

1 Evaluation of spalling of concrete pieces from tunnel lining employing joint shear
2 model

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15 **ABSTRACT**

16 This paper was conducted to quantify the spalling of tunnel lining concrete. This paper firstly
17 modeled the spalling of tunnel lining concrete and examined ways to quantitatively evaluate the safety
18 of the tunnel lining concrete against spalling by comparing the shear stress acting on the joint surfaces
19 of lining concrete cracks with the shear capacity of the joint surfaces. Secondly, based on double shear
20 tests simulating the spalling phenomenon, it was found that wider crack widths as well as greater
21 tapered angles of the joint surfaces resulted in lower shear capacities. Thirdly, a method for simulating
22 the shearing process of joint surfaces was proposed. The method was shown to be able to roughly
23 estimate the relationship between the joint aperture and the shear capacity. Using the improved
24 method, the shear capacity was considered using various widths and tapered angles of the cracks.
25 Furthermore, it was found that the *JRC* value of the joint surface samples collected from actual
26 tunnels was between the *JRC* value of the mortar beam joint surfaces that were made in the shear test
27 and that of the joint surfaces of the concrete beam, made in the shear tests with the maximum
28 aggregate size measuring 40 mm. A parameter study was also conducted under hypothetical
29 conditions for the relationship between the width of the cracks and the safety of the tunnel lining
30 against spalling.

31

32 **Keywords:**

33 Tunnel lining

34 Falling of concrete fragments

35 Shear strength

36 Crack surface roughness

37

38 1. INTRODUCTION

39 The problem of the spalling of tunnel lining concrete started drawing attention after the spalling
40 accident involving a Shinkansen (bullet train) railway tunnel in 1999 in Japan, as shown in Figures 1
41 and 2 (Ministry of Transport, 2000; Asakura et al., 2001). Railway and road tunnels are periodically
42 examined through visual inspections and hammering tests to evaluate their structural soundness.
43 When areas at risk of spalling are found, appropriate actions are taken to ensure the safety of the
44 tunnel. In the maintenance of railway tunnels, visual inspections are firstly conducted which may
45 reveal multiple connected cracks in closed, crossed, parallel or other forms. These are followed by
46 hammering tests, in which the tunnel linings are hit with a hammer or similar object. Based on the
47 results, the tunnel's structural soundness against spalling is rated as α , β or γ . For example, if an area
48 with closed cracks emits a dull sound when hit, the area's structural soundness is rated as α , in which
49 case measures to prevent spalling must be taken.

50 In an ordinary inspection, the tunnel lining surfaces are visually inspected for cracks, including their
51 shapes and widths. Cracks seen on tunnel lining surfaces have a range in widths, from small to large,
52 and it is assumed that closed cracks with greater widths have a greater risk of spalling. The current
53 method used for evaluating structural soundness is based on a qualitative approach in which the risk
54 of spalling is considered to be greater with multiple closed cracks. No methods for quantitatively
55 evaluating the safety against spalling have been proposed; and thus, there are no criteria available
56 today on the specific widths of cracks for a quantitative evaluation of the structural soundness. If there
57 were a method of quantitatively evaluating the relationship between the widths of lining surface
58 cracks and the safety against spalling, the structural soundness against spalling could be determined
59 more accurately, which would contribute to the rational maintenance of tunnels.

60 In mountain tunnels with plain concrete linings, for which there is no adhesion of the concrete to
61 reinforcing bars, any closed cracks that run across the width of the lining can cause spalling. In
62 practice, the risk of closed cracks causing concrete to spall off is reduced by the roughness of the

63 mating faces of the cracks causing frictional resistance. Therefore, the safety of the tunnel lining
64 against spalling is thought to vary widely depending on the roughness and the width of the cracks.

65 Some studies have investigated the anomalies of tunnel linings caused by outer force, such as
66 squeezing earth pressure, uneven earth pressure, loosening earth pressure, etc. For example, Asakura
67 et al. (1994) carried out loading tests with 1/30 scaled lining models and arranged the relationship
68 between the loading patterns and the configurations of the cracks observed in the tunnel lining. He et
69 al. (2009) investigated the failure mechanism of deformed tunnels. Wang (2010) and Chiu et al.
70 (2017) investigated the anomalies of tunnel linings caused by the instability of neighboring slopes.
71 However, these studies did not focus on the spalling of the tunnel lining, which is a local
72 phenomenon.

73 With reinforced concrete, a number of studies have examined the shear capacity and the stress
74 transfer that factor into the roughness of concrete cracks and joints (Yoshikawa and Tanabe, 1986;
75 Yoshikawa et al., 1989; Li and Maekawa, 1988; Maekawa and Qureshi, 1997). Chiaia et al. (2009)
76 also investigated the effect of fibers in the tunnel lining concrete by means of a block model. However,
77 there have been few studies on the spalling of pieces of plain concrete attributable to the concrete's
78 own weight, wind pressure, and other factors. The following is an example of studies conducted on
79 the plain concrete linings of road tunnels: a study in which cracking modes were examined for factors
80 involved in spalling and a study in which core samples of cracked linings were collected and their
81 shear strength was measured by direct shear tests (Ito et al., 2004). Those studies provided valuable
82 data for investigating the spalling of tunnel lining concrete, but did not propose quantitative methods
83 for evaluating spalling, including the relationship between the width of a crack, as well as the degree
84 of roughness of the mating faces of the crack (hereinafter defined as joint surface roughness), and the
85 safety against spalling.

86 With this background, the authors firstly modeled the spalling of tunnel lining concrete and
87 examined ways to quantitatively evaluate the safety against the spalling of the tunnel lining concrete

88 by comparing the shear stress acting on the joint surfaces of lining concrete cracks with the shear
89 capacity of the joint surfaces. Secondly, double shear tests simulating the spalling phenomenon were
90 conducted to clarify the relationship between the width and tapered angle of the cracks and the shear
91 capacity. Thirdly, double shear tests were simulated using an improved method to replicate direct
92 shear tests on rock joints (Kishida and Tsuno, 2001), and its applicability was examined. Employing
93 the method, the shear capacity was considered using various widths and tapered angles of the cracks
94 and various material strengths. Furthermore, the joint surface roughness in an actual tunnel was
95 measured, and calculations were performed under hypothetical conditions for the relationship between
96 the width of the cracks and the safety of the tunnel lining against spalling.

97

98 **2. MODELING THE SPALLING OF TUNNEL LINING CONCRETE**

99 **2.1 Modeling of the spalling**

100 In the past, pieces of concrete that fell off the lining of mountain tunnels were typically several
101 centimeters thick; and thus, the events were categorized as exfoliation. During one such accident,
102 however, a chunk of concrete (250 cm wide, 300 cm deep, and 45 cm thick) fell off the crown of a
103 railway tunnel, as shown in Figure 2. Such huge chunks of concrete could cause serious accidents.
104 Therefore, the present study looked at the spalling of huge chunks of lining concrete. While spalling
105 can also be caused by other factors, such as a void behind the lining, the aim of this study is to model
106 the spalling phenomenon that is caused by a reduced shear capacity of the joint surfaces of cracks that
107 have become wider apart or are due to other characteristics.

108 The falling of a chunk of plain lining concrete from the crown of a mountain tunnel was modeled as
109 follows: It was assumed that the concrete piece was formed by the closure of cracks and had the
110 dimensions of B by D by H shown in Figure 3.

111 1) The weight of the chunk and the external forces generate shear stress τ_a that acts on the joint
112 surfaces (the four sides of the chunk) of the cracks.

113 2) Shear resistance is generated as a result of the roughness of the joint surfaces of the cracks. The
114 maximum shear resistance, which depends on the joint surface roughness, the strength of the chunk
115 material, and other factors, is defined as the shear capacity τ_b of the joint surfaces.

116 3) The chunk falls off when shear stress τ_a , acting on the joint surfaces, exceeds the shear capacity τ_b
117 of the joint surfaces.

118 While possible adhesion to the ground and to waterproof sheets behind the lining can have an
119 influence on the shear capacity, those factors were not considered in the model. This is because it is
120 difficult to assess such adhesion appropriately and because the model used in the evaluation was on
121 the safe side.

122

123 2.2 Shear stress generated on the joint surfaces of cracks

124 In the case of Figure 3, shear stress τ_{a1} generated by the weight of the chunk can be calculated as
125 follows:

126

$$127 \quad W = \gamma BDH \quad (1)$$

$$128 \quad A = 2H(B + D) \quad (2)$$

$$129 \quad \tau_{a1} = \frac{W}{A} = \frac{\gamma BD}{2(B + D)} \quad (3)$$

130 where:

131 W : weight of the chunk

132 γ : weight per unit volume of the chunk

133 B : width of the chunk

134 D : depth of the chunk

135 H : thickness of the chunk

136 A : area of the joint surfaces of the cracks

137 Shear stress τ_{a2} , generated by external forces, can be calculated as follows:

$$\tau_{a2} = \frac{fBD}{2H(B+D)} \quad (4)$$

where:

f : stress caused by external forces (i.e., stress acting on the area measuring B by D)

Based on the above, shear stress τ_a , generated on the joint surfaces of the cracks, can be calculated by

$$\tau_a = \frac{\gamma BD}{2(B+D)} + \frac{fBD}{2H(B+D)} \quad (5)$$

2.3 Trial calculation of the shear stress generated on the joint surfaces of cracks

The shear stress acting on the joint surfaces of the chunk was calculated under various areas of closed cracks and thicknesses of the chunk using Equation (5). The trial calculation was performed under the same width B and depth D . Only the specific wind pressure of 5 kN/m², generated by the passage of trains, was considered as the stress caused by external forces f . For this wind pressure, the variation in air pressure (5 kN/m²), measured on the Sanyo Shinkansen line during a train passage, was used (Ministry of Transport, 2000).

Figure 4 shows the relationship between the area of closed cracks and the shear stress generated on the joint surfaces of the cracks. The area of closed cracks was calculated by multiplying the width of chunk B by depth D . As the figure shows, for the same thickness of the chunk, larger shear stress is generated on the joint surfaces when the area of the closed cracks is larger. Moreover, for the same area of closed cracks, the shear stress becomes smaller when the chunk is thicker.

3. DOUBLE SHEAR TESTS

3.1 Outline of the tests

To quantitatively evaluate the safety of the tunnel lining concrete against spalling, it is necessary to clarify the shear capacity of the joint surfaces of the cracks relative to the behavior of the area within the closed cracks. Accordingly, **mortar test pieces** were made, each with two cracks, as shown in

163 Figure 5, which were then subjected to the double shear tests using the loading device shown in
164 Figure 6 to simulate the falling of a piece of tunnel lining concrete.

165

166 3.1.1 Test pieces

167 The joint surfaces of each crack in the test pieces were made by making a plain concrete beam
168 (2000 mm long, 250 mm wide, and 500 mm high) with the maximum aggregate size measuring 20
169 mm, and then subjecting the beam to a shear test to generate the crack. In the next step, to ensure
170 uniform roughness on all test pieces, plaster casts of the crack were made and then mortar faces of the
171 crack were created from the plaster casts. Then, the mortar casts were placed in a mold in such a way
172 that, for both cracks of each test piece, the protruding mortar cast with a cracked face (250×250 mm)
173 was on the underload section and the receding mortar cast with an identical cracked face was on the
174 stub side. Finally, mortar was poured into the mold and left to cure at air temperature for 28 days. The
175 result was a test piece consisting of left and right stubs with an underload section between them, with
176 the three sections split by two cracks. Using the same procedure, plaster and mortar joints were made
177 to compare their roughness. Only a small difference was found between them that corresponded to a
178 *JRC* value, a measure of the joint surface roughness, of around 0.3 at most.

179 The test pieces were produced, including the curing process, at a constant room temperature of
180 20°C. The mortar was made from cement, sand, and water in a weight ratio of 1:2.8:0.6. The tests
181 showed that the mortar had a uniaxial compressive strength of 36 N/mm² and a modulus of static
182 elasticity of 2.50×10^4 N/mm².

183

184 3.1.2 Measurement of the roughness of the joint surfaces

185 Prior to testing, the roughness of the joint surfaces was measured at intervals of 1 mm using a
186 surface roughness measurement system consisting of a CCD laser displacement meter (spot diameter:
187 70 μm and resolution: 3 μm) and a sliding table. The measured joint roughness is shown in Figure 7.

188 The measurements have a *JRC* of 29.0, which roughly agrees with the average of 28.1 for actual
189 tunnels described in 5.1. One specific joint roughness was used, focusing mainly on the effect of the
190 crack width in the tests, although the joint surface roughness is different in actual situations. The
191 effects of the joint surface are discussed in Chapters 5 and 6.

192

193 3.1.3 Test cases

194 The test cases are shown in Table 1. In Cases 1 to 5, the tests were conducted using various crack
195 widths to clarify the relationship between the joint aperture and the shear capacity. In Cases 6 and 7,
196 the joints were tapered to facilitate the falling of the underload section. As described earlier, the joint
197 surfaces in those cases were a copy of the joint surface made in the shear test on a plain concrete beam
198 with the maximum aggregate size measuring 20 mm. Therefore, the joint surfaces all had the same
199 roughness profile.

200

201 3.1.4 Test device and method

202 As shown in Figure 6, the test device consisted primarily of reaction frames, a hydraulic jack to
203 apply vertical load (3,000 kN), and two screw jacks to hold the stub sections still. After a test piece
204 was placed on the test device, the stub sections were held in place with screw jacks to prevent their
205 vertical movement, thereby preventing them from rotating during the test. To allow the stub sections
206 to move horizontally, Teflon plates were inserted between the stub sections and the screw jacks and
207 beneath the stub sections. Having made these arrangements, the underload section was then subjected
208 to a vertical load using a hydraulic jack in a controlled displacement mode (0.015 mm/s).

209 For the measurements, a load cell was placed between the hydraulic jack and the underload section
210 to measure the shear stress, while load cells were placed between the reaction frames and the test
211 pieces to measure the horizontal reaction stress on both sides. In addition, the vertical displacement of
212 the underload section was measured with four displacement sensors (Ch1 to Ch4 in Figure 6 (b)) to
213 obtain the shear displacement, while the horizontal displacements of the stub and the underload

214 sections were measured with 6 displacement sensors (Ch5 to Ch10 in Figure 6 (b)) to obtain the
215 aperture growth of the joint surfaces.

216

217 3.2 Test results

218 3.2.1 Test results of test pieces with various crack widths

219 Figure 8 (a) shows the relationship between the shear displacement and the shear stress for Cases 1
220 to 5 with various crack widths. In the tests, as the two joint surfaces contribute to shearing, the shear
221 stress is calculated by dividing the vertical load applied on the underload section by the hydraulic jack
222 by the combined area ($2 \times 250 \times 250$ mm) of the two joint surfaces. The figure shows that the shear
223 stress grew as the shear displacement increased until a certain point at which the test pieces yielded
224 and the maximum shear stress was reached. Beyond that point, the shear stress remained
225 approximately the same. It was also found that test pieces with wider crack widths had smaller values
226 of maximum shear stress.

227 Figure 8 (b) shows the relationship between the shear displacement and the horizontal reaction
228 stress. The horizontal reaction stress was obtained by dividing the average stress measured with the
229 load cells (500 kN) on both sides by the combined area of the joint surfaces. The relationship between
230 the shear displacement and the horizontal reaction stress shows similarities to that between the shear
231 displacement and the shear stress. Test pieces with wider crack widths had smaller values of
232 maximum horizontal reaction stress. Likewise, test pieces with wider crack widths had smaller values
233 of aperture growth (Figure 8 (c)). The joint aperture growth of a test piece here is the average value
234 based on the piece's two joint apertures, starting at 0 mm when the shear displacement is 0 mm.

235

236 3.2.2 Test results of test pieces with various tapered angles of joint surfaces

237 Figure 9 shows the relationship between the shear displacement and the shear stress for Cases 1, 6,
238 and 7 with different tapered angles of the joint surfaces. As shown in the figure, with the tapered

239 angles of 5° and 10°, the shear stress rose as the shear displacement increased. Then, after peaking, it
240 declined. It was also found that the test pieces with greater tapered angles of the joint surfaces had
241 smaller values of maximum shear stress.

242

243 3.2.3 Summary of the test results

244 The double shear tests, simulating the spalling phenomenon, yielded the following results:

- 245 1) The shear stress grew as the shear displacement increased until a certain point at which the
246 maximum shear stress was reached. Beyond that point, the load approximately remained stable or
247 declined.
- 248 2) Test pieces with wider crack widths had smaller values of maximum shear stress, possibly allowing
249 them to fall more easily with weaker external force.
- 250 3) Test pieces with greater tapered angles of the joint surfaces had smaller values of maximum shear
251 stress, possibly allowing them to fall more easily with weaker external force.

252

253 **4. SIMULATION OF THE SPALLING OF TUNNEL LINING CONCRETE**

254 4.1 Outline of the analysis

255 In the double shear tests described in the previous section, test pieces were examined that all had
256 the same roughness of joint surfaces and were made of the same material. However, the shear capacity
257 is thought to change with the roughness and the material properties. Accordingly, methods for
258 simulating the shearing process of joints were examined to clarify the shear capacity for varying
259 roughness and material properties of joints.

260 In rock engineering, equations and models for the shearing behavior of rock joints have been
261 developed (Patton, 1966; Barton, 1973; Barton and Choubey, 1977; Ohnishi et al., 2000; Kishida and
262 Tsuno, 2001). Among them, the examination in this study was based on an analytical model (Kishida
263 and Tsuno, 2001) that is designed to simulate the post-peak shearing behavior of rock joints, from

264 softening up to the residual state, based on the inputs of confining pressure, material strength, material
265 friction angle, and three-dimensional digitized joint surface roughness data. The analytical model used
266 in this study was based on the following:

267 1) In the shearing process, confining pressure acts on the asperities (surface unevenness between the
268 measuring points) in contact with each other where stress concentrates.

269 2) As the area of contact depends on the angle of dilation (between the direction in which the test
270 piece moves in shearing and dilation and the shearing direction) and the profile of the roughness,
271 the stress concentrated on the asperities in contact with each other can be determined based on an
272 assumed dilation angle.

273 3) The angle of dilation is determined in such a way that the concentrated stress acting perpendicularly
274 on the asperities in contact with each other at the joint surfaces equals the uniaxial compressive
275 strength.

276 Although the crack width might be gradually varied along the thickness direction, the proposed
277 method assumes that the crack width is uniform in the thickness direction. It is well known that the
278 joint surface roughness of a rock fracture changes under the shearing process. In addition, the
279 condition of the joint surface roughness, such as the asperities and the contact conditions, strongly
280 affects the mechanical and hydro-mechanical behavior of the rock fracture. Kishida, et al. (2009)
281 carried out direct shear and flow through experiments on single rock joints in addition to flow
282 simulations using joint surface roughness data from the shear analytical model (Kishida and Tsuno,
283 2001). Obtaining a good agreement between the results of the flow through experiments and the flow
284 simulation of the single rock fractures, the validity of the shear analytical model is confirmed. In this
285 study, the shear analytical model (Kishida and Tsuno, 2001), designed for direct shear tests on joints
286 that are initially engaged completely under constant confining pressure, was modified for the analysis
287 in this study so that it would be able to address the constant stiffness condition and the joint aperture
288 that were considered in the double shear tests.

289

290 4.1.1 Analysis steps

291 The step intervals of the analytical model need to be integer multiples of the measuring intervals of
292 roughness. As the joint surface roughness was measured at 1-mm intervals in the double shear tests,
293 the shear displacement was seen to increase by 1 mm at each step of the analysis. The surface
294 unevenness between two adjoining measuring points was defined as an asperity. In addition, the
295 analysis assumed that the underload section of a test piece would move downward under shearing
296 force. While the joint surfaces had an identical profile in the double shear tests, the simulation
297 considered horizontal reaction stress σ_n and shear stress τ acting on just one of the identical surfaces.

298

299 4.1.2 Calculations in Step i

300 With the selected asperities shown in Figure 10, that are in contact with each other and have a
301 dilation angle of σ_{ni}' , the horizontal reaction stress acting on the asperities can be calculated by

302

$$303 \quad \sigma_{ni}' = \sigma_{ni} \times T / A_i \quad (6)$$

304 where:

305 σ_{ni} : horizontal reaction stress in Step i

306 T : total number of surfaces between two adjoining measuring points (total number of asperities)

307 A_i : number of asperities in contact with each other in Step i

308 As constant stiffness is considered, horizontal reaction stress σ_{ni} was calculated by

309

$$310 \quad \sigma_{ni} = K \times \left(1.0 \times \sum_{k=1}^{i-1} \tan \theta_k + 1.0 \times \tan \theta_i \right) \quad (7)$$

311 where:

312 K : spring constant

313 θ_k : dilation angle determined in Step k

314 θ_i : assumed dilation angle

315

316 Next, horizontal reaction stress σ_{ni}' and shear stress τ_i' , acting on the asperities in contact with each
317 other, can be resolved into stress P perpendicular to the joint surface and stress Q parallel to the
318 surface, as shown in Figure 10. With the length of the joint surface given by $1/\cos \theta$, P and Q can be
319 calculated using the following equations:

320

321
$$P = (\tau_i' \sin \theta_i + \sigma_{ni}' \cos \theta_i) \times \cos \theta_i \quad (8)$$

322
$$Q = (\tau_i' \cos \theta_i - \sigma_{ni}' \sin \theta_i) \times \cos \theta_i \quad (9)$$

323

324 On the joint surface, the following equation of balance is assumed to be true:

325

326
$$Q - P \tan \phi_b = 0 \quad (10)$$

327 where:

328 ϕ_b : material friction angle

329 By substituting Equations (8) and (9) into Equation (10), the following equation is obtained:

330

331
$$\tau_i' = \frac{\sigma_{ni}' (\sin \theta_i + \cos \theta_i \tan \phi_b)}{\cos \theta_i - \sin \theta_i \tan \phi_b} \quad (11)$$

332

333 Rearranging the equation leads to the following equation, which shows the relationship between σ_{ni}'
334 and τ_i' :

335

336
$$\tau_i' = \sigma_{ni}' \tan(\phi_b + \theta_i) \quad (12)$$

337

338 Furthermore, considering that the ratio of σ_{ni} to σ_{ni}' is equal to the ratio of τ_i to τ_i' , the following

equation is true:

$$\tau_i = \sigma_{ni} \tan(\phi_b + \theta_i) \quad (13)$$

In Step i , the flow chart shown in Figure 11 is followed to determine shear stress τ_i using an assumed dilation angle. Firstly, a dilation angle, θ_i , is assumed and σ_{ni}' is calculated using Equation (6). Secondly, τ_i' is calculated using Equation (12), and then σ_{ni}' and τ_i' are substituted into Equation (8) to calculate stress P perpendicular to the joint surface. Then, P is compared with the uniaxial compressive strength. If P is greater than the uniaxial compressive strength, the dilation angle is reduced by 0.1° ($\theta_i = \theta_i - 0.1^\circ$). This is repeated, reducing θ_i by 0.1° each time until P is equal to or less than the uniaxial compressive strength. The dilation angle θ_i , reached at the end of this process, is then determined as the final dilation angle in this step.

It is assumed that when the joint is completely engaged (Case 1 with a joint aperture of 0 mm), only asperities with a slope angle equal to or greater than the dilation angle are in contact with each other and stress concentrates on those asperities. Accordingly, the slope angle is calculated for all asperities (between the measuring points), as shown in Figure 12, and those asperities whose slope angles are equal to or greater than the dilation angle are counted.

As for the number of asperities in contact with each other when the joint is not completely engaged, the number is obtained by calculating *Weight* W for all asperities (between the measuring points) and totaling the calculations. The W between measuring point $k-1, j$ and measuring point k, j is calculated as follows:

Under the condition whereby the stub and the underload sections are in contact with each other at measuring point $k-1, j$: when the slope angle is greater than dilation angle θ_i , the asperity is considered to be in contact, and therefore, $W = 1$; when the slope angle is less than dilation angle θ_i , the asperity is considered not to be in contact, and therefore, $W = 0$.

364 When the stub and the underload sections are not in contact with each other at measuring point $k-1$,
 365 j , a straight line is drawn at an angle of θ , from $Y_i(k-1, j)$ on the underload section to the stub section,
 366 as shown in Figure 13, and the intersection of the line and the stub section is defined as the Transit
 367 Contact Point (TCP). When TCP is located between $X_i(k-1, j)$ and $X_i(k, j)$, the surface between TCP
 368 and $X_i(k, j)$ is considered to be in contact and *Weight* W is calculated accordingly by

$$369 \quad W = L / \Delta x \quad (14)$$

371 where:

372 L : distance between the x -coordinate of TCP and $x = k$

373 Δx : measuring interval (1.0)

374 The x -coordinate of TCP can be calculated by

$$375 \quad x = \frac{X_i(k-1, j) - Y_i(k-1, j)}{\tan \theta - X_i(k, j) / \Delta x + X_i(k-1, j) / \Delta x} \quad (15)$$

377
 378 When TCP is not located between $X_i(k-1, j)$ and $X_i(k, j)$, $W = 0$.

379

380 4.1.3 Roughness at the end of Step i

381 At the end of each step, the underload section is moved 1.0 mm in the shearing direction at the
 382 dilation angle determined in Step i . When the stub and the underload sections overlap each other
 383 ($X_{i+1}(k, j) > Y_{i+1}(k, j)$), they are assumed to be in contact with each other at the middle point of the
 384 overlap (Figure 14).

385

386 4.2 Analysis conditions and cases

387 Using the same joint surface roughness that was used in the double shear tests, a simulation was
 388 conducted of shear stress τ and horizontal reaction stress σ_n acting on one of the two joint surfaces as

well as of the joint aperture growth. The joint aperture growth corresponds to dilation. In calculating the horizontal reaction stress, spring constant K in Equation (7) was set to 1.5 N/mm^3 based on the relationship obtained in the experiments between the joint aperture growth and the horizontal reaction stress as the spring constant K was affected by the stiffness of concrete, screw jack and other factors. The simulation was conducted up to a shear displacement of 16 mm.

The analysis cases used here are shown in Table 2. Cases 1 to 9 employed various crack widths, including those from 5 mm to 8 mm that were not used in the double shear tests. Cases 10 to 15 were conducted with various tapered angles of the joint surface including 15° , 20° , 25° , and 30° that were not used in the double shear tests. Cases 1 to 15 used a uniaxial compressive strength σ_c of 36 N/mm^2 .

4.3 Results of the analysis

4.3.1 Results of the analysis with various crack widths

The results of the analysis for Cases 1 to 5 (crack widths of 0 mm to 4 mm), relative to the results of the experiments, are shown in Figure 15. As the roughness of the two crack surfaces was the same as in the double shear tests, the calculation results under the condition of one crack surface are comparable with the experimental ones. The results of Step 1 (shear displacements of 0 mm to 1 mm) are shown at 0.5 mm on the shear displacement axis, while those of Step 2 (shear displacements of 1 mm to 2 mm) are shown at 1.5 mm on the axis. Similar to the results of the experiment, the shear stress grew as the shear displacement increased until the peak was reached, beyond which the shear stress remained approximately stable. It was also found that smaller values of shear stress resulted from wider crack widths. The calculated shear stress slightly differs from the test results in case of crack width if 0mm. It is considered that the difference is caused by the influence of shaved asperities in the shear procedure. As for the horizontal reaction stress and joint aperture growth, smaller values resulted from wider crack widths as in the case of the experiments.

Figure 16 shows the relationship obtained from the analysis of Cases 1 to 9 between the joint

414 aperture and the shear capacity relative to the results of the experiment. The shear capacity
415 corresponds to the maximum shear stress. The results of the analysis roughly agree with those of the
416 experiment, indicating that the simulation method is capable of roughly estimating the relationship
417 between the joint aperture and the shear capacity. It was also found that wider crack widths resulted in
418 lower shear capacities on the joint surfaces.

419

420 4.3.2 Results of analysis with various tapered angles of the joint surface

421 Figure 17 shows the relationship obtained from the analysis of Cases 1 and 10 to 15 between the
422 tapered angle of the joint surface and its shear capacity relative to the results of the experiment.
423 Although there are three experimental results in the figure, the results of the analysis roughly agree
424 with those of the experiment, indicating that the simulation method is capable of roughly estimating
425 the relationship between the tapered angle of joint surfaces and their shear capacity. It was also found
426 that greater tapered angles resulted in lower shear capacities on the joint surface.

427

428 4.3.3 Summary of the analysis results

429 A simulation conducted using the same joint surface roughness that was used in the double shear
430 tests simulating the spalling phenomenon yielded the following findings:

- 431 1) The simulation method is capable of roughly estimating the relationship between the joint aperture
432 and the shear capacity and that between the tapered angle of the joint surfaces and the shear
433 capacity.
- 434 2) Wider crack widths resulted in lower shear stress and shear capacity of the joint surface.
- 435 3) Greater tapered angles of the joint surface resulted in lower shear capacity of the surface.

436

437 5. EXAMINATION OF JOINT SURFACE ROUGHNESS

438 5.1 Joint surface roughness of actual tunnels

439 Samples of a cracked plain concrete lining were collected by core boring at railway tunnels (double

track, around 30 to 40 years old), as shown in Figure 18, and the roughness of the joint surfaces was measured using the method described in 3 (1) b). In rock engineering, indices such as Z_2 and JRC have been proposed for the quantification of the joint surface roughness, while equations have been developed for the relationship between Z_2 (Tse and Cruden, 1979) and the JRC (Barton and Choubey, 1977). With the core samples, the roughness of the joint surfaces was measured at 0.5-mm intervals over an area measuring 40 mm by 40 mm. To evaluate the roughness in a simplified manner, Z_2 and then the JRC were calculated as follows:

$$Z_2 = \left[\frac{1}{M-1} \sum_{i=1}^{M-1} \left(\frac{y_{i+1} - y_i}{\Delta x} \right)^2 \right]^{1/2} \quad (16)$$

where:

Δx : measuring interval

y_i : height of i^{th} measuring point

M : number of measuring points on a measuring line

$$JRC = 64.22Z_2 - 2.31 \quad (17)$$

Figure 19 shows the JRC values of the 16 joint surface samples collected from a number of tunnels and those of two cold joint samples that were also collected.

The JRC values of the joint surfaces are distributed over a range of 20 to 36, averaging 28.1. The JRC values of the cold joint surfaces were smaller than those of the joint surfaces, indicating that the cold joints had lower shear capacity. Cold joints can have a significant impact on spalling as shown by the incident (Ministry of Transport, 2000). It must be noted that cold joints have different characteristics to those of other joints in a closed form with respect to shearing and spalling.

5.2 Surface roughness made in the shear tests on a concrete beam

The double shear tests used a joint surface that was made from a shear test on a concrete beam with the maximum aggregate size measuring 20 mm. Considering that the maximum aggregate size used in railway tunnels that have been built in recent years normally measures 40 mm, a test piece of concrete with the maximum aggregate size measuring 40 mm was made and then subjected to a shear test to obtain the rough surfaces of the cracks. The surfaces were measured at 0.5-mm intervals. A test piece of mortar was also made by following the same procedure, but using no aggregate, and it was also measured at 0.5-mm intervals.

Table 3 shows the *JRC* values that were calculated using the same method as in a). The table reveals that the *JRC* value of the joint surfaces of actual tunnels is between the *JRC* value of the mortar beam joint surfaces and that of the joint surfaces of the concrete beam with the maximum aggregate size measuring 40 mm. In addition, the *JRC* value of the joint surface used in the double shear tests is close to the average *JRC* value of the joint surfaces sampled from actual tunnels. This confirms the validity of using the joint surface used in the double shear tests as a typical roughness profile of the joint surfaces of actual tunnels.

The shear capacity of the joint surfaces of the concrete and mortar beams that were made in the shear tests was calculated. Figure 20 shows the relationship between the calculated shear capacities and the maximum aggregate size. As part of the calculation, the uniaxial compressive strength was 15, 22, 29 36, 43 and 50 N/mm² and the tapered angle of the joint surfaces was 0°. As shown in the figure, the shear capacity increases as the maximum aggregate size, or the joint surface roughness, is greater. The shear capacity also increases as uniaxial compressive strengths is larger.

6. EXAMPLE OF CALCULATION OF THE SAFETY AGAINST SPALLING IN ACTUAL TUNNELS

6.1 Outline of calculation

The safety against spalling of the lining concrete was calculated using an index, which was the shear capacity divided by the shear stress of the joint surfaces. A parameter study was conducted

490 using various width B , depth D , and thickness H of a chunk of concrete. Assuming plain concrete with
491 the maximum aggregate size measuring 20 mm, the shear capacity was calculated using the roughness
492 profile used in the double shear tests. In the investigation (Miura et al., 1959), in which core samples
493 of the lining were collected before it was removed as part of the tunnel reconstruction and the samples
494 were subjected to uniaxial compression tests, the uniaxial compressive strength of the samples ranged
495 from 15 N/mm² to 26 N/mm², as shown in Table 4. Then, the uniaxial compressive strength was set at
496 15 and 26 N/mm². The angles of the joint surfaces found during the core sampling described in 5.1
497 were 30° or less. Accordingly, the tapered angle of the joint surfaces was set as to be 0° and 30°. As
498 for the wind pressure induced by the train passage, the variation in air pressure (5 kN/m²) (Ministry of
499 Transport, 2000) measured on the Sanyo Shinkansen line during a train passage was used.

500

501 6.2 Calculation results of the safety against spalling

502 The calculated safety against spalling relative to joint aperture is shown in Figures 21 when the
503 tapered angle of the joint surfaces is 0°. It was found that wider crack widths and larger width B and
504 depth D of chunk of concrete results in lower safety factors against spalling. The safety against
505 spalling is still above 1 even with a joint aperture of 5 mm. Figure 22 shows the calculated safety
506 against spalling when the tapered angle of the joint surfaces is 30°. The tendency of results
507 corresponds to those in Figure 21. Table 5 shows the safety against spalling obtained by Figure 22
508 under the various size of chunk and uniaxial compressive strength. The safety against spalling
509 increases as uniaxial compressive strengths is larger. In some cases, the safety against spalling is
510 below 1 with a joint aperture of 3 mm. Accordingly, the results of the calculation show that the safety
511 against spalling is considered high when a joint with an aperture of 3 mm or more closes.

512 The calculations considered only the chunk's own weight and the train-induced wind pressure as
513 factors contributing to the falling of the chunk. When ground pressure on the tunnel lining is
514 considered, however, the shear stress on the joint surfaces will increase. This aspect requires further

515 study.

516

517 7. CONCLUSION

518 In order to evaluate the spalling of tunnel lining concrete quantitatively, it has been discussed the
519 mechanism through the actual field investigations and its modeling has been presented. Based on this
520 modeling, then, the double shear tests have been conducted. And, the simulation method, which was
521 applied to clarify the shear behavior of rock joints, has been modified and applied to the spalling of
522 tunnel lining concrete. Next step, the roughness of the cracks in the actual tunnel lining has been
523 measured and the crack width and safety factor against spalling relations has been estimated using the
524 proposed simulation method. The knowledges from this study are summarized below.

- 525 • The spalling of tunnel lining concrete was modeled and an equation to calculate the shear stress on
526 the joint surfaces was proposed. Based on that, a method was proposed for quantitatively evaluating
527 the safety against the spalling of the tunnel lining concrete by comparing the shear stress acting on
528 the joint surfaces with the shear capacity of the surfaces.
- 529 • Double shear tests, simulating the spalling phenomenon, found that wider crack widths resulted in
530 lower shear capacities of the joint surfaces. Greater tapered angles of the joint surfaces resulted in
531 lower shear capacities.
- 532 • A method for simulating the shearing process of joint surfaces was proposed. The method was
533 shown to be able to roughly estimate the relationship between the joint aperture and the shear
534 capacity and that between the tapered angle of the joint surfaces and the shear capacity. Wider
535 crack widths and greater tapered angles of the joint surfaces resulted in lower shear capacities of the
536 joint surfaces with specific rates of decline.
- 537 • The *JRC* value of the joint surface samples collected from actual tunnels was between the *JRC* value
538 of the mortar beam joint surfaces that were made in the shear test and that of the joint surfaces of
539 the concrete beam, made in the shear tests with the maximum aggregate size measuring 40 mm.

- Based on a parameter study, it was found that larger width B and depth D and smaller thickness H of chunk of concrete and wider crack widths result in lower safety factors against spalling. The safety against spalling also increases as uniaxial compressive strengths is larger.

It is planned that a quantitative method will be established in the future to judge soundness α , β , and γ by accumulating more data on joint surface profiles and using the proposed method. A simulation method will also be used to examine the effect of measures in preventing spalling.

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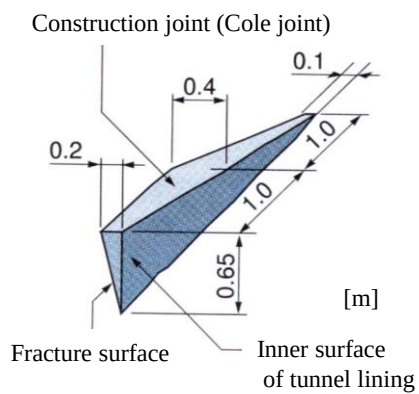


Figure 1 Lining of Fukuoka Tunnel after spalling (Asakura et al., 2001)

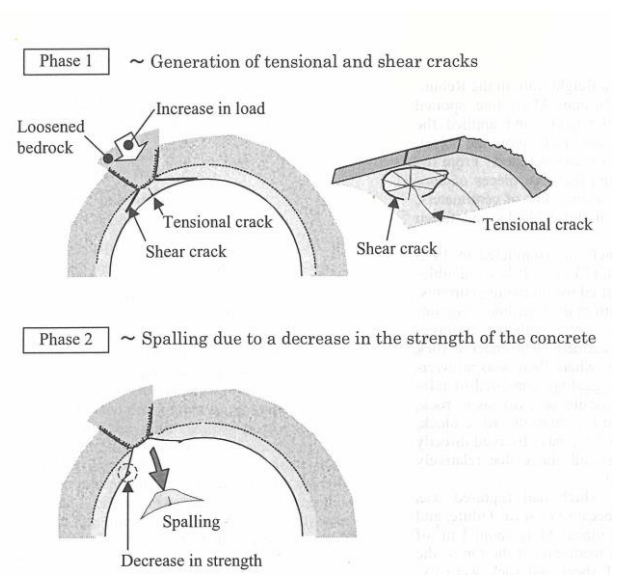
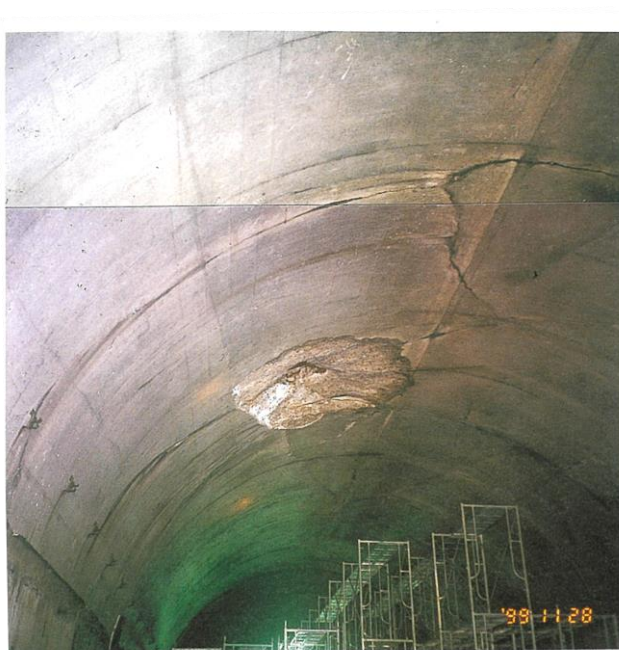


Figure 2 Lining of Rebunhama Tunnel after spalling (Asakura et al., 2001)

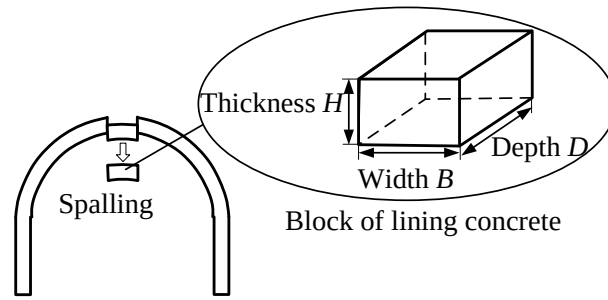


Figure 3 Modeling concepts of spalling

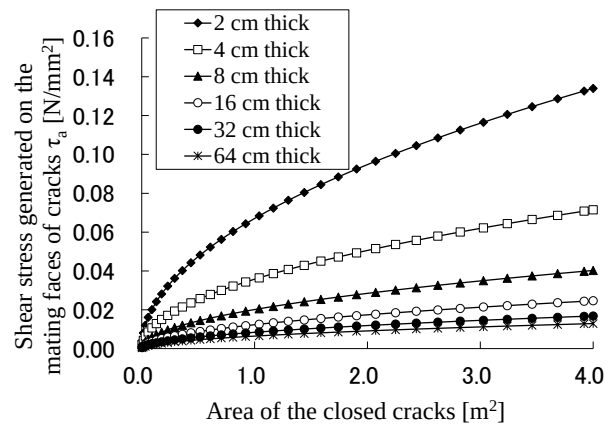
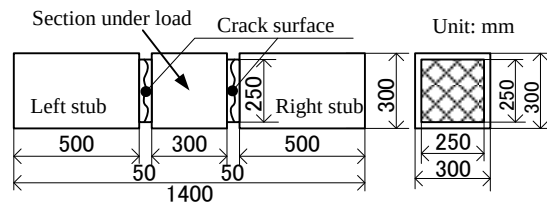
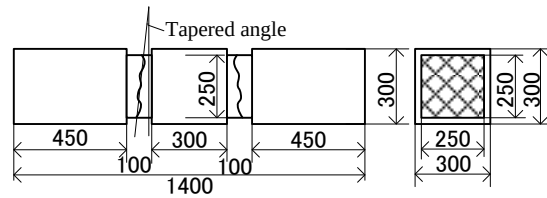


Figure 4 Calculated shear stress generated on the joint surfaces of cracks

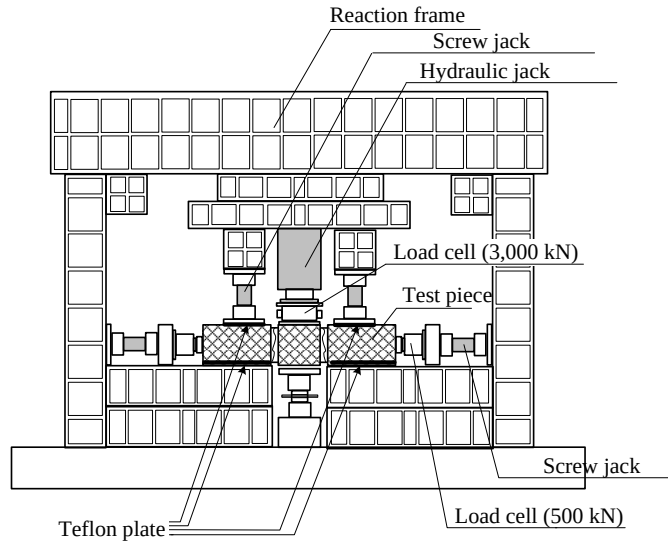


(a) No. 1 to No. 5 (with tapered angle of 0°)

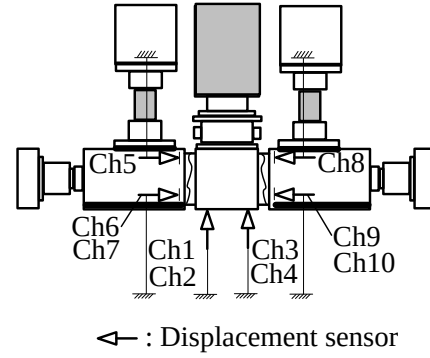


(b) No. 6 and No. 7 (with tapered angles of 5° and 10°)

Figure 5 Test pieces for double shear tests



(a) Outline of the loading device



(b) Location of displacement sensor

Figure 6 Outline of the test device

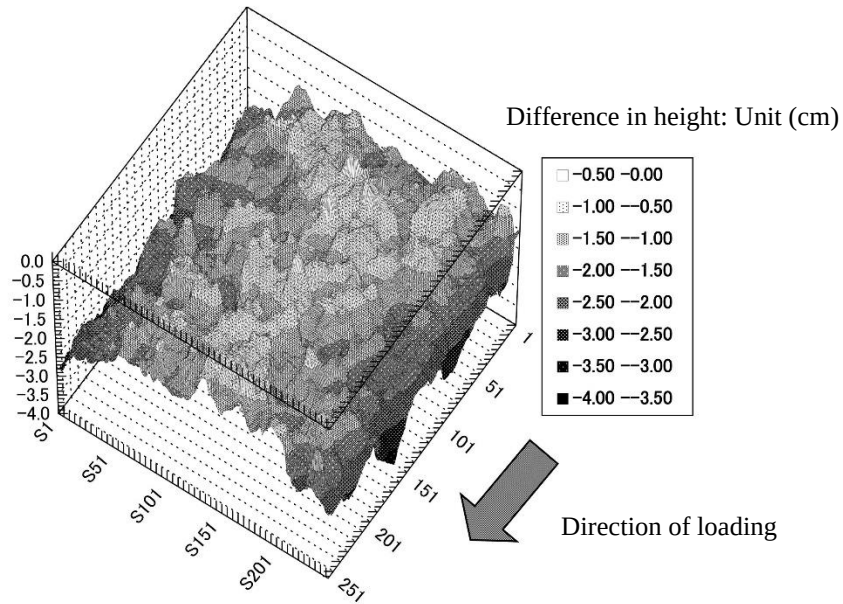
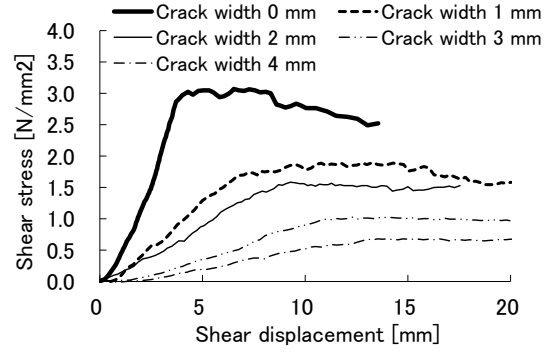


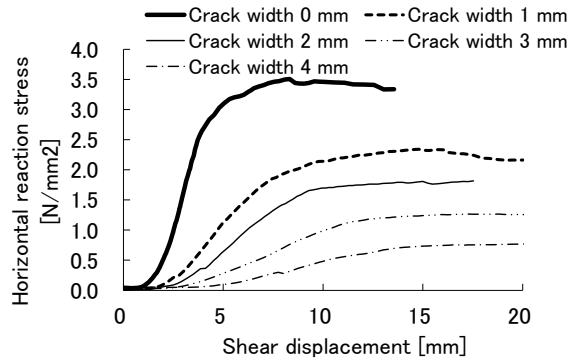
Figure 7 Measured roughness of joint surfaces

Table 1 Test cases

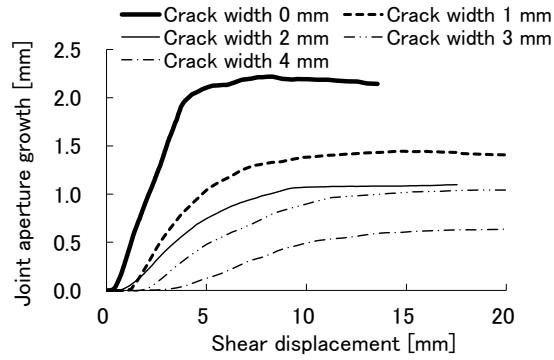
No.	Crack width	Tapered angle of joint surface
1	0 mm	No taper (0°)
2	1 mm	No taper (0°)
3	2 mm	No taper (0°)
4	3 mm	No taper (0°)
5	4 mm	No taper (0°)
6	0 mm	5°
7	0 mm	10°



(a) Relationship between shear displacement and shear stress



(b) Relationship between shear displacement and horizontal reaction stress



(c) Relationship between shear displacement and joint aperture growth

Figure 8 Test results of test pieces with various crack widths

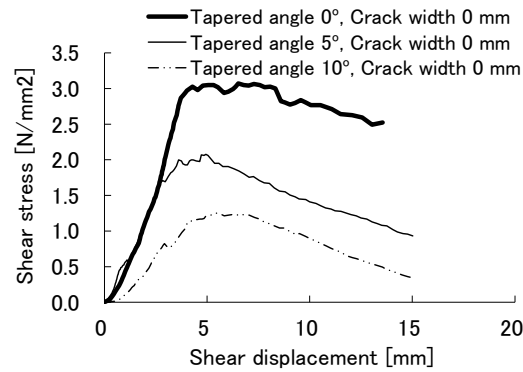


Figure 9 Test results of test pieces with various tapered angles of joint surfaces

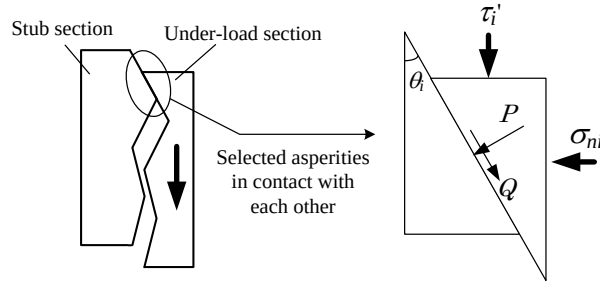


Figure 10 Asperities in contact with each other under stress

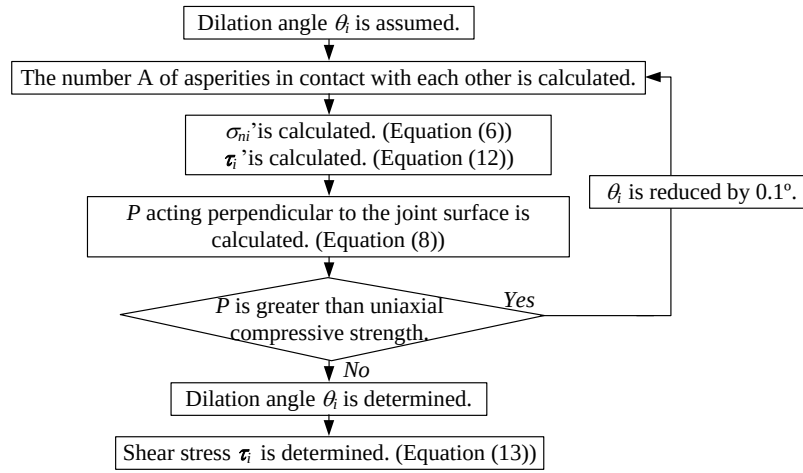


Figure 11 Flow chart in Step i

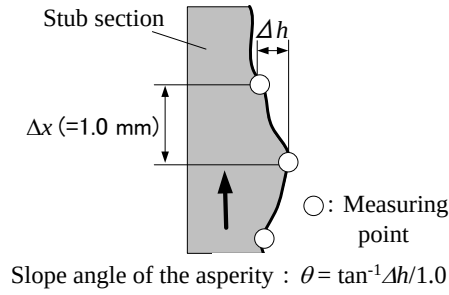


Figure 12 Slope angle of an asperity

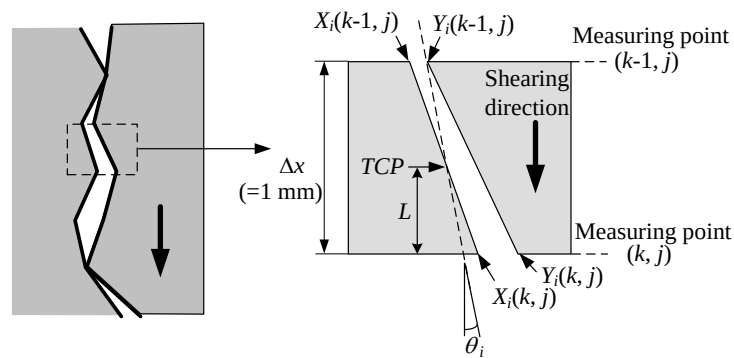


Figure 13 Concepts of *Weight* and *TCP*

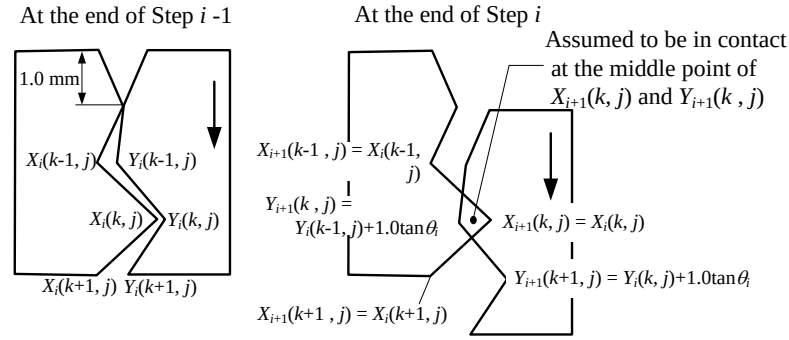
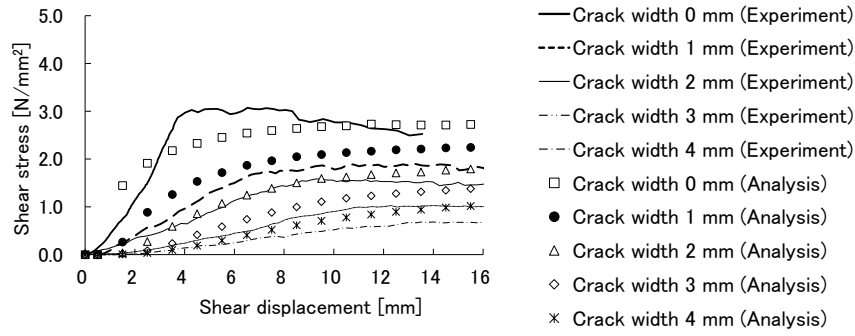


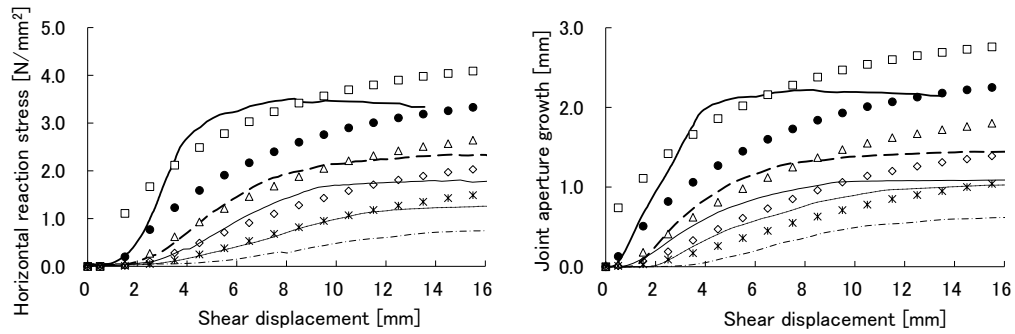
Figure 14 Coordinates at the end of Step i

Table 2 Analysis cases

No.	Crack width	Tapered angle of joint surface	Material strength (Uniaxial compressive strength)
1-9	0, 1, 2, 3, 4, 5, 6, 7, 8 mm	0°	36 N/mm ²
10-15	0 mm	5, 10, 15, 20, 25, 30°	36 N/mm ²



(a) Relationship between shear displacement and shear stress



(b) Relationship between shear displacement and horizontal reaction stress

(c) Relationship between shear displacement and joint aperture growth

Figure 15 Results of analysis in comparison with those of experiment (crack width of 0 to 4 mm)

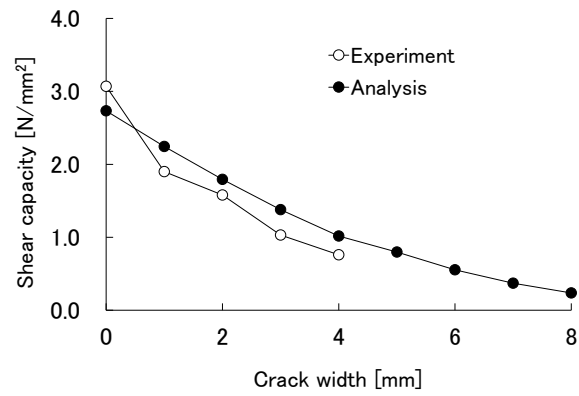


Figure 16 Relationship between joint aperture and shear capacity

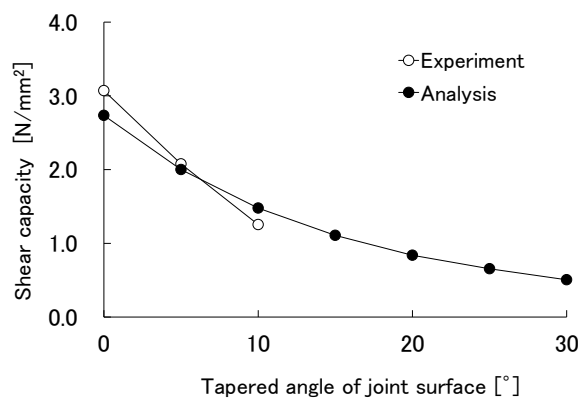


Figure 17 Relationship between tapered angle of joint surface and its shear capacity



(a) Sample A



(b) Sample B



(c) Sample C

Figure 18 Core samples of joint surfaces

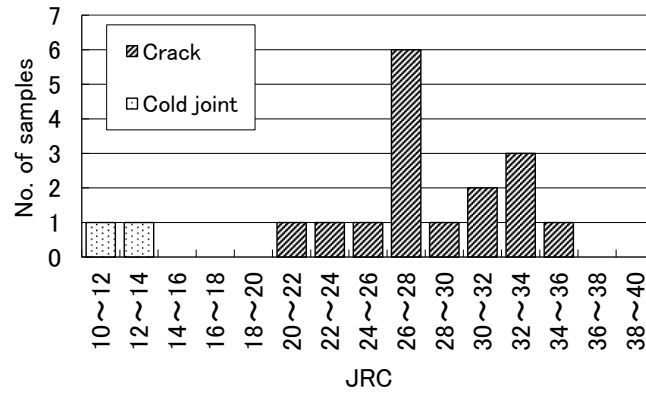


Figure 19 JRC values of joint surfaces on actual tunnels

Table 3 JRC values of joint surfaces made in beam shear test

Maximum aggregate size	JRC value
Without aggregate (mortar)	22
40 mm	36
20 mm	29

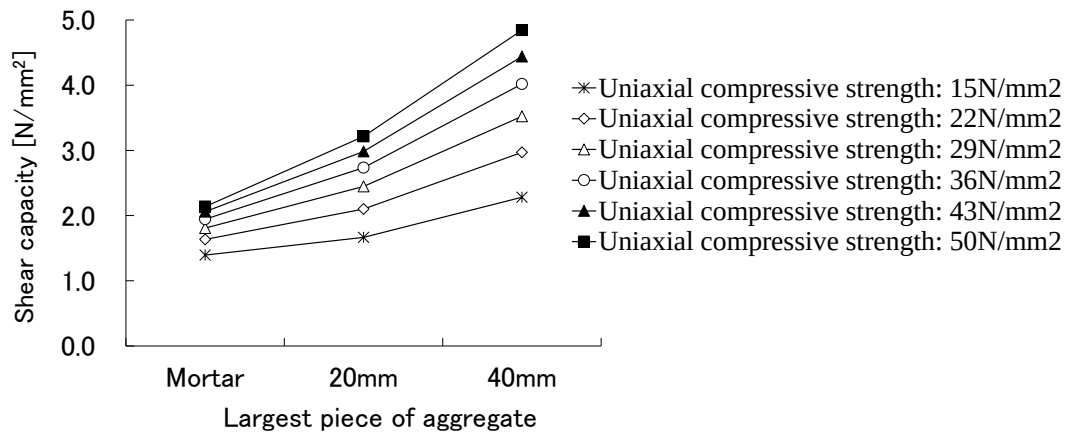
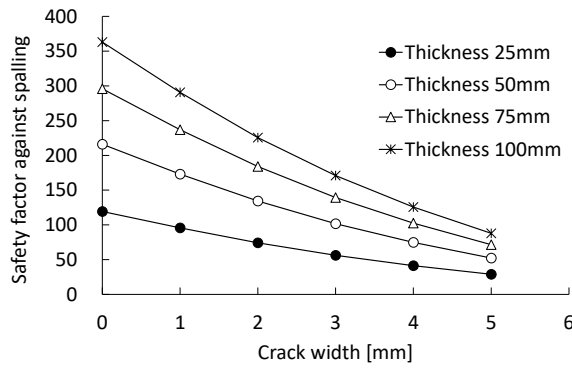


Figure 20 Calculated shear capacity (Tapered angle of joint surface: 0°)

Table 4 Uniaxial compressive strength of lining concrete (Miura et al., 1959)

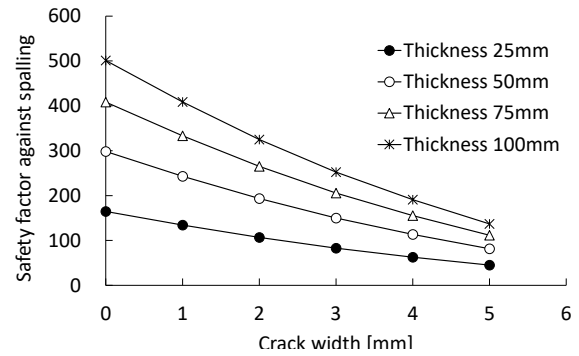
Tunnel	Age*	No. of test pieces	Uniaxial compressive strength
No.3 Otoshibe (Tunnel arch)	14 years	6	25.8 N/mm ²
Obusu Tunnel (Side wall)	20 years	6	17.6 N/mm ²
(Arch)		6	18.1 N/mm ²
Shimokuno Tunnel (Side wall)	26 years	6	23.2 N/mm ²
(Arch)		6	14.7 N/mm ²
No.2 Yubiso Tunnel (Side wall)	29 years	2	16.8 N/mm ²

*: At the time of the investigation



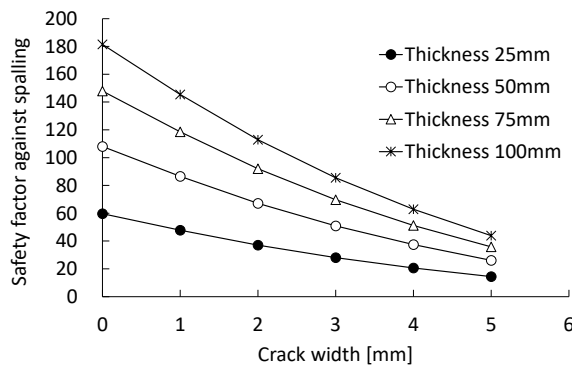
(a) $B = 250 \text{ mm}$, $D = 250 \text{ mm}$

Uniaxial compressive strength: 15 N/mm^2



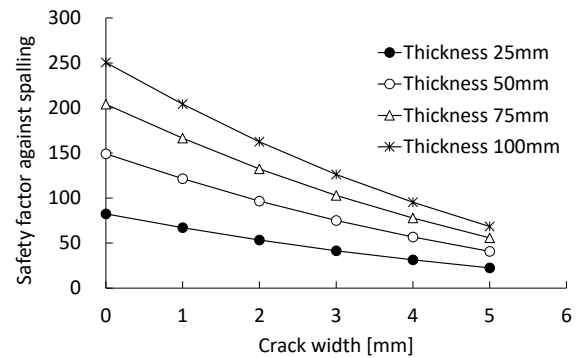
(b) $B = 250 \text{ mm}$, $D = 250 \text{ mm}$

Uniaxial compressive strength: 26 N/mm^2



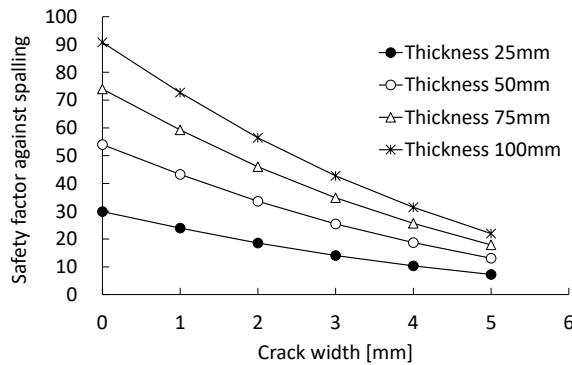
(c) $B = 500 \text{ mm}$, $D = 500 \text{ mm}$

Uniaxial compressive strength: 15 N/mm^2



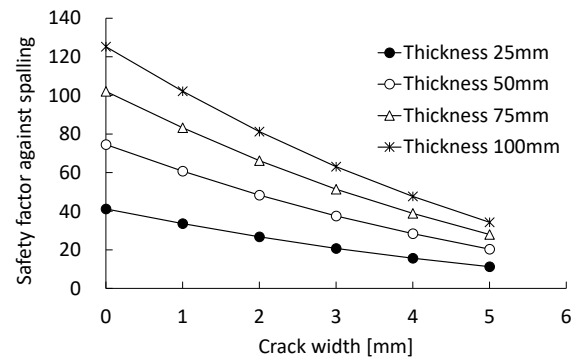
(d) $B = 500 \text{ mm}$, $D = 500 \text{ mm}$

Uniaxial compressive strength: 26 N/mm^2



(e) $B = 1000 \text{ mm}$, $D = 1000 \text{ mm}$

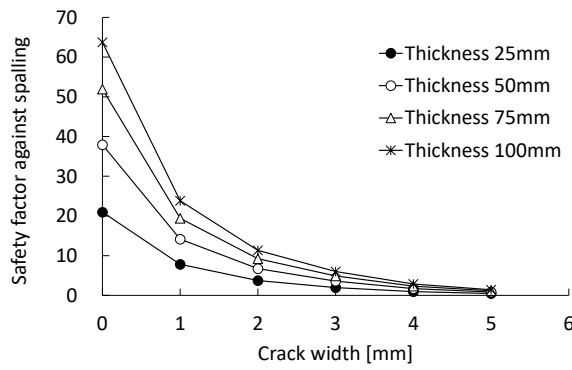
Uniaxial compressive strength: 15 N/mm^2



(f) $B = 1000 \text{ mm}$, $D = 100 \text{ mm}$

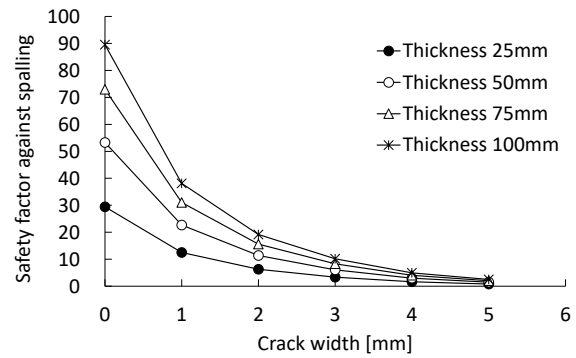
Uniaxial compressive strength: 26 N/mm^2

Figure 21 Calculated safety factor against spalling (Tapered angle of joint surfaces: 0°)



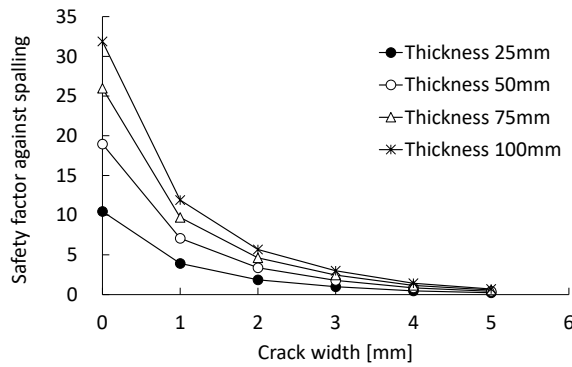
(a) $B = 250 \text{ mm}$, $D = 250 \text{ mm}$

Uniaxial compressive strength: 15 N/mm^2



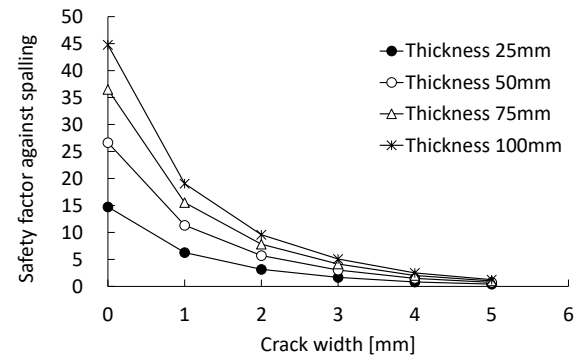
(b) $B = 250 \text{ mm}$, $D = 250 \text{ mm}$

Uniaxial compressive strength: 26 N/mm^2



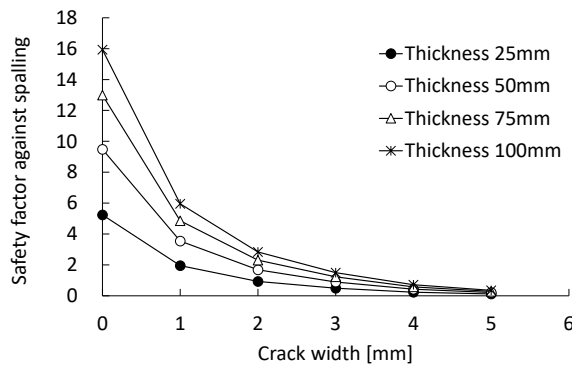
(c) $B = 500 \text{ mm}$, $D = 500 \text{ mm}$

Uniaxial compressive strength: 15 N/mm^2



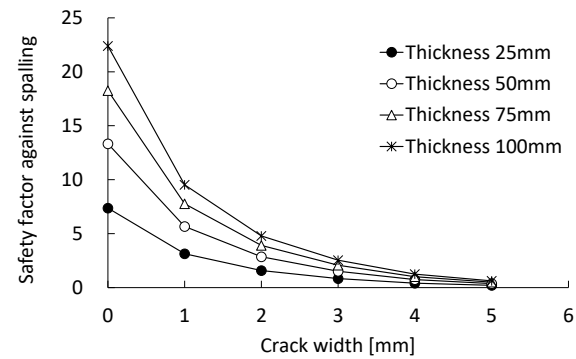
(d) $B = 500 \text{ mm}$, $D = 500 \text{ mm}$

Uniaxial compressive strength: 26 N/mm^2



(e) $B = 1000 \text{ mm}$, $D = 1000 \text{ mm}$

Uniaxial compressive strength: 15 N/mm^2



(f) $B = 1000 \text{ mm}$, $D = 100 \text{ mm}$

Uniaxial compressive strength: 26 N/mm^2

Figure 22 Calculated safety factor against spalling (Tapered angle of joint surfaces: 30°)

Table 5 Safety factor against spalling with taped angle of joint surface 30°

Size of chunk [mm]			Uniaxial compressive strength				
Width B	Depth D	Thickness H	15N/mm ²	22N/mm ²	26N/mm ²	29N/mm ²	36N/mm ²
250	250	25	1.98	2.78	3.34	3.69	4.50
		50	3.58	5.03	6.05	6.67	8.14
		75	4.90	6.89	8.29	9.14	11.14
		100	6.01	8.45	10.17	11.21	13.67
500	500	25	0.99	1.39	1.67	1.84	2.25
		50	1.79	2.51	3.03	3.34	4.07
		75	2.45	3.44	4.14	4.57	5.57
		100	3.00	4.22	5.08	5.61	6.84
1000	1000	25	0.49	0.69	0.84	0.92	1.12
		50	0.89	1.26	1.51	1.67	2.03
		75	1.22	1.72	2.07	2.28	2.79
		100	1.50	2.11	2.54	2.80	3.42