Analysis of heaving phenomenon in mountain tunnel and consideration to shallow shaped invert as countermeasures

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Abstract. Heaving in some mountain tunnels after completion occurs in roadbeds that were built without an inverted arch structure because the ground was relatively stable during construction. The stress-release by excavation, along with subsequent swelling by groundwater, cause the roadbed to lose strength, allowing upheaval of the road surface. In this study, we analysed geological degradation characteristics and the state of past tunnels in cases where inverted arches were installed. We also analysed measurement data for axial forces of inverts. The necessary performance was discussed through the result obtained by numerical analysis to compare support effectiveness of the invert shape and consider a shallow structure.

1. Introduction

As its nationwide renewal project, NEXCO (Nippon Expressway Company Ltd.) is examining the countermeasures against heaving in mountain tunnels on highways that occurred after they were put in service. Its basic policy for the countermeasure to install inverted-arch concrete, so-called ‘invert’, but in and around the metropolitan area, the installment has been carried out not with road closure but with lane regulation, in order to alleviate traffic jam. Since the construction speed is slower with lane regulation than with road closure, design rationalization and construction efficiency are becoming increasingly important.

This paper studies the cases of countermeasures against heaving phenomenon, identifies the state and causes of the deformations, finds out the current situations of the countermeasure works and outlines the challenges. Then, it proposes the shallow structure of the invert based on the numerical analysis of the structure and measured stress after the installment.

2. Current situations and challenges of the countermeasures against heaving

Table 1 shows the cases of the countermeasures and figure 1 shows the cross-section of the tunnels and their invert structures. tunnel A, tunnel B, tunnel C, tunnel D and tunnel E were all constructed without inverts. For tunnel F and tunnel G, inverts were installed at the time of construction but later damaged by plastic pressure. Most of the cases are the former.
2.1 Case study

As shown in table 1, in the case where the invert is not provided at the construction stage, the invert is damaged. The maximum road surface uplift speed is lower than that of the boom. The contraction speed of the inner sky is less than 1/10 of the road surface uplift speed in all cases.

In the cases without inverts, class of ground were CI, CII, or DI of “General design” (shown in table 2 in Design criteria III tunnel) and the displacements were small during the initial tunnel excavations. The tunnel behaviors were at the squeezing levels of “none” or “light” suggested, and basically considered to be in elastic zone. For the cases with damaged inverts, the ground grades were equivalent

Table1. Cases of the countermeasures against heaving phenomenon.

| Tunnel | A | B | C | D | E | F | G |
|--------|---|---|---|---|---|---|---|
| Maximum uplift of road surface (mm) | 153 | 175 | 110 | 70 | 124 | 500 ~ | 110 |
| Speed of road surface uplift (mm/y) | 11~28 | 17~27 | max 39 | — | 4 | 272~(mm/day) | 23 |
| Speed of inner displacement (mm/y) | 2 | 3~6 | slowly | slowly | 1 | 10~(mm/day) | 1~2 |
| Class of ground | CH, DI | CH, DI | CH, DI | CH, DI | CH, DI | DI | DI |
| Invert or not | without invert | without invert | without invert | without invert | without invert | with invert | with invert |
| Rock species | tuff breccia, andesite | tuff breccia, andesite | tuff breccia, andesite | tuff breccia, andesite | tuff breccia, andesite | tuff breccia, andesite | tuff breccia, andesite |
| qv (MPa) | 0.1~< 0.2 | 0.2~< 1 | 0.3 | 0.3 | 0.1 | 0.1 | 0.5 |
| Rock strength ratio | — | 0.06~< 1.6 | — | — | 0.1 | 0.1 | 0.5 |
| Inundation collapse degree | — | — | — | C, D | mainly D, C | mainly D | D |
| Cation Exchange Capacity | — | — | — | — | — | — | — |
| Cause of deformation | Due to the stress release of the ground containing swellable clay minerals, the clay absorbed water and expanded and deteriorated. | The ground with a small ground strength ratio and a high slaking rate has deteriorated due to stress release and water absorption. | Tuff breccia is altered and contains expansive clay, and underground drainage acts to induce ground deterioration. | High overburden and low rock strength ratio, and ground containing some smectite absorbed water leakage and the strength decreased. | Tuff breccia acted on tuff (including clay minerals), easily deteriorated by water, and a plasticized region was generated. | An invert was provided to suppress displacement during construction, but plastic pressure acted at a young age and cracks occurred. Spring water infiltrated there and water was supplied for a long time, expanding the plastic region and damaging the existing invert. |
| Design method | The frame is calculated with a load of 0.3N/mm² from the Teltourgi and Q value. Considering the large external force, R₁ was selected. | The frame is calculated, with the loosening height of DII being 6m as the load. The load bearing capacity of the lining of the arch and the invert are equivalent. | Considering future ground deterioration. | Considering future ground deterioration. | FDM analysis was performed. | With reference to similar cases, FDM analysis by applying an expansion pressure from the rock test. |
| Road raguration | Road closed | Road closed | Road closed | Road closed | Road closed | Road closed | Lane regulation |
| Road raguration (m) | 379 | 506 | 370 | 2,670 | 41 | 130 | 42 |
| Structure of invert | | | | | | | |
| Invert radius R₁ (m) | 2.0 R (R₁) = Arch structure radius | 2.0 R (R₁) = Arch structure radius | 2.0 R (R₁) = Arch structure radius | 1.9 R₁ | 2.0 R₁ | 1.9 R₁ | 1.9 R₁ |
| Thickness of invert | 100 (mm) | 100 (mm) | 100 (mm) | 100 (mm) | 100 (mm) | 100 (mm) | 100 (mm) |
| Spring water during excavation | With spring | With spring | With spring | With spring | With spring | A lot | A lot |
| Spring water during measures | With spring | With spring | With spring | With spring | With spring | A lot | A lot |

(Note) a: Criterion with expansive; qu< 4.0. Rock strength ratio< 2.0. Inundation collapse degree: “C”, “D”, Cation Exchange Capacity > 20(meq/100g)

Table2. Standard support pattern.

| Class of ground | Support pattern | Cycle length (m) | Length (m) | Yield strength (GPa) | Rockbolt installation pitch (mm) | Area (mm²) | Arch, Solid (cm) | Invert (cm) | Allowable deformation (cm) |
|-----------------|-----------------|------------------|-------------|---------------------|-----------------------------|------------|-----------------|-------------|--------------------------|
| B | B-a | 2.0 | 3 | 176 | 1.5 | Upper section 12° (mm) | 5 | - | - | 30 | 0 | - |
| CI | CI-a | 1.5 | 3 | 176 | 1.5 | Upper section 12° (mm) | 7 | - | - | 30 | 0 | - |
| CII | CII-a | 1.2 | 3 | 176 | 1.5 | Upper and lower section 12° (mm) | 7 | - | - | 30 | 0 | - |
| DII | DII-a | 1.0 or less | 4 | 296 | 1.2 | Upper and lower section 12° (mm) | 15 | HH158 | 30 | 50 | 10 |

(Note) a: Standard support pattern basically applicable to all rock types; b: applicable only to the ground consisting of such rock as slate, mud stone, shale, etc. where excavation is likely to cause larger deformation than expected. (Note) "( )" in invert is applicable to the ground consisting of mud stone, tuff, serpentinite, solfatarian clay and so on.
to DII with deformations suppressed by the cross section closure of the main inverts. The tunnel behaviors are "intense" in the squeezing level, and considered to be in "perfectly plastic zone."

The heaved bedrocks were mostly of soft quality such as mudstone, shale and tuff, but deformations were also observed in their adjacent semi-hard rocks including andesite and tuff breccia. The physical properties of the bedrocks were mostly about 3.0 MPa or less in uniaxial compressive strengths and lower than 2.0 in competence factors, and the rock grades deteriorated to the equivalent of DII of "General design (shown in table 2). Regarding the inundated collapse degrees of the bedrocks after immersion in water for 24 hours, most were graded "D: collapsed completely," while some were graded "C: sharp edges collapsed." In tests of cation exchange capacity, all exceeded the threshold for distensibility. In the five cases without inverts, existence of spring water was recorded during the countermeasure works. The roadbeds lost strength and the inundated collapse degrees and results of Cation Exchange Capacity (CEC tests) exceeded the threshold values, showing the characteristics of swelling. Therefore, it was assumed that the strength degradations occurred by swelling, with the ground characteristics that are easy to deteriorate by water (primary cause) and the existence of spring water (inducement). In the

![Figure 1. Cross section of tunnel and invert of countermeasures.](image-url)
two cases with damaged inverts, spring water was also observed in the bedrocks during the countermeasure works and they had the disposition of the swelling. But the deformations did not stop during the construction, and they were suppressed by the cross section closure of the main inverts. The study of phenolphthalein obtained from the invert concrete lumps showed that discontinuous planes appeared in the early stages of constructions and it was assumed that the inverts were damaged under the construction loads and thus the squeezing was the main phenomenon. As shown in table 1, \( R_1 \) is radius of arch structure. For tunnel A and tunnel B, the invert radius \( R_3 \) was set to be twice the upper half radius (\( 2.0R_1 \)) for high load resistance. For tunnel C and tunnel D, the invert shape was made to be \( 2.0R_1 \), based on the loosened height set one rank heavier than the existing support pattern. For tunnel F and tunnel G, numerical analyses were carried out to confirm the load bearing capacity of inverts. For tunnel E, the inverts were designed by setting the degradation area under the road surface and lowering \( c \) and \( \phi \) using the ground degradation model.

As shown in figure 1, the F tunnel has a rigid structure in the invert compared to the others due to its deformation characteristics. For other tunnels, the thickness of the invert is more than the standard design (C class: 40 cm, DI: 45 cm, DII: 50 cm) and all radii of curvature about \( 2.0R_1 \), smaller than standard design (approx.\( 2.7R_1 \)) with the support function of the structure strengthened (figure 1). As a result, the basic structures of tunnel G with damaged inverts and the five tunnels without inverts were basically the same.

2.2 Measured stress of inverts
The behaviors of invert stress were compared between tunnel G where squeezing was the main phenomenon and tunnel E where swelling was the main phenomenon. Figure 2 shows the progress of

![Image](image-url)

Figure 2. Axial stress of invert concrete in 5 years.
invert stress and the stress distribution after 5 years of the countermeasures.

In tunnel G, both cross sections of the inverts showed stress increase on the compression side, and the tendency to increase continued even after 5 years. The stress distribution of cross section 1 showed slightly larger stress on the side of the preceding construction than the following one and unsymmetrical pressure working due to the effect of the preceding construction of the passing lane and the fragile geology of that passing lane. The stress distribution in cross section 2 also showed comparatively large even after 5 years with maximum stress being 7.0 MPa. In tunnel E, the stress in the two cross sections showed gradual increase except for the center of the inverts and the stress converged after 5 years.

Except for the center part, the maximum stress was small enough, about 1.4MPa. Both tunnels were similar in the shapes of invert structures and compressive stresses act on the axial compressive structures, but the stress values were clearly different. The axial force (working earth pressure) was about 3,700 kN (0.37 MPa) in tunnel G, about 5 times as much as 690 kN (0.07 MPa) of tunnel E.

3. Examination by numerical analysis
Considering the future strength degradation of the roadbeds, numerical analysis was conducted to compare the invert shapes with conventional radius ratio 2.0 with the newly proposed radius ratio 2.7.

3.1 Tunnel for examination
The bedrock of tunnel is composed of mudstone and tuff breccia, and the road surface uplift has continued for a long time because the invert was not initially installed. Compared to the cases in table 1, the tunnel has slightly higher strength and better degree of inundated collapse and better results of CEC tests, but the bedrock near the central drainage works showed slightly high water content rate and deterioration assumed to be the result of swelling by water absorption. The rationalization of tunnel invert was studied to shorten the construction time by making the invert shape shallower with smaller curvature, considering the time needed for the roadbed excavation and the consistency of the cross sectional shapes with the adjacent sound sections.

3.2 Numerical analysis method
Two-dimensional analysis was conducted with three-dimensional finite difference analysis software (FLAC3D by ITASCA). The bedrock was set to be of elasto-perfect plasticity following the Mohr-Coulomb yield criterion (shown table 3), and shotcrete, permanent lining and inverts set to be solid element (shown table 4 and table 5) and steel supports as beam element, with rockbolts analytically neglected. For invert concrete, early-strength concrete (design strength 24 MPa), which has higher strength than lining, was used in order to quickly backfill after casting.

| Table 3. | Table 4. Physical characteristics of lining and invert. |
| --- | --- |
| **Ground physical characteristics.** | **Lining** | **Invert** |
| Modulus of deformation | 500 (Mpa) |  |
| Poisson's ratio | 0.35 |  |
| Unit volume weight | 22 (kN/m³) |  |
| \(c\) | 0.4 (Mpa) |  |
| \(\phi\) | 35 (deg.) |  |
| Tensile strength | 0.08 (Mpa) |  |
| Design strength (MPa) | 18 | 24 |
| Elastic modulus (MPa) | 22 000 | 25 000 |
| Poisson’s ratio | 0.2 | 0.2 |
| Element | Solid | Solid |

| Table 5. Physical characteristics of shotcrete and steel support. |
| --- | --- |
| **Shotcrete** | **Steel support** |
| Elastic modulus (MPa) | 4 000 (During excavation) | 210 000 |
| Poisson’s ratio | 0.2 | 0.3 |
| Cross-sectional area (m²) | — | 30.0x10⁻⁴ |
| Second moment of area (m⁴) | — | 389x10⁻⁸ |
| Element | Solid | Solid |
Figure 3. Analysis model.

Figure 4. Analysis procedure.

1. Prediction of future uplift
2. Setting the depth of deterioration
3. Estimating geological property values in future
4. Setting the invert shape
5. Evaluation of invert shape

Figure 5. Prediction of future uplift.

Figure 6. Strength deterioration.

Figure 7. Step of analysis.
The analysis mesh (shown figure 3) was set at about 5D on the left, right and bottom sides of the tunnel, and the height of the overburden at 85m. Figure 4 shows the analysis flow. At first as shown in figure 5, the prediction of the future uplift volume was approximated by an exponential function to obtain 225 mm, and the difference $\Delta = 44$ mm was calculated from the present displacement of 181 mm. And the degradation area under the roadbed was set at the depth of 6m based on the underground displacement measurement. Subsequently, in order to estimate the degradation of the physical properties of the bedrock, several cases of analysis were carried out in which the physical properties of the bedrock $c, \phi$ were reduced by trial and error so as to adjust to the future uplift volume, and obtained $c = 0.09$ MPa from 230 mm which was close to the total uplift volume (shown figure 6). Finally, after the analysis step Step6 in figure 7, this ground physical characteristic value was input, the invert shapes of the two cases (shown figure 8) were set, and each analysis was performed.

### 3.3 Analysis result

Figure 9 (a) illustrates the axial force and bending moment of the two types of inverts. Figure 9 (b) is the M-N diagram of the load bearing curves with the design strength 24 MPa (in blue) and 18 MPa (for reference, in red) for comparison. The deeper invert (2.0$R_1$) had smaller bending moment and slightly larger axial force than the shallow one (2.7$R_1$), resulting in more advantageous structure. However, in the M-N diagram, the two cases almost overlapped, with sufficient allowance for the load bearing curves. The sectional forces of the lining showed a similar result. Also, the deformation amount...
distribution around the tunnel roadbed is slightly advantageous because the deeper invert slightly suppresses the spread of displacement downward on the roadbed.

But it can be evaluated that shallow inverts for speedy construction are almost as effective as deep inverts, considering the fact that their sectional forces have enough capacity for the bearing force, they both have long-term durability, their deformation volumes are at about the same level, and there is time constraint of traffic regulation during construction. Therefore, the standard cross section with the invert of about 2.7 radius ratio was deduced as the shallow structure, considering the cross-sectional shape with the existing sound sections.

4. Conclusion
The new findings of this study are presented below.

・It was clarified that the ground showed elastic behavior in the case where the invert was not provided at the time of construction, and the ground showed the behavior in the completely plastic region in the case where the invert was damaged. But in all cases, the ground around the roadbeds deteriorated to the level equivalent to lowest class of ground due to the long-term decrease in the strength.
・The phenomenon of all seven cases can be explained as swelling mechanism in which the inundation collapse degrees and the results of material tests exceeded the threshold of expansibility due to the ground that is easy to deteriorate by water and the action of spring water.
・The existing inverts are deep-shaped with radius curvature of about 2.0R1 which is smaller than the standard, and the support structure of the inverts is strengthened. The two cases of tunnels with different initial ground conditions and deformation causes were dealt with by inverts with almost the same shape, but the time course and distribution of stresses generated in the inverts were different.
・In the cases with damaged inverts, the axial force was about 3,700 kN and the working earth pressure about 0.37 MPa, and they have yet to be converged. It is necessary to have the support performance such as primary invert on early ring closure which suppresses the squeezing-dominant phenomenon.
・In the cases where inverts were not initially installed and swelling was main phenomenon, the axial force of the inverts was about 1.0 MN and the working earth pressure about 0.1 MPa. The support performance to deal with this level of phenomenon is needed.
・Then, from the numerical analysis considering the deterioration of the ground, in the case without invert, almost the same effect was shown in the conventional deep shape and shallow shape, and it was verified that the invert shape can be made shallow.

References
[1] East Nippon Expressway Co., Ltd 2015 J. Design criteria III tunnel, 72-87.