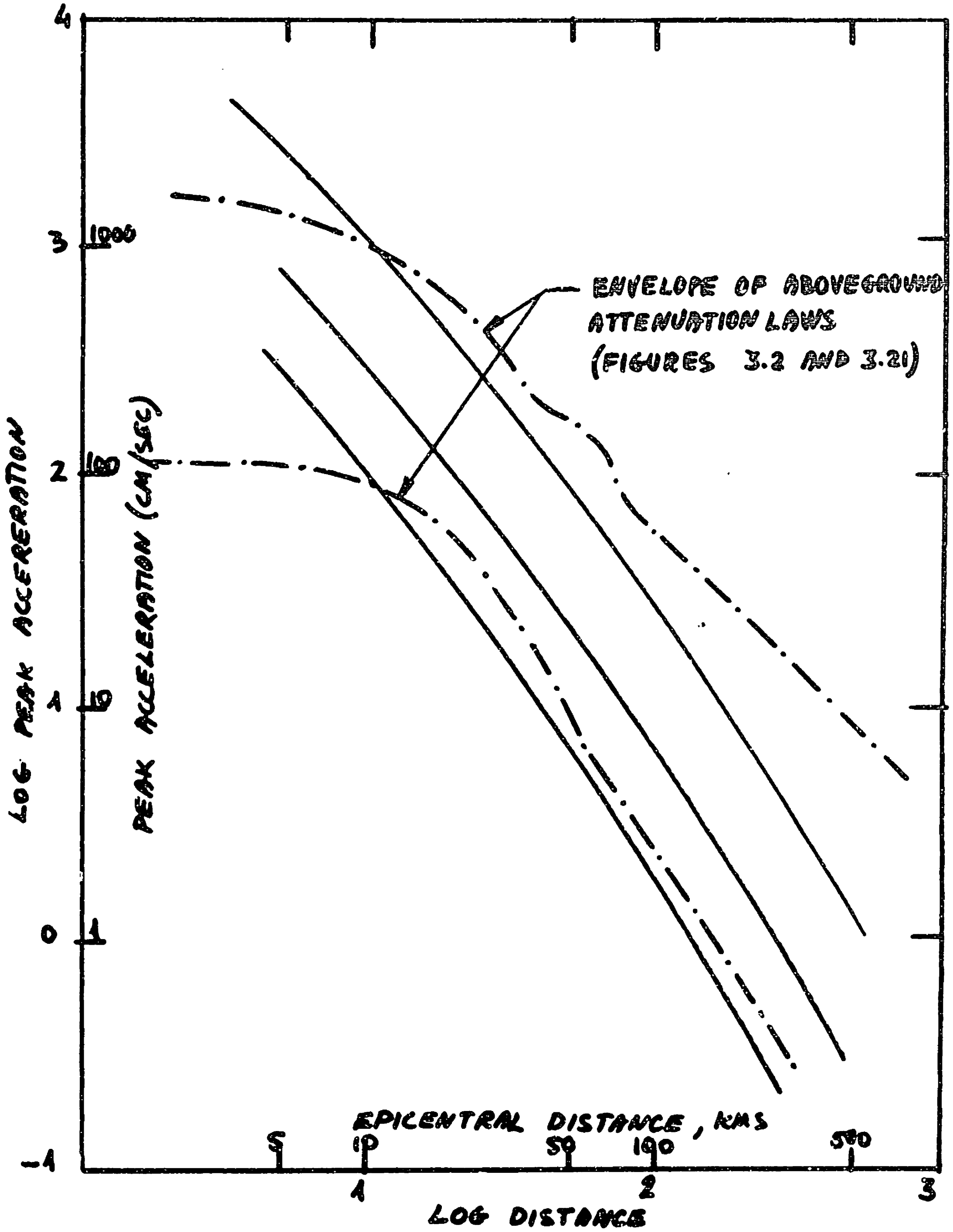


FIGURE 3.22 Comparison of Aboveground Attenuation Laws For Accelerations and Kanai Formula (-300 M , M = 6.5)

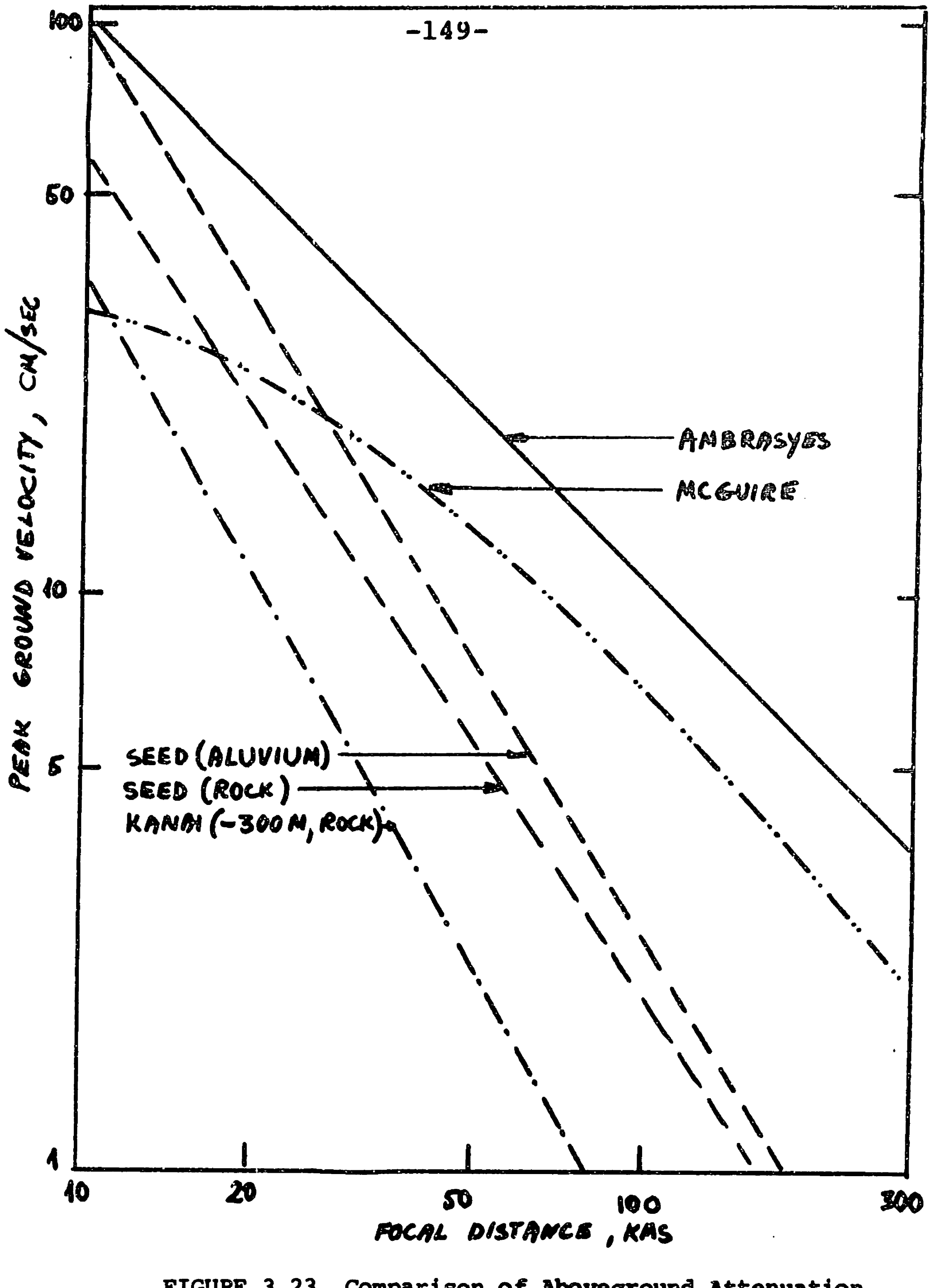


FIGURE 3.23 Comparison of Aboveground Attenuation Laws for Velocities and Kanai's Equation For 300M Underground.

to his own words "admittedly somewhat speculative", a qualitative tendency may be concluded. The ratio between the velocity amplitudes on ground surface to those measured below ground are varying from 3.5 at short distances (20 kms or 12.5 miles) to 7.3 and 10 at 50 and 100 kms (3.13 and 62.5 miles) respectively.

The reader must not accept these numerical results as theoretically or experimentally proven. They are not! As already mentioned, the Ambraseys and McGuire curves on one hand and Kanai curve on the other hand are based on different earthquakes, locations, and assumptions; and it is not justified to obtain any numerical conclusions from them.

Figure 3.24 shows a comparison between Okamoto's Equation for 67 m (220 ft) below ground and Kanai's Equation for 300 m (984 ft) below ground. As Okamoto proposed only an acceleration law, the writer used the usual assumption of sinusoidal motion to obtain a velocity rule which becomes:

$$V = \frac{aT}{2} \qquad 3.13$$

where a is given by Equation 2.23. As the velocity is now period dependent, it was plotted as a bandwidth from $T = 0.1$ to 1 second. Again, no conclusions may be

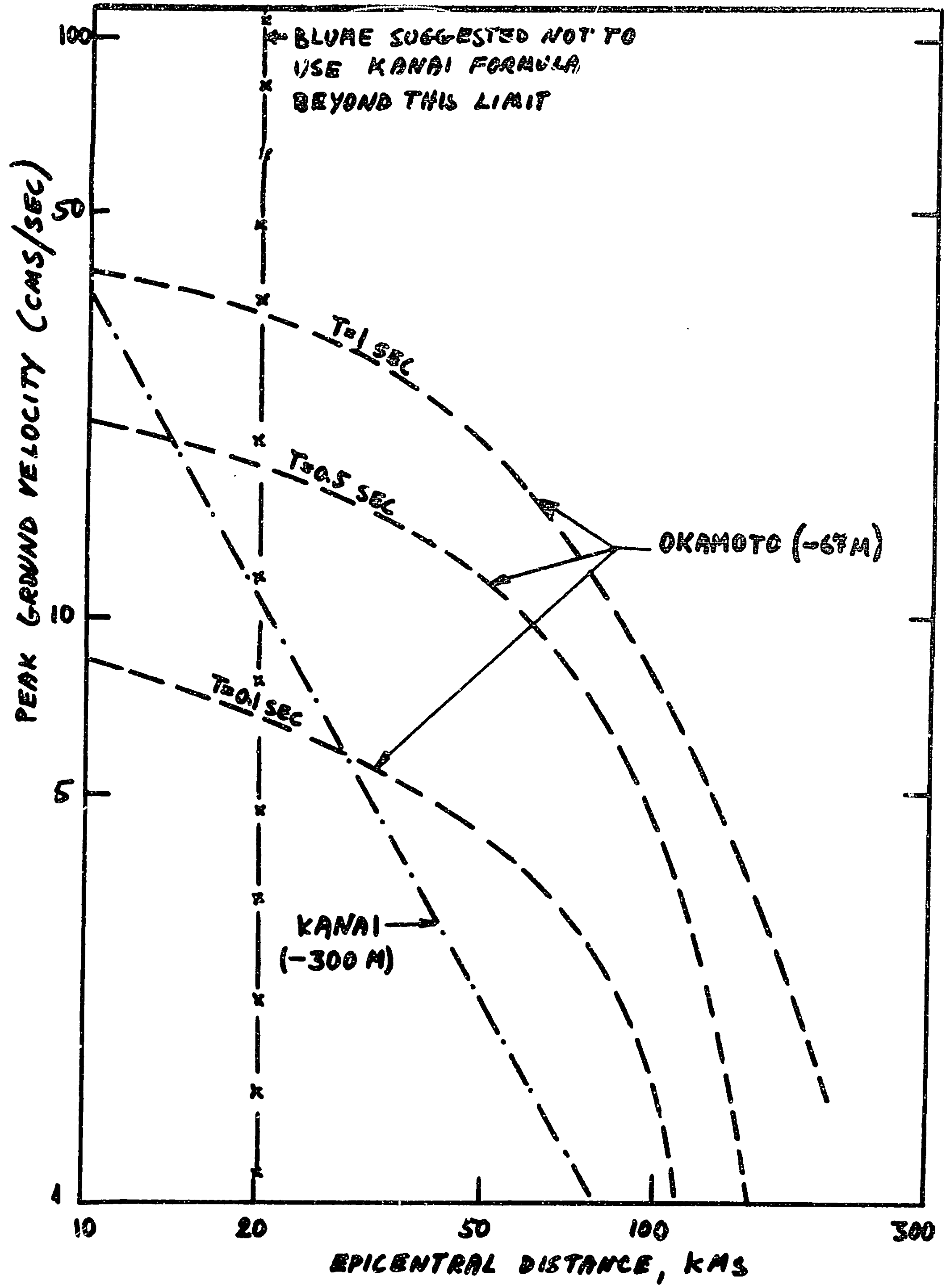


FIGURE 3.24 Comparison of Underground Attenuation Laws of Okamoto (-67m) and Kanai (-300m)

suggested from this comparison; and no explanation is available for the intersection of the curves of Kanai and Okamoto, for $T = 0.1$ seconds at 30 km.

It is of interest to note that, while Kanai's velocity equation is period independent, the opposite is true for Okamoto's equation.

Based on this discussion, it was decided to use only aboveground attenuation laws (those suggested by McGuire) and to neglect the attenuation with depth. The peak accelerations and velocities calculated in Chapter 5 (Table 5.5) may be regarded as upper bound, and the expected level of shaking will usually be less than that.

CHAPTER 3

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CHAPTER 4

SUMMARY OF CASE HISTORIES

4.1 EFFECTS OF EARTHQUAKES ON TUNNELS

4.1.1 Introduction

The first three chapters of this work were concentrated on understanding the fundamentals which may influence the behavior of tunnels or any other underground construction built in jointed rock when loaded by earthquake ground motion.

This chapter will be devoted to descriptive case histories in which tunnels and related underground construction have suffered damage of various extents by earthquakes. In some ways, one can look at this data as a large-scale experiment which must be used to verify any analytical model.

4.1.2 The Central California Earthquake (1905)

The San Francisco Earthquake was the first major earthquake in the United States in the twentieth century. The earthquake occurred on April 18, 1906, with a magnitude of 8.3 (Richter, p. 473). There is some confusion about its exact magnitude and location. Even Richter gave a second estimate to the magnitude ($M = 7.75$, p. 472), but the higher value now seems to be more acceptable.

The epicenter was at 121° 36'W 37° 49'N (some seismologists gave 122° 48'W 38° 03'N) and the focal depth, 16 - 20 km (10-12.5 miles). The duration of the strong shaking was 60 seconds. Most of the damage, either by "shaking" or by fire which followed the earthquake, occurred in aboveground structures. Some water conduits and two tunnels were also damaged. Gilbert et. al. (1907) stated that "while one of the main conduits was badly damaged wherever it crossed the fault, this damage was no greater than that done to any other structure that was situated at the fault line. Structures so located were torn apart." Concerning underground pipes, it was clearly found that "these breaks (in distributing mains) occurred wherever the pipe passed through soft or filled ground. No breaks occurred where the case iron pipe was laid in solid ground or rock."

The damage to tunnels was summarized by Gilbert et. al. (1907), Special Committee of ASCE (1907), and Carnegie Institute (1908) as follows: Two narrow-gauge Southern Pacific Railroad tunnels were damaged in the vicinity of Los Gatos (approximately 130 km (81.3 miles) from the eipcenter) on the rail line between Santa Cruz and Wrights. One of the tunnels was cut by the fault where it passed through the Santa Cruz Mountains. A portion of the loose roof, made of shale on a layer of serpentine or soapstone, caved in, completely closing the tunnel. A house on the same fault line near Wrights was split in two. The tunnel was 3.95 m (13 ft) wide, and lined with timber. Except for the local cave-in, some of the timber-crushed rails were heaved and ties broken. An offset of 1.35 m (4.5 ft) was measured in the tunnel.

The second tunnel with the same cross section and timber lining was not crossed by the fault, and the damage pattern was similar but to a somewhat lesser degree. The rocks looked like sandstone and jasper of the Franciscan Age.

A special inquiry by the Corps of Engineers (quoted by Gilbert et. al., 1906) confirmed that "no tunnels had collapsed and what damage had been done...had been wholly or partly repaired before I reached California." This report was two weeks after the earthquake.

New tunnels, which were under construction on the Bayshore Line in Southern San Francisco, remained undamaged; and other tunnels on the Santa Cruz-Los Gatos Line, which were located at the same area of very intense shaking (10 on the Rossi-Forel Scale), remained undamaged!

4.1.3 Kwanto Earthquake (1923)

The Kwanto (or Kanto Region) Earthquake, also known as the Great Tokyo Earthquake, occurred on September 1, 1923, with a magnitude of 8.16, which makes is one of the strongest earthquakes. Since the eipcenter was located near Tokyo and Yokohama, the damage was enormous; and approximately 100,000 lost their lives in the earthquake and the great fire that followed.

The map in Figure 4.1 shows the area with the epicenter about 10 km (6.25 miles) south of Mt. Tanzawa at a depth of 0-10 km (o-6.25 miles). (Seismological Notes, 1923; Jaggar, 1923)

The first assumption of the eipcenter region was SSW from Tokyo on the bottom of the Sagami Bay near Oshima Island (ISS, 1923; Richter, 1958), but an updated summary by Okamoto (1973) points out the location given near Mt. Tanzawa.

There are many reports about the earthquake and its interesting seismic phenomena; but unfortunately, most of the material is in Japanese. Some material appeared in English and is available: Kanamori (1970), Davison (1930), Richter (1958), Duke et. al. (1959), and Okamoto (1973); but unless the two original reports by Okamura (1926 and JEIC (1926) will be translated in detail, the only available reference on tunnels is Duke's report (1959).

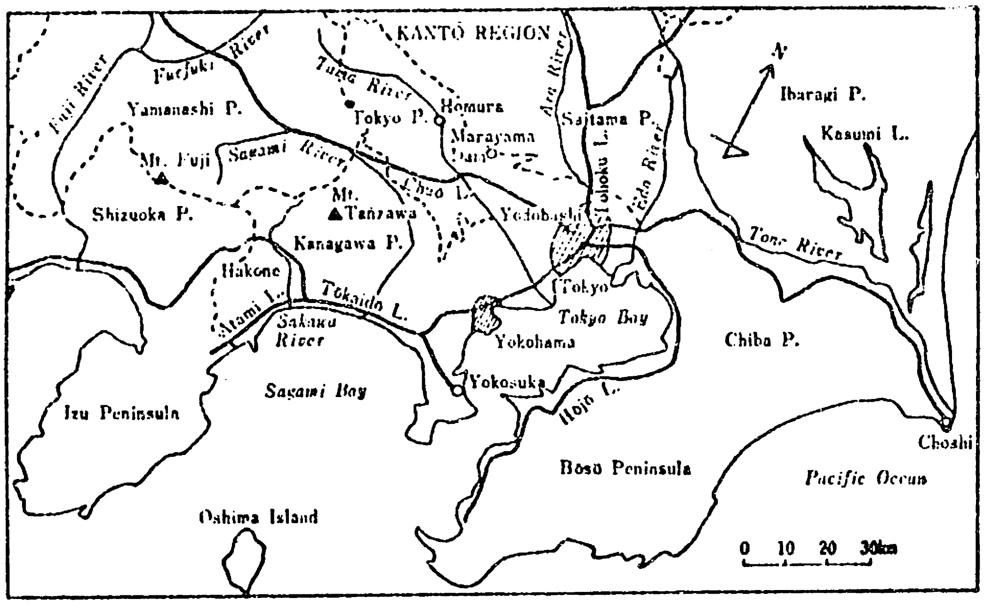

FIGURE 4.1 Area of the Kanto Earthquake (1923) (after Okamoto, 1973)

In the September 1, 1923 earthquake (magnitude 8.16), about 25 tunnels with different cross-sections, linings, and rock conditions were damaged around Tokyo in the area of intensity of 10 in the Rossi-Forel Scale. The damage was caused by either ground shaking or ground failure, as no case of fault intersecting the tunnels is known.

On the Yokohama and Yakouku Line, four tunnels were damaged near the portal, mainly through cracked bricks but also from landslides. There is no data about cross section and rock type. Damage at Honjo Line was minimal, mostly from slight fractures in concrete walls and some spalling.

Eleven tunnels suffered from a wide range of damage. A common factor was masonry damage at the portals and landslides which buried the entrances. All the damage in the brick lining was usually minor, except in two cases: A tunnel near Yonegami-Jama No. 2, also called Shimomaki-Yama, under 28.3 m (95 ft) of cover of unknown rock suffered from a deformed interior. The linings were made of masonry. A second tunnel, built in loose material on a steep slope under 18 m (60 ft) of cover, had a "badly cracked interior."

In contrast to the secondary damage caused in masonry lining, a tunnel lined with reinforced concrete blocks was "destroyed" by tilted blocks and caving of ceiling slabs. This tunnel was shallower, covered by only 1.5-6.0 m (20 ft) of unknown material. Of the seven tunnels in the Takaido-Main Line, only one was undamaged; and in all others, main damages were either buried entrances or some ceiling collapses near the portals. The interior portions suffered much less, usually from cracks, except in the Hakone No. 7 Tunnel where an interior section collapsed. This tunnel was under 30 m (100 ft) of fissured, faulted, and weathered rock.

The Yose Tunnel, under 19.5 m (65 ft) of soft, fine, grained rock, was daylighted and collapsed in several shallow portions. Similar damage occurred to another shallow tunnel at Doki. A brick and concrete tunnel at Namuya, under 75 m (250 ft) of unknown material, caved in, and cracks with displacements of up to 25 cm (10 in) were found. The same tunnel suffered also from landslides. A masonry tunnel in "deformed and destroyed rock" with some basalt had suffered from cracks, and in some places earth pressure caused buldge-in of the masonry.

It is quite impossible to analyze this data in its present state because all the important factors, including type of linings, depth of cover, character of rock, etc., are varying over a wide range with most of them either poorly defined or simply unknown; but nevertheless, it is clear that most of the damage was concentrated either near the entrances (or portals) or in shallow tunnels, and that ground failure (landslides) which overload the entrances may be the reason for the heavy-damage cases.

Okamoto (1973) and Okamoto et. al. (1963) studied 116 tunnels which were in the area hit by the 1923

Kwanto Earthquake. They found that 82 tunnels suffered damage consisting of failure of portal sections, transverse and longitudinal cracking of the linings, spalling, and deformations. In seeking a correlation between lining thickness and damage, regardless of geology, it was found that earthquake damage was greater in sections with thicker lining as summarized in Table 4-1.

Lining Thickness	Damage Ratio
40 cm (16 in)	82%
30 cm (12 in)	38%
20 cm (8 in)	10%

TABLE 4-1 CORRELATION OF LINING'S THICKNESS AND DAMAGE

Comparing areas considered to be of identical geological calssification, the thickness of the lining was still varied according to degree of deterioration of rock; and again it was found that the damage ratio was higher in sections with thicker lining.

Comparing damage ratio according to geological classification, neglecting the differences in lining thickness, it was found that the damage rate is progressively reduced from maximum damage rate in soil or soil and gravel to minimum in hard rock.

Geological Conditions	Damage Ratio
Hard rock	16%
Soft Rock	40%
Rock with Joints	44%
Soil and Soil with Gravel	61%

TABLE 4-2 DAMAGE RATE VERSUS GEOLOGICAL CONDITIONS

It may be concluded that the earthquake damage is greater, the poorer the geology and the thicker the lining. Furthermore, it seems that a more suitable method to improve the tunnel safety is to strengthen the ground and not only to increase the lining thickness.

4.1.4 Tanna Earthquake (1930)

Tanna (Japan) Earthquake, known also as Idu (or Isu) Peninsula Earthquake occurred on November 26, 1930 (Seismological Notes, 1931), with the epicenter at 35.1°N 139.0E and an intensity of 10 at the Tanna area of interest. Figure 4.2 shows the Idu Peninsula with the Tanna Fault crossing from North to South. The quake's magnitude in Japanese scale was $M_K = 4.3$, which is equivalent to Richter magnitude of 7.0.

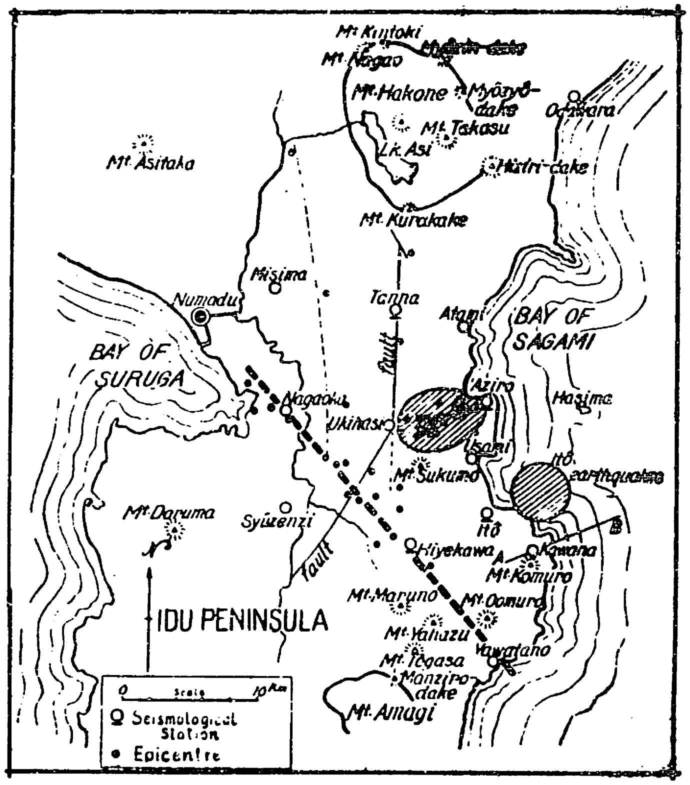

FIGURE 4.2 Distribution of the After-Shocks of the Idu Earthquake (after Nasu, 1931)

The Tanna region suffered from a long period of seismic activities resulting in more than 4880 shocks of sensible degrees in a period of more than two months. The main destructive shock is the one which occurred on November 26, 1930. The significant seismic phenomena in the Idu Peninsula was described in many Japanese papers, in which Otuka (1933), Nasu et. al. (1931), Yamagutti (1937), and Tsuya (1930) all summarized data pertinent to the earth movement along the Tanna Fault.

The Tanna Fault intersected the Tanna Tunnel lying 150 m (492 ft) below the ground surface of the Tanna Basin. According to a precise levelling summarized by Tsuboi (1931), the ground movement was a complex tilting which continued long after the earthquake, as measured later by Takahasi (1931). A second stage of precise levelling was made by Takahasi (1931), who found that "deformations are mostly combined with faults or with the place where the strata makes sudden changes in strength, and the remaining part of the tunnel is generally less active." Takahasi (1931) based his conclusions on measurements conducted in the Tanna Tunnel after the Idu Earthquake, while comparing his exact levelling with the geological profile of the tunnel. High deformations were observed while passing from solfataric clay to andasite, from fault breccia to andasite, from agglomerate to solfataric clay, from agglomerate to andasite (fault), etc.

The Tanna Tunnel was being excavated in nearly east-west direction, perpendicular to the fault and through the epicentral region of the earthquake! The total planned length of the tunnel was 7810 m (25,000 ft). Work was begun at both ends; and at the time of the Idu

Earthquake, the eastern portion had advanced about 3351 m (11,000 ft) and the western about 3652 m (12,000 ft), the latter just crossing the Tanna Fault. The tunnel had been excavated through soft materials, such as sand, volcanic scoria, agglomerate, and clay. During the earthquake, the tunnel was displaced 235 cm (7 ft, 10 in) horizontally by a left-hand strike slip and a westward downthrow of 60 cm (2 ft). On the surface above the tunnel, the displacements were described as "much less", with the strike-slip measured at one point up to 70 cm (2 ft, 3.5 in). Except for the displacement along the fault, the only damage to the tunnel was a few cracks in the concrete walls. This is an interesting point as the acceleration in the tunnel is assumed to be 0.4 g (10 km from M = 7). At the same time, in a village situated 160 m (525 ft) above the tunnel, 55% of the dwellings were destroyed, and 40% of the buildings were destroyed in two other nearby villages. The aboveground buildings were built on a lake deposit of sandy clay and boulders above 40 m (131.2 ft) of overlying volcanic materials and agglomerate. Richter (1958) noted that in both the Tanna Tunnel and the Tehachapi Tunnel (See paragraph 4.1.7) the displacements on the surface were less than in the tunnel.

4.1.5 <u>Fukui Earthquake (1948)</u>

Fukui Earthquake occurred on June 28, 1948 with a magnitude of 7.2, the epicenter being 36.1°N 136.2°E (kawasumi, 1951; Bull. SSA, 1948). The railway system suffered from serious aboveground damage including deformations of bridges, cave-ins of embankments, settlements in soft alluvium, lifting up and shifting of tracks, and collapse of stone walls. (Special Committee, 1950; Sakabe, 1960). In comparison, the damage to a tunnel near Kumasaka was light with the portal arches of a brick-lined tunnel partially fractured. (Far East Command, 1948). More damage was caused to a large concrete culvert which was badly cracked at midlength (Okamoto, 1973). No damage was reported during the aftershocks (Omote, 1950).

4.1.6 <u>Off-Tokachi Earthquake (1952)</u>

This earthquake, also called the Hokhaido Earthquake, occurred on March 4, 1952, with a magnitude of 8.1. Some estimates were ranging from 8.0 up to 8.25. The epicenter was 70 km (43.7 miles) off the coast, undersea, 42.5°N 143.5°E. Most of the damage was caused by tsunamis in the coast and settlements of embankments and river levees. The damage in tunnels -- in zones of Intensities IV-V (Japan scale) -- was minor with cracking of both concrete walls and brick-lined tunnels. (Special Institute

Committee, 1952; Duke, 1956).

4.1.7 <u>The Kern County, California Earthquake (1952)</u>

This earthquake of July 21, 1952, was the major shock in a region which suffered many earthquakes of magnitude greater than 5. Epicenters of major shocks are shown in Figure 4.3, and a close-up of the area of interest is given in Figure 4.4. The only earthquake which caused damage to the tunnels has a magnitude of 7.6; and even in this case, the damage was due to fault movement and not "shaking." The sever damage caused in four timbered tunnels on the Southern Pacific Railroad was concentrated near Bealville, about 24 km (15 miles) northwest of Tehachapi (Southern Pacific Company, 1955). The rail track in this area crosses the zone of the White Wolf Fault twice. (Figure 4.4). Other tunnels, eight more in the region between Tehachapi and Bakersfield, which were outside but adjacent to the area of the ground fractures, suffered only slightly. All of these tunnels, built in 1876, were constructed with timber linings which were covered several years later with 30-60 cm (12-21 in) of reinforced concrete linings without removing the timber.

The first tunnel, covered by 50 cm (150 ft) of decomposed Diorite, was faulted, wrecked, and 63 m (200 ft) out of 210 m (700 ft) were daylighted. This tunnel, without any ground cracks found above it, was crossed by an active fault which was found during daylighting.

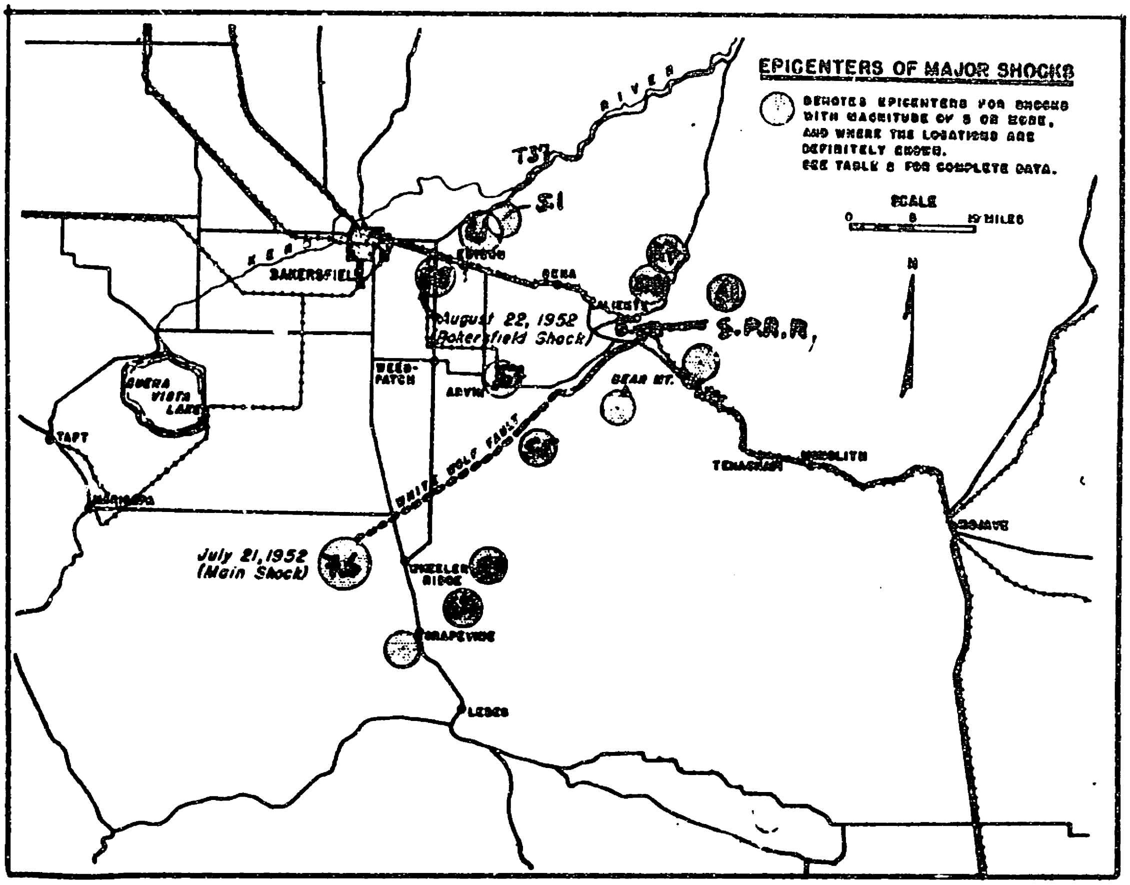

FIGURE 4.3 <u>Epicenters of Major Shocks in the Bakersfield Area</u> (after BSSA, 1954)

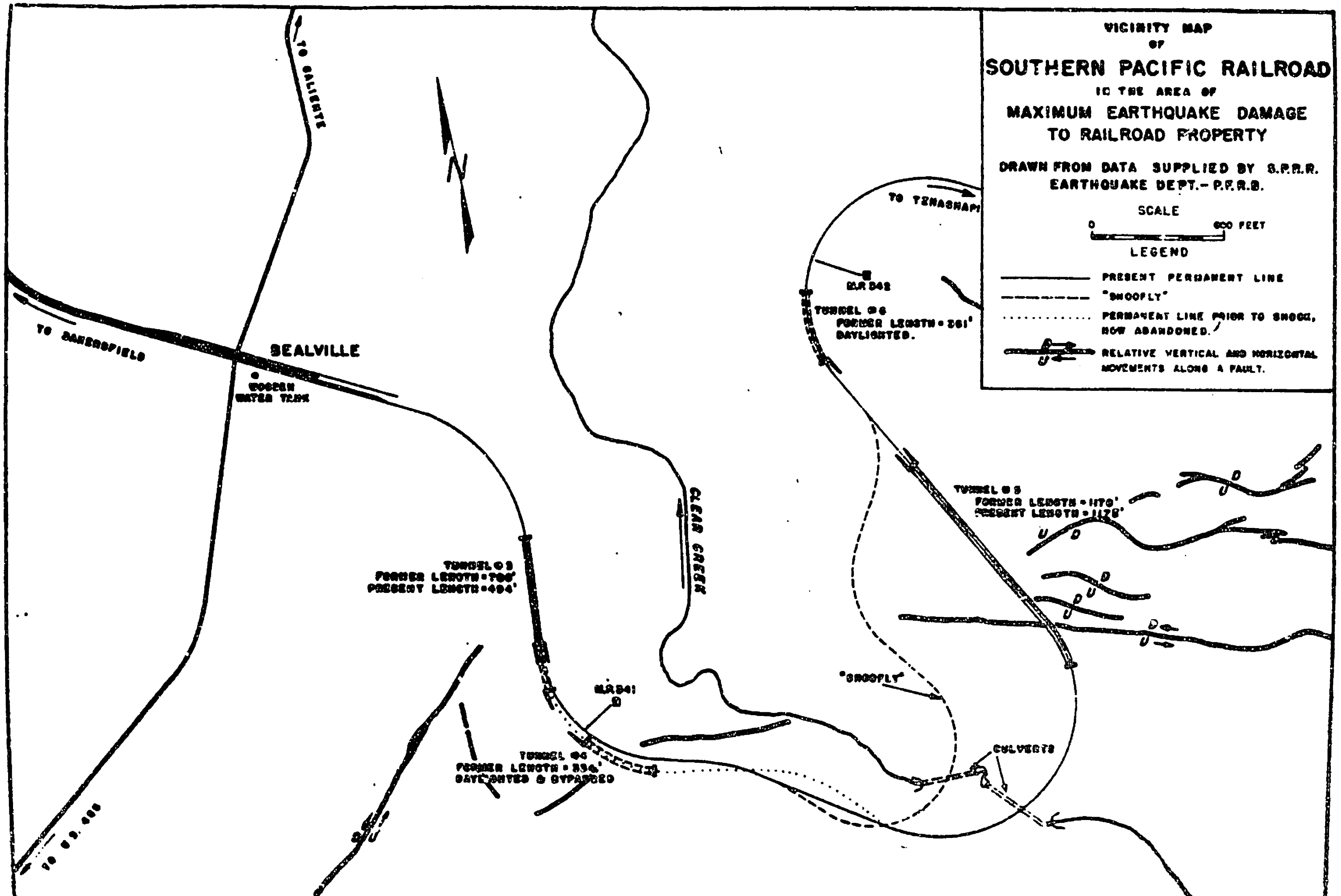

FIGURE 4.4 Fault-crossing in the Tehachapi Pass (after BSSA, 1954)

A second tunnel of 180 m (360 ft) was covered by only 15 m (50 ft) of decomposed Diorite and was daylighted to its full length.

A third tunnel which was covered with 37.5 cm (125 ft) of same Diorite, crossed a fault and was daylighted.

The fourth tunnel, which was the deepest, was covered by 67.5 m (225 ft) of the same Diorite and was wrecked by a fault crossing.

All the severe damage was due to thrusting on the White Wolf Fault, which broke and offset the tunnel structures.

Other types of damage were: shortening of the tunnels by displacements; bending of rails in the tunnels at the entrances; and raising of a wall which then covered a bent rail that moved horizontally under the wall.

The tunnels were located in the region of heaviest shaking of Modified Mercalli M, but it is clear that the damage was caused by fault movement and not by the shaking (Steinbrugge, 1954). Other aftershocks causing accelerations up to 0.2 g did not cause any damage. (See table in Chapter 5).

4.1.8 Kita-Mino Earthquake (1961)

This earthquake occurred on August 19, 1961, with a magnitude of 7.0 to 7.2 and focal depth of 17.5-25 km (10.9-15.6 miles) at 36.0°N 136.5E. It was different from other earthquakes reported in Japan, as the ground was mainly rock and the vibration seemed to be very short, but violent. (JSCE, 1953; Okamoto, 1973).

The area, comprised of jointed igneous rocks, had rock slides even in normal times. There were many large-scale landslides during the earthquake, and many above-ground structures were damaged by falling rocks (Yoshima et. al. 1961).

An underground powerhouse of 77 m (254 ft) in length, 22.5 m (74.3 ft) in width, and 42.8 m (140.2 ft) in height, which was located 20 km (12.5 miles) from the epicenter, did not suffer from any damage (Kishinoue, 1962; Okamoto, 1973). According to workers at the dam and the powerhouse, the earthquake motion appeared to have been slighter in the underground powerhouse than above the ground.

An aqueduct tunnel passing through soft ground was cracked.

4.1.9 Niigata Earthquake (1964)

The Niigata Earthquake occurred on June 16, 1964 with a magnitude of 7.5, the epicenter 203 km (126.9 miles) southwest of Awashima Island (139°E 382°N), and at depth of 40 km (25 miles) (JNCEE, 1964).

A tunnel built in soft soil on the Uetsu Line suffered cracking of the portal and spalling of concrete at the crown of the lining over a great length (Okamoto, 1973; Kawasumi, 1964). Another tunnel, Teraska, suffered both spalling of concrete along the crown and crushing of the invert at the bottom of the sidewall on the mountainside. (Figures 4.5, 4.6). On the surface above the tunnels, cracks were found perpendicular to the centerline of the tunnel. An interesting summary on the damage to the tunnels in soil was made by Kawasumi (1964): "After the earthquake we observed cracks on linings of many tunnels, particularly on those which have thin layers of burden or near the entrance of the tunnels. The Terasaka Tunnel,described earlier, has a record that it was driven through the landslides, and the engineers and laborers engaged in the construction work of the tunnel were so much troubled by the large earth pressures as to be compelled to consolidate the ground with cement milk grouting. The thickness of the concrete lining amounted to 1 m (3 ft) where the lining received maximum

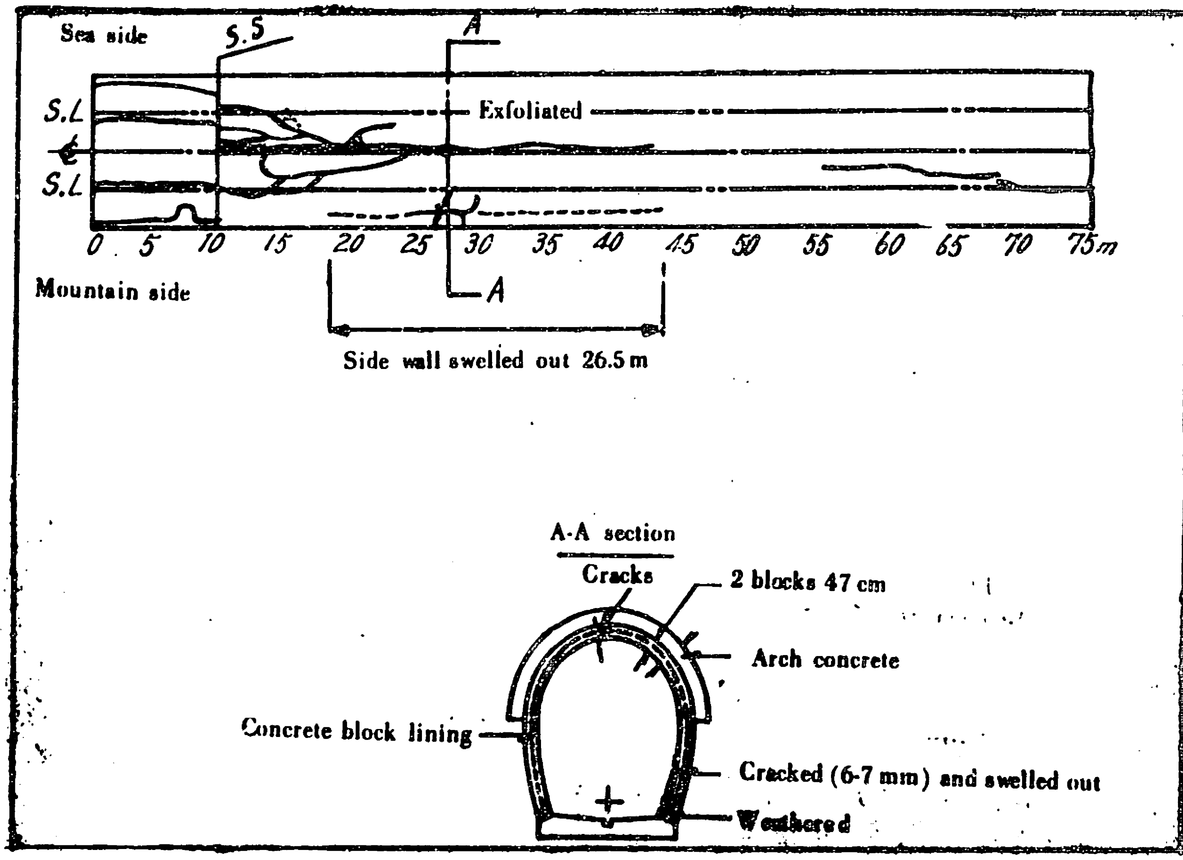

FIGURE 4.5 Cracks Found in the Arch Concrete of Terasaka Tunnel (after Okamoto, 1973)

earth pressure."

Many sewers in soft soil, both of box-type culverts and circular pipes, were violently damaged by separation, cracking, snaking, variation of height, and upheaval due to liquefaction.

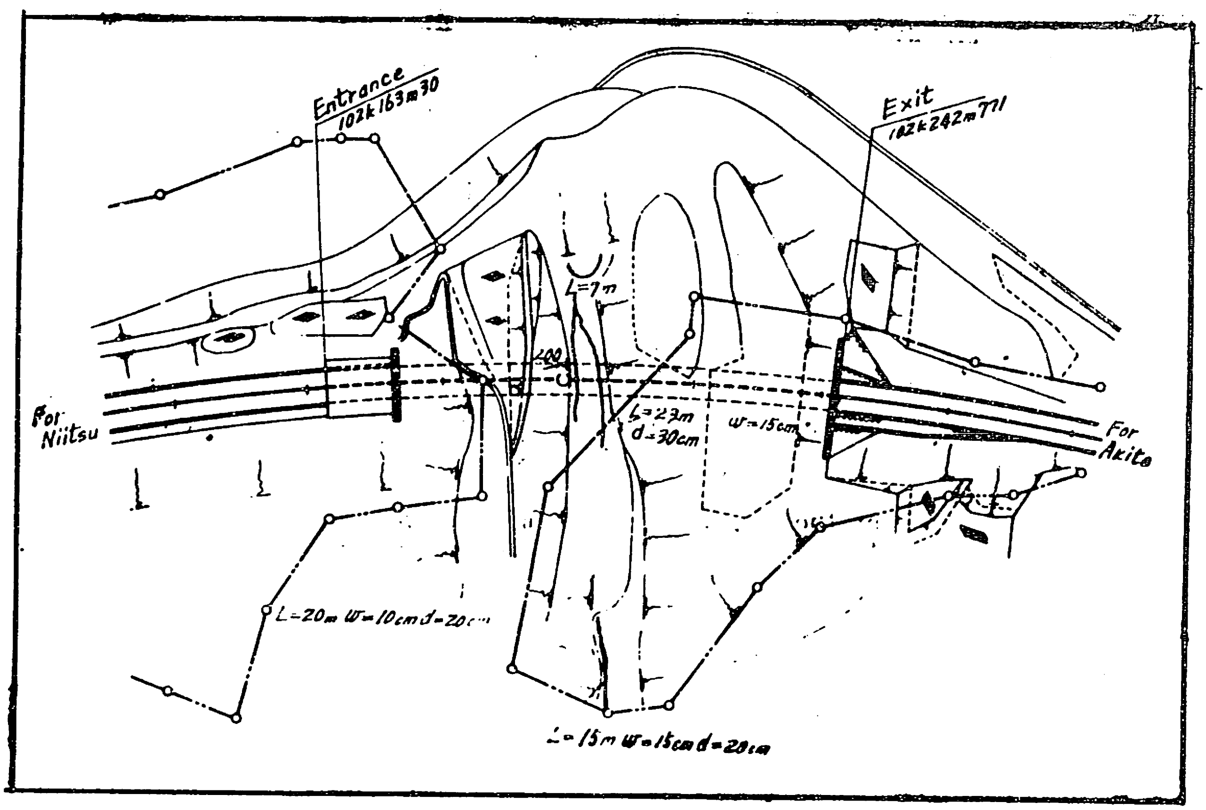

FIGURE 4.6 Cracks Found on the Ground Surface Just Above Terasaka Tunnel. (L: length, w: width, d: depth of a crack.) (after Okamoto, 1973)

4.1.10 Great Alaskan Earthquake (1964)

The Alaskan Earthquake occurred on March 27, 1964, and was the first earthquake studied carefully in detail. It had a Richter magnitude of 8.4, which classified it among the world's greatest. (Nat. Res. Counc., 1973).

During the earthquake, the Alaskan Railroad had six tunnels on the mainline between Seward and Portage and two tunnels on the Whittier Branch. All eight tunnels were unlined and had sustained "very little damage." The Whittier Tunnel, over 4500 m (15,000 ft) long, is constructed through a grey rock formation that required very little shoring. This tunnel had on each end a wooden door to reduce freezing which remained undamaged.

The only problem caused by the earthquake within the tunnels was an overhead ravelling of the material which fell on the track. According to the report, "damage to tunnels was considered insignificant, compared to severe damage to railroad surface facilities in other areas."

The Eklutna Project suffered from heavy damage to aboveground structures, intake structures, and earthdam. The damage to a precast conduit was mainly in the joints. No damage occurred to the circular, concrete-lined pressure tunnel. This tunnel, 7182 m (25,000 ft) long, 2.75 m (9 ft) inside diameter, is passing through varied graywacke rock ranging from broken graywacke with inclusions of argillite to thin bedded argillite with some thin graywacke beds.

4.1.11 San Fernando Earthquake (1971)

The San Fernando Eartnquake occurred on September 9, 1971, with a relatively moderate magnitude -- 6.4-6.6 -- at focal depth of 13 km (8.13 miles). The main shock was accompanied by more than 200 aftershocks; but, of engineering significance were four $M = 5.1$ and thirteen $M = 4.0$ to 4.5 aftershocks, which were measured during the first four hours after the main shock and which may have tributed to observed damage. The epicenter (34° 24'N 118° 23.7'W) was located about 13.9 km (8.7 miles) north-northeast of San Fernando, in the San Gabriel Mountains south of Soledad Canyon. (Figure 4.7). The shaking lasted for 10 seconds with a very violent vertical motion. The earthquake's effects were studied by NOAA, and the data herein is from Volumes II and III of this report (NOAA, 1973).

The Balboa Inlet Tunnel is 1.6 km (1 miles) long and 4.2 m (14 ft) in diameter. The damage was concentrated in an area 26 m (87 ft) long, which is approximately 40 m (133 ft) south of the lower branch of the Santa Susana Fault. The damage consisted of severe spalling and breaking of reinforced steel bars. The tunnel remained stable. In this zone, the tunnel passes under a canyon; and, "we believe that the zone of damage is a

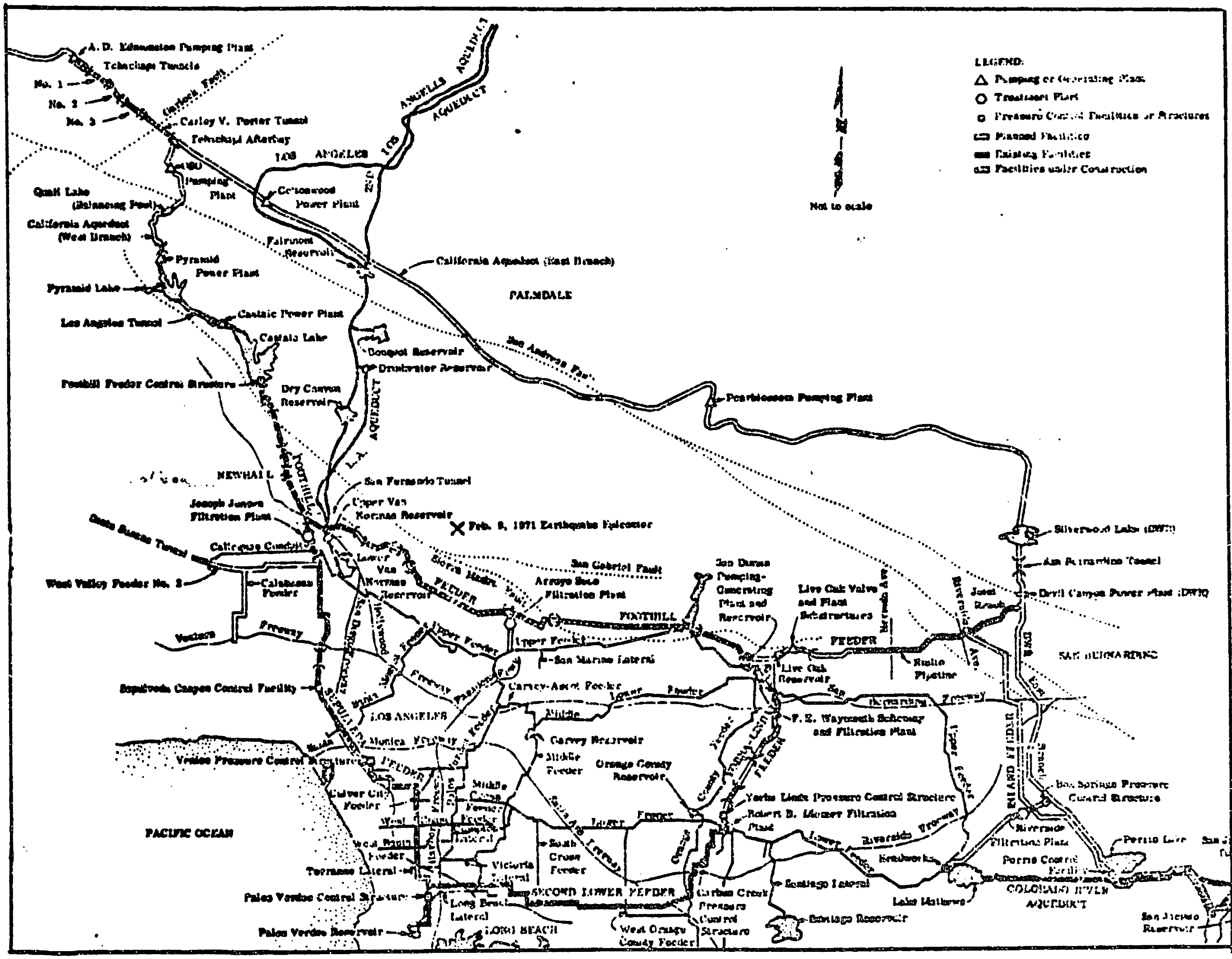

FIGURE 4.7 Major California Aqueduct Facilities Location Map

result of strong ground shaking in the tunnel under local shallow cover." (NOAA, 1973). This explanation seems logical when compared to damage done to shallow tunnels in Japan. (See preceeding sections.) The invert at the south portal rose 8 m (27 ft) relative to the damaged area. The tunnel remained on grade north of the damaged area.

It is important to note that while surface cracks appeared along the upper branch of the Santa Susana Fault, which caused lateral displacements in the Golden State Freeway and Foothill Boulevard, "damage to the tunnel lining was not observed directly beneath these cracks." The observation is opposite to that in the Kern Earthquake.

The San Fernando Tunnel is 5.5m (18 ft) in diameter, 8845 m (29,000 ft) long, suffered from cummulative total vertical displacements of 2.3 m (7.5 ft) along 9 km (5.6 miles). The east portal is just north of the Sylmar Fault where the displacement was maximum. The tunnel was driven through soft alluvium, saturated silt, sand, and gravel. No shear surfaces were recognizable in the tunnel. It is interesting to note that miners working at the face during the earthquake drove out easily with a locomotive which means that the rails did not suffer from excessive distortion during the quake. The structural damage to the tunnel itself was "very minor, consisting principally of cracking and spalling of a few of the concrete tunnel supports." (NOAA, 1971). This damage is far less than the excessive damage caused to Veteran's Hospital, Van Gogh Street, Joseph Jensen Filtration Plant, and many new buildings in the nearby area.

The Maclay Tunnel is 2 m (6.5 ft) in diameter, 2.9 km (1.8 miles) long, and was built before 1918. In comparison to the new tunnel of San Fernando and Balboa mentioned above, this tunnel suffered considerable structural damage consisting mainly of wide, long cracks, but no local buckling or other failure. The Chatsworth High Line, which contains several tunnels of 1.83-1.98 m (6-6.5 ft) diameters and built at the same period with identical design, suffered only slight damage.

Techapi crossing is part of California's Aqueduct which consists of four concrete-lined tunnels. (a) Tehachapi Tunnels 1, 2, and 3 are 7.2 m (23.5 ft) in diameter with total length of 4926 m (16,152 ft). (b) Carley V. Porter Tunnel is 6 m (20 ft) in diameter and 7650 m (25,075 ft) long. An inspection conducted immediately following the quake (as none of them contained water) revealed that "no structural damage had been sustained." (NOAA, 1973).

The first Los Angeles Aqueduct, constructed prior to 1913 and located immediately north of the Van Norman Reservoir, consists of tunnels lined with unreinforced concrete. The inside dimensions are approximately 2.95 m (9 ft, 8 in) wide by 3.2 m (10 ft, 6 in) high. There was no severe damage, but "hundreds of new fractures were observed in the southern entrance and became less severe proceddeding upstream. The fractures were primarily circumferential, but longitudinal and diagonal patterns were also quite prevalent. It is important to note that "while the lining was heavily cracked with separations ranging from hairline cracks up to 8 mm (1/4 in), all lining remained in place; and it was not necessary to remove or replace any sections.

4.2 RELATED EXPERIENCE

Except the case histories which were described in Section 4.1, some other cases were described which may illustrate some of the conclusions arrived at in Chapter 3.

Staunton (1918) told about violent shaking felt by some miners in a mine at Tombstone, Arizona during the Sonora Earthquake of 1887. Small rockfalls occurred in the mine, but there was no collapsing in the unlined walls. The damage aboveground consisted in falling plaster and chimneys and shifting of engines on their foundations.

Pardee (1926) stated that an earthquake in 1925 was hardly noticed in mines at Butte and Barker, Montana, but was felt at the surface.

Duke (1959) told about similar cases in which an aftershock (1952) of the Kern County Quake has not been felt in Crystal Cave, Sequoia, but was sharply felt outside the cave.

Duke and Leeds (1959) summarized their correspondence with the Southern Pacific Company, Los Angeles Department of Water and Power and the Pacific Gas and Electric Company, by stressing that "no earthquake damage has been found to date." A similar survey was made by the author during 1975-1976, including the US Bureau of Reclamation, California Department of Water Resources, Metropolitan Water District of Southern California in addition to the three agencies referred to by Duke. All the agencies stated clearly that no damage occurred to their tunnels from earthquake shaking except some minor damage reported in the case histories on Chapter 4.

The Bureau of Reclamation (1976) stated that: "Although many earthquakes have occurred in California in areas where thie Bureau has constructed tunnels, none of the tunnels have suffered severe damage. Inspections have revealed only minor cracking in some of the concrete linings, but this cracking could not be proved to be the result of the earthquake action."

The Los Angeles Department of Water and Power (1976) stated that "to date, we have not experienced significant tunnel damage due to earthquakes with the following exceptions:" During the San Fernando Earthquake (1971), "heavy cracking and fracturing of concrete lining occurred in First Aqueduct tunnels near the Cascades... The repair program consisted mainly of crack repair, patching, and grouting. There was no significant displacement occurring within the tunnel sections." The Second Aqueduct tunnels received no damage from this earthquake although pipeline damage occurred in six locations. The Pacific Gas and Electric Company (1976) stated that "we are not aware of any significant or measurable damage to any of these tunnels resulting from seismic activity."

All these related data contribute to the general qualitative conclusion that shaking damage to tunnels is quite minor and is negligible compared to damage caused to aboveground structures in the same area.

CHAPTER 4

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CHAPTER 5

ANALYSIS OF CASE HISTORIES

5.1 INTRODUCTION

This cahpter is basically a short summary, in a table format, of the case histories described in length, and somehwat "journalistically", in Chapter 4, coupled with the estimated peak amplitudes of the ground motions which are thought to be occurred during the described earthquakes as calculated from the attenuation laws described in Chapter 3. The data collected and summarized in this chapter should serve as the "input" data for any correlation between the observed damage and the level of shaking which is thought to cause it.

The chapter is divided into three parts: (a) Explanation of the format and contents of the tables; (b) The tables; and (c) Discussion on possible correlations. The table itself is divded into three main categories arranged in three sub-tables which may help the reader to concentrate on the essentials of each category. The three categories are: (a) The tunnel; (b) The damage; and (c) The earthquake. Table 5-1 serves as background material on the tunnels. It contains the name and location of the tunnel (maps or sketches may be found in the respective sections of Chapter 3), structural data such as cross-section and lining material, and geological data such as ground conditions and depth of cover.

As the original papers were somewhat descriptive and not systematic, and as the original writers were usually interested in damage to aboveground structures, the information given is hardly satisfactory, sometimes just not available, and sometimes too "journalistic" to be of engineering value. For example, "loose surface rock" can hardly serve as "input" for correlation: Which rock? What are its properties? What are its dimensions, both vertically and horizontally? Is the tunnel built in the same rock? Is the tunnel built in other material? etc. The writer may count many other questions as those, but the situation will remain the same: The available geological data is very poor!

One of the columns in this table links the two other tables by naming the earthquake causing the damage.

Table 5-2 gives a short description of the reported damage. As explained earlier in Chapter 2, there are usually four potential mechanisms for earthquake damage, but only one of them -- the ground shaking -- is of interest in this report. As the case histories in Chapter 3 contain all mechanisms, except damage from tsunamis, which were never reported in the literature, there arose a question how to screen out the damage caused not by shaking.

It is the writer's opinion that the damage caused by fault movement or ground failure should be included in the table, as it gives a better perspective on the range of damage and gives the reader a better understanding of what might be expected. The damage is divided into three categories (three columns):

(a) Damage caused by ground shaking;

(b) Damage caused by fault movement, where a tunnel crossed an active fault;

(c) Damage caused by either ground failure or portal failure. The first is usually either liquefaction (as in Niiagata) or landslides and rock falls on slopes. The second is usually either from landslide just above the portal or from shaking. As the portal of a tunnel is a special boundary problem which is not similar to the usual two-dimensional phenomenon of long tunnels, it was decided <u>not</u> to include damage to portals, even if it was caused by shaking, in the "shaking-damage" category.

Table 5-3 summarizes the assumed level of shaking of the destructive earthquakes. The first three columns --magnitude, distance, depth--give the basic data which were obtained from the references in Chapter 3 and especially in Seismological Notes (Bull. SSA), International Seismological Summary (Oxford's Observatory), and the reports of special committees for the various earthquakes. In many cases where different magnitudes, locations of focus, or depths were reported, a value was chosen either as an average or as the most recent information.

With the attenuation laws discussed in Chapter 3, the three peak amplitudes -- displacement, velocity, and acceleration -- were calculated. These peak amplitudes are "at surface"; and no reduction has been made for depth effect because, as already stated: The geological conditions at the sites are not clear enough, and there is not sufficient information to conclude a numerical value for the reduction (or deamplification) factor.

The last two columns are both for the sake of completeness and for a better perspective of the associated surface damage. Intensity may help to correlate the underground "minor" damage to the more severe aboveground damage. The duration is important to determine the role of fatigue or cycle-loading on the damage. As discussed in Chapter 3, it is known that cycle-loading may cause damage and failure even in low level stresses.

No references are given in this chapter or in the tables, as they are based on the sections of Chapter 3, which has a list of references for each earthquake.

5.2 SUMMARY OF DATA

Refer to Tables 5-1, 5-2, 5-3 beginning on page 195.

5.3 CORRELATION OF DAMAGE AND SHAKING LEVEL

As has already been mentioned several times during Chapter 4, the information accumulated in the case histories is poor, and it lacks the level of precision and detailed description which is so essential to any quantitative analysis. However, the data does permit a correlation between the level of damage and the level of shaking as determined by peak-ground acceleration and velocity.

Figure 5.1 demonstrates the basic data as summarized from Table 5-3. The abscissa is the ordinal number of the case histories as used in Tables 5-1 to 5-3, and the ordinate is the estimated peak ground acceleration as calculated by using McGuire's Attenuation Law. Figure 5.2 is similar to Figure 5.1, but for peak ground velocity.

TABLE 5-1

TUNNEL'S DATA

No.	Earthquake	Tunnel	Location	Cross Section	Lining	Ground Conditions	Depth of Cover ft(m)
1	San Francisco (Central CA) 1906	Wright-1	Santa Cruz Mts. R.R.	13.0 wide	Timber	Shale, Serpentine, Soapstone	702 (214)
1a		Wright-1	"	"	"	"	"
1b		Wright-1	"	"	"	"	"
2		Wright-2	Near Glendale	"	"	Sandstone, Jasper	678 (206)
2a		Wright-2	"	"	"	"	"
2b		Wright-2	"	"	"	"	"
3	Kwanto (Kanto) Region, 1923 or	Terao	Yokohama L.		Bricks		
4	Great Tokyo 1923	Hichigama	Yokosuka L.				
5		Taura	"			Loose surface rock	50 (15)

TABLE 5-1 Continued

No.	Earthquake	Tunnel	Location	Cross Section	Lining	Ground Conditions	Depth of Cover ft(m)
6		Numama	Yokosuka L.				
7		Nokogiri Yama	Honjo L.		Concrete		
8		Kanome Yama	Atami L.			Boulders in Slope	
9		Ajo	"				
10		Ippamatzu	"				
11		Nagoye	"				100 (30)
12		Komine	"		Reinforced Concrete		5-20 (1.5-6)
13		Fudu San	"			Thin, loose material on hillside	60 (18)
14		Meno Kamiama	"		Masonry	Loose rock	55 (16.5)

TABLE 5-1 Continued

No.	Earthquake	Tunnel	Location	Cross Section	Lining	Ground Conditions	Depth of Cover ft(m)
15		Yonegami-Yama	Atami L.		Masonry		165 (50)
16		Shimomaki-Matzu	"		"		95 (29)
17		Happon Matzu	"			Loose mat-erial on steep slope	65 (20)
18		Nagasaha Yama	"		Brick and concrete		300 (90)
19		Hakone-1	Tokaido L.				200 (61)
20		Hakone-2	"				
21		Hakone-3	"				150 (46)
22		Hakone-4	"				160 (49)

TABLE 5-1 Continued

No.	Earthquake	Tunnel	Location	Cross Section	Lining	Ground Conditions	Depth of Cover ft(m)
23		Hakone-7	Tokaido L.			Fissured, faulted, weathered rock	100 (31)
24		Yose	Chuo L.			Soft, fine grained rock	65 (20)
25		Doki	Sobo L.		Brick		
26		Hamuya	Hojo L.		Brick and Concrete		250 (75)
27		Mineoka Yama	Ampo L.		Masonry	Some basalt, deformed rock	
28	Idu Peninsula 1930	Tanna	Tanna Basin		Concrete	Agglomerate volcanic scoria sand, clay	490 (150)
29	Fukui, 1948	Kumasaka			Brick		
30	Off Tokachi 1952				Concrete and Brick		

TABLE 5-1 Continued

No.	Earthquake	Tunnel	Location	Cross Section	Lining	Ground Conditions	Depth of Cover ft(m)
31	Kern County 1952	S.P.R.R. 3	Tehachapi Pass		Timber and 12-21 in. of concrete	Decomposed diorite	150
31a		"	"		"	"	"
31b		"	"		"	"	"
32	Kern County 1952	S.P.R.R. 4	"		"	Decomposed diorite, many surface cracks	125
33		S.P.R.R. 5	"		"	"	250
33a		"	"		"	"	"
33b		"	"		"	"	"
34	Kern County 1952	S.P.R.R. 6	"		"	Decomposed diorite	50
34a		"	"		"	"	"

TABLE 5-1 Continued

No.	Earthquake	Tunnel	Location	Cross Section	Lining	Ground Conditions	Depth of Cover ft(m)
34b	Kern County 1952	S.P.R.R. 6	Tehachapi Pass		Timber and 12-21 inches of concrete	Decomposed diorite	50
35	Kita Mino 1961	Power-house		254(77) length 74 (22) width 140(43) height		Jointed igneous rock	
36		Aqueduct				Soft ground	
37	Niigata, 1964	Negugaseki	Uetsu Line				
38		Terasaka					
39	Great Alaska 1964	Whittier-1	Whittier		Unlined	Gray rock	
40		Whittier-2					

TABLE 5-1 Continued

No.	Earthquake	Tunnel	Location	Cross Section	Lining	Ground Conditions	Depth of Cover ft(m)
41	Great Alaska 1974	Seaward-1					
42		Seaward-2					
43		Seaward-3					
44		Seaward-4					
45		Seaward-5					
46		Seaward-6					
47	San Fernando 1971	Balboa Inlet	M.W.D. of South. CA	11 D	Reinforced concrete		
48		San Fer-nando		18 D	R.C.	Alluvium soft saturated silt, sand, and gravel	
49		McLay		6.5 D			
50		Chatsworth		6-6.5D			

TABLE 5-1 Continued

No.	Earthquake	Tunnel	Location	Cross Section	Lining	Ground Conditions	Depth of Cover ft(m)
51		Tehachapi 1		23.5 D			
52		Van Norman Inlet					
53		Tehachapi 2	CA Aque-duct	23.5 D			
54		Tehachapi 3		23.5 D			
55		Carley Porter		20 D			
56		Van Norman North	LA Aque-duct	8'8" x 10'6"	Unreinforced concrete		
57		Saugus					
58		San Fran-ciscquito					
59		Elizabeth					

TABLE 5-1 Continued

No.	Earthquake	Tunnel	Location	Cross Section	Lining	Ground Conditions	Depth of Cover ft(m)
60		Antelope					
61	Inyokern 1946	Jawbone					
62		"					
63		"					
64		Freeman					
65	Arvin Tehachapi	Saugus					
66		San Fran-ciscquito					
67		Elizabeth					
68		Antelope					
69		Jawbone					
70	Chalome, 1922	Jawbone					
71		Freeman					

TABLE 5-2

DAMAGE DATA

No.	Earthquake	Tunnel	Damage Due To Shaking	Damage Due To Fault Movement	Damage Due To Ground Failure & Other Reasons
1	Central CA (San Francisco)	Wright-1	Caving in of rock and some breaking of timber but to lesser extent compared to damage near the fault	Caving in of rock from roof and sides. Breaking in flexure of upright timber. Upward heaving of rails. Breaking of ties. Blocked in several points. Transverse horizontal offset of 4.5 feet under the fault.	
1a		"	No damage!		
1b		"	No damage!		
2	San Francisco 1906	Wright-2	Broken timber, roof caved in.		
2a		"	No damage!		
2b		"	No damage!		

TABLE 5-2 Continued

No.	Earthquake	Tunnel	Damage Due To Shaking	Damage Due To Fault Movement	Damage Due to Ground Failure & Other Reasons
3	Tokyo, 1923 (Kwanto)	Terao			Cracked brick portal
4		Hichigama			Landslide at entrance
5		Taura			Landslide at entrance
6		Numama			Cracked brick portal
7		Nokogiri-Yama	Concrete walls fractured slightly. Some spalling of concrete.		
8		Kanome-Yama			Entrance buried by landslide. Some damage to masonry portal.
9		Ajo			Landslides at entrance Damage to masonry portal.
10		Ippamatzu	Masonry dislodged near floor, in interior		Cracks in masonry near portals.

TABLE 5-2 Continued

No.	Earthquake	Tunnel	Damage Due To Shaking	Damage Due To Fault Movement	Damage Due to Ground Failure & Other Reasons
11		Nagoye	Interior cracked.		
12		Komine	Destroyed. RC blocks tilted. Ceiling slabs caved in. Formed section cracked.		
13		Fudu San	Clean interior!		Cracked masonry portal.
14		Meno-Kamiama	Partial collapse.		
15		Yonegami-Yama	Minor interior masonry damage.		Cracks near portal.
16		Shimomaki-Matsu	Deformed masonry in interior.		Portals closed by slides.
17		Happon-Matsu	Badly cracked interior.		Buried by slides.
18		Nagasahu Yama	Some interior fractures in brick and concrete.		

TABLE 5-2 Continued

No.	Earthquake	Tunnel	Damage Due To Shaking	Damage Due To Fault Movement	Damage Due To Ground Failure & Other Reasons
19		Hakone-1	Interior cracked.		
20		Hakone-2	Undamaged!		
21		Hakone-3	Cracks in interior.		Ceiling collapsed near portal. Some damage to masonry portal.
22		Hakone-4	Collapse of loose material.		Entrance almost completely buried.
23		Hakone-7	Interior collapse.		Landslides buried entrances.
24		Yose	Shallow portions collapsed and daylighted.		
25		Doki	Collapses at shallow parts.		
26		Humuya	Cave in. Cracks with 10 inch (25cm) displacement.		Landslide.

TABLE 5-2 Continued

No.	Earthquake	Tunnel	Damage Due To Shaking	Damage Due To Fault Movement	Damage Due To Ground Failure & Other Reasons
27		Mineoka-Yama	Cracks in bulges in masonry from local earth pressure.		
28	Idu Peninsula 1930	Tanna	Few cracks in walls.	7'10" horizontal displacement, 2' vertical displacement just across the Tanna fault.	
29	Fukui, 1948	Kumasaka			Brick arches of portal partially fractured.
30	Off Tokachi 1952		Minor cracks in both brick and concrete linings.		
31	Kern County 1952	S.P.R.R. 3		Wrecked under White Wolf Fault. Daylighted.	
31a	(aftershock)	"	No damage!		
31b	(aftershock)	"	No damage!		

TABLE 5-2 Continued

No.	Earthquake	Tunnel	Damage Due To Shaking	Damage Due To Fault Movement	Damage Due To Ground Failure & Other Reasons
32		S.P.R.R. 4		Wrecked under fault . Daylighted.	
33		S.P.R.R. 5		Wrecked under fault.	
33a	(aftershock)	"	No damage!		
33b	(aftershock)	"	No damage!		
34		S.P.R.R. 6		Fractured, day-lighted.	
34a	(aftershock)	"	No damage!		
34b	(aftershock)	"	No damage!		
35	Kita Mino 1961	Power-house	No damage!		
36		Aqueduct	Cracking.		
37	Niigata 1964	Nezugaseki	Spalling of con-crete at crown.		Cracking at portal.

TABLE 5-2 Continued

No.	Earthquake	Tunnel	Damage Due To Shaking	Damage Due To Fault Movement	Damage Due To Ground Failure & Other Reasons
38		Terasaka	Spalling of concrete at crown, crushing of invert at bottom of sidewalls.		
39	Great Alaska 1964	Whittier 1	Some overhead ravelling of loose rock which falls on the track.		
40		Whittier 2	No damage!		
41		Seward-1	No damage!		
42		Seward-2	No damage!		
43		Seward-3	No damage!		
44		Seward-4	No damage!		
45		Seward-5	No damage!		
46		Seward-6	No damage!		

TABLE 5-2 Continued

No.	Earthquake	Tunnel	Damage Due To Shaking	Damage Due To Fault Movement	Damage Due To Ground Failure & Other Reasons
47	San Fernando 1971	Balboa		Severe spalling, breaking of concrete lining, deformations where tunnel passed under canyon at shallow cover, only 120' (36m) south of Santa Suzana Fault. No breaking of reinforcing bar at RC Blocks.	
48		San Fer-nando		Maximum displace-ment and damage near Sylmar Fault.	A large vertical dis-placement of 7.5 ft along 5.6 miles, causing fluxural cracks.
49		McLay	Wide long cracks. No local buckling.		
50		Chats-worth	Slight damage.		
51		Tehachapi 1	No damage!		
52		Van Norman Inlet	No damage!		
53		Tehachapi 2	No damage!		

TABLE 5-2 Continued

No.	Earthquake	Tunnel	Damage Due To Shaking	Damage Due To Fault Movement	Damage Due To Ground Failure & Other Reasons
54		Tehachapi 3	No damage!		
55		Carley Porter	No damage!		
56		Van Norman North	Hundreds of new fractures in concrete lining. No structural damage. Fractures primarily circumferential, also longitudinal and diagonal.		
57		Saugus	No damage!		
58		San Francisquito	No damage!		
59		Elizabeth	No damage!		
60		Antelope	No damage!		
61	Inyokern 1946	Jabbine-1	No damage!		
62		Jabbine-2	No damage!		

TABLE 5-2 Continued

No.	Earthquake	Tunnel	Damage Due To Shaking	Damage Due To Fault Movement	Damage Due To Ground Failure & Other Reasons
63		Jabbine-3	No damage!		
64		Freeman	No damage!		
65	Arirn-Tehachapi	Saugus	No damage!		
66		San Francisquito	No damage!		
67		Elizabeth	No damage!		
68		Antelope	No damage!		
69		Jawbone	No damage!		
70	Chalome 1927	Jawbone	No damage!		
71		Freeman	No damage!		

TABLE 5-3

EARTHQUAKES' DATA

No.	Earthquake	Tunnel	M	R	Depth	a	v	d	I_o	Duration
1	San Francisco 1906	Wright-1	8.3	135	15	0.13	26.9	42.1	10 RF	40
1a			6.1	40	15	0.10	10.4	10.4		<10
1b			6.6	20	15	0.23	25.7	23.7		~10
2	San Francisco	Wright-2	8.3	135.8	15	0.13	26.8	41.9	10 RF	40
2a			6.1	42.7	15	0.10	9.9	10.0		<10
2b	-		6.6	25	15	0.20	22.7	21.6		~10
3	Kwanto, 1923	Terao	8.16	31.6	10	0.47	82.5	91.8		35
4		Hichigama	8.16	36.4	10	0.42	74.8	99.1		35
5		Taura	8.16	31.6	10	0.47	82.5	91.8		35
6		Numama	8.16	46.0	10	0.35	62.8	75.1		35
7		Nokogiri Yama	8.16	70.7	10	0.24	43.9	57.7		35
8		Kanome Yama	8.16	26.9	10	0.52	91.6	117.1		35

TABLE 5-3 Continued

No.	Earthquake	Tunnel	M	R	Depth	a	v	d	I_o	Duration
9		Ajo	8.16	25.0	10	0.55	95.8	112.5		35
10		Ippamatzu	8.16	25,0	10	0.55	95.8	112.5		35
11		Naguye	8.16	24.0	10	0.50	98.1	107.4		35
12		Komine	8.16	26.9	10	0.52	91.6	99.1		35
13		Fudu San	8.16	24.0	10	0.50	98.1	107.4		35
14		Meno Kamiana	8.16	32.0	10	0.46	81.8	91.2		35
15		Yonegami Yama	8.16	32.0	10	0.46	81.8	91.2		35
16		Shimomaki	8.16	36.5	10	0.42	74.7	85.3		35
17		Happon Matsu	8.16	20.0	10	0.63	108.7	112.5		35
18		Nagashu Yama	8.16	22.4	10	0.59	102.1	107.4		35
19	Kwanto, 1923	Hakone-1	8.16	15.6	10	0.72	123.0	123.2		35

TABLE 5-3 Continued

No.	Earthquake	Tunnel	M	R	Depth	a	v	d	I_o	Duration
20		Hakone-2	8.16	15.6	10	0.72	123.0	123.2		35
21		Hakone-3	8.16	17.2	10	0.69	117.4	119.0		35
22		Hakone-4	8.16	19.7	10	0.64	109.6	113.1		35
23		Hakone-7	8.16	22.4	10	0.59	102.1	107.4		35
24		Yose	8.16	26.9	10	0.52	91.6	99.1		35
25		Doki	8.16	61.0	10	0.27	49.9	63.4		35
26		Hamuya	8.16	63.0	10	0.26	48.5	62.1		35
27		Mineoka Yama	8.16	65.0	10	0.26	47.3	60.9		35
28	Idu, 1930	Tanna	7.0	0	--					15
29	Fukui, 1948	Kumasaka	7.2	25.0	10	0.30	39.5	39.3		15-20
30	Off Tokachi 1952	--	8.0	?	?				4-5	30-35

TABLE 5-3 Continued

No.	Earthquake	Tunnel	M	R	Depth	a	v	d	I_o	Duration
31	Kern County 1952	S.P.R.R. 3	7.6	46.0	20	0.24	37.5	42.9		10-15
31a	--	"	6.1	29.0	20	0.13	13.0	12.2		<10
31b	--	"	5.8	21.0	20	0.14	12.0	10.4		~5
32	Kern County 1952	S.P.R.R. 4	7.6	46.0	20	0.35	37.5	42.9		10-15
33		S.P.R.R. 5	7.6	46.5	20	0.24	37.2	42.7		10-15
33a	--	"	6.1	16.0	15	0.19	18.1	15.6		<10
33b	--	"	5.8	16.0	15	0.16	13.8	11.5		~5
34	Kern County 1952	S.P.R.R. 6	7.6	46.5	20	0.24	37.2	42.7		10-15
34a	--	"	6.1	16.0	15	0.19	18.1	15.6		<10
34b	--	"	5.8	16.0	15	0.16	13.8	11.5		~5

TABLE 5-3 Continued

No.	Earthquake	Tunnel	M	R	Depth	a	v	d	I_o	Duration
35	Kita Mino 1961	Powerhouse	7.2	32.0	25	0.25	33.7	39.3		15-20
36		Aqueduct	7.2	?	25					15-20
37	Niigata, 1964	Nezugaseki	7.5	?	40					20-25
38		Terasaka	7.5	?	40					20-25
39	Alaska, 1964	Whittier-1	8.4	75.0	30	0.26	52.0	79.4		45
40		Whittier-2	8.4	75.0	30	0.26	52.0	79.4		45
41		Seward-1	8.4	85.0	30	0.23	46.3	64.8		45
42		Seward-2	8.4	85.0	30	0.23	46.3	64.8		45
43		Seward-3	8.4	100	30	0.19	39.7	60.9		45
44		Seward-4	8.4	100	30	0.19	39.7	60.9		45
45		Seward-5	8.4	110	30	0.19	36.2	56.7		45
46		Seward-6	8.4	115	30	0.17	34.7	56.7		45

TABLE 5-3 Continued

No.	Earthquake	Tunnel	M	R	Depth	a	v	d	I_o	Duration
47	San Fernando 1971	Balboa	6.4	16.0	13	0.23	23.9	21.0	10	15
48		San Fer-nando	6.4	16.0	13	0.23	23.9	21.0		15
49		McLay	6.4	16.0	13	0.23	23.9	21.0		15
50		Chatsworth	6.4	20.0	13	0.20	21.4	19.4		15
51		Tehachapi	6.4	70.0	13	0.08	8.7	10.0		15
52		Van Norman	6.4	33.0	13	0.15	15.8	15.5		15
53		Tehachapi-2	6.4	73.0	13	0.07	8.4	9.7		15
54		Tehachapi-3	6.4	73.0	13	0.07	8.4	9.7		15
55		Carley Porter	6.4	65.0	13	0.08	9.3	10.5		15
56		Van Morrison North	6.4	23.0	13	0.19	19.8	18.3		15
57		Saugus	6.4	23.0	13	0.19	19.8	18.3		15

TABLE 5-3 Continued

No.	Earthquake	Tunnel	M	R	Depth	a	v	d	I_o	Duration
58		San Fran-ciscquito	6.4	24.5	13	0.18	19.1	17.8		15
59		Elizabeth	6.4	27.3	13	0.17	17.9	17.0		15
60		Antelope	6.4	37.5	13	0.13	14.4	14.5		15
61	Inyokern, 1946	Jawbone	6.3	26.0	15	0.16	16.8	15.7		
62		"	6.3	28.0	15	0.16	16.0	15.2		
63		"	6.3	31.0	15	0.14	15.0	14.4		
64		Freeman	6.3	22.0	15	0.18	18.5	16.9		
65	Arvin Tehachapi 1952	Saugus	7.7	90.0	20	0.14	23.0	31.0		
66		San Fran-cisquito	7.7	75.0	20	0.17	27.2	35.0		
67		Elizabeth	7.7	70.0	20	0.18	29.0	36.7		

TABLE 5-3 Continued

No.	Earthquake	Tunnel	M	R	Depth	a	v	d	Io	Duration
68		Antelope	7.7	48.0	20	0.25	39.7	46.3		
69		Jawbone	7.7	90.0	20	0.14	23.0	31.0		
70	Chalone, 1922	Jawbone	6.1	52.0	20	0.08	8.5	8.9		
71		Freeman	6.1	52.0	20	0.08	8.5	8.9		

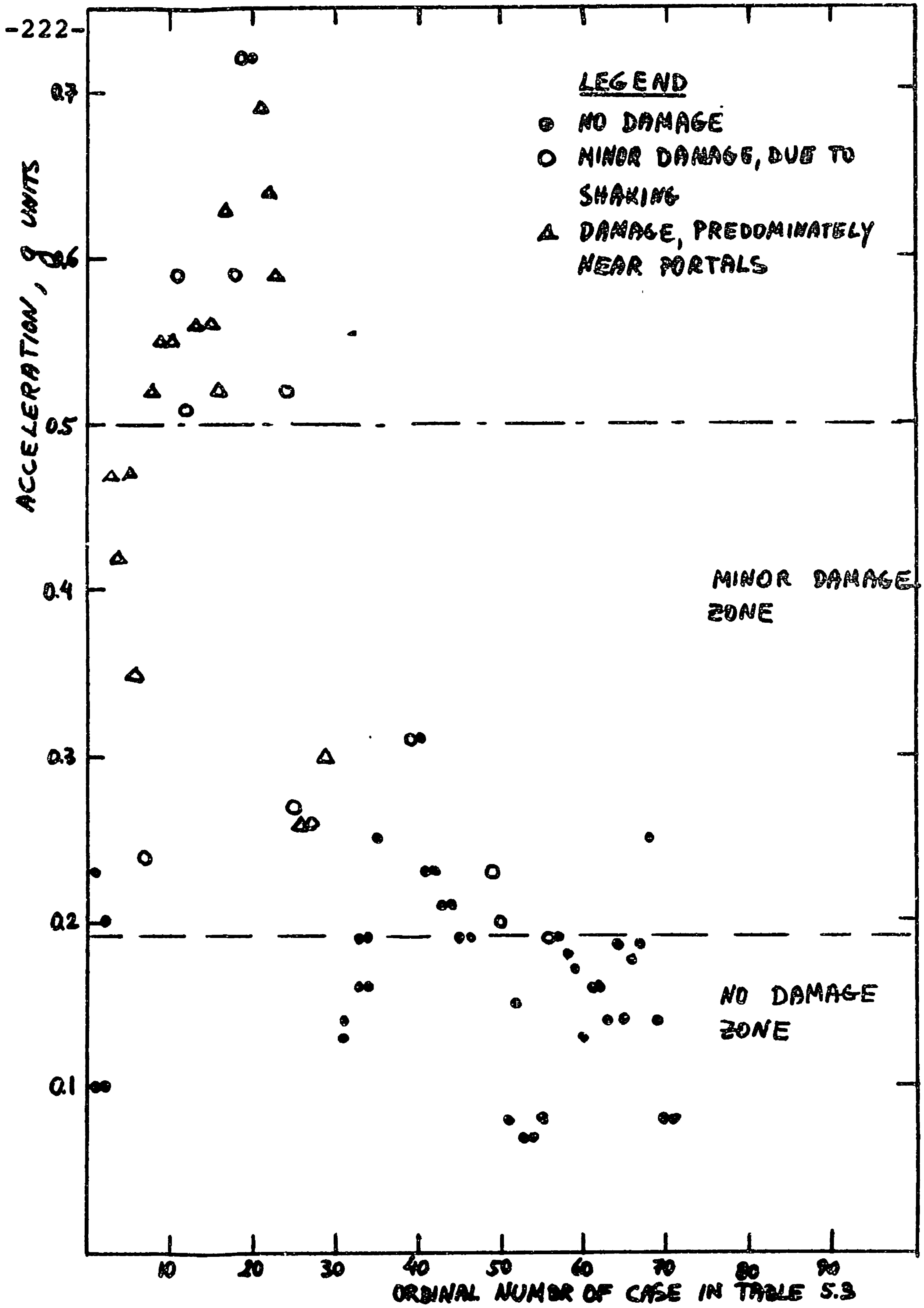

FIGURE 5.1 Correlation Between Peak Ground Acceleration and Damage, From Known Case Histories (Table 4-3)

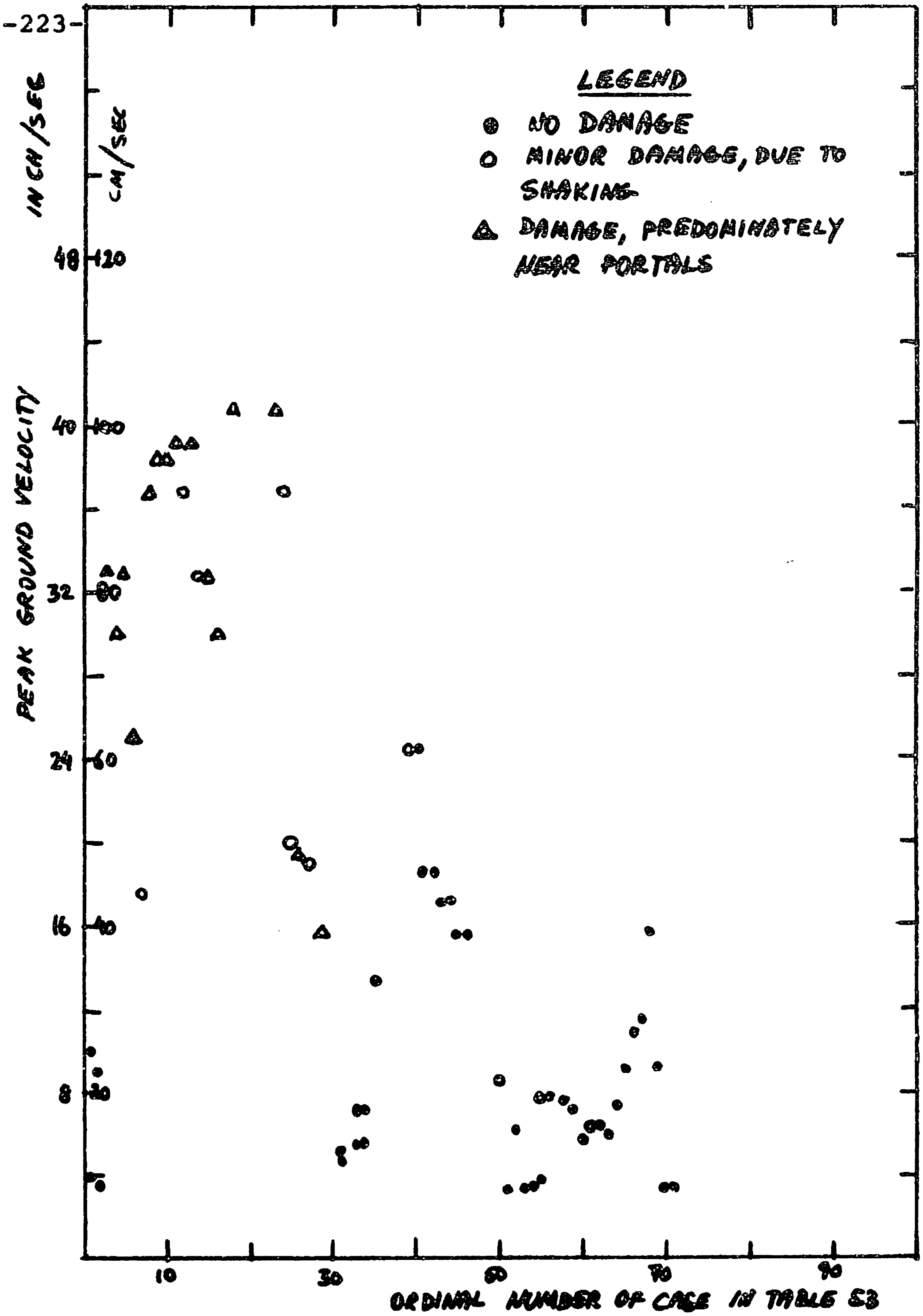

FIGURE 5.2 Correlation Between Peak Ground Velocity and Damage From Known Case Histories (Table 4-3)

At a first, quick glance, one may learn several interesting facts out of these two figures, even before using any other information: It is clear, from Figure 5.1, that at ground accelerations less than 0.19 g, there is absolutely not case of any damage for both lined and unlined tunnels. At accelerations between 0.19 g and 0.50 g, only a few cases of shaking damage occurred. More damage cases have been reported in this range, but most of the cases occurred at the portal, and were caused usually by ground failure and landslides at the slopes above the tunnel's entrance. The only shaking damage cases in the 0.19 g to 0.50 g range are:

(a) No. 7 - where the concrete lining was fractured slightly;

(b) No. 25 - where the collapses occurred at <u>shallow</u> zones;

(c) No. 27 - where many cracks and local bulges were caused by increased local earth pressure;

(d) No. 39 - where some loose rock fell from the unlined roof;

(e) No. 49 - where some long cracks occurred without any local buckling;

(f) No. 50 - where only "slight damage" has been reported; and

(g) No. 56 - where many cracks occurred at the concrete lining, but no structural damage.

From these seven cases, only one case of collapse was reported (No. 25), and this was in the case of a shallow tunnel. As was studied by Okamoto (1973), shallow tunnels are much more vulnerable to earthquake shaking damage than deep tunnels. In all the other six cases, the damage was slight, and no danger of collapse or any problems to the continuous use of the tunnels have been reported. It is possible, therefore, to conclude that 0.19 g is the threshold of minor damage. The range below 0.19 g may be called "Safe Zone", and the range of 0.19 g to 0.50 g may be called "Minor Damage Zone".

At accelerations above 0.50 g and up to 0.72 g, 18 cases of damage have been reported. Of them only six cases (33%) may be defined as caused by shaking within the tunnels. All other cases were usually caused by "ground failure" and most of them near entrances. There is no case of major damage, not to mention collapse, even at the high level of shaking of 0.50 g to 0.72 g.

It is of interest to compare the shaking levels just mentioned to the damage caused by them to aboveground structures. Figure 5.3 is a correlation between the damage levels as concluded from the case histories and the MM-Intensity levels summarized by Krinitzsky et. al. (1975). The "Safe Zone", with acceleration up to 0.19 g, is equivalent to MM VI-VIII; and the "Minor Damage Zone" is equivalent to MM VIII-IX. The upper zone of acceleration up to 0.72 g may be defined, by enlarging and modifying the boundaries of the "Minor Damage Zone", as no cases of real heavy damage are known to occur. The Modified Mercalli intensities mentioned above are (Housner, 1970) respectively:

VII : Damage negligible to buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly-built or badly-designed structures.

VIII: Damage slight in specially designed structures; considerable in ordinary substantial buildings with partial collapse; great in poorly-built structures.

IX : Damage considerable in specially designed structures; well-designed frame structures

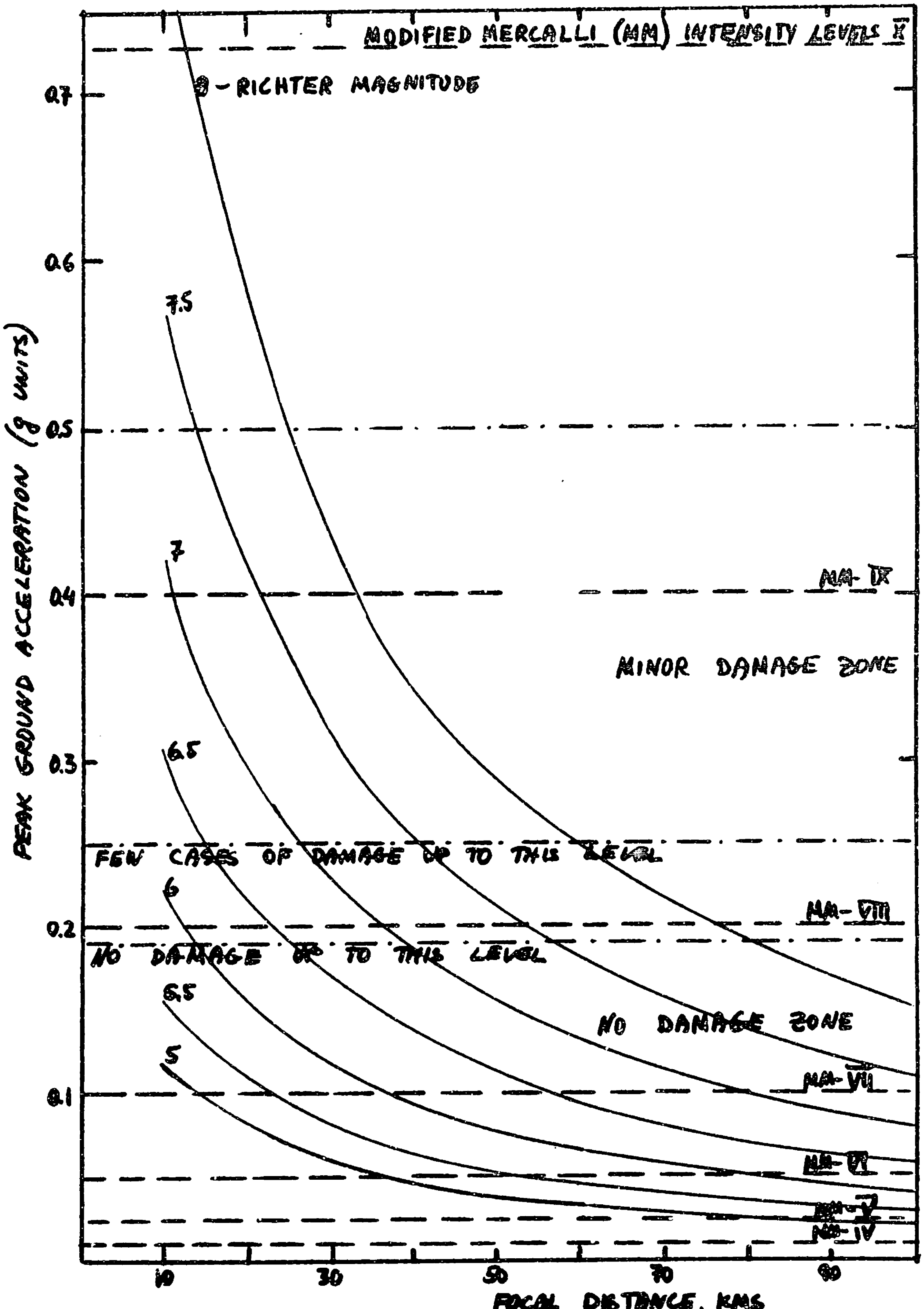

FIGURE 5.3 Correlation Between Damage, Peak Acceleration, Distance To Causative Fault of an Earthquake of Given Magnitude, MM-Intensities.

thrown out of plumb; great in substantial buildings with partial collapse; buildings shifted off foundations.

It is clear that at peak accelerations which are expected to cause heavy damage to aboveground structures (MM VIII-IX), there is only minor damage to tunnels. Comparatively, then, tunnels seem to be safer and more stable than aboveground structures at the same intensity level!

Another correlation demonstrated in Figure 5.3 is between the acceleration "oriented" damage level and the epicentral distance of an earthquake with known magnitude. In this case, the abscissa is the distance in kilometers. The "Safe Zone" is defined for $M = 6$ quakes at distances equal or greater than 15 km (9.4 miles); $M = 7$ at 27 km (16.9 miles); and $M = 8$ at more than 70 km (43.7 miles). The "Minor Damage Zone" ($a \leq 0.5$ g) may be expected for a quake of $M = 7$ at 7 km (4.4 miles), 7.5 at 14 km (8.8 miles), and 8 at 25 km (15.6 miles). The 0.72 g level (still slight damage) may be caused by quakes of $M = 8$ at 14 km (8.8 miles).

It is, of course, possible to assume that earthquakes of magnitude 7.5 at less than 5 km (3.13 miles) from the causative fault or $M = 8$ at epicentral distance less than 10 km (6.25 miles), may cause severe damage to tunnels, as the peak ground accelerations may be much higher; but there are not enough strong motion measurements at these short distances to justify a quantitative conclusion.

A correlation along the same pattern -- versus intensities and epicentral distances to quakes of known magnitude -- is shown in Figure 5.4. There are two other points concerning the use of particle velocity as a damage index which may be of interest: (a) Langefors (1963), and Chapter 2.7.4 of this thesis, suggested 30 and 60 cm/sec (12 and 24 in/sec) as "fall of stones" and "new cracking" respectively. Very few cases in this paper are for unlined tunnels; hence, any significant quantitative conclusion is premature. The only case in which the fall of stones from an unlined roof was reported was in the Alaska Earthquake (at Whittier) at velocities higher than the 30 cm/sec (12 in/sec) mentioned by Langefors. (b) Puchtov (1965) has suggested the use of a critical velocity (Section 2.6) which is related to the rock's strength. Velocities of 35 to 50 cm/sec (14 to 20 in/sec), which are estimated for the Alaska Earthquake, are supposed to cause tensile failure in most jointed rocks. As the eight tunnels at the Alaska Railroad remained undamaged (except for fall of small, loose

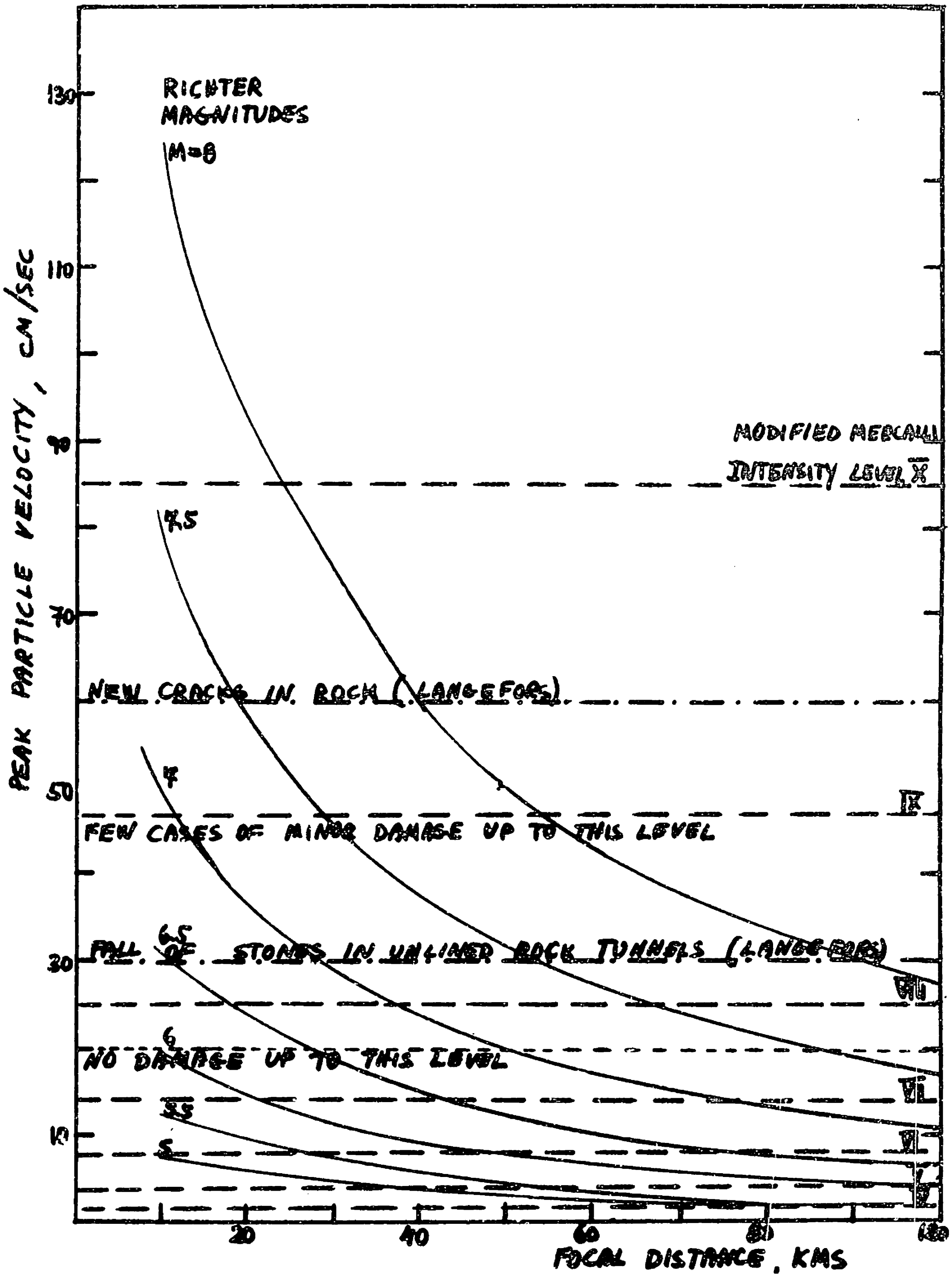

FIGURE 5.4 Correlation Between Damage, Peak Ground Velocity, Distance to Causative Fault of an Earthquake of Given Magnitude and MM-Intensity Levels

stones from the roof), the critical velocity approach seems to be unacceptable in these cases. The limited data at hand does not permit us to draw a final quantitative conclusion.

The correlation between partical velocity and MM-Intensities is based on Krinitzsky and Chang (1975). In this case, the "No Damage Zone" in tunnels falls between VII (damage negligible to buildings of good design) and VIII (damage slight in specially-designed structures). The "Minor Damage Zone" falls slightly above IX (damage considerable in specially-designed structures).

The correlation of particle velocity and acceleration to damage and/or intensity are similar. Hence, for earthquake engineering in which the acceleration is recorded, the use of acceleration seems to be slightly advantageous.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

6.1.1 Basic Conclusions From Case Studies

Based on the case histories summarized in Chapters 4 and 5 the following conclusions may be of practical value:

(a) No collapse of tunnels should be expected from shaking only. It was found that there was no damage in both lined and unlined tunnels at accelerations up to 0.19g or MM-VIII. In addition, very few cases of damage due to shaking were observed at accelerations up to 0.25 g or MM-VIII-IX. There were a few cases of local "minor damage," such as falling of loose stones, and cracking of brick or concrete linings for accelerations above 0.25 g. Most of the cases of similar damage appeared above 0.4 g or MM-IX. Some local "minor damage" may occur, as explained later, but there is no danger of total collapse.

(b) Tunnels are much safer than aboveground structures for given intensity of shaking. This is especially true if the aboveground structures are built on a soft layer which amplifies the ground motion. While only "minor damage", such as described in (a) earlier, was observed in MM-VIII and IX levels, the damage in aboveground structures at the same intensities is "considerable."

(c) More severe damage may be expected when the tunnel is crossed by a fault which displaces during an earthquake. The degree of damage is dependent on the fault displacement and on the conditions of both the lining and the rock.

(d) Tunnels in poor soil or rock, which suffer from stability problems during excavation, are more susceptible to damage during earthquakes.

(e) Tunnels deep in rock are safer than shallow tunnels.

(f) "Failure" of a tunnel, as defined later (6.1.6), was found associated only with fault movement, not shaking.

In the next sections, these and other conclusions will be discussed in detail.

6.1.2 Damage Near Portals

Chapters 4 and 5 show clearly that in many cases, the damage to tunnels was caused by slope stability problems near the portal. The portal failure results from aboveground slope instability, which can be solved by ordinary engineering measures. As learned from the case histories summarized in Chapter 5, damage at the portals may become significant at ground acceleration of 0.25 to 0.7 g, or Modified Mercalli intensities of VIII and above. Most of the cases (70% approximately) occurred above 0.4 g or MM-IX. In any case of potential slope fialure, the first several meters inside the tunnel from the portal must be designed to withstand the extra load from accumulation of slide debris.

6.1.3 Damage in Poor Ground Conditions

In the few cases where damage due to shaking was reported along the tunnel's interior, the soil or rock conditions were poor and caused difficulties to miners during excavation. Shaking damage can be eliminated by stabilizing the soil or rock around the tunnel along the critical zone and especially by improving the contact between the lining and the rock. If the lining is in contact with the rock around the perimeter (without small cavities which may allow local movements of small pieces of rock), then the danger of local damage may be minimized.

Improving the lining by using thicker and stiffer sections may result in excess seismic forces transmitted to the lining; hence improving the lining is only a secondary measure, while improving or stabilizing the ground itself is a better and more recommended approach.

6.1.4 Damage by Unsymmetric Load

Tunnels are more stable under symmetric load which improve the soil-lining interaction. Tunnels near slopes or tunnels which have variable depth of cover in a section perpendicular to their longitudinal axis are more vulnerable to damage during earthquakes. Backfilling with firm and stable material and rock-stabilizing measures may improve the safety and stability of these tunnels.

6.1.5 Resonant Behavior and Dynamic Loading

No resonant-type behavior of entire cavities should be expected when excited with frequencies between 1 and 100 Hz (which includes all significant motions due to earthquakes and construction blasting). No high frequency wave energy is expected to circulate around the inner surface of a cavity. Analytical results tend to suggest the existence of such phenomena, but this Rayleigh-type wave influence is important only for wavelengths equal or shorter than the radius of the tunnel. Such short-wavelength, high-frequency waves are highly unlikely for earthquake waves, except, possibly, very near the source.

Dynamic stress concentrations generated by "step-function waves" impinging upon lined and unlined tunnels by both dilatational and distortional waves are generally no more than 10 to 20 percent greater than the static values. For earthquake waves (which are not "step-functions"), it is expected that the stress concentration factors will be smaller.

6.1.6 Damage and Failure

It is most important to define the failure or damage according to the use and importance of the tunnel. While falling stones may derail trains (a failure) or may disrupt car traffic (an inconvenience), they can be washed out of a water tunnel (no failure).

"Failure" is the termination of safe use of the tunnel. Such may be a collapse of lining or rock which closes the tunnel or displacement which shears rails or roads, halting all traffic. Damage, on the other hand, may include any source of small, short-term disturbance to the operation of the tunnel. Such may be cracking of lining, falling of loose stones, et.

Most observed consequences of earthquake loading of tunnels were associated mainly in damage but not in failure. The small number of failure were associated with fault displacement or instability at the portals.

6.1.7 Summary

Thus, underground construction of facilities in rock appears to be seismically advantageous from several standpoints. In the first place, the structure is not affected by the amplification of body and surface waves due to soil layers. This advantage is not shared by structures constructed in deep excavations and buried. Second, when the cavity is located deeper than about one quarter wavelength from the surface, the structure is not affected by the doubling of displacement amplitude which occurs upon reflection of body waves at the earth's surface. Third, there is a possibility of additional reinforcement and structural stiffening by the "all around" connection of the liner to the cavity which is only possible underground. Overall, underground siting of utility tunnels in rock does indeed seem to have seismic advantages over surface siting.

6.2 RECOMMENDATIONS

Relatively little measured seismic data exists for tunnels and rock masses at depth. Therefore, it is highly recommended to install, by a cooperative action with the United States Geological Service, seismographs and strong-motion equipment in the following rock site configurations:

1. In a deep "homogeneous" rock, at different depths, to study the influence of depth alone for a given, unchanged rock material. This can be done either in existing mine shafts or in unused deep wells.

2. In several rock sites with different types of rock, to study the influence of rock type on the attenuation, frequency content, etc.

3. In rock tunnels near an active seismic zone like those of the BART system or the L. A. Water Supply systems, to better understand the resonance behavior of tunnels, if any, the influence of distance from the portals and the depth of cover.

4. In sites with similar seimisity, but which have different qualities of the same rock (RQD, etc.) to study the influence of joints and other discontinuities.

As the provision of resonance may be of interest for very large underground chamgers under relatively high frequency, short-wave loading it is suggested to investigate this problem separately. This focus was suggested by the fact that the ratio of wavelength to span or height of chamber is much smaller for large chambers than for tunnels.

It seems to the writer that the next step must be a concentrated effort to obtain much more in-situ measured data in known conditions which may then be compared with methods of analysis.