Dynamic Rock Testing and Performance Study of Underground Structures Subjected to Blasting and Seismicity

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ABSTRACT

The study of rock dynamics is important because many rock mechanics and rock engineering problems involve dynamic loading ranging from earthquakes to blast explosions. The subject deals with the distribution and propagation of loads, dynamic responses, and processes of rocks and rate-dependent properties, coupled with the physical environment. Rock dynamics has a wide range of applications in civil, mining, geological and environmental engineering. However, due to the additional "4th" dimension of time, rock dynamics remains, in the discipline of rock mechanics, a relatively more challenging topic to understand and to apply, where documented research and knowledge are limited. Much new researches are needed and are indeed on-going.

This paper covers the dynamic rock testing and study on the performance of underground structures subjected to blasting and earthquake vibrations.

Keywords: Dynamic rock testing; Seismic loading; Blast loading; Split Hopkinson pressure bar; Underground structures

1. INTRODUCTION

Rock dynamics, as a branch of rock mechanics, deals with the responses of rocks (materials and masses) under dynamic stress fields. Differing from static mechanics, dynamic stresses are in the forms of stress waves propagating with time, and therefore the response of rocks is influenced by and interacting with the stress motion.

Dynamic loads include explosion, impact, and seismicity, that are generally in the form of stress waves. Since rock masses generally contain fractures in different scales, the interaction between stress waves and rock materials and fractures affects not only stress wave attenuation, but also rock mass instability. The wave transmission and transformation across rock fractures and along the fracture and failure of rock materials are traditional issues in the rock dynamic study.

Dynamically induced rock instability is commonly associated with frictional slip on rock fractures. Dynamical triggering of fracture slip may take place close to or far from the main shock. The fracture damage depends on the incident wave energy and the stored strain energy at the fracture plane. If energy release from the damage is sufficiently large, it can cause seismic events, and further induce aftershocks in close proximity. Laboratory experiments offer a direct observation on energy release pattern (Wu, 2013).

Singh and Goel (2002) have developed computer programs for the seismic stability analysis of circular slides (SARC), planar slide (SASP), simple wedge failure (SASW), complex wedge failure (WEDGE) and talus or debris slide (SAST). The programs calculate the dynamic settlement of rock slopes due to single earthquake. The permissible dynamic settlement of rock slopes is recommended to be 25cm. Thus, factor of safety slightly less than 1.0 may be allowed in dynamic conditions.

Dynamic loads are usually associated with high amplitude and short duration. A proper understanding of the effect of loading rate on the mechanical properties of rocks is important in the analysis of rock behavior or the design of rock caverns subjected to dynamic loads. For example, in the event of an explosion in a cavern or under an external attack, shock waves are generated and propagate in the rock mass. The rock and rock structures at distances are subjected to shock loads at different loading rates. The amount of damage and instability that occurs to the rock and rock structures is highly dependent, thus, on the loading rate and is primarily governed by the dynamic strength properties of the rock. Rock dynamic properties are particularly important for numerical simulation of rock structures.

Rock dynamics has a wide range of applications in civil, mining, geological and environmental engineering. However, due to the additional "4th" dimension of time, rock dynamics remains, in the discipline of rock mechanics, a relatively more challenging topic to understand and to apply, where documented research and knowledge are limited. Much new researches are needed and are indeed on-going.

A text book by Yingxin and Zhao (2011) on *Advances in Rock Dynamics and Applications* provides a summary of the current knowledge of rock dynamics. The book covers fundamental theories of fracture dynamics and wave propagation, rock dynamic properties and testing methods, numerical modelling of rock dynamic failure, engineering applications in earthquakes, explosion loading and tunnel response, as well as dynamic rock support. This book shall be a good reference for pursuing work on rock dynamics.

This paper firstly covers the apparatus used for rock testing under dynamic loading in brief and then gives an overview on behavior of underground openings subjected to dynamic loading because of earthquake and blast/explosion.

2. DYNAMIC ROCK TESTING

Split Hopkinson pressure bar (SHPB) is a commonly used to test various materials at dynamic loading conditions. Set up of Split Hopkinson pressure bar is given in Figure 1.

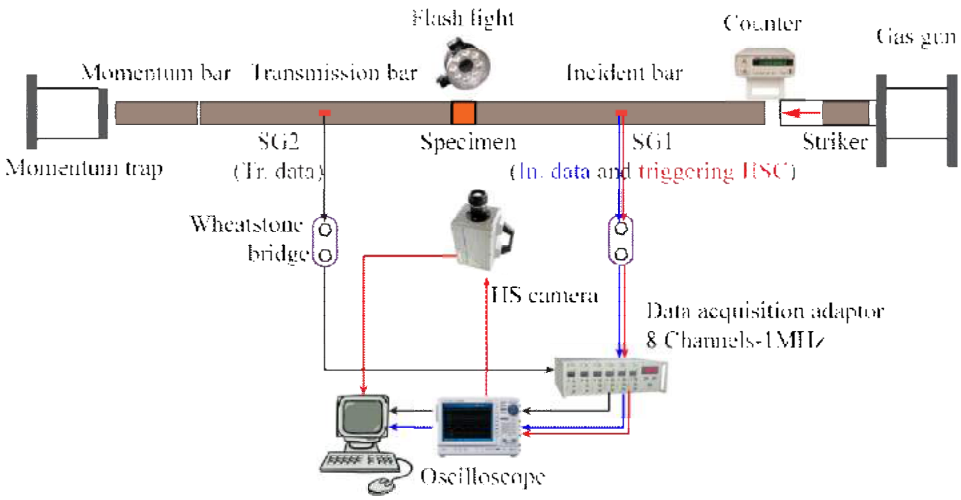


Fig. 1 - Schematic view of SHPB, high-speed camera and data acquisition (Zhao, 2013)

British electrical engineer Bertram Hopkinson first suggested such measurements in 1914. The setup used today is based on a modification developed by Herbert Kolsky in London in 1949. It is sometimes also called split Hopkinson Kolsky bar.

SHPB consisted of a gas gun, striker bar, incident bar, transmitted bar and the specimen is sandwiched between incident and transmitted bars. In this setup, gas gun is pressurized up to a predetermined pressure. As the pressurized gas is released from gas gun, striker bar is set into motion and an initial velocity is attained. Once the striker bar hits to the incident bar face, a stress wave is created at the impacted end of incident bar, then this wave travels down the incident bar from striker bar end to incident bar/specimen interface, called “incident wave (inc.)”. When stress wave reaches the incident bar/specimen interface, part of the stress wave is transferred to specimen and caused a rapid deformation while part of stress wave is reflected back to incident bar, called “reflected wave (ref.)”. As the plastic deformation occurs, stress wave travels in specimen down to transmitter bar/specimen end. At this point, part of the stress wave is again reflected in specimen while remaining part is transferred to transmitter bar, called “transmitter wave (trans.)”. Loading duration of stress wave, *T*, produced in a SHPB experiment is directly proportional with the length of striker bar. Velocity of striker bar is controlled with the pressure level in gas gun and magnitude of stress wave created is directly proportional with striker bar velocity.

As the stress wave propagates through the bars, incident, reflected and transmitted waves are recorded with the strain gages bonded to the surfaces of bars, as given in Figure 1. Due to the high strength of bar materials, the applied stresses remain in the elastic deformation region, so the stress and strain values of the specimen can be measured by acquired strains as a function of time from the full bridge strain gages. Strain and stress of specimen are then calculated.

Stress wave with high amplitude can influence rock material strength increase and rock material fails with more fractures. However, it is not clear yet the cause of high density of fracturing. The split Hopkinson pressure bar (SHPB) technique is a popular tool to explore the mechanical causes of loading rate effects on rock strength and failure pattern. The SHPB application is developing to investigate rate effects on fracture branching, multiple fracture initiation and crack propagation velocity.

Huang et al. (2010), using the SHPB on granites have obtained Eq. 1, which shows the linear increase in tensile strength with the loading rate.

$$\sigma_t = 11.6 + 0.0108\,\sigma' \tag{1}$$

where
σ_t = Tensile strength in MPa and
σ' = Loading rate in GPa/sec.

It has been observed by other researchers also that the dynamic tensile strength is more than the static tensile strength. Under dynamic condition, it has also been observed that the density of rock fracture on failure is also more. The reason for this perhaps is that more number of cracks are developed in dynamic testing and therefore more energy is required for cracks to join to give tensile strength, whereas in static tensile strength only one crack is developed and the energy is concentrated to fully crack the rock along this crack. This is to be further tested and analyzed to arrive at a definite conclusion.

Zhao (2013b) obtained Figure 2, a plot between the compressive strength and the loading rate. It can be seen in Figure 2 that the compressive strength slightly increases with the loading rate.

Sato et al. (1981) have attached the confining pressure vessel in to the SHPB apparatus to study the dynamic rock behavior under different confining pressures and obtained the following results.

- The dynamic deformation behavior of rocks in triaxial stress state becomes more ductile at a lower confining pressure than in the static condition, that is, rocks are inclined to be more ductile as the strain rate is higher.
- The dynamic fracture strength increases linearly with the increase of the confining pressure and is in parallel to the curve of static strength.

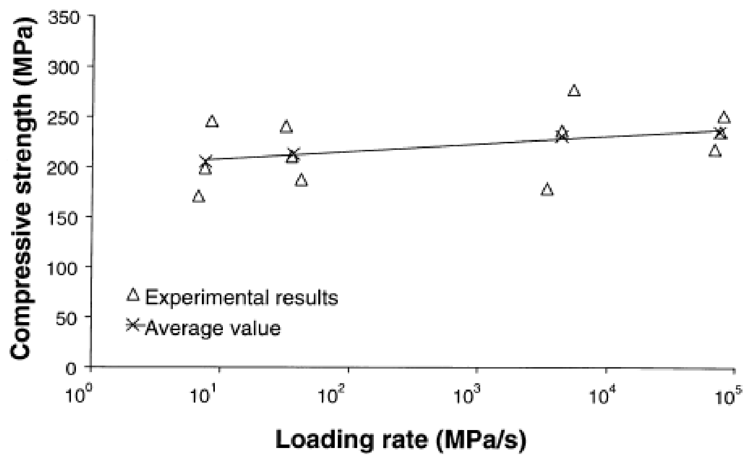


Fig. 2- Loading rate vs. the compressive strength (Zhao, 2013b)

Zhao et al. (1999) carried out experiments to study the properties of Bukit Timah granite from Singapore under dynamic conditions. Results of the studies showed that rate (time) and stress wave effects are two important factors influencing rock dynamics problems in addition to all the factors influencing the traditional rock mechanics problems. It is interesting to know that the P-wave velocity increases after saturation of the rock cores whereas the static elastic modulus decreases significantly after saturation (Mehrotra, 1992).

It is believed that numerical modelling with the proper understanding of the basic dynamic properties of the rock materials, rock joints and rock mass, are the key approach to study the shockwave propagation in fractured rock mass, the response of rock mass and the stability of rock structures under dynamic loads. Small-scale field experiments and laboratory tests provide the necessary input parameters for the numerical modelling. Limited large-scale field tests should be conducted mainly to calibrate the numerical simulation.

All software FLAC, UDEC and 3DEC analyze rock dynamic problems, predicting the damage mechanics. It should be remembered that the dynamic normal stiffness is much higher than the static normal stiffness of rock joints. The dynamic shear stiffness is also higher. In the seismic interaction analysis of concrete dams and rock foundations, the elastic modulus (E_e) should be used rather than the modulus of deformation (E_d). Same is the case with machine foundations in estimating the natural frequency.

The allowable bearing pressure is 50% more than the static bearing pressure which is obtained from plate load tests for a 12mm permissible settlement of the foundations on rocks. Further, the shear strength of very rough dilatating rock joints can be very high under dynamic shear stresses than the static shear strength.

As such the studies are carried out on dynamic behavior of rock material and rock joints. Further study is required to strengthen the understanding on the topic.

3. SEISMIC LOADING ON UNDERGROUND STRUCTURES

3.1 General

Underground structures have features that make their seismic behaviour distinct from most surface structures, notably (i) their complete enclosures in soil or rock and (ii) their significant length (i.e. tunnels) (Hashash et al., 2001). In general, underground structures have a lower rate of damage than surface structures. Nevertheless, some underground structures have experienced significant damage in large earthquakes (M>7), including 1995 Kobe, 1999 Chi-Chi, 2004 Chuetsu, 2005 Kashmir and 2008 Wenchuan earthquakes (Hashash et al., 2001 and Aydan et al., 2010).

The intensity of seismic force experienced by each tunnel differs owing to their different distances from the displaced fault zone and the epicenter of the earthquake. The distance to the ground surface or to nearby slopes also influences the seismic effect. Seismic waves propagate in the ground and lose energy because of dispersion and ground resistance, causing tunnels to be under greater seismic forces if they are closer to the displaced fault zone or the epicenter. Additionally, when seismic waves reach the ground surface, they release energy due to reflection or refraction, and thus tunnels near the surface, and especially those near slope faces, will absorb a greater seismic energy (Wang et al., 2001).

Most mountain tunnels generally run through very hard ground, and a few tunnels pass through the displaced fault zone and fractured zones. Seismic waves propagate faster in hard and dense materials, and thus less energy will be released at places where the tunnels lie in ground that is harder than the tunnel structure, meaning that such tunnels will tend to deform with the ground and suffer less damage. On the other hand, if the tunnels lie in relatively weaker ground they will absorb larger amounts of energy and thus suffer greater damage. Concrete linings can particularly be damaged easily by ground displacement or ground squeeze where soft and hard grounds meet, as soft and hard grounds behave differently during earthquakes. Any unfavorable events such as cave-in or collapse during tunnelling would extend the plastic zone around the tunnel, weaken the surrounding rock and cause excessive vibration when seismic waves pass through. In addition, if the ground has previously experienced vertical stress from loosening, plastic stress owing to squeezing, inclined stress or any other weakening processes, tunnels in these areas will suffer greater damage to their concrete linings during an earthquake.

Very few data are available on the earthquake induced damage to underground structures and tunnels before 1970's.

Owen and Scholl (1981) updated the work by Dowding and Rozen (1978), collecting 127 cases of damage to underground structures. An important addition came from the cut-and-cover tunnels damaged during the San Francisco (1906) and San Fernando (1971) earthquakes. These structures were shallow and generally constructed in poor soil.

Sharma and Judd (1991) enlarged the collection of the previous authors reaching a total number of 192 cases for 85 different earthquakes. To correlate seismic vulnerability of a tunnel to some relevant factors, six parameters were examined: tunnel cover, subsoil type, peak ground acceleration, magnitude of the earthquake, distance from the epicentre and type of lining support. Most of the damages (60%) affect shallow tunnels (depth lower than 100m); Many cases (42%) concern unlined tunnels in rock.

Hosseini et al. (2010) studied the stability of main access entry to C1 coal seam of Tabas collieries in Iran using Phase^2 software in static and dynamic states. They concluded that the stress and displacement balance of forces around the tunnel are adversely affected which leads to instability in the tunnel. They also concluded that increasing the stiffness of the support system can increase the effect of the seismic loads.

Dowding and Rozen (1978) divided their database using the damage level as a criterion. They considered three damage classes (no damage, minor damage, damage). Huang et al. (1999) and Wang et al. (2001) added a damage level to such classification, subdividing the second group in two classes (slight and moderate).

As such, the three damage levels are defined by using the crack width (W) and length (L), the tunnel functionality and the need of restoration after earthquakes (Lanzano et al., 2013):

- *Class A*: Slight damage. L<5m W<3mm. Perfect functionality. No restoration needed. No service stop;
- *Class B*: Moderate damage. L>5m W>3mm. Differential displacements cause deep cracks, spalling and exposed reinforcement. Compromised functionality. Service interruption until the complete restoration with aseismic expedients;
- *Class C*: Severe damage. Landslide and liquefaction. Structural collapse of the lining. Service stop without any possible restoration;

Corigliano (2007) more recently subdivided 230 worldwide cases from 35 different earthquakes in these three classes: severe damage occurred only for 6 seismic events.

Dowding and Rozen (1978) have compiled the seismic response of 71 tunnels. The Fig. 3 extracted from this study shows that the tunnels are less susceptible to damage than the surface structures. The peak acceleration at the surface of less than 0.2 g magnitude did no damage to the tunnels. The accelerations between 0.2 and 0.5 g did only minor damage. The damage was found to be significant only when the peak ground acceleration exceeded 0.5 g. In such cases most of the damage that occurred was located near portals.

3.2 Observed Response of Underground Structures on Seismic Loading

Several Japanese investigators measured earthquake motion simultaneously at the ground surface and at depth. The findings of these studies may be summarised as follows. Nasu (1931) determined the ratio of displacements due to earthquakes at the surface and tunnels up to depths of 160m. The geology consisted of lake deposit on the surface and volcanic andesite underneath. The surface/depth displacement ratios were 4.2, 1.5 and 1.2 for periods of 0.3, 1.2 and 4 seconds, respectively. Kanai and Tanaka (1951) measured acceleration at depth up to 600m in copper mines in Paleozoic rock. The ratio of maximum surface displacement to that at the depth of 300m was about 6:1. Iwasaki et al. (1977) obtained acceleration records up to a depth of 150 m during a period of 5 years. The borehole accelerometers were installed at four locations around the Tokyo bay. Three of these sites were in sand and clay while the fourth was in siltstone. During the period of measurement 16 earthquakes with magnitudes ranging from 4.8 to 7.2 were recorded. The analysis showed that the maximum acceleration depended heavily upon the soil conditions. The ratio of surface/depth accelerations are about 1.5 on a rocky ground; 1.5 to 3.0 in sandy ground and 2.5 to 3.5 in clayey ground.

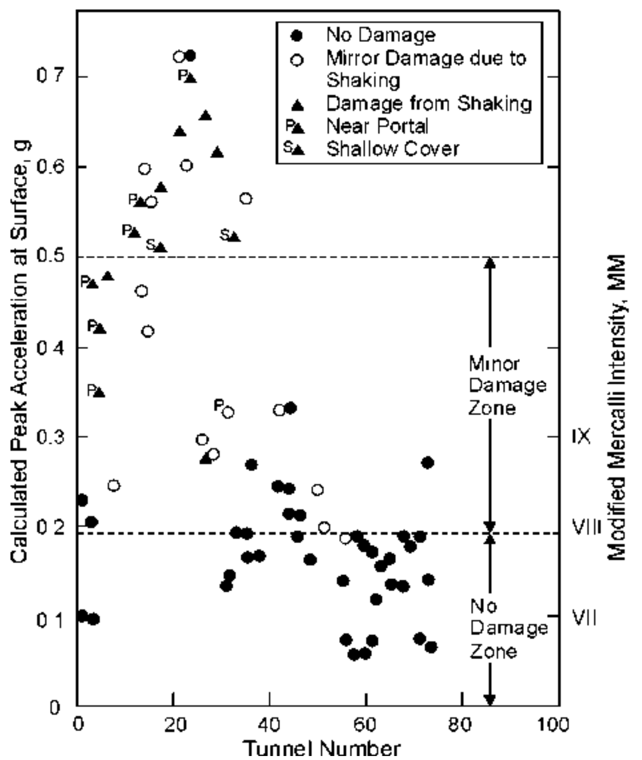


Fig. 3 - Calculated peak acceleration at the surface and associated tunnel damage (Dowding and Rozen, 1978)

The study of Alaskan earthquake which was one of the largest earthquake of 20th century (M = 8.5) showed that while the surface damage was extreme, the underground structures escaped without any significant damage (Eckel, 1970). Similar results were reported by Cooke (1970) on the Peru earthquake of May 31, 1970. The earthquake of 7.7 magnitude on Richter scale did no damage to 16 rail road tunnels of combined length of 1740m under small ground cover located in MM-VII and MM-VIII intensity zones. Similarly, no damage was reported to the underground hydro-electric power plant, 3 coal mines and 2 lead zinc mines located in MM-VII intensity zone.

The Himalayan experience may be added to the above. A large number of shrines are located in caves deep inside the Himalayas. Although, this is a seismically active region and several big earthquakes have occurred in this area over the centuries but nothing has happened to these shrines. It is understood that the size of natural caves, tunnels and caverns is smaller than the quarter wave length of seismic waves. Hence, underground openings are not noticed by the seismic waves and so there is no resonance and damage of the openings.

The study of the behavior of support system of large underground openings due to seismic excitation for over seven years period showed that earthquake vibrations do not appear to affect the rock pressure on steel rib support system. However, it has been found that earthquake shocks of moderate intensity (M<5) affect wall support pressure on rock anchors temporarily. The percentage increase in cable tension is less than 2%. Thus, the rock pressure theory need to be developed to account for accumulated strains in the rock mass due to recurring earthquakes in seismic regions (Mitra and Singh, 1988).

The monitoring of the pressure shafts for Chhibro underground powerhouse complex indicated that water level in the surge tank and the water temperature influence the stresses in the penstock liner although earthquake vibrations do not. Further, anchor loads in the machine hall have been observed to increase slightly during the rainy season and during earthquakes. This effect has not been observed in the shaft liner since the shaft is inherently more stable than the high caverns (Mitra and Singh, 1989).

There may be some residual strains in the rock mass due to effect of nearby thick shear/fault zone. The wall support pressure near the shear/fault zone seems to increase significantly with time due to strains accumulated after each earthquake shock. This problem may not be significant if shear zone is far away, i.e. 1.5 times the span of the opening (Mitra, 1991; Mitra and Singh, 1992). Consequently an empirical support pressure theory was proposed to account for accumulated strains in the rock mass due to recurring earthquakes in a seismic region. The proposed empirical equation relating seismic support pressure (based on Barton, 1984 Eqs. 6 & 8) near thick shear zone due to recurring earthquakes with rock mass quality of the cavern wall Q_{wall} is as follows.

$$\Delta p_{eq} = \frac{0.2 \, x \, 10^{-4}}{J_r} . N_{eq} \, (Q_{wall})^{-1/3} \qquad \text{(Mitra, 1991)} \qquad (2)$$

Where,

Δp_{eq} = Seismic support pressure due to recurring earthquake near thick shear zone in MPa,

J_r = Joint roughness number,

Q_{wall} = Wall rock mass quality, and

N_{eq} = Total number of earthquakes (2<M<5) within 110km range within the life time of an opening.

Equation 2 does not account for predominant period of shocks, which is found to have some effect on Δp_{eq} (Okamato, 1973).

An earthquake of 6.3 magnitude occurred on October 21, 1991, which was centered near Uttarkashi, approximately 100 km away from the project site, which devastated the entire Uttarkashi district of North Indian Himalayan state Uttarakhand. Recorded damage in the Chhibro underground power house cavern on account of this earthquake was limited to minor cracks in the region closest to the shear/ fault zones. There was no damage in the sections of the power house complex away (up to a distance of width of opening B) from the shear/ fault zone (Mitra & Singh, 1995).

An analysis by Mitra and Singh (1997) shows that the dynamic support pressures are negligible compared to long-term support pressures in the roof of the chamber near the shear zone due to residual strains in the nearby rock mass.

Figures 4 and 5 highlight the degree of damage to tunnels near to the ground surface and faults after the 1999 Chi-Chi earthquake (M=7.3) in Central Taiwan. The tunnels passing through the displaced fault zone suffered catastrophic damage, and the lining was sheared off. Meanwhile, for the 50 tunnels in the hanging wall group, 5 tunnels (10%) are classified as undamaged, 21 tunnels (42%) were lightly damaged, 11(22%) moderately damaged and 13 (26%) severely damaged. Finally, for the 6 tunnels in the footwall and other areas, 3 (50%) suffered no damage at all, 2 (34%) were lightly damaged and 1 (16%) was severely damaged. Evidently, the tunnels located in the hanging wall area suffered more damage than those in the footwall area. No damage was reported in the Taipei subway, which is located over 100 km from the ruptured fault zone (Hashash et al., 2001).

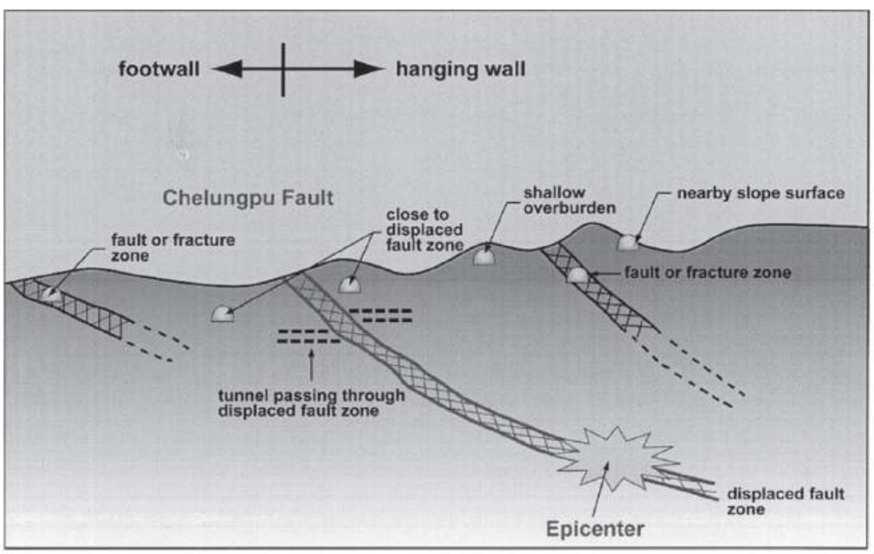


Fig. 4 - Tunnel locations relative to Chelungpu thrust (Wang et al., 2001)

3.3 Various Types of Tunnel Failure

Wang *et al.* (2001) suggested eight patterns of cracks induced into the tunnel lining during an earthquake (Fig. 6):

a) *Sheared off lining*: it occurs for tunnel passing through active faults;

b) *Slopes failure induced tunnel collapse*: it occurs when the tunnel runs parallel to slopes generating landslides passing through the lining;
c) *Longitudinal cracks*: it occurs when the tunnel is subjected to higher deformations due to surrounding ground;
d) *Traverse cracks*: it occurs when the tunnel has weak joints;
e) *Inclined cracks*: it occurs for a combination of longitudinal and transversal cracks;
f) *Extended cracks*: it occurs when there is the partial collapse of linings for seismic intense deformation;
g) *Wall deformation*: it occurs when there is a transverse reduction due to the invert collapse;
h) *Spalling of lining*: it occurs when the transversal section completely collapses.

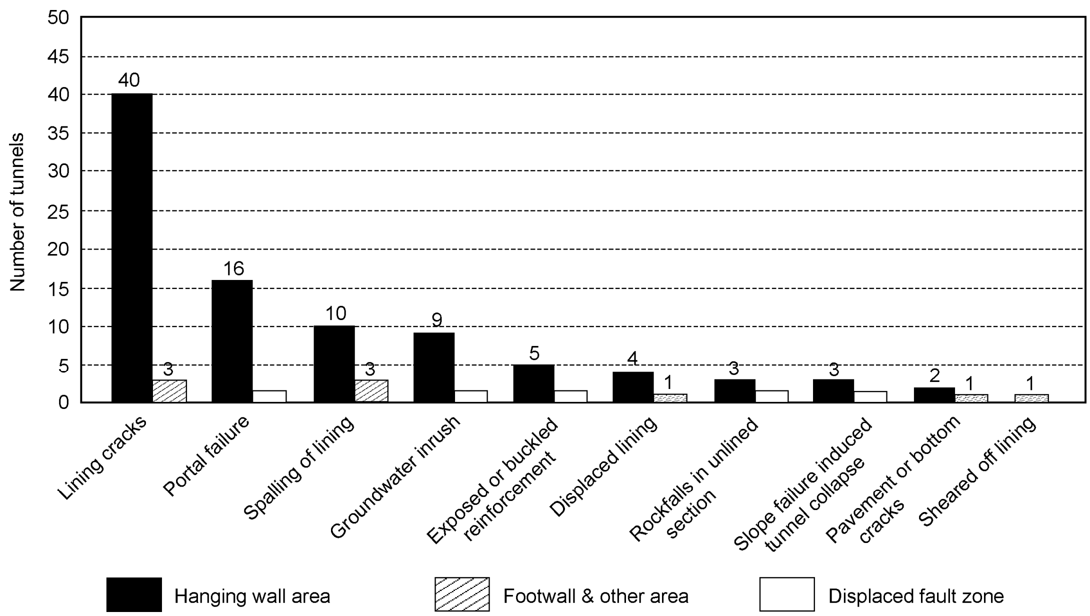


Fig. 5 - The numbers of tunnels suffering various types of damage (Wang et al., 2001)

In Table 1 the possible links between causes (geological, geotechnical and structural factors) and effects (type of damage according to Fig. 6) are reported, showing when the influence is weak or decisive.

3.4 Performance of Underground Structures Subjected to Dynamic Loading

Based on the above and as highlighted by Hashash et al. (2001) and Wang et al. (2001), following are the general observations pertaining to dynamic performance of underground structures.

- Underground structures suffer appreciably less damage than surface structures. All the cracks and collapses take place only for severe earthquakes, with high magnitude and without special a-seismic expedients. Generally for moderate earthquakes, the static design is enough to protect structures from seismic motion

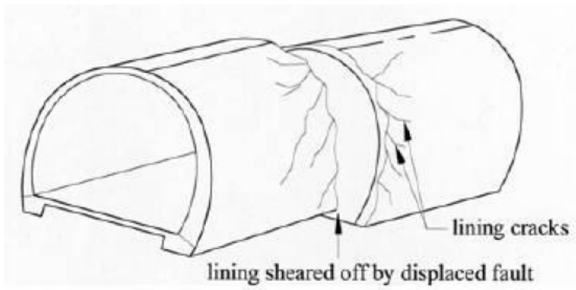


Fig. 6a - Sheared off lining

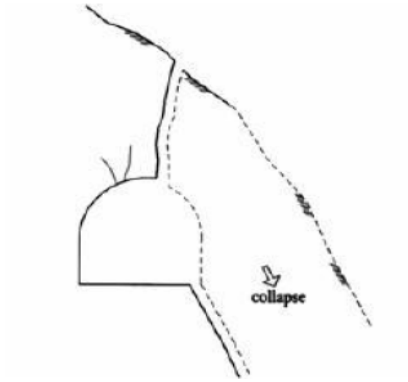


Fig. 6b - Slopes failure induced tunnel collapse

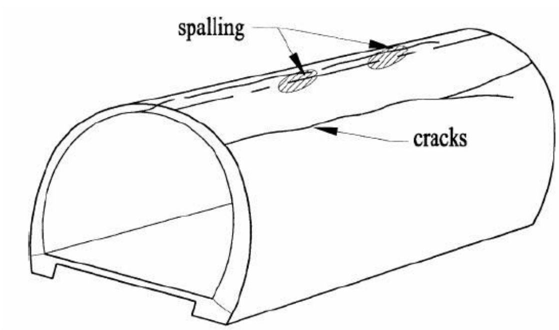


Fig. 6c - Longitudinal cracks

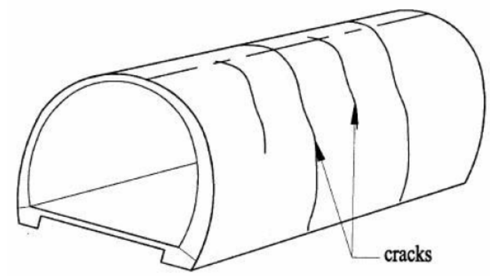
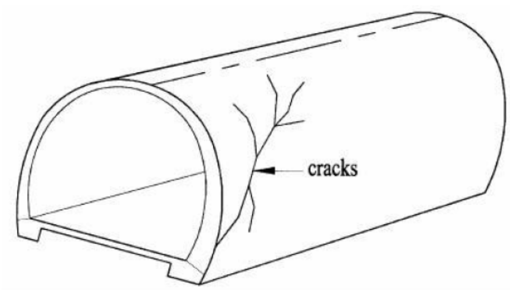
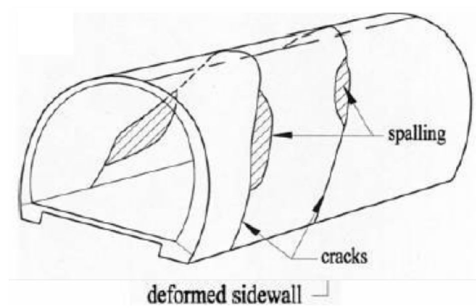


Fig. 6d - Transverse cracks



e. Inclined cracks



f. Extended cross cracks

Fig. 6g - Bottom cracks and wall deformation

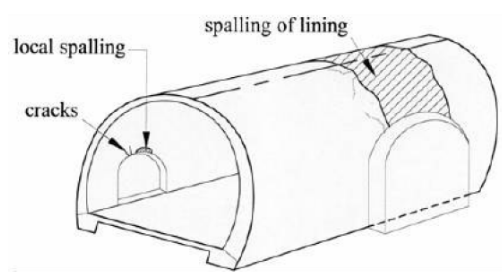


Fig. 6h - Spalling of lining

Fig. 6 - Type of damages in tunnels because of seismic loading (Wang et al., 2001)

Table 1 - Links cause/effects to tunnel damage (Wang et al., 2001)

S.No.	Possible factors	a	b	c	d	e	f	g	h
1	Passing through fault zones	*							
2	Unfavourable ground condition				o		*		
3	Interface hard-soft ground						*		
4	Nearby slope surface and portals		*		*	*	*		
5	Collapse during construction			o		o		o	
6	Lining cracks before earthquake			o	o				
7	Poor structural arrangements				o	o			*
8	Unreinforced concrete lining	o	o		o	o	o	o	*
9	Deteriorated lining material			o	o				
10	Cavity existed behind lining			*		o			

Notations: * = presents significant influence; o = presents moderate/weak influence

- Reported damage decreases with increasing overburden depth. Deep tunnels seem to be safer and less vulnerable to earthquake shaking than are shallow tunnels.
- Damage at and near tunnel portals may be significant due to slope instability.
- Underground facilities constructed in soils or poor to very poor rock masses are expected to suffer more damage compared to openings constructed in competent rock.
- Tunnels running across active faults suffer severe damage due to differential displacements which are incompatible with structure strength. Wherever it is possible, the tunnel should not pass through active faults. Alternatively, segmented concrete lining should be used across the active fault zone (Singh and Goel, 2006).
- In tunnels located near faults/thrusts (with plastic gouge) in seismic areas, the ultimate support pressure might be about 25 per cent more due to accumulated strains in the rock mass along the fault (Mitra, 1991).
- Larger accelerations are noticed in the hanging wall side compared to foot wall side (Aydan, 2009).
- Lined and grouted tunnels are safer than unlined tunnels in rock. Shaking damage can be reduced by stabilizing the ground around the tunnel and *by improving the bond strength* between the lining and the surrounding ground through grouting.
- Tunnels are more stable under a symmetric load, which improves ground-lining interaction. Improving the tunnel lining by placing thicker and stiffer sections without stabilizing surrounding poor ground may result in excess seismic forces in the lining.
- Backfilling with compressible material and rock-stabilizing measures may improve the safety and stability of shallow tunnels.
- Damage may be related to peak ground acceleration and velocity based on the magnitude and epicentral distance of the affected earthquake.
- Duration of strong-motion shaking during earthquakes is of utmost importance because it may cause fatigue failure and therefore, large deformations.

- High frequency motions may explain the local spalling of rock or concrete along planes of weakness. These frequencies, which rapidly attenuate with distance, may be expected mainly at small distances from the causative fault.
- Ground motion may be amplified upon incidence with a tunnel if wavelengths are between one and four times the tunnel diameter. This observation shows that high frequencies can be more dangerous than lower ones, but such frequencies are generally outside the range of a typical earthquake energy content.
- Water and gas supply system are more vulnerable compared to metro and road tunnels, as steel tubes have a thickness/diameter ratio lower than concrete tunnels. Most of the damage of such lines occurs in saturated sand due to liquefaction.
- Most of the metro lines and roadway tunnels are only damaged by extremely severe earthquakes. Some authors (Iida et al.,1996; Yoshida, 1999), describing the damage of the metro line of the city of Kobe during the earthquake of 1995, show that many sections suffered cracks and collapses for the absence of a-seismic expedients. On the other hand some American metro lines had good performance during the Loma Prieta earthquake (1989), thanks to special seismic joints used in the tunnel design.

A deep tunnel ($H>>350Q^{1/3}$ m and $J_r/J_a > 0.5$; where H is tunnel depth in meters, Q is Barton's rock mass quality; J_r & J_a are parameters of Barton's Q-system) may fail dynamically by rock burst (Singh and Goel, 2006). The strain energy release rate upon excavation is more than the permissible limit, so the high stored strain energy is suddenly released, causing violent failures of rock masses. If the stress-strain curve of rock material is type II, the rock burst is likely to take place under high in situ stress conditions (H>900m).

3.5 Psuedo-Static Approach for Estimating Seismic Support Pressure

A pseudo-static approach, which is quite popular in geotechnical engineering, is proposed to estimate the support pressure under dynamic conditions in underground openings. According to this approach, the vertical peak acceleration is $\alpha_v.g$ in roof and horizontal peak acceleration is $\alpha_h.g$ in the wall of the tunnel, where g is the acceleration due to gravity (Fig. 7). It is reasonable to assume in the case of jointed rock masses that the vibrating mass is the mass of rock wedge which is naturally formed by three critical rock joints. Psuedo-static analysis assumes that the unit weight of rock mass (γ) is modified to $(1+ \alpha_v).\gamma$. It follows that the increased (total of static+seismic) support pressure because of earthquake ($p_{seismic}$) may be taken approximately as follows (Singh and Goel, 2006).

<u>In roof</u>

$$\gamma = (1+ \alpha_v)\gamma \quad (3)$$

$$p_{seismic} = (1+\alpha_v).\ p_{roof} \quad (4)$$

<u>In walls</u>

$$p_{seismic} = (1+\alpha_h).\ p_{wall} \quad (5)$$

where

α_v = coefficient of the vertical peak acceleration at roof ,

α_h = coefficient of the horizontal peak acceleration at walls, and

γ = unit weight of the rock mass.

As per Barton (1984),

$$Q_{seismic} = Q/2 \tag{6}$$

Where Q is Barton's rock mass quality and $Q_{seismic}$ is Barton's rock mass quality for seismic conditions.

Equation 6 is shown in terms of coefficient of vertical peak acceleration as follows:

$$\begin{aligned} Q_{seismic} &= Q/(1+\alpha_v)^3 \qquad (7) \\ &= Q/2 \qquad \text{(for } \alpha_v = 0.25\text{, similar to Eq. 6)} \end{aligned}$$

Above approach may be used to estimate $Q_{seismic}$ for roof for different values of α_v. In walls, $Q_{seismic} = Q_{wall}/(1+\alpha_h)^3$.

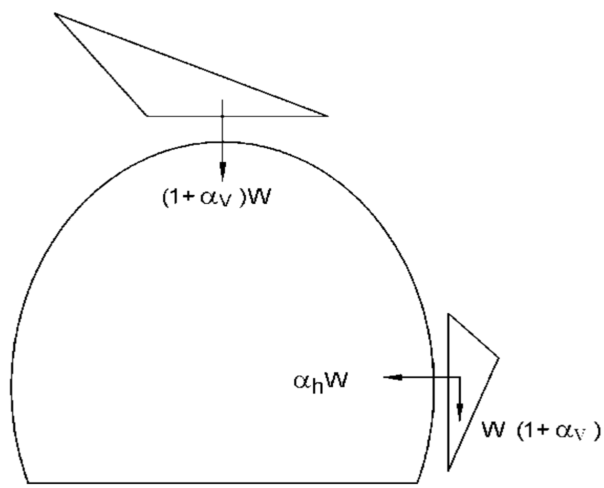


Fig. 7 – Peak acceleration experienced by rock wedges during earthquakes

Considering Eq. 6 and Barton's equation of estimating support pressure, $p_{seismic}$ will be,

$$p_{seismic} = 1.25 p_{static} \text{ (Barton, 1984)} \tag{8}$$

Another cause of seismic support pressure is continuous building up of the residual strains around an opening with successive earthquakes, particularly near the faults, etc. Nevertheless the hypothesis of Barton (1984) appears to be realistic in view of the fact that tunnels have seldom failed during even major earthquakes. The design of support system may be selected from the Grimstad and Barton (1993) chart for the seismic rock mass quality ($Q_{seismic}$) as per Eq. 7.

Bhasin and Thomas (2013) highlighted that 50% reduction of Q_{static} or Q (Eq. 6) actually gives a 25% increase in support pressure (Eq. 8). This 25% increase is in the range of 15-

44% increase in maximum axial force that was observed for seismic conditions through numerical modelling studies. It is however, important to add that the use of rock mass reinforcement and tunnel support using Q/2 from Grimstad and Barton (1993) support chart will not be appropriate in cases where adverse geological features such as wedges or faults exists. Such cases warrant special design of reinforcement based on the orientation and strength-deformation properties of the geological features (Barton, 1984, 1994).

Elastic and plastic analyses were carried out for tunnels of different sizes and the results are summarized in Fig. 8 (Bhasin et al., 2006). It can be seen from this figure that for the elastic analysis there is not much difference in the maximum axial force on the lining as the tunnel size increases from 5 to 20 m. In addition, there is not much difference in the maximum axial force on the lining when dynamic loading is applied as compared to static loading. For plastic analysis the load on the lining increases significantly with the tunnel diameter. This is in line with the studies carried out by Bhasin and Grimstad (1996) and the observations of Singh et al. (1992) which showed that the rock support pressure in tunnels in weak rocks is dependent on the dimensions of the tunnel.

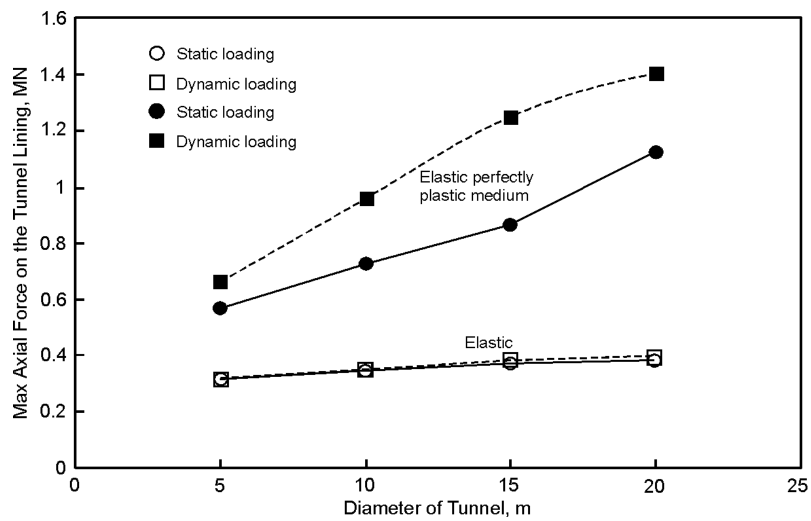


Fig. 8 - Variation of maximum axial force on tunnel lining with tunnel size (Bhasin and Thomas, 2013)

The dynamic increment in support pressure in rail tunnels may perhaps also be assumed to be negligible and of the same order as that of earthquakes. However, where overburden is less than 2B (where B is the width of the opening), the roof support pressure is taken equal to the overburden pressure. This conservative practice is due to errors inherent in the survey of hilly terrain. In case the shallow rail or road tunnels are excavated in the seismic rocky areas, concrete lining is provided with contact grouting to improve bond strength between concrete lining and rock mass. Consolidation grouting of loose rock mass should also done to prevent further loosening of the rock mass during earthquakes. Back grouting ensures intimate contact between concrete lining and rock surface which may not allow bending of the lining and no bending stresses are likely to develop during earthquakes (Singh and Goel, 2006).

4. BLAST LOADING

4.1 Blast on Surface

It is being realized now that underground openings may provide safety against nuclear or missile attacks on the surface above the underground structures. The depth of overburden is the most important factor. Rock engineers are now approached to design support systems which are safe against blast loading. The concept is same as for seismic loading, except that the peak acceleration may be of high intensity ($\alpha_v > 1$, sometimes 5).

The experience of tunnelling or mining through rock burst prone areas may be relevant here. Long resin bolts/anchors (without pre-tension) have been successfully used as they are able to withstand vibrations of high intensity and arrest propagation of fractures in the rock mass. The steel fibre reinforced shotcrete (SFRS) is also a ductile material and has high fracture toughness and high shearing resistance. The principle for transforming a catastrophic brittle failure into the plastic failure is that the brittle rock mass is converted into the ductile reinforced rock arch. The SFRS is also ductile obviously due to steel fibres. For tunnel stability the support system must be reinforced. However, it is cautioned that with the increasing thickness or stiffness of the support system, the inertia is increased and thus the tunnel flexibility is reduced. Consequently, the effect of the dynamic stress on the tunnel increases (Hosseini et al., 2010).

It may be mentioned here that the peak acceleration of blast waves do not attenuate rapidly in hard rocks. The damping coefficient of hard rocks is also low. As such the coefficient of peak acceleration (α_v) is likely to be quite high in shallow openings. Conservative approach is the need of design of underground structures of strategic importance, as future weapons and atomic bombs are going to be unimaginably disastrous in its life time.

The dynamic model tests show that rock wedge in the roof tends to slide down slightly on shaking. Hence wedge theory of support pressure would perhaps be applicable under heavy dynamic loading such as blast loading.

The dynamic support pressures are likely to be high according to Eqs. 4 and 5. In case $\alpha_v > 1$, the rock wedge at the bottom of the opening may also be dislodged in upward direction. Thus the required dynamic support pressure at the bottom of an opening is estimated by assuming the unit weight of rock mass equal to $(\alpha_v - 1)\,\gamma$ (Fig. 9). So full-column grouted resin rock bolts should be installed in bottom of tunnels also.

$$p_{bottom} = (\alpha_v - 1)\, p_{roof} \tag{9}$$

Hence rock anchors and SFRS may also be needed at the bottom of the opening. Perhaps it is not necessary to make bottom of the opening curved surface to reduce dynamic tensile stresses.

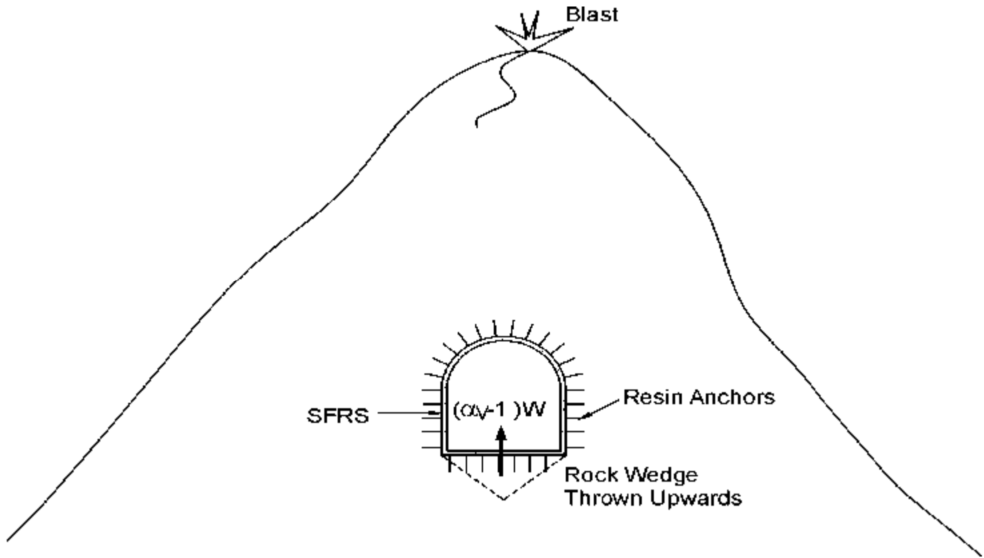


Fig. 9- Support system for blast loading on surface

One may also keep in mind that the overburden of rock mass at portal of the tunnel should be say 5.B in the blast prone area (but normally 3B), where B is the span of opening. Further the maximum overburden over an opening should be much less than 350 $Q^{1/3}$m where $J_r/J_a < 0.5$, this will ensure non-squeezing condition in the openings (Here Q is Barton's rock mass quality; J_r is Barton's joint roughness number and J_a is Barton's joint alteration number). Yet a minimum of rock cover of 300m above underground opening should be ensured for safety against mega nuclear attacks right above them. Needless to mention that the rock mass quality near portals is down graded to Q/2 and it is Q/3 near intersection of openings (Barton et al., 1974). So additional down grading of rock mass quality ($Q_{seismic}$) may be done using Eq. 7 near portals and intersections of underground openings for blast loading.

4.2 Explosion Effects in Underground Ammunition Storage Sites

- The blast wave originating from an explosion in an underground storage chamber will surge through the rock formation as ground shock and will escape as blast through the access tunnel into the open air. The strong confining effect of an underground storage site and the large amount of hot explosion gases generated will produce a relatively constant high pressure in the chamber. This pressure may break up the rock formation and produce a crater. The kinetic energy (dynamic pressure impulse) of the blast in the main passageway is very high compared to an explosion in free air. Objects like unexploded ordnance, rock, gravel, equipment, and vehicles will be picked up and accelerated up to velocities of several hundred metres per second before leaving through adits. In addition, engineered features can collapse and cause debris hazards. Break-up of the cover will cause projection of a heavy ejection of rock and earth in all directions on the surrounding surface area.

- The explosion gases will surge at a high velocity through the access tunnel into the open air where they will burn completely. The escaping gases will carry along ammunition, rock debris, installations, and lining onto surrounding areas.
- A disturbance near the surface of the ground will emit compression P-waves, shear S-waves, and Raleigh surface R-waves in a semi-infinite elastic medium. Deeply buried disturbances will emit only P-waves and S-waves, but in the far field, interface effects will result in R-waves on the surface. For all of these wave types, the time interval between wave front arrivals becomes greater and the amplitude of the oscillations becomes smaller with increasing standoff distance from the source.

The first wave to arrive is the P-wave, the second the S-wave, and the third the R-wave. The P-wave and S-wave cause minor tremors, as these waves are followed by a much larger oscillations when the R-wave arrives. The R-wave is cause of the major tremor because: (i) about two-thirds of the ground shock energy at the source goes into the R-wave, and (ii) the R-wave dissipates much less rapidly with distance than either the less energetic P-wave or S-wave. The P-waves and S-waves dissipate with distance r to a power of r^{-1} to r^{-2}. At the surface, P-waves and S-waves dissipate with distance as r^{-2}, while R-waves dissipate with distance as $r^{-0.5}$. The greater energies being transmitted by R-waves and the slower geometric dissipation of this energy causes R-waves to be the major tremor (the disturbance of primary importance for all disturbances on the surface).

4.2.1 Small-Scale Model Tests and Validity of Scaling Laws

(i) A portion of the blast energy from an underground detonation is used to compress the surrounding geological media. This allocation of energy should be considered when evaluating the experimental results of underground tests.

(ii) Small-scale model tests that are constructed of non-responding materials do not exhibit the non-linear energy loss effects which are typical of an underground explosion. Therefore, air blast results from non-responding models tend to be conservative for predicting hazards that would occur in an actual underground event. In spite of this limitation, small-scale model tests are still of value for design purposes.

For details on ‘underground storage of ammunitions and explosives’ book of Goel et al. (2012) may be referred.

4.2.2 Depth of Cover above Storage Chambers

Depth of cover considerations is significant for planning and evaluating underground explosive storage. It is intuitively evident that the amount of earth / rock cover over a given amount of explosive will have a significant effect in the blast and shock phenomenology in the event of a detonation. In the limiting case, an explosion of almost any size that would be conceivable for conventional storage situations would produce negligible surface effects if it occurred at a depth of several kilometers. However, for practical situations, the depth of cover is a factor that must be quantitatively considered in evaluating air blast and debris effects.

The depth of cover or chamber cover thickness is the shortest distance between the ground surface and the natural rock surface at the chamber's ceiling or, in some cases, a chamber's wall. For all types of rock, the critical cover thickness (Cc) required to prevent breaching of the chamber cover by a detonation of explosive charge W is (DoD, 2004).

$$Cc = 1.0W^{1/3} \quad , \qquad \text{meters} \tag{10}$$

where Cc is in meters and explosive charge W is in kg.

5. CONCLUSIONS

Following conclusions are drawn on the basis of above.

- The underground openings/structures suffer appreciably less damage than surface structures (M>7).
- The proposed pseudo-static approach may be used for estimating the seismic support pressure in tunnels.
- It is found that the $Q_{seismic}$ proposed by Barton (1984) matches when the coefficient of vertical peak ground acceleration at roof is 0.25. Accordingly, $Q_{seismic}$ can be estimated for different coefficient of peak ground accelerations at roof.
- In case of tunnels or underground structures required to be located near thick shear zone or fault zone, an additional dynamic support pressure may be considered for estimation of long-term support pressure (Eq. 2).
- To take care of high intensity acceleration it is suggested to support tunnel invert also.
- For tunnel stability the support system must be reinforced. However, with the increasing thickness or stiffness of the support system, the inertia is increased and thus the tunnel flexibility is reduced. Consequently, the effect of the dynamic stress on the tunnel increases.
- The overburden above the nuclear-underground-defence shelters should be adequate to withstand the high shock waves and severe thermal shock waves.

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