Stiffness Evaluation and Current Status of a Degraded Road Bridge Slab Located in a Mountainous Area

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Abstract

In the mountainous area of the Hokuriku region, bridges are experiencing early deterioration caused by salt damage, alkali-silica reaction (ASR), and frost damage. Survival time analysis was carried out using the inspection data to study the relationship between the degradation tendency of bridges and the regional characteristics. In addition, the causes of the degradation of the reinforced concrete (RC) slab of a road bridge, which deteriorated early, were investigated using cylindrical core extraction. Polarizing microscopic observation of the specimens collected from the slab confirmed that ASR was the cause of the deterioration. The reduction in the mechanical properties of concrete due to ASR was also studied and reported. Moreover, vehicle running tests using a test truck were carried out. Then, long-term monitoring of the responses of the test bridge due to live load based on the bridge weigh-in-motion method was also performed for ordinary vehicles. The stiffness of the RC slab was evaluated by comparing the results obtained from the tests and the numerical analyses. It was found that the current stiffness of the slab remarkably decreased as compared with the results when the slab was sound. Finally, this study proposes an approach for the soundness evaluation of RC slabs.

1. Introduction

While local bridges continue to age, the budget for their maintenance and management has decreased in the current fiscal situation owing to the reduction in tax revenue due to the declining birthrate in Japan. For bridges spanning more than 2 m, visual inspection is conducted once every 5 years. Because of the current budget constraint, it is not currently possible to implement repair and reinforcement systematically. In addition, measures are repeatedly carried out without evaluating the residual strength of the bridge after the inspection is completed.

In the Hokuriku region, salt damage, alkali-silica reaction (ASR), and frost damage cause early severe degradation of the bridges in the mountainous area without warning (Katayama et al. 2004; Torii et al. 2008; Arima et al. 2016; Ha et al. 2017). Although traffic volume is small, the influence of overloaded vehicles is also a safety concern. Under such a regional condition, how to evaluate the load-bearing capacity of these bridges and how to take appropriate countermeasures have become urgent issues. Moreover, the relationship between the degradation tendency of bridges and the regional characteristics causing salt damage, ASR, and frost damage is also a problem that needs to be solved. Since structural concrete usually experiences the combined effects of ASR, carbonation, freeze-thaw cycles, steel corrosion, and high-cycle fatigue loads (Gong et al. 2017), concrete degradation is a complex process. As a part of the present study, a statistical analysis (namely, survival analysis) (Maki et al. 2017) was performed using the inspection data for bridges in the Hokuriku region to solve the above problems. Specifically, a survival time analysis was conducted for the reinforced concrete (RC) slabs of the steel road bridges in the mountainous area that were built for the dam construction in the 1970s.

In addition, in a case study on the destruction process of RC slabs subjected to frost damage in the Hokkaido district, which is a snowy region, Mitamura et al. (2009) reported on the effect of freezing and thawing on fatigue life. Moreover, Ishikawa et al. (2011) reported on the damage condition of RC slabs that had been replaced owing to fatigue and salt damage caused by the anti-freezing agent on the expressway in the Hokkuri district. In 2016, Ha et al. carried out a destructive loading test to evaluate the sensitivity of nondestructive damage detection methods (Arima et al. 2016) and the residual load-bearing capacity and deformations of a full-scale prestressed concrete (PC) T-shaped girder that was removed nearly 25 years after its construction owing to salt damage. The result showed that, even though many damages were observed, the investigated girder performed a sufficient load-bearing capacity.

Since 1982, ASR has been observed in highway structures in this region, where deicing salts (mainly NaCl) are used in the winter (Katayama et al. 2004). Particularly in the Noto region, there is a wide distribu-
tion of andesite, which is the primary cause of ASR degradation in bridges (Torii et al. 2007, 2012; Torii 2010). Up to recent years, many infrastructures, not only in the Noto region but also in the Hokuriku region (Fukui, Ishikawa, Toyama, and Niigata prefectures), were also degraded by ASR (Torii et al. 2016; Kubo et al. 2016). The reaction starts the expansion of reactive aggregates by forming a swelling gel of alkali silicate hydrate (N(K)-C-S-H). The ASR gel increases in volume, exerting a significant pressure within the material, which causes spalling, cracking, and loss of strength in the concrete and rebar. The relationship between the characteristics of ASR-induced damage and the reduction in the elastic modulus have been proved by many previous studies (Giaccio et al. 2008; Ha et al. 2017). Over the years, many researchers have performed loading tests on both ASR-degraded PC and RC specimens.

Regarding RC structures, Miyagawa et al. (2006) studied the fracture mechanism of reinforcing steels in the case of concrete structures thoroughly damaged by ASR. The authors reported the results of the investigation on the fracture mechanism, nondestructive testing methods, and repair and strengthening methods for damaged concrete structures. In 2012, Wang and Morikawa studied the deterioration of concrete members due to ASR and its influence on shear failure mechanism and shear resistance performance of 1200-mm RC beams in both experimental and numerical approaches. The experimental results showed that undesirable shear collapse modes were obtained and that bond slip easily occurred in RC beams with ASR-induced cracks along the tensile reinforcing bars. In another study of Hajighasemali et al. (2014), the influence of ASR on the overall serviceability, strength, ductility, and deformability of RC beams under service load conditions was investigated by considering the creep and ductility phenomena. Twelve 1100-mm-long RC beam specimens were initially produced by utilizing nonidentical concrete mixtures containing reactive aggregates (R) and nonreactive aggregates (N). The authors’ conclusions proved that ASR has a significant effect on the R beams (where cracks occurred at lower loads) and that the N beams were stronger than their equivalent R specimens at the final loading.

In 1988, Kobayashi et al. described the structural performance of prestressed beams made of concrete affected by ASR in comparison with that of corresponding healthy beams. In their findings, the reduction in the ultimate strength of ASR beam was approximately 10% compared with that of the sound beam, even when the tensile strain in the vertical stirrup exceeded 1000 × 10^6 and many longitudinal cracks occurred. Takebe et al. (2013) verified the effects of the presence or absence of ASR deterioration on the shear strength of concrete members using two different PC specimens (height 300 mm × width 300 mm × length 2000 mm). The controlled beam contained normal aggregates, whereas the other one had ASR reactive aggregates in its mixture. The authors’ findings showed that both specimens failed in the flexural mode, with the final load of the ASR-affected beam being approximately 7% lower than that of the normal one. In a recent study, Hiroi et al. (2016) carried out a flexural loading test on real-scale large PC beams (height 1250 mm × width 1200 mm × length 7000 mm) that were exposed to 7.5 years of deterioration. Their results showed that the decreases in Young’s modulus and tensile strength were more obvious than that in the compressive strength and that the decreased rate was higher in the vertical cores in which confinement was absent than in the longitudinal cores. In another study by Ha et al. (2017), two PC girders, which were constructed and placed outside the laboratory, had been exposed to weather conditions for 1.5 years. The H girder was affected by ASR, whereas the FA girder was kept at an inactive state with ASR acceleration due to the addition of fly ash. The authors’ results proved that the addition of fly ash did not only increase the load-bearing capacity by nearly 5% but also enhanced the initial bending stiffness of the objective PC girder by 10% after more than one year under ASR deteriorations. Moreover, regarding ASR-affected specimens, andesite particles from sand and gravel reacted strongly, and many fine cracks (approximately 0.02 mm in width) occurred from broken pieces of aggregates to cement paste. Both the compressive strength and the elastic modulus of the cores taken from the H girder were smaller than those from the FA girder. Owing to the influence of fly ash content, the compressive strength and the elastic modulus of the FA specimen were approximately 1.3 to 2 times greater than those of the H specimen.

However, a few studies evaluated the status of full-size RC slabs under ASR deterioration. Using test specimens close to full size, Maeshima et al. (2016) analyzed the relationship between ASR deterioration and fatigue durability of RC slabs. Two years later, Takahashi et al. (2018) proposed a method using a data assimilation approach to estimate the remaining life of RC slab specimens deteriorated by ASR expansion and cyclic mechanical loads. Specifically, the responses of the real-scale RC slabs studied by Maeshima et al. (2016) were referred for verification and validation processes of the proposed model. Then, the authors developed a data assimilation procedure to combine the multi-scale analysis with visual site inspection data by extending the pseudo-cracking method for RC bridge decks (Fujiyama et al. 2013; Tanaka et al. 2017) to the case of ASR damages. In the result, it was shown that ASR expansion could enhance the fatigue life of RC slabs.

In this study, survival time analysis was firstly performed in order to examine the deterioration tendency of the RC slabs of steel road bridges over a specific route in the mountainous areas of the Hokuriku district where abnormal appearances were noticed. Based on the analysis results, the bridge group of the mountainous
route had a tendency to be deteriorated earlier than other bridges in the same district. Therefore, in order to identify the deterioration factor and evaluate the load capacity of the bridges, this study selected the RC deck slab of a steel road bridge in service from a group of bridges in a mountainous area suspected of being deteriorated by ASR (see Figs. 1 and 2). Then, degradation factors were identified from various analyses by core extraction and visual observation. The status of the bridge was evaluated by the change in the mechanical properties, the observation results of ASR gel under a polarized microscope, and the salt content analysis. In addition, the application of a soundness evaluation method based on a vehicle running test has attracted notable consideration recently. Kartopol'tsev et al. (2017) assessed the dynamic properties and stiffness of RC roadway slabs under live loads by considering the pavement defects. In addition, Miyamoto et al. (2017) proposed an effective monitoring method (called the "bus monitoring system") for condition assessment of existing short- and medium-span (10-30 m) RC/PC bridges using a public bus as part of a public transit system, and safety indices, namely, “characteristic deflection.” Regarding the present study, because the analysis result on a concrete core provides a local evaluation at a representative point, to evaluate the entire stiffness of the RC slab, vehicle running tests using a large test truck and a long-term monitoring of the responses of the test bridge due to ordinary vehicles based on the bridge weigh-in-motion (BWIM) method were performed. Moreover, this study proposes an approach to clarify the soundness of ASR-deteriorated RC slabs based on both the experimental and the numerical results of the displacement of the RC slabs.

2. Survival time analysis for routes in mountainous areas

In the Hokuriku district, salt damage, ASR, and frost damage are cited as the reasons leading to the early deteriorations of concrete structures, although they are closely related to site conditions. In this study, a survival time analysis using bridge inspection data was carried out that considered the deterioration tendency of the RC slabs in the mountainous routes.

2.1 Outline of the survival time analysis

Survival analysis is a statistical technique that focuses on the relationship between time and events from a certain point of time until an event occurs. This method is often used in medical and engineering fields. Although it is common to deal with deaths and faults in the medi-
Yamazaki et al. (2015) conducted a survival time analysis using inspection results in the Tohoku region for application in the maintenance of bridges and analyzed the importance and risk determination for each degradation mechanism.

The survival time \( T \) until an event occurrence is set as a random variable, and the probability density function \( f(t) \) is expressed by the following equation:

\[
f(t) = \lim_{\Delta t \to 0} \frac{\Pr(t < T < t + \Delta t)}{\Delta t}
\]

On the assumption that the cumulative probability distribution function is \( F(t) \), the survival function \( S(t) \) at a specific time point \( t \) is given by the following equation:

\[
S(t) = \Pr(T > t) = 1 - \Pr(t \leq T) = 1 - F(t)
\]

Under the condition that an event has not occurred by time \( t \), the hazard function \( h(t) \) indicating the instantaneous mortality rate at which the event occurs at the next moment \( t + \Delta t \) is given by the following equation:

\[
h(t) = \lim_{\Delta t \to 0} \frac{\Pr(t < T < t + \Delta t | T \geq t)}{\Delta t} = \frac{f(t)}{S(t)}
\]

Furthermore, the relationship between the cumulative hazard function \( H(t) \), the survival function \( S(t) \), and the hazard function \( h(t) \) is as follows:

\[
H(t) = \int_0^t h(t)dt = - \log S(t)
\]

When dealing with the survival time, it is necessary to consider the case where no event is observed before the end of the investigation as “censored.” The Kaplan-Meier method is an estimation method that considers censoring, and the Kaplan-Meier estimator \( \hat{S}(t) \) is given by the following equation:

\[
\hat{S}(t) = \Pi_{t_i \leq t} \left(1 - \frac{d_i}{r_i}\right)
\]

where \( d_i \) is the number of deaths at time \( t_i \) and \( r_i \) is the number of observers that may have an event just before time \( t_i \).

2.2 Survival curve of the target route

(1) Outline of the study

The bridges over a specific route in a mountainous region in the Hokuriku district were regarded as the “applicable group,” whereas the other bridges were regarded as the “nonapplicable group.” Then, an analysis was performed on both groups to investigate the relationship between the deterioration tendency of the RC slabs and the location conditions. Here, the definition of event occurrence refers to a case where damage requiring repair was found during a bridge inspection. According to the inspection record of the district, the bridge soundness was classified into five grades, from grade 5 (mild) to grade 1 (severe). This study considered the healthy degree of grade 3 or less as “damaged,” and it was regarded as an event occurrence.

The application of a filter such as classification class and construction year allowed a comparative examination of both groups. The number of steel road bridges having RC slabs in the target area is 276. Among them, the number of bridges in the applicable group built for the dam construction in the 1970s was 17 bridges. Here, “bridge age” indicates the elapsed year from construction to inspection. Because the RC slabs of the route have not been repaired extensively so far, the restoration of bridge soundness through repair was not considered and, therefore, its impact on the result was considered small. For the survival time analysis, the survival package of the statistical software R ver. 3.2.5 was employed (R Core Team 2017).

(2) Study results

The survival time obtained by the Kaplan-Meier method for the applicable and nonapplicable groups is shown in Fig. 3. The figure shows that the survival curve of the nonapplicable group gradually decreased, whereas that of the applicable group showed a sharp decline after 35–45 years. Although there was a difference in the number of samples and in the distribution of bridge age in both groups, it was clear that the RC slabs of the applicable group tended to deteriorate prematurely. Because the applicable bridge group is designated as an emergency transportation route, it is necessary to clarify the load carrying capacity and to determine the optimal repair and reinforcement work. Therefore, various analyses were performed on a steel road bridge selected from the applicable bridges in the mountainous area of the Hokuriku district to identify the degradation factor of its RC deck slabs and to evaluate the load carrying capacity. The results and discussions are presented in the following sections.

3. Description of the test bridge

As a part of the national highway, the bridge was con-
constructed in a mountainous area (altitude: 559 m) 45 years ago (completed in 1973). As can be seen in Fig. 1, the bridge is a simple composite bridge with a length of 36.0 m and four main girders. Moreover, because the A1 side is lower than the A2 side, the skew angle and the longitudinal gradient are 60° and 2%, respectively. The main girder spacing and the deck thickness are 2.5 m and 180 mm, respectively. Moreover, the volume of traffic (small cars + large vehicles) traveling on this bridge and the ratio of large vehicles according to the 2015 Census were 1177 vehicles per day and 20%, respectively (MLIT 2016). Because of the small traffic volume, it was difficult to consider that fatigue was a cause of the deterioration in this bridge.

4. Appearance investigation

Figure 4(a) shows the situation of patching of the pavement, which seemed to have been repaired several times at the vicinity of the joint. Patches were also confirmed at some places on the pavement. In addition, efflorescence was observed at the bottom of the RC slab and near the joint (see Fig. 4(b)). Signs of water permeation also appeared under the RC slab and near the span center, as shown in Fig. 4(c). Furthermore, the wheel guard section on the bridge surface (Fig. 4(d)) had many horizontal cracks in the longitudinal direction of the bridge.

5. Analysis of ASR-induced deteriorations using drilled cores

5.1 Overview of the experiment setup

Cylindrical core samples with a diameter of 55 mm were collected from the RC slab of the test bridge to investigate the mechanical properties of concrete such as the neutralization depth, the compression strength, the static modulus of elasticity, and the salinity.

Two sampling locations were selected with and without efflorescence (dry part and water-leakage part) on the bottom of the slab. The slab thickness at the dry part and the water-leakage part (near the joint) is 180 mm.
and 190 mm, respectively. The pavement thickness is 50 mm for both cases.

Figure 5 shows the observation result under a polarized microscope (open nicol) of the concrete core collected from the water leakage part. The figure shows that cracks propagated from the andesite particles to the cement paste and that they were filled with ASR gels. Therefore, it was inferred that the main cause of deterioration in the RC slab was ASR.

5.2 Compressive strength and static elastic modulus

The compressive strength and static elastic modulus were calculated by performing compression tests using cores collected at two places, i.e., the dry part and the water-leakage part. The results are summarized in Table 1.

Table 1 Compressive strength and coefficient of static elasticity.

|                     | Dry part | Water-leakage part |
|---------------------|----------|--------------------|
| Static elastic modulus (kN/mm²) | 21.5     | 10.0               |
| Compressive strength (N/mm²)     | 32.3     | 21.2               |

The table shows that the compressive strength of the specimens was reduced by approximately 30% in the water-leakage part as compared to that in the dry part. Regarding the elastic modulus, the results obtained from the water-leakage part showed that it decreased to about half of that obtained from the dry part. Previous studies pointed out that the elastic modulus was significantly degraded by the ASR-induced deterioration (Kubo et al. 2006; Ha et al. 2017). This outcome was also confirmed by the results on the RC slab of the test bridge. Because the test bridge was a composite bridge and the design strength and the static elastic modulus of concrete were 35 N/mm² and 29.5 kN/mm² for the RC slab, respectively, it was inferred that the reduction in compressive strength of both the dry and water-leakage parts affected the rigidity and load-bearing capacity of the entire structure.

5.3 Salt content analysis

Regarding the salt content analysis of the dry part and the water-leakage part, the sampled cores were sliced in 10-mm pitch. Then, the total chloride ion content was determined by a potentiometric titration method, and the chloride ion concentration distribution is shown in Fig. 6. The figure shows that the chloride ion concentration in the dry part was almost zero, whereas in the water-leakage part, it was estimated to be approximately 1.32 kg/m² at the concrete cover of the lower rebar, and that the concentration on the lower surface was considerably high. In addition, the measured results did not exceed the limit value of steel corrosion occurrence, which could be determined from the proposed formulas (W/C + 3.4; estimated W/C: 55% – 60%) by the Standard Specification for Concrete Structures of Japan (JSCE 2012). In addition, corrosion of the reinforcing bars was not observed at the place where the core was collected.

The neutralization depth is also shown in the same figure. On the lower surface, it was 1.3 cm in the dry part and 1.5 cm in the water-leakage part, and neutralization did not progress as the deck is over 40 years old. The explanation of this outcome is that rainfall and snowfall frequently occur in the mountains in the Hoku-riku district, the humidity continues to be high, and the humid environment slows the progress of neutralization. Moreover, because the bridge, which was built in the 1970s, does not have a waterproof layer, it is presumed
that the wet environment causes ASR to progress continuously, resulting in the deterioration of the RC slab.

6. Vehicle running test

6.1 Test description

As a preliminary stage of carrying out live load monitoring for ordinary vehicles on the target route, a vehicle running test using a test truck with a known load was conducted to investigate the response waveforms of each structural member and to estimate the conversion factors of the BWIM method. Because the result of the core extraction shown in the previous sections provided the evaluation at a representative point, another purpose of the running test was to investigate the entire rigidity of the RC slab. Regarding the test vehicle, a tri-axle truck having leaf springs was employed, as described in Fig. 7. In addition, the load distribution to each wheel axle was obtained by sequentially loading each axle of the vehicle (one axis at a time with or without loading) on a vehicle weighing scale installed on the site, as summarized in Table 2.

The arrangement of measurement points in the running test is shown in Fig. 8. The double circles in the figure show the locations where the displacements of the RC slab and the main girders were measured (see Fig. 8). For the displacement of the RC slab, immovable beams as the fixed line were installed just below the upper flange between the main girders, and then the relative displacement of the slab with the beam was measured (see Fig. 9(a)). In addition, a steel stretch rod was lowered from the immovable beam to the ground to obtain the displacement with the ground (see Fig. 9(b)). The displacements of the RC slab and the main girders were measured with the CDP-10 and CDP-25 displacement transducers (Tokyo Sokki Kenkyujo), respectively. The measurement locations of both the RC slab and the main girder displacements were in the state of being wet. Furthermore, strain gauges were installed at the bottom of the RC slab and at the lower flanges of the main girders to estimate the vehicle load. For discriminating the running speed and the number of axles, pi-shaped gauges and other strain gauges were installed under the RC slab near the pier wall and at the vertical stiffeners at the main girder ends, respectively. The test truck ran in both directions with running speed patterns of 20, 40, and 60 km/h. To be close to the conditions of actual traffic under one side regulation, the running speed (20–60 km/h) and running directions were changed and tested multiple times for each lane, as shown in Table 3 and Fig. 10. In addition, the sampling frequency was set to 200 Hz.

6.2 Displacement response by the running test

Figure 11 shows the displacements of the main girder and the RC slab at the quarter span location when the
test truck traveled on the downstream side lane (from A2 to A1 direction) at approximately 20 km/h.

Since the dynamic components can be observed even at 20 km/h, as for the displacement of the main girder, a low-pass filter by fast Fourier transform of 2.0 Hz was applied to eliminate the vibration around 3 Hz of the sprung mass of the vehicle. For the RC slab displacement where an unsprung vibration of 10–20 Hz was dominant, filter processing by moving average (a low-pass filter equivalent to 3.5 Hz) was performed.

The result of the filtering process showed that the displacement of the main girder became one large mountain-shaped waveform and that the maximum value was ~3 mm. With regard to the displacement of the RC slab, it was possible to confirm two large mountain-shaped waveforms for the front wheel and the two rear wheel axles (front rearmost axle and second rearmost axle). The maximum displacement on the rear wheel side was ~0.25 mm.

Displacement occurred approximately in the range of the front and back panels of the observation point, and the maximum of the slab displacement (after filtering) occurred when two rear axle parts passed over the observation point. Therefore, it can be seen that the load of two rear wheel axles exerted a large influence on the slab displacement (Fig. 11(b)).

| Traveling position | Case 1 | Case 2 | Case 3 | Case 4 | Case 5 | Case 6 | Case 7 |
|--------------------|--------|--------|--------|--------|--------|--------|--------|
| Outside            |        |        |        |        |        |        |        |
| Center             | 40     | 60     | 40     | 60     | 40     | 60     | Inside |
| Inside             |        |        |        |        |        |        |        |

![Image](image_url)
6.3 Calibration work and estimation accuracy of the BWIM method

The BWIM approaches used for the control and monitoring of the weight of trucks on the bridges have three methods focusing on the peak value of the member response (Fukada et al. 2011), the influence area, and the reaction force (Ojio 2006). This study focused on the peak value of the member response that requires an influence line of the bridge member and sensors for detecting vehicle axle arrival time. The calibration value was estimated by dividing the known load of the test truck by the average of the peak strain values obtained from the traveling cases when the test truck passed through the bridge for the first time in each direction. The gross weight of the test truck was estimated again from the average of the peak strain values obtained from the second time of the running test in each direction and from the obtained calibration value. The estimation accuracy of the method was confirmed by comparing the estimated weight with the vehicle weight measured beforehand shown in Table 2. When evaluating the running load with the dynamic waveforms, the BWIM method has a tendency to decrease in accuracy owing to the unevenness of the road surface and to the level difference in the vicinity of the measurement devices.

From the result of the vehicle running test, Fig. 12 shows the response waveforms after the low-pass filtering of the strains of the main girder and the RC slab when the test truck traveled on the downstream lane (A2 to A1) at approximately 40 km/h. The strain response waveforms of the main girder and the RC slab shown in this figure were the sum of the strain value of the strain gauges at each target area surrounded by a box in Fig. 8 for reducing the load estimation error due to the running position. In Fig. 12, the main girder strain shows one large mountain-shaped waveform, whereas two mountain-shaped waveforms could be observed with respect to the strain response of the RC slab. The peak value of the main girder strain was used to calculate the total weight, whereas the two peak values of the slab strain corresponded to the weights at the front axle and the two rear axles of the test truck. The conversion coefficient of the BWIM method was generated from the correlation of each response waveform acquired multiple times from the test cases (see Table 3) and the weight of test truck. At first, the peak strain values of the response waveforms of seven test cases in each lane were averaged. Then, the corresponding load was divided by the average peak strain value to estimate the conversion coefficient from the strain values to the loads. Using the obtained conversion factor (Table 4), the running load was estimated again from the result of the second run, and the comparison result of the actual load and the estimated load is shown in Fig. 13. In addition, the dotted

Table 4 Conversion coefficient (method focusing on the peak value of the member response) (unit: kN/με).

|                | Results obtained from the strain data of the main girder | Results obtained from the strain data of the RC slab |
|----------------|--------------------------------------------------------|-----------------------------------------------------|
|                | Area 1        | Area 2        | Area 3        | Area 4        |
| Downstream lane| 1.357         | 0.8545        | –             | –             |
| Upstream lane  | 1.413         | –             | 0.6242        | –             | 0.9563        |

Fig. 12 Example of the response waveforms after the low-pass filtering of the strains of the main girder and the RC slab.

Fig. 13 Relationship between the actual load and the estimated load.
lines, dashed lines, and dash-dot lines indicate errors of ±10%, ±20%, and ±60%, respectively, with respect to the actual load. The total weight of the truck calculated from the main girder strain was similar to the actual load, with a variation of less than 10%, which was considered as a precise estimation. Moreover, the weight at the two rear axles and the gross weight of the truck calculated from the slab strain incurred differences of approximately ±20%. Therefore, the BWIM approach using the peak value of the member response provided reasonable estimations for the running load and, thus, was employed for the long-term monitoring of the responses of the test bridge due to ordinary vehicles in the next sections. However, the results from the slab strain also demonstrated an overestimation of the weight at the front axle, with a variation of ~60% at the maximum. Besides the influence of the running position, one of the other possible reasons for the overestimation of the front axle weight from the slab strain was that, when the front axle ran near the measurement point, the rear wheels were immediately passing through the expansion joint, thereby exciting the entire bridge and leading to a significant dynamic influence on the obtained data.

7. Long-term monitoring of the responses of the test bridge

7.1 Overview
Traveling load monitoring was performed to evaluate the current rigidity of the RC slab and to understand the traffic load situation of the vehicles traveling on the test bridge. The measurement period was 11 days in total from 20 October to 1 November 2016, and the sampling period was 0.005 s. The total vehicle weight was estimated from the correlation between the peak value of the strain waveform after the filtering of the main girder and the vehicle weight, using the method of Fukada et al. (2011). The axle weight and axial balance were estimated from the peak value of the slab strain by the same method as the total vehicle weight. In addition, not all vehicles traveling on the target bridge were recorded, as this study only considered vehicles with a total weight of 40 kN or more, as estimated from the strain waveform of the main girders. Therefore, to be able to reliably measure a vehicle having a total vehicle weight of 40 kN or more, the peaks value of the main girder strain of 29.0 με or more were analyzed. Regarding the relationship between running load and slab displacement, the measured results are described together with the analysis results in the next section.

7.2 Monitoring results
During the monitoring period, there were 2419 vehicles with a total weight of 40 kN or more passing through the target bridge. Specifically, the number of vehicles going on the downstream lane was 1121, whereas that going on the upstream lane was 1298. The daily traffic volume was estimated to be ~220 vehicles, which was reasonably in agreement with the traffic volume per day of large cars (259 vehicles) according to the FY2015 Traffic Census (MLIT 2016).

Figure 14 shows the frequency distribution of the distance between axles. The distance between the axles was calculated by multiplying the passing time of each axle obtained from the strain gauges or the pi-shaped gauges installed at the bottom of the RC slab by the traveling speed estimated from the strain gauges installed on the vertical stiffeners at both ends of the main girders. From Figure 14, because the most common distance between the axles was obtained as ~1.4 m (1.35–1.45 m) for both target lanes, it was clear that many large vehicles having a tandem axis passed through the bridge during the monitoring period. In addition, there were also a significant number of two-axle vehicles running across the bridge because of the prominent frequencies in the vicinity of the axle distance of 5 m.

Figure 15 shows the relationship between total vehicle weight and total wheelbase, which was the distance between the front-most axle and the rear-most axle of the vehicle. The plotted total vehicle weights were estimated from the strain data of the main girders. It was inferred that the influence of large vehicles in the upstream lane was higher than the observed results in the downstream lane. In addition, only one vehicle, whose gross weight (~480.3 kN) exceeded the permitted weight limit (431.2 kN) for eight special models, referring to the semitrailer connected vehicles and full-trailer connected vehicles, was recorded in the upstream lane. If a vehicle falls under the eight special models, permissions from the road administrator and the local government are required.

Because of the small traffic volume per day, fatigue damage was unlikely to be the main cause of the deteriorations of the target bridge. From the monitoring data of the vehicle weight, the possibility that overloaded vehicles were causing serious damage to the slab was considered low. The results also indicated that the frequency of vehicles with tandem axes across the bridge was more prominent than that of other vehicles. As can be seen from the displacement and strain responses of

![Figure 14 Frequency distribution of the distance between axles.](image-url)
the RC slab in Figs. 11 and 12, it was believed that the tandem axle of the test truck exerted a considerable influence on the responses of the slab.

8. Parametric study by numerical analysis

8.1 Analysis model

To analytically identify the stiffness of the RC slab of the bridge, this study performed numerical analyses using DIANA 10.2, which is a commercially available program for nonlinear finite element analyses. Figure 16 shows the analysis model. To reproduce the loading state corresponding to the actual traveling lane of the vehicle, the analytical model encompassed the steel main girders, the steel bracing system (sway braces, lateral braces), the pavement, and the RC slab, including the cross section where the slab displacement was measured. Regarding the element types used in the analyses, the steel main girders and the steel stiffeners were modeled as four-node shell elements, whereas beam elements were utilized for the models of the steel bracing system. Eight-node solid elements reproduced the concrete slab, the pavement, and the wheel guards. For the models of other reinforcing bars, embedded steel reinforcement elements were employed and symmetric conditions were applied as boundary conditions in the bridge axis direction. The bonding force between the concrete and the steel was assumed to be complete. The energy-controlled convergence norm and the modified Newton-Raphson method were selected as the iterative methods. The analysis was carried out by applying incremental load factors with specified sizes. A nonlinear structural analysis was employed for the analysis command, and four scenarios of the concrete properties were analyzed for the parametric study to obtain the load-displacement curves, which were then expected to be the thresholds for the stiffness evaluation of the RC slabs, as compared with the experimental displacement results. In particular, Scenario 1 was the design values of the compressive strength and the static elastic modulus, i.e., 35 N/mm² and 29.5 kN/mm², respectively. Regarding Scenario 2, the static elastic modulus was estimated as 13.3 kN/mm² from the elastic coefficient ratio \( n = 15 \) when ignoring the tensile side, while the compressive strength was set to 24.4 N/mm² from the proportional distribution of the compressive strengths of concrete samples obtained from the leakage part and the dry part. Scenarios 3 and 4 utilized the results from the concrete core tests, which provided static elastic moduli of 21.5 kN/mm² and 10.0 kN/mm² for the dry part and the water-leakage part, respectively. Regarding the pavement, the asphalt concrete was assumed as a linear elastic material model with a static elastic modulus \( E_c \) of 700 N/mm² and a Poisson ratio \( \nu \) of 0.35.

8.2 Material properties of concrete

The rotating total strain crack model for concrete (DIANA FEA (Manie 2016)) was employed herein, which includes the JSCE tension softening model for the tensile behavior (JSCE 2012) and the Thorenfeldt model for the compressive behavior (Thorenfeldt et al. 1987). Specifically, the Thorenfeldt compression curve was utilized to reproduce the concrete’s compression stress-strain relationship. This curve is described by a single equation but with different parameters for the ascending and descending branches:
\[
\epsilon_c = \frac{n f'_{ck}}{n - 1 E_c}; \quad n = 0.8 + \frac{f'_{ck}}{17}; \quad k = \begin{cases} 1 & (0 < \epsilon_c < \epsilon_p) \\ 0.67 + \frac{f'_{ck}}{62} & (\epsilon_p \leq \epsilon_c) \end{cases}
\]

where \( \epsilon_c \), \( f'_{ck} \), \( E_c \), and \( \epsilon_p \) are related to the stress value in the compression (N/mm\(^2\)), compressive strength (N/mm\(^2\)), static elastic modulus (N/mm\(^2\)), and strain of concrete, respectively; \( \epsilon_p \) denotes the compressive strain corresponding to the maximum compressive stress; and \( n \) and \( k \) are the parameters of the Thorenfeldt curve. The parameters for the Thorenfeldt model and compression curves are presented in Table 5 and Fig. 17, respectively. Figure 17 shows that the compressive strength and the static elastic modulus showed decreased amplitude, which was associated with the increased degradation and the increase in the compressive stress corresponding to the maximum compressive stress. A previous study of Aoyama et al. (2016) found that the compressive strength and the static elastic modulus of concrete decreased as ASR degradation progressed and that the maximum strain at the time of compression failure increased. Therefore, the Thorenfeldt model appears to be suitable for reproducing the compressive behavior of concrete deteriorated by ASR.

In addition, the critical parameters governing the JSCE tension softening model for the tensile behavior were the fracture energy and the tensile strength of concrete. The fracture energy of concrete is the energy consumed to form cracks per unit area and is calculated according to Eq. (7) (JSCE 2012):

\[
G_f = 10(d_{max})^{1/3} f'_{ck}^{1/3}
\]

where \( G_f \), \( d_{max} \), and \( f'_{ck} \) are related to the fracture energy (N/mm), the maximum aggregate size (mm), and the compressive strength of concrete (N/mm\(^2\)), respectively. In the present study, the value of \( d_{max} \) was assumed to be equal to 25 mm. Additionally, owing to the absence of experimental data, the estimation of the tensile strength of concrete \( f_{tk} \) can be achieved using Eq. (8), based on the characteristic compressive strength \( f'_{ck} \) (JSCE 2012):

\[
f_{tk} = 0.23 f'_{ck}^{2/3}
\]

Figure 18 and Table 6 show the tension softening curves and the material parameters utilized for reproducing the tensile stress-strain relationship of concrete.

### 8.3 Steel materials

Von Mises plasticity with no hardening behavior was
employed to simulate the behaviors of the reinforcing bars, main steel girders, bracing system, and steel stiffeners. Specifically, the reinforcing bars (SD295) were defined by a property with a static elastic modulus $E_s$ of 200,000 N/mm$^2$ and a Poisson ratio $\nu$ of 0.3. The yield stress was defined as 295 N/mm$^2$. Regarding the other steel properties (SM490), a value of 325 N/mm$^2$ was utilized for the yield stress. The tensile stress-strain relationship was assumed to be bilinear, as shown in Fig. 19. Moreover, Table 7 shows the material parameters utilized for reproducing the behaviors of the steel properties.

8.4 Comparison between experimental and numerical results

To evaluate the entire rigidity of the RC slab of the target bridge, the numerical results of the analysis were compared with the slab displacement obtained by the running test using the test truck with a known load. Regarding the experimental results of the running test, Figs. 20(a) and (b) show the relationship between the load and the slab displacement of the test and the simulation at the quarter span location of the downstream lane and at the 5/8 span location of the upstream lane, respectively. As aforementioned in the previous section, the load of two rear wheel axles exerted a large influence on the slab displacement because the maximum displacement was recorded when eight rearmost wheels passed over the observation point, as shown in Fig. 11(b). Therefore, the maximum displacement of the RC slab caused by the two rear wheel axles was employed as the comparison value with the numerical results to evaluate the current soundness of the structure. From Table 2, the total load of the rearmost wheels can be obtained as 169 kN in regard to the case with loading. Noise occurred in the slab displacement waveform when the truck traveled at each traveling speed, thereby leading to the variation in the evaluation. Therefore, the obtained data were filtered by moving average (low-pass filter equivalent to 3.5 Hz) and evaluated with the maximum value of the quasi-static component. The results showed that, although there were slight variations in the measurement results at either measurement location, the influence of the running speed of the test vehicle was small. The slight influence of the vehicle speed on the maximum displacement at the bridge midspan was also reported numerically in the study of An et al. (2016), which evaluated five vehicle running speeds for the impact analysis, i.e., 100, 110, 120, 130, and 140 km/h. In comparison with the simulation results, at the quarter location of the span of the downstream lane (location D2), the experimental results were mostly distributed within the result range obtained by the simulation scenarios using the elastic moduli of the collected cores. Moreover, at the 5/8 span location of the upstream lane (location D3), the measured displacement was large and close to the numerical result obtained with the static elastic coefficient (10.0 kN/mm$^2$) of the concrete sample of the water-leakage part.

| Table 7 Material parameters for the steel materials. |
|----------------------------------|----------|----------|
| Static elastic modulus (N/mm$^2$) | 200,000  | 200,000  |
| Poisson ratio $\nu$              | 0.3      | 0.3      |
| Yield stress (N/mm$^2$)          | 295      | 325      |

Fig. 19 Stress-strain curves of the steel materials.

Fig. 20 Relationship between load and slab displacement from the vehicle running test and the numerical analysis.
From the long-term monitoring responses of the test bridge due to ordinary vehicles, Figs. 21 (a) and (b) show the relationships between the largest axle weight estimated from the slab strain for each vehicle across the upstream and downstream lanes and the corresponding displacements of the RC slabs at the quarter and 5/8 span locations of the test bridge. Simulation results (elastic range) are also shown for comparison. In Figs. 21 (a) and (b), the experimental displacement results measured at the bottom of the RC slabs at locations D2 and D4 (see Fig. 8) when the vehicles across the bridge via the downstream lane were distributed in the vicinity of the analysis result of Scenario 2 ($E = 13.3 \text{ kN/mm}^2$). Besides, the vertical displacement of the RC slabs at locations D1 and D3 (see Fig. 8) when the vehicles traveling on the upstream lane were similar to the results of Scenario 4 using the core test result ($E = 10.0 \text{ kN/mm}^2$) of the water-leakage part. Moreover, almost the monitoring results were distributed in the lower range than the analysis results derived from the design properties of concrete. In addition, as the results of the visual investigation, efflorescence was observed at the bottom of the RC slabs and near the joint. Signs of water permeation also appeared under the RC slabs of both upstream and downstream lanes and near the span center. Therefore, it was revealed that the stiffness of the target slab of each lane of the bridge remarkably decreased as compared with the sound condition (Scenario 1). The monitoring results were then compared to the simulation results of the loading test until the final failure, as shown in Figs. 22 (a) and (b). The analysis was performed by applying incremental load factors with specified sizes until cracks occurred on the compressed side of the slab. As the load increased, punching shear failures arose from the formation of diagonal tension cracks around the loaded area, which resulted in a conical failure surface, as illustrated in Fig. 23. From the monitoring results, the maximum axle load was observed to be $\sim 208 \text{ kN}$, which was lower than the lowest crack initiation load ($P_{\text{crack}} = 338 \text{ kN}$) obtained from the simulations over four scenarios. Although the load-carrying capacity of the slab is not significantly deteriorated at the present stage, it is necessary to establish a maintenance management for these specimens to avoid further damage.

Although the result of the core extraction was the evaluation at a representative point, the comparison between the results obtained from the running test and the
numerical analyses revealed that there existed a correlation between the rigidity of the RC slab and the test result of the sampled core. Therefore, the proposed approach, which employed the slab displacements from the vehicle running test and the long-term monitoring and the results of the simulation using the properties of concrete cores as input data, could provide a reasonable evaluation on the soundness of the RC slab. Non-contact and high-precision equipment has been developed in recent years for measuring the floor slide displacement. Using this modern technology, stiffness evaluation using the displacement of the slab can be an effective evaluation approach in the maintenance management of bridges.

9. Conclusions

This study employed the survival time analysis using inspection data for bridges in the Hokuriku district to identify the tendency of early degradation of the RC slabs of the road bridges over a specific route in the mountainous areas where abnormal appearances were noticed. In order to investigate the degradation causes and the current load carrying capacity of these bridges, numerous analyses including cylindrical concrete cores, vehicle running test, long-term monitoring, and simulation were performed on the RC slabs of a road bridge, which deteriorated early in a mountainous area of the Hokuriku district. The primary results obtained from this study are summarized as follows.

1. Survival time analysis using inspection data was carried out for steel road bridge groups with RC slabs in a mountainous area. It was revealed that the RC slabs of the applicable group deteriorated earlier than those of other bridges.

2. Polarizing microscopic observation of concrete cores collected from the RC slabs confirmed that cracks developed from the andesite particles to the cement paste and that those cracks were confirmed to be filled with ASR gels. Therefore, it was confirmed that the primary cause of the deterioration in the concrete of the RC slab was ASR. In addition, in the result of the investigation on the distribution of chloride ion concentration, corrosion of the steel bar was not confirmed in the water-leakage part and the corrosion occurrence limit of the steel material was not exceeded.

3. The result of the investigation on the mechanical properties of the cores showed that the compressive strength decreased by approximately 30% in the water-leakage part as compared to the dry part. Regarding the static elastic modulus, the results obtained from the water-leakage part showed that it decreased to about half of that obtained from the dry part.

4. From the long-term monitoring of the ordinary vehicles, the actual condition of overloaded vehicles across the bridge was confirmed for each lane. Because of the small traffic volume per day, fatigue damage was unlikely to be the main cause of the deteriorations of the target bridge.

5. Although the result of the core extraction was the evaluation at a representative point, it was possible to evaluate the current stiffness of the target slab by comparing the results obtained from the vehicle running test and the long-term monitoring with those of the numerical analyses. Because the proposed approach, which employed the slab displacements from the vehicle running test, the long-term monitoring of the ordinary vehicles, and the simulation using the properties of concrete cores as input data, could provide a reasonable evaluation on the soundness of RC slabs, and, thus, further research is needed on the subject.

6. Regarding the long-term prospects, with the use of modern technologies such as a non-contact and high-precision measuring equipment, stiffness evaluation using the displacement of the RC slab can be an effective evaluation approach in the maintenance and management of bridges.

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