Response and Earthquake Induced Damage of Underground Structures in Rock Mass

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ABSTRACT

Underground structures are generally known as earthquake-resistant structures as compared to surface structures. Nevertheless, the recent earthquakes showed that underground structures are also vulnerable to earthquakes. There may be several reasons for such damage such as high ground motions and permanent ground movements. This study attempts to describe various forms of damage to underground structures such as tunnels, caverns, natural caves and abandoned mines during major earthquakes. Results of various model tests on shaking table are also presented to show the effect of ground shaking on the response and collapse of underground structures in continuum and discontinuum. Furthermore, some empirical equations are proposed to assess the damage to underground structures, which may be useful for quick assessments of possible damage.

Keywords: Underground opening; Earthquake damage; Fault; Shaking; Concrete lining

1. INTRODUCTION

It is well known that the underground structures such as tunnels and powerhouses are generally resistant against earthquake-induced motions (Dowding and Rozen, 1978; Sharma & Judd, 1991). However, they may be damaged when permanent ground movements occur in/along the underground structures. There are several examples of damage to tunnels due to permanent ground movements during, for example, 1930 Tanna, 1978 Izu-Oshima-Kinkai, 1995 Kobe, 1999 Düzce-Bolu, 1999 Chi-chi, 2004 Chuetsu, 2005 Kashmir and 2008 Wenchuan earthquakes. Most of tunnels have un-reinforced concrete linings. Since the lining is brittle, the permanent ground movements may induce the rupture of the linings and so falling debris may cause disasters with tremendous consequences to vehicles passing through. Therefore, this current issue must be urgently addressed. It should be also noted that the same issue is valid for the long-term stability of high-level nuclear waste disposal sites.

The seismic response and stability of areas situated above the abandoned lignite mines in Japan due to urbanization in recent years are of great concern. The July 26, 2003 Miyagi-hokubu earthquake (Mj6.2) caused some damage in the area of abandoned lignite mines nearby Yamamoto town, which is just above the hypocenter of the earthquake. It is also known that many collapses occurred in abandoned mines exploited by room and pillar technique in the past earthquakes in Japan.

This article is concerned with the stability problems and performance of underground openings during earthquakes. A brief description of ground motions in underground openings and ground surface and an example of permanent ground deformation observed during the 2004 Chuetsu earthquake are first given. Some results of model tests on underground openings tested on a shaking table are presented. Numerical studies on the deep tunnels and shallow underground structures are also described. Then, several examples of damage to underground structures in various countries during major earthquakes are explained and their causes are discussed. In the final part, damage to underground structures by earthquakes is classified and some empirical equations are proposed for assessing the seismic stability of underground structures.

2. GROUND MOTIONS

It is well-known that the underground motions are smaller than those at ground surface. First instrumental studies on tunnels were carried out by Nasu (1931) during the aftershock activity following 1924 Izu earthquake with 2.4m offset. Kanai and Tanaka (1951) measured ground acceleration in underground cavern and on the ground surface. These measurements indicated that the surface acceleration was generally 2 times or greater than 2 times of underground values.

Komada and Hayashi (1980) reported the results of extensive monitoring of ground motions during earthquakes on underground caverns and adjacent tunnels. They also investigated the frequency content and amplification in relation to magnitude and distance of earthquakes. However, these caverns were not in the epicentral area.

Figure 1 shows the acceleration records measured on the ground surface (GSA) and underground (GSG) during the 2009 Mw 6.3 L'Aquila earthquake. The GSA station is at Assergi and the GSG station is located in an underground gallery. Both stations are founded on Eocene limestone with a shear wave velocity of 1 km/s. Although the epicentral distances and ground conditions are the same, the acceleration at ground surface is amplified almost 15 times that in the underground gallery.

The recent global positioning system (GPS) also showed that permanent deformations of the ground surface occur after each earthquake (Fig. 2). The permanent ground deformation may result from different causes such as faulting, slope failure, liquefaction and plastic deformation induced by ground shaking. Although this type of ground deformations will have limited effect on structures except the surface breaks pass beneath structures, they may cause tremendous forces on long and/or large structure. The ground deformation may induce large tensile or compression forces as well as bending stresses in structures depending upon the character of permanent ground deformations. Blind faults and folding processes may also induce some peculiar ground deformations. The ground deformations shown in Fig. 2 were caused by a blind fault and associated folding of soft overlaying sedimentary layers. The deformations caused tremendous damage on tunnels during the 2004 Chuetsu earthquake (JGS, 2004).

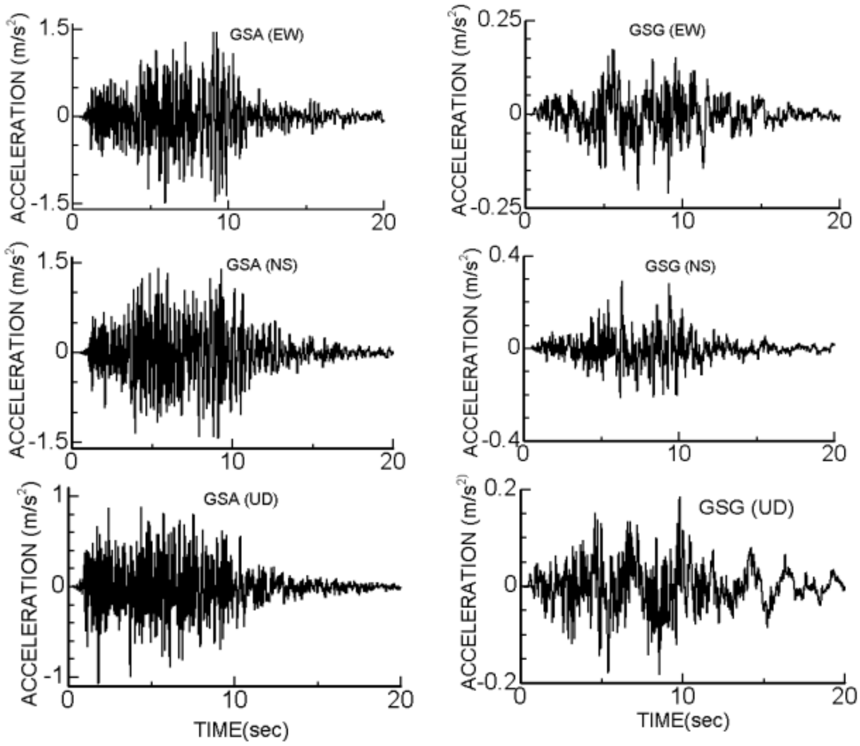


Fig. 1 - Acceleration records at GSA and GSG strong motion stations (Aydan et al., 2010)

3. DYNAMIC MODEL EXPERIMENTS ON UNDERGROUND OPENINGS

3.1 Underground Openings in Continuous Medium

A series of model tests on underground openings in a comparable granular material was carried out by varying the ratio of overburden to tunnel diameter and the inclination of ground surface. After compaction, the tunnel was excavated in a similar way to TBM excavation. Model tunnels were unsupported and some collapses from the roof occurred after the excavation. Then, tunnels were subjected to horizontal shaking perpendicular to longitudinal axis of the tunnels. Applied accelerations and movements of the ground surface were measured using laser displacement transducers. Figure 3 shows some views of the experiments on model tunnels. Figure 4 shows the applied acceleration wave and response of model tunnel for ground with sloping surface during shaking. Although the material is extremely weak, the maximum ground acceleration is greater than 0.6g. This simple example clearly implies that underground openings are quite resistant to ground shaking.

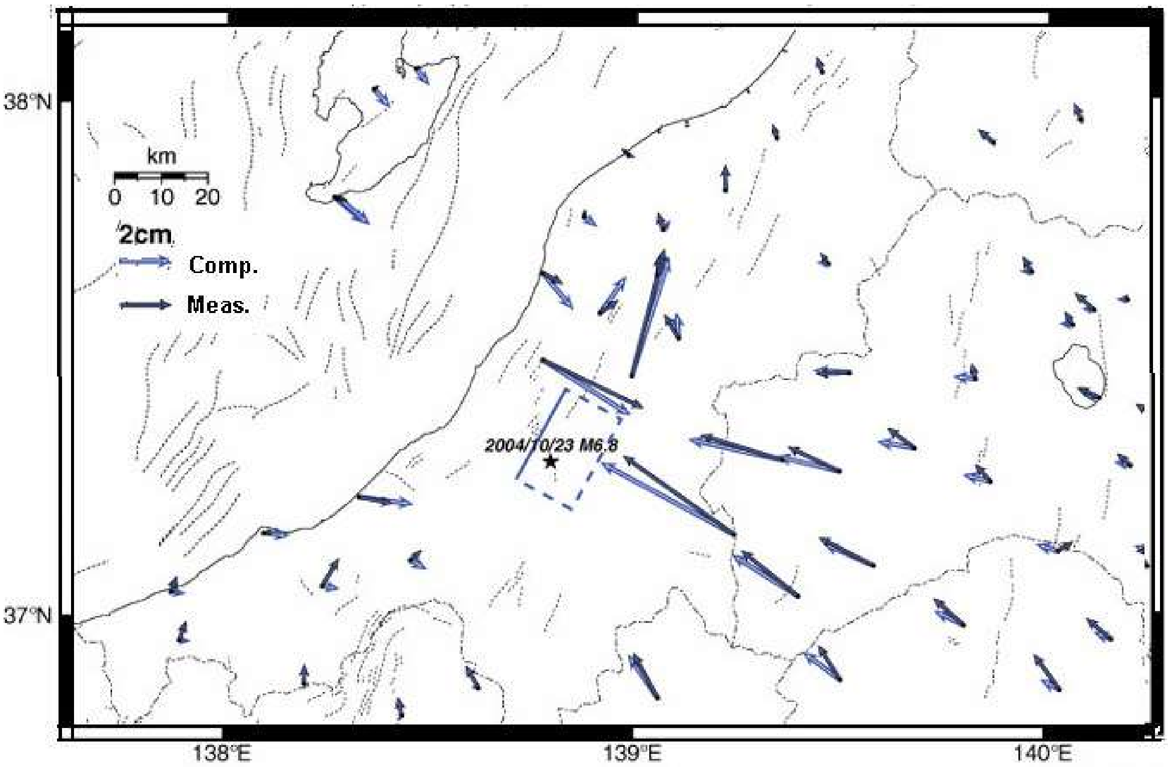
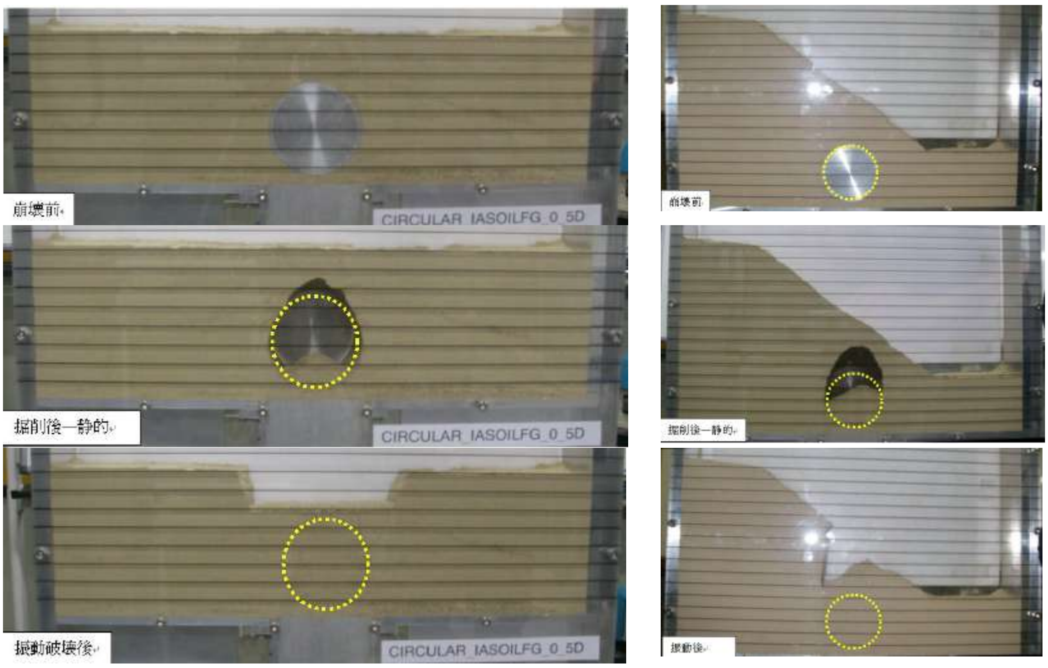


Fig. 2 - Permanent ground deformations induced by the 2004 Chuetsu earthquake



(a) Tunnel below flat ground (H/D=0.6) (b) Tunnel below sloping ground

Fig. 3 - Views of model tunnels before and after excavation and shaking

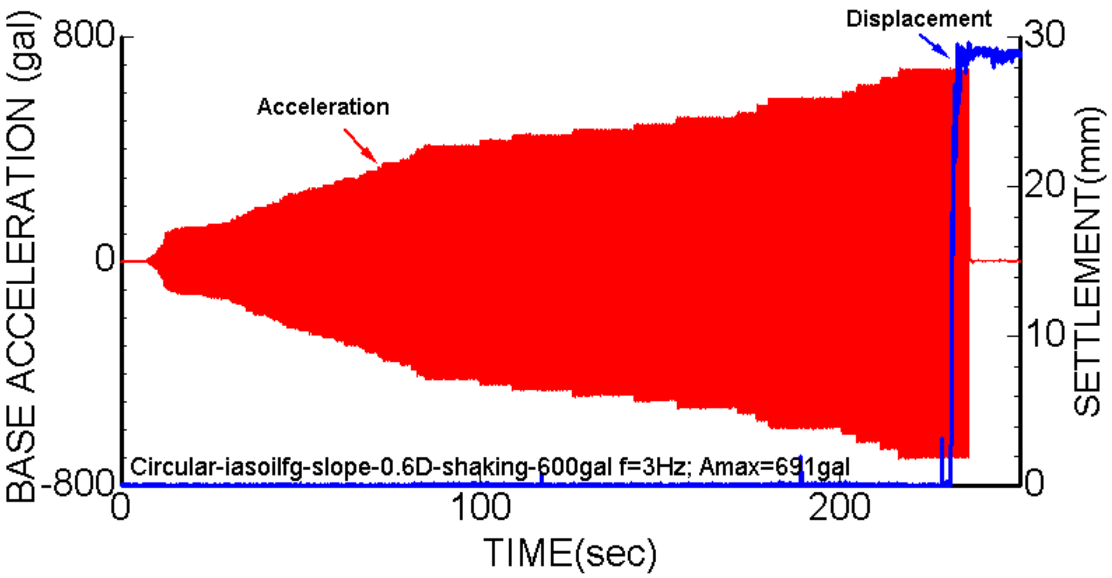


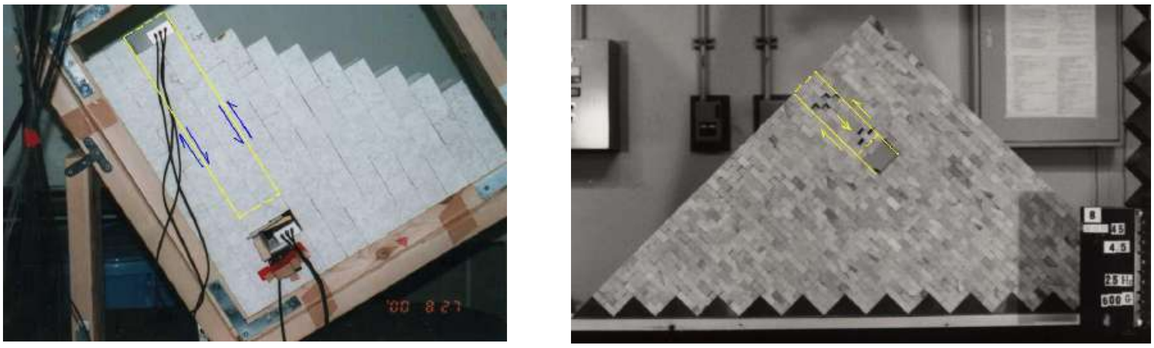
Fig. 4 - Displacement response of model tunnel next to sloping ground during shaking

3.2 Underground Openings in Discontinuous Medium

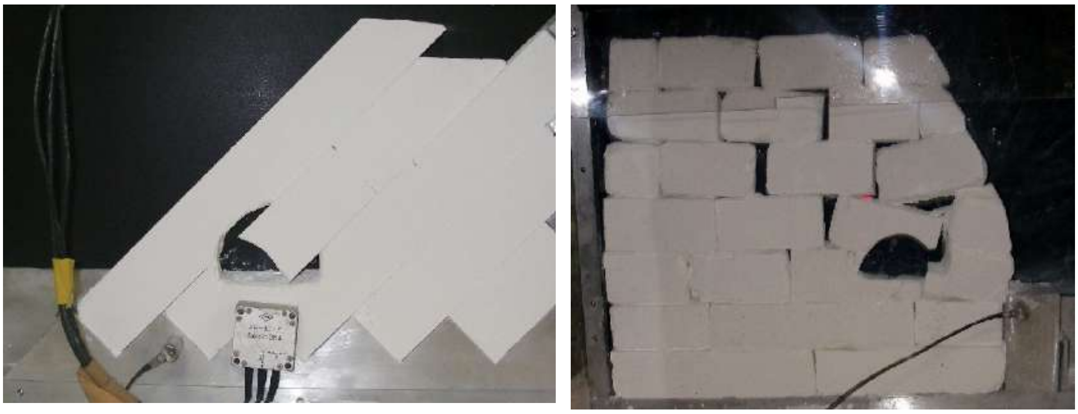
Aydan and his co-workers (Aydan et al., 1994; Aydan et al., 2007; Aydan and Genis, 2008; Aydan and Kawamoto 2004; Genis and Aydan, 2002, 2008) have reported some of their experimental results on the response and stability of underground structures in discontinuous medium. Some of these experiments involved model tests in non-breakable media. The instability merely takes place in relation to the orientation of discontinuities (Fig. 5(a)). In recent years, these experiments are repeated using breakable blocks (Fig. 5(b)).

Some laboratory model tests on abandoned lignite mines were carried out. The material used in modeling rock layers consisted of $BaSO_4$, ZnO and vaselin oil, which were used in base-friction model tests. The strength of this material depends upon the consolidation pressure so that it is possible to achieve the failure of models. The model is fixed on a uniaxial shaking table apparatus and shaked. In these particular model tests, the failure was due to the compression failure of pillars for the excavation ratio (A_t/A_p) of 2 (Fig.6a) while the failure was due to failure of the roof layers for the excavation ratio (A_t/A_p) of 3 as seen in Fig. 6(b). Here A_t is total supported area and A_p refers to pillar area.

In some of model tests, the effect of faulting was also investigated. Fig. 7 shows some views of a model test of an abandoned room and pillar mine in jointed rock mass induced by vertical normal faulting.



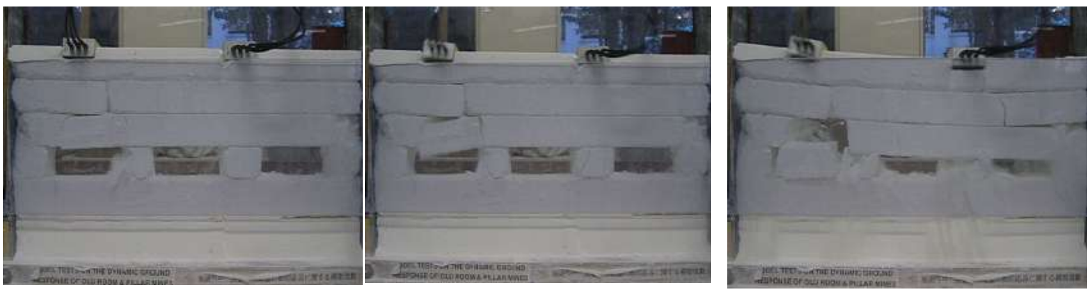
(a) Model tests using non-breakable blocks



(b) Model tests using breakable blocks

Fig. 5 - Views of some model tests

(a) Compressive failure of pillars



(b) Bending failure of roof layers

Fig. 6 - Some views of models of abandoned lignite mines during shaking table tests

Fig. 7 - Views of damage to an abandoned room and pillar mine by vertical normal faulting

4. NUMERICAL STUDIES

A series of parametric numerical analyses on the shape of underground openings under different high in-situ stress regime and direction and amplitude of earthquake induced acceleration waves were carried out. The details of these numerical analyses can be found in a publication by Genis and Gercek (2003). Figure 8 illustrates yield zone formations around a circular tunnel subjected to in-situ hydrostatic stress condition under static and dynamic conditions. Three different acceleration records were used in these particular analyses. If the maximum amplitudes of the earthquake records are same the yield zones formed under dynamic conditions are same and they are almost circular although the acceleration record was uni-directionally applied. The yield zone index (IYZ) is defined as the ratio of the area of yield zone to the area of the opening.

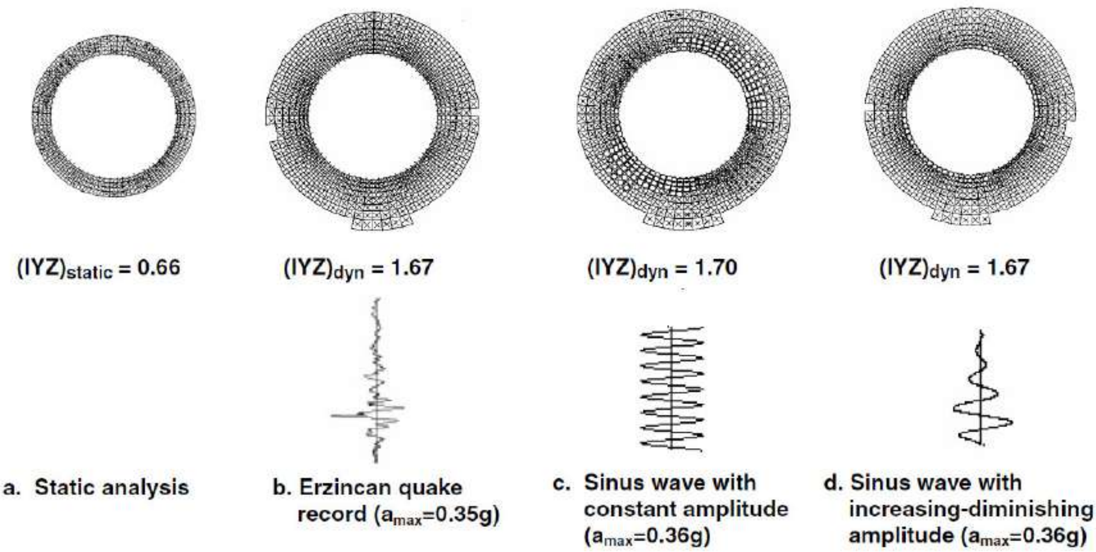


Fig. 8 - Yield zone formations around a deep circular opening under different waveforms (Genis and Gercek, 2003)

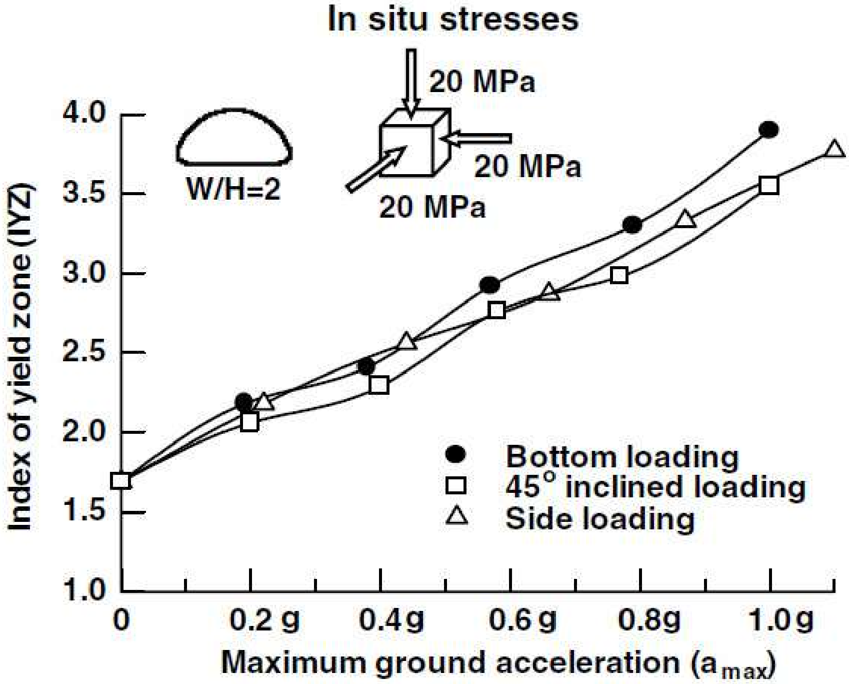


Fig. 9 - Effect of amplitude of ground acceleration and direction of wave propagation (Genis and Gercek, 2003).

Authors also investigated the effect of the direction of wave propagation on the geometry of the failure zone of an underground opening with aspect ratios (W/H) of 1 and 2 subjected to a hydrostatic in-situ stress state of 20 MPa. Applied acceleration waves were sinusoidal with a frequency of 1.5 Hz and its amplitude was varied out. The propagation direction of acceleration waves was vertical, 45^{o} and horizontal. Figure 9 illustrates the variation of yield zone index as a function the amplitude of acceleration waves for three different propagation directions. Under assumed conditions, the size of yield zone increased as the amplitude of acceleration waves increased and the results were almost unaffected by the direction of wave propagation. Furthermore, the aspect ratio of the opening was not so pronounced in terms of the size of yield zone. If the initial in-situ stresses are anisotropic, the shape and size of yield zone becomes different compared with those under hydrostatic in-situ stress conditions.

A series of numerical studies for the static and dynamic stability assessments of a large underground opening for a hydroelectric power house was carried out (see Genis and Aydan (2007) for details). The cavern is in granite under high initial stress condition and approximately 550 m below ground surface and the area experienced the largest inland earthquake with a magnitude of 8 in 1891 in Japan. In the numerical analyses, the amplitude, frequency content and propagation direction of waves were varied (Fig. 10). The numerical analyses indicated that the yield zone formation is frequency and amplitude dependent. Furthermore, the direction of wave propagation has also large influence on the yield zone formation around the cavern. When maximum ground acceleration exceeds 0.6-0.7g, it results in the increase of plastic zones around the opening. Thus, there will be no additional yield zone around the cavern if the maximum ground acceleration is less than this threshold value.

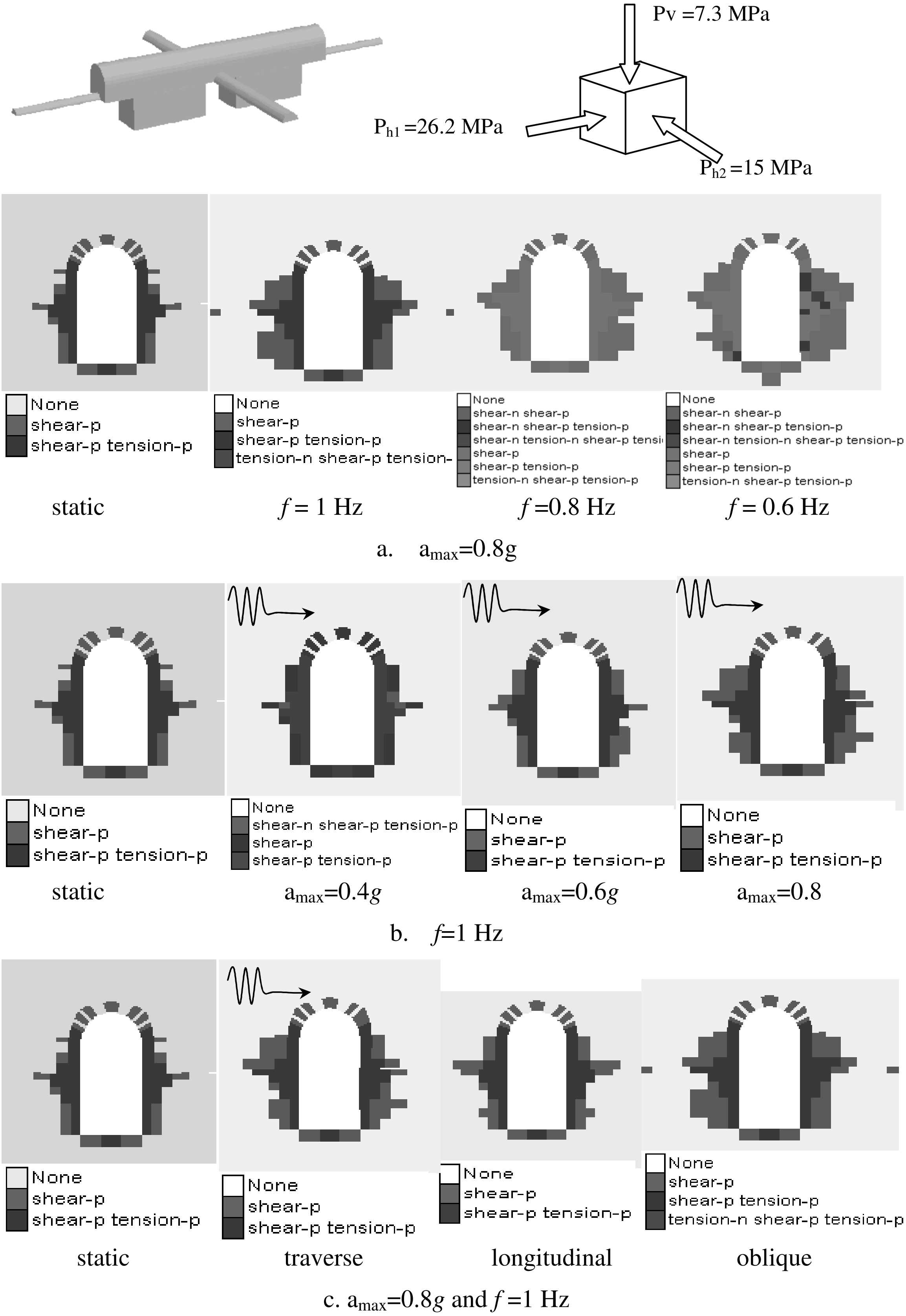
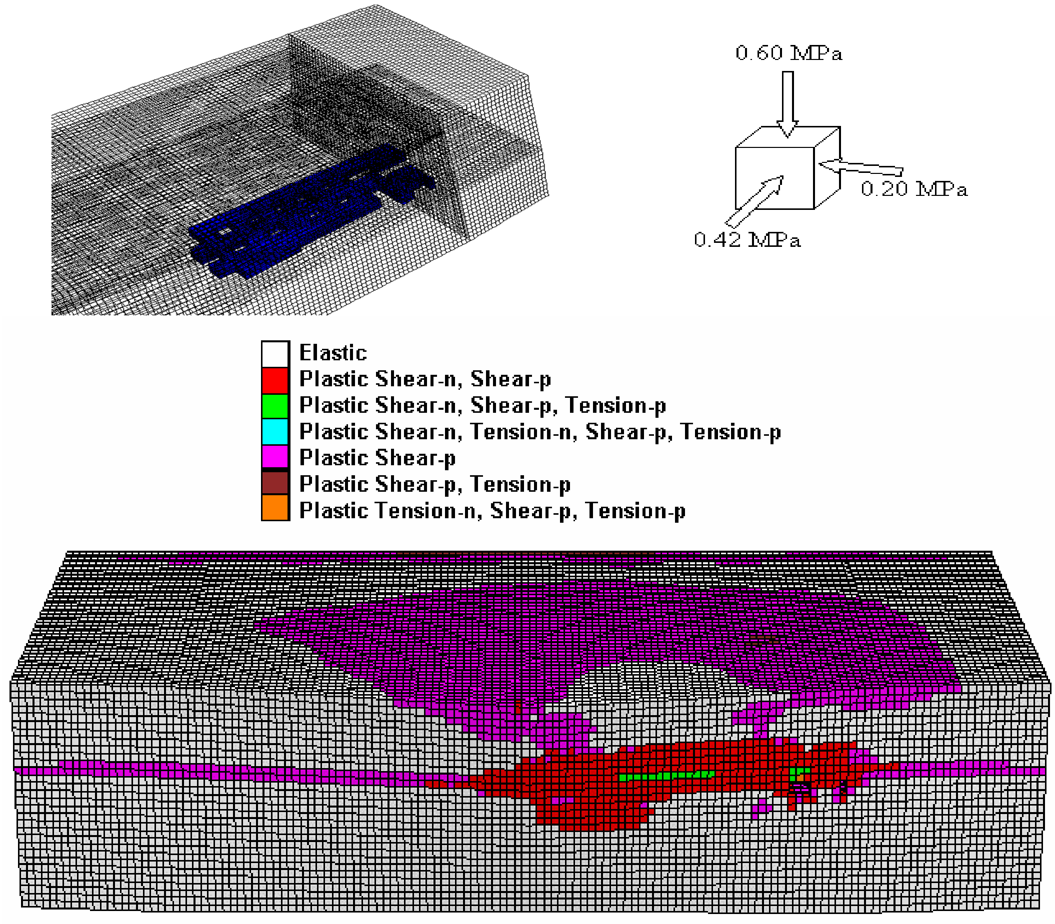
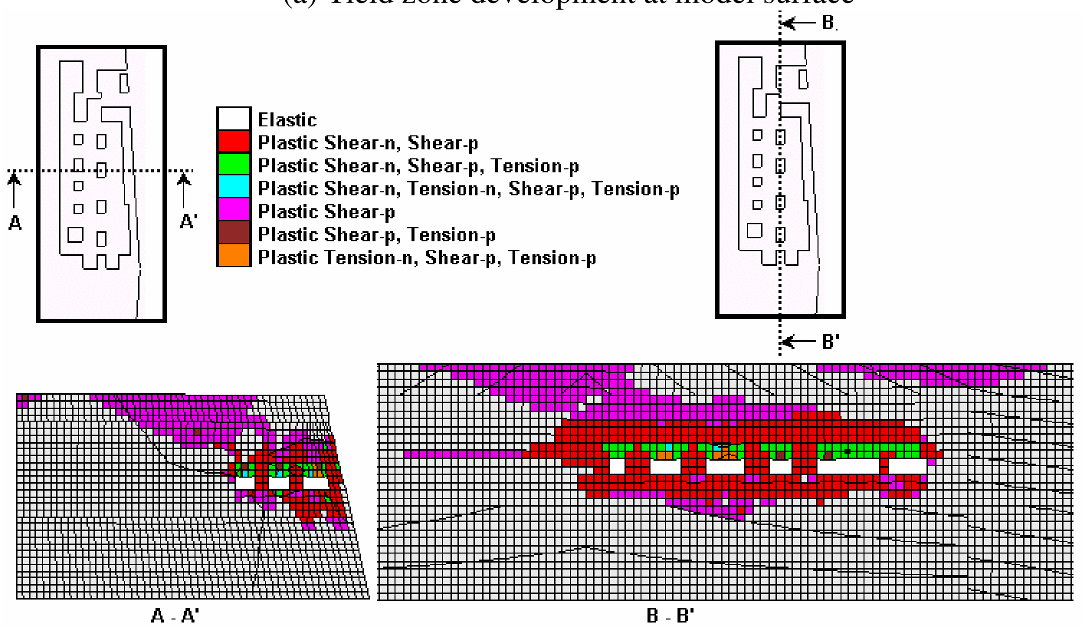


Fig. 10 - Yield zone formation of underground power house for different cases of input ground motions (Genis and Aydan, 2007)



(a) Yield zone development at model surface



(b) Yield zone development at EW and NS cross-sections.

Fig. 11 - Yield zone around mine for ground with reduced strength properties (Genis and Aydan, 2008).

The elasto-plastic dynamic response of abandoned lignite mine is chosen as an actual example and its dynamic response and stability are analyzed (see Genis and Aydan, 2008 for details). Figure 11 shows a three-dimensional perspective view of the mine and in-situ stress state inferred from the fault striations using Aydan's method (Aydan, 2000). The input ground motion is assumed to be sinusoidal with a chosen period and various situations were analyzed. A computation was carried out for the case, in which the properties of rock mass reduced to 1/8 of those of the intact rock while the ground acceleration amplitude was assumed to be 80 gals. This situation will correspond to a state of degradation of rock mass properties with time.

Figure 11 shows the yield zone development at the model surface and at EW and NS sections. This situation probably corresponds to a state of the total collapse of the abandoned mine. As seen in Fig. 11, the yielding at ground surface extends to a large area and the yielding in EW (A-A') cross section resembles to a slope failure containing the abandoned mine.

5. EARTHQUAKE INDUCED DAMAGE OF UNDERGROUND STRUCTURES

In this section, damage to underground structures since 2006 is summarized. The main aim is to provide an overview of types, patterns and causes of damage during earthquakes.

5.1 San Francisco Earthquake - USA (1906)

Wrights tunnel is the first example of damage to tunnels by the earthquakes and it is associated with the 1906 San Francisco earthquake (Mw 7.9, where Mw is moment magnitude). A right-lateral displacement of 1.7-1.8 m occurred along the San Andreas shear zone intersecting the Wrights tunnel about 120 m from the northeast portal (Fig. 12). Damage in the vicinity of the maximum offset included crushing of timber supports, heaving of rails, and rock fall. Some railroad ties were broken (Prentice and Ponti (1997).

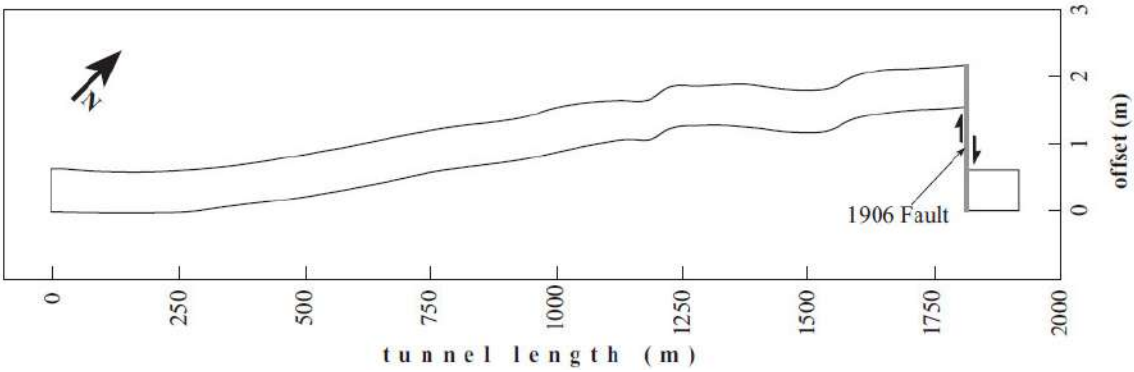


Fig. 12 - Deformation of Wrights tunnel during the 1906 San Francisco earthquake

5.2 Kern Earthquake - USA (1952)

The Kern County earthquake with magnitude on Richter scale (M) = 7.7 (Moment magnitude, Mw = 7.5) seriously damaged three tunnels. The epicentral distance was 47 km. The tunnels have shallow overburden in the range of 15 to 75 m and are close to the side of the valley. In one of the tunnels (overburden 38 m), a 1.22 m displacement was found, while there was a fracture with practically no vertical offset directly above the tunnel (Rozen, 1976).

5.3 San Fernando Earthquake - USA (1971)

The San Fernando earthquake of M = 6.6 in 1971 activated the Santa Susana thrust fault, which crossed the Balboa inlet tunnel about 300 m from the portal. The epicenter was at a distance of about 16 km from tunnel. The reinforced concrete liner spalled along a 90 m section at the fault crossing and longitudinally cracked the liner for some 300 m on each side of the fault (Rozen, 1976). The tunnel was under construction in sedimentary rock when the earthquake hit. The overburden was around 5m.

5.4 Great Kanto Earthquake - Japan (1923)

The Great Kanto earthquake with a moment magnitude of 7.9 occurred on September 1, 1923. This earthquake was the most disastrous earthquake in the seismic history of Japan. Although the epicenter of the earthquake was in Sagami Bay , Tokyo, Yokohama and prefectures of Kanagawa, Shizuoka and Chiba were greatly affected by the earthquake. 93 tunnels were affected and 25 of them received various levels of repairs (Okamoto, 1973; Yashiro et al., 2007). The tunnels were either single or double track tunnels and most of them were lined using brick liners and they were all situated on the hanging wall of the earthquake fault, which dip into NE with an inclination of about 30^o. The damage to tunnels and their portals occurred mainly by the intense ground shaking and the degree of the damage was very high in the vicinity of epicentral area (Fig. 13). The complete collapse of the tunnels were observed when the ground conditions were relatively poor.


(a) Yose Tunnel


(b) Nenokamiyama Tunnel


(c) Fudousan Tunnel

Fig. 13 - Examples of damaged tunnels by the 1923 Kanto earthquake (JSCE Archive)

5.5 North-Izu Earthquake-Japan (1930)

The 7.8km-long Tanna Tunnel between Hakone and Tanzawa was under construction when the 1930 North-Izu earthquake (M=7.2) occurred. The tunnel was crossing the fault-fracture zone beneath the Tanna basin where it encountered with serious problem of water inflow through fault zone and many drainage drifts were bored. The epicentral distance of the tunnel was 15 km while the distance to the causative fault was 0 km. The overburden of the tunnel at fault crossing was 160m and the width of fracture zone was about 45m consisting of sandy clay lake deposits overlying lower Pleistocene tuff, agglomerate and lava flows (Kuno, 1935). Near the heading of one of the drainage tunnels, an existing shear zone displaced by the earthquake and the drainage drift was completely closed (Figure 14). However, only a few cracks appeared in the main tunnel walls, in spite of the fact that the main tunnel heading was only 0.5 m east of the active shear zone (Sakurai, 1999).

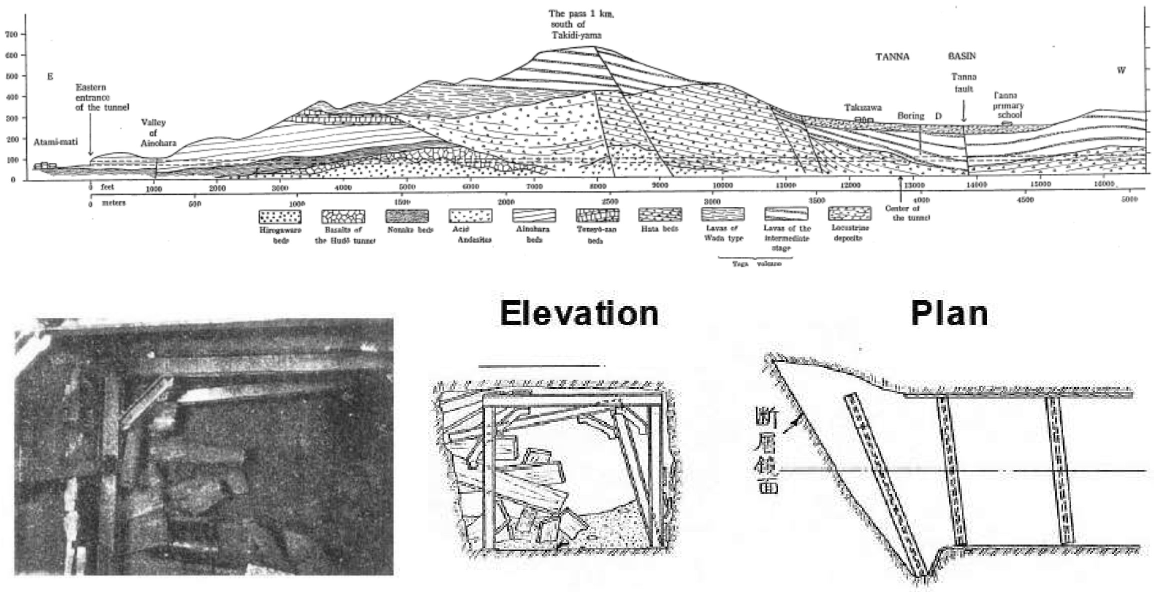


Fig. 14 - Geology of Tanna tunnel and damage at tunnel heading by the earthquake (arranged from Kuno (1935) and Sakurai (1999))

5.6 Izu-Oshima Earthquake-Japan (1970)

The 1970 Izu-Oshima earthquake (M=7.0) damaged the Inatori tunnel, which is a 6 m diameter 906 m long single-track railway tunnel. Overburden is about 100 m at the most. The tunnel was excavated through a deposit of volcanic mudflow deposits that have been hydrothermally altered to clay. The tunnel was supported with 0.7 m thick concrete lining. The centre line of the tunnel was displaced by 50-70 cm within a zone of 20 m at 440 m from the tunnel portal of the Ito-side where a fault segment crossed the tunnel (Fig. 15). Cracking and deformation occurred in a zone up to 300-400 m from the main fault rupture (Tsuneishi et al. 1978; Kawakami, 1984). The northeastern half of the tunnel was displaced about 0.85 m seaside (east); while its western part was displaced about 0.2 m towards mountain side (west). The tunnel was seriously damaged at location where the tunnel was cut by the fault.

5.7 Kobe Earthquake – Japan (1995)

The 1995 Kobe earthquake (Hanshin-Awaji) (Moment by Japan meterological agency Mj = 7.2 and moment magnitude Mw = 6.9) caused some damage to tunnels (Asakura et al., 1996; Asakura and Sato, 1998). Typical damage patterns were cracking in the lining, spalling of concrete in the arch and the sidewalls, expansion of existing cracks, heave and cracking of the invert, settlement of the arch crown, pounding of construction joints, collapse of portals. The closest tunnel, the Maiko tunnel under construction, with an epicentral distance of 4 km received only slight damage while a four-story building on the surface on top of the tunnel was completely destroyed. The 16km Rokko tunnel excavated in granitic rock was severely damaged by the earthquake. Damage was mostly observed at the location of faults with poor rock conditions and small overburden. This tunnel is crossed by five major fault of the dextral Rokko fault zone. The inferred relative movements are expected to range between 10-30cm while the maximum relative dextral motion along Nojima segment was more than 180cm. Huge chunks of concrete lining were fallen over the rails (Fig. 16). As the earthquake occurred before the start of shinkansen trains (running), no major secondary disasters was caused.

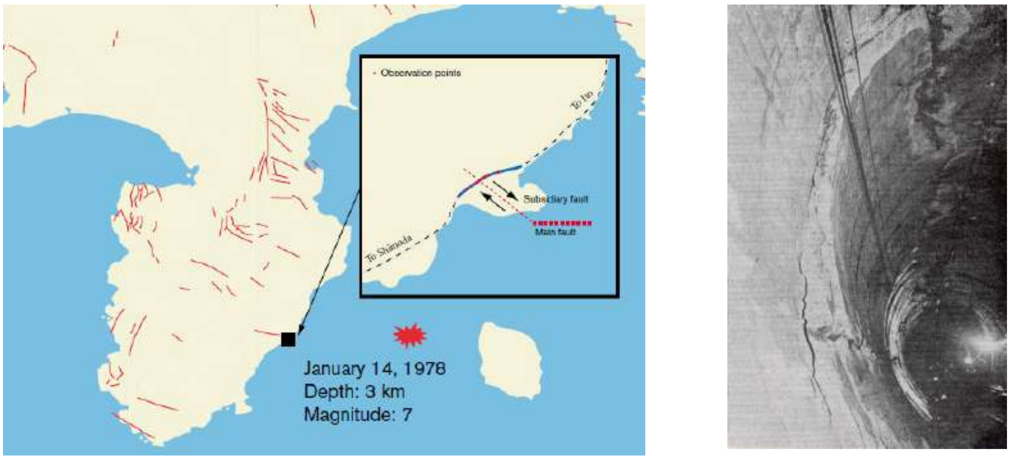


Fig. 15 - Damage to Inatori tunnel by 1970 Izu-Ohshima earthquake

The 1766m long Bantaki tunnel was excavated through fractured granite and spalling of concrete lining and buckling of the reinforcement bars were observed at the location where the tunnel passed through fault zone. The lining was reinforced at this section due to difficulties during the construction. Further, the tunnel invert was heaved by 15cm. It is pointed out that this tunnel is aligned with Otsuki fault and a building to the NE side of this tunnel collapsed by the earthquake.

5.8 Iwate Earthquake - Japan (1998)

1998 Iwate earthquake (M = 6.1) caused damage to tunnels of Kakkonda hydroelectric power plant (Hashimoto et al., 1999). The averaged upward movement by the reverse faulting was 10 to 20 cm. A concrete outlet tunnel crossing the fault was ruptured and the surrounding soil intruded into the tunnel. The tunnel was damaged and had cracks extending to a length of 7 m.

5.9 Western Tottori Earthquake-Japan (2000)

The Western Tottori earthquake with a magnitude M=7.3 occurred on October 6, 2000.

One headrace tunnel located 200 m below the surface for a hydropower plant intersects the earthquake fault and damage was mostly confined to the area where the tunnel intersected the fault with left-lateral deformation of 10-20 cm (Ueta et al., 2001).

(a) Cracking in the roof

(b) Side wall spalling

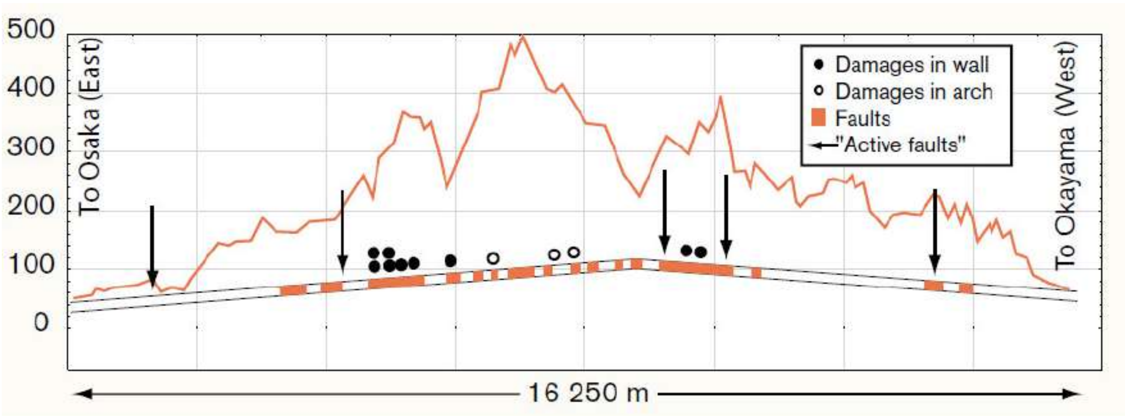


Fig.16 - Damage at the Rokko tunnel by the 1995 Kobe earthquake (Asakura and Sato, 1998)

5.10 Miyagi-hokubu Earthquake - Japan (2003)

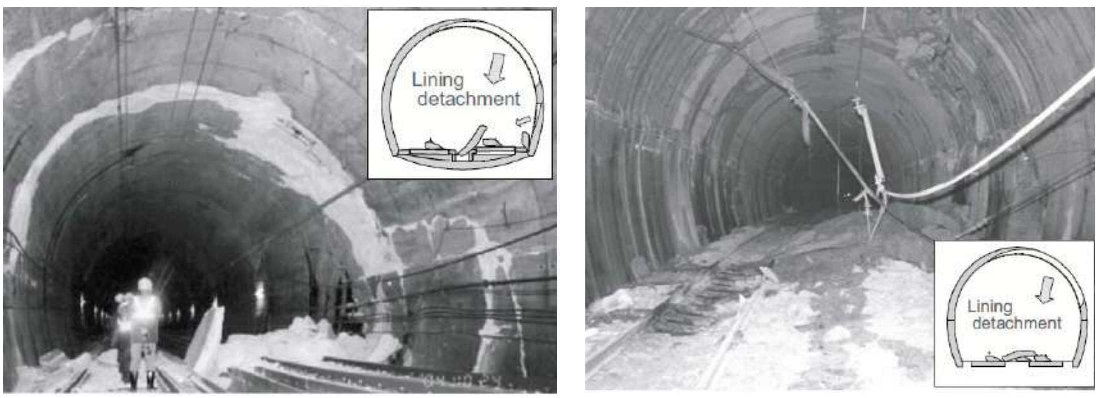
The 2003 Miyagi-Hokubu earthquake caused damage of abandoned mines in the vicinity of Yamato town although the magnitude of the earthquake was 6.2. The damage surveys on abandoned lignite surface are restricted to surface damage. Although there is no doubt that some damage in the form of roof collapse and pillar failure may take place, it is very difficult to observe those damage due to the limitation of access to old workings. The local authorities (Yamoto town) identified 28 locations where the surface damage was observed (Aydan and Kawamoto, 2004). The damages were in the form of caving (sink-hole) and water discharge. Figure 17 shows some examples of damage observed. Some sandy material was ejected due to the sloshing of the ground water in the abandoned-lignite mines as a result of ground shaking caused by the earthquake.

The local people showed that some wells adjacent to damaged locations had very high water level, implying that the abandoned-mine workings were fully submerged with ground water. Further, it is reported that the water discharge from caving locations continued for a considerable period of time. This also implies that the motion of the ground water may breach submerged old mining workings at higher elevations and flow towards the old mine workings at lower elevations since such events generally took place at locations with lower elevations.

Fig.17- Caving and sand-boiling.(Aydan and Kawamoto, 2004)

5.11 Niigata-ken Chuetsu Earthquake - Japan (2004)

The Niigata-ken Chuetsu earthquake occurred on October 23, 2004 and it had a magnitude Mj = 6.8 on the magnitude scale of Japan Meteorological Agency. The earthquake caused various levels of damage to 24 tunnels and five of them required repair and reinforcement measures (Yashiro et al., 2007). The most heavily damaged tunnels were Uonuma Shinkansen tunnel, Myoken Shinkansen railway tunnel and Wanatsu local tunnel and they were in the area of 10 km from the epicenter. Uonuma tunnel is 8625 m long and constructed in Neogenic mudstone. The tunnel is about 1km away from the earthquake epicenter. Concrete lining was ruptured and had fallen on the railway tracks (Fig. 18). The damage was concentrated at fracture zones observed during the construction where overburden was less than 50m. The concrete lining at the roof and spring level were ruptured and had fallen down. The JR Wanatsu tunnel was damaged by the earthquake for a length of 40 m.



(a) Uonuma tunnel (b) Wanatsu tunnel

Fig.18 - Damage to some of tunnels by the 2004 Chuetsu earthquake (Yashiro et al. 2007)

The 10 roadway tunnels were damaged by the 2004 Chuetsu earthquake (TEC-JSCE, 2005). Wanatsu tunnel is a double-lane 300m long tunnel excavated in silty sandstone. About 100m away from Nagaoka side portal the concrete lining had fallen in a length of 20m. The tunnel axis is parallel to the strike of the earthquake fault and there is contraction at the bottom of the tunnel.

5.12 Düzce Earthquake (1999)

The epicentre of the 1999 Düzce earthquake (M_w = 7.2) earthquake was approximately 20 km from Asarsu portals of the Bolu tunnels. The surface rupture of the fault was within 3 km of these portals. In the main Asarsu tunnels within the metasediments, slight-moderate damage was observed as slabbing and spalling of shotcrete, and also longitudinal cracking and deformation of the potentially weak rock.. In the Asarsu Left Tunnel-Bench Pilot Tunnels (BPT) in fault gouge clay, moderate to severe damage was observed. Invert heave of 0.5 to 1.0 m occurred, together with damage to the shotcrete arch lining comprising of shotcrete/concrete slabbing, spalling, compression crushing and associated steel rib buckling.

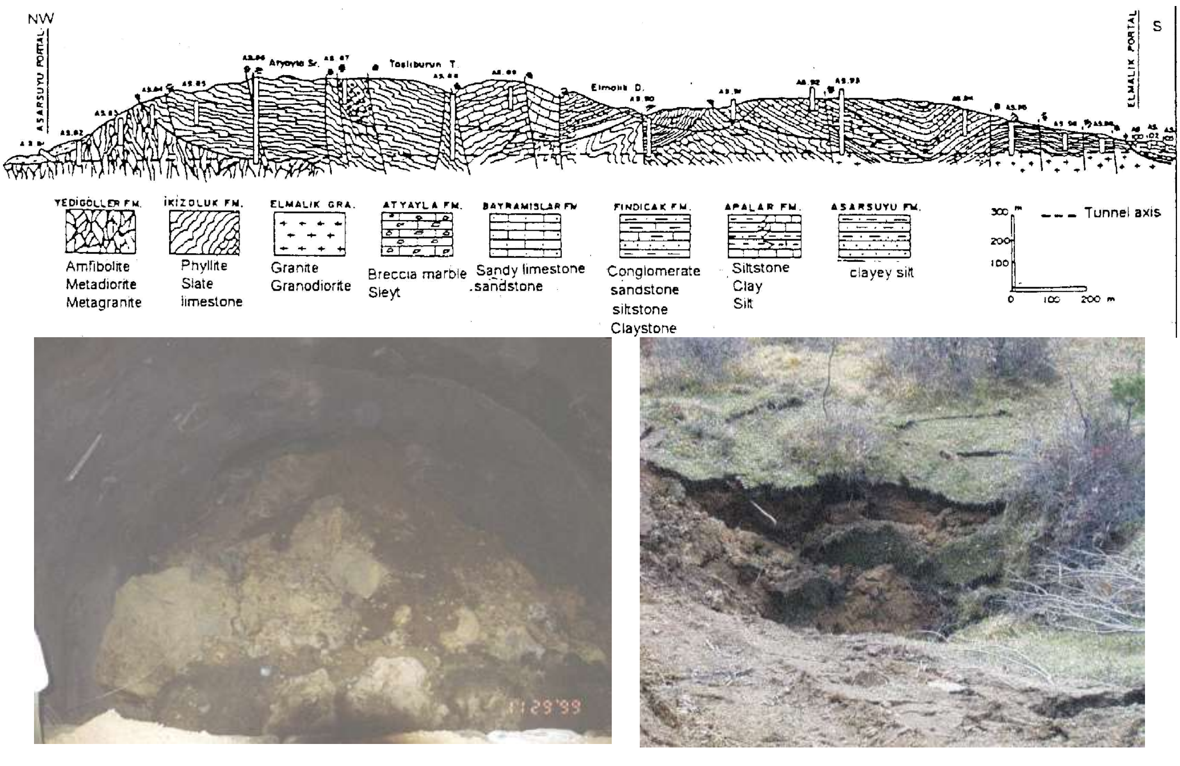


Fig.19 - Geology and views of tunnel and surface depression

Severe damage in the form of complete collapse of two parallel 16.5 m diameter main tunnels occurred during the earthquake near the southern Elmalik portals, where heavy squeezing problem was encountered during excavation (Fig. 19). The collapses caused the complete closure of the tunnels and sinkholes occurred on the ground surface (Aydan et al. 2000). The collapsed sections had no concrete lining yet and support system consisted of shotcrete and rockbolts.

5.13 Chi-chi Earthquake (1999)

The 1999 Chi-chi earthquake (M_w = 7.6) struck central Taiwan on September 21, 1999. The depth of the hypocenter was around 7.5 km and the epicentre close to the Chi-Chi town 12 km to the west of the Sun Moon lake. The shaking during the earthquake was extremely strong. Two locations for free-field strong motion instruments experienced more than 1g of horizontal shaking and at several other locations more than 0.8 g. The earthquake generated surface rupture along approximately 96 km of the pre-existing Che-Lung-Pu fault. The thrust fault dipped 25^o-30^o toward the east, with the hanging wall uplifted, ranging from 1 m in the south and 9 m in the north.

The damaged tunnels are mostly located on the hanging wall of the fault and constructed in soft rock (Wang et al., 2001). The maximum surface ground acceleration was more than 0.45 g in the area where most of the damaged tunnels are located. One of the tunnels, the Shi-Kang Dam water intake tunnel was the only tunnel that intersected the Che-Lung-Pu fault. The tunnel was completely closed due to the 3.5 m vertical and 3 m horizontal displacement. The tunnel is 4.1 m high and 3.8m wide and it has an ovaloid cross-section.

Various types of damage were observed, including lining cracks, portal failure, concrete lining spalling, groundwater inrush, exposed and buckled reinforcement, displaced lining, rockfalls in unlined sections, lining collapses caused by slope failure, pavement or bottom cracks, and sheared off lining. Out of 50 tunnels, 13 tunnels were severely damaged by the earthquake while 1 tunnel out of 6 tunnels was severely damaged by the earthquake. Further, there was portal damage to 26 tunnels on the hanging wall side of the earthquake fault. The pillar part of the Chi-Shue tunnel next to steep slope was chopped away due to slope failure. The most severely damaged tunnel sections in the hanging wall are those close to surface slopes or portal openings, while sections with a thick overburden generally suffered less.

5.14 Zirkuh Earthquake (1997) and Zarand Earthquake (1998)

The qanats (known also as *kareze* in Turkistan) are underground conduits, which collect the water from an aquifer on the slope of a hill and exploit the natural gradient of the land to transport the water underground to the agricultural areas below. The historical records indicate that earthquake may damage or cause the collapse of the tunnels and access wells of qanats. Due to partial or complete collapse of the qanats tunnels, water flow can be affected in different levels and land subsidence in form of sinkholes can occur on the ground surface. Damage to the qanats has been reported in several seismic events in Iran as well as in neighboring countries. Berberian et al. (1999) recently reported that 250 qanats together with 20 deep wells were damaged during 1997 Zirkuh earthquake. It is also reported that 25 out of 64 qanats in the region were damaged during the 1998 Zarand Earthquake.

5.15 Bam Earthquake (2003)

The 2003 Bam earthquake with a magnitude of 6.5 occurred near the city of Bam that is located at southeast of Iran. The maximum ground accelerations for the horizontal components are 0.7 and 0.8g, and 1.01g for the vertical component in Bam. Before Bam earthquake there were about 126 active qanats in the area to supply 50% of the required water for the city. The rest of the required water supplied with deep wells. In addition there are several trends of old qanats related to the past decades or centuries that their locations are unknown now. Most of these old qanats are now dry and partially collapsed. Most of the qanats of the area have been damaged due to Bam earthquake (Eshgi and Zare, 2003). In some cases the collapse of some of theses qanats caused severe damages to the building and lifelines. Based on the preliminary evaluations, about 40% of these qanats have collapsed or experienced severe damages due to the Bam earthquake. In some cases the collapse of the qanats stopped the water flow completely. Most of these qanats have been supported with hand made arches called "Kaval". Qanats in Bam and its vicinity experienced severe damages during the earthquake. The most important effects of Bam earthquake on qanat systems are damages to the access wells and tunnels. The earthquake caused several sinkholes above tunnels and collapse of shafts. Most of the damages were observed at the access wells and tunnels near the Bam fault. Most of the (occurred) collapses were also observed in a narrow band close to the Bam fault. When qanats were far from the Bam fault, the effects of earthquake on qanats were negligible. Only some fissures and cracks can be observed along the tunnels of access wells.

5.16 Kashmir Earthquake (2005)

The only tunnel in the epicentral area of 2005 Kashmir earthquake was located about 4km south of Muzaffarabad (Aydan et al., 2009a). The tunnel is lined with stone masonry lining and it is excavated in shale with an overlaying conglomeratic deposit just above the tunnel crown. The south portal of the tunnel was lightly damaged by the slope failure of overlaying conglomeratic deposit as seen in Fig. 20.

Fig. 20 - Lightly damaged roadway tunnel

5.17 Wenchuan Earthquake (2008)

There are many roadway and railway tunnels in the epicentral area of the 2008 Wenchuan earthquake (Aydan et al., 2009b). Some of them are under construction. It is well known that the underground structures are earthquake-resistant compared to surface structures when ground shaking is concerned. The damage to tunnels is generally limited to portals, which are mainly caused by slope failures and rockfalls. Several examples of portal damage are shown in Fig. 21. Similar damage to portals of railway tunnels was also observed. Nevertheless, such damage is light in scale and did not cause any major structural stability problems so far.

Fig. 21 - Damage to tunnel portals

The authors had the chance to visit five tunnels, three of which were under construction and suffered heavy damage during the earthquake. The tunnel nearby Zipingpu dam (N31.04888, E103.56463) is a two-lane tunnel with a length of about 1000m. The lining was damaged at several locations perpendicular to its longitudinal axis, which is aligned NW-SE direction. This tunnel was already repaired at the time of the investigation (June 21, 2008) (Fig. 22). The tunnel is about 18.5km away from the earthquake epicenter and it is very close to the Yingxiu-Beichuan fault. The tunnel floor was stepped at the damaged zones implying that the NW side is moved upward. Since this earthquake is of great scale and its relative slip was presumed to be about 7m, it is very natural to expect a diluted deformation zone as the fault approaches the ground surface due to the reduction of vertical stress. The damaged section of the tunnel wass rebolted and shotcreted.

Fig. 22 - Views of repaired damaged section of the tunnel near the Zipingpu dam

Longxi tunnel is a part of Wenchuan-Dujiangang expressway. Two double laned tunnels are excavated with a 3D wide pillar. The maximum overburden is about 480m and the maximum in-situ stress measured is about 26 MPa. The fault named as F8 is 900m from the SE portal and its strike is perpendicular to the longitudinal axis of the tunnel. The engineers informed the authors that they had difficulty of maintaining the stability and controlling the deformation of the tunnel during excavation, indicating that the tunnel experienced squeezing phenomenon. The tunnel was excavated using the rockbolts and shotcrete as an initial support and then the unreinforced concrete lining was constructed. The concrete lining was ruptured and the crown concrete had fallen down (Fig. 23). However, the overall rupture trace dips NW. The floor of the tunnel was also ruptured and uplifted by buckling. The tunnel collapsed at the location where the overburden is about 70m. The collapse may have created a sinkhole at the ground surface, which should be checked. The similar type of the collapse occurred at the Elmalik section of Bolu Tunnel during the 1999 Duzce-Bolu earthquake (Aydan et al. 2000). The ground conditions at both tunnels were remarkably similar to each other. The engineers still had not checked the zone beyond the collapsed section of the tunnel. The fundamental causes of the damage to this tunnel are probably the permanent ground deformations along the fault zones as well as high ground shaking.

Figure 23. Views of earthquake damage at Longxi (Longqi) Tunnel

Jiujiaya Tunnel is a 2282m long double lane tunnel and the coordinate of its south portal is N32.41288 and E105.12046. The tunnel is 226.6km away from the earthquake epicenter and it is about 3-5km away from the earthquake fault of the Wenchuan earthquake. The tunnel face was 983m from the south portal at the time of the earthquake (Figure 24). The concrete lining follows the tunnel face at a distance of approximately 30m. The tunnel suffers from methane gas problem due to coal seams below the tunnel. The rock consists of phyllite with intercalations of dolomite. The bedding planes have the orientation of NW50/64, which is roughly similar to that of the Longmenshan fault zone. The geological investigation indicated that the tunnel passes through several fracture zones. The engineers informed the authors that 30 workers were working at the tunnel face and one worker was killed by the flying pieces of rockbolts, shotcrete and bearing plates which were caused by intense deformation of the tunnel face during the earthquake. The concrete lining was ruptured and had fallen down at several section (Figure 24). However, the effect of the unreinforced concrete lining rupturing was quite large and intense in the vicinity of the tunnel face. The rupturing of the concrete lining generally occurred at the crown sections although there was rupturing along the shoulders of the tunnel at several places. Furthermore, the invert was uplifted due to buckling at the middle sections. Sometimes, the whole invert was pushed upward. The authors noticed that the tunnel heading was offset in a dextral sense with an upward movement with respect to the rest of the tunnel. The lateral and vertical offset displacements are approximately 100mm. This sense of deformation is very similar to that of the Longmenshan fault zone, which probably cuts through the tunnel with an acute angle of 50-60^{o}. Besides the high ground shaking, the permanent ground deformations definitely played an important role on the damage observed in Jiujiaya tunnel.

Figure 24. Views of damage at Jiujiaya Tunnel

5.18 L'Aquila Earthquake - Italy (2009)

The l'Aquila earthquake caused a number of sinkholes in l'Aquila (Figure 25). One of the sinkholes was well publicized worldwide (Aydan et al., 2010). The width was about 10 m and its depth is not well-known. A car had fallen into it. Another sinkhole had a slightly smaller size. Its length and width were about 8 m and 7 m, respectively. The depth was about 10 m. The layers between the roof and road level were breccia with calcareous cementation, breccia with clayey matrix and top soil from bottom to top. A side trench was excavated to a depth of 3 to 4 m from the ground surface on the south side of the sinkhole. The reconnaissance team of the GEER from USA also reported that a sinkhole occurred in Castelnuovo and it had a length and width of 5 and 3 m respectively, with a depth of 5 m. The roof thickness was about 1.5-2.5 m.

Fig. 25 - Sinkholes in L'Aquila (from Aydan et al. 2010)

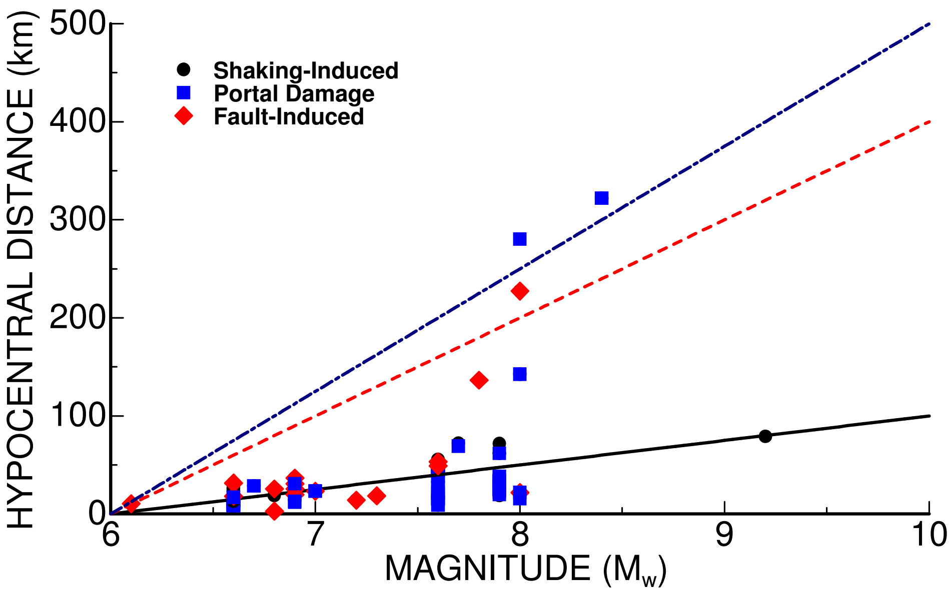


Fig. 26 - Empirical relations between magnitude and limiting damage distance

6. DISCUSSIONS AND CONCLUSIONS

The damage to underground structures excavated in rock can be induced depending upon the permanent ground deformations such as faulting or slope movements and ground shaking. The authors tried to compile a series of data-bases for earthquake induced damage of underground structures in rock mass with the consideration of ground shaking, faulting and slope movements. Because of the lack of space, the outcomes of the processing of these databases are briefly presented herein. Figure 26 shows the relation between the moment magnitude of the earthquake and hypocentral distance of the underground structure. As expected, the hypocentral distance of the damaged underground structures increases as the magnitude of the earthquake becomes larger. Furthermore, the limiting relations for fault-induced, ground shaking induced damage on tunnels would be different. The limiting distance of damage for portals of underground openings would be more (far-distant) and the relation proposed by Aydan (Aydan, 2007; Aydan et al., 2009a) for slope failure may be used for this purpose.

Table 1 - Earthquake-induced damage level index (DLI) for underground structures with the consideration of support members

Damage Level Index (DLI)	Remarks
1	No cracking of concrete lining, shotcrete, no deformation of rockbolts and steel ribs, no heaving of invert
2	Hair cracking of concrete lining, shotcrete, none noticeable deformation of rockbolt platens and steel ribs, no heaving of invert
3	Visible cracking of concrete lining, shotcrete, noticeable deformation of rockbolt platens and steel ribs, slight heaving of invert
4	Exfoliation of concrete lining, shotcrete, noticeable bending of rockbolt platens and steel ribs, heaving of invert. Nevertheless structurally is stable
5	Spalling of concrete lining, shotcrete, and considerable deformation of rockbolt platens and bending of steel ribs, heaving of invert. Structurally problematic and require repairs and reinforcement
6	Collapse of concrete lining, shotcrete, and extreme deformation of rockbolt platens and rupturing rockbolts and buckling of steel ribs, buckling and rupturing of invert. Collapse of blocks of ground from roof and shoulders. It is structurally unstable and requires immediate repairs and reinforcement
7	Complete closure of the section by failed surrounding ground. Crushing of concrete lining and shotcrete, rupturing of rockbolts and twisted steel ribs and extreme heaving of invert. Underground openings either to be abandoned or re-excavated with extreme precautions.

The definitions of structural damage of underground structures are generally too broad and a more refined classification of damage is felt to be necessary. The authors would propose a classification for this purpose as given in Table 1. Based on this classification, the authors re-plotted data shown in Fig. 27 as a function of distance from the surface trace or extrapolated surface trace of the earthquake fault. However, this is just a trial plot and how to consider the magnitude of the earthquake, which is a measure of the power of the seismic load, support system and ground properties, still remains as a challenging problem.

As proposed by Aydan (Aydan, 2003; Aydan et al., 1999; Ulusay 2002) the enlargement of cross sections of underground opening (i.e. tunnels) are necessary in order to accommodate the possible relative permanent ground deformations along definite faults with the consideration of their sense of deformation. Nevertheless, none of structural geologists can definitely say which fault rupture would move in the next earthquake. Furthermore, there may be negative and positive flower structures in the vicinity of ground surface. Therefore, the possible ground movements would be diluted into a wide zone and such a counter-measure may be difficult to implement.

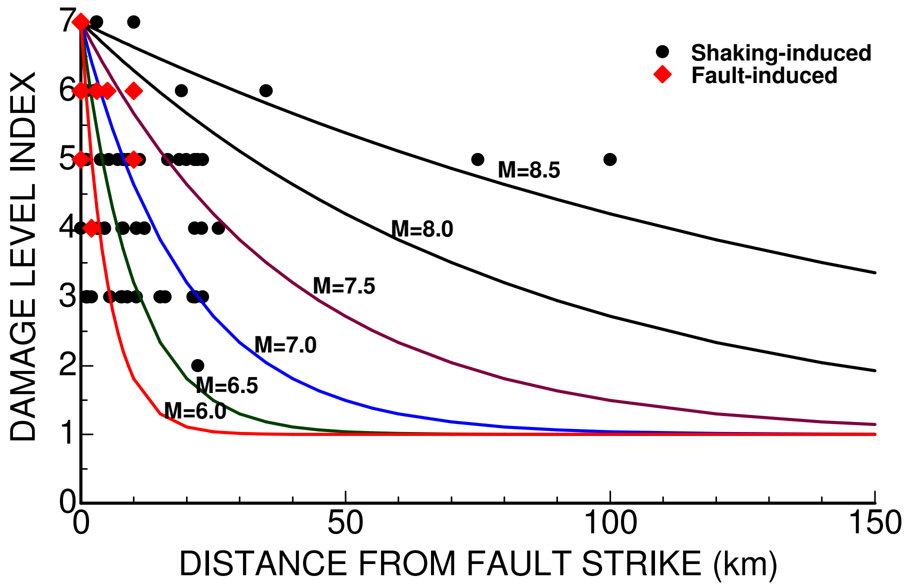


Fig. 27 - Relation between distance (Rf) from surface trace of the fault and damage level index (DLI)

The non-reinforced concrete linings are commonly used for tunnels worldwide except the tunnel portals. As the non-reinforced concrete linings fail in a very brittle manner, this may result in secondary disasters during their service life. Therefore, some counter-measures are necessary to deal this problem. As suggested by Aydan et al. (2009b), there may be two possible ways to deal with this issue. The first alternative would be to line the concrete lining with thin steel platens together with rockbolts. The other alternative may be to use fiber reinforced polymers together with rockbolts to line the inner side of the concrete lining.

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