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Damage to Rock Tunnels from Earthquake Shaking

By Charles H. Dowding,[1] M. ASCE and Arnon Rozen[2]

Introduction

Observations of rock tunnel response to earthquake motions is compared with calculated peak ground motions for 71 cases to determine damage modes. Damage, ranging from cracking to closure, is recorded in 42 of the observations. These tunnels, located in California, Alaska, and Japan, served as railway and water links and were 10 ft–20 ft (3 m–6 m) in diameter. This comparison of peak motion with observed damage can serve as a framework both for development of analytical models and for estimation of expected losses resulting from earthquake shaking failure of tunnels in rock. This is the first such correlation besides a consulting report by Cooke (2).

The potential of damage to tunnels from earthquakes is a factor to be considered in the siting of any subsurface project whose failure would result in severance of life-line supply. This paper focuses upon the evaluation of rock tunnel damage caused by shaking, but treats damage from other causes. There are three reasons for the more restricted focus: (1) Damage from other sources, such as ground failure or displacement from fault movement, is location specific, and potential damage may be minimized through careful siting; (2) shaking can result from movement of a number of faults (i.e., is not location specific) and therefore potentially affects long lengths of tunnel; and (3) it is useful for project planning to compare damage to tunnels with that to above-ground structures at the same intensity of shaking.

Damage Mechanisms

Damage in tunnels resulting from earthquakes is generally manifested in one

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or a combination of the following forms: (1) Damage by earthquake induced ground failure, such as liquefaction or landslides at tunnel portals; (2) damage from fault displacement; and (3) damage from ground "shaking" or ground vibration. The potential of tunnel damage from ground failure may be evaluated through established geotechnical analyses, geological exploration, and testing. Prudent siting can avoid this problem.

Tunnel displacement by fault movement usually results in serious damage. Similar to ground failure, siting to avoid intersection with active faults capable of movement can minimize this problem for new tunnels. It was found (15) that most of the tunnel damage from fault movement was caused by unavoidable location of tunnels across active faults.

Damage from ground shaking differs from the preceding two sources of potential damage. The first two are related to gross geological features that can be located before design and taken into account. In addition, they affect only limited lengths

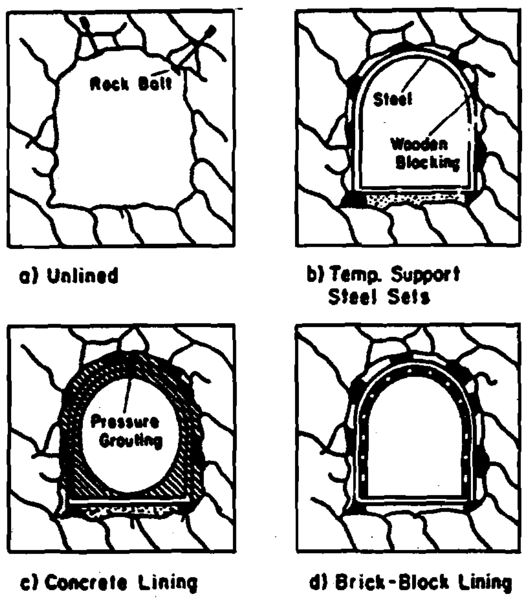

FIG. 1.—Rock Tunnels—Construction Details

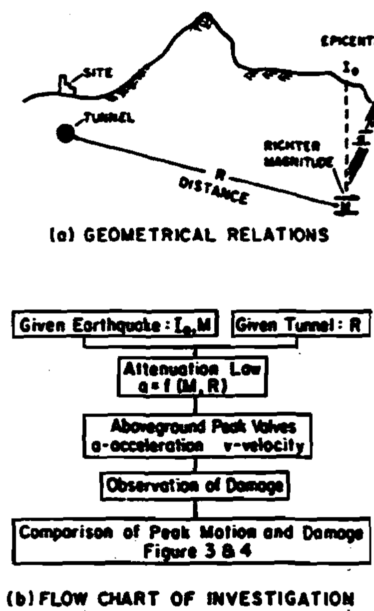

FIG. 2.—Concept of Comparison

of tunnel. On the other hand, damage from shaking can result from earthquakes caused by any number of faults at various distances and affects long lengths of tunnels.

Rock Tunnels: Distinguishing Characteristics

Consideration of the construction methodology for rock tunnels is critical to an understanding of their response to earthquake loading. Fig. 1 shows four types of lining configurations for rock tunnels: (1) No lining or few rock bolts; (2) temporary steel set support with wooden blocking; (3) final concrete lining which engulfs the temporary support; and (4) final masonry lining. Final masonry lining is not presently employed but was associated with early tunnels damaged by earthquake shaking. The final concrete lining with intermittent temporary support (steel sets or rock bolts) is typical of present construction practices.

In the final stages of construction, the concrete liner and the rock mass may be further wedded by pressure grouting. Through the concrete lining process, the tunnel and the surrounding rock mass become one entity and must move together. Virtually no free response of the tunnel liner with respect to the surrounding rock is possible.

The rock mass differs substantially from a soil mass on a scale the order of the size of a tunnel diameter. As can be seen in Fig. 1 the rock mass is characterized by intact blocks bounded by joints or discontinuites. Therefore the rock mass must be analyzed or conceived as a discontinuum, whereas soil masses (on a tunnel diameter scale) can be thought of as continuous where average properties govern. The weaknesses in rock or "what is not rock"—joints, joint fill material and shear zones govern rock mass response. Only when rock blocks are prevented from moving into the opening, in most cases by the liner and adjacent blocks, will failure take place through the intact blocks.

Choice of Shaking Measure

What are the measures of vibration that can be related in a meaningful and readily applicable manner to disturbance or structural damage? In studies of blasting vibrations, particle velocity is commonly employed as a damage index. In earthquake engineering, however, the peak ground acceleration is, by far, the most widely accepted index of the ground shaking intensity and damage.

The use of acceleration as an index of damage does not mean that maximum acceleration is the cause of damage, but simply that the use of acceleration as an index will result in a workable method for determining the imminence of gross levels of damage. Detailed study indicates structural damage is a function of number of cycles or duration of shaking, ratio of structural frequency to input frequency, and structural damping as well as peak acceleration. Therefore tunnel damage is correlated with peak particle velocity as well as peak acceleration.

Correlation of Damage and Peak Ground Motions

Fig. 2 shows how the peak acceleration and peak particle velocity correlated with damage were retrospectively calculated at the surface above a damaged tunnel. At a specific site, such a calculation can be based upon the earthquake's magnitude, m, and the distance between the source and site, R, through "attenuation laws" developed from regression analyses of accelerations measured at the surface. The writers chose McGuire's (9) attenuation relationships, since he derives attenuation relationships for both acceleration and particle velocity.

The study involved 71 tunnels subjected to earthquake shaking and distortion. These tunnels served as railway and water links. Two were as small as 6 ft (2 m) in diameter. However, the majority were 10 ft–20 ft (3 m–6 m) in diameter. Of these 71 tunnels, detailed geologic information was avilable for only 23. Twelve tunnels were in relatively competent rock and 11 in sheared, weathered, or broken rock, and three tunnels were located in soil-like materials. These geological details are contained in the original work (15). There was no available geological data for the other 45; however, from project descriptions and tunnel locations the tunnels were located in nonsoil media.

The tunnels were built between the late 1800's and the present, and thus represent a wide variety of construction methods and lining types. For the 27 tunnels where the lining was described, two were unlined, two were timbered, seven were lined with brick or masonry, and 13 were concrete lined. The importance of lining will be considered in the section dealing with damage observations.

The 71 cases involve 13 different earthquakes whose Richter magnitude varied from 5.8 to 8.3. Focal depths varied between 13 km and 40 km (8 miles–25 miles), however, depths of 15 km–20 km (9 miles–12.5 miles) predominated. Six of the earthquakes occurred in California, six in Japan, and one in Alaska.

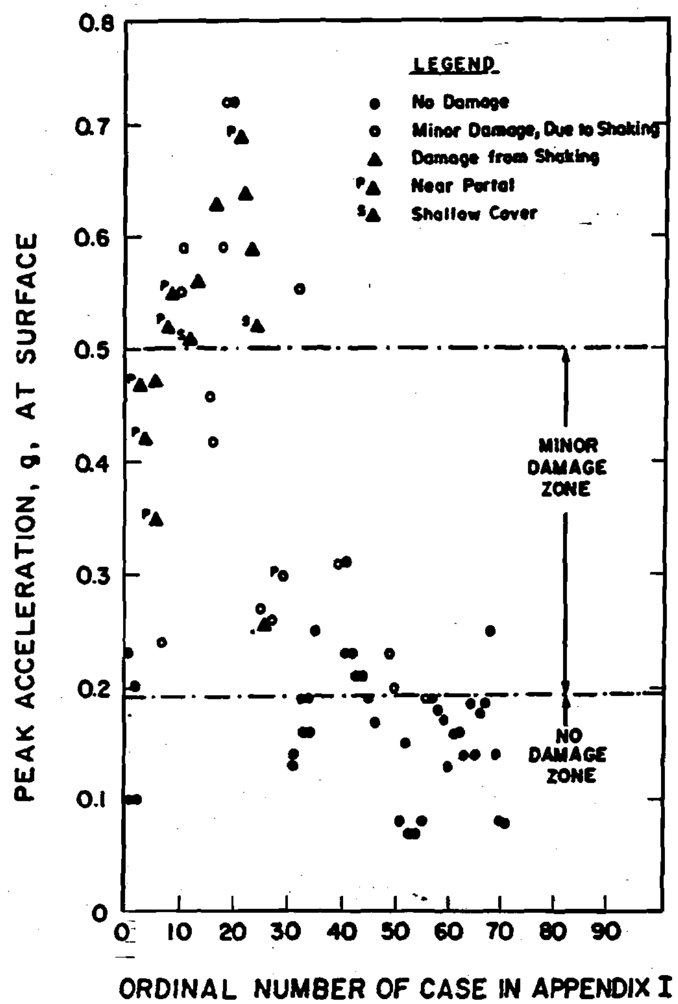

FIG. 3.—Calculated Peak Surface Accelerations and Associated Damage Observations

The reports of damage were separated into three main groups; shaking, fault movement (active fault intersection), and ground or portal failure. The last grouping contains cases predominantly related to landslides and the special boundary conditions at the portals. It was decided not to include portal damage in the final determination of damage thresholds, because of the intimate relationship with landsliding. However, damage associated with the portals was plotted along with the other data. Since the investigation focused upon shaking damages, those related to fault intersection were also not considered in the final comparison.

Figs. 3 and 4 summarize the basic data from the case histories. The abscissa is the ordinal number of the case histories described in Appendix I. In Appendix

I, the "no-damage" cases were not detailed because of space limitations; thus, their numbers are missing. The ordinate is the calculated peak surface acceleration (Fig. 3) or particle velocity (Fig. 4), as calculated with McGuire's (10) attenuation law. Three levels of response were distinguished, as shown on the figure, without regard to geologic media or lining *No damage* implies post-shaking inspection revealed no apparent new cracking or falling of stones. *Minor damage due to shaking* includes fall of stones and formation of new cracks. *Damage* includes major rock falls, severe cracking, and closure. These *damage* cases occurred predominantly at the portals.

The three levels of response are stratified with respect to the calculated peak surface motions. There are no reports of even falling stones in unlined tunnels

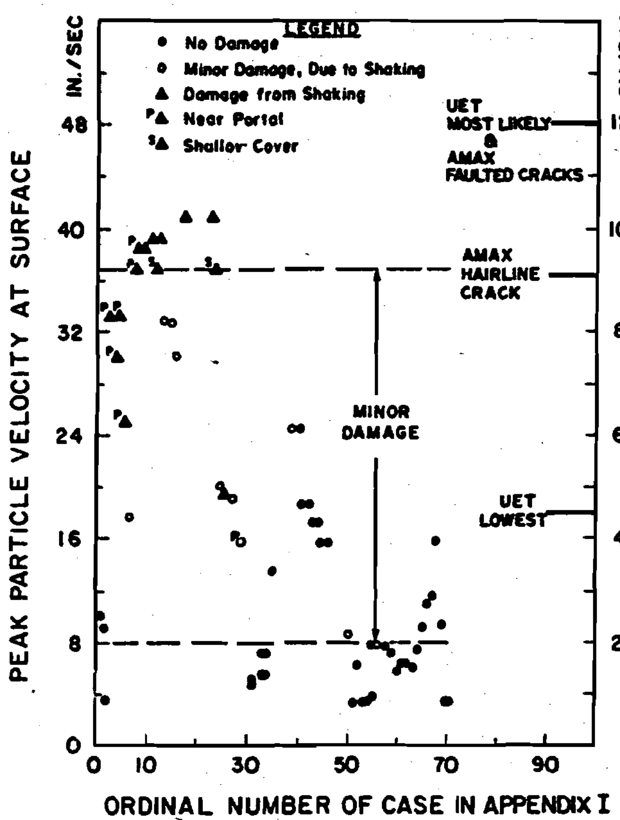

FIG. 4.—Calculated Peak Particle Velocities and Associated Damage Observations: Earthquake and Explosive Shaking

or cracking in lined tunnels up to 0.19 *g* and 8 in./sec (20 cm/s). Up to 0.25 *g* and 16 in./sec (40 cm/s) there are only a few incidences of minor cracking in concrete lined tunnels. Between 0.25 *g* and 0.52 *g* or 32 in./ sec (80 cm/s) there was only one partial collapse (No. 26). It was associated with landsliding and was lined with masonry.

Many of these post-event observations of damage suffer from lack of pre-event documentation. Cracking of tunnel liners from nonearthquake circumstances such as shrinkage is common. Therefore, observed cracks may or may not have been caused by the earthquake itself. With no pre-event documentation earthquake-related cracking must be separated from the other by circumstantial evidence such as freshness. The cracking and damage included in this study was that reported by the field observers. Observations may include some pre-event cracking and are, therefore, likely to be conservative.

All of the partial collapses, not associated with fault displacement or portal instability, occurred during the 1923 Kwanto earthquake in Japan. Many of these tunnels had final linings of masonry and concrete block. No doubt wooden blocking transferred load from the rock to a steel or wooden construction lining. Quite possibly these cases of heavy damage, including case 26, resulted from local ground failure after blocking shifted or response of the final linings that may not have been grouted into place, or both.

Fig. 5 summarizes two relationships involving tunnel damage. First, the observed damage is compared to Modified-Mercalli (MM) Intensity levels for above-ground structures. Secondly the damage level is correlated to Richter Magnitude and distance between epicenter and tunnel location. The "No Damage Zone" with acceleration up to 0.19 g, is equivalent to Modified-Mercalli (MM)

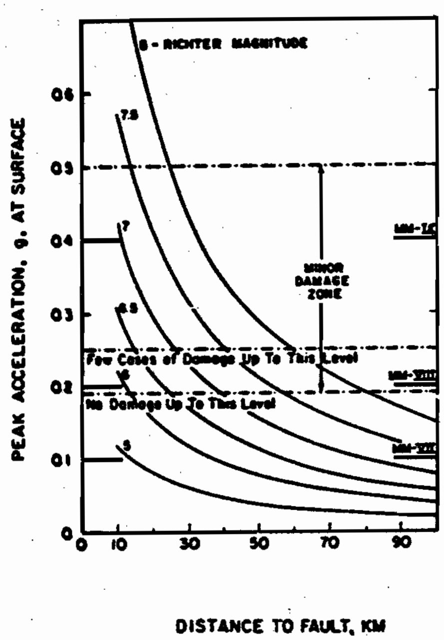

FIG. 5.—Comparison of Modified Mercalli Intensity of Surface Motions and Observed Damage

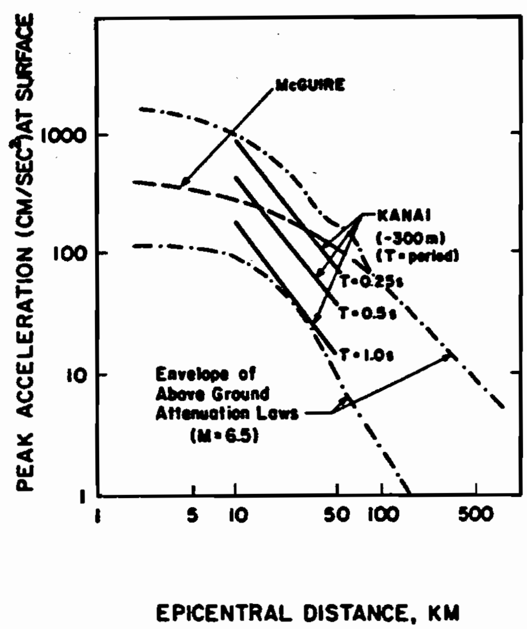

FIG. 6.—Comparison of Surface Attenuation Laws and Kanai's Subsurface Relationships

VI–VIII; the "Minor Damage Zone" with acceleration up to 0.5 g is equivalent to MM VIII–IX. These findings agree with Cook's report.

It is clear that at peak surface accelerations which are expected to cause heavy damage to above-ground structures (MM VIII–IX), there is only *minor damage* to tunnels. Comparatively, then, tunnels are safer and more stable than above-ground structures at the same intensity level as determined from surface motions. Furthermore, the difference between failure and formal damage level is a function of the use of the tunnel. While a few 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). Thus, "failure" is the termination of safe use of the tunnel. Such termination may be a collapse of lining or rock that closes the tunnel, displacement that shears roads, and halts all traffic or falling stones that can derail trains.

CONSIDERATIONS BEYOND PEAK SURFACE MOTIONS

A number of factors that could affect response and thus damage, other than peak surface motions, were considered, but not included in the analysis. These factors either required details of the earthquake time-history at depth which are unknown, or resulted in modifications that fell within the range of predicted values of peak motions. A brief description of each additional consideration and reason for its elimination follow.

Most attenuation relationships have been derived from measurements made at surface stations located on a wide range of ground conditions, both soil and rock, without differentiation between the different geological conditions. Because of site amplification effects, this lack of discrimination in correlations is a serious disadvantage when dealing with tunnels located at depth in rock.

Specific site studies point to deamplification of peak amplitude with depth, greater for soil and smaller for rock (8,12,14). However, no quantitative deamplification was employed in this study because the spread or variation in attenuation laws at a constant scaled epicentral distance is greater than the observed deamplification effect. Fig. 6 compares the spread in attenuation relationships with those of Kanai (8) and McGuire (10). Kanai's relationship was derived for motions 980 ft (300 m) below the surface. For most of the focal distances in this study (10 km–40 km) Kanai's and McGuire's relationships are similar.

Ground motion may be amplified upon intersection with a tunnel if, and only if, wavelengths are the same as the tunnel's diameter, or at most, up to four times the diameter. Since measured peak accelerations are recorded at wavelengths much longer than normal tunnel diameters, the interaction amplification was not quantitatively employed in this study. In future work, high frequency motions (not normally measured by strong motion equipment) should receive more attention as they may contribute to the possibility of relative displacement between blocks, along planes of weakness. This high frequency effect may explain the local spalling of rock or concrete which was reported in several cases after earthquakes.

As the higher frequency components attenuate more rapidly than the lower frequency components, the destructive frequencies (from the tunnel point of view) may be expected mainly at small distances from the causative fault. The present knowledge of the ground motions near the causative fault is limited, as few, if any, measurements have been made at small distances from faults (16).

Duration of strong-motion shaking during an earthquake is of utmost importance as it may cause fatigue failure and lead to large deformations. This mode of failure is dependent on the total number of cycles induced by the ground shaking. Haimson and Kim (5) found that long duration cyclic loading may cause fatigue failure in intact rock, and Brown and Hudson (1) proved it experimentally for jointed media. The large number of cycles required to cause fatigue failure usually is too large to be of importance in a single earthquake. The cumulative cyclic effect, if any, was not incorporated in this study due to a lack of available field data.

COMPARISON WITH FIELD EXPERIMENTS

It is valuable to compare the surface particle velocity and damage correlation in Fig. 4 with damage observed in shallow, unlined tunnels near the large Underground Explosion Tests (UET) conducted for the U.S. Army Corps of Engineers (17). The *unlined* tunnels located in sandstone were 6 ft, 15 ft, and 30 ft (2 m, 5 m, and 10 m) in diameter. The close explosions were single delays of 320 lb–320,000 lb (145 kg–145,450 g) of TNT located above and slightly off axis from the tunnels.

Hendron (6) has analyzed the results of the UET tests by comparing calculated particle velocities with the observed damage zones shown in Fig. 7. The particle velocities for each zone were calculated from locally derived attenuation relationships. The analysis showed that occasional rock drops (intermittent failure) in an unlined and unbolted tunnel were associated with calculated particle velocities that may have been as low as 18 in./sec (46 cm/s) for one of the 14 test blasts. The average particle velocity associated with this damage zone was 48 in./sec (120 cm/s).

Another comparison was obtained from an experiment conducted at the Climax, Colo. mine of AMAX (9) to determine cracking susceptibility of shotcrete liners. A 6-ft × 8-ft (1.8-m × 2.4-m) tunnel—in heavily jointed biotite schist bolted

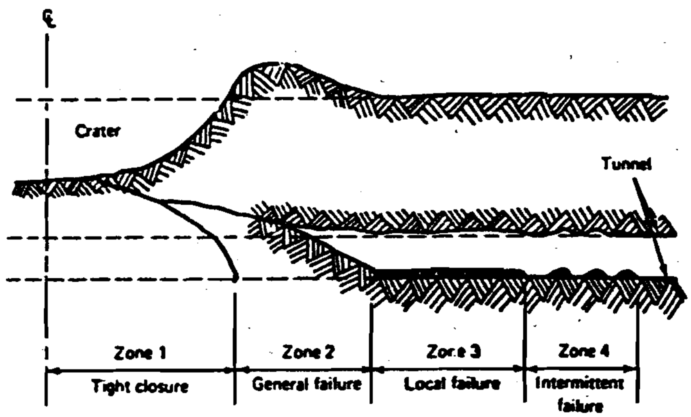

FIG. 7.—Zones of Damage Resulting from Underground Explosion Tests (6)

and lined with 2 in.–11 in. (5 cm–28 cm) of shotcrete—was subjected to vibrations from detonation of 400 lb (181 kg) of ammonium nitrate and fuel oil at distances of 40 ft and 30 ft (12 m and 9 m). Hendron's (6) attenuation relationships for close-in blasting indicate development of hair-line cracks after the 40-ft (12-m) blast were associated with peak particle velocities of approx 36 in./sec (91 cm/s). Faulting of the cracks in the shotcrete liner were associated with peak particle velocities of approximately 48 in./sec (120 cm/s).

The UET and AMAX results are compared with the results of the earthquake damage observations in Fig. 4. The damage thresholds determined from the case histories are lower than those of the experiments. Because of the frequency differences between experiment and earthquake, as examined herein, the case-study thresholds may be even more conservative than indicated by the particle velocity comparison in Fig. 4.

The experiments involved close-in blasting where peak particle velocity and acceleration occur at frequencies of 20 Hz–200 Hz (3), whereas peak earthquake motions occur at between 0.4 Hz and 10 Hz. In a given rock mass the higher frequency blast motions have short wavelengths and can differentially accelerate rock blocks on the order of the size of the tunnel. On the other hand, the lower frequency earthquake motions have wavelengths 20–50 times longer than the blast pulses and are much less likely to cause differential acceleration (and damage) across a tunnel. This frequency scaling difference is somewhat compensated by the greater number of pulses in an earthquake but is most likely only important in poorly lined tunnels as considered in connection with the Kwanto earthquake.

Perhaps the most important distinction illustrated by the experiments is the difference in damage mode for fully grouted-in-place tunnel linings as opposed to unlined and lined but nongrouted tunnels.

Comparison of the AMAX and UET tests indicate by analogy that only cracking would occur in fully grouted and lined tunnels at velocities associated with occasional rock drops in unlined tunnels. Thus lined and grouted tunnels are safer than unlined tunnels.

Observed Modes of Tunnel Damage

The following analysis is based upon the case histories summarized in Appendix I and the literature review. These case histories and literature reviews are detailed in the original study (15).

Damage Near Portals.—The table in Appendix I shows that in many cases, the damage to tunnels was caused by slope instability near the portal. An analysis of dynamic slope stability can be found elsewhere (7,13). Damage at the portals may become significant at a ground acceleration of 0.25. Most of the portal damage (approx 70%) occurred at accelerations above 0.4 *g*. In any case of potential slope failure, the tunnel lining near the portal must be designed to withstand the extra load from accumulation of slide debris.

Damage in Poor Ground Condition.—In the few cases where damage due to shaking was reported along the tunnel's interior, the soil or rock conditions were poor and created excavation difficulties during construction. Thus, 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 a lining is in contact with the rock around the perimeter (without small cavities that may allow local movements of small blocks of rock), then the danger of local damage may be minimized.

Improving the lining by placing thicker and stiffer sections without stabilizing surrounding poor ground may result in excess seismic forces transmitted to the lining; thus improving the lining must be accompanied by a stabilization of the ground itself (14).

Damage Associated with Shallow Cover and Unsymmetric Load.—Deep tunnels seem to be safer and less vulnerable to earthquake shaking than are shallow tunnels. Tunnels 12 and 24 had only 5 ft–20 ft (1.5 m–6 m) and 65 ft (20 m) of cover. Tunnels are more stable under a symmetric load which improves the rock-lining interaction. Backfilling with noncyclically-mobile material and rock-stabilizing measures may improve the safety and stability of shallow tunnels.

Resonant Behavior and Dynamic Loading.—No resonating of entire cavities that behave elastically should be expected when excited with frequencies between 1 Hz and 100 Hz, which includes all significant motions due to earthquake and construction blasting (4). 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 not associated with peak motions as measured today during earthquakes.

Dynamic Stress Concentration of the Ground Motions.—Concentration of dynamic stresses caused by waves impinging upon lined and unlined tunnels are generally no more than 10%–20% greater than the static values (11). For earthquake waves (which are not "step-functions"), it is expected that the stress concentration factors will be smaller.

Conclusions

Based on the case histories, the following conclusions may be of practical value:

1. Collapse of tunnels from shaking occurs only under extreme conditions. It was found that there was no damage in both lined and unlined tunnels at surface accelerations up to 0.19 *g*. In addition, very few cases of minor damage due to shaking were observed at surface accelerations up to 0.25 *g*. There were a few cases of minor damage, such as falling of loose stones, and cracking of brick or concrete linings for surface accelerations above 0.25 *g* and below 0.4 *g*. Most of the cases of similar damage appeared above 0.4 *g*. Up to surface acceleration levels of 0.5 *g*, no collapse (damage) was observed due to shaking alone.
2. Tunnels are much safer than aboveground structures for given intensity of shaking. While only minor damage to tunnels was observed in MM-VIII to IX levels, the damage in above-ground structures at the same intensities is considerable. Furthermore, it should be noted that the effect of the damage is a function of the use of the tunnel and building.
3. More severe but localized damage may be expected when the tunnel is crossed by a fault that 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.
4. Tunnels in poor soil or rock, which suffer from stability problems during excavation, are more susceptible to damage during earthquakes, especially where wooden lagging is not grouted after construction of the final liner.
5. Lined and fully grouted tunnels will only crack when subjected to peak ground motions associated with rock drops in unlined tunnels.
6. Tunnels deep in rock are safer than shallow tunnels.
7. Total collapse of a tunnel was found associated only with movement of an intersecting fault.

Acknowledgments

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TABLE 1.—Summary of Known Damage in Rock Tunnels

Number (1)	Earthquake (2)	Tunnel (3)	Damage due to shaking (4)	Damage due to fault movement (5)	Damage due to ground failure and other reasons (6)
1	Central California (San Francisco-1906) Magnitude 8.3	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 ft (13.7 m) under the fault.	
2	San Francisco 1906	Wright-2	Broken timber, roof caved in.		
3	Tokyo, 1923 (Kwanto) Magnitude 8.16	Terao			Cracked brick portal.
4		Hichigama			Landslide at entrance.
5		Taura			Landslide at entrance.
6		Numama			Cracked brick portal.
7		Nogogiri-Yama	Concrete walls fractured slightly. Some spall-		

TABLE 1.—*Continued*

(1)	(2)	(3)	(4)	(5)	(6)
			ing 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.
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-Jama	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.		
19		Hakone-1	Interior cracked.		
21		Hakone-3	Cracks in in-		Ceiling col-

TABLE 1.—*Continued*

(1)	(2)	(3)	(4)	(5)	(6)
			terior.		lapsed 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-in. (250-mm) displacement.		Landslide.
27		Mineoka-Yama	Cracks in bulges in masonry from local earth pressure.		
28	Idu Peninsula 1930 Magnitude 7.0	Tanna	Few cracks in walls.	7-ft 10-in. (2.39-m) horizontal displacement. Two-foot (0.6-m) vertical displacement just across the Tanna fault.	
29	Fukui, 1948 Magnitude 7.2	Kumasaka		Brick arches of portal partially fractured.	
30	Off Tokachi		Minor cracks		

TABLE 1.—*Continued*

(1)	(2)	(3)	(4)	(5)	(6)
	1952 Magnitude 8.0		in both brick and concrete linings.		
31	Kern County 1952 Magnitude 7.6	SPRR 3		Collapse under White Wolf Fault. Daylighted.	
32		SPRR 4		Collapse under fault. Daylighted.	
33		SPRR 5		Collapse under fault.	
34		SPRR 6		Fractured, daylighted, near fault.	
36	Kita Mino Magnitude 7.2	Aqueduct	Cracking.		
37	Niigata 1964 Magnitude 7.5	Nezugaseki	Spalling of concrete at crown.		Cracking at portal.
38		Terasaka	Spalling of concrete at crown, crushing of invert at bottom of sidewalls.		
39	Great Alaska 1964 Magnitude 8.4	Whittier 1	Some overhead ravelling of loose rock that falls on the track.		
47	San Fernando 1971 Magnitude 6.4	Balboa		Severe spalling, breaking of concrete lining, deformations where tunnel passed under canyon at	

TABLE 1.—*Continued*

(1)	(2)	(3)	(4)	(5)	(6)
				shallow cover, only 120 ft (36 m) south of Santa Suzana Fault. No breaking of reinforcing bar at RC Blocks.	
48		San Fernando		Displacement and damage near Sylmar Fault.	A vertical displacement of 7.5 ft (2.29 m) along 5.6 miles (9.02 km), caused flexural cracks.
49		McClay	Wide long cracks. No local buckling.		
50		Chatsworth	Slight damage.		
56		Van Norman North	Hundreds of new fractures in concrete lining. No structural damage. Fractures primarily circumferential, also longitudinal and diagonal.		

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Appendix I.—Summary of Case Histories

The case histories are summarized in Table 1 and Figs. 3 and 4. Rozen has tabulated these data in much greater detail (15).

Appendix II.—References

1. Brown, E. T., and Hudson, J. A., "Fatigue Failure Characteristics of Some Models of Jointed Rock," *Earthquake Engineering and Structural Dynamics*, Vol. 2, No. 4, Apr.-June, 1974, pp. 379-386.
2. Cooke, J. B., "Earthquake Risk—Savannah Bedrock Storage Project," Consulting Report to the Savannah River Project, U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, Miss., Oct., 1971.
3. Dowding, C. H., "Response of Buildings to Ground Vibrations Resulting from Construction Blasting," thesis presented to the University of Illinois, at Urbana, Ill., in 1971, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.
4. Glass, C. E., "Seismic Considerations in Siting Large Underground Openings in Rock," thesis presented to the University of California, at Berkeley, Calif., in 1973, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.
5. Haimson, B. C., and Kim, C. M., "Mechanical Behavior of Rock Under Cyclic Loading," Stability of Rock Slopes, *Proceedings of the 13th Symposium of Rock Mechanics*, E. J. Cording, ed., ASCE, 1972, pp. 845-863.
6. Hendron, A. J., "Engineering of Rock Blasting on Civil Projects," *Structural and Geotechnical Mechanics*, W. J. Hall, ed., 1977.
7. Hendron, A. J., Cording, E. J., and Aijer, A. K., "Analytical and Graphical Methods for the Analysis of Slopes in Rock Masses," *NCG-Technical Report No. 36*, 1971.
8. Kanai, K., "An Emperical Formula for the Spectrum of Strong Earthquake Motion," *Proceedings of the Second World Conference on Earthquake Engineering*, Vol. III, Japan, 1960.
9. Kendorski, F. S., Jude, C. V., and Duncan, W. M., "Effect of Blasting on Shotcrete Drift Linings," Report Submitted to Superintendent of Climax Mine, AMAX, 1973.
10. McGuire, R. K., "Seismic Structural Response Risk Analysis, Incorporating Peak Response Regression on Earthquake Magnitude and Distance," thesis presented to the Massachusetts Institute of Technology, at Cambridge, Mass. in 1974, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.
11. Mow, C. C., "Dynamic Response of Lined and Unlined Underground Openings," *Mem. RM-3962-PR*, RAND Corp., 1964.
12. Nasu, N., "Comparative Studies of Earthquake Motion Above Ground and in a Tunnel," Part I., *Bulletin, Earthquake Research Institute (Tokyo)*, Vol. 9, 1931, pp. 454-472.
13. Newmark, N.M., "Effects of Earthquakes on Dams and Embankments," Fifth Rankine Lecture, *Geotechnique*, London, England, 1965, pp. 139-160.
14. Okamoto. S., *Introduction to Earthquake Engineering*, University of Tokyo Press, Tokyo, Japan 1973.
15. Rozen, A., "Response of Rock Tunnels to Earthquake Shaking," thesis presented to the Massachusetts Institute of Technology, at Cambridge, Mass., in 1976, in partial fulfillment of the requirements for the degree of Master of Science.
16. Seed, H. B., et al., "Relationships Between Maximum Acceleration, Maximum Velocity, Distance from Source and Local Site Conditions for Moderately Strong

Earthquakes," *Report EERC-75-7*, University of California, Berkeley, Calif., July, 1975.
17. "Underground Explosion Test Program, Final Report," Engineering Research Associates, Vol. II: Rock, 1953.