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Review

Mountain tunnel under earthquake force: A review of possible causes of damages and restoration methods

Xuepeng Zhang [a,b], Yujing Jiang [a,b,*], Kazuhiko Maegawa [c]

[a] *State Key Laboratory of Mining Disaster Prevention and Control Co-founded by Shandong Province and the Ministry of Science and Technology, Shandong University of Science and Technology, Qingdao, 266590, China*
[b] *School of Engineering, Nagasaki University, Nagasaki, 8528521, Japan*
[c] *Nippon Expressway Research Institute Company Limited, Tokyo, 194-8508, Japan*

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ABSTRACT

Accurate seismic assessment and proper aseismic design of underground structures require a comprehensive understanding of seismic performance and response of underground structures under earthquake force. In order to understand the seismic behavior of tunnels during an earthquake, a wide collection of case histories has been reviewed from the available literature with respect to damage classification, to discuss the possible causes of damage, such as earthquake parameters, structural form and geological conditions. In addition, a case of Tawarayama tunnel subjected to the 2016 Kumamoto earthquake is studied. Discussion on the possible influence factors aims at improving the performance-based aseismic design of tunnels. Finally, restoration design criterion and methods are presented taking Tawarayama tunnel as an example.

1. Introduction

Over the past few decades, underground structures such as tunnels, metro stations, and underground parking stations have become major components of any transportation systems. They are increasingly constructed to facilitate different needs in a wide range of engineering applications, including subways, railways, highways, material storage, and sewage and water transport. Historically, underground structures experienced a lower rate of damage than aboveground structures subjected to static or seismic loadings. As a result, underground structures, for example mountain tunnels, were assumed to be seismic resistant since they are buried deeply in rock/soil layers (Dowding and Rozen, 1978; Sharma and Judd, 1991). Nevertheless, if a tunnel experiences a strong shaking, which is located close to an earthquake fault or has difficult geological or construction conditions, there is a high probability that the tunnel can be damaged. Many insurances of noticeable seismic damage were reported to indicate that mountain tunnel could be damaged at different levels by earthquakes, such as the 1923 Kanto earthquake, the 1995 Kobe earthquake, the 1999 Chi–Chi earthquake, the 2004 Niigataken-Chuetu earthquake, the 2007 Niigataken Chuetu-Oki earthquake, the 2008 Wenchuan earthquake, and the 2016 Kumamoto earthquake. Table 1 lists the damages to mountain tunnels by earthquakes over the past years. Underground structures in seismic active area suffered from damages that range from minor cracking to even failure.

All of these records indicate the urgent requirement for investigation on the seismic performance of underground structures subjected to seismic loadings and aseismic design for future underground structure planning. In order to obtain an accurate seismic assessment and aseismic design for underground structures, this study presents a review on damage classification and possible causes of damage. Finally, restoration design criterion and methods are also presented taking Tawarayama tunnel as an example.

2. Review of classification of seismic damage to mountain tunnels

Seismic performances of tunnels have been extensively studied (e.g. Duke and Leeds, 1959; Dowding and Rozen, 1978; Owen and Scholl, 1981; Sharma and Judd, 1991; Power et al., 1998; Wang et al., 2001;

* Corresponding author.
E-mail address: jiang@nagasaki-u.ac.jp (Y. Jiang).
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Table 1
Damages to mountain tunnels by earthquakes over the past years modified after Asakura et al., 2000; Wang et al., 2012; Isago and Kusaka, 2018; Okano, 2018; Zhang et al., 2018.

Year	Location	Magnitude	Tunnel performance
1906	San Francisco (USA)	8.3	Extensive and severest damage to 2 tunnels crossing San Andreas Fault
1923	Kanto (Japan)	7.9	Extensive and severest damage to more than 100 tunnels in southern Kanto area
1927	Kita-Tango (Japan)	7.3	Very slight damage to 2 railway tunnels in the epicentral region
1930	Kita-Izu (Japan)	7.3	Very severe damage to one railway tunnel due to earthquake fault
1948	Fukui (Japan)	7.1	Severe damage to 2 railway tunnels within 8 km from the earthquake fault
1952	Tokachi-Oki (Japan)	8.2	Slight damage to 10 railway tunnels in Hokkaido
1952	Kern (USA)	7.7	severe damage to 4 railway tunnels
1961	Kita-Mino (Japan)	7	Cracking damage to a couple of aqueduct tunnels
1964	Niigata (Japan)	7.5	Extensive damage to about 20 railway tunnels and one road tunnel
1968	Toikchi-Oki (Japan)	7.9	Slight damage to 23 railway tunnels in Hokkaido
1970	Tonghai (China)	7.7	Severe damage to road tunnels, especially portal collapse
1971	Los Angeles (USA)	6.6	Several damages to mountain tunnels crossing through Thelma Fault, slight damage to 3 mountain tunnels
1978	Izu-Oshima-Kinkai (Japan)	7	Very severe damage to 9 railway and 4 road tunnels in a limited area
1978	Miyagiken-Oki (Japan)	7.4	Slight damage to 6 railway tunnels mainly existing in Miyagi Prefecture
1982	Urakawa-Oki (Japan)	7.1	Slight damage to 6 railway tunnels near Urakawa
1983	Nihonkai-Cyubu (Japan)	7.7	Slight damage to 8 railway tunnels in Akita, etc.
1984	Naganoken-Seibu (Japan)	6.8	Cracking damage to one hydraulic power tunnel
1987	Chibaken-Toho-Oki (Japan)	6.7	Damage to the wall of one railway tunnel at Kanagawa-Yamanashi border
1993	Notohanto-Oki (Japan)	6.6	Severe damage to one road tunnel
1993	Hokkaido-Nansei-Oki (Japan)	7.8	Severe damage to one road tunnel due to a direct hit of falling rock
1995	Hyogoken-Nanbu (Japan)	7.2	Damage to over 20 tunnels, about 10 tunnels required repair and reinforcement
1999	Chi–Chi (Taiwan, China)	7.6	Damage to about 57 tunnels, of which 14 are severely damaged, 11 are moderately damaged, and 23 are slight damaged
2004	Niigataken-Chuetsu (Japan)	6.8	Damage to about 50 tunnels, of which 25 or so needed reinforcement or repair
2007	Niigataken Chuetu-Oki (Japan)	6.8	Damage to about 21 tunnels, of which 5 are severely damaged, 13 are moderately damaged, and 4 are slightly damaged
2008	Wenchuan (China)	8	Damage to about 55 mountain tunnels in the seismic active area, of which 10 are collapsed or severely damaged, 11 are moderately damaged, and 17 are slightly damaged
2016	Kumamoto (Japan)	7.3	Severely damage to 1 mountain tunnel, moderate/slight damage to 3 railway and road tunnels

Chen et al., 2012; Wang and Zhang, 2013; Yu et al., 2013, 2016a,b; Lai et al., 2017). Currently, several global databases are available. For example, Dowding and Rozen (1978) created a database with 71 cases of seismic damage to tunnels in Japan and USA. Owen and Scholl (1981) extended the aforementioned database with up to 127 cases. Sharma and Judd (1991) compiled a database with 192 cases during 85 earthquakes. Power et al. (1998) used the previous database to study the performance of bored tunnels, adding cases from more recent earthquakes (e.g. the 1995 Kobe earthquake). After investigating 10 strong earthquakes, Chen et al. (2012) established a database for the damage situation of 81 mountain tunnels.

Even though such progress has been made for seismic analysis, criteria for damage classification of tunnels have not been unified yet. Dowding and Rozen (1978) suggested three forms of seismic damages to tunnels, i.e. damage from earthquake-induced ground failure, damage by fault deformation, and damage by ground shaking or vibration. Wang et al. (2001) summarized six forms of damage patterns based on the characteristics and distribution of lining cracks: sheared-off lining, tunnel collapse by slope failure, longitudinal crack, transverse crack, inclined crack, extended cross crack, pavement or bottom cracks, wall deformation and cracks that develop near the opening. Yashiro et al. (2007) classified the damage patterns into three types: damage to shallow tunnels, damage to tunnels in poor geological conditions and damage to tunnels by fault sliding. Wang et al. (2009) illustrated eight major patterns of seismic damages: portal failure, longitudinal crack, transverse crack, inclined crack, shear failure of lining, pavement crack, lining spalling and groundwater inrush. Li (2012) analyzed the Wenchuan earthquake to Categorize the damage to mountain tunnel as avalanches and sliding towards tunnel portal, cracking of tunnel portals, collapse of lining and surrounding rock, cracking and displacement of lining, heaving and cracking of invert, and deformation and failure of primary lining. Based on 10 well-documented earthquakes, Chen et al. (2012) summarized six most frequent damage characteristics regarding the seismic performance of mountain tunnels, including lining cracks (Fig. 1a, Shen et al., 2014), shear failure of lining (Fig. 1b, Wang et al., 2001), collapse by slope failure (Fig. 1c, Li, 2012), portal cracking (Fig. 1d, Shen et al., 2014), groundwater leakage (Fig. 1e, Chen et al., 2012), and wall/invert damage (Fig. 1f, Yu et al., 2016a). Shen et al. (2014) analyzed typical seismic damage characteristic and mechanism of mountain tunnels based on three different damage patterns: damage to shallow tunnel, damage to deep tunnel structure, and damage to pavement.

Zhang et al. (2018) classified the seismic damages to Tawarayama tunnel into five patterns: cracks of tunnel lining, damage of construction joint, groundwater leakage, spalling and collapse of concrete lining, and damage of pavement. Fig. 2 illustrates the statistic result of each seismic damage pattern. The crack of concrete lining is the most frequent characteristic of seismic damage, which can be as high as 66.53%. The cracks can be classified into ring crack (23.91%, see Fig. 3a), transverse crack (57%, see Fig. 3b), longitudinal crack (13.04%, Fig. 3c), and inclined crack (20%, Fig. 3d). Some occasional cases are construction joint damage, lining concrete spalling/collapse and groundwater leakage.

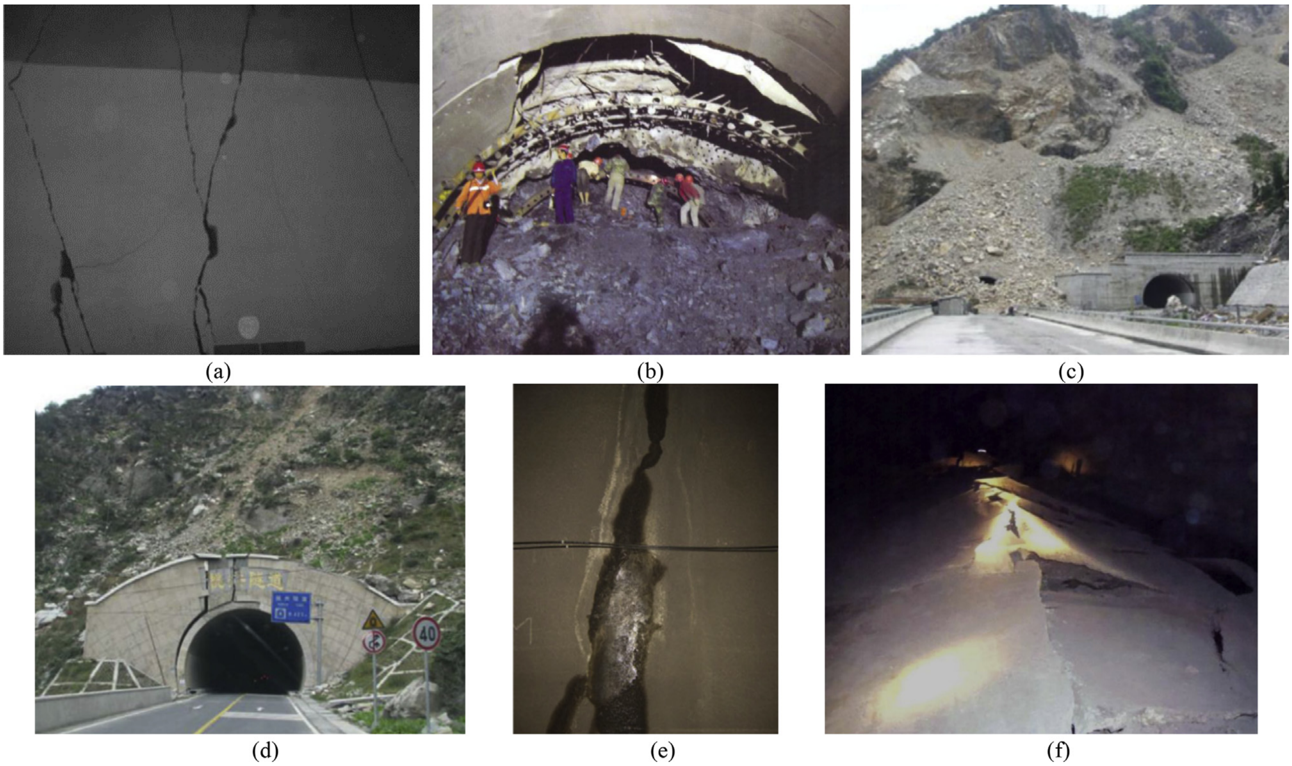


Fig. 1. Six most frequent damage characteristics of mountain tunnel subjected to earthquakes: (a) Lining cracks (Shen et al., 2014); (b) Shear failure of lining (Wang et al., 2001); (c) Collapse by slope failure (Li, 2012); (d) Portal cracking (Shen et al., 2014); (e) Groundwater leakage (Chen et al., 2012); and (f) Invert damage (Yu et al., 2016a).

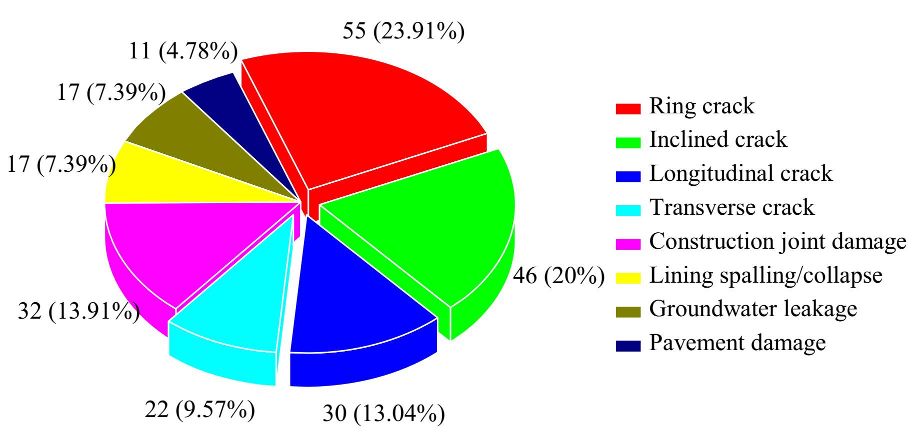


Fig. 2. Statistic result of each seismic damage pattern in Tawarayama tunnel.

3. Possible causes for seismic damages to mountain tunnels

3.1. Review of possible causes for seismic damages

Earthquake effect on mountain tunnels can be grouped into two categories: ground shaking due to wave propagation, and ground failure due to lateral spreading, landslide and fault rupture (Dowding and Rozen, 1978; St. John and Zahrah, 1987; Hashash et al., 2001). Three primary types of influence factors are suggested: earthquake parameters, structural forms, and geological conditions (Chen et al., 2012).

For the earthquake parameters, four main aspects are considered, including magnitude, focal depth, epicentral distance, and wave propagation direction (Sharma and Judd, 1991; Chen et al., 2012; Li, 2012; Wang and Zhang, 2013). The first three jointly determine the earthquake intensity of a particular area. The earthquake will be much more intense and its influence will be much stronger in the area with higher magnitude, shallower focal depth and shorter epicentral distance (Chen et al., 2012; Roy and Sarkar, 2017). In addition, Jiang et al. (2010) and Li (2012) focused on the influence of epicentral distance on seismic damage to the tunnel. On the other hand, the deformation mode and resulting damage to tunnels due to earthquakes are significantly influenced by the propagation direction of seismic waves (Li, 2012). Fig. 4 presents a simplified deformation mode of tunnels subjected to seismic loads. Three types of deformations could be used to express the effect of wave propagation direction on response of underground structure as follows: (1) axial compression and extension (Fig. 4a, b and d); (2) longitudinal bending (Fig. 4c); and (3) ovaling/racking (Fig. 4e and f) (Owen and Scholl, 1981; Maugeri and Soccodato, 2014).

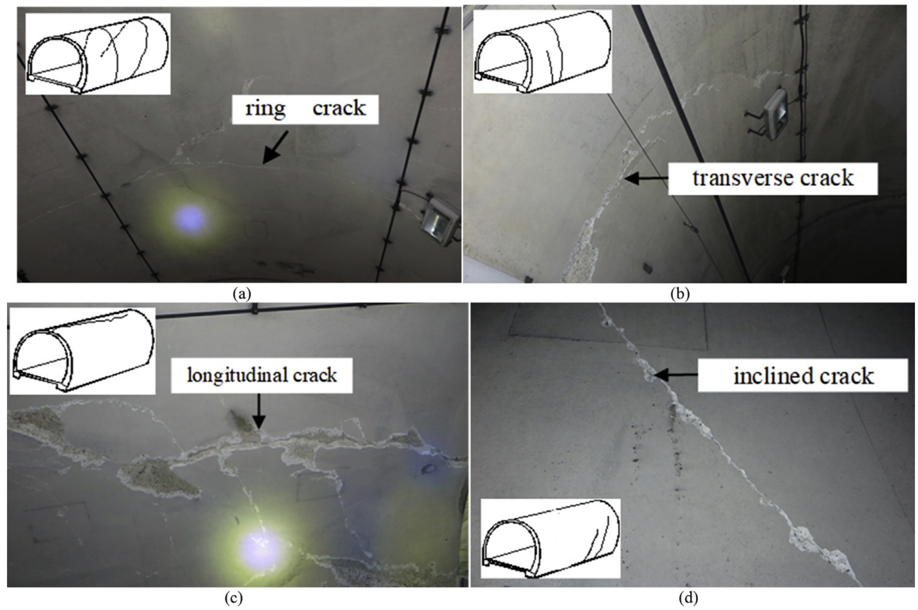


Fig. 3. Classification of cracks at Tawarayama tunnel subjected to the 2016 Kumamoto earthquake (after Zhang et al., 2018).

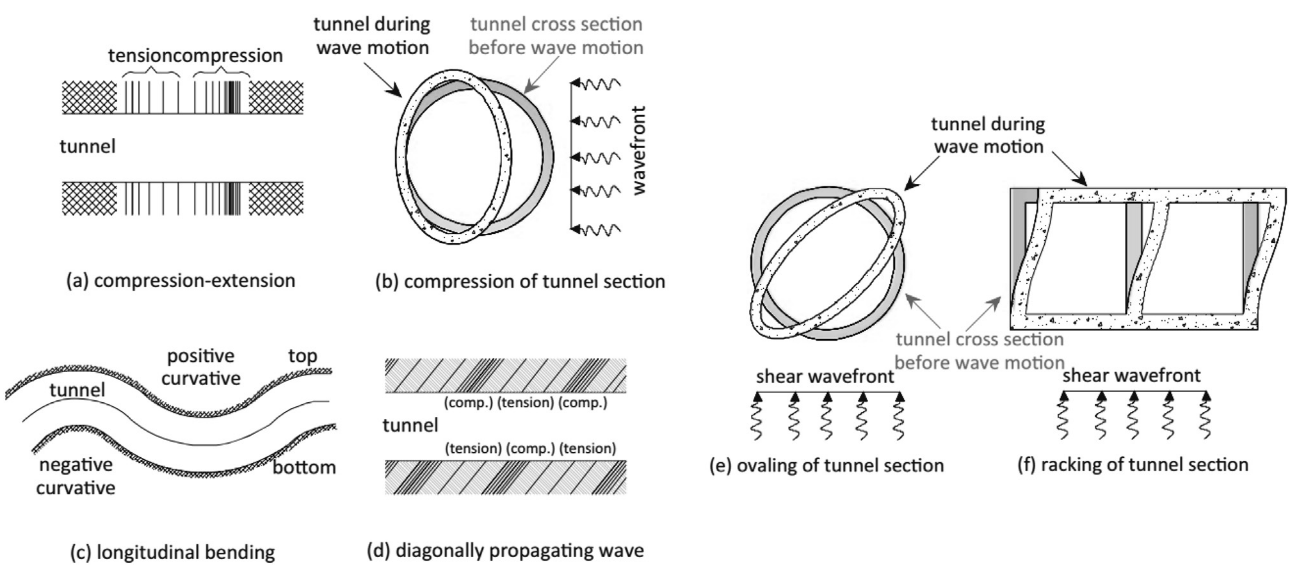


Fig. 4. Simplified deformation modes of tunnels due to seismic waves (Owen and Scholl, 1981; Maugeri and Soccodato, 2014).

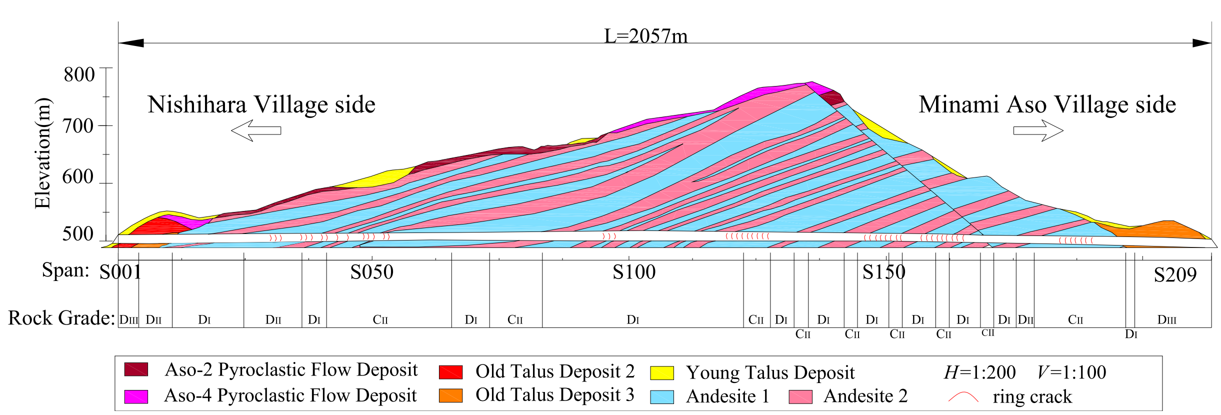


Fig. 5. Geological profile of Tawarayama tunnel. Based on Japan Road Association (JARA) (2003), rock mass along the tunnel is divided into four classes: C_{II}, D_{I}, D_{II}, and D_{III} (Zhang et al., 2018). S is the short for span.

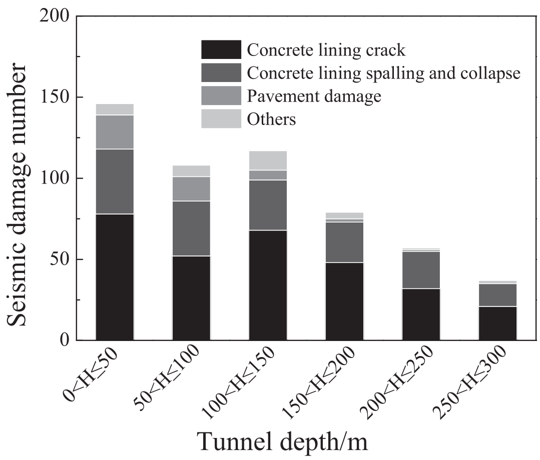


Fig. 6. Relationship between burial depth of Tawarayama tunnel and resulting seismic damages due to the 2016 Kumamoto earthquake.

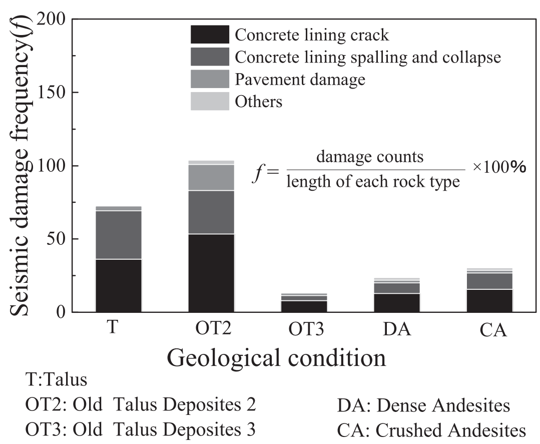


Fig. 7. Relationship between rock type and seismic damage frequency in Tawarayama tunnel subjected to the 2016 Kumamoto earthquake.

The influence factors regarding structural form mainly include burial depth, condition of tunnel lining, construction method, loading form, and cross-section with sudden change of tunnel structure (Yashiro et al., 2007; Chen et al., 2012; Li, 2012;Wang and Zhang, 2013). The geological conditions exercise a great influence on the structural seismic performance due to rock/soil-structure interaction effect. Two site conditions regarding the geological conditions are considered as follows: permanent ground deformation, and deterioration of site conditions (Chen et al., 2012). On one hand, fault movement and deformation of surrounding rock/soil are taken into account for the permanent ground deformation (Li, 2012; Roy and Sarkar, 2017). Seismic motions often induce large shear movement of fault. There is a high probability that seismic damages occur due to collapse, squeezing and pulling, when tunnel passes through shear areas (Wang et al., 2001; Chen et al., 2012; Li, 2012). On the other hand, obvious deterioration of site conditions includes liquefaction and degradation of seismic subsidence for the soft-soil area and high weathering and decompression for the hard-rock area (Yakovlevich and Borisovna, 1978; Chen et al., 2012; Li, 2012). Slope failures often occur due to poor rock mass quality of the slopes at tunnel opening.

3.2. Case study on the possible causes in Tawarayama tunnel

Tawarayama tunnel was excavated using new Austrian tunneling method (NATM) under Mt. Tawarayama on the Takamori Line of Kumamoto Prefectural Route 28. The total length of the tunnel is 2057 m with a horseshoe-shaped cross-section. The typical cross-section has a total width of 10.2 m and a maximum height of 7.97 m. The tunnel is located at about 22.4 km away from the epicenter of the mainshock ($M_j = 7.3$, at 01:25 on April 16, 2016) of the 2016 Kumamoto earthquake and was destroyed severely by the earthquake.

According to the geological investigation of Tawarayama tunnel, a Series of factors affecting its seismic response subjected to the 2016 Kumamoto earthquakes are summarized as follows: earthquake parameter, tunnel burial depth, surrounding rock mass quality, fault zone, and slope at the portal.

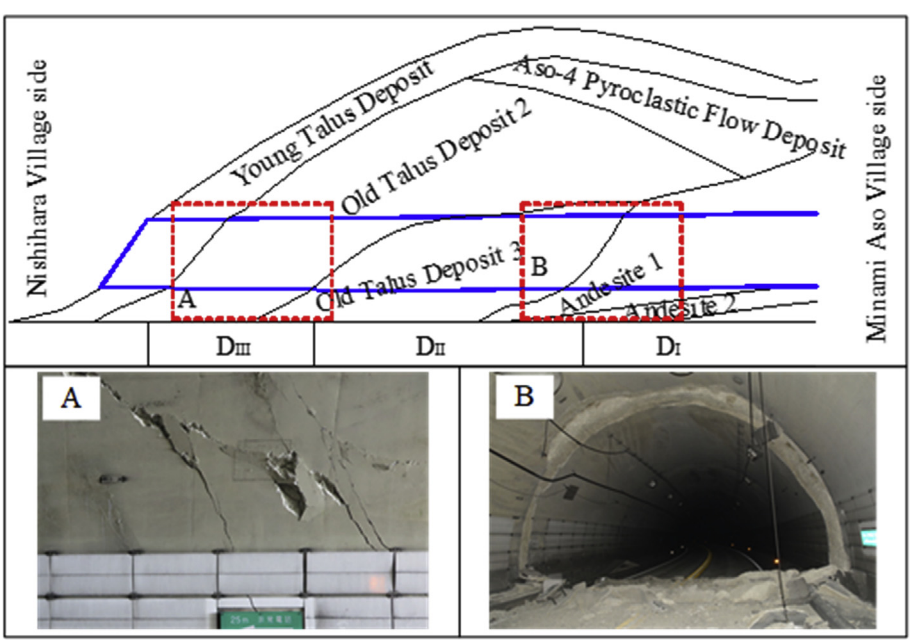


Fig. 8. Seismic damage at change zones between different grade rock masses.

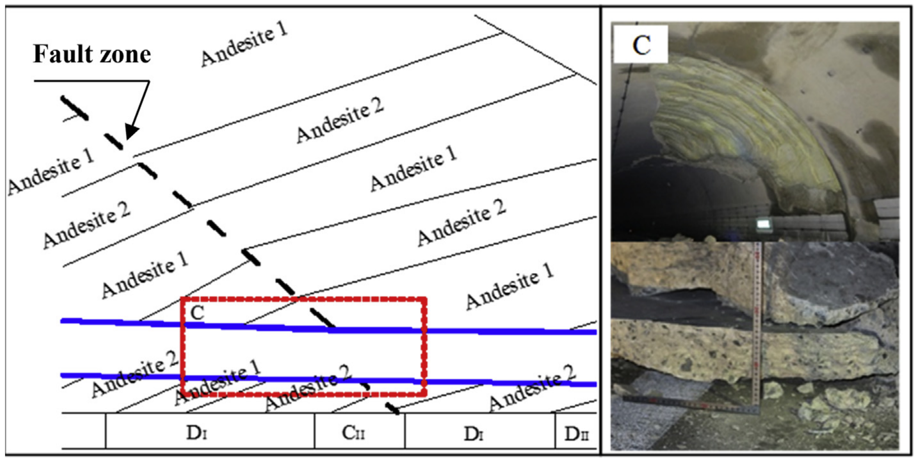


Fig. 9. Lining collapse at fault zone.

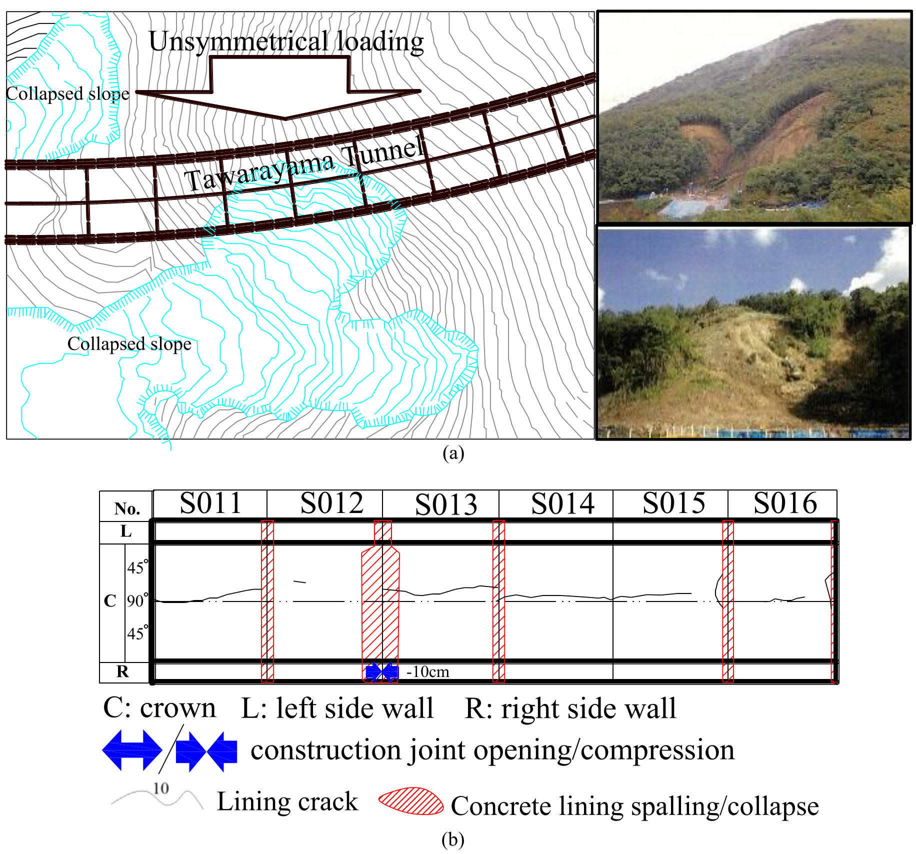


Fig. 10. Slope deformation in the Nishihara Village side (a) and resulting tunnel seismic damage (b).

3.2.1. *Earthquake parameter*

As aforementioned previously, magnitude, focal depth, and epicentral distance are the three key influence factors for earthquake parameters. Seismic waves with higher magnitude and shallower focal depth will cause tunnels to be under greater seismic forces if they are closer to the epicenter. For the 2016 Kumamoto earthquake, Tawarayama tunnel is about 22.4 km away from the epicenter of the mainshock at a focal depth of about 12 km with magnitude 7.3 (M_j). The investigation results are in accordance with previous results after analyzing 192 cases of underground structures influenced by 78 earthquakes (Sharma and Judd, 1991; Chen et al., 2012). Most of the tunnels were damaged or even failed when the magnitudes of the earthquakes were 7 and above. The percentage of the damaged tunnel was up to 71% when the epicentral distance is less than 25 km. The percentage is near 75% when the epicentral distance is less than 50 km.

Additionally, the propagation direction of the seismic waves may significantly affect tunnel seismic responses and result in damages. Epicenters of both mainshock and foreshock are located in the southwest direction of Tawarayama tunnel (Zhang et al., 2018, 2019).

Table 2
Soundness classification for road tunnel (JARA, 2015).

Classification		Definition
I	Sound status	There is no problem with the function of the road tunnel
II	With necessity for preventive maintenance	There is no hindrance to the function of the road tunnel, but it is desirable to take measures from the viewpoint of preventive maintenance
III	With necessity for an early measure	There is a possibility that the function of the road tunnel may be impaired, and it is necessary to take early measures
IV	With emergency measure	There is a situation in which the function of the road tunnel has been hindered or the hindrance is likely to occur, and emergency measures should be taken

Besides, near the western portal of the tunnel, there is a fault zone named Futagawa fault zone with the NE–SW general strike. The axis of Tawarayama tunnel strikes west-east. The strike of the Futagawa fault zone obliquely crosses the axis of Tawarayama tunnel. Site investigation showed that the fault zone was dislocated with a maximum displacement of about 2.2 m (in Mashiki). Seismic waves will also cause tunnels to be under greater seismic forces if they are closer to a displaced fault (Wang et al., 2001). The seismic wave in this area propagates crossing the axis of the tunnel obliquely. The west-east direction of the seismic ground motion validated the propagation direction of the seismic wave through the spatial correlation of the ground deformation at Mt. Tawarayama with the seismic damages to Tawarayama tunnel (Zhang et al., 2019). Axial tension-compression deformations or bending deformations occur when seismic wave propagates parallel to or obliquely crossing the axis of the tunnel. The lining cracks and pavement and invert failure due to compression/tension in Tawarayama tunnel are the typical examples of these types of failures.

3.2.2. Burial depth of tunnel

Fig. 5 illustrates the geological condition around Tawarayama tunnel. The maximum overburden of Mt. Tawarayama above the tunnel is nearly 300 m. Fig. 6 provides a relationship between the burial depth of Tawarayama tunnel and resulting seismic damages due to the Kumamoto earthquake. With increase of burial depth, the seismic damage intensity decreases due to the stronger constraint of strata. At depth less than 150 m, a large percentage of seismic damages occurred in Tawarayama tunnel. Previous literature suggested a limit burial depth of 50 m for the damage probability level with reference to the results of site investigation (Sharma and Judd, 1991; Chen et al., 2012). The difference in the depth limit is also related to rock mass quality and other geological conditions (Li, 2012).

3.2.3. Rock mass quality

There are three types of geological formations along the axis of Tawarayama tunnel, as illustrated in Fig. 5. They are Quaternary Holocene, Quaternary Pleistocene, and Tertiary Pliocene. At the portal of the tunnel, there is talus deposit (it is a mixture of andesite gravel and welded tuff gravel with matrix part of

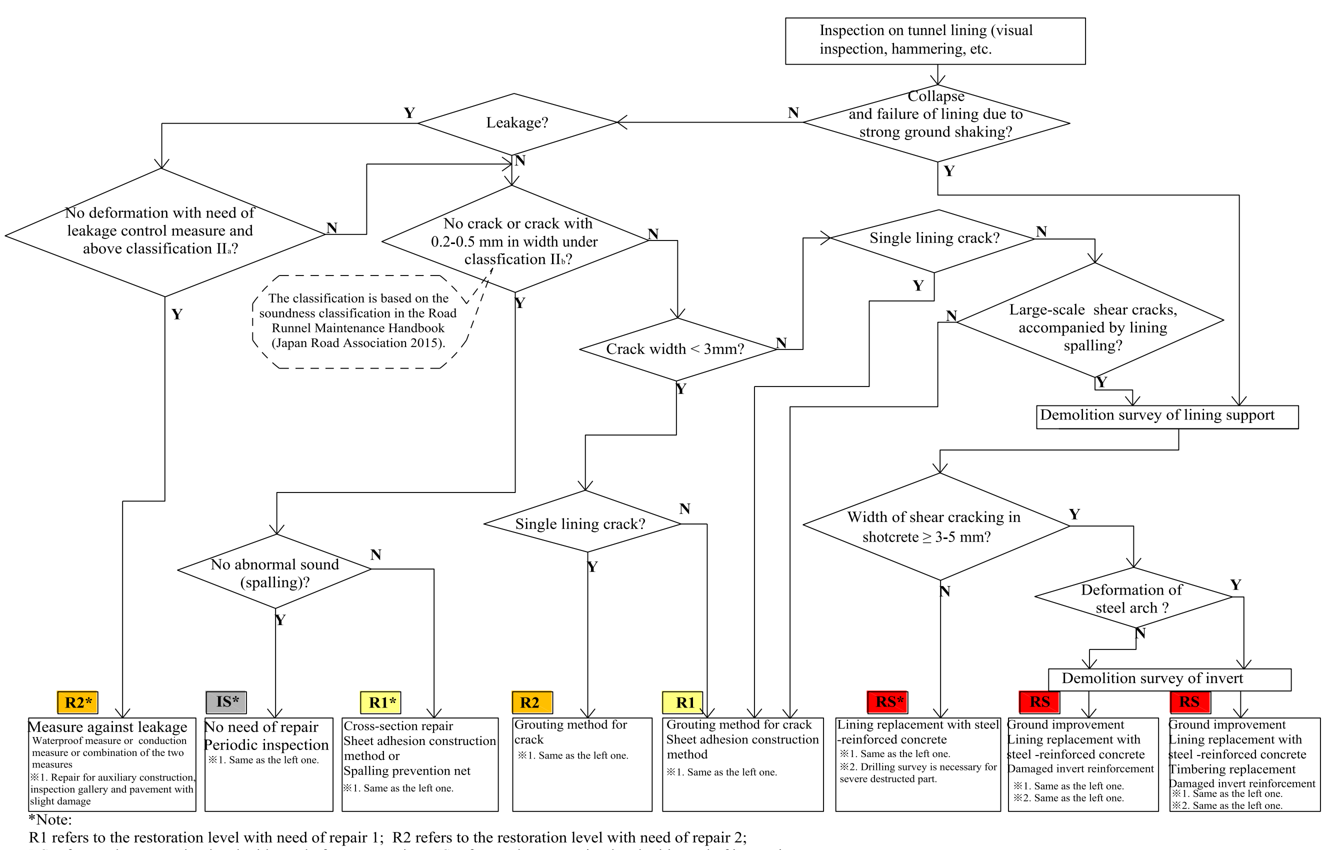


Fig. 11. Flowchart of the newly proposed restoration criterion for Tawarayama tunnel (Kumamoto River and National Highway Office, Kyushu Regional Development Bureau, Ministry of Land, Infrastructure, Transport and Tourism, 2017).

Span	001	002	003	004	005	006	007	008	009	010	011	012	013	014	015	016	017	018	019	020	021	022	023	024	025	026	027	028
Level	RS	R1	R1	RS	RS	R1	R2	R2	R2	RS	R2	RS	RS	R2	R2	R2	R1	R2	R2	IS	R2	R2	R2	R1	R1	R2	R1	RS
Span	029	030	031	032	033	034	035	036	037	038	039	040	041	042	043	044	045	046	047	048	049	050	051	052	053	054	055	056
Level	RS	R1	R2	R1	R2	R2	R2	R1	R1	R1	R1	R1	R1	R2	R2	R2	R2	IS	IS	R2	R2	R2	IS	R1	R1	IS	R2	R2
Span	057	058	059	060	061	062	063	064	065	066	067	068	069	070	071	072	073	074	075	076	077	078	079	080	081	082	083	084
Method	IS	IS	IS	IS	R1	R1	R2	R2	R2	R1	R1	R2	R1	R1	R1	R2	R2	R2	R2	R1	R1	R2	R2	IS	R2	R2	R2	R2
Span	085	086	087	088	089	090	091	092	093	094	095	096	097	098	099	100	101	102	103	104	105	106	107	108	109	110	111	112
Level	R2	R2	R2	R2	IS	IS	IS	R2	R2	R2	R1	RS	RS	IS	IS	R2	R1	R2	R1	R2	R2	IS	IS	IS	IS	IS	IS	R2
Span	113	114	115	116	117	118	119	120	121	122	123	124	125	126	127	128	129	130	131	132	133	134	135	136	137	138	139	140
Level	R1	R2	IS	R1	R2	R1	R1	R1	R1	R1	R1	R1	R1	R2	IS	IS	IS	R1	IS	IS	IS	R2	R2	R1	IS	IS	R2	R1
Span	141	142	143	144	145	146	147	148	149	150	151	152	153	154	155	156	157	158	159	160	161	162	163	164	165	166	167	168
Level	IS	IS	IS	IS	R2	R1	R1	R1	R1	R1	R1	R1	R1	IS	IS	R1	R1	RS	RS	R1	R1	R1	R1	R1	RS	RS	RS	RS
Span	169	170	171	172	173	174	175	176	177	178	179	180	181	182	183	184	185	186	187	188	189	190	191	192	193	194	195	196
Level	IS	IS	IS	IS	IS	IS	IS	IS	IS	IS	IS	IS	IS	IS	IS	R1	R1	R1	R1	R1	R1	R1	R2	IS	IS	R2	R2	IS
Span	197	198	199	200	201	202	203	204	205	206	207																	
Level	IS	IS	IS	IS	IS	R2	R1	R2	R1	R2	IS																	

Note: RS Restoration level with need of reconstruction; IS Restoration level with need of inspection; R1 Restoration with need of repair 1; R2 Restoration with need of repair 2

Fig. 12. Restoration level at each span in Tawarayama tunnel according to the proposed restoration design criterion (Kumamoto River and National Highway Office, Kyushu Regional Development Bureau, Ministry of Land, Infrastructure, Transport and Tourism, 2017).

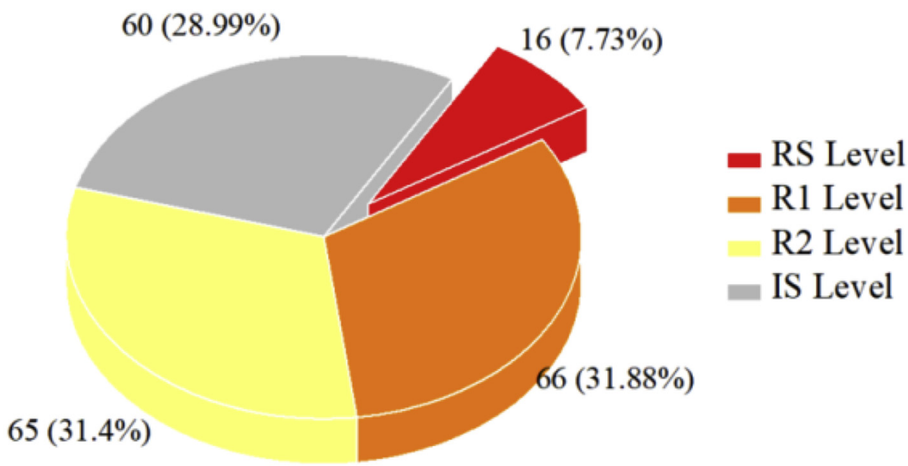


Fig. 13. Span percentage of each restoration countermeasure for Tawarayama tunnel.

volcanic ash clayed soil) and old talus deposit (old talus deposit 2 is welded tuff, and old talus deposit 3 is welded pyroclastic flow containing a large amount of scoria). Besides, andesite lava is the most common for the other section of the tunnel. A ratio of seismic damages and length of each rock type that the tunnel crosses is defined as seismic damage frequency *f*. Fig. 7 presents the relationship between rock type and seismic damage frequency in Tawarayama tunnel subjected to the 2016 Kumamoto earthquake. Due to large damping and limited capability to constrain the lining structure of loose deposits and broken rock masses (i.e. talus and old talus deposits at the portal of Tawarayama tunnel) with low strength surrounding the tunnel, seismic damages occurred more frequently with relatively high seismic frequency. For andesite in the deeper mined part, tunnel structures in the crushed one are more vulnerable to seismic damages than those in the dense one due to the poor mechanical properties of the crushed andesite.

Around Tawarayama tunnel, there are four types of rock grades categorized according to Japan Road Association (JARA) (2003): C_{II}, D_I, D_{II}, and D_{III}, as shown in Fig. 5. The condition of the rock mass is much worse as the grade number increases. When grounds with different rock grades meet around the tunnel, seismic damages suffered by tunnel structure during an earthquake usually occur due to the ground squeeze in soft ground or ground relative deformation at the intersection of different grounds. The damages at the western portal near the Nishihara Village side are the representative cases, as shown in Fig. 8.

3.2.4. Fault zone

One fault was detected crossing the axis of the tunnel during the site investigation of Tawarayama tunnel, as shown in Fig. 9 (the black dashed line). The serious collapse of the secondary lining (S167) occurred due to the large shear movement of fault during the 2016 Kumamoto earthquake, as illustrated by Area C in Fig. 9. Besides, some gravel deposits at the waterproof for the span S167 were also observed. The contact between the primary and secondary linings was imperfect. The stress and displacement behaviors of tunnel lining strongly depend on the contact status (Son and Cording, 2007). The contact status between tunnel linings and that between tunnel lining and its surrounding rock is one significant factor that cannot be ignored for seismic performance of underground structure during an earthquake.

3.2.5. Slope at portal

Fig. 10 shows the slope deformation in the Nishihara Village side and resulting damages to Tawarayama tunnel. During earthquake, landslides occurred due to the unsymmetrical loading of the slope. As a result, the lining structure underneath the ground was forced to move 10 cm towards the southern direction (the direction of the landslide). Deformation due to compression and bending occurred, which even induced longitudinal crack and spalling of concrete lining, as shown in Fig. 10b. Moreover, the dislocation of the tunnel lining due to the southwestward movement contributed to uplift of the maintaining roadway at the left side.

Table 3
Summary of restoration measures adopted for Tawarayama tunnel (Kumamoto River and National Highway Office, Kyushu Regional Development Bureau, Ministry of Land, Infrastructure, Transport and Tourism, 2017).

Tunnel section			Damage condition	Restoration method and soundness
Exterior of portal section	Electric room	Before entrance	Ground collapse of the electric room	Demolition and reconstruction of electric room; Ground re-embankment
	Unsymmetrical terrain	S001–S006	Continuous and longitudinal cracks due to tension along the left shoulder of the tunnel, tunnel lining deformation; Partial collapse of the slope right above the tunnel	Soil removal work to reduce the influence of unsymmetrical terrain
Entrance	S001		Movement deformation, shear cracking of lining shoulder	Reconstruction of the lining at the entrance
Tunnel interior	Remarkable damage	S004 and S005	Collapse, movement deformation, shear crack, spalling, exposure of reinforcing steel, etc.	Reconstruction method; Auxiliary construction method (FRP injection rock-bolt for sewing effect)
		S010, S012, and S013	Collapse, movement deformation, compressive damage	Reconstruction with reinforcement steel
		S096, S097, S166, S167 and S168	Collapse, movement deformation, shear crack, spalling, compressive damage	Reconstruction with reinforcement steel; Auxiliary construction method (FRP injection rock-bolt for ground improvement and sewing effect)
		S028, S029, S158, S159 and S165	Collapse, movement deformation, shear crack, spalling, compressive damage	Reconstruction with reinforcement steel
	Inclined crack	S002, S003, S006, S036, S037, S121, etc.	Inclined crack due to tension with width 3 mm or more, length of 5 m or more; Splitting of concrete along the crack and slight spalling; Partial concentration of crack with a width of 0.3 mm or more and density of 0.2 m/m^2 or more and concrete splitting	Sheet adhesion construction method with carbon fiber or FRP ※Repair 1 (R1)
	Transverse crack	S038, S041, S061, S062, S067, S095, etc.	Transverse crack due to tension with width 3 mm or more, length of 5 m or more; Splitting of concrete along the crack and slight spalling; Partial concentration of crack with a width of 0.3 mm or more and density of 0.2 m/m^2 or more and concrete splitting	Sheet adhesion construction method with carbon fiber or FRP ※ Repair 1 (R1)
	Slight damage	S007, S008, S009, S011, etc.	Slight crack, concrete splitting of construction joint and leakage	Grouting method for crack, measures against leakage and splitting ※ Repair 2 (R2)
	Slight damage without need of repair	S038, S041, S061, S062, etc.	Slight damage or no damage	Periodic inspection
Exit	S207		No damage	Sound status
Auxiliary construction	Pavement, inspection-gallery, drainage		Slight damage without repair need except for sections of partial pavement heave and movement deformation	Replacement of pavement, reconstruction of circular gutter, replacement of inspection-gallery, etc.

3.3. Restoration design criterion and methods for damaged mountain tunnel

After an earthquake, the damaged tunnel should be restored as soon as possible for later usage. Discussion on the influence factors is to provide a fundamental suggestion for restoration and aseismic design for mountain tunnel directly and eventually to improve the performance-based aseismic design of the tunnel. Kunita et al. (1994) presented a restoration work of Kinoura tunnel by the Noto Peninsular offshore earthquake. Most studies regarding seismic effect on mountain tunnel were focused on the aseismic design method (e.g. St. John and Zahrah, 1987; Sharma and Judd, 1991; Hashash et al., 2001; Wang et al., 2001; Yashiro et al., 2007; Chen et al., 2012; Li, 2012; Maugeri and Soccodato, 2014; Shen et al., 2014; Isago and Kusaka, 2018; Zhang et al., 2018). Herein, the case of the restoration work for Tawarayama tunnel by the 2016 Kumamoto earthquake is presented with respect to the restoration design criterion and restoration method.

3.4. Restoration design criterion for Tawarayama tunnel

According to the soundness classification of tunnel structure in Road Tunnel Maintenance Handbook (JARA, 2015) as listed in Table 2, the soundness of tunnel lining can be preliminarily classified into four categories: (1) soundness I with sound status; (2) soundness II with necessity for preventive maintenance; (3) soundness III

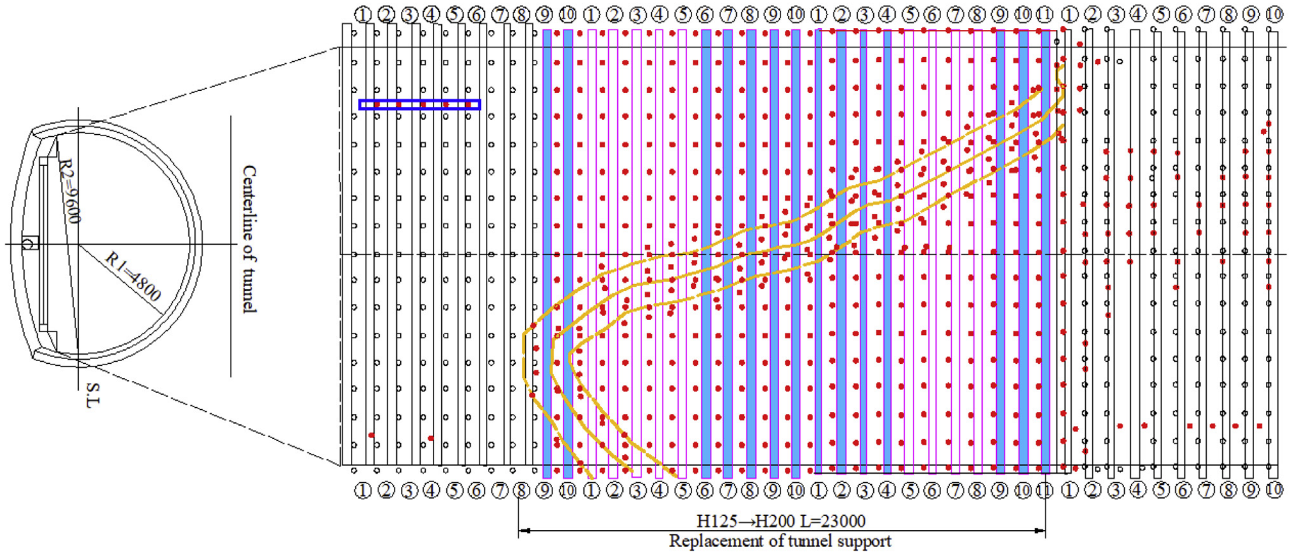
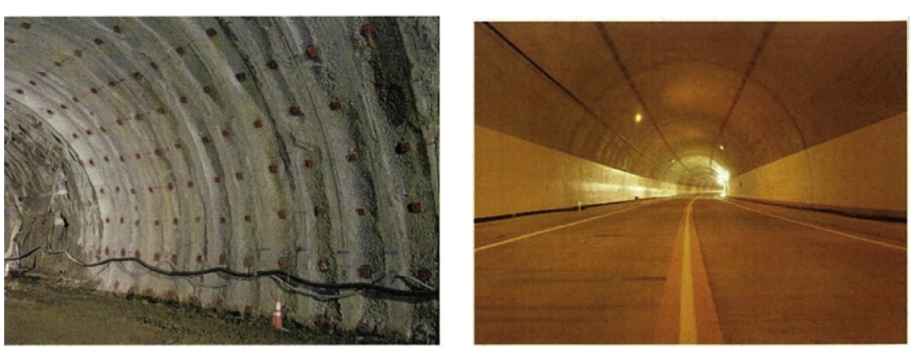


Fig. 14. Reconstruction of the lining from spans of S165 to S167 (unit: mm). Area with the yellow line is the fault zone behind the tunnel lining, red solid circle denotes the replaced rock-bolt, and blue lines denote the replaced steel arch.

(a) (b)

Fig. 15. Damage condition of steel support: (a) Right side of span S166, and (b) Left side of span S167.



(a) (b)

Fig. 16. Tunnel condition after reconstruction: (a) Condition after replacement of steel support at the left side in the spans of S166 and S167, and (b) Condition after reconstruction in the spans from S165 to S168.

with necessity for early measure; and (4) soundness IV with emergency measure. For convenience, the four degrees are referred to as I, II, III, and IV in sequence. Diagnosis results showed that 54 spans among the 209 spans in Tawarayama tunnel are with soundness I, 66 spans with soundness II, 31 spans with soundness III, and 58 spans with soundness IV (Isago and Kusaka, 2018). For the spans with the soundness from II to IV, repair and reinforcement works are necessary. The preliminary classification provides a reference for a further selection of detailed restoration measures. It can avoid unnecessary work for the condition without necessity for repair.

Since restoration measures depend on the status of damage, a design criterion based on the site investigation of Tawarayama tunnel is established, in order to accurately determine the reasonable restoration measures. This new criterion is developed with reference to the restoration design criterion for Touya tunnel (Suzuki et al., 2001). The restoration criterion for Touya tunnel was

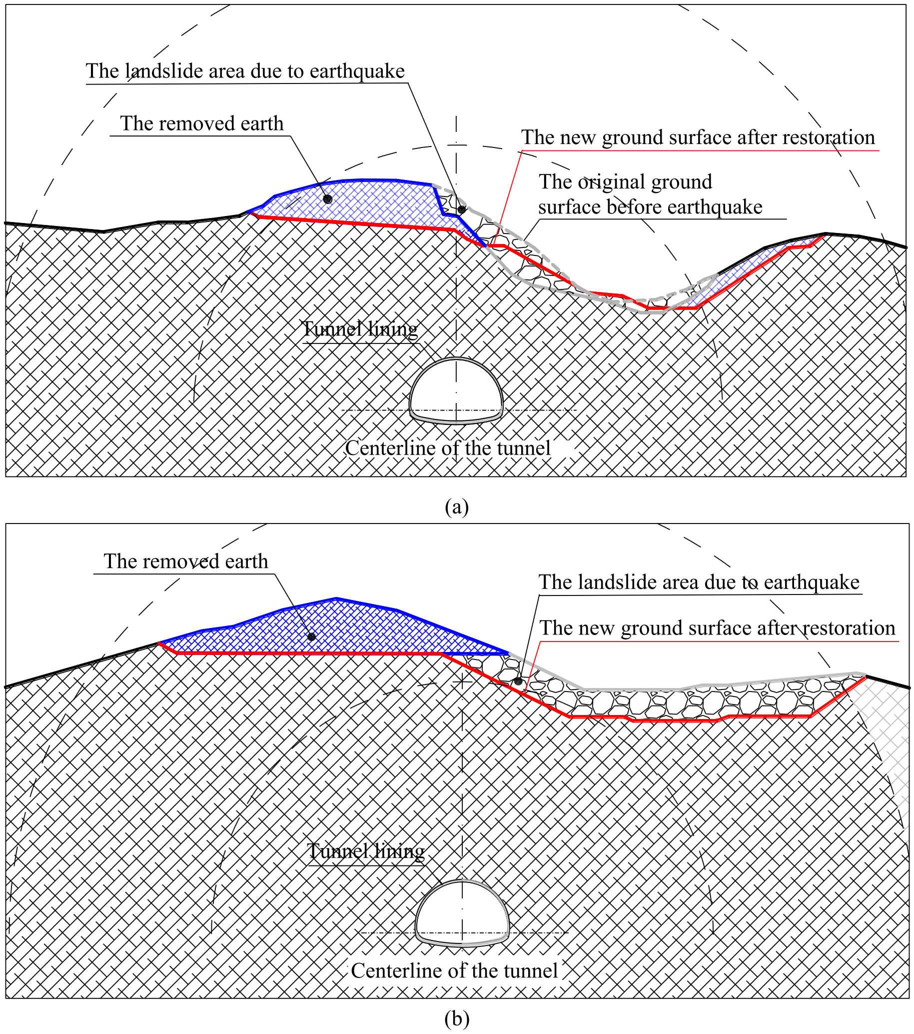


Fig. 17. Excavation of the slope: (a) Cross-section of spans S004 and S005, and (b) Cross-section of spans S006 and S007. Area within blue lines denotes removed earth.

developed against the eruption of Mt. Usu, yet the earthquake effect was not fully taken into consideration.

However, spalling of lining concrete in spans of S004 and S005 and portal sections occurred due to strong ground shaking. Moreover, the collapse of lining concrete was observed in spans of S166 and S167. Therefore, deformation and failure of tunnel linings due to strong ground shaking should not be ignored in the design criterion for tunnel restoration after an earthquake. Fig. 11 shows the flowchart of the proposed restoration design criterion. Similar to the design criterion for Touya tunnel, crack width, crack distribution, and geological condition are considered in the new design criterion. Besides, two aspects, spalling/collapse of tunnel lining due to ground shaking and groundwater leakage, are introduced into the present criterion.

Based on the proposed design criterion, the restoration level can be classified into four categories: reconstruction (RS), repair 1 (R1), repair 2 (R2), and inspection (IS). For lining in level RS, ground improvement, lining replacement with steel-reinforced concrete, steel arch support replacement and invert reinforcement are necessary. Lining in level R1 requires restoration such as section repair method, spalling prevention net method, and grouting method for cracks. Measures against water leakage and grouting method for cracks are necessary for lining in level R2. There is no special required restoration for lining in level IS, but periodic inspection is required.

3.5. Restoration method for Tawarayama tunnel

Two hundred and seven (207) spans of Tawarayama tunnel were investigated based on the proposed design criterion. Fig. 12 shows the restoration level at each span in the tunnel. Reconstruction is needed in the portal (S001) and the spans with lining spalling/collapse and other large deformation (S004, S005, S010, S012, S013, S028, S029, S096, S097, S158, S159, S165, S166, S167, and S168). In total, there are 16 spans among the 207 spans that need to be reconstructed. Except for the portal (span S001), lining concrete of other spans with the requirement for reconstruction was removed to provide a direct visual inspection on the steel arch support. Moreover, the pavement in 11 spans among the 16 spans was also removed. On the other hand, 66 and 65 spans are in the restoration levels of R1 and R2, respectively. No special restoration is required for 60 spans. Fig. 13 shows the statistics for spans with each restoration design level for Tawarayama tunnel. Based on the restoration level and specific condition of each damaged lining span, repair and reinforcement measures are determined. Table 3 summarizes the specific restoration measures adopted for Tawarayama tunnel.

For the cases of cracks with width $\geq$3 mm and length $\geq$5 m, partial concentration of cracks with width $\geq$0.3 mm and density $\geq$0.2 m/m^2, or concrete splitting and slight spalling, carbon fiber sheet or fiber-reinforced polymer (FRP) reinforcement method was adopted. For the cases of slight damage (i.e. S007, S008, S009 and S011) with slight cracking, concrete splitting of construction joint and leakage, grouting method was conducted.

The collapse of the lining structure (S167), buckling of steel support, and widespread spalling of the lining concrete (S012 and S028) deteriorated the bearing capability of the primary support and concrete lining. In spans from S165 to S167 with restoration level of RS, since the primary support and lining cannot function well with a very low capability, reconstruction is needed. Fig. 14 shows the reconstruction design for the lining from spans of S165 to S167. Due to 10 cm deformation both in transversal and longitudinal directions, the steel arch buckled severely (see Fig. 15), and replacement of the primary support was necessary. The rock-bolt (red solid circle in Fig. 14) and steel arch (blue line in Fig. 14) were replaced and the support pattern was changed from D_I to D_{IIIa}. On the other hand, removal of the failed steel support might affect the stability of the surrounding lining and ground and even induce secondary damage. Therefore, before removing the steel arch support, the injection type forepoling as an auxiliary method was conducted around the buckled areas to reinforce their stability.

Reconstructing the primary support such as steel support, shotcrete and rock-bolt should be evaluated with respect to their damage degrees. Since no obvious damage was observed for other 13 spans in restoration level RS, the steel arch support was left as it was Only the failed shotcrete was replaced and then the number of rock-bolts was increased. For the reconstructed lining, concrete was reinforced with reinforcing steel bar of ϕ19 mm @ 200 mm as the primary reinforcement and ϕ16 mm @ 300 mm as the secondary one. In addition, the invert concrete of 6 spans (S012, S013 and S165–S168) with large deformation due to compression was also replaced by steel-reinforced concrete. Fig. 16 shows the tunnel condition after reconstruction.

With respect to the entrance of Tawarayama tunnel (at the Nishihara Village side), unsymmetrical loading of slope due to semsmic loading induced serve deformation or even failure of the lining structure. Even worse, heavy rainfall caused collapse of part of the earthquake-induced loosen slope. The slope collapse aggravated the effect of unsymmetrical loading on the deformation of the lining structure. If the unsymmetrical loading is not addressed, the continuous pressure from the left side might induce continuous deformation by the aftershocks. Thus, there is a probability that secondary damage may occur. Excavation of the slope had to be conducted to relive the unsymmetrical loading, as illustrated in Fig. 17. The earth at the upper side above the tunnel denoted by the blue area in Fig. 17 was removed. In addition, some loosen soil due to the earthquake was also removed to avoid secondary damage to the slope. After removal, a new ground surface noted by the red line in Fig. 17 was formed to relive the unsymmetrical loading effect.

4. Conclusions

To better understand the seismic performances and responses of underground structures subjected to earthquake force, a wide collection of case histories has been reviewed in terms of damage classification. Several databases are available globally; however, there has no mature classification standards for damages to tunnels.

In this context, damage classification is proposed to highlight the possible causes of damages. For this, the case of Tawarayama tunnel subjected to the 2016 Kumamoto earthquake is discussed. To improve the performance-based aseismic design of tunnel, it is necessary to analyze the factors influencing the seismic responses of Tawarayama tunnel subjected to the 2016 Kumamoto earthquakes, including earthquake parameters, tunnel depth, surrounding rock mass quality, fault zone and slope at the portal.

The restoration design criterion for Tawarayama tunnel is developed with reference to the restoration design criteria for Touya tunnel. In addition to crack width, crack distribution and geological condition considered in traditional design criterion, two other aspects, the spalling/collapse of lining due to ground shaking and groundwater leakage, are introduced into the present criterion.

Declaration of Competing Interest

The authors wish to confirm that there are no known conflicts of interest associated with this publication and there has been no significant financial support for this work that could have influenced its outcome.

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Yujing Jiang is an academician in Engineering Academy of Japan. He obtained his BSc and MSc degrees from Shandong University of Science and Technology (SDUST), China, in 1982 and 1985, respectively, and his PhD degree from Kyushu University, Japan, in 1993. He is currently the director of the State Key Laboratory of Mining Disaster Prevention and Control Co-founded by Shandong Province and the Ministry of Science and Technology, vice chairman of the Soft-rock Engineering and Deep Disaster Control Sub-society of Chinese Society for Rock Mechanics and Engineering (CSRME). He is also winner of Joint Research Fund for Overseas Natural Science of China. His main research interests include (1) environmental geotechnical engineering; (2) rock mechanics and rock engineering; (3) development and utilization of underground space; (4) remote monitoring forecast and control for geological disaster; (5) mining of deep-sea resources (natural gas hydrate) and control of seabed strata environment; and (6) mine pressure and strata control. He has published 15 books in Chinese and foreign languages and over 300 research papers. He is vice editor of *Rock Mechanics and Rock Engineering*, and also serves as the editorial board member of several journals, including *Journal of Rock Mechanics and Geotechnical Engineering (JRMGE)*.