A strong earthquake with JMA (Japan Meteorological Agency) magnitude of 6.8 occurred with its epicenter located in Chuetsu-oki in Niigata Prefecture, on July 16 2007. This earthquake caused extensive damage, completely destroying about 1,000 homes, triggering 105 landslide disasters, and damaging 1,090 agricultural facilities and peripheral facilities of a nuclear power plant. The damage included a shield tunnel used as a drainage canal: damage unprecedented in Japan. This paper reports the results of a detailed survey of this damage, and describes dynamic response analysis carried out using near-source seismic waves. The cause of damage is considered to be the topographical effects of the shape of the mountain above the underground shield tunnel. In the future, it will be necessary to perform seismic design carefully in areas with similar topographical conditions.

1. INTRODUCTION

A shield tunnel is, as a structure many joints, a structure with superior ability to follow ground displacement, and has been described as one with good earthquake-resistance. There have been no reports of any severe damage to shield tunnels by large earthquakes such as the Hyogo-ken Nanbu Earthquake. No serious earthquake damage of this kind has been reported elsewhere in the world 1).

Although there are rare cases of minor shield tunnel damage caused by large earthquakes, damage such as failure of the tunnel section causing it to lose its tunnel functions has never been reported. The following are major types of damage which have been reported 1).

1. Chipping of the ends of the concrete segments.
2. Damage to ring joints near vertical shaft connections.
3. Lateral cracks of the secondary lining concrete at bends, longitudinal cracks at the vertical 45° position from the spring line of the secondary lining concrete in a straight sections; crack width of 0.1 to 0.7mm.
4. Differential settlement and leaking

Because large earthquakes do not cause severe damage to shield tunnels, it has been said that shield tunnels are highly resistant to earthquakes, but the Niigata Chuetsu-oki earthquake caused unprecedented damage. This occurred regardless of the fact the tunnel was designed with adequate care based on the normal design method and was properly constructed. This paper reports on the state of the damage to the shield tunnel caused by the Niigata Chuetsu-oki earthquake and on dynamic response analysis performed using near-source seismic waves to consider the causes of the damage.

2. LOCATION OF THE SHIELD TUNNEL AND OUTLINE OF THE DAMAGE

1. Location and outline of the structure of the shield tunnel
   As shown in Fig. 1, the damaged shield tunnel

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is located north of Kashiwazaki City and about 1.5km from the Japan Sea coastline. The tunnel alignment is generally straight at an angle of about 60° to the Japan Sea coastline. Ground conditions around the tunnel are, as shown in Fig. 2, diluvial clay with N value of about 30 and well compacted sandy soil with N value above 50. It is good quality ground corresponding to the engineering ground.
surface in seismic studies. A sand-bank covered plateau is above the tunnel, and the overburden of the tunnel varies abruptly along the length of the tunnel.

The minimum overburden is about 10m at both ends of the tunnel, and the maximum overburden is about 60m approximately 1/3 of the distance from the Japan Sea end of the tunnel.

The linings of the shield tunnel consist of a primary lining which is a reinforced concrete flat panel-shaped segment, and a secondary lining which is non-reinforced concrete. As shown in Fig. 3, the external tunnel diameter is 4,500mm and the combined thickness of the segment and the secondary lining is 200mm. When the thickness of the secondary lining was measured during the damage survey, it was determined that the execution quality was high, because its error was within ±10mm. In addition, its concrete strength was 26 – 30N/mm² which exceeds the design strength of 21N/mm², and there were no gaps between the primary and secondary linings at the top of the tunnel.

(2) State of damage to the shield tunnel

The damage to the shield tunnel was classified as two patterns: as damage considered generally similar to the longitudinal cracks at the upper and lower 45° positions from the spring line of the secondary lining concrete reported as damage to shield tunnels caused by past earthquakes (Fig. 4), and as severe damage including breakage of ring joints (Fig. 5). The severe damage was concentrated at approximately 200m or about 1/3 the length of the drainage channel from the Japan Sea.

[Damage Pattern 1]
- Cracking in the axial direction is conspicuous, occurring at 45° in the top half and at 45° in the bottom half.
- Crack width is between 0.5mm and 1.5mm.
- The scale of the damage is smaller than that of pattern 2

Fig. 4 Shield tunnel damage (Pattern 1).

[Damage Pattern 2]
- Cracking in the circumferential direction is conspicuous, and there is a gap of about 0.9m (segment width).
- Large cracks occurred at several locations, with widths between 20mm and 50mm.
- Total crack width in the circumferential direction reveals it is approximately 1% of tunnel length.
- The width of axial direction cracking in the ceiling is between 1 and 5mm

Fig. 5 Shield tunnel damage (Pattern 2).
Japan Sea, while damage in other sections was minor. It was confirmed that on box culvert tunnels constructed at opposite ends of the shield tunnel, cracking, opened joints, and unevenness occurred, presumably caused by the earthquake.

The results of longitudinal measurements at the top of the shield tunnel confirmed that as shown in Fig. 6, in sections where severe damage occurred, the tunnel top had settled severely. This settlement is thought to be the result of the combined effects of deformation of the tunnel section, which is described below, and settlement of the tunnel shaft line, but maximum settlement of 80mm was found in a section of approximately 50m. Considering the fact that there is a little difference in the planned alignment in the longitudinal direction in the sections at opposite ends, it can be concluded that this settlement was caused by the Niigata Chuetsu-oki Earthquake.

Because based on records of past visual inspections and results of on-site inspections, it was decided that the state of damage pattern 2, which appears to have been caused in the shield tunnel by...
the earthquake, differs from past earthquake damage, a detailed survey was carried out in order to confirm the impact on structural members at the locations damaged by this earthquake.

The detailed survey was carried out to confirm whether a more than 20mm wide crack found by the damage survey of the secondary lining surface was limited to the secondary lining or included the primary lining. The following is the state of damage to the shield tunnel obtained from the survey results.

(a) At the location where a crack wider than 20mm occurred in the secondary lining, the ring joint of the primary lining was broken (see Photos 1 to 4).

(b) At the location where the ring joint was broken, deformation of the interior section exceeding the outer diameter ratio of 2% occurred (see Figs. 7 and 8).

(c) At the location of the interior section deformation, the top of the tunnel settled between 20mm and 80mm.

(d) Cracks wider than 1mm occurred continuously on the ceiling.

At sound sections, the circularity of the tunnel was good, and the back of the secondary lining on the ceiling was free from quality problems such as inadequate filling or inadequate strength.

This concludes the description of the damage, but as stated initially, as far as the authors know, such serious damage to a shield tunnel has never been reported. Assuming ground conditions are relatively good, it is difficult to hypothesize the causes of the damage based only on conventional knowledge and past survey results. So it is important to clarify the causes of the damage in order to improve the seismic resistance of future shield tunnels. This paper describes dynamic response analysis performed to clarify these causes. The following are the results of the dynamic response analysis.

| Span | Remarks | Damaged | Sound |
|------|---------|---------|-------|
|      | Interior shape displacement | 67mm | 10mm |
|      | Interior shape | 46mm |
|      | Photo of interior surface | |

Fig. 7 Damage to the shield tunnel (interior section).

Fig. 8 Shield tunnel damage in survey section I.
3. DYNAMIC RESPONSE ANALYSIS

(1) Ground conditions
At the shield tunnel, the ground conditions are shown in Fig. 2. The shield tunnel runs under a sand dune consisting of sandstone and sediment sand, and its overburden thickness varies between 6m and 65m. Layers passing through are, on the ocean side, a diluvial consolidated clay called the Yasuda Layer, and with a Neogene layer of mudstone as the boundary, the land side is an alluvial sandy soil layer. The shear elastic wave velocity $V_S$ estimated from the N value is 250 to 300 m/s for the consolidated clay, 370 m/s for the mudstone, and 260 m/s for the alluvial sandy soil. The bedrock can be set on the top surface of the mudstone layer where the shear elastic wave velocity $V_S$ is more than 300 m/s if the Design Specifications for Highway Bridges is followed. But for the initial design, the ground was surveyed only to a depth of 2 to 15m below the tunnel, so it was impossible to confirm the depth distribution of the mudstone layer. Therefore, the bedrock in this paper was estimated and set based on a schematic diagram of geological composition of the location hypothesized based on the geological survey report prepared at the time of the design. The distribution of the mudstone layer set as the bedrock is shown by the FEM model diagram (Fig. 14) described below.

(2) Setting the input earthquake motion
The input earthquake motion used for the analysis was earthquake wave motion recorded and released at an observation location called the service hold ground system on the grounds of the Kashiwazaki Nuclear Power Plant about 3km horizontally from the shield tunnel (provided by Tokyo Electric Power Co.). At the service hole ground system, seismometers are installed at several depths. In this paper, hypothesizing that the earthquake motion wave form observed in ground at a depth of 99.4m (elevation -31.9m) with shear elastic wave velocity equivalent to the base ground for this analysis, does not attenuate with distance, it was input to the bedrock for the site assuming that it was a composite wave E+F: the incident wave of the earthquake motion from directly below and the wave F reflected from the surface ground. Figs. 9, 10, and 11 show the input earthquake motion and the velocity response spectrum used for the analysis.

The vertical component of the earthquake motion was ignored with reference to the fact that the cause of damage to underground structures by the Hyogo-ken Nanbu Earthquake is assumed to have been the horizontal component of the earthquake motion.

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Fig. 9 Input earthquake motion used for the analysis (NS component).

Fig. 10 Input earthquake motion used for the analysis (EW component).

Fig. 11 Velocity response spectrum of input earthquake motion used for the analysis.

Fig. 12 Tunnel longitudinal direction structural analysis method.
The longitudinal alignment of the tunnel almost conforms to the east – west direction, so the EW component is used when the tunnel is vibrated longitudinally and the NS component is used when it is vibrated laterally. Fig.11 also shows the level 2 earthquake motion velocity response spectrum which is adopted by the present Design Standards for Sewerage System Facilities 4). As a result of the analysis discussed below, damping by the ground is approximately 20%. The primary natural period after the fall of stiffness based on the FEM full model is about 2.8 seconds. The velocity response spectrum of the input earthquake motion can evaluate the EW component as almost identical to, and the NS component as smaller than, the level 2 earthquake motion adapted by the design standard.

(3) Analysis method
The analysis method is explained below. The analysis was done in the longitudinal and in the lateral direction of the tunnel.

a) Longitudinal direction
The longitudinal direction of a shield tunnel consists of segment rings and ring joints with varying stiffness. So analysis in the longitudinal direction was, as outlined in Fig. 12, based on dynamic analysis: modeling a shield tunnel as a beam with equivalent axial stiffness and bending stiffness 5), and introducing ground displacement at a depth in the center of the tunnel during an earthquake to the beam through a ground spring based on the time history. Analysis in the lateral direction was done to reproduce failure of the ring joint. Consequently, the elasto-plastic analysis described next was done.

The equivalent stiffness considered the non-linearity of compression and tension as shown in Table 1, because while compressive force is resisted by the main segment rings, tensile force is resisted by both the ring joints and segments. And the equivalent tensile stiffness was modeled also considering the yield of the surface plate of the ring joint 6). Nothing concerning the ring joint was entered on the completion drawing other than its plate thickness of 12mm, so the other specifications were read from drawings. To ensure stability of the analysis, ring joint breakage such as that observed at the site was not considered. And because the secondary lining was non-reinforced concrete, it is assumed it does not resist tensile force, so it was modeled resisting only compressive force. Ground displacement was modeled with the surface layer of the ground as a finite element, and a modified R-O model 7) which can calculate residual strain was applied to each element to perform the calculation. Fig. 12 shows the FEM model used to calculate the ground displacement. The ground displacement in the longitudinal direction of the tunnel was calculated by entering earthquake motion at the bottom end of the model in Fig. 14 and vibrating the entire model in the axial direction.

So the basic values of the ground spring were calculated based on the relationship of the forces necessary to cause unit displacement in the tunnel using an FEM model which modeled the surface ground and tunnel as shown in Fig. 15. The FEM model used to calculate ground spring applied stiffness obtained as the (maximum shear response)/(maximum shear strain) of each element calculated by the dynamic analysis performed using the model in Fig. 14 in order to consider the decline of stiffness of ground during an earthquake. The FEM model used to calculate ground spring was prepared for six sections where the ground layer conditions shown in Fig. 14 change, and the ground spring at intermediate points was obtained by performing linear interpolation of the basic values.

b) Lateral direction
The lateral direction of the shield tunnel was the object of analysis of damage pattern 2: a vertical decline of the interior section. The analysis method was the response displacement method which is a seismic design method for normal underground structures. For the response displacement method, as segment rings, a spring model which can consider differences in stiffness of main segments and joints between them as shown in Fig. 13 was adopted. A spring model usually considers the splice effect of ring joints, but because the ring joints were broken at

| Equivalent compression stiffness | $(EA)_c$ | $1.52 \times 10^6$(kN) |
|---------------------------------|----------|---------------------|
| Equivalent tension stiffness    | $(EA)_t$ | $7.05 \times 10^6$(kN) |
| Equivalent bending stiffness    | $(EI)$   | $3.68 \times 10^3$(kN•m²) |

Fig. 13 Tunnel lateral direction structural analysis model.
the damaged locations, a 1 ring model which organizes ring joints was adopted.

4. ANALYSIS RESULTS

The analysis results are described below.

(1) Longitudinal direction

The analysis in the longitudinal direction was done by entering the EW component of the earthquake motion because the alignment of the tunnel is almost east–west. From ground displacement calculated by entering earthquake motion at the bottom end of the FEM model shown in Fig. 14, Figs. 16 to 18 are deformation drawings at the time when ground displacement vertically at the peak was its maximum value (t = 7.08 seconds), and the maximum value distribution in the horizontal and vertical directions at the central depth of the tunnel is shown in Fig. 19. In a case where the horizontally stratified ground was shaken in the horizontal direction, almost no vertical component of ground displacement occurs as can be seen at both ends of the analysis model, but as seen in Fig. 17, a vertical component occurs in the Dc layer below the Asd layer. In the Dc layer, when horizontal inertia acted on the mountain peak causing deformation in a mode just like rotational deformation of the mountain peak, it is assumed that deformation occurred because the stiffness is not as great as that of mudstone. In this paper, ground below a mountain being deformed in the horizontal direction and vertical direction in a case where it is vibrated in the horizontal direction in this way is called the topographical effect.

The most detailed dynamic analysis model is one obtained by modeling the entire body based on three-dimensional FEM, but because the analysis model becomes too large, it is not realistic. If the tunnel is modeled within the two-dimensional ground displacement calculation model shown in Fig. 14, the tunnel is handled as a structure which is continuous in the depth direction. Therefore, the seismic study of the longitudinal direction of the tunnel is, generally, often done using a beam-spring model such as that shown in Fig. 12 9). The gap in the ground spring of the beam-spring model in dynamic analysis of tunnel longitudinal direction was set at a gap of 9m so that it would be possible to reflect deformation at each location of ground displacement in detail.

Ground displacement input to dynamic analysis of the shield tunnel was the horizontal component and vertical component based on a time history at the central depth of the tunnel obtained by ground FEM analysis.

Focusing on the state of damage at the site revealed that circumferential cracking of the secondary lining occurred almost entirely at ring joints. Among these, leaking occurred at many locations of joints in the secondary lining, located at every 10th ring (9m)

![Fig. 15 Ground spring calculation model (outline).]

![Fig. 14 FEM model used to calculate ground displacement.]

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Fig. 16 Horizontal component of ground displacement at the time of maximum vertical displacement at the peak (t = 7.08 seconds).

Fig. 17 Vertical component of ground displacement at the time of maximum vertical displacement at the peak (t = 7.08 seconds).

Fig. 18 Ground displacement at the time of maximum vertical displacement at the peak (t = 7.08 seconds).
as shown in Fig. 7. Fig. 20 shows that locations of relatively large cracks are concentrated in the section from 132.7m to 150.7m. Generally, the strength of concrete joints is lower and cracking occurs more easily than at other locations. It can be assumed that if large cracks occur at joints where the strength is lower than at other locations, deformation will be concentrated at these cracked locations. The survey result shows that at locations where large cracks exceeding 20mm occurred in the secondary lining, the ring joints broke and residual openings up to 50mm occurred. For these reasons, hypothesizing that openings of ring joints were concentrated at joints, opening of one ring equivalent to 10 rings was added to calculate the opening which occurs at joints as shown in Fig. 20.

Fig. 21 shows large peaks in openings at three locations—on the ocean side, near the mudstone, and near the start point—and although the locations do not strictly conform, the figure generally reproduces the state of damage in survey section I. The openings which were observed were about 50mm at the most, but the analysis results indicated they were between 20 and 30mm smaller. The analysis modeled the yield of ring joints, but did not model the breakages in order to avoid lowering the stiffness, reducing the stability of the analysis. It is assumed that one reason the analysis results were smaller than the observed values was that the modeling was done in this way. At locations other than the peak locations, openings of 10 rings are lower than 15mm, which corresponds closely to the fact that at secondary lining crack width of 20mm or less, ring joint breakage was not seen. The yield strength of the ring joints is about 1/3 of the tensile strength of the secondary lining obtained according to the Standard Specifications for Concrete [3]. The hypothesis that the strength of construction joints of the secondary lining is lower than the strength of normal parts and that openings are concentrated on the construction joints is generally correct.

(2) Lateral direction

Generally, seismic design for the lateral direction of a shield tunnel is done considering the impact of the horizontal relative displacement of the ground at the top and bottom ends of the tunnel during an earthquake. In this case, cyclic deformation of the tunnel occurred at an angle of 45°. Damage pattern 1 was definitely cracking in a case where such deformation occurred. Damage pattern 2 on the other hand, was a decline in the vertical direction of the interior section of the tunnel, which spread laterally. A deformation mode of this kind cannot be hypothesized by normal seismic design.
So if the study focuses on ground displacement at the tunnel center depth shown in Fig. 17 to Fig. 19, as a result of the topographical effect described above, displacement also occurs in the vertical direction. Based on the results of FEM analysis performed to compute ground displacement in the tunnel longitudinal direction, horizontal and vertical relative displacement of the ground at the depths at the top and bottom of the tunnel were investigated. Fig. 22 shows both the maximum values of horizontal and vertical relative displacement at the top and bottom ends of the tunnel and the maximum displacement in the longitudinal direction of the tunnel at the tunnel center depth (both absolute values). Fig. 22 reveals that although the ground was shaken in only the horizontal direction, relative displacement of the ground occurred at the top and bottom ends of the tunnel.

Based on the above, the analysis of the lateral direction was done by the response displacement method as specified below at the 189m point where the shape deformation of the interior section of the tunnel was measured.

- The horizontal relative displacement at the top and bottom ends of the tunnel (9.8mm) used were the maximum values in a case where the modified R-O model was applied to the one-dimensional ground model at the 189m point and the NS component of the earthquake motion was input.

![Fig. 23 Normal bending Moment.](image)

![Fig. 24 Bending moment caused by horizontal ground displacement.](image)

![Fig. 25 Bending moment caused by vertical ground displacement.](image)

![Fig. 26 Bending moment during earthquake.](image)
The vertical relative displacement at the top and bottom ends of the tunnel (7.7mm) used was the maximum value obtained by longitudinal direction analysis which input the EW component of the earthquake motion.

The ground spring obtained by applying the skin shear force and the reduced stiffness value obtained by the one-dimensional model was used.

The joints were modeled as the rotating spring which applied the reference values shown in the Handbook for Structural Design of Tunnel Lining Subject to the Action of Internal Water Pressure.

Normal loading complied with the initial design calculations.

The earthquake loading was loaded on the beam provided with section force based on normal loading as the initial value.

Fig. 23 shows bending moment generated by normal loading. Fig. 24 shows bending moment in cases where only horizontal ground displacement, skin shear force, and vertical inertial force were applied, and Fig. 25 shows bending moment in a case where only vertical ground displacement was applied. These reveal that the maximum value of bending moment occurs at a location different from the normal location, under horizontal ground displacement, but the bending moment by normal loading under vertical ground displacement occurs in the almost the same mode.

Based on this fact, examining the bending moment during an earthquake shown in Fig. 26 reveals the impact of horizontal and vertical ground displacement.

It shows which parts of the M-φ function where there is bending moment produced in the main body and joint in Figs. 27 and 28. Here the M-φ function of joints was obtained by viewing the anchor iron as re-bars in order to properly design the segments. Fig. 27 shows that the maximum value of the bending moment which is the interior tension is produced near the crown, and conforms almost perfectly with the location of cracking of the secondary lining in damage pattern 2. In Fig. 28, large bending causing yield of joints near the spring line occurred. This is thought to have been a cause of the reduction of the interior section in the vertical direction. The displacement obtained by the analysis is 1.5cm, which is smaller than the measured 8cm shown in Figs. 6 and 7, but as shown in Fig. 29, the deformation mode could be generally reproduced. Seismic waves transmitted from 85° relative to vertical axis (5° to the horizontal axis) through the bedrock where the shear elastic wave velocity \( V_s \) is 3,000 m/s. The seismic waves are assumed to have entered into consolidated clay where \( V_s \) is 300m/s.

If the angle of seismic waves transmitted through consolidated clay is calculated using Snell’s method, it advances almost straight upward at 5.7° to the vertical axis. For this reason it is concluded that ignoring the vertical component of the earthquake motion under analysis is appropriate. The input earthquake motion wave form which was entered for the analysis was not obtained based on tunnel position, and this is assumed to be one reason why the quantity of deformation from the analysis and that obtained by measurement results do not match.

5. CONCLUSIONS

Dynamic response analysis of the shield tunnel damaged by the Chuetsu-oki Earthquake was performed, obtaining the following knowledge.

(a) Fig. 11 shows that during the Chuetsu-oki Earthquake, the site experienced earthquake motion which is equivalent to the level 2 earthquake motion adopted by the present Seismic Design Standard.

(b) When the two-dimensional FEM model of the surface ground was shaken horizontally, a vertical component of the ground displacement in the order of a few cm was produced. The cause is presumed to be the topographical effect.

(c) The breakage of the ring joints was caused by ground deformation during the earthquake, and occurred at construction joints with weak secondary lining.

(d) The cause of damage pattern 1 was the dominance of the horizontal ground displacement at the top and bottom ends of the tunnel during the earthquake.

(e) The cause of damage pattern 2 was the action of vertical relative displacement almost identical to the horizontal component at the top and bottom ends of the tunnel during the earthquake.

(f) The analysis succeeded in qualitatively reproducing the mode of damage in the longitudinal and lateral directions of the tunnel, but it did not adequately reproduce it quantitatively. This occurred because ground conditions, input earthquake motion etc. included numerous hypotheses. But it did qualitatively clarify causes of this unprecedented severe damage.
6. SUMMARY

Shield tunnels have been considered to be structures with seismic performance superior to that of other underground structures judging from damage caused by past earthquakes. In addition, because there have been no cases of failure in the past, methods of evaluating behavior when an earthquake damages a shield tunnel have not been verified. But it has been confirmed that the Niigata Chuetsu-oki Earthquake caused unprecedented damage to a shield tunnel. A simulation of this damage to the shield tunnel caused by the Niigata Chuetsu-oki Earthquake has clarified the behavior of the shield tunnel during the earthquake and revealed the causes of its damage, making an important contribution to improve and advance future seismic design technologies.

The correlation of the analysis results with the damage which actually occurred has provided a qualitative explanation of its behavior, but the precision of the qualitative analysis cannot be defined as sufficient. This is assumed to be a result of differences between the setting precision of ground conditions, setting of the seismic waves, and the analysis model etc. It is presumed necessary to study methods of setting various analysis conditions including ground conditions, in order to perform more precise analysis in the future.

This paper presents a case which actually shows that shield tunnels previously thought to be structures with good seismic performance may be severely damaged under certain conditions, and the dynamic response analysis clarifies, although only qualitatively, that the topographical effect may have caused the damage. Under such conditions, measures to improve seismic performance, such as inserting seismic resistance joints at locations there are needed, are important. In particular, if the groundwater level is high, water seeps into shield tunnels, causing serious conditions.

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