Evaluation of Deformation and Failure Behavior of Mountain Tunnels on Lining Material

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This paper presents conclusions drawn from model tests of mountain tunnel linings consisting of various materials. Plain concrete linings did not show any decrease in load, indicating good deformability, whereas compressive cracks and spalling occurred. Brick linings demonstrated lower structural stiffness and bearing capacity than plain concrete linings, and interlayer cracks occurred. Short-fiber-reinforced concrete linings showed good anti-spalling performance, whereas structural stiffness and bearing capacity were almost the same as in plain concrete linings. Reinforced concrete linings had the highest structural stiffness and bearing capacity, but displayed frequent shear failures.

Keywords: mountain tunnel, lining, model test, deformation, failure, lining material

1. Introduction

Mountain tunnels are subject to ground pressure and seismic motion depending on the geological and other conditions of where they are built. Studies (e.g. [1]) have shown that mountain tunnel linings can withstand significant deformation, however, these studies do not fully take into account the characteristics of arch-structured mountain tunnels. In practice, alternative parameters are used to evaluate safety in the design process, such as tensile cracking, yielding of reinforcing bars, etc., and crack width, convergence rate, etc. for maintenance management [2]. The use of these parameters can be explained by the lack of a clear definition of what the critical state is for mountain tunnel linings. There are a number of possible reasons why defining a critical state for tunnel linings is difficult. Firstly, a mountain tunnel is an arched structure encased in the ground and the axial forces and bending moments intensify as deformation increases, leading to complex deformation and failure behaviors. Secondly, the quality of the tunnel lining itself which depends on the quality of construction: poor work can lead to deficiencies such as insufficient tunnel lining thickness and cavities behind the tunnel lining, which are quite common. Thirdly, the wide range of materials used for linings. Figure 1 shows the materials used for mountain tunnel lining at different times in history. Figure 1 shows that a variety of materials have been used, including bricks, concrete blocks and cast-in-place concrete. In the Meiji Era, linings were made with bricks and stones. In the Taisho Era, cast-in-place concrete was gradually introduced. At that time, concrete mixing and pouring was done manually. For a certain time, concrete blocks were used to achieve the design thickness of linings. In the Showa Era, cast-in-place concrete became the norm.

Depending on geological and topographical conditions tunnel linings may need to be reinforced. Figure 2 shows the different places where plain concrete and reinforced concrete are used for reinforcement in mountain tunnels. Plain concrete is normally used on stable ground, whereas reinforced concrete is used for tunnel entrances, shallow-overburdened sections and for fractured ground sections in anticipation of loads which will be exerted on the tunnel. More recently, fiber-reinforced concrete has in some cases been used.

Given this background, experiments were conducted using 1/5 scale models of tunnel linings made with different materials including plain concrete, brick, short-fiber-reinforced concrete and reinforced concrete in order to observe their deformation and failure behavior and thus clarify what the critical state is for mountain tunnel linings thereby facilitating related safety evaluation. This paper discusses the results of these experiments.

Fig. 1 Materials used through history up to the present day for mountain tunnel linings

Fig. 2 Typical distribution of plain concrete and reinforced concrete use in a mountain tunnel
2. Experiments

2.1 Experimental device

The experiments were conducted using an experimental device that included scale models of tunnel linings [3] (Fig. 3). The experimental device was a 1/5 scale model (with dimensions corresponding to a standard section of Shinkansen double-track tunnel). The device was composed of a hydraulic load application jack, hydraulic reaction cylinders, reaction frames, etc. Disc springs were arranged across the outer circumference of the model lining to simulate interaction between tunnel linings and the ground. Table 1 shows the specifications of the experimental device. Using a reduced 1/5 scale is less onerous than conducting full scale experiments and at the same time enables use of materials that are actually employed in tunnels, making it possible observe failure conditions similar to those found in real tunnels.

2.2 Experimental cases

Five experimental cases, shown in Table 2, were used to identify any difference between the model lining materials. Figure 4 shows the dimensions of the model linings used. The model linings had a standard thickness of 150 mm, based on the thickness of 70 cm for Shinkansen tunnels used in the conventional method of the 1960s, and a longitudinal length of 300 mm.

In Case 2, a brick lining was used. The bricks used were of the kind that are freely available on the market and had similar properties to those employed for tunnel linings in the Meiji Era. As shown in Fig. 5, the bricks were arranged in a staggered pattern lengthways and stacked two layers high in the radial direction using mortar joints.

Cases 3 and 4 focused on short-fiber-reinforced concrete. In Case 3, short polypropylene fibers (PPF) were mixed with concrete according to the specifications for reinforced concrete structures designed to prevent the cover concrete from spalling to verify the spalling prevention performance when used in mountain tunnel linings. As the PPF mixture ratio of Case 3 was considered to be rather low and therefore have little effect on the load bearing performance of the model, a second ratio was also used to detect any difference between them, so the model was made with two different PPF mixture ratios, one for the right half and the other for the left half. In Case 4, steel fibers (SF) were mixed with concrete at a mixture ratio of 0.5 vol%, a ratio expected to improve flexural ductility, for comparison with Case 3.

Case 5 was designed to simulate reinforced concrete linings. Mortar was used instead. There were no design standards for reinforced concrete linings of tunnels constructed according to the conventional method. Consequently, reinforcing bars were arranged in a double reinforced arrangement as shown in Fig. 6 by referring to examples (D22, bars approximately 150 mm apart) from actual railway tunnels. Shear reinforcement bars are round, therefore a reinforcement ratio of 0.22% was chosen based on the standards for reinforced concrete structures. Table 3 shows specifications for the materials used.

Experiments were conducted by applying a load perpendicularly onto the crown using a hydraulic load application jack until a controlled displacement of 50 mm (40 mm in Case 2) was reached.

Figure 7 shows the measurements that were considered. While loads were being applied, the following measurements were taken: the load being applied, the displacement of the crown where the load was being applied and normal displacements (at nine angles) on the inner surface of the lining. In Case 5 (reinforced concrete), the strain on the reinforcing bars was measured at the angles of the lining where normal displacement, mentioned above, was measured.

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Table 1 Specifications of the experimental device

<table>
<thead>
<tr>
<th>Element</th>
<th>Item</th>
<th>Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydraulic load application jack</td>
<td>Maximum load</td>
<td>500 kN</td>
</tr>
<tr>
<td></td>
<td>Maximum pressure</td>
<td>5.6 MPa</td>
</tr>
<tr>
<td></td>
<td>Stroke</td>
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</tr>
<tr>
<td>Hydraulic reaction cylinders</td>
<td>Stroke</td>
<td>200 mm</td>
</tr>
<tr>
<td>Disc spring</td>
<td>Spring coefficient</td>
<td>3,000 kN/m</td>
</tr>
<tr>
<td></td>
<td>Modulus of subgrade reaction</td>
<td>16 MN/m²</td>
</tr>
</tbody>
</table>

Table 2 Experimental cases

<table>
<thead>
<tr>
<th>No.</th>
<th>Designation</th>
<th>Material</th>
<th>Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Plain concrete</td>
<td>Concrete</td>
<td>Plain concrete</td>
</tr>
<tr>
<td>2</td>
<td>Brick</td>
<td>Brick</td>
<td>Brick 2 layers</td>
</tr>
<tr>
<td>3</td>
<td>PPF</td>
<td>Concrete</td>
<td>Polypropylene fiber, 0.05vol% (Left half)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Polypropylene fiber, 0.1vol% (Right half)</td>
</tr>
<tr>
<td>4</td>
<td>SF</td>
<td>Concrete</td>
<td>Steel fiber, 0.5vol%</td>
</tr>
<tr>
<td>5</td>
<td>RC</td>
<td>Mortar</td>
<td>Reinforced concrete</td>
</tr>
</tbody>
</table>

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Fig. 3 The experimental device [3]

Fig. 4 Dimensions of the model linings

Longitudinal length: 300 mm.
Standard size

Arrangement of bricks (Case 2)

**Table 3 Specifications for the materials**

<table>
<thead>
<tr>
<th>Item</th>
<th>Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>Maximum size of coarse aggregate: 20 mm</td>
</tr>
<tr>
<td></td>
<td>Strength: 24 N/mm² (28 day)</td>
</tr>
<tr>
<td>Mortar</td>
<td>1:5Mortar, Strength: 22 N/mm² (28 day)</td>
</tr>
<tr>
<td>Brick</td>
<td>Size: 195 - 70 mm, Joint: 1:3 mortar</td>
</tr>
<tr>
<td>Steel fiber</td>
<td>Size: φ0.7 mm - 43 mm, Mixture ratio: 0.5 vol%</td>
</tr>
<tr>
<td>Polypropylene fiber</td>
<td>Size: φ64.8 μm - 12 mm, Mixture ratio: 0.05 vol% and 0.1 vol%</td>
</tr>
<tr>
<td>Reinforcing bar</td>
<td>Double reinforcing, SD345</td>
</tr>
<tr>
<td></td>
<td>Longitudinal bar: D6@27.2 mm (Reinforcement ratio 1.55 %)</td>
</tr>
<tr>
<td></td>
<td>Stirrup: φ3 - 5 @60 mm (Reinforcement ratio 0.22 %)</td>
</tr>
</tbody>
</table>

![Fig. 5](image1.png)

**Fig. 5** Arrangement of bricks (Case 2)

**Fig. 6** Arrangement of reinforcing bars (Case 5)

**Fig. 7** Measurement items

3. Results of experiments

3.1 Failure modes

There were three main failure modes. After the appearance of some tensile cracks caused by bending, the following was observed: bending compression failure and spalling in Case 1 (plain concrete), Case 3 (PPF) and Case 4 (SF); interlayer cracking in Case 2 (brick); and inclined cracking in Case 5 (reinforced concrete).

With Case 1 (plain concrete), Case 3 (PPF) and Case 4 (SF), all of which used concrete, when \( \delta \) was about 2 mm, tensile cracks appeared on the inner side of the crown and on the outer side of the lining at the left and right shoulders, loading the lining into a three-hinged arch shape, and when \( \delta \) was about 20 mm compressive failure occurred on the inner side of the lining at the left and right shoulders. Cracking progressed in a similar manner with all of the materials, with and without fibers. However, with the compressive failure observed at \( \delta = 50 \) mm (Fig. 8), Case 1 (plain concrete) saw a lump as large as a hand coming off and falling, while Case 3 (PPF) and Case 4 (SF) saw pieces no larger than those of coarse aggregate (about 20 mm) coming off and falling. There were no major differences between the PPF mixture ratios or the materials. The lining spalling preventive effect was observed even with the lower PPF mixture ratio of 0.05 vol%. This is considered to be because mortar-jointed fibers limited the spalling of small pieces of concrete even with closed cracks.

Similarly, in Case 2 (brick), when \( \delta \) was about 2 mm, tensile cracks appeared on the inner side of the crown and on the outer side of the lining at the left and right shoulders, pushing the lining into a three-hinged arch shape. When \( \delta \) was about 20 mm, cracks appeared at the crown, in the joint between the upper and lower brick layers. As loading continued, compressive failure did not appear on the inner side of the shoulders. When \( \delta \) was about 32 mm and onward, however, cracks at the crown between the brick layers spread rapidly to the shoulders on both sides (Fig. 8). In both Case 1 (plain concrete) and Case 2 (brick), the lining took on a three-hinged arch shape, despite different materials being used. However, in Case 2 the brick joints made the lining structurally uneven, and the difference in curvature between the upper and lower layers caused by deformation is thought to have led to shear force being applied to the interlayer joint as the lining deformed and consequently led to the joint splitting.

In Case 5 (reinforced concrete), when \( \delta \) was about 3 mm, tensile cracking as well as inclined cracking appeared at a number of locations on the inner side of the crown. Then, tensile cracking appeared in a number of locations on the outer side between the arch shoulders and spring lines, and the inclined cracking gradually grew in width. When \( \delta \) was about 25 mm and onward, the inclined cracking widened rapidly. On the other hand, the tensile cracking grew little on the inner side of the crown and in the areas between the arch shoulders and spring lines on the outer side, and shearing failure occurred as a tunnel structure (Fig. 8). Compared with Case 1 (plain concrete), tensile cracks at the crown and shoulders were relatively narrow and shallow, and no compressive failure was observed on the inner side of the shoulders. This is considered to be because longitudinal reinforcement bars were provided, making the lining more resistant to bending.
3.2 Load-displacement curve

Figure 9 shows the relationship between load $P$ and displacement $\delta$ (hereafter “the load-displacement curve”) for each of Cases.

In Case 1 (plain concrete), Case 3 (PPF) and Case 4 (SF) in which the lining was made of concrete, when $\delta$ was about 2 mm tensile cracking appeared and lining became a three-hinged arch shaped, and the load temporarily weakened. There were no significant differences in the curve between the different types of fiber and PPF mixture ratios. With Case 4 (SF), however, the decline in load after the lining became shaped like a three-hinged arch was less than Case 1 and Case 3 and the load was larger than Case 1 and Case 3 when $\delta$ was less than 20 mm. This was probably because the decline in bending rigidity after the appearance of the tensile cracks was limited as a result of the comprehensive impact of the difference in fiber material, fiber mixture ratio, fiber geometry and fiber length. When deformation progressed up to a point and $\delta$ was about 20 mm and above, the load became almost the same for all Cases. After the appearance of compressive failure at the crown, rigidity declined slightly.

With Case 2 (brick), the load was lighter than the other Cases. This was considered to be due to discontinuity in the lining because of the numerous joints and that the modulus of elasticity of each single brick was smaller than the concrete, making the brick lining less rigid than the other materials.

Case 5 (reinforced concrete) displayed the highest rigidity and the largest maximum load among the materials. No significant decline in rigidity caused by tensile cracking was observed after $\delta$ reached about 2 mm, as was seen in Cases 1, 3 and 4. The rigidity declined when $\delta$ was about 3 mm and again about 14 mm. The load peaked when $\delta$ was about 22 mm and started declining rapidly when $\delta$ was about 27 mm. With Case 5, the load on the legs of the lining increased to such an extent that the steel stopper for the legs broke, resulting in both legs moving outward by about 10 mm by the time $\delta$ reached about 20 mm. The legs did not move after $\delta$ reached about 20 mm.

Figure 10 shows the deformation profiles when $\delta$ was 10 mm for Cases 1, 2 and 5. Cases 3 and 4 showed similar deformation profiles to Case 1 and therefore were omitted. Generally, the lining was displaced toward the inner space in the crown area directly below the load application point and toward the disc springs on the left and right shoulders. The outward displacement of the shoulders from highest to lowest was: Case 1 (plain concrete), Case 2 (brick) and Case 5 (reinforced concrete). Case 1 (plain concrete) saw the largest outward displacement of the shoulders probably because the location of the shoulders roughly corresponded to two hinges in the three-hinged arch where bending rigidity declined significantly. In Case 2 (brick), the lining had joints, which were absent in the plain concrete of Case 1, making the structure less rigid. It is therefore considered that the displacement caused by load application was absorbed in the area just at and around the load application point, resulting in the outward displacement of the shoulders being less than that for plain concrete. Case 5 (reinforced concrete) saw the smallest displacement of all the materials. This is probably because the reinforcement bars enhanced the bending rigidity of the structure.
3.3 Strain of the reinforcing bars

In Case 5 (reinforced concrete), Fig. 11 shows distribution of the strain on the longitudinal reinforcement bars when \( \delta \) was 10 mm and Fig. 12 shows the relationship between displacement and strain on the reinforcing bars. Figure 11 shows the reinforcing bars having a tendency to deform more at the crown and both shoulders. Figure 12 shows tensile yield at A (inner reinforcement bars at the crown) when \( \delta \) was 3 mm, compressive yield at B (outer reinforcement bars at the crown) when \( \delta \) was 8 mm, tensile yield at D (outer reinforcement bars at the shoulders) when \( \delta \) was 12 mm and compressive yield at C (inner reinforcement bars at the shoulders) when \( \delta \) was 22 mm. Figure 9 shows decline in rigidity when \( \delta \) was about 3 mm and again about 14 mm. The former decline appears to have been caused by yielding at A (inner reinforcement bars at the crown) while the latter likely resulted from yielding at D (outer reinforcement bars at the shoulders). In addition, the load stopped increasing when \( \delta \) was 22 mm, which is probably because the lining was pushed into a three-hinged arch shape after yielding at C (inner reinforcement bars at the shoulders).
formance between the materials. Mountain tunnels are constructed with a variety of section profiles and structures to meet various ground conditions. In addition, tunnel linings are often found to have cavities, or insufficient thickness. The subsequent section will discuss tendencies of tunnels satisfying the following assumptions:
- The ground is either sediment or low-strength soft rock.
- The lining has a sectional force that is generated due to moderate bending moments and axial forces.
- The tunnel lining does not have cavities and is sufficiently thick.

1. Plain concrete lining

As deformation progresses, cracking occurs, reducing rigidity, while the load being applied holds until the deformation becomes relatively large. These are some of the benefits of the arch structure. That said, compressive failure occurs on the inner side of the lining before the load peaks.

Plain concrete with reasonable levels of load bearing capacity and deformation performance is considered to be appropriate as the material for the lining of mountain tunnels being constructed in ordinary ground, which requires some margin and does not require application of specific loads. Nevertheless, as deformation progresses, compressive failure and spalling can occur. Therefore, a tunnel with cracks and deformation needs to be monitored regularly and retrofitted with anti-spalling and other measures suited to the amount of deformation to prevent compressive failure and spalling.

2. Brick lining

Brick linings have lower rigidity and load bearing capacity than concrete, and develop interlayer cracks as deformation progresses. Having low bending rigidity, brick linings contain deformation within a small area around the load application point.

Given the many hinges in the brick lining structure of a mountain tunnel, brick linings have lower bending rigidity and load bearing capacity and are more vulnerable to deformation than plain concrete linings. Being a layered structure, a brick lining needs inspection to detect interlayer separation if the tunnel is deformed.

3. Short-fiber-reinforced concrete lining

Ordinary short-fiber-reinforced concrete linings can have different mixture ratios and has roughly the same levels of rigidity and load bearing capacity as plain concrete linings. The decline in load with steel-fiber-reinforced concrete linings is less than for plain concrete linings when the lining becomes shaped like a three-hinged arch as long as deformation is moderate. Spalling occurs less often, and this still holds with lower mixture ratios of fiber.

For large deformation, fiber-reinforced concrete linings have the same level of load bearing capacity as plain concrete linings but is effective in spalling prevention and has higher deformation performance. Therefore, it can be used as a measure to prevent spalling for a tunnel constructed in fractured ground sections and other adverse conditions where deformation caused by earthquakes etc. is expected.

For small deformation, fiber-reinforced concrete linings with appropriate types of fiber and mixture ratios can help limit the opening of tensile cracks and deformation.

4. Reinforced concrete lining

Reinforcing bars greatly improve rigidity and load bearing capacity over plain concrete linings. Load increases even after reinforcing bars yield, which is the arch effect. On the other hand, bending reinforcement and resultant increase in bending strength can induce a shear failure mode.

Reinforced concrete linings has highest rigidity and load bearing capacity and is most appropriate where high load bearing performance is needed such as at tunnel entrances and in shallow-overburdened sections. In that context, however, shear reinforcement also needs to be considered.

5. Conclusion

Experiments were conducted using 1/5 scale models of tunnel linings made of various materials, to observe their deformation and failure, with a view to helping clarify critical states for mountain tunnel linings and facilitate related safety evaluations. Assuming that the materials experimented above were adoptable as mountain tunnel linings, differences in their load bearing capacity and deformation performance were examined. These exercises clarified, applicable to limited conditions though, critical states, load bearing capacity and deformation performance of mountain tunnels. Mountain tunnels are constructed with a variety of section profiles and structures to meet various local geological conditions. In addition, deficiencies such as insufficient tunnel lining thickness and cavities behind the lining are seen in many tunnels. It must be noted that what is presented in this paper may not be applicable to certain conditions. Further study is needed to be undertaken to clarify the deformation and failure behaviors of tunnel linings in an even wider range of conditions.

References


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