

QUARTERLY REPORT of

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CONTENTS

RTRI

PAPERS

Method for Predicting Rail Creep and Track Buckling Using Wide-Area 2-D FEM of Railway Tracks Based on GIS Data **I N T**

Evaluation Method for the Risk of Scouring Based on Case Studies of Railway **I N**

Development of Grouted Ballasted Track for Existing Shinkansen Asphalt Roadbed **I**

Method for Predicting Fatigue Strength of Corroded Rails Using Stress and Stress Gradient around Corrosion Defects **I**

Study of Infrared Coating Thickness Measurement Method for Repainting Coated Steel Bridges **I**

Indirect Bridge Deflection Estimation Method for Bridges under Resonant Condition Based on Onboard-Measured Track Geometries **I**

Evaluation Method for Flange-Climb Derailment Focusing on Contact Position and Transverse Creepage Between Wheel and Rail **O R T**

Method for Estimating Factors of AI Decision Errors in Train Forward Surveillance **O T**

Proposal for a Train Rescheduling Evaluation Metric and a Method for Analyzing and Evaluating Rescheduling Actions **O**

Impact Tests on Railway Vehicle Aluminum Alloy Members Considering Buckling and Shear Fractures **R**

RESEARCH REPORT

Comparison of Driver Gaze Allocation Between GOA2.5 Automatic Operation and Manual Operation in Revenue Service **H O**

SUMMARIES

Summaries of Papers in RTRI REPORT (in Japanese)

- H** Human factors
- I** Infrastructure
- N** Natural hazards
- O** Operations
- R** Rolling stock
- T** Technical system integration and interaction



CONTENTS

PAPERS

- 167 Method for Predicting Rail Creep and Track Buckling Using Wide-Area 2-D FEM of Railway Tracks Based on GIS Data **I****N****T**F. URAKAWA, T. WATANABE
- 177 Evaluation Method for the Risk of Scouring Based on Case Studies of Railway **I****N**S. WATANABE, S. FUJIWARA
- 183 Development of Grouted Ballasted Track for Existing Shinkansen Asphalt Roadbed **I**K. ITO, N. TAKAHASHI, T. TAKAHASHI
- 191 Method for Predicting Fatigue Strength of Corroded Rails Using Stress and Stress Gradient around Corrosion Defects **I**J. MIZUTANI, R. TAKAYAMA, T. DESHIMARU, Y. TERASHITA, M. HOSODA, R. YAMAMOTO
- 197 Study of Infrared Coating Thickness Measurement Method for Repainting Coated Steel Bridges **I**T. SAKAMOTO, S. YAMANAKA
- 203 Indirect Bridge Deflection Estimation Method for Bridges under Resonant Condition Based on Onboard-Measured Track Geometries **I**K. MATSUOKA, K. HATTORI
- 210 Evaluation Method for Flange-Climb Derailment Focusing on Contact Position and Transverse Creepage Between Wheel and Rail **O****R****T**S. KUNIYUKI, K. NAKANO
- 218 Method for Estimating Factors of AI Decision Errors in Train Forward Surveillance **O****T**N. NAGAMINE, W. GODA
- 224 Proposal for a Train Rescheduling Evaluation Metric and a Method for Analyzing and Evaluating Rescheduling Actions **O**M. TAKADA, T. KUNIMATSU, S. TANAKA
- 230 Impact Tests on Railway Vehicle Aluminum Alloy Members Considering Buckling and Shear Fractures **R**K. NAGATA, T. OKINO, J. TAKANO

RESEARCH REPORT

- 238 Comparison of Driver Gaze Allocation Between GOA2.5 Automatic Operation and Manual Operation in Revenue Service **H****O**C. NAKAGAWA, H. AKATSUKA, T. AOYAGI, A. FUJII

SUMMARIES

- 244 Summaries of Papers in RTRI REPORT (in Japanese)

H Human factors
I Infrastructure
N Natural hazards
O Operations
R Rolling stock
T Technical system integration and interaction

Method for Predicting Rail Creep and Track Buckling Using Wide-Area 2-D FEM of Railway Tracks Based on GIS Data

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This study developed a horizontal two-dimensional static elasto-plastic FEM analysis program for rail tracks. The validity of the developed program was confirmed by comparing it with an existing one-dimensional analysis model. The study also found that the program was able to analyze the behavior prior to track buckling and the buckling temperature with the same level of accuracy as the existing buckling analysis model. Furthermore, corner breakage in curves was a problem in the analysis using GIS railway line data. Therefore, a smoothing method was proposed in which corner breakage was replaced with a circular arc of a set radius, followed by a moving average, and confirmed its effectiveness.

Key words: rail creep, track buckling, GIS (Geographic Information System), wide-area structural analysis, smoothing of track alignment, shade/sunlight

1. Introduction

Many major railway tracks have continuous welded rails (CWRs), which are individual rails welded together to eliminate rail joints (here, “rail joint” refers to conventional joints where rail ends are connected by fishplates and fish bolts). CWRs offer several advantages, such as improved running safety, reduced noise and vibration, and enhanced ride comfort. However, because they cannot accommodate rail expansion and contraction at the joints, compressive axial forces develop in the rails when the rail temperature increases. Consequently, this increases the risk of track buckling, in which the track laterally deforms perpendicular to the rail direction, during high temperatures in summer, particularly on ballasted tracks. For CWRs, the compressive axial force P_R corresponding to the rail temperature T_R is calculated using Equation (1).

$$P_R = E_R A_R \beta_R (T_R - T_0) + \Delta P \quad (1)$$

Here, E_R , A_R , and β_R denote the Young's modulus, cross-sectional area, and linear thermal expansion coefficient of the rail, respectively. T_0 is the neutral temperature (i.e., the temperature at which the axial force becomes zero) of a newly welded rail. ΔP is the additional axial force caused by a change in the neutral temperature relative to the time of rail installation. This change arises due to rail creep (the longitudinal movement of the rail) or track maintenance work.

In Japan's management practice for CWRs, safety against track buckling is evaluated before the high-temperature summer season. This evaluation compares the maximum axial force obtained by substituting the expected maximum summer rail temperature and the amount of rail creep into T_R in Equation (1), with the buckling resistance calculated from the track stiffness and geometry. Furthermore, depending on the evaluation results, safety against track buckling is ensured through measures such as adding ballast, performing track maintenance, resetting the rail position (restoring rails that have shifted longitudinally), and conducting track patrols [1].

In general, CWR tracks are designed to ensure sufficient buckling resistance at the expected maximum rail temperature. Most locations that pose safety concerns in assessments exhibit large rail creep. However, since current measurements of rail creep are taken at intervals of approximately 200 meters, it is difficult to accurately identify concentrated local movements. Furthermore, the remedial actions described above require considerable labor. Therefore, it is desirable to implement appropriate countermeasures (such as in-

stalling anti-creepers [2]) at locations where significant rail creep is anticipated, thereby reducing the creep itself. In this context, the prediction of rail creep is important for improving safety against track buckling and reducing maintenance and management costs.

Previous studies have suggested several factors that contribute to rail creep, which can be broadly categorized into three groups:

A) loads induced by train passage and braking [3],

B) track repair and maintenance operations [3, 4, 5],

C) rail temperature (annual maximum and minimum temperatures [4, 5], temperature differences between shaded and sunlit areas, and the movement of shadows due to the diurnal motion of the sun [6, 7]).

The impacts of factors A and B on rail creep are not well understood, making quantitative prediction difficult. In contrast, factor C has been increasingly clarified through theoretical investigations by Abe et al. [6, 7].

Furthermore, the rail temperature prediction model developed by the authors [8, 9] can predict rail temperature distributions over a wide area of approximately 100 km using Geographic Information System (GIS) data and meteorological data. The model also accounts for shading effects from surrounding objects. By performing a structural analysis of the track using these predicted rail temperatures as input, it is possible to quantitatively predict rail creep induced by temperature variations.

A similar study was conducted previously [10]; however, the analysis was limited to a one-dimensional model along the longitudinal direction of the rail. Therefore, it was unable to reproduce various conditions that are expected to significantly influence track deformation, such as curvature, track irregularity, and temperature differences between the left and right rails caused by shading.

To reproduce these conditions, a two-dimensional horizontal analysis is required. Such an analysis not only enables a more accurate representation of rail creep, but also offers the advantage of reproducing track buckling within the computational framework. As discussed in Section 4, previous FEM-based buckling analyses were primarily intended to evaluate buckling strength indices. To achieve this, a nonlinear arc-length method, which is computationally demanding, was employed. Consequently, the target track section was limited to approximately 1 km, and the analysis was confined to restricted track irregularity conditions [11].

In this study, a two-dimensional horizontal static elasto-plastic FEM program (hereafter referred to as the “developed program”) is

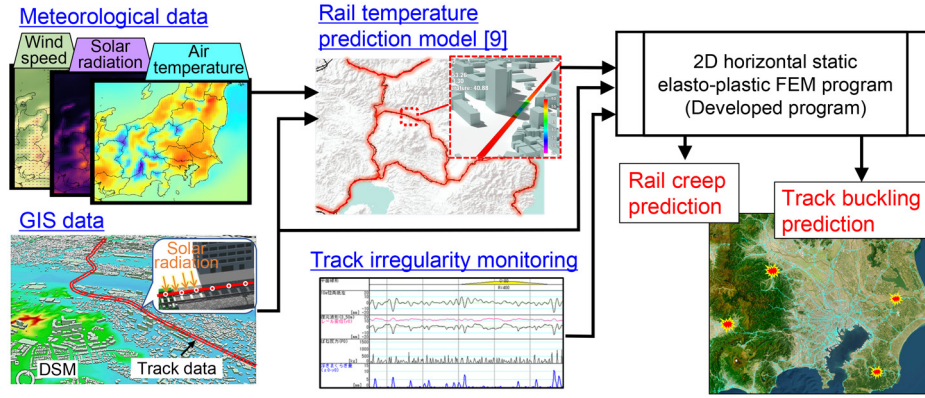


Fig. 1 Prediction method for rail creep and track buckling using two-dimensional horizontal static elasto-plastic FEM program for large-scale railway track analysis

implemented in Fortran to model rail creep induced by rail temperature with higher fidelity. The developed program simplifies nonlinear solution procedures and can compute large-scale rail creep and buckling phenomena over track lengths of approximately 100 km. Combining this program with an existing rail-temperature prediction model, we propose a simulation-based method for analytically predicting both rail creep and track buckling on ballasted tracks (Fig. 1). Furthermore, the validity of the developed program is examined through comparisons with existing analytical models.

2. Two-dimensional horizontal static elasto-plastic FEM program (the developed program)

2.1 Overview of the model

As shown in Fig. 2, the rails and sleepers are modeled using Euler-beam elements, the rail fastening systems are represented by translational and rotational spring elements, and the ballast is modeled by translational springs. In the developed program and throughout this paper, the longitudinal direction of the rail is defined as the local x -axis and the lateral direction as the local y -axis, where the positive y -direction corresponds to the left-hand side of the track when viewed along the positive x -direction. The global coordinate system (X, Y) follows the imported GIS data.

The spring elements of the rail fastening systems connect the rail and sleeper nodes, while the ballast springs connect each sleeper node to a fixed point. A positional index $i_x = 1, 2, \dots, n_x$ is assigned in the x -direction, and a rail element i_x consists of two rail nodes i_x and i_x+1 (Fig. 2). Sleeper nodes are independently defined and not shared with the rail nodes, but are located at the same spatial positions as the left and right rail nodes. A sleeper beam element i_x is defined as a beam element whose two end nodes are the left and right sleeper nodes associated with index i_x (Fig. 2).

As shown in Fig. 3, the loading conditions apply axial forces resulting from the thermal expansion of the rails, derived from the left and right rail temperatures $T_{\text{Rl}}(i_x, t)$ and $T_{\text{Rr}}(i_x, t)$ obtained from the thermal analysis, to the left and right rail elements i_x . The boundary conditions fully restrain the left- and right-rail nodes at both model ends ($i_x = 1, n_x$), as shown in Fig. 3.

In the developed program, rail nodes are placed at approximately 1-m intervals along the GIS track data. The program automatically reads the spatial coordinates of each node and the time-history rail-temperature data (from the rail-temperature predic-

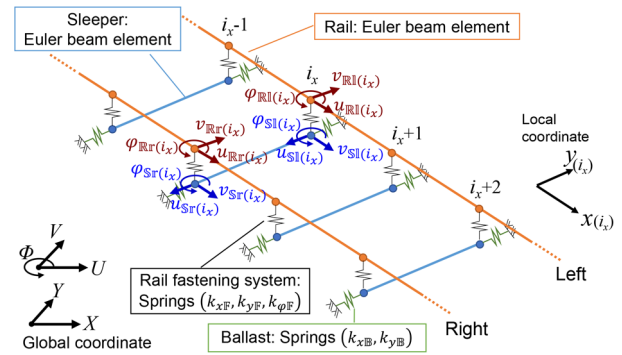


Fig. 2 Structure and coordinate system of the analytical model

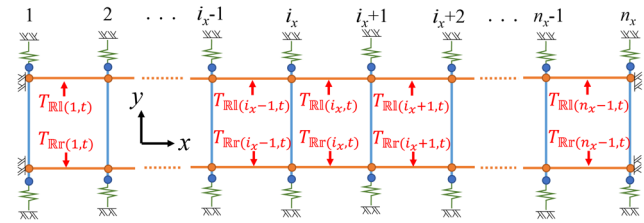


Fig. 3 Loading and boundary conditions of the analytical model

tion models [8, 9]) and automatically constructs the aforementioned elements, loading conditions, and boundary conditions. The spacing between sleepers is equal to the spacing between the rail nodes.

2.2 Linear and nonlinear treatment in the analysis

The linear and nonlinear modeling approaches are summarized in Table 1. To reproduce the sliding behavior of sleepers on ballast, the ballast spring elements are modeled as perfectly elasto-plastic with a yield load γ and a yield displacement d_p (Fig. 4). Although slip can also occur in the rail fastening system of actual tracks, the fastening springs are modeled as linear elastic. This follows the conventional design practice for CWRs on ballasted tracks, which assumes that rail-fastening resistance is much greater than ballast resistance, so rail creep predominantly occurs in the ballast layer [12].

To deal with large deformations such as track buckling and rail

lateral displacement in curved sections, the rotation of the rail beam elements is modeled as a geometric nonlinear element. Specifically, as illustrated in Fig. 5, the local coordinate system of each rail element is updated sequentially according to the rail deformation. All other components are modeled as linear.

In the developed program, as in general FEM procedures, the nodal displacement vector $\{D\}$ is treated as the unknown. The element stiffness equations derived from static equilibrium are assembled into a global stiffness equation that is solved for $\{D\}$. To solve the nonlinear problem, the nonlinear parameters, namely the plastic displacements of the sleepers and the local coordinate systems of the rail elements, are updated using the computed displacement vector $\{D\}$. The calculations and parameter updates are repeated until the difference between successive displacement vectors, $\{D\} - \{D'\}$, becomes smaller than the convergence tolerance D_c , which is set to 0.05 mm in this study. For details of the formulation and computational procedure, the reader is referred to Ref. [13].

Table 1 Linear and nonlinear modeling approaches for each element

Part	Element type	Nonlinearity		
		Material	Geometric	Boundary condition
Rail	Euler-beam element	Linear	Rotation of element	Linear
Sleeper	Euler-beam element	Linear	Linear	Linear
Rail fastening system	Springs (x, y, ϕ)	Linear	Linear	Linear
Ballast	Springs (x, y)	Perfectly elasto-plastic	Linear	Linear

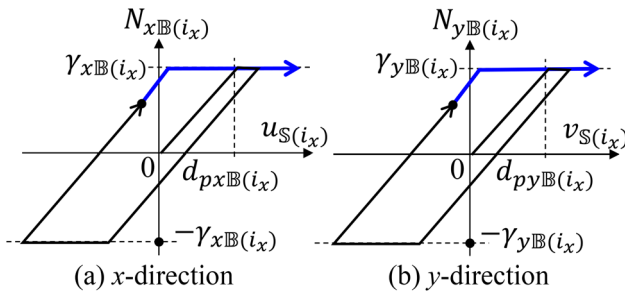


Fig. 4 Perfectly elasto-plastic spring characteristics of the ballast

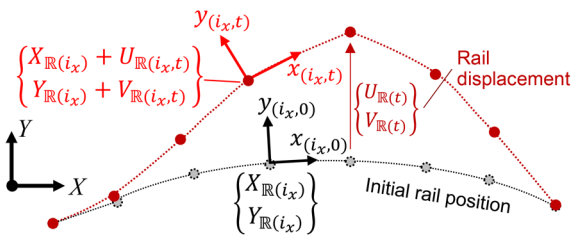


Fig. 5 Definition of the rail element local coordinate system based on nodal displacements

3. Verification of the rail creep analysis

3.1 Analysis conditions

The track length is set to 200 m, and the alignment is modeled as a straight line, a circular curve, and a curve extracted from GIS data, as shown in Fig. 6. The results for the straight line are compared with those obtained from the one-dimensional analytical program in [10] (hereafter referred to as the “existing program”).

For the circular curves, CWR design practice generally classifies curves with radii less than approximately 600 m as sharp curves. The lower design limit for the radius is commonly taken as 300 m due to track-stiffness considerations [12]. To examine the stability of solutions on sharp curves, two radii are analyzed: 600 m (“R600m” in Fig. 6) and a sharper radius of 200 m (“R200m” in Fig. 6), the latter of which is smaller than the usual minimum radius. For the curve “GIS data” in Fig. 6, a 200 m curved section is extracted from actual track GIS data.

The analytical parameters listed in Table 2 assume a typical ballasted track with CWRs. The same parameters are used in the existing program, except that the rail is modeled as a longitudinal spring rather than a beam. Therefore, the rail spring constant is taken as $(E_{R(i_x)} A_{R(i_x)})/L$ ($= 0.00775 \text{ m}^2 \times 206 \times 10^9 \text{ N/m}^2 / 1 \text{ m} = 1.60 \text{ MN/m}$), where L is the nodal spacing in the x -direction. The rail temperatures are taken from Fig. 7, which shows the results of the coupled solar-radiation and heat-conduction analysis in [8, 9], with shading effects considered. The neutral temperature of the rail is set

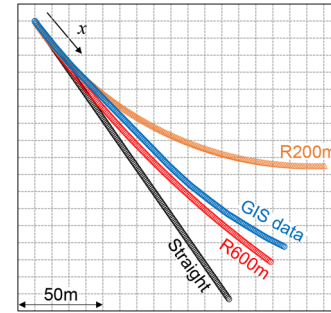


Fig. 6 Track-geometry data (left rail) used for the analysis

Table 2 Analysis parameters for rail creep analysis (per meter per rail)

Part	Symbol	Item	Value
Rail (JIS 60kg)	E_R	Young's modulus	$206 \times 10^9 \text{ N/m}^2$
	A_R	Cross-sectional area	0.00775 m^2
	I_R	Second moment of area	$5.12 \times 10^6 \text{ m}^4$
	β_R	Thermal expansion coefficient	$11.4 \times 10^{-6} / ^\circ\text{C}$
PC sleeper	E_S	Young's modulus	$88 \times 10^9 \text{ N/m}^2$
	A_S	Cross-sectional area	0.033 m^2
	I_S	Second moment of area	$3.4 \times 10^4 \text{ m}^4$
Rail fastening system	k_{xF}	Spring constant in the x -direction	$62.5 \times 10^6 \text{ N/m}$
	k_{yF}	Spring constant in the y -direction	$31.3 \times 10^6 \text{ N/m}$
	$k_{\phi F}$	Rotational spring constant	$4.33 \times 10^4 \text{ rad/m}$
Ballast	γ_{xB}	Yield load in the x -direction	8000 N
	γ_{yB}	Yield load in the y -direction	4000 N
	d_{pxB}	Yield displacement in the x -direction	0.002 m
	d_{pyB}	Yield displacement in the y -direction	0.002 m

to 30°C.

3.2 Analysis results

3.2.1 Straight line

Figures 8(a) and 8(b) show the analytical results of the rail axial force and the rail displacement in the x -direction, respectively, for the straight line. From Fig. 8(a), the axial forces obtained from the developed program (solid line) and the existing program (dashed line) are in good agreement.

Regarding the rail displacement, Fig. 8(b) shows that the values obtained from the two programs are almost identical, although a slight phase shift in the waveform can be observed along the x -direction. This shift is likely due to the difference in temperature applied to the rail elements when computing the thermal axial force. The existing program uses the average temperature of the two nodes forming rail element $i_x(T_{R(i_x)} + T_{R(i_x+1)})/2$, whereas the developed

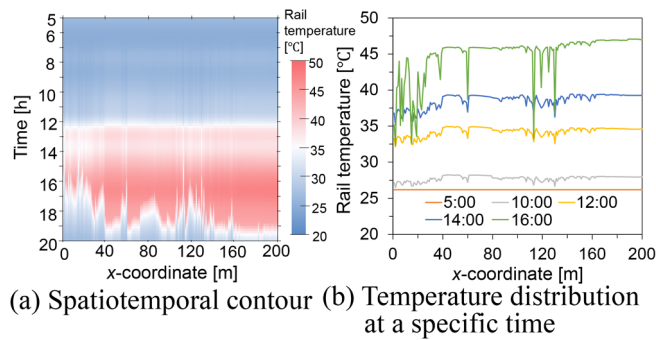


Fig. 7 Rail temperature used in analysis

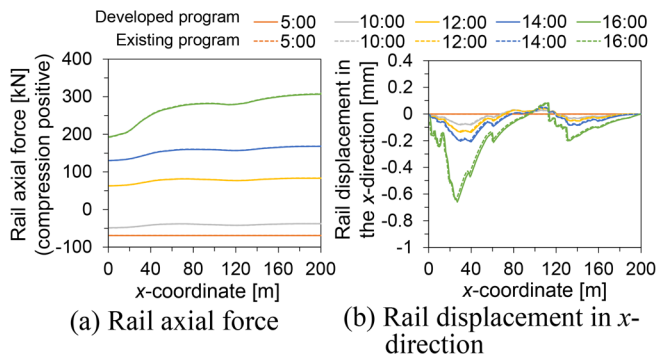


Fig. 8 Analysis results for the straight line and comparison with existing program

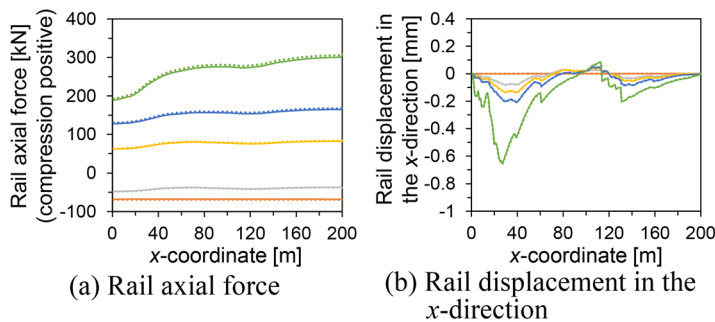


Fig. 9 Analysis results for circular curves

program uses only $T_{R(i_x)}$.

3.2.2 Circular curve

Figure 9 shows the analytical results of the axial force of the right (outer) rail and the rail displacements in the x - and y -directions for the circular curves. In Fig. 9(c), the y -direction rail displacement indicates that the entire curve bulges outward after 12:00, when the rail temperature becomes high. The displacement for the R200 m case becomes approximately three to four times larger than that for the R600 m case.

The rail axial force and the x -direction displacement show little difference between R600 m and R200 m, as seen in Fig. 9(a) and Fig. 9(b). Furthermore, Fig. 9(b) and Fig. 9(c) show that the rail displacements in both the x - and y -directions remain below 1 mm, which is smaller than the ballast yield displacement of 2 mm (Table 2). Therefore, no plastic deformation occurs in the ballast. Consequently, the rail axial force (Fig. 9(a)) and the x -direction displacement (Fig. 9(b)) are nearly identical to those in the straight-line case (Fig. 8(a) and Fig. 8(b)).

3.2.3 Curve (GIS data)

Figure 10 shows the curvature of the initial rail geometry obtained from the GIS track data. Several locations within the analysis section have curvature values exceeding 0.04 m^{-1} , which corresponds to a radius of curvature of less than 25 m. This occurs because the GIS track data represent the alignment as a series of straight-line segments, whereas an actual track consists of straight lines and circular arcs that are smoothly connected by transition curves. Consequently, angle breaks appear at the joints of these straight segments.

Figure 11 shows the analytical results using the GIS track data. As seen in Fig. 11(c), significant local displacements exceeding 10 mm in the y -direction occur at the angle-break locations. Com-

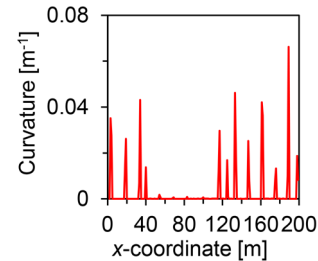
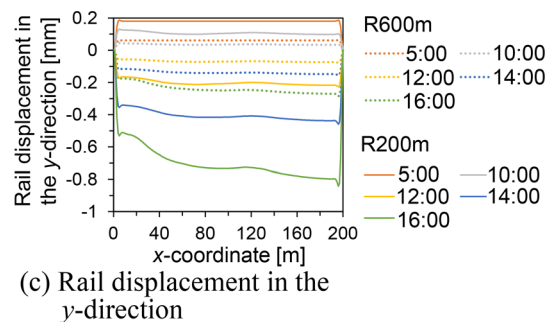


Fig. 10 Curvature of GIS track data



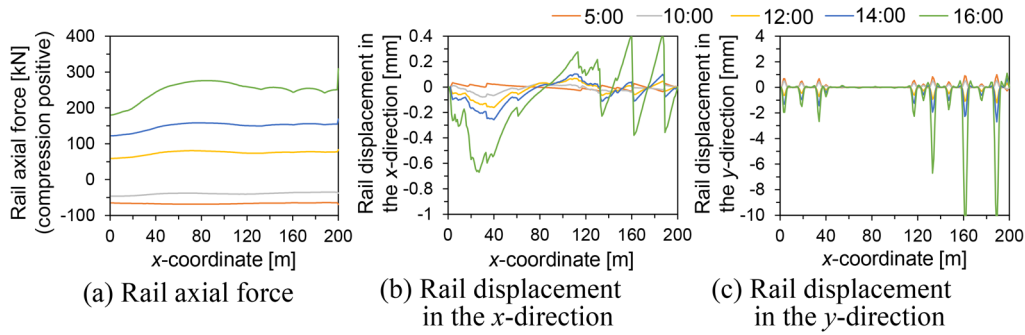


Fig. 11 Analysis results for curve (GIS data)

pared with the circular-curve cases, the rail axial force decreases (Fig. 11(a)) and the x-direction displacement increases (Fig. 11(b)). These large displacements occur at the angle-break locations and are caused by deficiencies in the input alignment data. Even under these exceptional conditions, the computation converges without difficulty, and no unrealistic behavior appears in the analytical results.

In summary, the developed program produced results that were consistent with those of the existing program for straight lines and circular curves. Stable solutions were obtained even for sharp R200 m curves and for cases with y-direction displacements exceeding 10 mm. These results confirm the validity of the developed program. However, the angle breaks contained in the GIS track data caused problems in the analysis. The developed program is designed to perform coupled analysis with solar-radiation calculations linked to geographic information (see Fig. 1), so the use of GIS data as input for the rail alignment is unavoidable. Therefore, it is necessary to adjust the GIS track data before the analysis. This can be done by smoothing the rail alignment or correcting it using the designed track geometry. A smoothing method for eliminating angle breaks in the GIS track data is examined in Section 5.

4. Verification of accuracy of buckling analysis

Figure 12 shows the relationship between the temperature rise from the neutral temperature and the rail displacement in the y-direction. In an actual track, as indicated by the red dashed line in Fig. 12, a small displacement develops with increasing rail temperature. Once the displacement reaches the limit point (A), a sudden large displacement occurs, and the equilibrium shifts instantaneously to point (B). The section to the left of point (A) and the section to the right of point (C) represent stable equilibrium states, corresponding to the pre-buckling and post-buckling conditions, respectively. The section between points (A) and (C) represents an unstable equilibrium state, and the track cannot sustain equilibrium within this range under actual conditions.

The temperature T_A at point (A) is the temperature at which track buckling actually occurs. T_A is referred to as the upper buckling temperature, and the corresponding axial force is referred to as the upper buckling resistance. The temperature T_C at point (C) is the theoretical temperature below which track buckling does not occur. The axial force corresponding to T_C is referred to as the lower buckling resistance [12]. For reference, the axial force corresponding to the temperature increase T' from the neutral temperature is defined as the axial force generated under a fully restrained condition, and is given by $E_R A_R \beta_R T'$. Here, E_R , A_R , and β_R are the Young's modulus, cross-sectional area, and coefficient of linear thermal expansion of the rail, respectively.

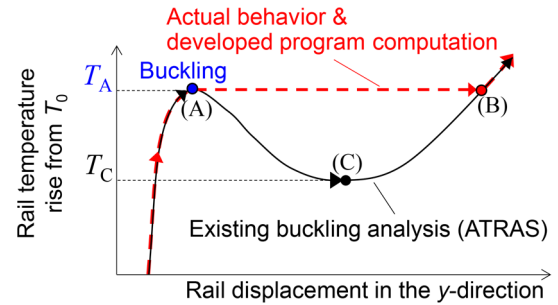


Fig. 12 Relationship between rail temperature rise from neutral temperature T_0 and y-direction rail displacement during track buckling

The temperatures T_A and T_C have been used as indices representing track-buckling strength. In existing buckling analysis models [11], an unstable equilibrium branch is solved using the arc-length method, which allows the black solid curve in Fig. 12 to be computed and thereby enables the determination of T_A and T_C . In contrast, the developed program reproduces the actual physical behavior indicated by the red dashed line in Fig. 12. When the rail temperature reaches T_A , track buckling occurs, and the y-direction displacement increases abruptly. This occurs because the developed program simulates real physical behavior by solving only the stable equilibrium condition with respect to rail temperature at each time step. Therefore, the developed program cannot compute T_C , but can compute T_A .

In this section, a static buckling analysis is performed using the developed program, and the upper buckling temperature T_A is determined from the relationship between the rail temperature rise and the y-direction displacement. The obtained T_A is compared with the corresponding result from the existing buckling-analysis model to verify the analytical accuracy. The existing buckling-analysis model used for comparison is the FEM-based track-stability analysis tool ATRAS [11, 13].

4.1 Analysis conditions

For track alignment, a circular curve with a radius of 600 m is used, as described in [14]. An alignment irregularity δ is superimposed toward the outside of the curve, with a wavelength l , an amplitude $\delta_0 = 20$ mm, and a center position x_0 , as shown in Fig. 13. The alignment irregularity δ in Fig. 13 is referred to as the first-mode irregularity, and is commonly used in existing buckling analyses under small-radius conditions (approximately less than 1000 m) [15].

To simulate the temperature reduction caused by shading ef-

fects, the longitudinal rail-temperature difference ΔT_x (Fig. 14(a)) and the temperature difference between the left and right rails ΔT_y (Fig. 14(b)) are used. The longitudinal temperature distribution is characterized by ΔT_x , the temperature-drop length W , and the distance λ from the center of the alignment irregularity to the midpoint of the W range, as shown in Fig. 15(a). The temperature differences ΔT_x and ΔT_y are introduced as follows: Once the rail temperature T_1 reaches the prescribed values of ΔT_x and ΔT_y , the other rail temperature T_2 increases at the same rate as T_1 (Fig. 15(b)). Because track

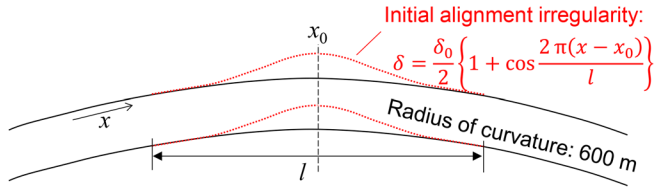


Fig. 13 Track alignment used for buckling analysis

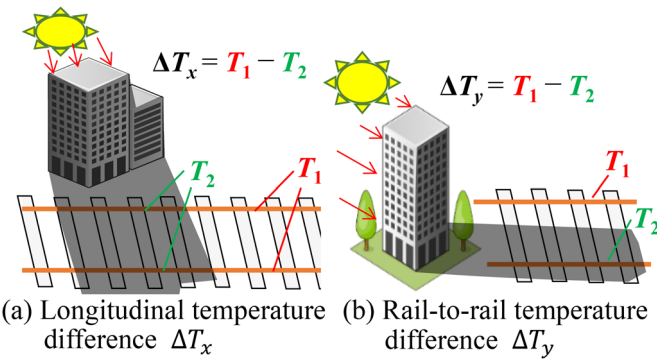


Fig. 14 Rail temperature differences caused by shading

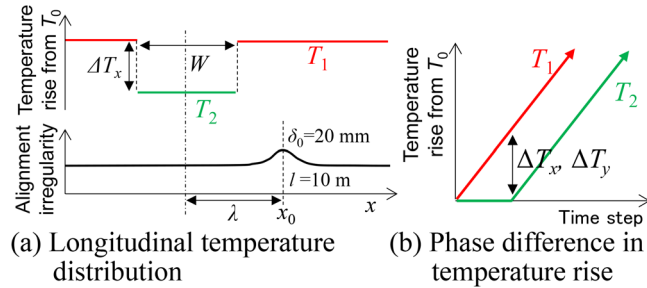


Fig. 15 Introducing rail temperature differences ΔT_x and ΔT_y into the analysis

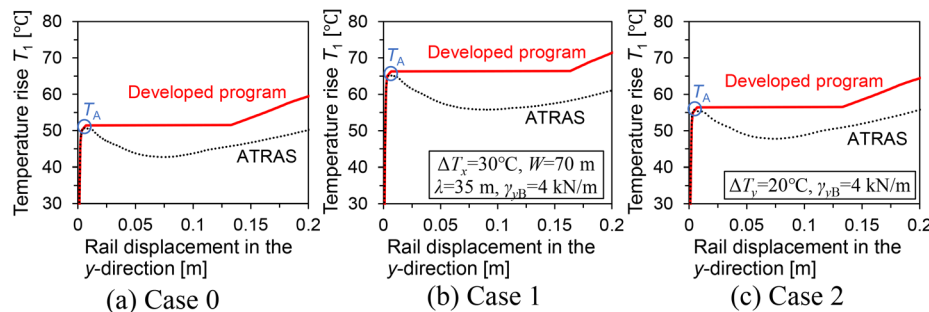


Fig. 16 Results of buckling analysis (rail temperature rise vs. y-direction rail displacement). Cases 1 and 2 show representative examples only.

buckling occurs as the alignment irregularity grows with increasing rail temperature, the center position x_0 of the alignment irregularity coincides with the center of the buckled location.

The analytical parameters are listed in Table 3, following the values used in previous buckling analyses [13]. The analysis cases are summarized in Table 4. Case 0 represents the standard condition without temperature differences. Case 1 consists of 29 cases in which ΔT_x , ΔT_y , W , λ , and the ballast lateral strength (ballast yield load γ_{yB}) are varied. Case 2 consists of nine cases in which ΔT_y and γ_{yB} are varied.

4.2 Analysis results

Figure 16 shows the relationships between the rail temperature rise and the y-direction rail displacement for Case 0, Case 1, and Case 2 (Cases 1 and 2 are shown for selected examples). As described previously, the developed program cannot trace the unstable

Table 3 Analysis parameters for rail creeping analysis (per meter per rail)

Part	Symbol	Item	Value
Rail (JIS 50 kgN)	E_R	Young's modulus	$206 \times 10^9 \text{ N/m}^2$
	A_R	Cross-sectional area	0.00642 m^2
	ν_R	Poisson's ratio	0.3
	β_R	Thermal expansion coefficient	$11.4 \times 10^{-6} / ^\circ\text{C}$
PC sleeper	E_S	Young's modulus	$88 \times 10^9 \text{ N/m}^2$
	A_S	Cross-sectional area	0.033 m^2
	ν_S	Poisson's ratio	0.3
	k_{xF}	Spring constant in the x-direction	$62.5 \times 10^6 \text{ N/m}$
Rail fastening system	k_{yF}	Spring constant in the y-direction	$31.3 \times 10^6 \text{ N/m}$
	$k_{\phi F}$	Rotational spring constant	$4.33 \times 10^4 \text{ rad/m}$
Ballast	γ_{xB}	Yield load in the x-direction	8000 N
	γ_{yB}	Yield load in the y-direction	4000 N
	$d_{\phi xB}$	Yield displacement in the x-direction	0.002 m
	$d_{\phi yB}$	Yield displacement in the y-direction	0.002 m

Table 4 Buckling analysis cases

	ΔT_x [°C]	ΔT_y^* [°C]	W [m]	λ [m]	γ_{yB} [kN/m]	Number of cases
Case0	0	0	0	0	4.0	1
Case1	15, 30	0	35, 70	0~85	2.0, 2.8, 4.0, 6.0	29
Case2	0	-20, 10, 20	0	0	2.0, 4.0, 6.0	9

※ A positive value indicates that the temperature on the outside of the curve is higher.

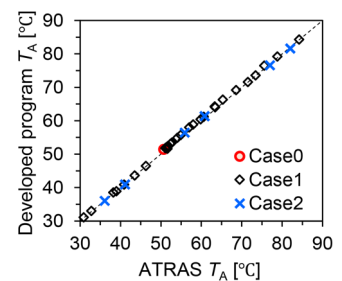


Fig. 17 Comparison of upper buckling temperature T_A

equilibrium state that occurs immediately after buckling due to reduced stiffness. Consequently, the y -direction displacement increases abruptly after buckling. However, the temperature at which track buckling occurs, that is, point (A) in Fig. 12, is nearly the same as that obtained by the existing model (ATRAS).

A comparison of the upper buckling temperature T_A obtained from the developed program and that obtained from ATRAS is presented in Fig. 17. The results show that the two values agree well. These findings confirm that the developed program can evaluate the upper buckling temperature T_A with an accuracy comparable to that of the existing buckling-analysis model.

5. Study on smoothing methods for GIS track data

5.1 Smoothing methods

5.1.1 Smoothing by moving average

The target nodal coordinates $\{X_{(i)}, Y_{(i)}\}^T$ are replaced by the average coordinates of the nodes within a prescribed range centered at the target node.

5.1.2 Smoothing by circular arcs

As shown in Fig. 18, circles with radii $R_{(i)}$ that are tangent to the GIS track data at the node positions are placed on both sides. At locations where a circle contacts the GIS track data at two points, the original data between these two points are replaced by the corresponding circular arc.

5.1.3 Smoothing by circular arcs followed by moving average

As will be discussed later, the circular-arc-based smoothing method defined in Section 5.1.2 can result in large curvature at the junctions between circular arcs (smoothed data) and straight segments (original data). Therefore, the data are first smoothed using this circular-arc-based method and then further smoothed using a moving average.

5.2 Evaluation of the smoothing methods

In this section, the smoothing methods described in Section 5.1 are evaluated by applying them to GIS track data. Approximately 12 km of GIS track data, shown in Fig. 19, are used, and the smoothing is performed under the seven conditions (i)–(vii) listed in Table 5. Figures 20 and 21 show a portion of the track geometry before and after smoothing and the corresponding curvature distributions, respectively. These analyses focused on the region around $x = 911$ m, where the curvature of the original GIS track data is particularly large.

As shown in Fig. 20, a maximum change of approximately 2 m is observed at the angle-break location of the curve. When smoothing is applied using a moving average, as in conditions (i) and (ii) in Table 5, the angle breaks in the original data are eliminated, and the maximum curvature is reduced to 0.013 m^{-1} for condition (i) (20 m moving average) and 0.007 m^{-1} for condition (ii) (40 m moving average).

When smoothing is performed using circular arcs corresponding to conditions (iii) and (iv) in Table 5, and focusing on the angle break at $x = 911$ m, the curvature in the vicinity of this location becomes equal to the reciprocal of the prescribed arc radius R in both

cases. However, localized increases in curvature are observed at $x = 927$ m for condition (iii) (200 m radius circular arc) and at $x = 938$ m for condition (iv) (300 m radius circular arc), as shown in Fig. 21. This occurs because the track data are discretized at 1 m intervals,

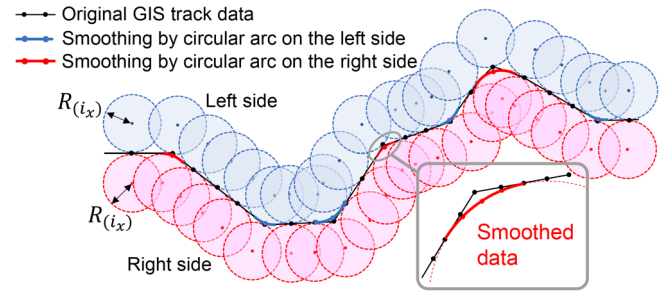


Fig. 18 Smoothing of GIS track data using circular arcs

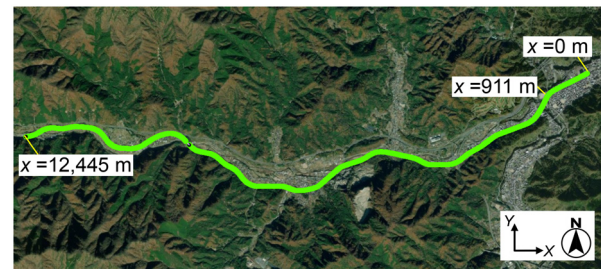


Fig. 19 GIS track data (original data)

Table 5 Smoothing conditions

Smoothing condition	Smoothing method	Averaging range	Arc radius
(i) 20 m moving average	Section 5.1.1	20 m	—
(ii) 40 m moving average	Section 5.1.1	40 m	—
(iii) 200 m radius circular arc	Section 5.1.2	—	200 m
(iv) 300 m radius circular arc	Section 5.1.2	—	300 m
(v) 200 m radius circular arc + 20 m moving average	Section 5.1.3	20 m	200 m
(vi) 300 m radius circular arc + 20 m moving average	Section 5.1.3	20 m	300 m
(vii) 300 m radius circular arc + 20 m moving average	Section 5.1.3	40 m	300 m

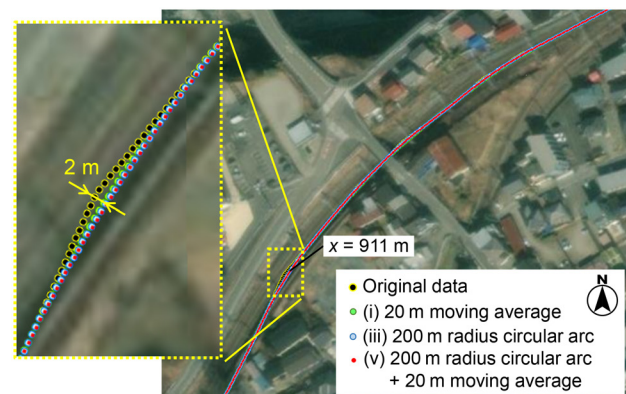


Fig. 20 Change in track geometry due to smoothing (around $x = 911$ m)

which results in deviations between the theoretical tangency positions of the circle and the nodal locations. These deviations introduce new angle breaks at the junctions between the circular arcs (smoothed data) and the straight segments (original data), leading to localized increases in curvature.

In the case of smoothing using circular arcs followed by a moving average, as in conditions (v)-(vii) in Table 5, the angle breaks at the junctions between the circular arcs and the straight segments are mitigated. As a result, the maximum curvature is reduced to 0.007 m^{-1} for condition (v) (200 m radius circular arc + 20 m moving average), 0.005 m^{-1} for condition (vi) (300 m radius circular arc + 20 m moving average), and 0.004 m^{-1} for condition (vii) (300 m radius circular arc + 40 m moving average), as shown in Fig. 21. In addition, in the vicinity of the angle-break location around $x = 911 \text{ m}$, the curvature becomes equal to the reciprocal of the arc radius (0.005 m^{-1} for a 200 m radius and 0.0033 m^{-1} for a 300 m radius) and gradually decreases as the distance from the angle-break location increases, connecting smoothly to the straight segments.

These results show that smoothing using either a moving average (conditions (i) and (ii)) or circular arcs followed by a moving average (conditions (v)-(vii)) effectively removes angle breaks in the GIS track data. However, when using a moving average alone, the resulting track alignment is considered to be strongly influenced by the resolution and accuracy of the original data. This makes it difficult to apply this method broadly to various GIS track datasets in the present context. In contrast, smoothing using circular arcs followed by a moving average not only removes angle breaks but also creates transition curves with gradually varying curvature between circular curves and straight lines. Therefore, the smoothing approach defined in Section 5.1.3, in which circular arcs are fol-

lowed by a moving average, is adopted for GIS track data smoothing in this study.

In the present analysis, the radius of the circular arc used for smoothing is set to be uniform over the entire section. It is expected that a more realistic track alignment can be reproduced by assigning an individual curve radius to each curve section. Figure 22 shows the results obtained by applying condition (v) in Table 5 to the “GIS data” shown in Fig. 6 and performing the same analysis as in Section 3.2.3. As shown in Fig. 22, smoothing eliminates the localized large displacements observed at angle-break locations in the unsmoothed data (Fig. 11(c)).

5.3 Analysis using smoothed GIS track data

The GIS track data shown in Fig. 19 are smoothed using a 200 m radius circular arc followed by a 20 m moving average. An analysis using the developed program is then performed by inputting the rail temperatures shown in Fig. 23. The temperature distribution shown in Fig. 23 is estimated from the GIS data for the track geometry and elevation shown in Fig. 19, under the assumption of typical rail temperatures on a clear summer day (September 1). Locations where the rail temperature locally decreases correspond to tunnel sections or areas shaded by surrounding mountains and other topographic features.

The analytical parameters consist of two conditions. The first is the standard parameters given in Table 2 for a typical continuous welded rail section on a ballasted track. The second is a condition susceptible to track buckling, in which the ballast yield load in the y -direction, γ_{yB} , given in Table 2 is reduced to one half (2 kN).

Figures 24(a) and 24(b) show the analytical results for the x -direction and y -direction displacements of the right rail at 14:00, respectively. Fig. 24(a) shows that when $\gamma_{yB} = 4 \text{ kN}$, the maximum rail creep (x -direction rail displacement) of approximately 5 mm occurs in areas where there is a steep longitudinal gradient of rail temperature due to shading. When $\gamma_{yB} = 2 \text{ kN}$, a large lateral displacement

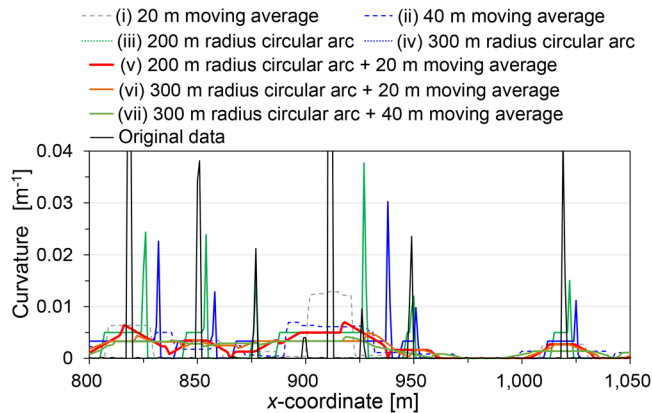


Fig. 21 Change in curvature distribution due to smoothing

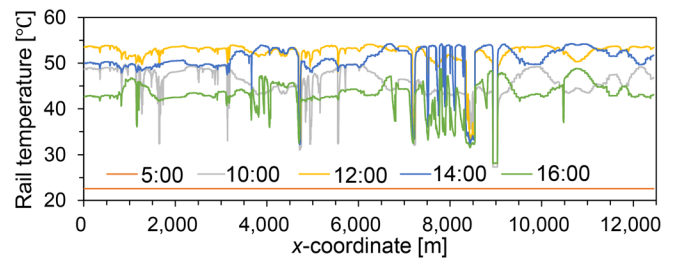


Fig. 23 Spatiotemporal distribution of rail temperature used in analysis

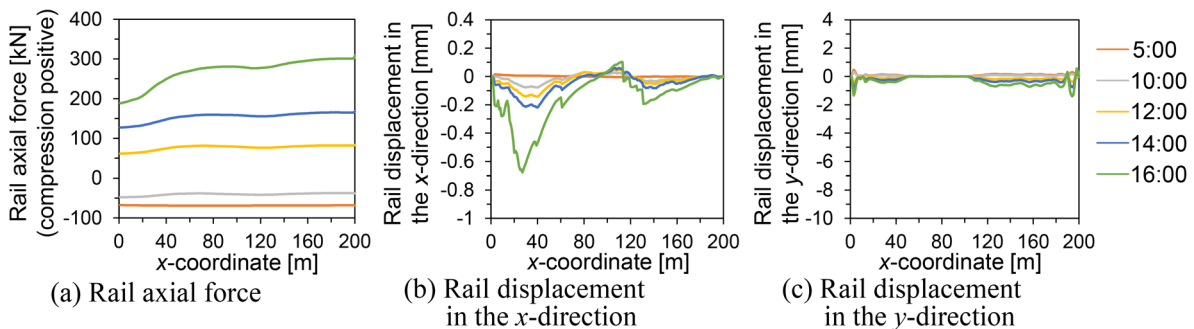


Fig. 22 Analytical results using smoothed track data based on “GIS data” shown in Fig. 6

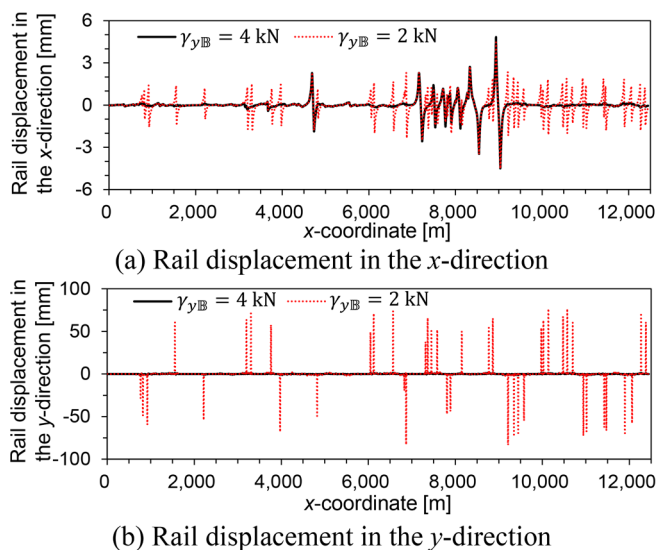


Fig. 24 Analytical results at 14:00 using track geometry smoothed by a 200 m radius circular arc followed by a 20 m moving average based on GIS track data shown in Fig. 19

(y -direction rail displacement) exceeding 50 mm occurs in the curved section (Fig. 24(b)), and a rail creep of approximately 2 mm is generated in the vicinity of this location (Fig. 24(a)).

Such rail creep induced by rail displacement in the direction perpendicular to the rail cannot be reproduced by the previously proposed one-dimensional analysis [10]. These results demonstrate the effectiveness of the two-dimensional static elasto-plastic FEM program developed in capturing this behavior.

In summary, the proposed method allows rail creep and track buckling to be analyzed over a wide area exceeding 10 km in length while accounting for conditions that were difficult to incorporate in conventional one-dimensional analyses. Furthermore, this analysis can take into account various conditions, including track curvature, lateral displacement, and temperature differences between the left and right rails, which are expected to influence rail displacement in the direction perpendicular to the track.

6. Conclusions

This study involved the development of a two-dimensional horizontal static elasto-plastic FEM analysis program, and the proposal of a method for analytically predicting rail creep and track buckling through coupled analysis with an existing rail temperature prediction model. The validity of the program was also verified through comparisons with existing analytical models. The main findings obtained in this study are summarized as follows.

- (1) The analytical results obtained using the developed program for straight tracks and circular curves were in good agreement with those from existing one-dimensional analyses. In addition, stable solutions were obtained even under severe conditions, including sharp curves with a radius of 200 m and cases in which the rail displacement in the y -direction exceeded 10 mm. These results confirm the validity of the program.
- (2) The developed program was shown to be capable of evaluating the upper buckling temperature T_{A^*} , which is one of the indices of buckling resistance, with an accuracy comparable to that of ex-

isting buckling analysis models.

- (3) When GIS track data are used as rail position information in the analysis, it was confirmed that angle breaks in curved sections required correction. To address this issue, a smoothing method was introduced in which the angle break sections in the GIS track data are replaced by circular arcs with a prescribed radius R , followed by the application of a moving average. This method was shown to provide highly accurate reproductions of actual track geometry, including the emulation of transition curves between circular curves and straight sections. Furthermore, this approach successfully solved the problem of localized deformation amplification at angle break locations caused by compressive rail axial forces.
- (4) Using the developed program, rail creep behavior induced by rail displacement in the direction perpendicular to the rail, which could not be reproduced by existing one-dimensional analyses, was successfully reproduced, particularly in curved sections.
- (5) These results confirm the accuracy and effectiveness of the developed program, demonstrating that the proposed method enables analytical prediction of rail creep and the occurrence of track buckling.

Future work will focus on modeling track displacement, seasonal variations in meteorological conditions, and the annual motion of the sun, as well as clarifying their effects on the progression of rail creep and the occurrence of track buckling.

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Evaluation Method for the Risk of Scouring Based on Case Studies of Railway

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In order to optimize maintenance and management of railway bridges and mitigate the risk to railway safety of scouring disasters, assessing risk of localized scouring at bridge piers is essential. This study proposes an indicator to represent risk of scouring by deriving dimensionless tractive forces acting on the riverbed from hydrological statistics during disasters. The proposed indicator is validated by comparing and organizing the results between disaster-affected and non-affected conditions. As a result, we found a consistent linear relationship between the normalized value and the embedment ratio when disasters occur.

Key words: scouring, bridge pier, hydrological statistics, dimensionless tractive force

1. Introduction

Localized scouring around railway bridge pier foundations (hereinafter referred to as scouring) during high water events can lead to settlement or tilting, potentially resulting in significant structural damage. To optimize the resources required for bridge maintenance and management, it is crucial to identify piers at high risk of damage due to scouring. Past efforts include statistical analysis-based scouring hazard assessments [1] as well as score-based identification methods [2] [3]. However, these approaches primarily focus on empirical evaluations of predisposing factors for scouring but fail to sufficiently consider the external forces causing scouring. Scouring is a complex phenomenon contingent on the interaction between the bearing capacity characteristics of the riverbed and the external force characteristics of the river flow. Therefore, a quantitative method for evaluating scouring risk is required which considers the influence of external forces, specifically, the flow rate during flood events.

2. Subjects of analysis

According to data held and provided by the Railway Technical Research Institute, since 1974 there have been 91 documented cases of flooding causing some form of damage to railway river bridges. Water level observation records are available for 11 of these cases. Cases from within this group were then selected based on the following criteria: (1) damage type limited to pier scouring and tilting; (2) availability of riverbed cross-section diagram near the bridge; (3) flow and water level data obtainable from hydrological observation points within approximately 10 km of the damaged bridge; and (4) availability of data on riverbed gradient and representative grain size of riverbed material around the damaged bridge. Table 1 shows the resulting eight bridges selected for analysis. The methods for determining each item in Table 1 are described later.

Since all damaged bridge piers were unprotected, the analysis of scouring risk indicators presented later includes both damaged piers and unprotected piers. Figure 1 shows the river cross-sections at the locations of the damaged bridges. Among the rivers associated to the eight bridges mentioned above, only Bridges C and H have flood-channels with low-water revetments. Furthermore, on the levee side of the damaged pier of Bridge G, a concrete walkway has been constructed as a promenade. In the other rivers, sandbars have formed. The damaged piers of Bridges A, B, D, E, F, and G are located on these alternate bars, while those of Bridges C and H are

situated within low-water channels.

3. Index representing risk of scouring occurrence at railway river bridge piers

3.1 Calculation of dimensionless tractive force

Scouring is a phenomenon caused by riverbed variations that involve erosion of sediment around bridge piers. Therefore, we focused on the dimensionless tractive force as an index for representing the risk of scouring occurrence. The dimensionless tractive force was calculated for the flow rate at the time of damage and compared with the values corresponding to the annual maximum flow rates.

First, the peak flow rates for both the disaster event and for each year are extracted from the water level and discharge data at the river flow observation cross-section. Next, the water level, flow rate (discharge), and average flow velocity at the damaged pier were determined by routing the extracted annual maximum flows through the bridge location. The dimensionless tractive forces are then calculated from these values. The flow rate is determined using a compound channel uniform flow calculation that neglects boundary mixing [5], with the dimensionless tractive forces calculated for each subdivided channel cross-section.

Under the above-mentioned assumptions, the average flow velocity for each cross-section (1, 2, i , ..., N) of the river channel was calculated using (1). Subsequently, the flow rate for each section and the total flow rate for the entire channel cross-section were determined using (2) and (3), respectively. Here, n (s/m^{1/3}) is the Manning's roughness coefficient of the riverbed, A_i (m²) is the cross-sectional area, L_i (m) is the wet perimeter of each cross section and I represents the riverbed gradient. The hydraulic radius is defined as the ratio A_i/L_i (m). It was assumed that roughness coefficient n and the riverbed gradient I are constant across the river's cross section, and that the water surface elevation remains uniform in the cross-sectional direction.

$$v_i = \frac{1}{n} \left(\frac{A_i}{L_i} \right)^{2/3} I^{1/2} \quad \dots (\text{m/s}) \quad (1)$$

$$q_i = A_i v_i \quad \dots (\text{m}^3/\text{s}) \quad (2)$$

$$Q = \sum_{i=1}^N q_i \quad \dots (\text{m}^3/\text{s}) \quad (3)$$

Table 1 summarizes the structural specifications of the eight bridges, along with the parameters used in the calculations. The data

in the table were obtained from supplementary materials of river improvement plans, various property maps, and disaster records. For the representative grain size of the riverbed d_R , d_{60} (m) was adopted. Since representative grain size data for Bridges B and F could not be obtained, their roughness coefficients were calculated using Manning-Strickler's formula, as expressed in (4).

$$n = \frac{d_R^{1/6}}{7.66\sqrt{g}} \quad (4)$$

The specific gravity of the riverbed soil particles, G_s , was set to 2.65 for all rivers. g (m/s²) denotes gravitational acceleration. For bridges A, B, F, G, and H, where flow records were unavailable, the flow rate at each observation point was estimated using water level data and river cross-section, based on the calculation method described above. The dimensionless tractive force τ^* was calculated for the damaged bridge location using this flow rate.

Figure 1 shows the approximate location of the normal channel and the water level when the calculated flow rate during the disaster event was flowing. The dimensionless scouring force was calculated using (5) based on the parameters shown in Table 1 and (1) to (3).

$$\tau^* = \frac{u_*^2}{(G_s - 1)gd_R} = \frac{gRI}{(G_s - 1)gd_R} \quad (5)$$

3.2 Normalization of dimensionless tractive force

Figure 2 compares the dimensionless tractive force at the time of the disaster with the annual maximum flow rate prior to the disaster year for each damaged bridge pier (hereafter referred to as the annual maximum dimensionless tractive force). Focusing on the position of the annual maximum dimensionless scouring force $\diamond$ at the time of the disaster, all damaged bridges exhibited values surpassing the upper limit of the distribution group, thereby, exceeding all previously recorded maximums. However, for Bridge G, there are cases where damage was avoided even when the value at the time of damage was exceeded.

The dashed lines in the figure indicate reference values for the dimensionless critical tractive force τ_c^* corresponding to each representative grain size of riverbed material. Here, the dimensionless critical tractive force was determined using Iwagaki's equation. The dimensionless critical tractive force is a theoretical value represent-

Table 1 Overview of cases and parameters used in analysis [4]

Bridge	A	B	C	D	E	F	G	H
Date of disaster	2003/8	2004/9	2006/7	2012/7	2015/9	2016/8	2017/10	2017/10
Damaged pier	19 th : Slope 20 th : Exposed foundation	1 st	3 rd	2 nd	10 th	3 rd	5 th	3, 4 th : Exposed foundation
Overview of disaster	The slope of pier on sandbars where water does not flow during normal water levels		The slope of bridge piers on low-water channel		The slope of pier on sandbars where water does not flow during normal water levels		Exposed foundation of pier on low-water channel	
Type of foundation	Shallow	Shallow: 1-5P Pile: 6-10P	Shallow: 1-5P Pile: 6P	Shallow	deep (caisson type)	Shallow	Pile: 1P Shallow: 2-5P	Shallow and pile
Number of unprotected piers / Total number of piers	21/21	7/10	6/6	2/6	15/21	4/4	3/6	8/9
River segment	2-2~3	2-2	2-2	1	1	1	2-1	2-2
Riverbed gradient I	1/2596	1/1200	1/2880	1/200	1/196	1/152	1/260	1/1210
Representative grain size of riverbed d_R (m)	0.0105	0.7: Estimated value	0.01	0.05	0.063	0.3: Estimated value	0.035	0.004
Roughness coefficient of riverbed n	0.0195	0.04	0.03	0.025	0.0263	0.035	0.024	0.027
Observed data type and location	Water level: 17 km upstream	Water level: 1.5 km upstream	Flow rate: 2.5 km downstream	Flow rate: 1.5 km downstream	Flow rate: 8 km upstream	Water level: 7 km upstream	Water level: 0.5 km upstream	Water level: 4 km downstream
Data period	2002-2019	1992-2020	2001-2017	2001-2017	2001-2017	2008-2019	2001-2017	2006-2019
Flow rate during disaster Q (m ³ /s)	1824.6	1334.3	643.8	387.7	2194.6	291.3	351.9	2385.2

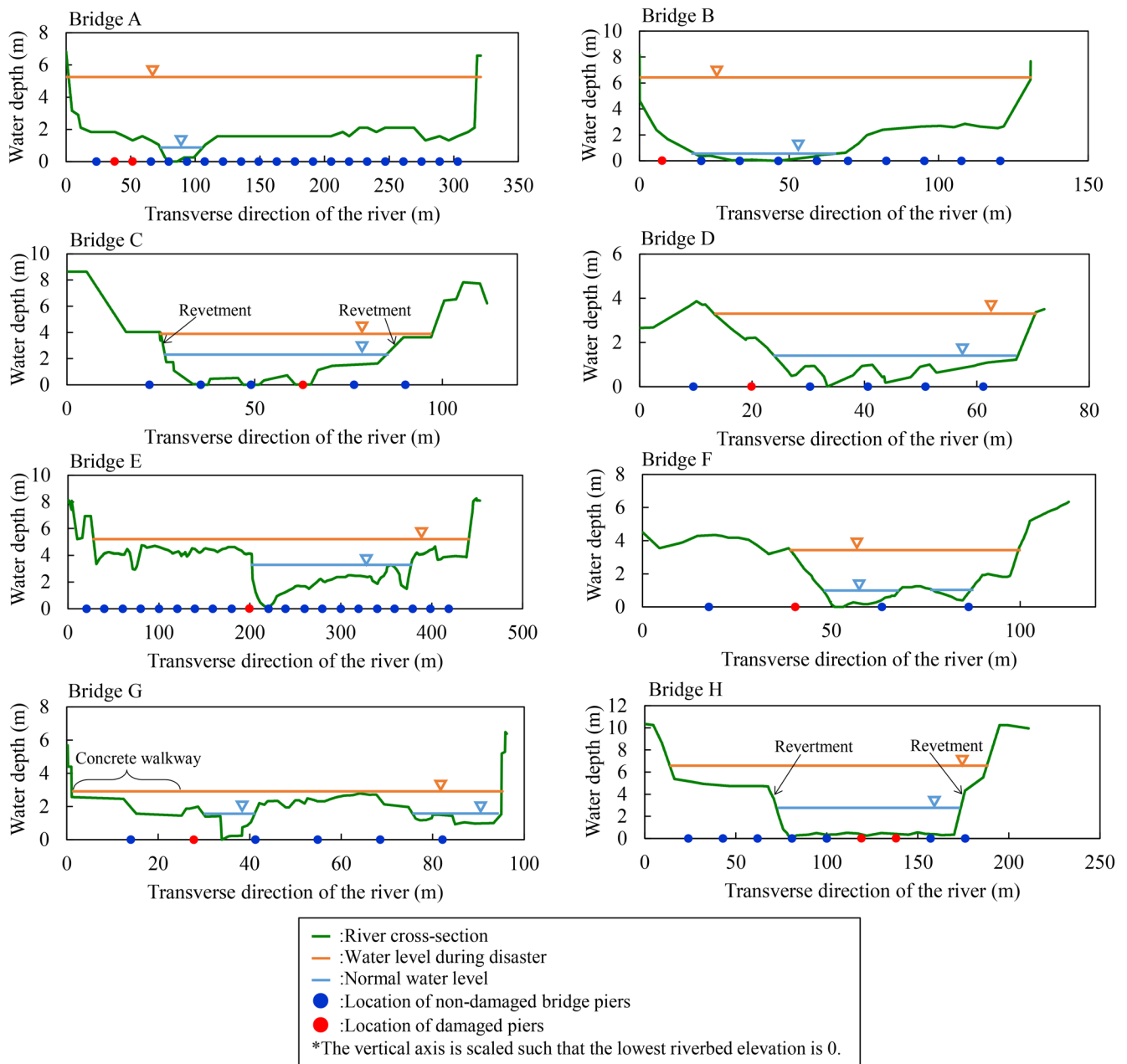


Fig. 1 C cross-sectional view of river at location of damaged bridge [4]

ing the threshold at which riverbed material begins to move. It is derived from the equilibrium between the scouring force acting on soil particles and the resistance value of the soil particles. Here, $\tau_c^* = 0.050$ for all cases.

Although direct comparison is difficult due to the assumptions involved, for Bridges C, E, G, and H, in numerous years, values at the reference sites were significantly exceeded, not only during the disaster but also prior to it. Therefore, evaluating the relative differences in scouring resistance between bridges and piers solely by comparing the annual maximum dimensionless tractive forces is inherently limited.

Yamamoto [6] states that the mean annual maximum flow is the appropriate flow rate for defining channel-scale and planform characteristics, such as the formation of natural levees and floodplain deposition. Therefore, this paper calculates the average value of the annual maximum dimensionless tractive force obtained for each

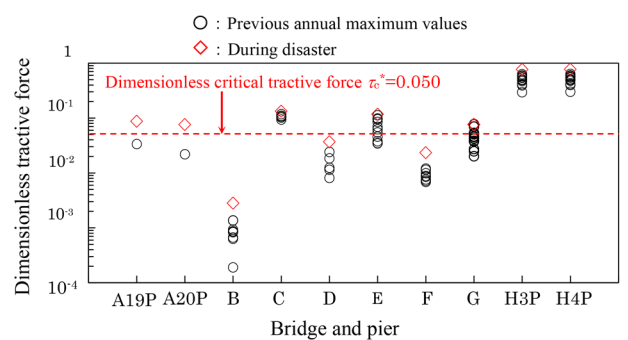


Fig. 2 Dimensionless tractive force on each bridge (Up to the year of the disaster) [4]

year. A relative hazard assessment is performed by comparing the extent to which this value was exceeded when scouring damage occurred among the bridges. From this perspective, the dimensionless tractive force at the time of damage (as shown in Fig. 2) was defined as the upper limit of the scouring resistance capacity of the pier, while its maximum historical value was taken as the lower limit. We then calculated the normalized value by dividing both limits by the average annual maximum dimensionless tractive force. The definition of the normalized dimensionless tractive force is given by Equation (6), and Fig. 3 illustrates the calculation concept.

$$\text{Normalized value} = \frac{\text{Value at the time of the disaster or previous maximum value}}{\text{Average annual maximum dimensionless tractive force}} \quad (6)$$

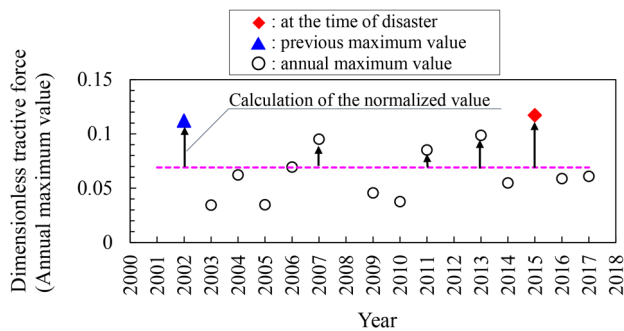


Fig. 3 Conceptual illustration for normalized value calculation [4]

The “average annual maximum dimensionless tractive force,” which is used as the denominator for normalization, is the mean of the annual maximum dimensionless values recorded over the entire observation period, including the year of the disaster. The entire observation period was used to minimize variations in the normalized values, since a short period from the start of observations to the disaster would provide insufficient data points for calculating the reliable average. For the same reason, we excluded piers located outside the low-water channels from the average calculations, as their annual maximum dimensionless tractive force may be zero in some observation years.

Figure 4 illustrates the relationship between the average annual maximum dimensionless tractive force for each bridge and the annual maximum value recorded prior to the disaster. The figure shows the values during disaster and the previous maximum values separately. The dashed line represents the 1:1 ratio; the $\diamond$ symbols denote the values at the time of the disaster; and the $\triangle$ symbols represent the previous maximum values. The figure demonstrates a strong positive correlation between the average annual maximum dimensionless tractive force and both the values at the time of damage, and the historical maximum values. It can be seen that, with the exception of a few historical maximum values, these values exceed the average annual maximum dimensionless scouring force. In other words, this suggests that the average annual maximum dimensionless tractive force can serve as a reliable base line for evaluating the relative magnitude of forces that potentially lead to damage.

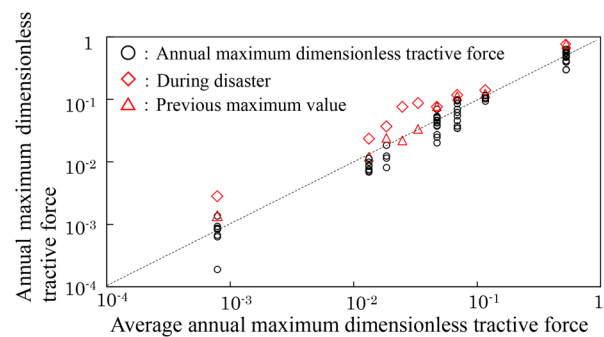


Fig. 4 Relationship between average annual maximum dimensionless tractive force and annual maximum dimensionless tractive force [4]

4. Reference measures for normalized value indicating the risk of bridge scouring

By focusing on the embedment ratio of each bridge pier, this chapter examines the setting of reference values for normalized values associated with an increase in the likelihood of scouring. The embedment ratio is calculated by dividing the embedment depth of a pier by its average pier width in the stream flow direction. Figure 5 shows the relationship between the embedment ratio and normalized values. The plot distinguishes between several categories of normalized values: those recorded during disaster, the previous maximums, values for prior to the disaster, the normalized values of bridge piers other than the damaged ones at the time of the disaster and at the time the historical maximum occurred, and the normalized values of other undamaged bridges. Furthermore, the figure shows a linear approximation of the relationship between the embedment ratio and the normalized value during the disaster and at the historical maximum. It is generally known that the stability of a bridge pier foundation decreases rapidly once scouring reaches the bottom of the foundation. Therefore, the approximation line assumes that when the intercept is 0—representing the point at which embedment is lost—the foundation becomes unstable due to river flow forces, regardless of the foundation type.

Figure 5 demonstrates a clear positive correlation between the embedment ratio and the normalized value for both the disaster-affected cases and the historical maximum cases. The coefficient of determination is 0.958 for the disaster-affected cases and 0.877 for the historical maximum cases. The approximation line for the disaster-affected cases lies near the upper limit, encompassing nearly all data points. By organizing the normalized values in relation to the embedment ratio, it is evident that a correlation with minimal variation has been obtained. Furthermore, an examination of the pier locations reveals that, as the embedment ratio increases, the plots are distributed in the following order: piers within the low-water channel, those adjacent to the channel, and those on sandbar. In other words, this suggests the possibility of uniquely determining the threshold normalized flow rate at which scouring is likely to occur for a given embedment ratio. Furthermore, the approximation line for the maximum historical value lies slightly below that of the disaster event, and yet remains above the majority of the plotted data. This suggests that the line can become a guideline for the embedment ratio at which the risk of scouring increases.

These results are based on limited data and accuracy must be improved for application in maintenance and management settings. However, we have determined that it is possible to use the normalized value corresponding to a given embedment ratio as a reference

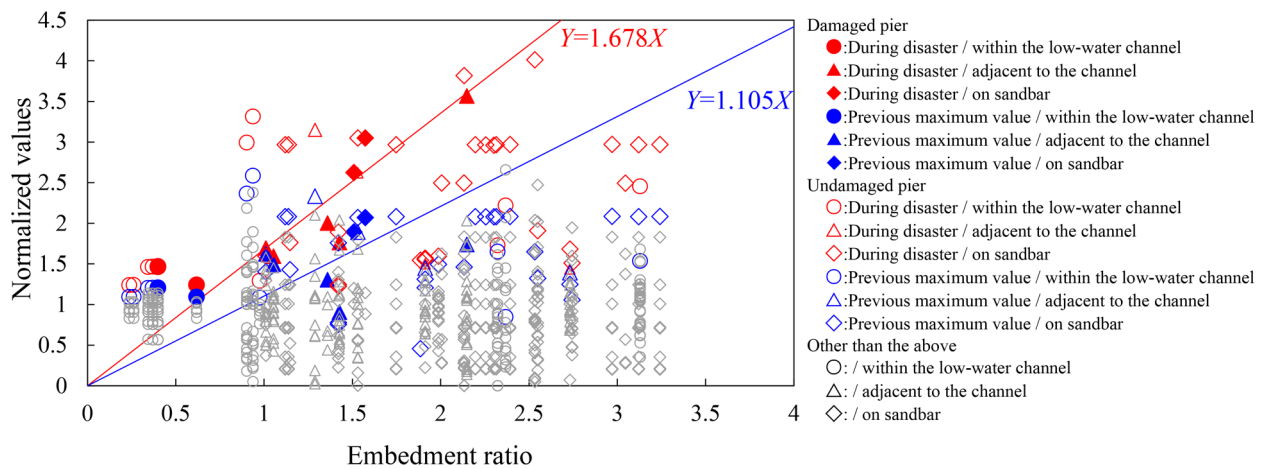


Fig. 5 Relationship between embedment ratio and normalized value [4]

value for the risk of scouring at that embedment ratio.

5. A proposed method for comparing the risk of scouring

This chapter proposes a method for the relative comparison of scouring risk among multiple bridges. The method uses a reference measure defined by the normalized value corresponding to the embedment ratio, as derived from the approximate line presented in Chapter 4.

As a model case, we present a comparative analysis of fictitious Bridge X and Y. First, when comparing fictitious Bridges X and Y, it is necessary to identify the piers on each bridge where scouring is a primary concern. One approach involves selecting target piers based on their weighting scores [2] [3]. In this case, we select an unprotected pier with an embedment ratio of 1.1 for Bridge X and one with an embedment ratio of 1.3 for Bridge Y, to determine which of the two carries a higher risk of scouring. Substituting the reference values into the linear regression equation from Fig. 6 yields normalized values of approximately 1.85 for Bridge X and 2.18 for Bridge Y.

In studies comparing the rainfall resistance of slopes, some approaches assess risk based on the return intervals of rainfall events that trigger landslides [7]. Furthermore, Hoshino et al. evaluate flood risk by focusing on the non-exceedance probability of the annual maximum discharge [8]. The proposed method follows a similar approach by focusing on the normalized values relative to the reference measures and their return period. It assumes that variations in return periods reflect historical flood data, which incorporates the characteristics of each river basin and runoff.

Figure 6 shows the extracted bridge piers, with the return period on the horizontal axis and the normalized values corresponding to the reference measures on the vertical axis. For example, in the case of Bridge X, the return period corresponding to the reference value of 1.85 is indicated by a triangle (▲) on the horizontal axis of Fig. 6. Conversely, for Bridge Y, the return period corresponding to the reference value of 2.18 is indicated by a circle (●) on the horizontal axis of Fig. 7. Comparing the two, since the return period for the circle (●) is shorter than that for the triangle (▲), external forces which can induce scouring are expected to occur more frequently. This leads to the determination that Bridge Y carries a higher relative risk.

If the return periods for the normalized values at the time of the disaster are approximately equal, the relative risk can be assessed by comparing historical maximum data. Specifically, the risk is evalu-

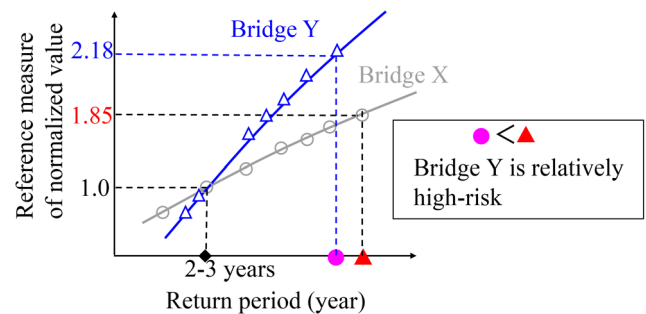


Fig. 6 Comparison of pier risk levels based on the return period corresponding to reference measure of normalized value [4]

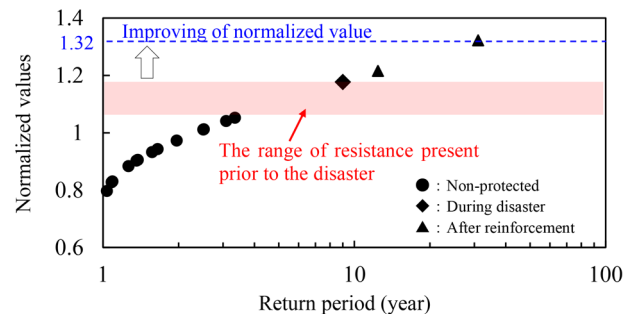


Fig. 7 Improved resistance through application of reinforcement (Bridge C) [4]

ated based on the relationship between the normalized values experienced by the bridge in question up to the time of the analysis, the historical maximum normalized values and the approximation lines in Fig. 5, alongside their corresponding return periods.

This proposed method permits comparison of relative risk levels based on past experience, even when evaluating multiple bridges with piers at high risk of scouring.

In this study, the range where the normalized value exceeds 1 is considered appropriate for applying the relationship between the embedment ratio and the normalized value. This is because the return period for a normalized value of 1.0 is generally 2 to 3 years. From a maintenance perspective, it is critical for this period not to be shorter than the 2-year cycle required for periodic railway inspections. An embedment ratio result with a normalized value less than

1 would signal the need for additional individual inspections, and depending on the situation may require implementation of countermeasures, for example, foundation reinforcement or the installation of scouring protection.

To verify the effectiveness of the proposed method, it is important to confirm that the normalization values reflect improved conditions following the installation of scouring protection or other countermeasures. The limited number of cases limits the generalizability of these findings. However, Bridge C experienced flooding that exceeded the levels observed during of the original disaster. This occurred after riverbed-reinforcing blocks were installed across the full width of the riverbed as part of post-disaster restoration measure. Figure 7 illustrates the relationship between the normalized value and the return period experienced during and after the disaster. The horizontal axis of the figure corresponds to the same return period scale used in Fig. 6. The figure shows that a flood with a normalized value of 1.32 occurred after the reinforcement. This clearly demonstrates that the measures were effective in increasing the bridge's capacity to withstand higher normalized values.

6. Conclusions

For bridges in past railway scouring disaster cases where external forces could be calculated, we determined a normalized value defined as the ratio of the dimensionless tractive force acting on the bridge piers to the average of annual maximum tractive forces. Based on the relationship between this calculated normalized value and the embedment ratio, we proposed a method for comparing the risk of scouring damage among bridge piers. The findings of this study are summarized as follows:

(1) For eight bridges with available water level or discharge records, the annual maximum dimensionless tractive force at the cross-sectional location of the railway bridge was calculated based on the observation records for each year. Furthermore, to compare the scouring resistance of each pier, the normalized value was determined as the ratio of the annual maximum dimensionless tractive force to its long-term average. Since a linear correlation was observed between the calculated normalized value and the embedment ratio of damaged piers, we proposed using the normalized value corresponding to a given embedment ratio as a reference value for the risk of scouring at that ratio.

(2) By focusing on the return period of the normalized value associated with the embedment ratio, we developed and proposed a method for quantitative comparison of scouring risk. As a result, we demonstrated that bridges at risk of scouring can be identified by

comparing the return period of the normalized values corresponding to their embedment ratios.

It should be noted that the approximate linear relationship between the embedment ratio and the normalized value described above is based on a limited number of cases. In order to apply this as a general standard value to current maintenance and management methods, further improvements in accuracy and verification of validity based on the accumulation and analysis of case studies are necessary.

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Development of Grouted Ballasted Track for Existing Shinkansen Asphalt Roadbed

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One way to reduce maintenance costs for ballasted track is to replace existing lines with grouted ballasted track. In the construction of grouted ballasted track, the voids within the ballast bed are filled with a cement-based grout to form a pre-packed concrete bed. In this study, we proposed a design method for installing grouted ballasted tracks on an existing 40-year-old asphalt roadbed. We confirmed that the safety requirements for preventing structural and fatigue failure in both the asphalt roadbed and pre-packed concrete bed were satisfied.

Key words: asphalt roadbed, existing lines with grouted ballasted track, superconducting magnet

1. Introduction

In ballasted tracks, plastic settlement occurs primarily within the ballast bed due to repeated train loading, necessitating periodic track maintenance. In recent years, the increased operating speeds of Shinkansen trains have led to a corresponding rise in maintenance costs associated with higher-speed operation [1]. For example, ballast screens (Fig. 1) [2] have been installed to prevent ballast scattering caused by snow falling from high-speed trains along ballasted Shinkansen track sections crossing cold and snowy regions. Consequently, the need to remove and reinstate ballast screens during track maintenance further increases overall maintenance costs.

To reduce plastic settlement of ballast beds, a practical approach involving the transition to grouted ballasted track (hereafter referred to as the “ballastless track”) has been implemented on conventional railway lines. This track structure incorporates a pre-packed concrete bed, constructed by grouting the voids of the existing ballast layer with a cement-based grout (Fig. 2) [3]. Transitioning from ballasted to ballastless tracks on Shinkansen lines is expected not only to reduce maintenance costs but also prevent ballast scattering caused by snow falling from high-speed trains. Previous studies have examined application of this transition to ballasted tracks installed on viaducts [4]. This paper, however, focuses on asphalt roadbeds. As illustrated in Fig. 3, asphalt roadbeds consist of an asphalt mixture layer and a layer of graded crushed stone.

In a previous study [5] on Shinkansen ballasted tracks constructed on asphalt roadbeds, a 300-mm ballast layer using aggregate with a maximum particle size of 63 mm was investigated. However, since the particle size was too large relative to the thickness of the pre-packed concrete layer, crushed stone 4020 (commonly used for concrete) was adopted as the aggregate. Furthermore, when applying the ballastless track to an existing asphalt roadbed, it should be noted that approximately 40 years have already elapsed since asphalt roadbeds were first introduced on Shinkansen ballasted tracks. Assuming a design service life of 50 years for ballastless tracks, the existing asphalt roadbed would thus be required to remain in service for a total of 90 years.

The *Design Standards for Railway Structures and Commentary—Earth Structures* (hereafter referred to as the “Earth Structure

Standards”) [6] provide a verification method for newly constructed asphalt roadbeds based on the allowable number of load cycles. However, neither these standards, nor related specifications from the road engineering field, specify a verification method for asphalt roadbeds that have been in service for 40 years.

Accordingly, this study aims to develop a ballastless track structure applicable to asphalt roadbeds that have been in service for 40 years and propose a corresponding performance verification method. Previous findings for conventional railway lines indicate that the flexural fatigue strength of pre-packed concrete using ballast as the coarse aggregate can be evaluated using the same fatigue equation as ordinary concrete [7]. The present development adopts ballast for Shinkansen as the aggregate in the pre-packed concrete bed, while employing a grout material with higher strength than that used in conventional lines.

To evaluate the flexural strength and flexural fatigue strength required for the performance of the pre-packed concrete bed, flexural fatigue tests were first conducted. Next, to determine the allowable number of load cycles required for assessing the existing asphalt roadbed that has been in service for 40 years, flexural fatigue tests of asphalt mixtures with simulated long-term aging were performed. Finally, FEM analysis was conducted to compute the stresses in the pre-packed concrete bed and the strains in the existing asphalt roadbed. Based on these results, the performance of the ballastless track installed on the asphalt roadbed was verified.

2. Strength characteristics of the pre-packed concrete bed

2.1 Test methods and specimens for pre-packed concrete

To evaluate the strength characteristics of the pre-packed concrete used for the track bed, flexural tests (JIS A 1106) and flexural fatigue tests were conducted in accordance with JIS A 1106. The loading conditions for the flexural fatigue tests were based on the standard. The loading configuration is shown on Fig. 4.

The dimensions of the pre-packed concrete specimens were set at 800×200×200 mm (L×W×H). The specimens were produced by placing and compacting ballast inside the formwork to a target den-

sity of 1.6 g/cm³. Then the grout material, described in Table 1, was injected and left to harden. The grout injection was performed by applying vibrations to the formwork using a hand vibrator. The grout material employed was a commercially available premixed type that satisfies the following specifications: a flowability of 8 ± 2 s (J14 flow cone), a compressive strength of at least 24 N/mm² at 3 hours (and 45 N/mm² at 28 days), a workability time of 15-35 minutes, and sufficient freeze-thaw resistance. The specimens were cured in air, and the curing period was set to 28 days.

2.2 Test results and discussion for pre-packed concrete

The results of the flexural fatigue tests are presented in Fig. 5. The figure also includes the estimated flexural fatigue strength calculated using the flexural strength obtained from the flexural test (3.1 MPa). The fatigue strength was estimated using Eq. (1), an empirical equation for ordinary concrete proposed in a previous study [8]. The flexural fatigue strength of the pre-packed concrete was found to be comparable to, or slightly higher than, the values estimated from Eq. (1). This confirms that the flexural fatigue strength of pre-packed concrete can be evaluated conservatively using the fatigue equation developed for ordinary concrete.

$$f_{crd} = k_{cr} \cdot f_k \cdot \left(1 - \frac{\sigma_{min}}{f_k}\right) \cdot \left(1 - \frac{\log N}{K}\right) / \gamma_c \quad (1)$$

where

f_{crd} : fatigue strength of concrete (MPa),

f_k : characteristic strength of concrete (MPa),

K : 17,

k_{cr} : 1.0,

N : number of load cycles to failure,

σ_{min} : minimum stress (taken as 0 MPa in this study),

γ_c : material factor for concrete (1.0 for estimation and 1.3 for verification).

3. Strength characteristics of existing asphalt mixture

3.1 Test methods and specimens for the existing asphalt mixture

To evaluate the strength characteristics of the asphalt mixture that has been in service for 40 years, flexural failure tests (Pavement Investigation and Testing Manual B005) and flexural fatigue tests were conducted in accordance with Pavement Investigation and Testing Manual (B005 and B018T), respectively. The flexural failure tests were performed under displacement control at a loading rate of 50 mm/min and a test temperature of 20°C. Similarly, the flexural fatigue tests were also conducted under displacement control with a loading frequency of 10 Hz at the same temperature. The strain applied to the bottom surface of the specimens during the flexural fatigue tests were set to four levels: 250 μ , 200 μ , 150 μ , and 100 μ . The strains were calculated from the measured displacement of the specimens.

The dimensions of the specimens prepared from the existing asphalt mixture were 300×100×50 mm (L×W×H) for the flexural failure tests, and 400×40×50 mm for the flexural fatigue tests. As shown in Fig. 6, the asphalt mixture layer consists of aggregates and asphalt binder, which serves to bond the aggregates together. It is generally known that the viscosity of asphalt increases, so that the material becomes harder due to aging induced by traffic loading and temperature fluctuations [9].

To evaluate the viscosity of the asphalt binder in the aged as-

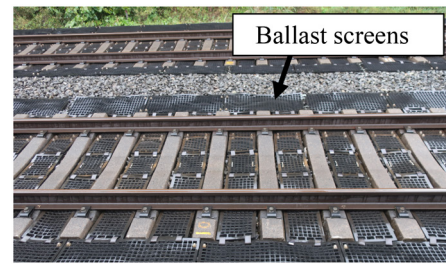
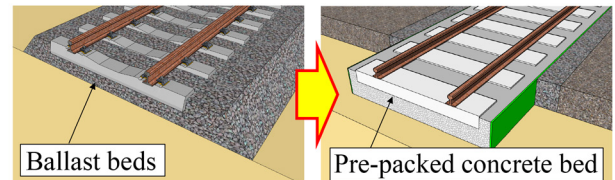


Fig. 1 Installation condition of ballast screen



(a) Ballasted track (b) Existing lines with ballastless track

Fig. 2 Renewal of ballastless track on existing lines

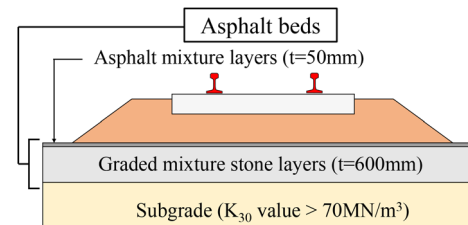


Fig. 3 Example of layer configuration of an asphalt road-bed for ballasted track

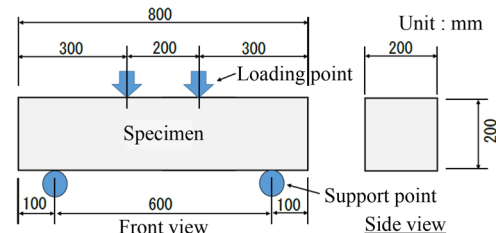


Fig. 4 Loading method for the flexural failure test and flexural fatigue test on pre-packed concrete specimens

Table 1 Mix proportions of the grout material (12 L)

Ultra-rapid hardening cement	Resin powder	Mixing water	air-entraining agent
25 kg	0.25 kg	5.0 kg	0.0625 kg

phalt mixture layer, the asphalt was recovered from the existing layer using the Abson recovery method (Pavement Investigation and Testing Manual G029). A penetration test (Pavement Investigation and Testing Manual A041) was then conducted to assess the hardness of recovered asphalt. The penetration test evaluates hardness based on the depth of penetration made by a standard needle, where one penetration unit corresponds to 1/10 mm. The results indicated

a penetration value of 11 for the aged asphalt, a substantial decrease from the typical value of approximately 60 at the time of construction, confirming a significant hardening of the material.

While exposing specimens fabricated under actual environmental conditions would be ideal for simulating the long-term aging of asphalt, such an approach is not practical. Therefore, a thermal aging method was employed to simulate long-term aging of the asphalt binder of the asphalt mixture. In accordance with the “Laboratory Preparation Method for Thermally Aged Asphalt Mixtures without Compaction” (Pavement Investigation and Testing Manual B020), the asphalt mixture was aged in a temperature-controlled chamber at 160°C for 24 hours. This process was designed to reach a penetration value of 11, after which the mixture was compacted to fabricate the test specimens.

3.2 Test results and discussion for the existing asphalt mixture

Figure 7 shows the results of the flexural tests. This figure indicates that the flexural toughness of the existing asphalt mixture—defined as the area under the load-displacement curve—was lower than that of the new asphalt mixtures. However, the flexural failure load increased by a factor of 2.8.

Figure 8 presents the results of the flexural fatigue tests that are presented. The figure also includes the allowable number of load cycles calculated using Eq. (2), which is provided in the Earth Structure Standards. In this calculation, the elasticity coefficient E_A of the elastic modulus of the As mixture layer corresponding to a test temperature of 20°C was set to 2,500 MPa. Meanwhile, the air void ratio V_V and asphalt content V_b were set to 5.0% and 5.25%, respectively, based on the values measured during specimen preparation. It should be noted that Eq. (2) is obtained by multiplying the failure criterion equation used for asphalt pavement design by a factor of 0.6. This reduction factor is applied because, unlike roadways where vehicle wheel paths vary laterally, railway loading occurs along a fixed track. This results in minimal lateral load dispersion and a consequently higher accumulation of fatigue damage.

$$N_A = 0.6 \times 18.4 \times C \times 6.167 \times 10^{-5} \varepsilon_t^{-3.291} E_A^{-0.854} \quad (2)$$

where

N_A : allowable number of load cycles for fatigue failure of the As mixture layer,

ε_t : tensile strain at the bottom of the As mixture layer,

E_A : elastic modulus of the As mixture layer,

C : a function of air void ratio V_V and asphalt content V_b of the As mixture layer, expressed as $C=10^M$,

M : parameter defined as $M=4.84 \left[\frac{V_b}{V_b+V_V} - 0.69 \right]$.

Figure 8 shows that, for the new asphalt mixture, the number of load cycles to failure obtained from the test results is in good agreement with the allowable number of load cycles calculated using Eq. (2). In contrast, for the aged asphalt mixture, the test results deviated significantly from the allowable load cycles predicted by Eq. (2). Compared with the allowable load cycles from Eq. (2), the number of cycles failed decreased at a strain level of 250 $\mu\epsilon$, while it increased to strain levels of 200 $\mu\epsilon$ or lower. This behavior is attributed to the increase in the flexural strength of the asphalt mixture layer resulting from long-term aging. This increase reduced the ratio of flexural stress to flexural strength, thereby extending the number of cycles to failure. Therefore, the allowable load cycle equation shown in Eq. (2) was found to be inapplicable to the existing asphalt mixture.

To address this issue, the number of cycles to failure for the

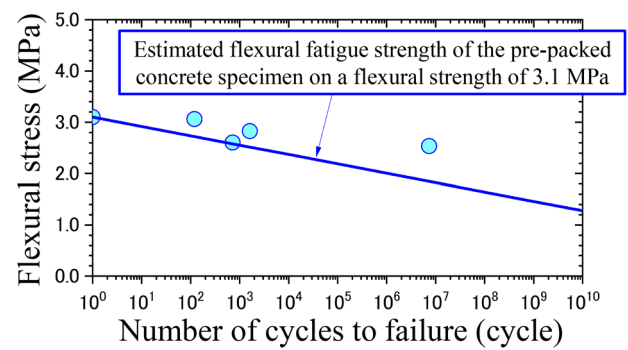


Fig. 5 Flexural fatigue test

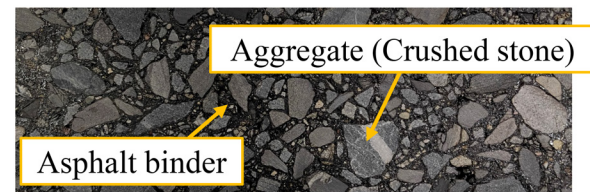


Fig. 6 Cross-section of the asphalt mixture layer

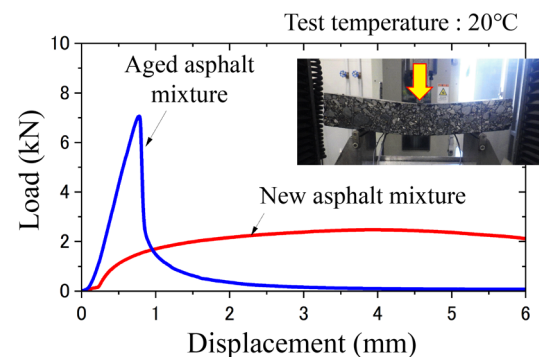


Fig. 7 Results of the flexural test

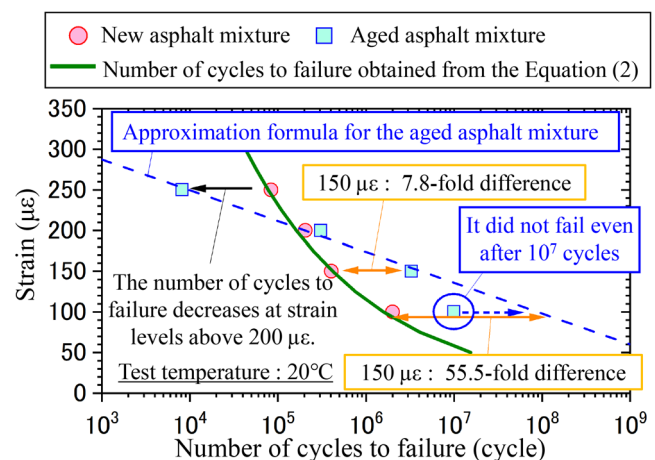


Fig. 8 Results of flexural fatigue test

existing asphalt mixture was estimated using logarithmic trendlines fitted to the experimental results. Fig. 9 shows the ratio of the number of cycles to failure of the existing asphalt mixture to the new

asphalt mixture. Under the test conditions used in this study, the ratios were 0.1, 1.2, 7.8, 55.5, and 118.9 at strain levels of 250, 200, 150, 100, and 50 $\mu\epsilon$, respectively. This indicates that the allowable number of load cycles decreased when the strain exceeded 200 $\mu\epsilon$. Previous studies in the road engineering field [10, 11] have also reported this trend. At strains of 200 $\mu\epsilon$ or less, asphalt mixtures in service for approximately 13 years tend to exceed the allowable cycles estimated for new mixtures.

4. Performance verification of ballastless track on an existing asphalt roadbed

4.1 Performance verification of pre-packed concrete bed

A performance verification was conducted for both pre-packed concrete bed and asphalt roadbed under the assumption that the ballastless track would be installed on an aged asphalt roadbed. In this verification, the service period of the existing asphalt roadbed prior to the track installation of the ballastless track was set at 40 years, with a design service life of 50 years specified for the post-installation period.

Figure 10 shows the dimensions of the ballastless track. The analytical models used to calculate response values are illustrated in Fig. 11. Figs. 11(a) and 11(b) represent the models for the ballastless track and for the ballasted track, respectively. The material properties used in the analysis, including the elastic modulus E and Poisson's ratio ν , are also provided in Fig. 11. The subgrade reaction coefficient K_{30} was set to 70 MN/m³, which is the minimum required value for new railway lines.

4.1.1 Method for performance verification of the pre-packed concrete bed

The performance items and required performance for the pre-packed concrete bed were determined in accordance with the *Design Standards for Railway Structures and Commentary - Track Structures* (hereafter referred to as the "Track Standards") [12]. The verification targets included safety against structural failure and fatigue failure, as well as serviceability related to appearance and damage.

For safety verification, the limit states for the pre-packed concrete bed were defined as the flexural strength for structural failure and the flexural fatigue strength for fatigue failure. Furthermore, since cracking of the pre-packed concrete bed is not permitted during service, serviceability verifications related to appearance (based on crack width) and damage (based on flexural strength) were omitted.

Figure 12 shows procedure for performance verification, and Eq. (3) indicates the method for performance verification.

$$\gamma_i \cdot (I_{Rd} / I_{Ld}) \leq 1.0 \quad (3)$$

where

γ_i : track structural factor,
 I_{Rd} : design response value,
 I_{Ld} : design limit value.

Table 2 presents the safety factors used for the performance verification of the pre-packed concrete bed. The track member factor γ_b was set to 1.1 rather than the standard 1.0 for reinforced concrete (RC) track structures, since the pre-packed concrete bed is an unreinforced concrete member. This higher factor reflects the variability of the material and its structural importance as direct support for the sleepers.

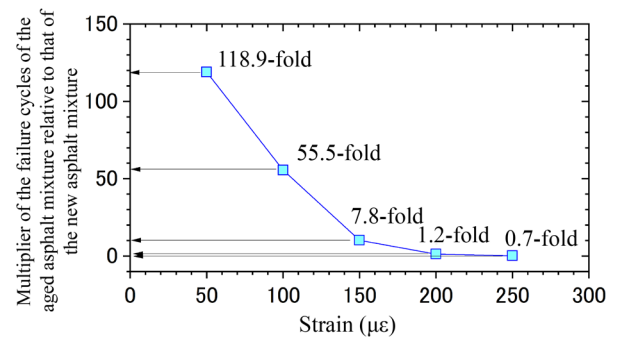


Fig. 9 Multiplier of failure cycles of aged asphalt mixture relative to new asphalt mixture

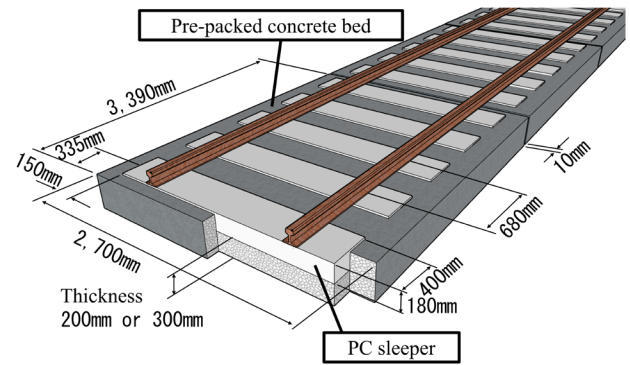
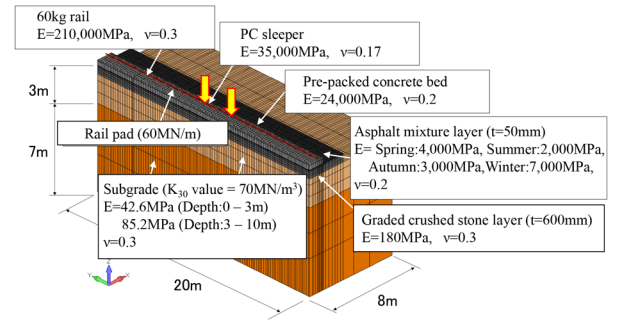
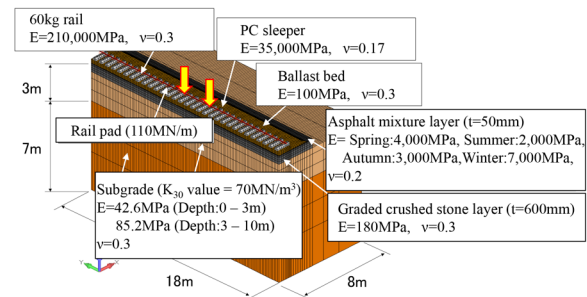


Fig. 10 Dimensions of existing lines with ballastless track



(a) Existing lines with ballastless track



(b) ballasted track

Fig. 11 Analytical model (1/2 symmetry)

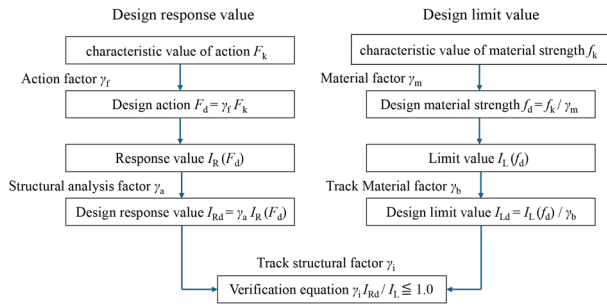


Fig. 12 Procedure for performance verification

Table 2 Safety factors used for performance verification of the pre-packed concrete bed

Required performance	Action factor γ_f	Structural analysis factor γ_a	Material factor γ_m	Track Material factor γ_b	Track Structural factor γ_i
Safety (failure)	1.1	1.0	1.3	1.1	1.1
Safety (fatigue failure)	1.0	1.0	1.3	1.1	1.1

4.1.2 Design response values for pre-packed concrete bed

The response value used for performance verification is the flexural stress at the bottom of the pre-packed concrete bed caused by train loading. This value was obtained through FEM analysis using the analytical model shown in Fig. 11(a). The elastic modulus of the asphalt mixture layer was set to 2,000 MPa (summer condition), the minimum value used in design. This is because a lower elastic modulus of the asphalt mixture layer results in larger response values in the pre-packed concrete bed. In calculating the response value, the dynamic component of the wheel load was considered by multiplying the static wheel load by a dynamic load factor.

For the performance verification against fatigue failure, the dynamic load factor was set to 2.1. This value was derived using Eq. (4), which defines the speed-impact ratio for long-welded rail sections in design of asphalt roadbeds [6]. The calculation assumes a maximum operating speed of 360 km/h, the upper limit speed for domestic high-speed rail operations.

$$i = 1 + 0.3 V / 100 \quad (4)$$

where

i : speed-impact ratio,
 V : train speed (km/h).

The dynamic load factor used for the verification of structural failure at a train speed of 360 km/h was determined based on a previous study [5]. In that study, the dynamic load factor related to fatigue failure acting on the pre-packed concrete bed was 1.45 at a train speed of 260 km/h, whereas the dynamic load factor related to structural failure was 2.2, giving a ratio of 1.52. Accordingly, for a train speed of 360 km/h, the dynamic load factor for structural failure was taken as 3.2, which was obtained by multiplying the dynamic load factor for fatigue failure at 360 km/h (2.1, as mentioned earlier) by 1.52.

The flexural stress generated at the bottom of the pre-packed concrete bed under a static wheel load of 85 kN (per bogie) was

0.295 MPa. Therefore, the design response value I_{Rd} for fatigue failure was calculated by multiplying this stress by the dynamic load factor of 2.1, the load factor of 1.0, and the structural analysis factor of 1.0, resulting in 0.62 MPa. Similarly, the design response value I_{Rd} for structural failure was obtained by multiplying the stress by the dynamic load factor of 3.2, the load factor of 1.1, and the structural analysis factor of 1.0, resulting in 1.04 MPa.

4.1.3 Design limit values for pre-packed concrete bed

The characteristic value of flexural fatigue strength, used as the design limit value I_{Ld} for fatigue failure, was determined using Eq. (1). The standard ballast-bed thickness in sections for asphalt roadbeds is 300 mm, differing from the 200 mm specimen thickness used in the flexural tests. Therefore, the size effect was applied to the characteristic flexural strength used in Eq. (1). By applying the same size-effect relationship as that for ordinary concrete [13], the characteristic flexural strength was calculated to be 2.92 MPa corresponding to the measured value of 3.1 MPa (Fig. 5).

The number of load cycles was taken as 74.46 million, corresponding to 50 years of design service life with 60 trainsets (each consisting of 17 cars) passing per day. Therefore, the design limit I_{Ld} for fatigue failure, i.e., the design flexural fatigue strength, was obtained by dividing the characteristic flexural fatigue strength of 1.57 MPa by a material factor of 1.3 and a track member factor of 1.1, resulting in 1.10 MPa.

Similarly, the characteristic value of flexural strength used as the design limit value I_{Ld} for structural failure was set at 2.92 MPa, considering the same size effect as described above. Thus, the design limit value I_{Ld} for structural failure, or the design flexural strength, was obtained by dividing 2.92 MPa by the material factor of 1.3 and the track member factor of 1.1, resulting in 2.04 MPa.

4.1.4 Performance verification results for pre-packed concrete bed

For fatigue failure, the design response value I_{Rd} was 0.62 MPa, and the design limit value I_{Ld} was 1.10 MPa. Using a track structural factor of 1.1, Eq. (3) gave a ratio of 0.62.

Similarly, for structural failure, the design response value I_{Rd} was 1.04 MPa, and the design limit value I_{Ld} was 2.04 MPa. Using a track structural factor of 1.1, Eq. (3) resulted in a ratio of 0.56.

Based on these results, it was verified that the pre-packed concrete bed satisfies the safety requirements for both fatigue failure and structural failure.

4.2 Verification of the existing asphalt roadbed

4.2.1 Method for performance verification of the existing asphalt roadbed

The performance items and required performance for verifying the existing asphalt roadbed following installation of the ballastless track were established based on the design provisions for asphalt roadbeds in the *Earth Structure Standards*. The verification focuses on safety against fatigue failure. The performance index used in the verification is the allowable number of load cycles of the existing asphalt mixture layer following installation of the ballastless track. The verification target is the asphalt mixture layer that constitutes the asphalt roadbed.

The performance verification of the existing asphalt roadbed employs the damage ratio defined in Eq. (5). The seasonal damage ratios m_{Ai} (for spring, summer, autumn, and winter) for the asphalt

mixture layer are determined by Eq. (6) as the actual number of load cycles n_i to the allowable number of load cycles N_{Ai} for each season.

$$M_A = \sum_{i=1}^4 m_{Ai} \leq 1 \quad (5)$$

$$m_{Ai} = n_i / N_{Ai} \quad (6)$$

4.2.2 Number of load cycles acting on existing asphalt roadbed

Figure 13 presents the strain distribution generated at the bottom of the asphalt mixture layer, obtained from the FEM analyses conducted using the analytical models shown in Figs. 11(a) and 11(b). The elastic modulus of the asphalt mixture layer was set to 2,000 MPa. For each, the location of the maximum strain varied by track structure: in the ballasted track, it occurred near the sleeper end (approximately 1,110 mm), while in the ballastless track, it occurred near the edge of the pre-packed concrete bed (approximately 1,350 mm). This indicates that the position of the maximum tensile strain at the bottom of the asphalt mixture layer varies depending on the track structure. In addition, at approximately -1,350 mm (corresponding to the location of 1,350 mm in the ballastless track), the strain at the bottom of the asphalt mixture layer supporting the ballasted track was zero, with compressive (negative) strain occurring farther outward. Based on these observations, it is considered unnecessary to include the damage accumulated in the pre-renewal to that accumulated post-renewal when verifying the safety of the asphalt mixture layer for fatigue failure in the transition from ballasted track to ballastless track.

Accordingly, the number of load cycles used for evaluating the damage ratio of the existing asphalt mixture layer was limited to only the period following the track structure renewal. As described in Section 4.1.3, this value was set at 74.46 million cycles for a design service life of 50 years. It should be noted that the number of load cycles per season was therefore 18.615 million.

4.2.3 Allowable number of load repetitions for existing asphalt roadbed

The allowable number of load repetitions for the existing asphalt mixture layer was determined based on the maximum seasonal tensile strain at the bottom of the asphalt mixture layer, derived from FEM analysis using the analytical model shown in Fig. 11(a). Since no established formula exists for calculating the allowable number of load repetitions for an existing asphalt mixture layer, the allowable load repetitions for the existing asphalt mixture layer were derived by applying the relationship for new asphalt shown in Fig. 9.

The procedure for calculating the allowable number of load repetitions for the existing asphalt mixture layer consists of the following four steps (step 1 to 4):

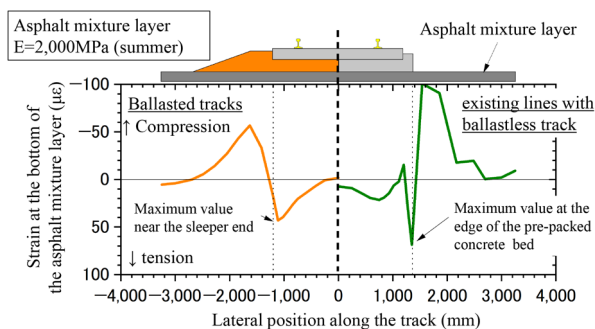


Fig. 13 Analysis results

Step ①: Determine the maximum tensile strain at the bottom of the asphalt mixture layer through FEM analysis, using the elastic modulus of a new asphalt mixture layer;

Step ②: Calculate the allowable number of load repetitions for new asphalt using Eq.(2), based on the maximum strain obtained in Step ① and the elastic modulus of the asphalt mixture layer;

Step ③: Determine the multiplication factor for the existing asphalt mixture from Fig. 9, based on the maximum strain from Step ①;

Step ④: Calculate the allowable load repetitions for the existing asphalt mixture layer by multiplying the allowable number of load repetitions from Step ② by the factor derived in Step ③.

In Step ①, the seasonal elastic moduli of newly constructed asphalt mixture layers were used for the FEM analysis. The values were set to 4,000 MPa for spring, 2,000 MPa for summer, 3,000 MPa for autumn, 7,000 MPa for winter [6]. It should be noted that the elastic modulus of asphalt mixture layers used in asphalt pavement design is typically up to approximately 12,000 MPa [14]. The dynamic load was set at 178.5 kN per wheel, calculated by multiplying the static wheel load of 85 kN by a dynamic amplification factor of 2.1 (as described in Section 4.1.2), assuming a train speed of 360 km/h.

FEM analysis was conducted using the analytical model shown in Fig. 11(a). to verify the relationship between the elastic modulus of the existing asphalt mixture layer and the resulting strain. Based on previous estimates derived from measured deflections in load tests [15], the elastic modulus of asphalt mixtures in service for over 15 years varied widely, reaching values up to approximately 50,000 MPa.

The analysis results, shown in Fig. 14, indicate that the generated strain remained below 100 με. Within the modulus range of new asphalt mixtures, the maximum strain was observed at approximately 7,000 MPa. As the elastic modulus increased, the strain decreased. This trend can be explained as follows (see Fig. 15). At an elastic modulus of 2,000 MPa, the asphalt mixture layer becomes more flexible directly beneath the track. This flexibility leads to the development of tensile strain under the rail and compressive strain under the sleeper ends, where train loads are most influential. As a result, the strain beneath the edge of the pre-packed concrete bed is smaller than that at 7,000 MPa.

In Step ②, using the seasonal maximum strains obtained in Step ① (Fig. 16), the allowable number of load repetitions for a newly constructed asphalt mixture layer is calculated using Eq. (2).

In Step ③, the corresponding multiplication factors are determined from Fig. 9 using the seasonal maximum strains obtained in Step ① (Fig. 16). In this study, strains in both the aged asphalt mix-

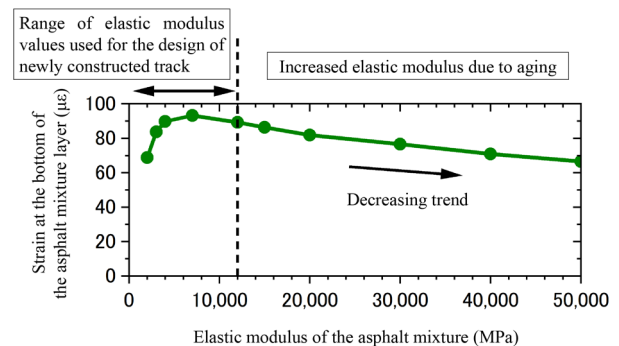


Fig. 14 Relationship between elastic modulus and strain of asphalt mixture

ture layer and the new asphalt mixture layer were found to be less than $100 \mu\epsilon$. Therefore, the multiplication factor of 55.5 (the value at $100 \mu\epsilon$) was applied uniformly to ensure a conservative evaluation. Although temperature-dependent seasonal effects are expected in existing asphalt mixtures as well as in new mixtures, the same multiplication factor was adopted.

In Step ④, the allowable number of load repetitions for the existing asphalt mixture layer is determined by multiplying the value (from Step ②) by the factor derived in Step ③ (55.5 in this study). It should be noted that fatigue test results for aged asphalt mixtures [16] indicate an increase in the number of repetitions to failure as the test temperature decreases. This trend is consistent with that observed in new asphalt mixtures.

Table 3 summarizes the elastic moduli, maximum strains, al-

lowable load repetitions, and multiplication factors obtained from Fig. 9, as well as the number of load repetitions and damage indices described in Section 4.1.2.

4.2.4 Performance verification results for existing asphalt roadbed

The total damage index M_A was calculated by summing the seasonal damage indices m_{Ai} using Eq. (5). As shown in Table 3, the resulting damage index for the existing asphalt mixture layer was 0.76, which is less than 1.

Based on the above, it was confirmed that the existing asphalt roadbed satisfies the safety requirements against fatigue failure.

5. Conclusions

The material properties of pre-packed concrete and asphalt mixtures were examined through flexural fatigue testing. Furthermore, the performance of a ballastless track structure installed on an existing asphalt roadbed was evaluated. The main findings are summarized as follows:

1. Applicability of the fatigue equation to pre-packed concrete

It was demonstrated that the conventional flexural fatigue strength equation for ordinary concrete is also applicable to pre-packed concrete, which is formed by injecting high-strength grout into the voids between ballast particles on Shinkansen tracks.

2. Proposed method for evaluating allowable load repetitions of existing asphalt roadbed

A new evaluation method is proposed for estimating the allowable number of load repetitions of an existing asphalt mixture layer. In this method, the allowable repetitions for new asphalt—calculated using the standard design equation—are multiplied by a factor derived from the fatigue lives of aged asphalt mixtures measured in flexural fatigue tests.

3. Performance assessment of a ballastless track on 40-year-old asphalt roadbeds

This study assumed the installation of a ballastless track on an existing asphalt roadbed that has been in service for 40 years. Both the pre-packed concrete bed and the underlying asphalt mixture layer were evaluated under the conditions of a train speed of 360 km/h and a design service life of 50 years. The results confirmed that both structural components satisfy the required performance criteria.

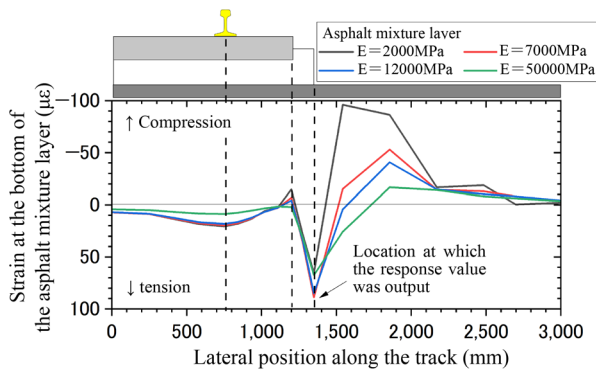


Fig. 15 Strain distribution at bottom of asphalt mixture layer

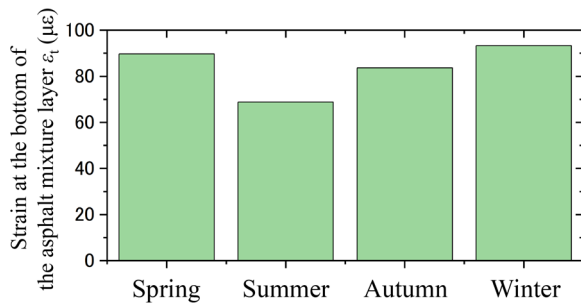


Fig. 16 Strain occurring in each season

Table 3 Verification results of existing asphalt roadbed

Parameter	Spring	Summer	Autumn	Winter
Elastic modulus of the asphalt mixture layer used for design at the time of new construction E_A (MPa)	4,000	2,000	3,000	7,000
Strain at the bottom of the asphalt mixture layer ϵ_t ($\mu\epsilon$)	89.7	68.8	83.6	93.2
Allowable number of load applications for new asphalt mixture layer (cycle)	1,642,511	7,117,744	2,646,740	897,762
Allowable number of load applications for aged asphalt mixture layer N_{Ai} (cycle) [Allowable number of load applications for new asphalt mixture layer $\times 55.5$]	90,338,081	391,475,934	145,570,675	49,376,933
Number of load applications n_i (cycle) [17-car train $\times 4$ axles $\times 60$ trains/day $\times 364$ days/year $\times 50$ years /4]	18,615,000	18,615,000	18,615,000	18,615,000
Seasonal damage level m_{Ai}	0.21	0.05	0.13	0.38
Damage level M_A	0.76			

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Method for Predicting Fatigue Strength of Corroded Rails Using Stress and Stress Gradient around Corrosion Defects

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Rail corrosion, particularly in humid environments, accounts for around 20% of broken rails in Japan. However, traditional methods for predicting the fatigue strength of corroded rails, such as using corrosion thickness and S-N curves, have limitations. The study involved collecting corroded rails, scanning their surface profiles, and applying finite element analysis to evaluate stress at fracturing points. A new method using stress and stress gradients was proposed to predict the number of load cycles required for corroded rails to fracture in bending fatigue tests, with an estimated range of 0.5 to 2 times.

Key words: rail, corrosion, fatigue strength, finite element method

This paper is an extended and revised version of previously published works [1] [2] [3].

1. Introduction

One cause of broken rails is rail corrosion, which occurs mainly in humid environments, such as tunnels and level crossings [4]. As rail corrosion progresses, pitting and thinning occur on the bottom surface and bottom side of the rail, as shown in Fig. 1. When train loads are applied to the rail, stress concentration at corroded areas can cause cracks to initiate and propagate, potentially leading to rail breakage. Additionally, it is known that corroded rails have lower fatigue strength than rails laid in typical open sections, which refer to ordinary track sections excluding special environments such as tunnels and level crossings. This reduction in fatigue strength results in fewer load cycles to failure at specific loads and an absence of a true fatigue limit [4]. Therefore, a detailed evaluation of the fatigue strength and remaining life of corroded rails is necessary.

In Japan, the degree of rail corrosion has been classified using a corrosion thickness index. S-N curves, which relate applied stress to the number of cycles, have been developed for each classification and are used to estimate the fatigue life of corroded rails. In general, rails have a lower limit of applied stress, known as the fatigue limit, below which fatigue failure does not occur in non-corrosive envi-

ronments. By contrast, corroded rails lack a fatigue limit, and fatigue strength decreases as the degree of corrosion increases [4]. However, the conventional method for evaluating fatigue strength based on the amount of corrosion has room for improvement, as it cannot eliminate the influence of variability in fatigue test results.

In recent years, a method that uses the stress and stress gradient calculated by finite element analysis has been proposed for evaluating the bending fatigue strength of various mechanical element shapes [5], [6]. Therefore, evaluating the stress and stress gradient at the location where fracture actually occurred enables a more accurate assessment of rail fatigue strength than previous methods.

Based on the above, this study evaluated fatigue strength by focusing on stresses and stress gradients generated in the vicinity of corrosion defects when bending loads were applied to rails.

2. Collection of corroded rails

In this study, various tests were conducted on rails collected from inside a tunnel. Table 1 lists the collected rails. The rails, fifteen 60kg rails and ten 50kgN rails specified in JIS E 1101, were cut

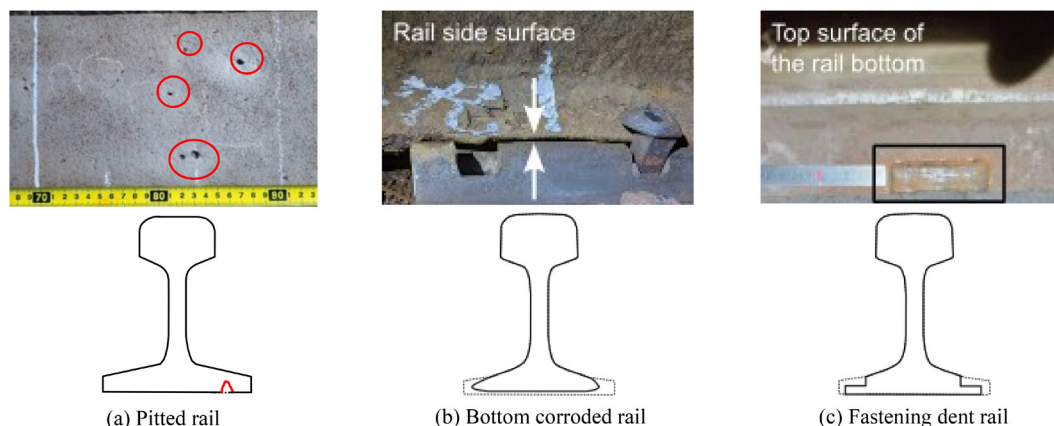


Fig. 1 Example of corroded rails [1][2][3]

to 1.5 meters. Pitting corrosion was observed on the bottom of the rails, as shown in Fig. 1(a), and thinning was observed on the undersides of the rails, as shown in Fig. 1(b) and 1(c). Hereafter, rails with pitting corrosion on the bottom are referred to as “pitted rails,” rails with reduced thickness at the contact area with the track pad are referred to as “Bottom corroded rails,” and rails with reduced thickness on the upper surface of the rail bottom in contact with the flat spring type rail fastening device are referred to as “fastening dent rails.” Before conducting the tests, any rust adhering to the rails was removed by sandblasting, as shown in Fig. 2. Alumina was used as the abrasive. The discharge pressure during blasting was 0.4 MPa, the distance from the nozzle to the test rails was approximately 200–300 mm, and the abrasive consumption rate was 10 kg/h.

3. Methods of testing and stress analysis

3.1 Measurement of rail geometry

The measurement of rail geometry altered by corrosion was carried out using two different 3D scanners. Pitted rails were measured using a handheld 3D scanner (3DSL Rhino-01, manufactured by Seiko Wave) shown in Fig. 3(a). Bottom corroded rails and fastening dent rails were measured using an arm-type 3D scanner (Quantum X FaroArm, manufactured by FARO) shown in Fig. 3(b). For the subsequent discussion, the coordinate system was defined, as shown in Fig. 4. The origin was set at the center of the bottom surface of the rail in its new rail condition, at the central position along the length of the rail. The x -direction was defined as the lateral direction perpendicular to the longitudinal axis of the rail, the y -direction as the longitudinal direction along the rail, and the z -direction as the vertical direction. The measurement range for rail geometry was set to include the area directly below the loading point of the rail bending fatigue test, with a range of $-100 \text{ mm} \leq y \leq 100 \text{ mm}$ for pitted rails, and $-250 \text{ mm} \leq y \leq$

250 mm for Bottom corroded rails and fastening dent rails. The pitted rails showed pitting corrosion on their bottom surfaces, while the other surfaces of the same rails remained almost unchanged from their original geometry. Therefore, only the bottom surface of the rail was measured. On the other hand, the Bottom corroded rails and fastening dent rails showed overall thinning at their bottoms; therefore, the entire circumference of each rail was measured.

3.2 Rail bending fatigue tests

A rail bending fatigue test, as shown in Fig. 5, was conducted. Loading conditions were set to achieve nominal peak-to-peak stress ranges of 220, 250, 280, and 370 MPa under sinusoidal loading, as indicated in Table 2. It is noted that the nominal fatigue strength at 2 million load cycles for actual rails laid in typical open sections is 232 MPa [7], and that corroded rails have reduced fatigue strength compared with rails in typical open sections [4]. Therefore, one of the loading conditions was set to a stress fluctuation range lower than the strength of actual rails laid in typical open sections after 2 million load cycles. Additionally, to prevent fretting fatigue failure at the loading and support points, each contact point was equipped with rotatable rollers. The loading frequency was set to 3.0 Hz, and the maximum number of load cycles was set to 2 million. If the rail did not fracture after 2 million cycles, it was classified as “unfractured.”

3.3 Stress analysis of rail bending fatigue tests considering shape change due to corrosion

Twenty-five models were created considering the corroded geometry of the rails, and a stress analysis was conducted using the finite element method to simulate the rail bending fatigue test for

Table 1 Classification of collected rails [3]

Test rail No.	Rail sections	Corrosion state
1~15	JIS 60kg	Pitted rail
16~20	JIS 50kg N	Bottom corroded rail
21~25	JIS 50kg N	Fastening dent rail



Fig. 2 Side view of test rail [1][3]



(a) Handy type



(b) Arm type

Fig. 3 3D scanner used in the test [3]

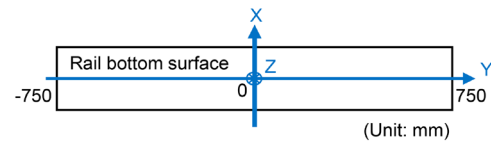


Fig. 4 Coordinate system for rail model [3]

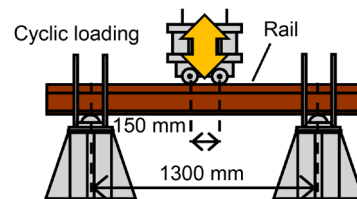


Fig. 5 Overview of rail bending fatigue tests [1][2][3]

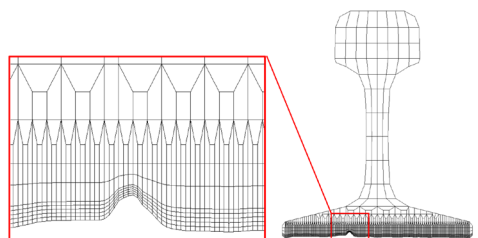
Table 2 Loading conditions in rail bending fatigue tests [1][2][3]

	Nominal stress fluctuation ranges σ_A [MPa]	Maximum load [kN]	Minimum load [kN]
JIS 60kg rail	220	344	42
	280	427	41
	370	551	42
JIS 50kgN rail	220	239	29
	250	268	28
	280	297	29

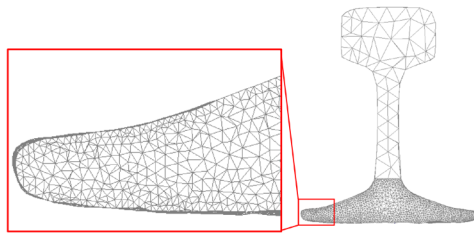
each model. NX Nastran V10.0 was used for the stress analysis. This analysis was performed as an elastic analysis, with Young's modulus E of 2.1×10^5 MPa and Poisson's ratio ν of 0.3. The total length of each model was set to 1,300 mm. Loading conditions were set with a loading interval of 150 mm and a loading force, as shown in Table 2. The boundary conditions constrained the x and z directions at both lower ends of the rail and the y direction at one end of the rail.

The element divisions were varied according to the type of 3D scanner used for geometry measurement. The models of pitted rails and bottom corrosion/fastening dent rails are shown in Fig. 6. Hexahedral first-order elements were used for the pitted rails. The range for reproducing the corroded geometry was set to $-100 \text{ mm} \leq y \leq 100 \text{ mm}$ on the bottom surface of the rail, and the z coordinates of the bottom surface of the rail were moved in 0.01 mm increments according to the measurement results of the 3D scanner. The mesh size of the bottom surface of the rail in the basic model before setting the corrosion depth was, 0.5 mm.

Tetrahedral second-order elements were used for the Bottom corroded rails and the fastening dent rails. The range for reproducing the corroded geometry was set to $-250 \text{ mm} \leq y \leq 250 \text{ mm}$ and $0 \text{ mm} \leq z \leq 30 \text{ mm}$ on the bottom of the rail. In other areas, the geometry was constructed by extending the end shapes of the measurement data from the 3D scanner, and the boundaries of each area were defined using tied-element constraints. The mesh size was set to 1 mm in areas reproducing corrosion and 10 mm in other areas.



(a) Pitted rail



(b) Bottom corroded/ Fastening dent rail

Fig. 6 Example of mesh of XZ plane [1][2][3]

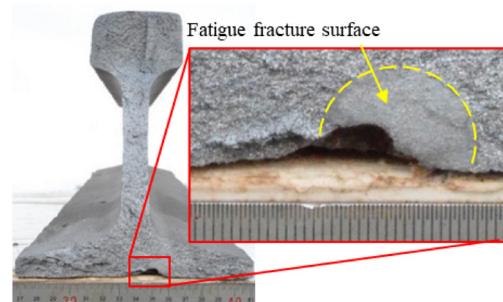
4. Results and discussion

4.1 Results of rail bending fatigue tests

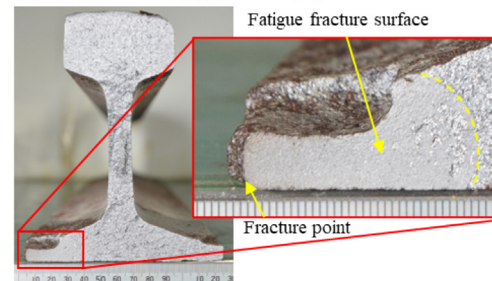
Table 3 shows the results of the fatigue tests. Of the 25 test rails, No. 5 and No. 18 remained unfractured, while the rest fractured. Of these, No. 2 and No. 7 fractured outside the geometry measurement range of the bottom surface of the rail. This is believed to be due to the presence of deeper pitting corrosion outside the measurement range than within it. Figure 7 shows examples of the fracture surfaces of the rails after testing. The smooth fracture surface within the yellow dotted line in the enlarged view indicates the

Table 3 Results of rail bending fatigue tests [3]

No.		σ_A [MPa]	Number of cycles	Feature of fracture initiation point
Pitted rail	1	370	49,217	Bottom surface
	2	220	392,301	Out of measurement range
	3	370	88,131	Bottom surface
	4	370	46,959	Bottom surface
	5	220	Unfractured	-
	6	280	146,649	Bottom surface
	7	280	280,758	Out of measurement range
	8	220	903,620	Bottom surface
	9	220	184,637	Bottom surface
	10	280	134,530	Bottom surface
	11	370	70,554	Bottom surface
	12	220	511,719	Bottom surface
	13	220	263,311	Bottom surface
	14	370	41,490	Bottom surface
	15	280	70,252	Bottom surface
Bottom corroded rail	16	250	111,225	Bottom surface
	17	250	163,558	Bottom surface
	18	220	Unfractured	-
	19	280	50,155	Bottom surface
	20	220	255,098	Bottom surface
Fastening dent rail	21	250	182,396	Bottom surface
	22	250	188,509	Bottom surface
	23	220	191,095	Bottom surface
	24	280	121,409	Bottom surface
	25	220	324,973	Side surface



(a) No. 13 [1]



(b) No. 25

Fig. 7 Fracture surface after rail bending fatigue tests

fatigue fracture area. The rougher fracture surface outside this area corresponds to the brittle fracture area, where the crack propagated rapidly. Observing the initiation points of the fatigue fractures, only No. 25 had its initiation point on the side surface of the rail bottom, while the others were on the bottom surface of the rail.

Figure 8 shows the test results and the S-N curve of the aged rails obtained from previous studies [7]. Except for the unfractured rails No. 5 and No. 18, it was confirmed that the fatigue strength of the corroded rails was lower than that of the aged rails reported in previous studies. The decrease in fatigue strength is attributed to pitting and thinning of the rail bottom caused by corrosion.

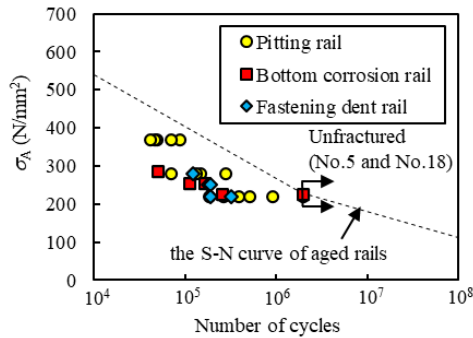


Fig. 8 Results of rail bending fatigue tests [3]

4.2 Results of stress analysis

Figure 9 shows the stress distribution in the y -direction from the fracture initiation point to the z -direction, calculated using FEM. The vertical axis represents dimensionless stress normalized by the maximum stress $\sigma_{y\max}$ at the bottom surface of the rail at the fracture initiation point. In this study, the fracture initiation point is defined as the deepest point of pitting where fatigue cracks occurred. From these results, the stress gradients at the fracture initiation point were calculated using (1).

$$\chi = \frac{1}{\sigma_{y\max}} \left| \frac{\partial \sigma_y}{\partial z} \right|_{z=0} \quad (1)$$

The slope of σ_y at the fracture initiation point, as shown by the dotted line in Fig. 9, refers to the slope of the nonlinear regression model of σ_y at $z = 0$, which corresponds to the solid line in Fig. 9. The nonlinear regression model is represented by (2), and the coefficients, α , β , γ , δ were determined to minimize the sum of squared residuals.

$$\sigma_y / \sigma_{y\max} = \alpha \exp(\beta + \gamma z) + \delta \quad (2)$$

Table 4 presents the range of stress fluctuations and stress gradients at the fracture initial point, calculated using FEM. In the next section, the number of loading cycles at fracture for corroded rails in bending fatigue tests will be estimated using these values.

As mentioned above, the fracture initiation point of No.25 was on the bottom side. In this study, the stress gradient in the z -direction was calculated under the assumption of fatigue crack propagation in the z -direction. Therefore, the results of No.25 are excluded from the following discussion. Additionally, we examined the analysis results for No.15, which showed the largest stress fluctuation range at the fracture point. The stress fluctuation range at the fracture point of No.15 was 916 MPa, and the stress generated when the maximum load was applied in the fatigue test was 1014 MPa. This value exceeds the tensile strength of rail steel, which is 992 MPa [8], indicating that the elastic FEM assumption is no longer valid due to the

onset of plastic deformation. Therefore, the analysis results for No.15 may not accurately reproduce the actual fatigue phenomenon. Consequently, the results of No.15 are also excluded from the following discussion.

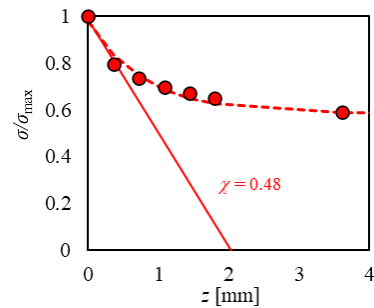


Fig. 9 Calculation examples of stress gradients (No.1) [3]

Table 4 Range of stress fluctuations and stress gradients at fracture initiation point, calculated using FEM [3]

No.	σ_{AFEM} [MPa]	χ	No.	σ_{AFEM} [MPa]	χ
1	664	0.48	14	777	0.87
3	619	0.31	15	916	0.81
4	685	0.38	16	599	0.30
6	344	0.06	17	560	0.29
8	248	0.05	19	604	0.16
9	514	0.57	20	508	0.29
10	588	0.59	21	518	0.25
11	415	0.02	22	533	0.30
12	258	0.04	23	632	0.40
13	487	0.51	24	635	0.56

4.3 Evaluation of fatigue strength by focusing on the stresses and the stress gradients

Siebel et al. found that the ratio of “maximum stress at the stress concentration point, $\sigma_{\max}$ ” to “fatigue limit of smooth material, σ_{w0} ,” $\sigma_{\max} / \sigma_{w0}$, is determined by the stress gradient [9]. Based on Siebel et al.’s concept, the relationship between “maximum stress at the fracture location when a corroded rail fractures after N loading cycles, $\sigma_{\max}$ ” and “strength at finite life of a new rail at N loading cycles, σ_{t0} ” is expressed by (3).

$$q = \frac{\sigma_{\max}}{\sigma_{t0}} = \frac{K_t \sigma_t}{\sigma_{t0}} = \frac{K_t}{K_f} \quad (3)$$

K_t is the stress concentration factor, K_f is the notch factor, and σ_t is the strength at finite life of a corroded rail. Additionally, “new rail” refers to a rail that has been manufactured but has not been installed on the track and still has its mill scale.

In other words, the coefficient q in (3) is a factor to correct the apparent strength at finite life of the corroded rail using the strength at finite life of a new rail, hereafter referred to as the “finite-life strength correction factor.” This is equivalent to the ratio of the maximum stress at the stress concentration point, $\sigma_{\max}$, to the fatigue limit of the smooth material, σ_{w0} , as given by Siebel et al. However, this relationship does not hold when the notch is extremely small or shallow, $\sqrt{\text{area}} < 1 \text{ mm}$, as the dimensions of the notch and the di-

mensions of the area related to the fracture at the notch bottom become comparable [10]. Here, we call the boundary at which a notch can be considered a crack a “transition point” [11]. If the notch radius at the transition point is ρ_0 , depending on the value of ρ_0 , there are cases where the fatigue limit is determined by the fatigue strength, and cases where a non-propagating crack occurs, and the crack strength determines the fatigue limit. The notch radius at the transition point is almost determined solely by the material. Although the ρ_0 for rail steel is unknown, the ρ_0 for carbon steel with similar tensile strength is approximately 0.1 to 0.25 mm [12]. In this study, the notch radius at the fracture initiation point was derived, as shown in Table 5. To derive the notch radius at the fracture point, a quadratic curve matching the cross-sectional shape in the yz plane was used, including the fracture initiation point. For samples No.6, 8, 11 and 12, the irregularities near the fracture origin were small, and the notch radius was judged to be less than 1 mm. For the other rails, the notch radius was sufficiently larger than that of carbon steel, allowing the proposed method using maximum stress and stress gradient to be applied.

Figure 10 shows the relationship between the finite-life strength correction factor q and the stress gradient χ for the 15 rails that fractured within the shape measurement range, with fracture initiation points at the bottom of the rail, and with notch radii of 1 mm or more. The dashed line represents the regression line with an intercept of 1. The correlation coefficient R at this time was 0.79, indicating a high positive correlation between q and χ .

Using the relationship between q and χ , we verify whether it is possible to predict the number of loading cycles N at the time of fracture for corroded rails. Assuming that the fracture point, the stress at the fracture point $\sigma_{\max}$ and the stress gradient χ are known for the test rails, the strength at finite life of new rails σ_{i0} can be calculated using the finite-life strength correction factor q . Using

Table 5 Notch radius values that are fracture initiation points [3]

No.	Notch radius [mm]	No.	Notch radius [mm]
1	2.34	14	1.64
3	6.10	16	1.53
4	2.01	17	4.61
6	Less than 1mm	19	3.44
8	Less than 1mm	20	6.09
9	2.94	21	2.27
10	3.51	22	10.77
11	Less than 1mm	23	1.43
12	Less than 1mm	24	2.93
13	2.34		

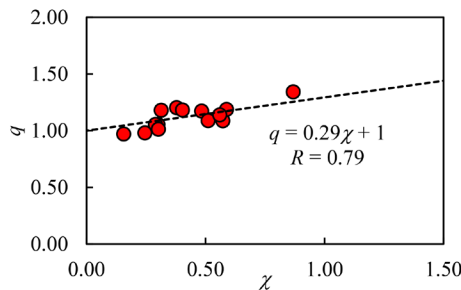


Fig. 10 Relationships between stress gradient and finite-life strength correction factor

this strength at finite life σ_{i0} and the S-N curve of new rails, the estimated number of loading cycles at the time of fracture for the corroded rail N_E was calculated. The specific procedure for estimating the fatigue test results is described below:

- Obtain the shape of the corroded rail and perform a FEM analysis to calculate the y -direction stress $\sigma_{\max}$ and the stress gradient χ at the fracture initiation point.
- Using the relationship between the finite-life strength correction factor q and the stress gradient χ obtained from Fig. 10, calculate the finite-life strength correction factor q from the stress gradient χ at the fracture location.
- Determine the stress fluctuation range σ_A , which is the difference in stress in the y -direction between the maximum and minimum loads at the fracture initiation point.
- Using (3), calculate the strength at finite life σ_{i0} of the corroded rail, by treating it as a new rail and dividing the stress fluctuation range σ_A by q .
- Using the S-N curve of the new rail, calculate the estimated number of loading cycles at the time of fracture N_E for σ_{i0} . The S-N curve of the new rail was calculated using the least-squares method based on previous bending fatigue test results [13], as shown in (4).

$$S = -167 \log_{10} N + 1362 \quad (4)$$

Figure 11 shows a comparison between the estimated number of loading cycles at the time of fracture N_E and the actual number of loading cycles N for the 15 rails used to derive the relationship between the finite-life strength correction factor q and the stress gradient χ . The dashed lines in the figure indicate the range where the ratio of the predicted value to the experimental value is between 0.5 and 2. The estimated number of loading cycles N_E , in this study was generally plotted within the range of the dashed lines. Therefore, by evaluating the stress $\sigma_{\max}$ and the stress gradient χ at the fracture location using the relationship between the finite-life strength correction factor q and the stress gradient χ , it is possible to estimate the number of loading cycles at which fracture may occur in the bending fatigue tests for various shapes caused by corrosion, provided that the fracture initiates at the bottom of the rail and that the notch radii are 1 mm or more. Consequently, this method may also be used to estimate the lifespan of rails installed on real tracks. Specifically, if the rail bottom shape can be obtained by removing or lifting the corroded rail, it may be possible to estimate the fatigue life of the rail without conducting fatigue tests.

At present, there is no practical method available for precisely evaluating the entire bottom of rails on tracks of railway lines in operation. To apply this method to rails in operation, further investi-

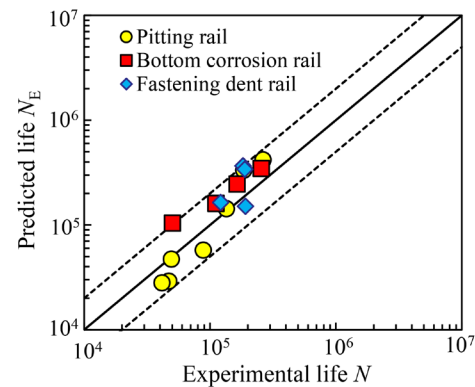


Fig. 11 Comparison of predicted and experimental number of fracturing

gation, including inspection methods, is necessary. Additionally, corrosion may continue to develop while fatigue cracks propagate in laid rails. This aspect cannot be considered in the current fatigue tests. Therefore, a future challenge is to conduct fatigue tests on full-scale rails that also take corrosion progression into account.

5. Conclusions

A method for evaluating rail fatigue strength was investigated by focusing on the stress and stress gradient that occur in the vicinity of corrosion defects when rails are subjected to bending loads. It was confirmed that the proposed method could predict the number of load cycles required for corroded rails to fracture in bending fatigue tests within a range of 0.5 to 2 times the number of load cycles.

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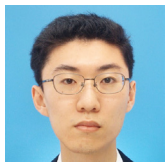
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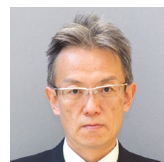
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Study of Infrared Coating Thickness Measurement Method for Repainting Coated Steel Bridges

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Repainting of coated steel bridges is performed manually, and construction management is required to prevent variations in coating thickness. However, the equipment used on site to measure coating thickness only measures it locally, which makes it difficult to measure a large number of points over a wide area. The purpose of this paper is to determine the applicability of a coating thickness measurement method using an infrared camera. To this end, we report the results of the evaluation of the optical properties of coatings applied to steel plates with various surface profiles and the results of a study on the material composition of coatings suitable for this coating thickness measurement.

Key words: coating, measurement, infrared, coating thickness, coating durability

1. Introduction

When steel bridges are in service for extended periods, coatings are applied to each component to provide long-term corrosion protection. The corrosion resistance of these coatings is significantly influenced by their thickness [1]. On the other hand, repainting coatings that have deteriorated over time is usually done manually, making it prone to variations in coating thickness. The primary instruments used to measure coating thickness are wet film gauges and electromagnetic coating thickness gauges (Fig. 1) [2]. However, these instruments measure coating thickness only at the point of contact, which makes it difficult to accurately assess coating thickness over a wide area. Therefore, there is a need to develop a method for quantitatively measuring coating thickness over a wide area during repainting.

Against this backdrop, the authors focused on a method of measuring coating thickness using infrared radiation. This method uses images captured by an infrared camera to estimate coating thickness over a wide area. Previous studies have reported measurement examples in the near-infrared wavelength range (approximately 700–2,500 nm), targeting the outermost coating layer of painted steel road bridges [3]. However, these studies aimed to assess the degree of coating degradation caused by ultraviolet radiation and not to measure coating thickness at the time of repainting.

In this paper, with the aim of assessing the applicability of coating thickness measurement methods using infrared cameras, we report on the results of evaluating the optical properties of coatings applied to various types of steel plates with different surface conditions. We also present the results of a study on the material composition of paints suitable for infrared coating thickness measurement [4].

2. Setting the target for infrared coating thickness measurement

2.1 Setting the target coatings

Painted steel bridges typically use a multi-coat coating system. Within this, the primer applied to the steel plate provides corrosion resistance. The first coat, in particular, must exhibit long-term adhesion to the steel plate and long-term durability as a material. Furthermore, when repainting painted steel bridges, it is common for the same worker to apply all layers up to the final coat. To detect application defects early, it is desirable to know the coating thickness of



Fig. 1 Measurement of coating thickness using an electromagnetic coating thickness gauge

the first-layer coating. Therefore, this paper selected epoxy resin paint, commonly used as the first layer in repainting steel bridges, as the target coating for evaluation.

2.2 Measurement principle of coating thickness using infrared

Figure 2 shows a schematic diagram of the energy behavior of light that is irradiated onto a coating film applied to a steel plate. The energy radiated from the coating surface is the sum of the component E_1 , which reflects at the surface without penetrating the coating interior, and the component E_2 , which penetrates the coating interior, reflects at the steel plate surface, and is then radiated again from the coating surface, as shown in equations (1) and (2). Here, the light energy incident on the coating surface from outside is represented by E , the reflectance and transmittance of the coating are represented by ρ_p and τ_p , respectively, and the reflectance of the steel plate is represented by ρ_m .

$$E_1 = \rho_p E \quad (1)$$

$$E_2 = \tau_p [\rho_m (\tau_p E)] \quad (2)$$

E_1 may vary depending on the surface condition of the coating. E_2 may vary with coating thickness according to Lambert-Beer's law [5]. Furthermore, it may also vary depending on the surface condition of the steel plate.

3. Evaluation of material optical properties using a spectrophotometer

This chapter evaluates the optical properties of epoxy resin paint at various coating thicknesses using a spectrophotometer. It

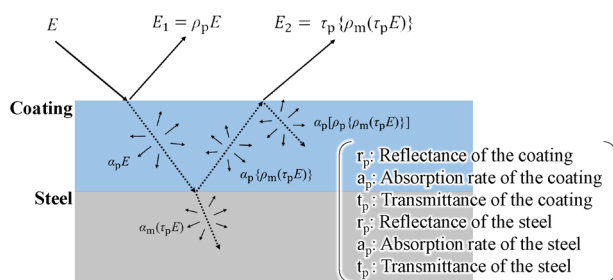


Fig. 2 Image of the behavior of light incident on the coating

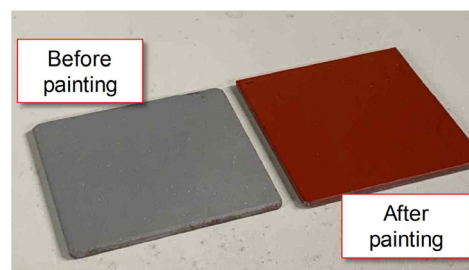


Fig. 3 Appearance of the coated test specimen

also evaluates the optical properties of epoxy resin paint on steel plates with different surface conditions.

3.1 Evaluation of optical properties of epoxy resin paint

3.1.1 Test method

The epoxy resin paints used were the reference paint specified in “Manual of Painting-Design and Application for Steel Structure” [2], and commercially available epoxy resin paints from three paint manufacturers (referred to as General-purpose paint A to C). The formulation of the reference paints is shown in Table 1. All General-purpose paints satisfy Type C No. 1 as specified in JIS K 5551. The corrosion resistance of these coatings is comparable. The color of all paints was set to rust brown.

Two types of test specimens were prepared using the aforementioned paints: film-type specimens and specimens coated on steel plates (referred to as coated test specimens). The film-type specimens were prepared by air-spraying each paint onto polyethylene plates, allowing them to dry, and then peeling them off. The coated test specimens were prepared by coating blasted steel plates with a surface roughness of approximately 30 μm (10-point average roughness) as specified in JIS B 0601 (Fig. 3). Since the target dry coating thickness for each paint is approximately 60 μm , this test targeted thicknesses of 20, 40, 60, and 80 μm .

The transmittance and reflectance of each test specimen were measured using a spectrophotometer. Here, transmittance is defined as the ratio of transmitted light intensity to incident light intensity, and reflectance is defined as the ratio of reflected light intensity to incident light intensity. The measured wavelength range was 200–2,500 nm.

Table 1 Formulation of reference paint [2]

Material			Blending ratio
Principal agent	Resin	Epoxy resin	30.0
		Modified resin	5.0
	Pigment	Anticorrosive pigment	5.0
		Extender pigment	30.0
		Color pigment	10.0
	Additive agent	Dispersing agent	1.0
		Antifoam agent	0.1
		Silane coupling agent	1.0
	Solvent	Xylene	9.9
		Methyl isobutyl ketone	9.0
Curing agent		Solvent-borne polyamidoamine	32.0

3.1.2 Test results and discussion

The evaluation of the optical properties of each paint revealed similar trends across all paints. Figure 4 shows the transmittance and reflectance of the film-type specimen using the reference paint, and the reflectance of the coated test specimens. The transmittance of the film-type specimen tended to decrease across all measured wavelengths as the coating thickness increased. This is thought to occur because the coating film is colored by pigments, which makes light transmission more difficult as the coating thickness increases. Conversely, reflectance tended to decrease as the coating thickness decreased in the wavelength range of approximately 1,000 nm and above. One reason for this is the dispersion state of the coloring pigments within the paint. Figure 5 shows the appearance of the film-type specimen when illuminated from the back with an LED light. At a coating thickness of 20 μm , a mottled distribution of red hue was observed, and even at a thickness of 40 μm , variation in hue was evident.

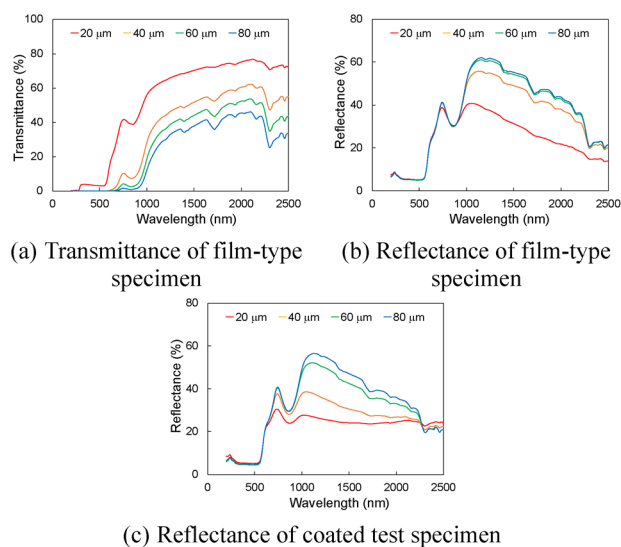


Fig. 4 Transmittance or reflectivity with respect to the thickness of the reference paint

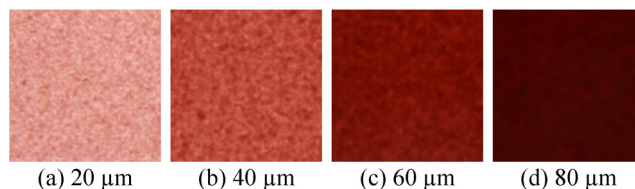


Fig. 5 Appearance of the film-type specimen when LED irradiation is applied from the back side

Thus, it is presumed that the colored pigment particles in the reference paint are not uniformly dispersed within the coating film. Consequently, it is presumed that at smaller coating thicknesses, light passes through more easily, resulting in a decrease in reflectance.

The reflectance of the coated test specimens showed results similar to those of the film-type specimens. One contributing factor is the rough surface of the blasted steel plate, which causes light transmitted through the coating film to scatter on the steel plate surface, resulting in minimal reflection. Thus, since there exists a wavelength range where reflectance varies with coating thickness, it is possible to measure coating thickness by measuring the intensity within that wavelength range.

Figure 6 shows the relationship between coating thickness and reflectance or transmittance for various coatings at a wavelength of 1,100 nm, where the effect of coating thickness was significant. Since the mechanism linking coating thickness to optical properties is not fully understood, the analysis yielding the strongest correlation for each measurement result was selected. Linear regression was used for transmittance with the vertical axis on a logarithmic scale; and regression using a quadratic function was employed for reflectance. The results showed strong correlations in all cases, with transmittance decreasing and reflectance increasing as coating thickness increased. Therefore, it was confirmed that coating thickness can be quantitatively calculated from either reflectance or transmittance within the measured coating thickness range. According to Reference [3], the brightness of images captured by infrared cameras varies with the intensity of reflected light. Therefore, using a paint formulation that exhibits significant changes in reflectance or transmittance with coating thickness is expected to enable high-precision measurement of coating thickness via infrared imaging.

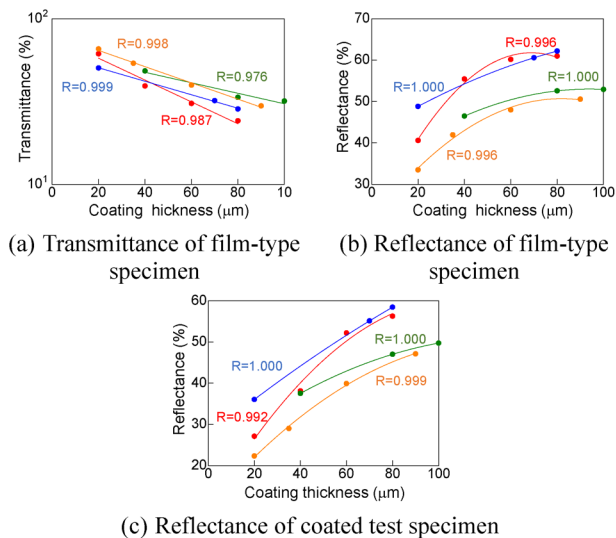


Fig. 6 Relationship between the thickness and transmittance or reflectance at a wavelength of 1,100 nm

3.2 Evaluation of optical properties of steel plate surfaces

3.2.1 Test method

This test used hot-rolled steel plates with mill scale (referred to as hot-rolled steel plates) and bright steel plates in addition to the blasted steel plates as discussed in the previous section. This is because hot-rolled steel plates were used in older painted steel bridges,

and because metal-luster surfaces partially form during substrate preparation when repainting. The hot-rolled steel plate used hot-rolled steel plate conforming to JIS G 3101, while the polished steel plate used bright steel plate conforming to JIS G 3141. The coating method was the same as in the previous section. The reflectance of each coated test specimen was measured using a spectrophotometer. Measurements were also taken on the uncoated steel plate surface.

3.2.2 Test results and discussion

The reflectance of each steel plate is shown in Fig. 7. The reflectance was highest for the polished steel plate, followed by the blasted steel plate, and lowest for the hot-rolled steel plate. Factors contributing to this include scattering due to surface roughness for the blast steel plate and scattering and absorption by the surface oxide layer for the hot-rolled steel plate.

Figure 8 shows the reflectance measurement results for each coated test specimen. For the coated test specimens on polished steel plate, reflectance decreased with increasing coating thickness in the wavelength range above approximately 1,700 nm, showing a trend similar to the transmittance measurement results for the film-type specimens in the previous section (Fig. 4(a)) (Fig. 8(a)). However, in the 1,000–1,700 nm wavelength range, reflectance increased with increasing coating thickness, showing a trend similar to the reflectance measurement results for the film-type specimens in the previous section (Fig. 4(c)). Thus, one factor contributing to the tendency for the reflectance of the coated test specimens to vary by wavelength range is the absorption characteristics of the resin. In the wavelength range above approximately 1,700 nm, there are numerous absorption peaks believed to originate from epoxy resins and pigments in the paint. Therefore, from Fig. 2, it is estimated that light penetrating the coating film loses energy as it progresses, and the effect of E_2 , which depends on the optical path difference (i.e., coating thickness) of the light penetrating the coating film, becomes significant. Furthermore, as shown in Fig. 7, the reflectance of the polished steel plate surface is higher compared to other steel plates. Therefore, the reflectance is considered similar to the transmittance of the film-type specimens discussed in the previous section. Consequently, the reflectance may decrease with increasing coating thickness.

For the coated test specimens on hot-rolled steel plates and blasted steel plates, a trend similar to the reflectance of the film-type specimen (Fig. 4(b)) was confirmed across the entire wavelength range (Fig. 8(b) and 8(c)). This can be attributed to the reflective characteristics of hot-rolled steel plates and blasted steel plates. As shown in Fig. 8, the reflectance of each steel plate is lower than that of the polished steel plate. Therefore, the influence of E_1 was greater than that of E_2 , and it is presumed that the results resembled the reflectance of the film-type specimens.

From the above results, it was found that measuring the coating thickness of conventional coatings applied to steel plates using reflectance is possible for hot-rolled steel plates and blasted steel plates. For polished steel plates, however, two wavelength ranges exist where the correlation between coating thickness and reflectance reverses. Therefore, this test could not determine the feasibility of thickness measurement for polished steel plates, and this remains a topic for future investigation.

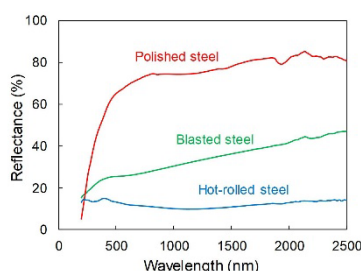


Fig. 7 Reflectivity of various types of steel plates with different surface conditions

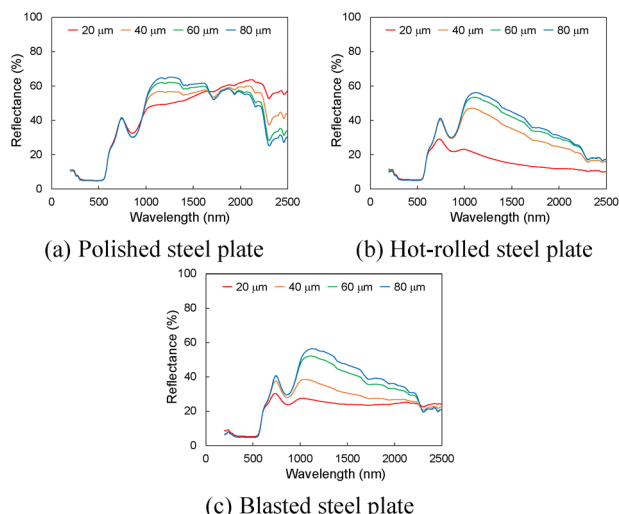


Fig. 8 Reflectance when painted on various steel plates with different surface conditions

4. Evaluation of the influence of color pigments in coatings on film properties

This chapter evaluates the optical properties of epoxy resin paint with different color pigment content ratios to understand the relationship between pigment concentration and the optical characteristics of the paint film. Additionally, the long-term durability of coatings with altered pigment ratios was assessed through an environmental barrier performance and an accelerated aging test.

4.1 Evaluation of coating film optical properties using infrared spectroscopy

4.1.1 Test method

Seven reference paints A to G were used, with the coloring pigment content adjusted within the range of 0 to 40%. To minimize changes in coating workability, the total weight of extender pigments and coloring pigments was kept constant for each reference paint. The compositions of each reference paint are shown in Table 2. The standard formulation corresponds to reference paint D. Using these, coated test specimens were prepared following the same procedure as in the previous chapter. Target coating thicknesses were 20, 40, 60, and 80 μm . The reflectance of the coated test specimens was measured using a spectrophotometer, and infrared camera imaging was also performed. The infrared camera used had a measurable wavelength range of approximately 950–1,700 nm.

The images captured by this infrared camera are represented with 256 levels of brightness corresponding to the intensity of light energy. For imaging, a halogen lamp with luminous intensity in the infrared region was used as the light source (Fig. 9).

Table 2 Formulation of reference paints with different pigment blending ratios

Material			Blending ratio							
			A	B	C	D	E	F	G	
Principal agent	Resin	Epoxy resin	30.0							
		Modified resin	5.0							
	Pigment	Anticorrosive pigment	5.0							
		Extender pigment	40	37	34	30	20	10	0	
		Color pigment	0	3	6	10	20	30	40	
	Additive agent	Dispersing agent	1.0							
		Antifoam agent	0.1							
		Silane coupling agent	1.0							
	Solvent	Xylene	9.9							
		Methyl isobutyl ketone	9.0							
Curing agent		Solvent-borne polyamidoamine	32.0							

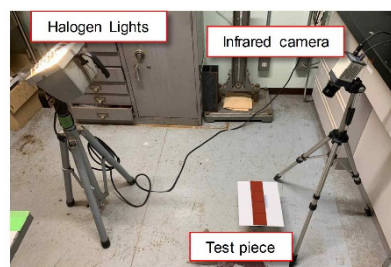


Fig. 9 Shooting situation using the infrared camera

4.1.2 Test results and discussion

Similar to the previous chapter, an analysis of the coated test specimens was performed at specific infrared wavelengths to gain a detailed understanding of the relationship between coating thickness and optical properties. Here, the wavelength region of 1,200 nm was selected because the change in reflectance with respect to coating thickness became significant under the test conditions. A key factor differing from the 1,100 nm wavelength selected in the previous chapter was the change in the formulation of the coating used in this chapter. The relationship between coating thickness and reflectance is shown in Fig. 10. These results indicate a strong correlation between coating thickness and reflectance in all cases, suggesting the possibility of quantitatively calculating coating thickness from reflectance within the measured coating thickness range.

From the curves in Fig. 10, the difference in reflectance between the reference paint at coating thicknesses of 80 μm and 40 μm was calculated (Table 3). Reference paint D showed the largest difference, suggesting that coating thickness can be measured with high precision for coatings formed at the target thickness ± 20 μm .

Figure 11 shows the relationship between the brightness of the

infrared camera image and the coating thickness at the center of each coated test specimen (approximately 40×40 mm). The results shown in Fig. 11 differ from the spectrophotometer measurements shown in Fig. 10. Reference paints E, F, and G show curves that are closer to the reference curve. However, the small change in brightness at large coating thickness suggests that high-precision coating thickness measurement may not be possible. One factor contributing to this difference is that the energy distribution in the infrared region differs between the light source used for spectrophotometer measurements and the light source used for infrared camera imaging. Additionally, the wavelength ranges targeted in Figs. 10 and 11 differ.

The difference in reflectance between the 80 μm and 40 μm coating thicknesses for each reference paint was calculated from the curves in Fig. 10 (Table 4). This table also shows that reference paint D exhibited the largest difference.

4.2 Evaluation of coating durability

4.2.1 Test method

Using reference paints A to D, which yielded favorable results

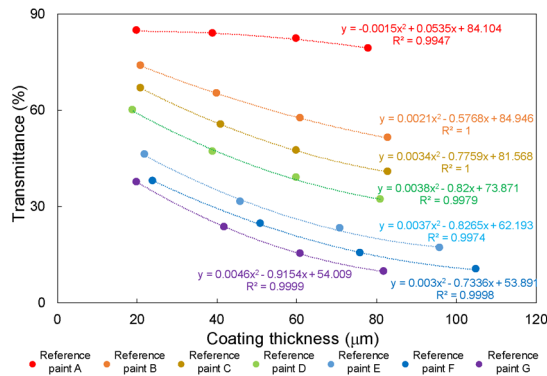


Fig. 10 Relationship between the coating thickness and transmittance at a wavelength of 1,200 nm

Table 3 Difference in reflectance between 80 μm and 40 μm thickness for reference paint A to G in Fig. 10

A	B	C	D	E	F	G
0.4	8.4	12.7	13.9	12.7	11.0	8.6

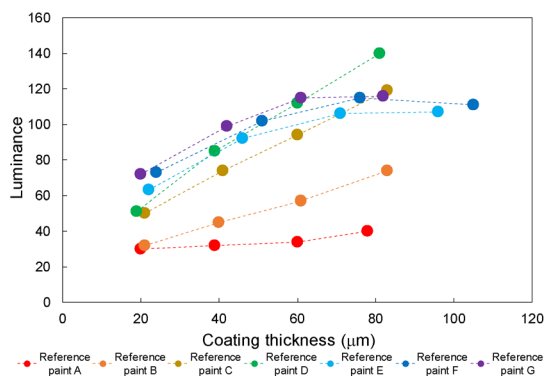


Fig. 11 Relationship between the coating thickness and luminance of coated test specimen

Table 4 Difference in luminance between 80 μm and 40 μm thickness for reference paint A to G in Fig. 11

A	B	C	D	E	F	G
9.0	26.0	43.0	50.0	21.0	23.0	17.0

in the previous section, film-type specimens and coated test specimens were prepared following the same procedure as in the previous chapter. Blasted steel plates measuring 150 × 70 × 3.2 mm were used for this.

The durability of the coatings was evaluated using the water vapor transmission coefficient [6] of the film specimens and the results of the composite cycle test specified in “Manual of Painting-Design and Application for Steel Structures” [2]. For the composite cycle test, coating specimens with one, two and four coats were tested, with each coat having a thickness of 60 μm. Scratches were made on the specimens to evaluate corrosion resistance in the event of coating damage.

4.2.2 Test results and discussion

The water vapor transmission coefficients for each test specimen are shown in Table 5. The water vapor transmission coefficients for reference paints A to D were around one, showing no significant difference.

After conducting the composite cycle test up to ten cycles, equivalent to approximately ten years in a typical environment, no abnormalities were observed in any of the intact specimens. Subsequently, the relationship between the volume of blistering occurring near the scratch and the number of cycles was measured using a laser displacement sensor. The results are shown in Figs. 12 and 13. For a single coat, reference paint C exhibited the smallest blister volume at ten cycles, achieving less than half the blister volume of the reference paint A, which exhibited the largest blister volume. However, this volume difference decreased with an increasing number of coats, and no significant difference was observed with four coats. Since the typical coating thickness of coating systems exceeds 200 μm, the influence of pigment loading on corrosion resistance is considered negligible in practical applications.

The above results indicate that varying the colorant pigment

Table 5 Water vapor transmission coefficient of reference paint A to D

	A	B	C	D
Water vapor transmission coefficient, g · cm/m²/day/mmHg	0.9	0.9	0.9	1.0

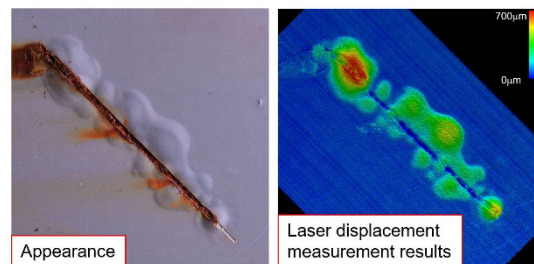


Fig. 12 Appearance and the measurement result with laser displacement meter of scratch area

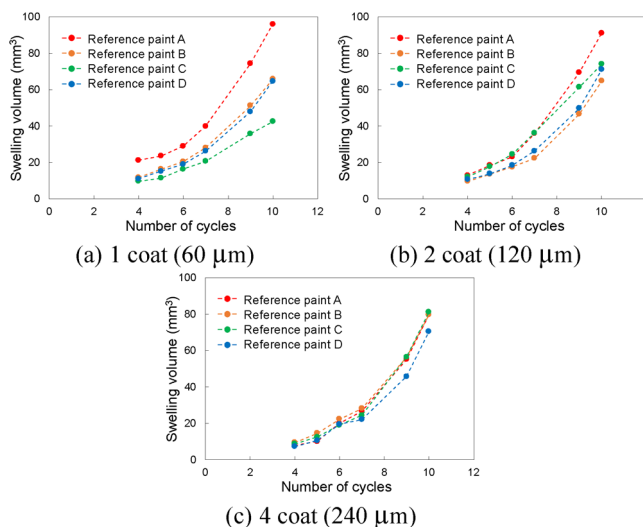


Fig. 13 Relationship between number of cycles and swelling volume

content from 0 and 10% does not significantly affect the long-term durability of the coating film. Therefore, considering the optical property evaluation results of the coating film discussed in the previous section, reference point D is considered the most appropriate formulation from the perspectives of coating durability and infrared coating thickness measurement accuracy.

5. Conclusion

To assess the applicability of an infrared camera-based method for measuring coating thickness during the repainting of coated steel bridges, this study evaluated the optical properties of epoxy resin paint applied to steel plates. It also investigated coating formulations suitable for infrared coating thickness measurement. The following findings were obtained:

- (1) The optical properties of coated steel plates were evaluated using a spectrophotometer. The results show that the reflectance of the coated steel plates correlates with coating thickness, confirming the potential for infrared-based measurement of coating thickness. It is also found that surface conditions, including coloring pigments in the paint and the composition and roughness of the steel plate surface, influence the measurement results.
- (2) Infrared camera imaging of coated steel plates revealed that coating thickness can be measured from the brightness of the cap-

tured images. This is useful when visual inspection makes it difficult to discern differences in coating thickness.

(3) An evaluation of the optical properties of epoxy resin paint with varying amounts of color pigments revealed that an optimal amount of color pigment in the coating exists for accurate measurement of its coating thickness.

(4) Evaluating the impact of varying pigment content on long-term coating durability revealed, through an accelerated aging test using composite cycle testing and water vapor permeability measurements, that for practical coating thicknesses, the ratio of pigment content does not significantly affect the long-term durability of the coating film.

However, as the coating thickness increases, the light transmittance through the coating decreases. This raises concerns that measuring the thickness of a coating during recoating, specifically for the second layer and beyond—i.e., coatings with multiple layers—may be difficult. Therefore, this method is considered most appropriate for assessing the coating skills of painters, focusing on measuring the thickness distribution and unevenness of the first layer. We plan to continue investigating the applicability of this infrared camera-based method of measuring coating thickness, utilizing the findings obtained in this study.

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Indirect Bridge Deflection Estimation Method for Bridges under Resonant Condition Based on Onboard-Measured Track Geometries

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For some high-speed railway bridges, substantial cost and effort are required for in situ measurements during train passage to manage the effects of resonance. This paper presents a method for estimating the deflection of railway bridges in resonance based on onboard measurements of the track geometry. Based on a theoretical analysis that explicitly considers the dynamic response component, the proposed method estimates the deflection from three peak differences in the track geometry between the first and last cars. Numerical simulations and field measurements demonstrated that the proposed method accurately estimates the maximum deflection with an error margin of <0.5 mm.

Key words: onboard measurement, bridge, resonance, high-speed railway

1. Introduction

High-speed trains passing over bridges can induce large vibrations due to resonance between the excitation frequency generated by the regular axle arrangement of the train and the natural frequency of the bridge (i.e., bridge frequency) [1]. The resulting vibrations may lead to crack initiation and propagation in bridge girders [2, 3] as well as adversely affect onboard ride comfort [4]. Resonance has also been reported to induce excessive vibrations in utility poles installed on bridges causing damage to overhead contact line equipment [5, 6]. Thus, extensive efforts have been devoted to understanding resonance and establishing design methods to mitigate this problem [7]. Consequently, bridge resonance has largely been eliminated as a practical problem for modern high-speed railways. However, resonance can still affect ride comfort in the case of passing over older railway bridges in Japan. In particularly severe cases, countermeasures such as train speed reduction or large-scale structural strengthening have been implemented [8]. On such lines, significant time and labor are required to appropriately assess the resonance conditions of railway bridges and determine the need for countermeasures because the deflection of each individual bridge during the passage of a train must be measured in situ [9].

To reduce the work required for this, various methodologies have been proposed for estimating bridge deflections based on onboard measurements of the track geometry [10, 11]. The maximum difference between two track geometries obtained by a running train has been theoretically derived to be proportional to the maximum bridge deflection measured in situ, and this difference can be used to estimate the bridge deflection [12]. However, this method is based on static theory, so it cannot be applied to bridges in resonance for which the dynamic response is dominant during train passage [12]. A new methodology to estimate the deflection of railway bridges in resonance is required that explicitly accounts for the dynamic response.

The aim of this study was to develop a method for estimating the deflection of railway bridges in resonance based on onboard measurements of the track geometry. As in previous studies [12, 13], the difference between track geometries obtained at the leading and trailing cars was used. This approach eliminates track geometry components other than bridge deformation such as rail irregularities, which are common to both track geometries. First, the relationship between the resonant condition of a bridge and peak displacements of the track geometry was clarified through a theoretical

analysis of the dynamic response of a railway bridge [14]. Based on this theoretical analysis, a method for estimating railway bridge deflection in resonance was developed using two parameters: bridge frequency and the modal damping ratio [14].

2. Theoretical Analysis

The theoretical analysis considered the case in which the displacement of a railway bridge in resonance was observed as differences in the track geometry, which were measured by a track inspection device installed at an arbitrary position on a running train. In this study, the term “resonance” refers not only to the typical state of resonance in which the excitation period of a running train coincides with the bridge frequency but also to states of near-resonance in which the two frequencies differ slightly and a beating phenomenon occurs. As shown in Fig. 1, the bridge can be modeled as a simply supported beam characterized by the span length L_b , flexural rigidity EI , mass per unit length $\bar{m}$, and modal damping ratio ξ while the passage of a train can be modeled as a series of moving loads arranged with the same spacing as the train axles (i.e., car length L_c). All moving loads have an identical magnitude P and move from left to right at a constant speed v . When transformed into a modal coor-

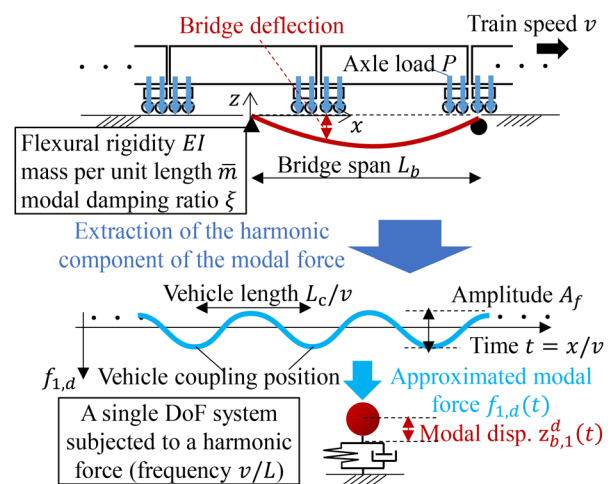


Fig. 1 Theoretical and transfer function model of first bending mode for a railway bridge

dinate system, the modal displacement of the first bending mode $z_{b,1}$ is expressed by the mass-normalized modal force f_1 , natural circular frequency of the beam $\omega_{b,1}$, and modal damping ratio $\xi_{b,1}$:

$$\frac{\partial^2 z_{b,1}}{\partial t^2} + 2\xi_{b,1}\omega_{b,1}\frac{\partial z_{b,1}}{\partial t} + \omega_{b,1}^2 z_{b,1} = f_1 \quad (1)$$

Here, the modal force f_1 can be decomposed into a harmonic component with a period corresponding to the car length and a steady component:

$$f_1 = f_{1,d} + f_{1,s} \quad (2)$$

The harmonic component is expressed as $f_{1,d} = A_f \exp(i\omega_f t)$, where A_f is the amplitude of the harmonic component $f_{1,d}$, ω_f is the circular frequency of its variation, and i is the imaginary unit. Because $f_{1,d}$ attains a minimum each time the car coupling position (i.e., where axles are densely arranged) passes the midspan of the bridge (i.e., one cycle per car length), the circular frequency is given by $\omega_f = 2\pi v/L_c$, where L_c is the car length and v is the train speed [15]. The displacement component $z_{b,1}^d$ corresponds to the harmonic component $f_{1,d}$ in the modal displacement $z_{b,1}$ of (1). If the bridge response is assumed to be in a steady state, then the displacement response of the beam to the harmonic component $f_{1,d}$ can be assumed to take the form $B_f \exp(i\omega_f t)$. Consequently, the following relationship is obtained:

$$B_f \left\{ \omega_{b,1}^2 - \left(\frac{2\pi v}{L_c} \right)^2 + 2i\xi_{b,1}\omega_{b,1} \frac{2\pi v}{L_c} \right\} = A_f \quad (3)$$

The transfer function G , which is defined as the ratio of the dynamic response amplitude B_f to the quasi-static displacement amplitude $A_f/\omega_{b,1}^2$ under the action of the harmonic component of the modal force, is given by $G = \omega_{b,1}^2 B_f / A_f$. Because the excitation period is a function of the train speed v , the transfer function can be expressed as a function of v , as given in (4) and (5) [14]:

$$|G(v)| = \left(\sqrt{1 - (\beta v)^2} + (2\xi_{b,1}^2 v^2) \right)^{-1} \quad (4)$$

$$\angle G(v) = \tan^{-1} \left(\frac{2\xi_{b,1} v}{1 - (\beta v)^2} \right) \quad (5)$$

Here, $\beta = 2\pi/(\omega_{b,1} L_c)$. Converting the phase lag in (5) into a spatial lag by using the excitation circular frequency ω_f obtains:

$$\angle G(v) = \frac{L_c}{2\pi} \tan^{-1} \left(\frac{2\xi_{b,1} v}{1 - (\beta v)^2} \right) \quad (6)$$

If the displacement component $z_{b,1}^d$ corresponding to the harmonic component $f_{1,d}$ is transformed back into the physical coordinate system, then $\omega_f = 2\pi v/L_c$ can be used with the relation $vt = x$ to express the bridge dynamic displacement $z_b^d(x)$ as follows assuming $t = 0$ at the instant the car coupling position enters the beam:

$$z_b^d(x) = B_f \sin(2\pi x/L_c + \theta) \sin(\pi x/L_b) \quad (7)$$

Equation (7) represents the dynamic response of a bridge in resonance as observed by the car coupling position traveling over the bridge. The dynamic response can be expressed as the product of one wave with a wavelength equal to the car length and another wave with a half-wavelength equal to the bridge span (i.e., the mode shape of the first bending mode). Here, θ is determined by the phase of the bridge dynamic response to the harmonic excitation $f_{1,d}$ and distance L_i between the location of the track inspection device and the car coupling position:

$$\theta = \frac{2\pi}{L_c} \left(dG(v) - \frac{L_b}{2} - \frac{L_c}{2} + L_i \right) \quad (8)$$

Figure 2 shows the transfer function of the first bending mode of the bridge with respect to the harmonic component of the modal force, which is calculated by using (4) and (6) (car length: 25 m, bridge frequency: 2.5 Hz, and modal damping ratio: 2%). The spatial lag represents the phase lag of the bridge displacement component $z_{b,1}^d$ relative to the harmonic component $f_{1,d}$, which is expressed in terms of the distance traveled by the car. When the spatial lag is zero, the response is in phase, and the bridge displacement component $z_{b,1}^d$ also reaches a minimum (negative peak) when the harmonic force $f_{1,d}$ becomes minimum as the car coupling position is located at midspan. In contrast, when the bridge and train are in perfect resonance ($\omega_f = \omega_{b,1}$), which corresponds to a speed of 225 km/h in Fig. 2, the displacement component $z_{b,1}^d$ lags behind the harmonic component $f_{1,d}$ by $\pi/2$. Accordingly, the spatial lag becomes $L_c/4 = 25/4 = 6.25$ m.

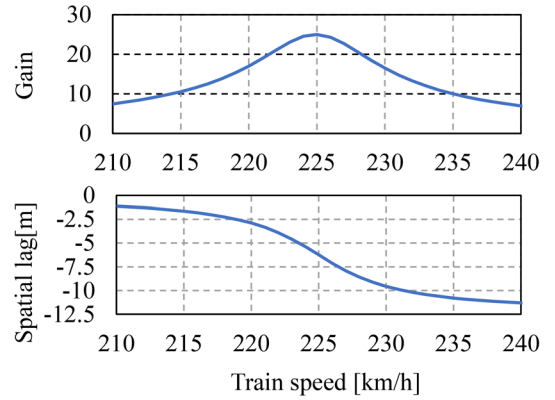


Fig. 2 Transfer function of beam to modal force

Figure 3 presents a numerical example of the bridge dynamic response observed from a train based on (7). The bridge span was set to 30 m, and the train speed was varied within $\pm 5\%$ of the resonant speed. The position of the track inspection device was assumed to be approximately 3 m toward the car center from the center of the trailing bogie, which is consistent with actual installations. In all cases, the response within the bridge section was represented by the product of one wave with a wavelength of 25 m (i.e., car length) and another wave with a half-wavelength of 30 m (i.e., bridge span). The dynamic response of the bridge in resonance was observed by on-board measurements, which exhibited a minimum value (i.e., negative peak) at midspan when the train speed corresponded to the resonant speed. When the train speed was lower or higher than the resonant speed, the minimum value shifted toward the entrance side

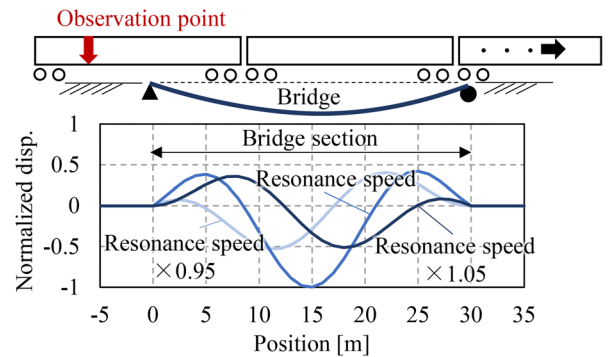


Fig. 3 Theoretical solution for bridge response based on onboard observation

or exit side, respectively, of the bridge because of the influence of the spatial lag. Therefore, the difference between the train speed and resonant speed (i.e., bridge frequency) is reflected in the position (i.e., phase) of the negative peak. The amplitude of the negative peak is primarily governed by the modal damping ratio. Because the dynamic response of a bridge in resonance is controlled by the bridge frequency and modal damping ratio, the amplitude and position of the negative peak observed in the track geometry correspond to these two parameters. However, the phase of the transfer function is also influenced by the modal damping ratio, and the peak position is additionally affected by positional synchronization errors in track geometry measurements. Consequently, it is difficult to uniquely determine the bridge frequency and modal damping ratio solely from the amplitude and position of the peak. The proposed method for estimating bridge deflection was developed to address these limitations.

3. Proposed Method

3.1 Observed and Calculated Track Geometry

For the observed data, this study assumed the use of 20-m mid-chord offset track geometry, which is commonly employed in practical track maintenance in Japan. The difference between the 20-m mid-chord offset track geometries observed at the leading and trailing cars (hereafter referred to simply as the track geometry difference) was taken to eliminate static track geometry components and quasi-static bridge deflection components and to extract dynamic response components that gradually increase during train passage due to resonance of the bridge. As the model for inverse analysis, the bridge was represented as a simply supported beam, and the train was represented as a series of moving loads, as given in (1) and (2). The input parameters of the model included the train speed v , number of cars N_w , axle load P , modal mass m_1 of the first bending mode of the bridge, bridge frequency $n_{b,1}$ ($n_{b,1} = \omega_{b,1}/2\pi$), modal damping ratio $\xi_{b,1}$, span length L_b , and number of modes considered in the calculation. The maximum number of modes was set to three to follow the example from previous studies [5]. These parameters were used to obtain the bridge response during train passage through numerical integration of (1). Among the input parameters, the train speed v is measured onboard together with the track geometry. The number of cars N_w , axle load P , span length L_b , and modal mass m_1 can be obtained from specifications or design values. The bridge frequencies of the second and higher modes can be uniquely determined from the first-mode value based on beam theory [5], and the modal damping ratios of the second and higher modes were assumed to be identical to that of the first mode. Accordingly, the only unknown parameters are the bridge frequency $n_{b,1}$ and the modal damping ratio $\xi_{b,1}$. These two unknown parameters are denoted as $\theta = [n_{b,1}, \xi_{b,1}]$.

For comparison with the observed data, it was necessary to compute the track geometry difference from the bridge response obtained by numerical calculation. In this study, a measurement scenario was assumed in which the track geometries were measured at the leading and trailing cars. The bridge response at the installation positions of the inertial midchord offset track inspection devices was first calculated and then converted into a distance series at 0.25-m intervals in the same manner as the measured track geometry. Subsequently, the response was converted into 20-m midchord offset track geometries through the same processing applied to the measured data, and the track geometry difference was obtained by

synchronizing the positions of the leading and trailing cars and taking their difference. The calculated track geometry difference was denoted as dI . This computational procedure involves inputting the unknown parameter θ and outputting the track geometry difference dI :

$$dI = G(\theta) \quad (9)$$

Under these conditions, the problem of estimating the deflection of a bridge in resonance is reduced to determining $\hat{\theta} = [\hat{n}_{b,1}, \hat{\xi}_{b,1}]$, which minimizes the error between the calculated track geometry difference dI and observed track geometry difference.

3.2 Definition of Evaluation Points

Next, evaluation points were defined to estimate the above unknown parameters from the track geometry difference. When the difference between the measured and calculated track geometry difference waveforms is evaluated directly, a high level of positional alignment accuracy is required. With insufficient accuracy, errors will theoretically increase in regions where the slope of the track geometry difference waveform is large. In contrast, the bridge frequency and modal damping ratio are characterized by the amplitude and position of the negative peak (i.e., slope of zero) in the track geometry difference. Based on this, it is possible to exclude regions with large waveform gradients from the estimation and to use only the negative peak of the track geometry difference. The amplitude and position of the negative peak correspond to the bridge frequency and modal damping ratio. However, if the positional alignment accuracy is insufficient, the estimation error of the negative-peak position also increases. To address this issue, the estimation of the negative peak position is replaced by estimating the amplitudes of other peaks. Specifically, in addition to the negative peak near midspan, the amplitudes of two adjacent positive peaks are used as evaluation points: one before and one after the negative peak. This yields a total of three evaluation points.

Figure 4 shows the track geometry difference observed on a bridge in resonance and at the three evaluation points calculated by using (9). The relative magnitudes of the two positive-peak amplitudes before and after the negative peak vary depending on the difference between the resonant speed and train speed. This behavior is caused not only by the spatial lag associated with deviation from the resonant speed but also by the fact that the waveform of the track geometry difference within the bridge section is influenced by the bridge mode shape, which reaches a maximum near midspan and a minimum near the supports [14]. That is, when the negative peak shifts toward the exit side of the bridge (e.g., at a train speed of 1.05 times the resonant speed), the first positive peak moves toward midspan and the second positive peak moves toward the exit side, which results in a larger amplitude for the first positive peak and a smaller

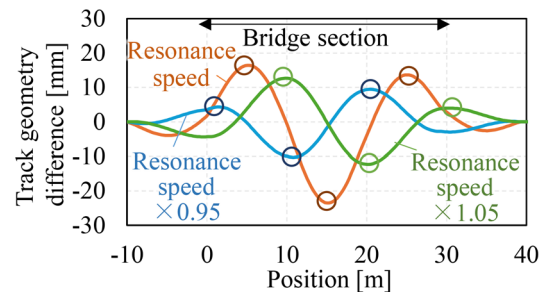


Fig. 4 Characteristics of the three peaks in track geometry difference

amplitude for the second. By incorporating the amplitudes of the two positive peaks into the estimation, the shift in position of the negative peak can be evaluated in an alternative manner. Under these conditions, the bridge frequency and modal damping ratio can be estimated by using the amplitudes of the three peaks in the track geometry difference as evaluation indices to minimize the error between the observed and calculated values.

3.3 Numerical Verification

A numerical simulation was conducted to verify the proposed method. The moving-load series was assumed to represent a 12-car Shinkansen train, and the train speed was set to 260 km/h for all cases. The bridge span was set to 25 m with a mass per unit length of 25 t/m and modal damping ratio of 2.5%. The bridge frequency was set to 2.9 Hz so that the train running speed (i.e., 260 km/h) corresponded to the resonant speed. The track geometry difference obtained under these conditions was used together with the train speed, bridge span length, and mass per unit length as inputs to estimate the bridge frequency and modal damping ratio. Then, the track geometry difference waveform and maximum bridge deflection were estimated. The Markov chain Monte Carlo (MCMC) method, which is a type of Bayesian estimation technique, was employed [15]. Details on the estimation procedure are given in [14]. By setting the estimated bridge frequency and modal damping ratio as parameters of the beam model and performing a moving-load simulation, the bridge deflection was subsequently estimated.

The upper part of Fig. 5 shows the estimation process for the track geometry difference. Although the estimated track geometry difference at the 10th iteration deviated from the true value, repeated estimation yielded an estimate at the 100th iteration that agreed well with the true value. In addition, even though only the three evaluation points were used, not only the peak amplitudes but also the peak positions were updated to match the true values. This indicates that the peak position can be estimated in an alternative manner by using the local minimum near the midspan together with the two adjacent local maxima before and after it. The lower part of Fig. 5 shows the estimation process for the bridge deflection estimated via the track geometry difference. Similar to the track geometry difference, the bridge deflection estimated at the 10th sampling deviated substan-

tially from the true value, but by the 100th iteration the estimated bridge deflection waveform matched the true waveform with high accuracy. Therefore, at least under the numerical simulation conditions, the proposed method could be used to estimate the deflection of a bridge in resonance from the track geometry difference by estimating the bridge frequency and modal damping ratio so that the three evaluation points match.

The above verification was performed for bridges with span lengths of 25 and 50 m. In addition to the resonant speed, cases where train speeds were varied within $\pm 5\%$ and $\pm 10\%$ of the resonant speed were also examined. The results confirmed that the bridge frequency could be estimated within an error of 1%, the modal damping ratio within an error of 40%, and the bridge deflection within an error of 10% relative to the true values [14]. It should be noted that the bridge deflection estimation error increased when the train speed was -10% of the resonant speed because of a reduction in the dynamic response when the trailing car was passing caused by a beating phenomenon [14]. Excluding this case, the bridge deflection estimation error was generally within 2%, which confirms that the proposed method provides sufficient accuracy.

4. Validation on an Actual Railway Line

4.1 Validation Conditions

A Japanese high-speed railway line for which resonance during train passage has been reported on several bridges was selected as the target. The onboard measurement of track geometries was carried from the leading and trailing cars of a train, and the proposed method was applied to the track geometry differences obtained for four bridges (A—D). In addition, the bridge deflection during the passage of the same train was simultaneously measured in situ by using a U-Doppler system [16], and the estimation accuracy was verified by comparing the measured maximum bridge deflection with that estimated by the proposed method.

Track geometries were measured by using inertial midchord offset track inspection devices installed on the leading and trailing cars of a 12-car revenue-service train [17] and were converted into 20-m midchord offset track geometry [14]. Bridges A—D all had the same structural configuration: posttensioned simply supported T-girder bridges with four main girders (double-track bridges), a span length of 29.2 m, a bridge length of 30 m, and a girder depth of 1.9 m. A cross-sectional view is shown in Fig. 6. This bridge type is representative of structures for which resonance has become a maintenance concern, and large-scale strengthening measures have been implemented on other bridges of the same type.

Figure 7 shows the train speed and the track geometry differ-

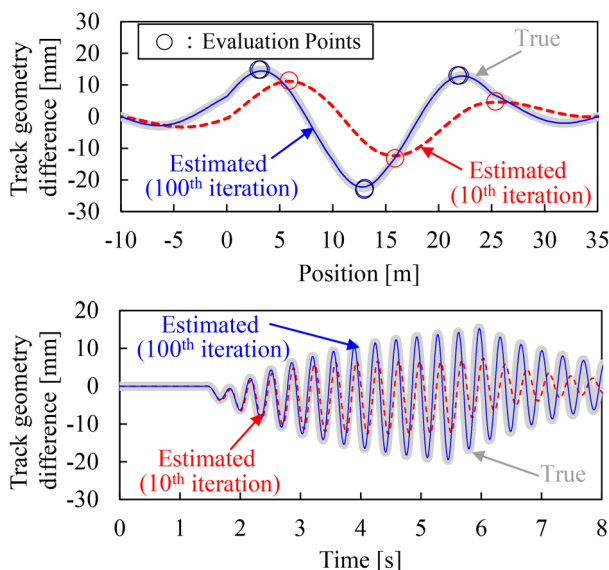


Fig. 5 Results of numerical verification (span length: 25 m)

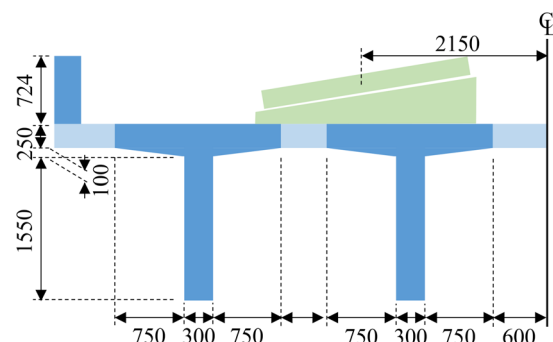


Fig. 6 Bridge cross-sectional view (span length: 29.2 m)

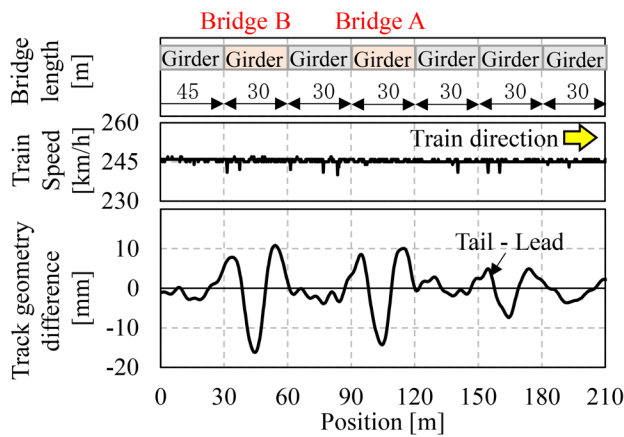


Fig. 7 Onboard measurement data for a railway line section including Bridges A and B

ence between the leading and trailing cars measured over an approximately 200-m section of the line that includes Bridges A and B. The top panel presents structural information including the structure type (girder) and bridge length. The second panel indicates that the train traveled from left to right at a speed of approximately 247 km/h. The bottom panel shows the track geometry difference obtained by subtracting the track geometry of the leading car from that of the trailing car. For Bridges A and B, fluctuations with amplitudes exceeding 20 mm were observed in the track geometry difference. The negative peak near midspan and the positive peaks before and

after it associated with these fluctuations are in good agreement with the results of the theoretical analysis.

4.2 Estimation Results

The proposed method was applied to a 40-m segment of the track geometry difference for each bridge, which included 5 m before and after the bridge. The bridge properties were set based on design calculation values with a span length of 29.3 m and mass per unit length of 24.6 t/m for all bridges. The initial values of the bridge frequency and modal damping ratio were set to 2.9 Hz and 2.0%, respectively. For the MCMC analysis, the total number of samples was set to 5,000. The first 500 samples were discarded as burn-in, and the remaining 4,500 samples were used as samples from the posterior distribution.

Figure 8 shows the evolution of the estimated track geometry difference and bridge deflection for Bridge A during the estimation process together with the measured values. The upper panel presents the evolution of the estimated bridge frequency and modal damping ratio. The middle and lower panels show the estimated track geometry difference and bridge deflection at representative iterations. In the first iteration (Fig. 8a), the estimated track geometry difference had a smaller amplitude than the measured one, and two of the three evaluation points differed substantially from the observed values. As the sampling proceeded, the estimated track geometry difference gradually approached the measured data. By the 500th iteration (Fig. 8c), the negative peak of the estimated track geometry difference was almost identical to the measured value. In accordance with

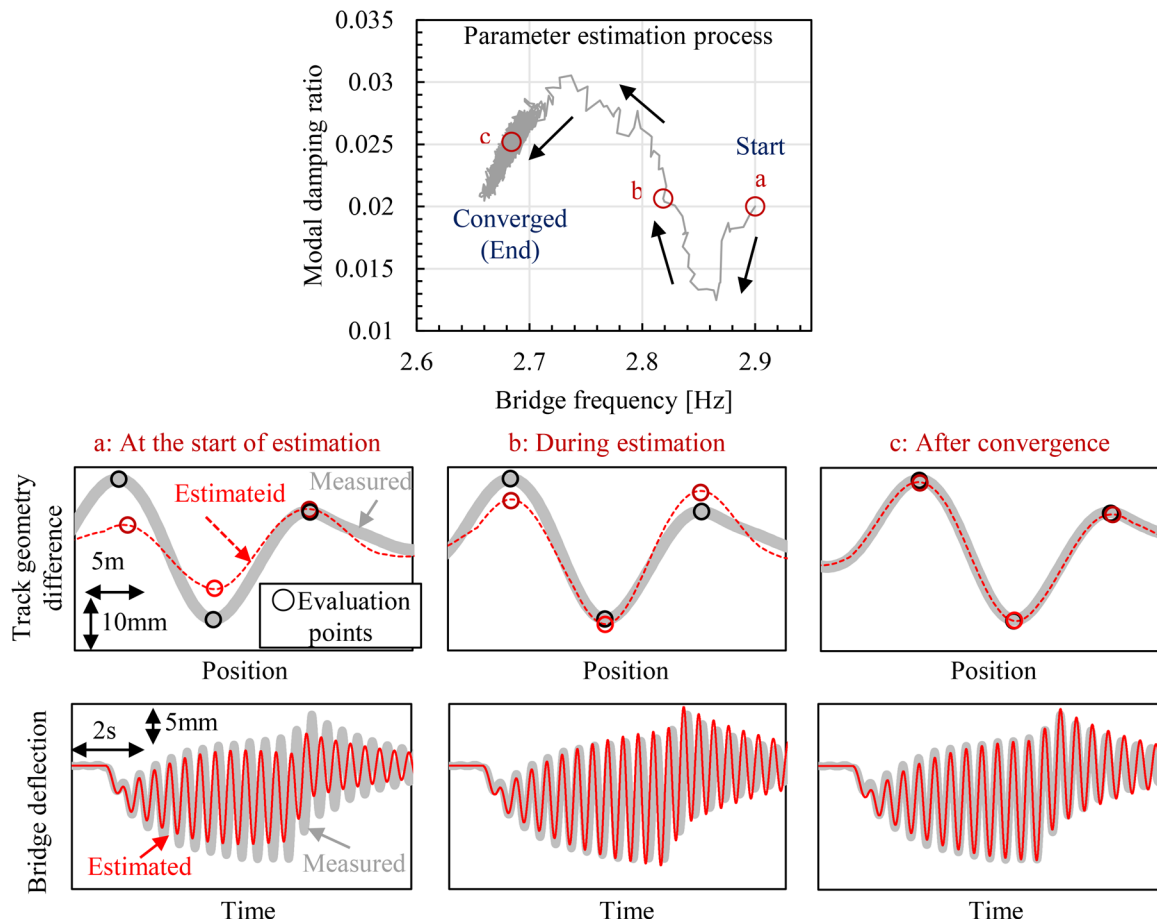


Fig. 8 Track geometry difference and bridge deflection during estimation process (Bridge A, train speed: 247 km/h)

the convergence of the parameters, the estimated bridge deflection waveform also approached the in situ measured waveform. During the updating process of the track geometry difference, only the minimum value matched the measured data at the 50th iteration (Fig. 8b). However, in the bridge deflection waveform, the maximum deflection already agreed well with the measured value by this iteration. From the 50th to 500th iterations, the estimated bridge deflection waveforms showed little variation and consistently matched the measured waveform with high accuracy. These results confirm that, even on an actual railway line, if the track geometry difference can be reproduced accurately, the bridge deflection waveform can also be reproduced with high accuracy. Furthermore, if the minimum value of the track geometry difference obtained from on-board measurements is reproduced well, the maximum value of the bridge deflection waveform will match in situ measurements with reasonable accuracy.

Table 1 summarizes the maximum bridge deflection values estimated by the proposed method for each bridge and the corresponding measured values. The estimated maximum bridge deflections were 15.3 mm for Bridge A (measured: 15.2 mm), 13.8 mm for Bridge B (measured: 14.2 mm), 11.9 mm for Bridge C (measured: 11.4 mm), and 9.4 mm for Bridge D (measured: 9.7 mm). For all bridges, the estimated values agreed with the measured values within approximately 0.5 mm (5%), which indicates that the proposed method can estimate the maximum bridge deflection of bridges in resonance with good accuracy. It should be noted that the estimation error tended to increase slightly as the difference between the train speed and estimated resonant speed increased. This is due to the influence of a beating phenomenon, which occurs when the train and resonant speeds deviate slightly. Nevertheless, even in cases where this beating phenomenon occurred, the estimation error of the maximum bridge deflection remained below 5%. Although not presented in this paper, the bridge frequency and modal damping ratio of each bridge were also obtained during the estimation process for the maximum bridge deflection; for details, refer to [14].

Table 1 Bridge Deflections Estimated and Measured in Situ

Bridge	Estimated onboard [mm]	Measured in situ [mm]	Train speed [km/h]	Resonant speed [km/h]
A	15.3	15.2	247	241.4
B	13.8	14.2	235	241.3
C	11.9	11.4	255	244.0
D	9.4	9.7	245	258.0

5. Conclusions

This study aimed to develop a method for estimating the deflection of bridges in resonance based on onboard measurements of the track geometry, which has strong practical demand but has been difficult to achieve to date. The main findings are summarized as follows:

- A theoretical analysis was conducted where a bridge in resonance with a passing train was modeled as a simply supported beam subjected to a series of moving loads to determine the dynamic response component. The results of the theoretical analysis clarified that the difference between the resonant and

train speeds can be represented by the peak values and phase of the track geometry within the bridge section.

- A method for estimating the maximum bridge deflection was proposed, which involves estimating the bridge frequency and modal damping ratio that match the amplitudes of three peaks in the track geometry difference obtained from the leading and trailing cars.
- Application of the proposed method to an actual high-speed railway demonstrated that it accurately estimated the maximum deflection of bridges in resonance with an error of <0.5 mm relative to in situ measurements.

The proposed method can utilize inertial midchord offset track inspection devices already installed on revenue-service trains for track maintenance [17] as well as structural information stored in the track maintenance management database system LABOCS [18]. Therefore, no additional equipment investment is required for practical implementation. Efforts toward practical application on selected railway lines are currently underway. However, the computational cost of the estimation remains an issue for practical implementation. If dozens of bridges in resonance are detected in a single onboard measurement run, the MCMC-based approach used in this study would require a considerable amount of computation time. Possible solutions include the use of faster estimation methods such as maximum likelihood estimation or the development of evaluation methods based on nomograms for specific bridge types. Because of the limited number of bridge types where deflection is a critical issue, development and verification of such nomograms are currently in progress.

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Evaluation Method for Flange-Climb Derailment Focusing on Contact Position and Transverse Creepage Between Wheel and Rail

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This paper proposes a new method for evaluating the running safety of railway vehicles in the event of flange-climb derailment, focusing on the contact conditions between the wheel and rail. The method uses two key variables: lateral contact position and normalized transverse creepage, defined as the ratio of transverse creepage to the wheelset angle of attack. Through vehicle dynamics simulations under various running conditions, the relationship between these variables and the running safety was investigated. The results show that the transit domains for the loci of these variables differ significantly between derailment and non-derailment cases. This paper provides fundamental insights into the development of advanced safety assessment techniques for railway operations that take detailed wheel/rail contact dynamics into account.

Key words: running safety, wheel/rail contact, flange-climb derailment, creepage

1. Introduction

In recent years, techniques for identifying wheel/rail contact conditions have been extensively investigated. These conditions are difficult to measure directly while railway vehicles are running. For example, methods have been proposed for capturing the following contact-related state quantities:

- Lateral contact position [1-4]
- Wheelset angle of attack [5-9]
- Friction coefficient [10-15]

At present, running safety against flange-climb derailment is typically evaluated by measuring the derailment quotient and comparing it with prescribed reference limits. However, several studies have reported that explicitly accounting for wheel/rail contact conditions could improve the accuracy of these evaluations [16-19]. While these studies proposed methods that still use the derailment quotient as the primary indicator, there is still room to improve the precision of safety assessments by directly considering the relationship between contact conditions and running safety.

This study examines a running-safety evaluation method under the assumption that wheel/rail contact conditions are known. In previous work, another safety evaluation method was proposed which considered the relationship between the lateral displacement and lateral velocity of a wheelset corresponding to derailment-quotient levels at which flange-climb derailment can occur [20]. The study reported that the method provides appropriate evaluations when the vehicle is running on a straight track [20]. However, this previous approach has not yet reached a stage where the safety of negotiating curves can be clearly evaluated, as the influence of the wheelset angle of attack becomes pronounced in this situation. Incorporating the effect of the wheelset angle of attack within the same framework would require considering a four-dimensional evaluation domain, making the evaluation process cumbersome.

In view of these issues, this study presents a methodology and results for an evaluation method that utilizes the transverse creepage normalized by the angle of attack together with the lateral contact position among the contact-condition variables. This paper is an English translation of the essential parts of a previously published article [21], extracted without altering the original line of argument.

2. Consideration of evaluation method for running-safety

2.1 Phases leading up to flange-climb derailment

This paper classifies the process leading to flange-climb derailment into four distinct phases according to the wheel/rail contact position, as illustrated in Fig. 1. Each phase is described below.

• Tread-contact phase

In this phase, the wheel contacts the rail in the area indicated in green in Fig. 1. Contact occurring in the flange-throat region is also included in this phase.

• Wheel-rising phase

In this phase, the wheel contacts the rail in the area indicated in purple in Fig. 1. The contact angle of the wheel reaches its maximum value (the flange angle α_f), and the wheel rise increases rapidly.

• Flange-climb phase

In this phase, the wheel contacts the rail in the area indicated in orange in Fig. 1. As this phase progresses, the contact angle decreases from its maximum value α_f .

• Derailment phase

In this phase, the wheel contacts the rail at the flange-top position indicated by the red point in Fig. 1. The wheel is then fully supported on top of the rail, and if this condition persists, derailment eventually occurs. Because this phase is extremely hazardous, this study treats entry into this phase as representing derailment.

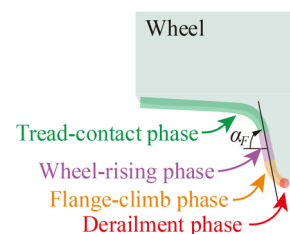


Fig. 1 Four phases leading to flange-climb derailment

2.2 Indicators and corresponding variables

In this study, two types of safety margin are considered as evaluation indicators, as illustrated in Fig. 2. The adequacy of the running-safety evaluation is assessed by examining whether the current running state is at risk of reaching the flange-climb phase from the viewpoint of these two indicators.

- Indicator 1 - Geometric margin (Fig. 2(a))

This indicator represents the geometric margin from the current phase to the derailment phase. When the current contact patch lies in an area close to the wheel flange top, the risk of flange-climb derailment is regarded as high.

- Indicator 2 - Dynamic margin (Fig. 2(b))

This indicator represents the dynamic margin from the current phase to the derailment phase. A large rate of change in the contact patch toward the flange top indicates a high risk of flange-climb derailment.

The coordinate systems and variables around the wheelset and wheel that can represent candidates of these indicators are shown in Fig. 3. The variables are categorized into the following three groups:

- (1) Wheelset motion variables, including the wheelset running speed xV_w , lateral displacement y_w , angle of attack ψ_w , and roll angle ϕ_w (Fig. 3(a)): Larger y_w , ψ_w , and ϕ_w generally indicate that the wheelset is closer to making severe contact with the outer rail.
- (2) Geometric contact variables, including wheel/rail lateral contact position s , wheel rise z , rolling radius r , and contact angle α (Fig. 3(b)): These quantities describe where and how the wheel makes contact with the rail. A contact condition closer to the flange (large s , z , r , and α) corresponds to a higher risk of flange-climb derailment.
- (3) Friction condition, represented by the friction coefficient μ (Fig. 3(b)): This quantity prescribes the maximum creep forces that can be generated.

In this study, the variables corresponding to each indicator are chosen as follows. For Indicator 1 (geometric margin), the lateral contact position s as shown in Fig. 3(b) is adopted. For Indicator 2 (dynamic margin), the lateral contact velocity $\dot{s}$ could in principle be used. A related concept is the method that evaluates running safety

using the domain relationship between the lateral displacement and lateral velocity of the wheelset [20]. However, as discussed in the Introduction, explicitly accounting for the influence of the angle of attack under curve-running conditions inevitably complicates the evaluation. Therefore, instead of $\dot{s}$, a new state variable termed a normalized transverse creepage ξ is defined and employed as an indicator presenting the dynamic margin.

2.3 Definition and characteristics of normalized transverse creepage

A normalized transverse creepage ξ is derived from a transverse creepage v_y . v_y is a quantity calculated from the running speed and the difference in velocity between the contact patch and the wheel and rail. v_y can be expressed theoretically using the variables in Fig. 3 as follows:

$$v_y = -\frac{r}{r_0 \cos \alpha} \psi_w + \frac{\dot{y}_w + r \dot{\phi}_w}{^xV_w \cos \alpha}. \quad (1)$$

Here, r_0 is the average rolling radius of the left and right wheels. Equation (1) applies to both the left and right wheels.

Assuming the flange-climb phase and a nonzero wheelset angle of attack $\psi_w \neq 0$, the normalized transverse creepage ξ is defined by v_y divided by ψ_w :

$$\xi := \frac{v_y}{\psi_w} = -\frac{r}{r_0 \cos \alpha} + \frac{\dot{y}_w + r \dot{\phi}_w}{\psi_w ^xV_w \cos \alpha}. \quad (2)$$

The quantity ξ can be expressed as variables representing the contact conditions, and can be decomposed into two components as follows:

$$\xi = \xi_s + \xi_F, \quad (3)$$

$$\xi_s := -\frac{r}{r_0 \cos \alpha}, \quad (4)$$

$$\xi_F := \frac{\dot{y}_w + r \dot{\phi}_w}{\psi_w ^xV_w \cos \alpha}. \quad (5)$$

Here, ξ_s , given by (4), is a state quantity that is mostly determined by the lateral contact position s , although the contact angle α is af-

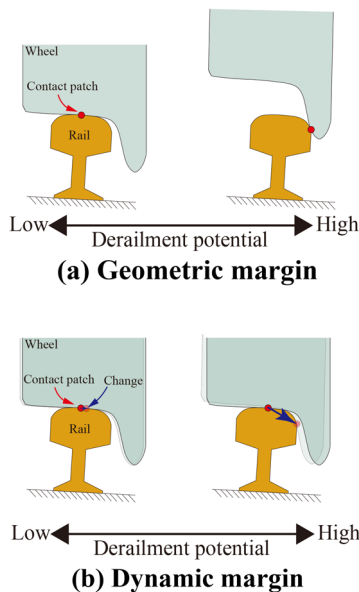


Fig. 2 Two indicators for evaluating running safety

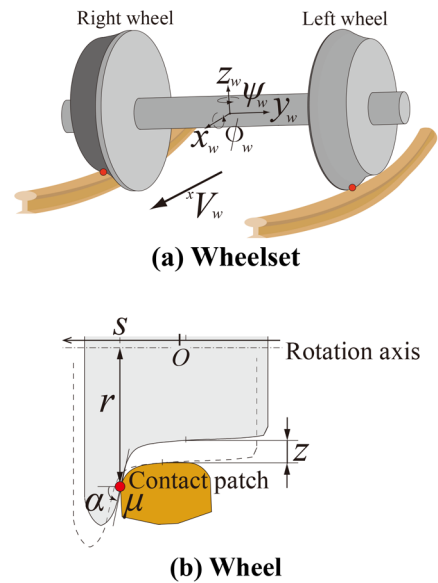


Fig. 3 Variables and coordinate systems representing contact condition between wheel and rail

affected by the wheelset roll angle ϕ_w and other factors. ζ_s is termed the normalized standard transverse creepage. ζ_F , given by (5), is a state quantity governed by the velocity components of the wheelset and is termed the normalized fluctuated transverse creepage. ζ_s corresponds to the slip component that generates a transverse creep force tending to diverge in the wheelset lateral displacement direction, whereas ζ_F corresponds to the slip that produces a transverse creep force acting in the deceleration direction.

As an example, Fig. 4 illustrates the normalized transverse creepage ζ for contact positions $s > 0$ when the wheel tread profile is the design-modified arc tread, commonly used in current Japanese commuter trains. In the neutral state, the wheel rolling radius has a value of 0.43 m. For reference, the wheel rolling radius r is plotted on the right axis (blue line). In Fig. 4, the red line represents the normalized standard transverse creepage ζ_s under the assumptions that the wheelset roll angle ϕ_w is approximated as zero and the rolling radius of the paired wheel is 0.43 m.

The characteristics of ζ_s observed in Fig. 4 are summarized as follows:

- In the tread of the wheel (approximately $s \leq 26$ mm), ζ_s decreases with an approximately constant gradient.
- As the contact point enters the flange throat area (approximately $26 \text{ mm} \leq s \leq 36$ mm), the gradient changes nonlinearly and ζ_s decreases more steeply than in the tread.
- When contact occurs on the flange straight section, corresponding to the wheel-rising phase (approximately $36 \text{ mm} \leq s \leq 40.2$ mm), the gradient becomes nearly constant again and ζ_s decreases gradually.
- During the flange-climb phase, when contact occurs at the flange top from the end of the flange straight section (approximately $40.2 \text{ mm} \leq s \leq 51$ mm), ζ_s increases nonlinearly.

The normalized fluctuated transverse creepage ζ_F takes values within the green band around ζ_s (red line) in Fig. 4, depending on the dynamic behavior of the wheelset. Regardless of whether the vehicle is negotiating a left or right curve, the numerator and denominator of (5) generally change sign simultaneously, and therefore the sign of ζ_F remains unchanged. The normalized transverse creepage ζ , which is the sum of ζ_s and ζ_F , thus lies within the green range. This study hypothesizes that the trajectories plotted on the s - ζ plane differ between states with and without the risk of flange-climb derailment. The study then attempts to distinguish between these states from the observed differences.

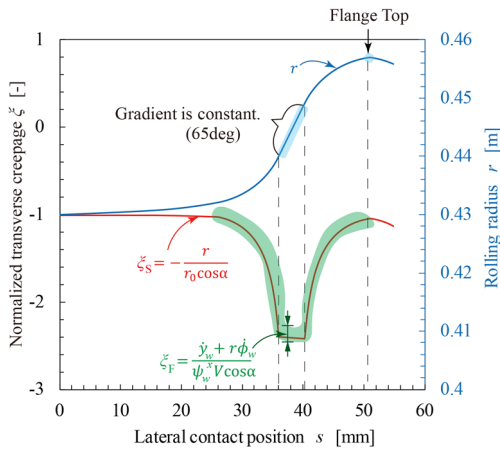


Fig. 4 Conceptual diagram of relationship between lateral contact position s , normalized transverse creepage ζ , and wheel rolling radius r

3. Investigation of relationship selected state quantities and running safety

To investigate the relationship between selected state quantities and running safety, vehicle dynamics simulations were performed under the assumption of flange-climb derailment. These simulations were conducted using Simpack (Dassault Systèmes, ver. 2024x.2), which allows various curve and running conditions to be specified easily and provides access to quantities that are difficult to measure experimentally, such as the wheel/rail contact position.

3.1 Simulation conditions

3.1.1 Vehicle model

The vehicle model represents a typical Japanese narrow-gauge railway vehicle operating on track with a gauge of 1067 mm. It consists of one car body, two bogie frames, and four wheelsets modeled as rigid bodies, as illustrated in Fig. 5. Each rigid body (shown in blue) has six degrees of freedom, and the connections between them include spring and damper elements (shown in red), which represent the primary and secondary suspension characteristics. The wheel profile is the design-modified arc tread.

3.1.2 Track model

As shown in Fig. 6(a), the track layout is modeled as a single left-hand circular curve consisting of five consecutive sections: an initial straight section, an entrance transition curve, a circular curve, an exit transition curve, and a terminal straight section. As illustrated in Fig. 6(b), the curvature and superelevation along each transition curve vary linearly with the longitudinal distance from its starting point. The entrance transition curve and the circular curve are both 100 m long. The length of the exit transition curve is varied by changing the end position X , as indicated in positions (i) - (iv) of Fig. 6(b), according to the specified superelevation run-off rate described later. A smoothing distance of 3.0 m in total (1.5 m at each boundary) is applied at the interfaces between adjacent sections to ensure geometric continuity. The rail profile follows the JIS 50 kgN design rail with a rail inclination of 1/40, corresponding to the standard tie-plate installation angle. The creep forces at the wheel/rail contact are computed using the FASTSIM algorithm [22].

A guard angle was installed on the inner rail; once the derailment phase was reached, the inner-side wheel came into contact with the guard. Since this condition increases the number of contact points and leads to a substantial rise in computational cost, the simulation was terminated when contact occurred. This state was treated as “derailment” in the data analysis.

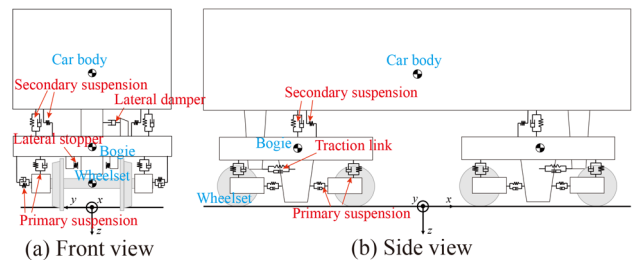
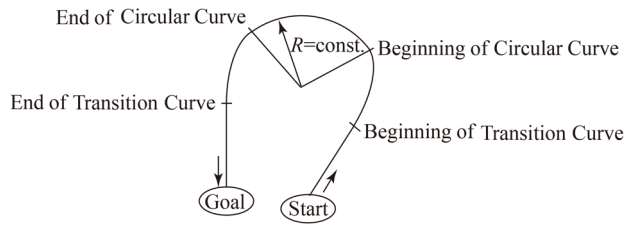
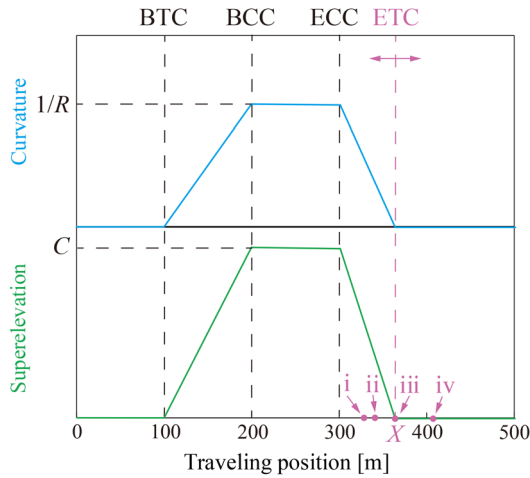


Fig. 5 Vehicle model used in simulation



(a) Track layout



(b) Track curvature and superelevation

Fig. 6 Track information for simulation

3.1.3 Parametric study variable settings

The simulation conditions were designed so that the wheel/rail contact conditions in the wheel-flange would vary over a wide range. The factors influencing the contact conditions were classified as static or dynamic effects, and the main factors are listed as follows.

- Static effects: static wheel load, static wheel-load imbalance, circular curve radius, superelevation, slack, friction coefficient, wheel tread profile, rail shape profile, primary stiffness, and secondary stiffness.
- Dynamic effects: cant run-off rate, running speed, track irregularity, friction coefficient, wheel tread profile, rail cross-section, primary stiffness, and secondary stiffness.

Herein, factors that are considered to affect both static and dynamic behavior are included in both categories.

To clarify the fundamental characteristics, five factors were selected as variables in the parametric study to represent influences that modify the contact condition.

- (1) Static wheel-load imbalance: The static wheel load on the outer rail side of the leading wheelset was reduced by 0%, 15%, and 30%.
- (2) Circular curve radius R : Two sharp curves with radii of 100 m and 160 m prone to flange-climb derailment were considered.
- (3) Friction coefficient μ : The friction coefficient on the inner and outer rails during running was assumed to be constant and identical, and was varied in the range $0.1 \leq \mu \leq 0.7$.
- (4) Superelevation run-off rate: The length of the exit transition curve (the distance $X=300$ m in Fig. 6(b)) was adjusted so that the exit superelevation run-off rate became (i) 300, (ii) 400, (iii) 600, or (iv) 1000. A value of 300 corresponds to the minimum run-off rate for a typical Japanese narrow-gauge railway vehicle

with a 2.1 m wheelbase.

- (5) Running speed sV_w : A constant running speed was assumed; for $R = 100$ m, the speeds were 10 km/h and 30 km/h, and for $R = 160$ m, the speeds were 10 km/h and 40 km/h.

All other factors were held constant under the following conditions:

- Static wheel load: A vehicle model representing a typical Japanese narrow-gauge commuter train yielded a static wheel load of 29.8 kN.
- Superelevation: Considering that a larger superelevation tends to reduce the outer-rail wheel load on curves and thus leads to more adverse conditions with respect to derailment, the maximum superelevation of 105 mm for narrow-gauge commuter rolling stock in Japan was adopted.
- Slack: Since a track with gauge slack is favorable in terms of derailment safety, the slack was set to 0 mm.
- Primary and secondary stiffnesses: These were set to typical values for Japanese narrow-gauge commuter rolling stock.
- Track irregularity: No track irregularities were applied.

3.2 Simulation results

3.2.1 Behavior of normalized transverse creepage with respect to contact position

Figure 7 shows the loci traced on the $s - \zeta$ plane by the lateral contact position s and the normalized transverse creepage ζ , obtained from a subset of the vehicle dynamic simulation results. The analysis conditions for Fig. 7 are as follows: a 30% reduction in the static-wheel load on the outer rail side of the leading wheelset, a circular curve radius of 100 m, an exit-side superelevation run-off rate of 400, a running speed sV_w of 10 km/h, and a friction coefficient μ that varies from 0.1 to 0.7 in increments of 0.1.

Figure 7(a) shows an overview from the flange-throat to the flange-top. For $\mu \geq 0.3$, the contact point reaches the start of the flange straight section; for $\mu \geq 0.5$, the contact point passes beyond the end of the flange straight section, reaches the flange top, and thereby enters the derailment phase. Figure 7(b) provides an enlarged view of the vicinity of the straight flange section, corresponding to the range where the contact position lies approximately between $36.0 \leq s \leq 40.2$ mm. As shown in Fig. 7(b), under simulation conditions in which the contact point passes beyond the straight flange section and ultimately reaches the derailment phase, the normalized transverse creepage ζ remains higher than the normalized standard transverse creepage ζ_s throughout. By contrast, when the contact point reaches the straight flange section but does not progress to the flange-climb or derailment phases, the relative magnitude of ζ and ζ_s reverses as s increases. Subsequently, the contact position moves back toward the neutral position. In this process, the wheelset angle of attack approaches zero, causing the response ζ to tend toward $-\infty$.

3.2.2 Transit domains of loci with and without flange climb after entering the exit transition curve

Under the simulation conditions adopted in this study, flange-climb occurred frequently after the vehicle entered the exit transition curve. Therefore, the following analysis focuses on the end of transition curve section.

The dataset was divided according to whether flange-climb occurred or not. Then, the transit domains of the loci traced by the lateral contact position s and the normalized transverse creepage ζ on the $s - \zeta$ plane were investigated under various simulation condi-

tions. In the analysis, ζ was regarded as a function of s , $\zeta=\zeta(s)$, and, for each case, $\zeta(s)$ was constructed using data from the moment the vehicle entered the exit transition curve until s reached its maximum value. Subsequently, for each data group, the maximum and minimum values of ζ at each s , denoted by $\zeta_{\max}(s)$ and $\zeta_{\min}(s)$, were determined and treated as the boundaries of the transit domain.

The results are shown in Fig. 8. The red-shaded area represents the transit domain of the data that reached the derailment phase (hereafter referred to as “derailment data”). The orange-shaded area represents the transit domain of the data that reached the flange-climb phase but did not reach the derailment phase (“flange-climb data”). The blue-shaded area represents the transit domain of the data that did not reach the flange-climb phase (“non - flange-climb data”).

The main findings from Fig. 8 are summarized below.

- The transit domain of the derailment data existed only for contact positions $s \geq 37.3$ mm. The lower boundary of this domain was approximately constant in the straight flange section (about $37.3 \text{ mm} \leq s \leq 40.2 \text{ mm}$).
- The transit domain of the flange-climb data existed for $s \geq 36.9$ mm. Its lower boundary was almost onstant around the middle of the straight flange section (about $36.9 \text{ mm} \leq s \leq 39.5 \text{ mm}$), but for $s > 39.5$ mm. However, the domain extended downward in terms

of ζ .

- For a given s , the lower boundary of the non - flange-climb data was below those of both the derailment data and the flange-climb data, and the corresponding domain width in the ζ direction was wider.
- Except in the vicinity of the exit of the straight flange section (approximately $40.0 \text{ mm} \leq s \leq 40.2 \text{ mm}$), the lower boundary of the transit domain for the non—flange-climb data was below the normalized standard transverse creepage ζ_s .
- There were areas where the three transit domains overlapped.

3.3 Discussion

From equations (2) - (5), the difference between the normalized transverse creepage ζ and the normalized standard transverse creepage ζ_s in Fig. 8 is equal to the normalized fluctuated transverse creepage ζ_F . ζ_F represents the degree of transverse creepage that includes the velocity effect relative to the angle of attack; the larger ζ compared to ζ_s , the more strongly the wheelset tends to move outward relative to the track gauge. As a result, the transverse creep force acts to reverse this motion and return the wheelset center of gravity toward the track center (hereafter, this force is referred to as the “restoring transverse creep force”).

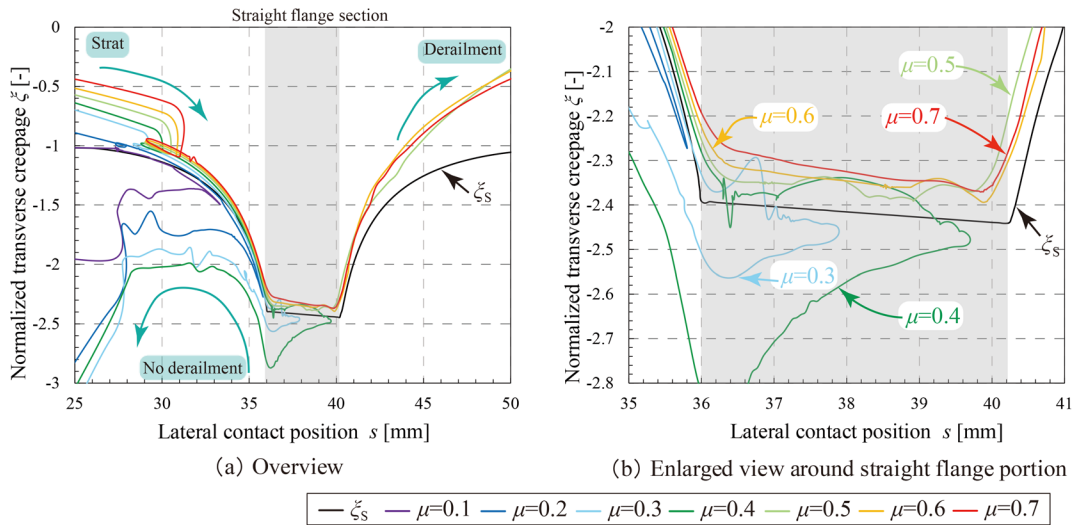


Fig. 7 Example of relationship between lateral contact position s and normalized transverse creepage ζ

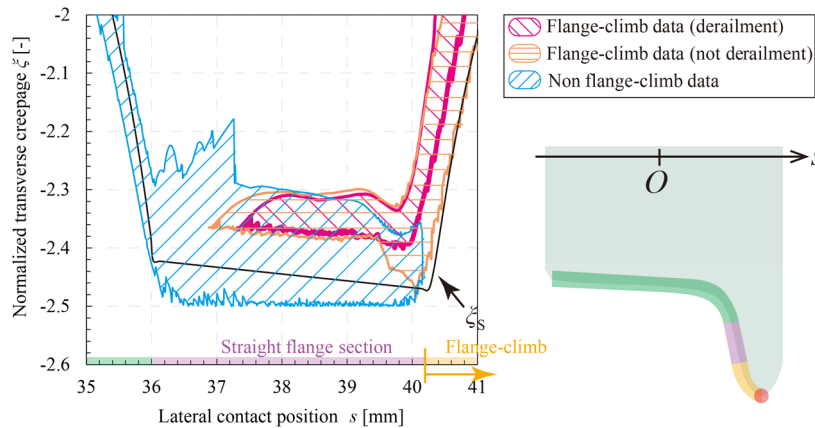


Fig. 8 Transit domains on the s - ζ plane for loci of derailment data, flange-climb data, and non-flange-climb data in the exit transition curve section

In Fig. 8, the difference in width of the transit domains in ξ among the derailment data, flange-climb data, and non - flange-climb data at the same contact position s can be interpreted as follows: the running conditions for the non - flange-climb data are relatively less severe than for the cases with flange-climb, so the lower boundaries of their transit domains extend to smaller ξ values, resulting in a wider domain. Despite the increase in ξ_p , flange climb or derailment is considered to occur because of the following factors:

- The wheel and rail are in contact on the straight section of the flange, large spin creepage is generated, and the transverse creep force that restores the wheel does not act effectively.
- The lateral acceleration of the wheelset is directed outward relative to the track gauge, so the lateral deceleration is insufficient and the contact condition progresses to the flange-climb or derailment phase.

For a more detailed discussion of these points, the reader is referred to Reference [21].

4. Proposal of evaluation methodology

This section describes the proposed method for evaluating running safety using indicators investigated in section 3. The method uses the evaluation diagram shown in Fig. 9. In this diagram, the state $[s, \xi]^T$, which is composed of the lateral contact position s and the normalized transverse creepage ξ , is classified into either a safe domain D_s or a dangerous domain D_D . Herein, Fig. 9 also shows the domains obtained from the simulation results in Fig. 8. The basic idea is to evaluate running safety according to whether the state $[s, \xi]^T$ lies in D_s or D_D .

The two domains are defined based on the preceding results according to the following concepts:

- Cases where the contact patch is on the tread side of the wheel ($s < 36.0$ mm): since this corresponds to the tread-contact phase and the wheel flange does not come into contact with the rail, this area is classified as the safe domain D_s .
- Cases where the contact point is on the straight section of the flange ($36.0 \text{ mm} \leq s < 40.2$ mm): this corresponds to the wheel-rising phase, and attention is focused on the relative magnitude of the normalized transverse creepage ξ and the normalized standard transverse creepage ξ_s . If $\xi < \xi_s$, the subsequent motion causes the contact patch to move toward the tread side, and the state is assigned to the safe domain D_s . Conversely, if $\xi \geq \xi_s$, the contact state may subsequently enter the flange-climb phase, and the state is classified as belonging to the dangerous domain D_D .
- Cases where the contact point is beyond the straight section of the flange ($s \geq 40.2$ mm): since contact has already reached the flange-

climb phase, this area is treated as being in the dangerous domain D_D , regardless of the value of ξ .

In this example, the boundary in the interval $36.0 \leq s < 40.2$ mm is set at $\xi = \xi_s$. However, Fig. 8 shows that some cases with $\xi > \xi_s$ do not reach the flange-climb or derailment phases, and that derailment does not occur if s remains below approximately 36.8 mm. Therefore, in this interval, the boundary line can be placed above ξ_s . This issue will be studied in more detail in future work to refine the domain definition.

Figure 10 presents a flowchart for evaluating running safety using the proposed method, where two modes of application are considered.

- Evaluation using the proposed method alone (Fig. 10(a)): Running safety is assessed from a new perspective, independent of conventional criteria. If the state $[s, \xi]^T$ lies within the safe domain D_s , the running condition is judged “safe” in terms of running safety; otherwise, it is judged “dangerous.”
- Combined evaluation with the conventional method (Fig. 10(b)): The proposed method is used as an auxiliary tool to improve the accuracy of the conventional safety assessment. First, it is checked whether the derailment quotient Y/Q , composing of the lateral force Y and the wheel load Q , exceeds the reference value δ , and safety is evaluated according to the conventional criterion. If $Y/Q > \delta$, the proposed method is then applied to determine whether the state $[s, \xi]^T$ lies within the safe domain D_s ; if it does, the condition can still be judged “safe” from the viewpoint of running safety. When $Y/Q \leq \delta$, the condition is judged “safe” by the conventional method alone, and no further evaluation using the proposed method is performed, because this practice has not caused problems in past operation.

As described above, the proposed approach allows for the evaluation of running safety using the state variables s and ξ , either independently or in combination with the conventional derailment-quotient-based assessment; however, as noted, the validity of the domain definition exemplified in Fig. 9 still requires more detailed verification.

5. Conclusions

Future advances in research and development are expected to make it easier to identify wheel/rail contact conditions. This motivates the present study on running-safety evaluation methods for flange-climb derailment that explicitly account for contact conditions. Among the various quantities that characterize the contact condition, this study adopted the normalized transverse creepage (defined as the transverse creepage divided by the wheelset angle of

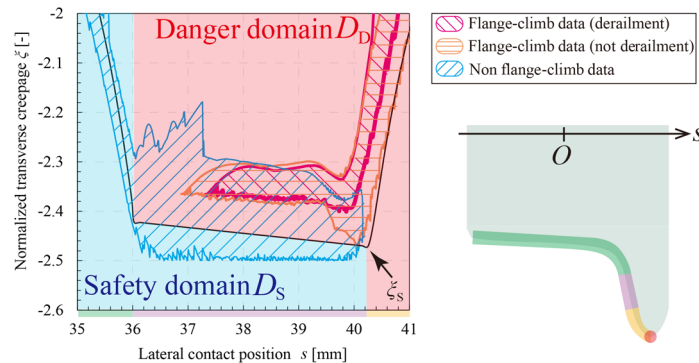
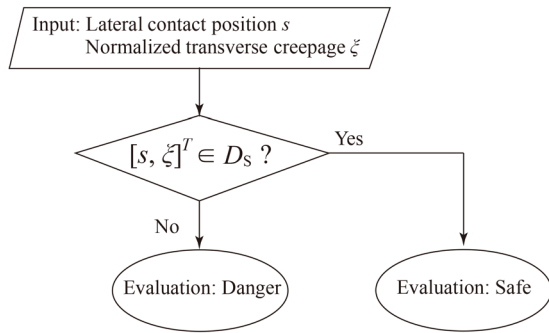
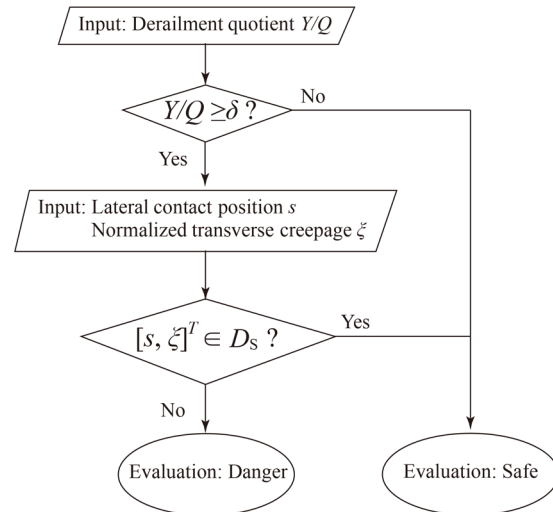


Fig. 9 Example of safety domain D_s and danger domain D_D on the s - ξ plane



(a) Using only proposed method



(b) Combination with conventional evaluation method

Fig. 10 Two flowcharts for evaluating running safety using the proposed method

attack) and the lateral contact position as evaluation indicators. Using railway vehicle dynamics simulations, trajectories traced by these two indicators on a common plane were examined under various simulation conditions, and the relationship between their transit domains and running safety was investigated.

The results showed that, when the data were classified according to whether flange-climb occurred after entering the exit transition curve or not, the corresponding transit domains differed clearly. Specifically, when the contact patch was located on the straight section of the flange, larger values of the normalized transverse creepage (i.e. values closer to zero) were associated with states more susceptible to flange-climb and derailment. Therefore, even if the contact position lies on the straight portion of the flange, it may be possible to judge the running state as safe if the normalized transverse creepage falls below a certain threshold. In other words, some cases that would be judged as dangerous using conventional derailment-quotient-based criteria could be evaluated as safe using the present method.

Conversely, within the scope of the present investigation, the following aspects and their associated influences could not be fully verified:

- Track irregularities were not taken into account, and thus the assumed running conditions may differ from actual operating conditions.
- The wheel and rail profiles were limited to a modified-arc tread and a 50 kgN rail design profile; other shapes and combinations were not considered, and the vehicle parameters were restricted to a single vehicle type.
- In each simulation run, the friction coefficients on the inner and outer rails were assumed to be identical, and situations involving abrupt changes in friction coefficient due to friction modifiers were not considered; instead, the friction coefficient was held constant. This implies that the simulations may not fully cover the diversity of contact conditions.

Consequently, the present study does not yet establish a concrete procedure for defining the evaluation domain boundaries that explicitly incorporates safety margins. These challenges are reserved for future work, which will involve more detailed investigations. Furthermore, running tests using real vehicles are planned to examine the proposed method in greater detail and further validate

its practical applicability.

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Method for Estimating Factors of AI Decision Errors in Train Forward Surveillance

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In the railway industry, AI-based automation using camera images is increasingly being introduced to tasks that were previously performed visually. For future application to safety-critical operations such as autonomous driving, it is essential to investigate the causes of AI decision-making errors. This study proposes a method to trace train forward surveillance and estimate the causes of decision-making errors, such as missed detections, at three stages: input images, AI architecture, and training data. This paper describes the proposed cause-estimation method and reports verification results using evaluation images.

Key words: train forward-facing obstacle detection, artificial intelligence, object detection, decision-making error, missed detection

1. Introduction

Recent advances in machine learning, particularly deep learning, have enabled artificial intelligence (AI) to achieve high performance in multiple domains, including image recognition, natural language processing, and predictive analytics. Accordingly, AI utilization has expanded across diverse fields such as healthcare, manufacturing, finance, and education, and it is expected to contribute to addressing labor shortages caused by demographic changes.

At the same time, AI (especially deep learning) often lacks transparency in its decision process, which makes it difficult to understand how specific outputs or decisions are produced. This means that, when AI makes an erroneous decision, identifying reasons to explain the error can be difficult. This issue is also mentioned in the Ministry of Internal Affairs and Communications “AI Utilization Guidelines” [1]. These guidelines emphasize that when AI is used in a given social context, it is necessary to analyze and identify the type of explanations [for actions] that would be required and to implement appropriate measures accordingly. In particular, for safety-related railway systems, it is necessary to be able to explain the causes of decision errors and what corrective actions are envisaged in such cases, to ensure transparency.

Although “white-box” AI with high transparency has been explored, no effective and practical method has been established for deep-learning-based image processing as of today. Considering that non-transparent AI is already being deployed in other fields, a realistic approach would be to establish a framework that can secure a certain level of transparency.

Against this background, this paper addresses on-train forward-facing obstacle detection systems using AI and proposes a method for estimating the causes of errors in AI decision-making, such as missed detections. Forward-facing obstacle detection is safety-critical because it is designed to support train drivers and may in future be applied for autonomous train operations.

By using the proposed method, railway operators can inspect the detection performance of an AI-based forward-facing obstacle detection system before implementation and can also investigate the causes for missed detections and consider follow-up countermeasures.

2. Use of AI in on-train forward-facing obstacle detection

This paper considers a train forward-facing obstacle detection system in which sensors are installed at the front of the train and an AI system monitors the forward area using sensor information, as shown in Fig. 1. In practical deployment, the sensing unit may consist of a single sensor or a combination of sensors such as a camera, LiDAR, and millimeter-wave radar.

3. Method for estimating causes of AI decision-making errors

3.1 Overview

A critical hazardous event in camera-based train forward surveillance is a missed detection of an obstacle. Therefore, a method has been developed to estimate the causes behind a failure of a forward-facing obstacle detection AI to detect an obstacle. This method involves three stages: inspection of input images, inspection of AI architecture, and inspection of training data. Figure 2 presents the overall flow.

If a factor can be identified at any one of the inspection stages in the 3-step factor estimation method, this factor can then form the basis for a follow-up countermeasure. Inspections are conducted in order of how easily corrective measures can be implemented once a

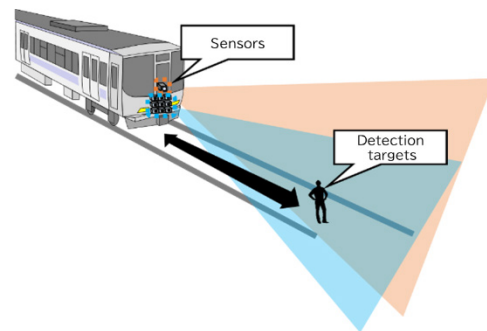


Fig. 1 Proposed train forward-facing obstacle detection system

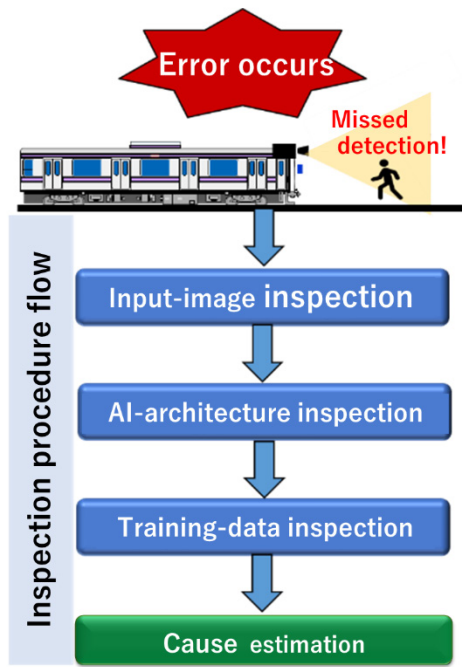


Fig. 2 Overall flow of proposed cause-estimation method for AI decision-making errors

factor has been identified, starting with the stage where corrective action is easiest. The rationale for this approach is based on the assumption that if an error does occur, a rapid response is required. The first stage, input-image inspection, focuses on causes related to camera settings or imaging conditions rather than the AI itself; such causes are often easier to address (e.g., changing camera settings) than AI-related causes.

The second stage, AI-architecture inspection, identifies decision-making errors caused by differences in recognition mechanisms across architectures. If architecture-related causes are identified, they may be addressed by modifying or adding to the architecture, or by tuning it.

The third stage, training-data inspection, investigates biases in the distribution or characteristics of the training data that may cause errors. Possible solutions for these issues include undersampling the biased data or collecting and adding missing data. Since adding training data usually requires substantial effort in terms of data col-

lection and annotation, this step is placed last in the flow.

The following subsections describe each type of inspection in detail.

3.2 Inspection of input images

Figure 3 illustrates the procedure for detecting decision-making errors caused by input images. Image processing is used to reproduce changes in camera settings for images in which an error (e.g., missed detection) occurs. The forward-facing obstacle detection AI is then re-run on the processed images to check whether the decision has been improved. If an improvement is observed for a certain setting, that setting is regarded as an estimated cause of the error.

Ten items were defined: rotation, position, contrast, white balance, saturation, brightness, defocus blur, motion blur, focal length (optical zoom factor), and noise. These items were selected from error factors observed in previous studies on AI-based object detection, with a focus on factors that can be countered by adjusting camera settings and related measures (Table 1).

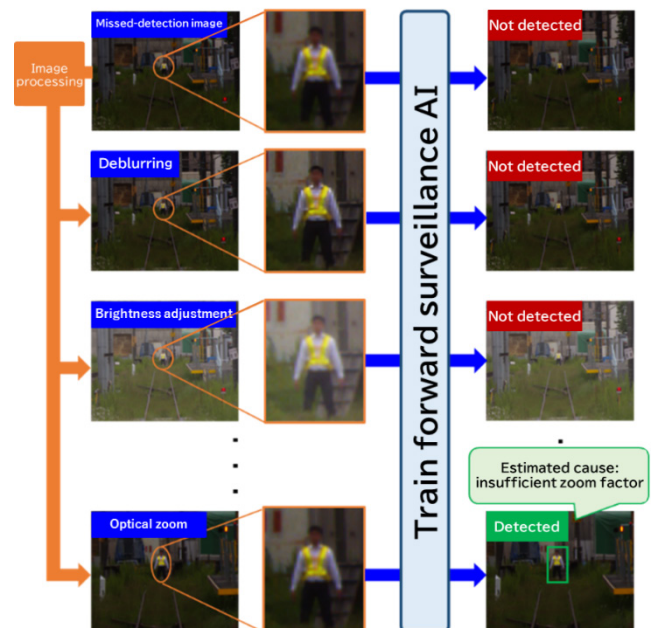


Fig. 3 Cause estimation by input-image inspection

Table 1 Correspondence between setting items (possible causes) and countermeasures

Setting item (possible cause)	Example image processing to reproduce	Feasible countermeasures
Defocus blur	Deblurring filter	Focus adjustment
Motion blur	Deblurring filter	Change shutter speed; change train speed
Brightness	Adjust V in HSV; gamma correction	Change shutter speed; change depth of field; install trackside light source
Contrast	Histogram equalization	Change contrast setting
White balance	Adjust color temperature	Change white-balance setting
Saturation	Adjust S in HSV	Change camera saturation setting
Rotation	Affine transformation	Change camera installation orientation
Position	Affine transformation	Change camera installation position
Optical zoom	Cropping + super-resolution	Change camera lens
Noise	Median filter; bilateral filter	Change ISO sensitivity; change shutter speed

3.3 Inspection of AI architecture

Deep-learning-based object detection algorithms are known to exhibit different recognition characteristics depending on the network architecture (hereafter, AI architecture) [2]. AI-architecture inspection aims to estimate decision-making errors caused by such differences.

Architecture differences include the base architecture (e.g., CNN or Transformer) and variations in network depth. The base architecture affects recognition mechanisms. For example, CNNs are often considered to focus on fine details, whereas Transformers tend to focus on global features such as silhouettes [2]. Input resolution also affects feature extraction and recognition: higher resolution allows finer features to be considered, while lower resolution may fail to capture such details.

To confirm whether these tendencies cause a decision-making error, the proposed inspection constructs multiple forward-facing obstacle detection AI models with the same input resolution that is generated from the same training data, but with different AI architectures (Table 2). The error images are then detected using these models. If some models can detect the target while others fail, the differences between the failed condition (architecture–resolution combination) and the successful condition are regarded as the cause of the error (Fig.4). Once a cause is identified, it is recommended that the forward-facing obstacle detection system be reconstructed using an improved architecture (i.e., an architecture that succeeded).

3.4 Inspection of training data

Multiple cases have been reported in which AI decision-making errors are caused by biased data distributions. For example, a hiring AI system was discontinued because it produced discriminatory evaluations against women, which was attributed to training data dominated by male résumés [7, 8].

Table 2 Models (network architectures) and input resolutions

Model (network architecture)	Input resolution
YOLOX-nano (CNN) [3]	(416, 416), (640, 640), (1280, 1280)
YOLOX-s (CNN) [3]	(416, 416), (640, 640), (1280, 1280)
YOLOX-l (CNN) [3]	(416, 416), (640, 640), (1280, 1280)
YOLOv8n (CNN) [4]	(640, 480), (1280, 960)
YOLOv8s (CNN) [4]	(640, 480), (1280, 960)
YOLOv8l (CNN) [4]	(640, 480), (1280, 960)
DETR (Transformer) [5]	(640, 480), (1280, 960)
Co-DETR (Transformer) [6]	(640, 480), (1280, 960)

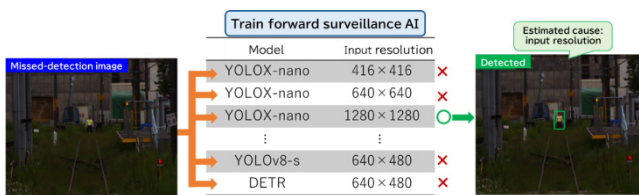


Fig. 4 Cause estimation by AI-architecture inspection

The proposed method estimates such training-data-related causes as follows. First, distributions of image features are computed from the training data. Secondly, the position of the error image in the training-data distribution is visualized. If the error image corresponds to a low-frequency region in the distribution, the bias is regarded as an estimated cause.

Figure 5 shows example features for visualizing when humans are the detection targets. Training data for object detection include bounding-box annotations. Using these annotations, 108 features are visualized by combining the global features of the whole image and the local features within the bounding-box regions.

Figure 6 illustrates the concept of cause estimation by examining training data using feature distributions. Feature distributions (e.g., bounding-box aspect ratio, hue, saturation, and brightness) are plotted, and the position of the miss detection image in each distribution is shown. If the miss detection image corresponds to a rare region, the corresponding feature can be estimated as a cause.

The above inspection covers relatively simple features. However, neural networks make decisions based on complex, nonlinear features. Therefore, it is desirable to inspect complex features as well, although such features are difficult for humans to interpret. To incorporate complex features without direct interpretation, a similarity search is used to examine how many similar images are included in the training data.

For the similarity search, CLIP, a vision-language model widely used in image retrieval, is employed [9]. CLIP learns multimodal representations by associating images with text. Figure 7 outlines CLIP, and Fig. 8 shows an example of the results of a similarity search.

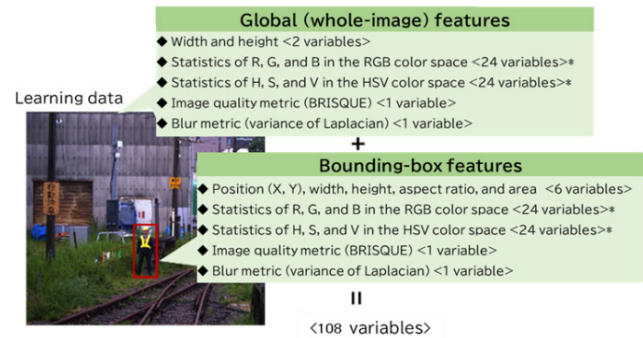


Fig. 5 Features visualized from training data

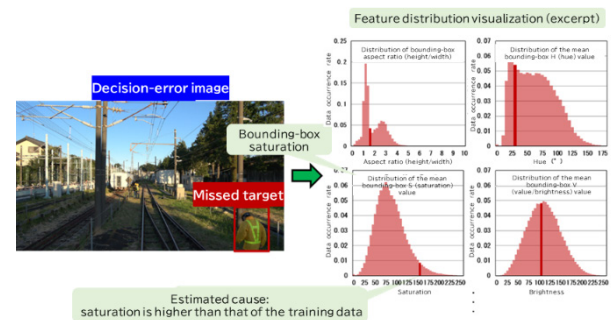


Fig. 6 Cause estimation using training-data feature distributions

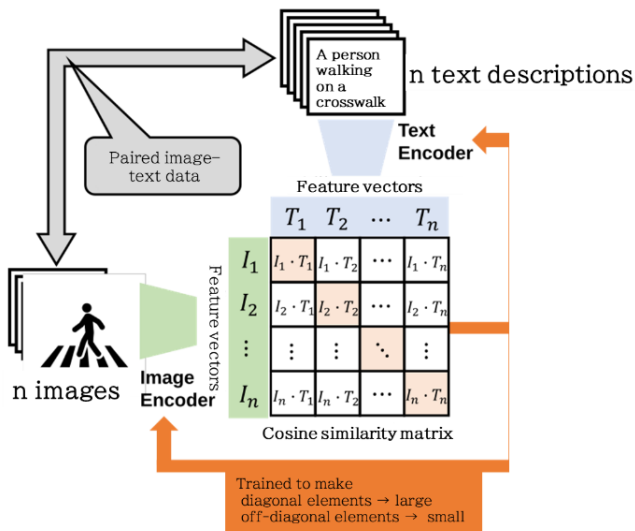


Fig. 7 Overview of CLIP

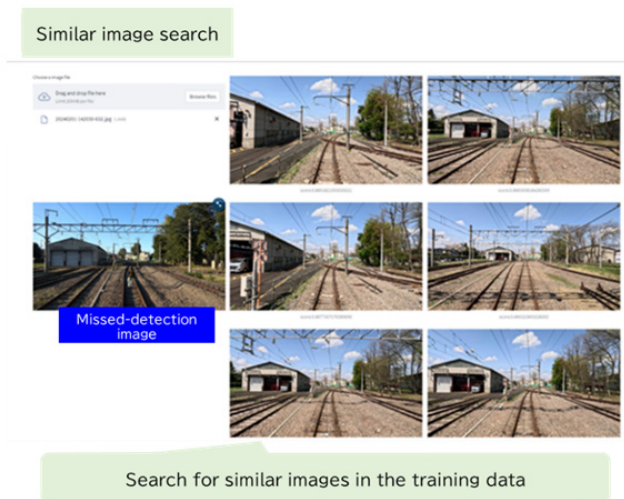


Fig. 8 Similarity search using CLIP

4. Evaluation of the method

4.1 Creation of evaluation images

To evaluate the proposed method, images are required in which the forward-facing obstacle detection AI actually fails to detect an

obstacle. These images must satisfy both conditions: (i) images represent hazardous situations with an obstacle present, and (ii) the forward-facing obstacle detection AI fails to detect the obstacle.

In practice, each condition is rare, and collecting images that satisfy both simultaneously is extremely difficult. In particular, hazardous situations with obstacles are difficult to reproduce and capture in real environments, and available locations are limited (e.g., test tracks). Meanwhile, the causes of decision-making errors can vary widely depending on weather, sunlight, and line conditions; thus, evaluation images should cover common operational conditions, and preferably be taken from commercial lines.

Therefore, evaluation images were generated by inserting obstacles synthetically into real forward-facing images captured on commercial lines using generative AI (Fig. 9). GLIGEN was used as the image generation model [10]. GLIGEN supports control of object placement using bounding boxes in text-to-image generation, enabling the creation of pseudo-hazardous images from real operational images.

4.2 Evaluation using the generated images

Using the above generation method, hazardous images were created from approximately 215 hours of forward-facing videos (7.6 TB) captured from the front of running trains. Recorded between 2007 and 2023, the videos include various lines across Japan, as well as different seasons, weather conditions, and times of day. Since the camera equipment also varied, the resolutions ranged from HD to higher.

Next, the forward-facing obstacle detection AI developed at RTRI for a train forward-facing obstacle detection system was applied to the generated hazardous images [11]. Among approximately 75,000 images, missed detections were observed in 2,668 images.

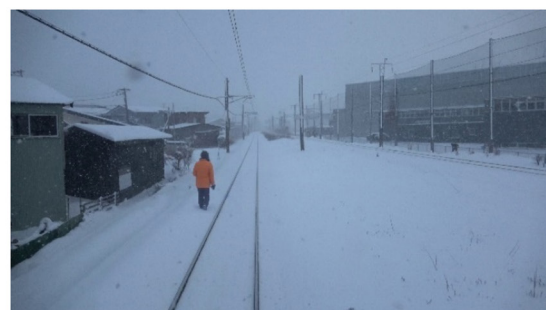
The proposed cause-estimation method was then applied to these 2,668 missed-detection images to evaluate its performance. In this evaluation dataset, the cause could be estimated for every image. Examples of missed detections for which causes were successfully estimated are shown in Figs 10–13. For data protection purposes, images in this paper have been pixelated. However, for the actual evaluations, original unprocessed images were used.

Figure 10 shows an example of input-image inspection. In this case, changing the brightness correction made the target detectable, indicating that brightness-related factors were the cause. Feasible countermeasures include installing light sources, changing the shutter speed, or adjusting the gain.

Figure 11 shows an example of AI-architecture inspection. In this case, changing the architecture enabled detection, indicating that architecture-related factors were the cause. Because the target



(a) Fine weather



(b) Snowy weather

Fig. 9 Examples of hazardous images generated from commercial-line forward images

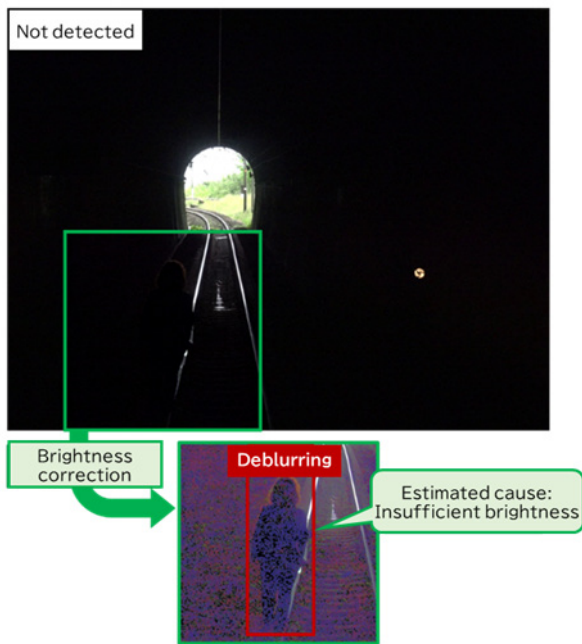


Fig. 10 Example estimated by input-image inspection

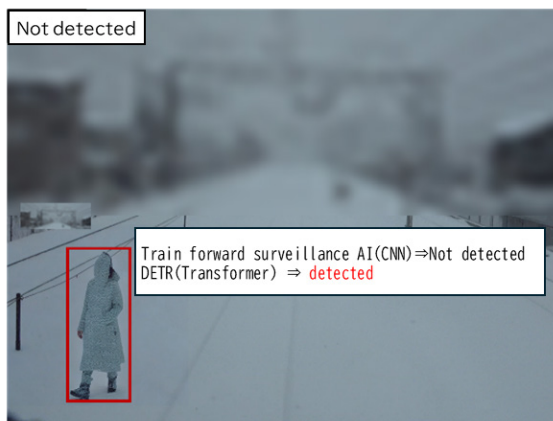


Fig. 11 Example estimated by AI-architecture inspection.

contained limited exposed fine details, the recognition mechanism likely contributed to the miss.

Figure 12 shows an example of training-data inspection using feature distributions. The target was a person lying on the track. The distribution of bounding-box aspect ratios (height/width) indicated that horizontally elongated boxes were rare in the training data. Thus, the lack of training data for persons lying on the ground was estimated to be the cause.

Figure 13 shows an example of training-data inspection using a similarity search. For missed detections involving water droplets on the front windshield of the train, the similarity search revealed that only a few similar images existed in the training data. Therefore, the scarcity of such images was estimated to be the cause.

5. Conclusions

This paper proposes a method for estimating the causes of decision-making errors, such as missed detections, in AI-based train



Fig. 12 Example estimated by training-data inspection (feature distribution)

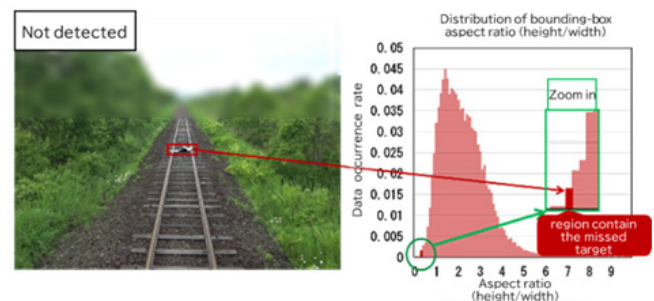


Fig. 13 Example estimated by training-data inspection (similarity search)

forward-facing obstacle detection systems. The method involves a three-stage procedure comprising inspection of input images, AI architecture, and training data; and permits estimation of causes behind decision-making errors.

To evaluate the method, approximately 75,000 hazard images were generated by modifying real on-train forward-facing images using generative AI in order to reproduce rare hazardous situations. The method was applied to 2,668 images with missed detections, for which the causes could be estimated. This demonstrates the feasibility of the proposed approach.

The proposed method is expected to support the pre-introduction inspection of obstacle detection performance for railway operators. It will also facilitate the investigation of causes and the planning of follow-up countermeasures when missed detections occur after deployment. Future work includes extending the approach to other railway AI applications such as maintenance and further studies on explainability and transparency to promote the safe utilization of AI in the railway domain.

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Proposal for a Train Rescheduling Evaluation Metric and a Method for Analyzing and Evaluating Rescheduling Actions

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To support dispatchers during disruptions, this paper describes a proposal for a metric to evaluate the “number of late-arriving passengers”, the development of a train rescheduling simulator; and presents a method for analyzing and evaluating the impact of actual train rescheduling plans on passenger convenience. The evaluation metric counts passengers whose delay in arrival at their destination station exceeds a threshold, enabling the comparison of train rescheduling plans. The simulator predicts timetables under different train rescheduling actions. Using the evaluation metric, the analysis identifies when and where passenger convenience changes due to changes in train rescheduling actions. We describe these methods in detail.

Key words: train rescheduling, train timetable, transport planning, single-track line, passenger behavior, evaluation metric

1. Introduction

When train operations are disrupted by accidents and other incidents, trains are rescheduled to restore the train timetable. Effective train rescheduling methods are required to minimize adverse effects on passenger convenience.

In order to minimize these adverse effects, railway operators evaluate how dispatchers reschedule trains, by considering what other more effective rescheduling plans there might be and comparing actual train rescheduling plans with other possible alternatives (Fig. 1). For these comparisons to be made, it is essential to predict train operations under other plans, and to have a method that allows quantitative relative evaluation using a metric that reflects passenger convenience.

Currently, there are no evaluation metrics to quantitatively evaluate the impact of alternative train rescheduling plans on passengers. Consequently, no established method exists to analytically determine which train rescheduling plan is relatively more effective. In practice, several common metrics are used, such as total train delay time (sum of delays over trains and stations), the number of canceled trains, and time from when a disruption occurs to when the train timetable resumes. However, these do not adequately reflect the impact of disruption on passengers. Previous studies have employed a method based on passenger disutility. This approach estimates travel time, transfers, and crowding during disruptions through simulation, converting the resulting disutility into time units for comparison with normal operations [1]. Another approach is to evaluate the actual impact on passengers based on the difference between normal travel time and travel time derived from smart card data (IC cards) during disruptions [2]. Since both methods incorporate the total arrival delay time of passengers as part of the

evaluation value, they cannot distinguish between different delay distributions, such as ten passengers delayed by one minute versus one passenger delayed by ten minutes. A proposed index, termed “POINT”, evaluates the magnitude of a disruption by calculating the product of affected duration and the number of passengers. This index is derived from the location and time of disruption, the time services resume, and the number of canceled trains. However, the value remains unchanged if train rescheduling actions do not affect the time services resume or the number of train cancellations. Moreover, the IC-card-based and POINT-based methods rely on observed data from actual disruptions. Consequently, they cannot calculate evaluation values for alternative train rescheduling plans that differ from the actual actions taken.

Accordingly, to improve the post-disruption reviewing process, we propose an evaluation metric that can also be applied to alternative plans. We further develop a method for comparing and analyzing multiple train rescheduling plans using the proposed evaluation metric, thereby enabling evaluation of the appropriateness of each plan. Finally, we conduct a case study on a commuter line consisting of both single-track and double-track sections to demonstrate the effectiveness of the proposed method.

2. Analyzing and evaluating train rescheduling

2.1 Basic concept of evaluation metric

To reflect changes in passenger convenience resulting from different train rescheduling plans, the evaluation value should change with shifts in passenger behavior. Since the IC-card-based method in a previous study [2] relies on data observed during dis-

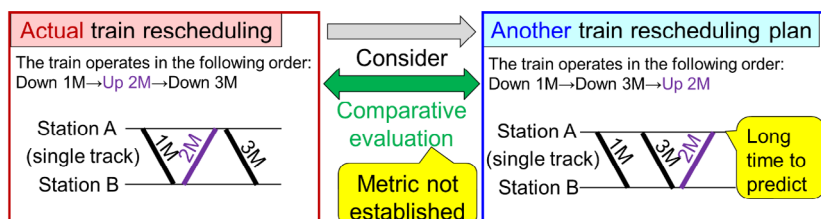


Fig. 1 Review of train rescheduling

ruptions, it cannot be used to evaluate other train rescheduling plans. The POINT-based method [3] calculates an index of disruption magnitude, so that the evaluation value remains constant unless the resumption time of operations or the number of canceled trains changes. Therefore, there is a need for an evaluation metric that can be applied for alternative train rescheduling plans and can capture differences in passenger behavior, even when resumption time and the number of canceled trains remain unchanged. Therefore, we propose an evaluation metric based on individual arrival time at the destination stations and establish a method for the relative evaluation of multiple train rescheduling plans.

First, we define “arrival delay time” as the difference between a passenger’s arrival time at their destination under a specific rescheduling plan and scheduled arrival time. We then define the “number of late-arriving passengers” as the total number of passengers whose arrival delay time exceeds a specified threshold [4]. This value is computed for various train rescheduling plans, and then we propose that a plan with a smaller number of late-arriving passengers is superior.

Arrival delay time can be estimated from automatic ticket-gate data under normal conditions, the scheduled timetable, and the predicted timetable for the train rescheduling plan. Therefore, automatic ticket-gate data observed during the disruption are not required. Consequently, arrival delay time and the number of late-arriving passengers can also be calculated for train rescheduling plans other than the actual train rescheduling.

2.2 Calculation flow for the number of late-arriving passengers

Figure 2 shows the procedure for calculating the number of late-arriving passengers.

First, the operational conditions (predicted timetable) expected when a certain train rescheduling action is implemented are estimated using a train rescheduling simulator. Next, using automatic ticket-gate data under normal conditions, train transfer routes for each passenger are estimated for both the scheduled timetable and the predicted timetable. Note that passengers are assumed to select the trains that enable them to arrive at their destination stations at the earliest possible time under each timetable.

We compute the number of passengers whose arrival is delayed beyond the threshold under the predicted timetable by comparing the estimated train transfer routes for each passenger under the planned and predicted timetables, and by setting various delay thresholds.

Section 2.3 outlines the method for estimating train transfer

routes and arrival delay time. Section 2.4 describes the method for comparing and analyzing multiple train rescheduling plans while Section 2.5 introduces the simulator used to generate predicted timetables that reflect rescheduling actions.

2.3 Calculation method for arrival delay time

The arrival delay time for each passenger is defined as the additional time required to reach the destination station compared with the scheduled timetable. Figure 3 shows how arrival delay time is calculated. According to the scheduled timetable, Passenger P traveling from Station A to Station D boards Rapid 3M at Station A. If Rapid 3M is delayed, Passenger P continues to board Rapid 3M and arrives later at Station D. Likewise, Passenger Q traveling from Station A to Station C boards Rapid 3M and transfers at Station B to Local 1M according to the scheduled timetable. If Rapid 3M is delayed, Passenger Q misses the connection to Local 1M and instead boards the following Local 5M. This results in Passenger Q arriving at Station C later than expected. In this way, we compute each passenger’s route and arrival time at the destination station for both the scheduled timetable and the predicted one to compute the arrival delay time by taking their difference.

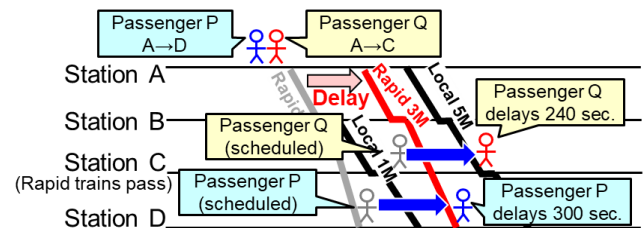


Fig. 3 Arrival delay time of passengers

2.4 Methods for comparing and analyzing multiple train rescheduling plans

The methods described in the previous section enable the estimation of passengers’ transfer routes between trains. This is achieved by analyzing automatic ticket-gate data under normal conditions and using predicted timetables generated by other train rescheduling plans. This in turn allows various train rescheduling plans to be evaluated quantitatively from the perspective of passenger convenience, based on predicted train operations. Moreover, the results of train transfer route estimations can be used to perform

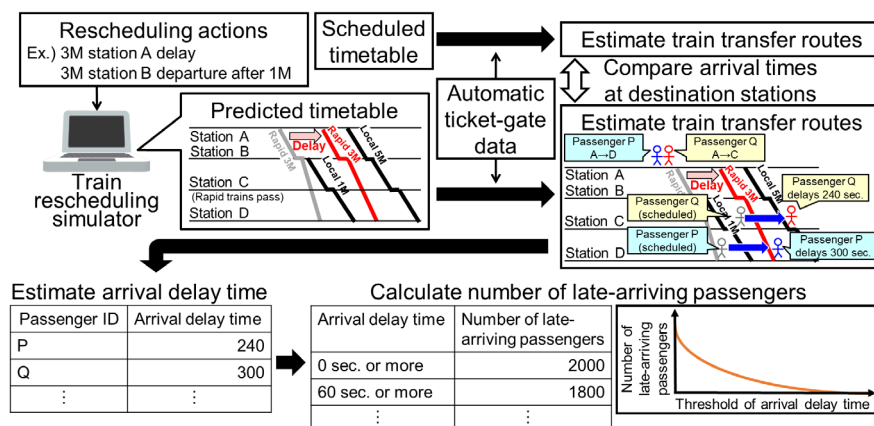


Fig. 2 Calculation flow of number of late-arriving passengers

detailed comparisons and analyses of the trains used by passengers who experience delays. The comparison and analysis methods are described next.

First, we compare the number of late-arriving passengers to determine which train rescheduling plan is better. Specifically, for each plan we compute the number of late-arriving passengers for a range of various arrival delay time thresholds. We then visualize the number of late-arriving passengers, by plotting the threshold of arrival delay time on the horizontal axis, and the number of late-arriving passengers on the vertical axis. Based on the plot, we conclude that a plan yielding a smaller number of late-arriving passengers across multiple thresholds is better.

Next, we conduct an analysis to identify the train rescheduling actions that cause differences in the number of late-arriving passengers between the two train rescheduling plans. First, we focus on passengers whose arrival times at their destination stations differ between the two train rescheduling plans. We cross-tabulate their arrival delay times and visualize the results as a two-dimensional histogram. Furthermore, we then use a colored timetable showing the number of passengers on trains used by passengers with large differences in arrival time. This visualizes the trains causing the differences and identifies the associated train rescheduling actions.

2.5 Train rescheduling simulator

There are no historical timetables available when evaluating train rescheduling plans other than the one actually implemented, so it is necessary to generate a predicted timetable. However, given complex train rescheduling actions, calculations must be performed sequentially to predict the arrival and departure times for all trains across all stations. Specifically, in addition to various constraints such as ensuring standard operating times and stopping times, train headway times and train sequence relationships also have to be taken into account. This makes manual estimation difficult, except for simple line sections. Therefore, to efficiently execute the procedure in Fig. 2, we developed a train rescheduling simulator.

Once train rescheduling actions are entered into the simulator through a graphical interface, it calculates and displays the predicted timetable based on the timetable prediction method proposed in previous research [5]. Furthermore, it computes the number of late-arriving passengers for the resulting predicted timetable.

When using the train rescheduling simulator to compare actual train rescheduling actions with their alternatives, the computational conditions must be consistent. However, actual timetables include

fluctuations in stop times and running times that occur on the day. These include increased station stop times due to passengers rushing to board and variations in running times between stations due to the driver's driving operations. However, since the predicted timetables generated by the train rescheduling simulator are based on minimum stop times and standard running times, they do not include such fluctuations on the day. Therefore, when evaluating the actual train rescheduling actions, we do not directly use the reconstructed actual timetable. Instead, we input the train rescheduling actions applied on the day into the train rescheduling simulator and use the predicted timetable generated by the simulator as the basis for evaluation.

The train rescheduling simulator consists of three main functions, as shown in Fig. 4.

(1) Input of train rescheduling actions

Once the timetable is displayed on the screen, selecting a train line allows the user to enter train rescheduling actions related to that train, including cancellations, changes in train order, time adjustments, and vehicle operations.

(2) Computation of predicted timetables

The train rescheduling simulator calculates and displays a predicted timetable based on the input train rescheduling actions. A graph-theoretic approach [5] is adopted to prioritize computational speed. By viewing the computed timetable, the user can iteratively add further train rescheduling actions. This supports the creation of train rescheduling plans through repeated editing and computation.

(3) Computation of evaluation values

Based on the computed predicted timetable, the train rescheduling simulator calculates and displays the number of late-arriving passengers for a specified delay threshold. In addition, it can visualize delays and crowding by coloring train lines according to inter-station occupancy and delay level.

3. Case study

3.1 Overview of case study on train rescheduling

To examine the effectiveness of the proposed evaluation metric and analysis/evaluation method, we conducted a case study based on an actual train rescheduling event on a commuter line featuring both single-track and double-track sections. Specifically, the following two timetables were compared:

(a) A timetable reflecting the rescheduling actions that have actually

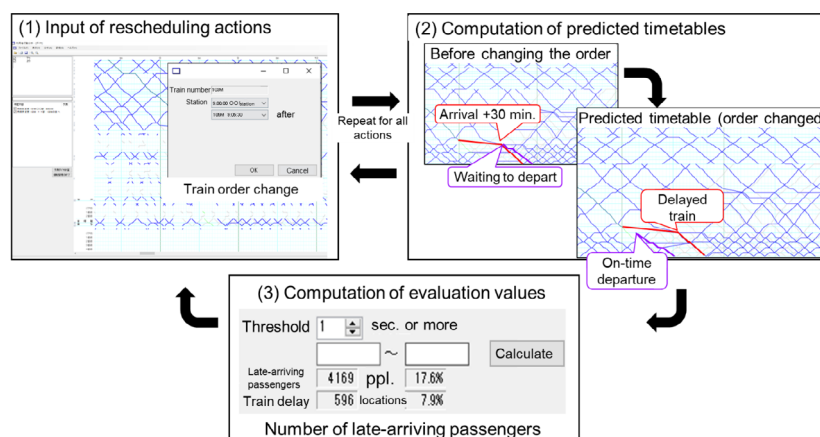


Fig. 4 Overview of the train rescheduling simulator functions

been implemented (hereinafter, the “reconstructed actual timetable”)

- (b) A virtual timetable reflecting rescheduling actions that differ from the actual ones (hereinafter, a “virtual timetable”)

The virtual timetable was created by replacing some of the original actions with less desirable ones. We computed and compared the number of late-arriving passengers for the reconstructed actual timetable and the virtual timetable.

We conducted various case studies including a case where no train rescheduling actions were made at all and a case where additional trains were canceled. In this paper, we report on a case study in which we assumed that a train that was actually canceled would not be canceled. Figure 5(a) shows the actual timetable that was analyzed. Although Train T_1 shown in Fig. 5(a) was scheduled to depart Station C at 8:17, it resumed operation 27 minutes late due to an accident. In the reconstructed actual timetable, four trains running between Stations A and G were canceled, passing loop stations were changed, and arrival and departure times were adjusted.

On the other hand, the virtual timetable was created assuming that the four trains would not be canceled. Specifically, it was created by adding two inbound and two outbound trains to the gaps between inbound trains at Station A and between outbound trains at Station G in the reconstructed actual timetable. Additionally, it was created by changing the passing loop stations and adjusting arrival and departure times. According to dispatchers who actually handle train rescheduling, the virtual timetable is undesirable, as it will result in delays to the departures of trains with longer travel distances compared to the reconstructed actual timetable. Figure 5(b) shows a virtual timetable created by the train rescheduling simulator. The four additional trains are indicated by thick red lines. Departures of trains running between Stations A and T will be delayed by more than three minutes until around 11:15.

3.2 Comparison of the number of late-arriving passengers

Using the train rescheduling simulator, we calculated the number of late-arriving passengers for the reconstructed actual timetable and the virtual timetable, and the results are shown in Fig. 6. For almost all thresholds, the number of late-arriving passengers based on the reconstructed actual timetable is smaller than that based on the virtual timetable. Thus, the reconstructed actual timetable is judged to be the better train rescheduling, consistent with the dispatcher’s assessment. In particular, a clear difference appears for

thresholds between 3 and 18 minutes.

3.3 Visualizing the distribution of arrival delay time using a two-dimensional histogram

To investigate the cause of the difference for the number of late-arriving passengers between the reconstructed actual timetable and the virtual timetable, we focus on the distribution of arrival delay time for each passenger under each timetable. Figure 7 shows a two-dimensional histogram of arrival delays, cross-tabulated for all passengers for whom destination arrival times differ between the reconstructed actual and the virtual timetables [6].

Overall, a concentration of warm-colored cells on the left side of Fig. 7 indicates that many passengers arrive earlier under the reconstructed actual timetable compared to the virtual timetable.

Furthermore, the warm-colored squares, which demonstrate significant differences in arrival delay time, are believed to be the primary cause of the difference in the number of late-arriving passengers shown in Fig. 6. Specifically, these are passengers for whom the arrival delay time was less than 5 minutes in the reconstructed actual timetable, but for whom arrival delay time was between 15 and 20 minutes in the virtual timetable. In the next section, we analyze which trains these passengers use and identify the trains that contribute significantly to increased delays, as well as the associated train rescheduling actions.

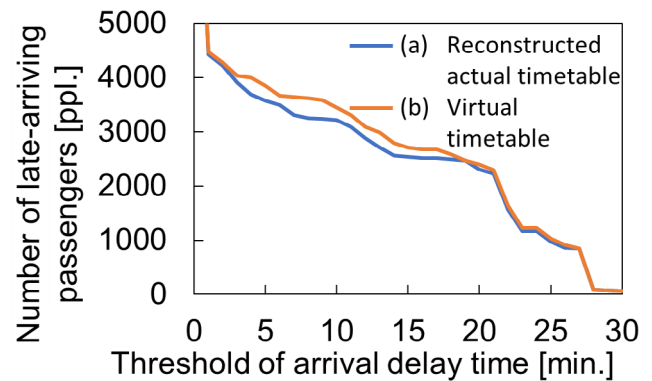


Fig. 6 Comparison of number of late-arriving passengers between reconstructed actual and virtual timetable

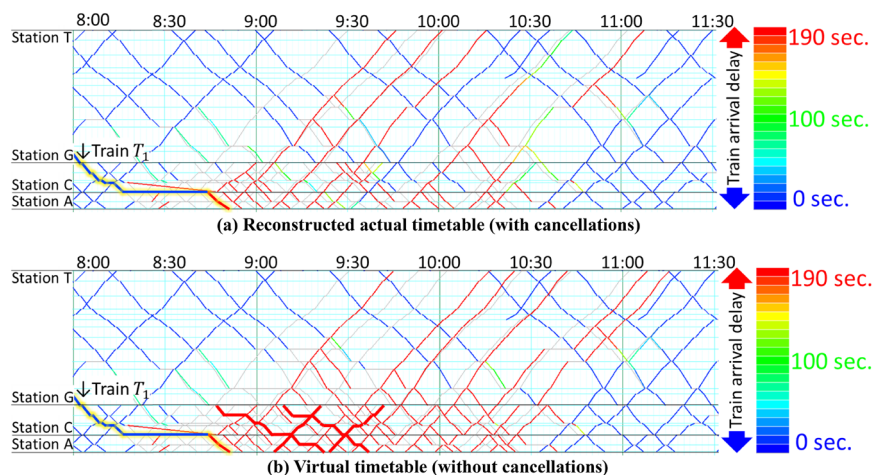


Fig. 5 Timetable colored by train arrival delay

3.4 Method for identifying train rescheduling actions that have a large impact on the number of late-arriving passengers

Figures 8(a) and 8(b) show colored timetable diagrams summarizing the trains used by the 89 passengers identified in the previous section under the reconstructed actual timetable and the virtual timetable, respectively. The four trains shown as thick black lines in Fig. 8(b) are those canceled in the reconstructed actual timetable.

Figures 8(a) and 8(b) show that most of the analyzed passengers are found to use Train T_2 indicated in the diagrams. A comparison of the operating time of Train T_2 in Figs. 8(a) and 8(b) in the virtual timetable shows that the train departing from Station G, which was canceled in the actual timetable, runs between Stations G and A. Therefore, the Train T_2 's departure from Station I is delayed compared to the actual timetable. As a result, Train T_2 's arrival at Station A in the virtual timetable is delayed by an additional 15 minutes and 30 seconds compared to the actual timetable. In other words, the delayed arrival of Train T_2 at Station A will delay the arrival of 83 passengers on the train for which destination is Station A. As a result, the train rescheduling actions in the actual timetable including canceling four trains between Stations G and A and advancing the departure of Train T_2 from Station I, which runs between Stations T and A, were effective in preventing an increase in passenger arrival delay time.

In addition to the example presented in this paper, we con-

firmed that the proposed method can distinguish the train rescheduling that dispatchers consider more effective and identify train rescheduling actions that result in differing evaluation results. Thus, we confirmed the proposed methods are effective for comparing and analyzing multiple train rescheduling plans based on the number of late-arriving passengers on this target line.

3.5 Evaluation metrics used in practice

For the two timetables in Section 3.1, we compared evaluation metrics commonly used in practical traffic control—specifically, time to timetable recovery, the number of canceled trains, and total train delay time—to verify the effectiveness of the train rescheduling plan.

Table 1 shows the calculation results based on the evaluation metrics. When the results based on the reconstructed actual timetable are compared with those based on the virtual timetable, the increase in time to timetable recovery is almost the same (the difference is 0.1 hours). Furthermore, there is generally a trade-off between the number of canceled trains and the total delay time. This is because train cancellations reduce the number of trains used in the calculation, which lowers the total train delay time. Therefore, these two evaluation metrics must be compared simultaneously, so that if the number of canceled trains is about the same, the train rescheduling plan with smaller total delay time is superior. However, when comparing train rescheduling plans with different numbers of canceled trains, such as the case study in this paper, it is impossible to assess which train rescheduling plan is better. This confirms that evaluation metrics used in practice cannot clearly assess between

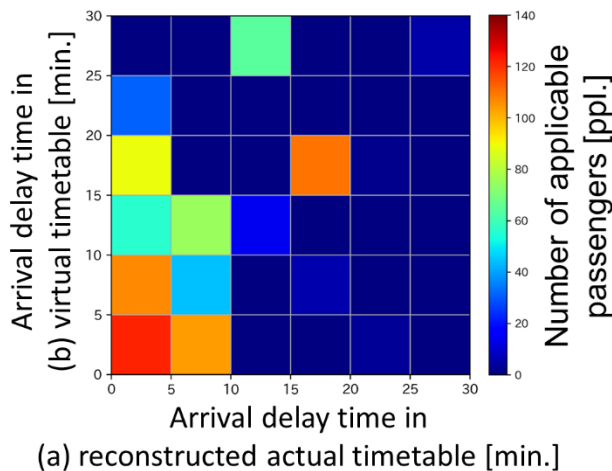


Fig. 7 Two-dimensional histogram

Table 1 Comparison of evaluation metrics

Evaluation metrics	(a) Reconstructed actual timetable	(b) Virtual timetable
Time to timetable recovery [hours]*1	2.4	2.5
Number of canceled trains [trains]	4	0
Total train delay time [hours]*2	29.9	63.0

*1: Time from service resumption until train delays fall below 1 minute

*2: Sum of arrival delays for all stations and all trains

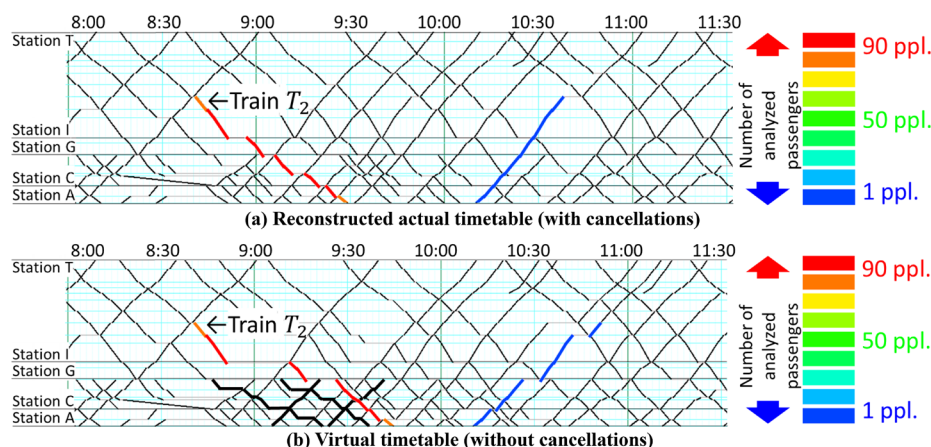


Fig. 8 Timetable based on the number of analyzed passengers

the two plans considered here.

In contrast, as described in Section 3.2, the number of late-arriving passengers proposed in this study was smaller for the reconstructed actual timetable than for the virtual timetable for almost all thresholds. Therefore, it was possible to determine that the reconstructed actual timetable was better, which was consistent with the view of the dispatcher. The results of the above case study confirmed that the number of late-arriving passengers is a better measure of the quality of a service, which cannot be determined by evaluation metrics used in practice.

4. Conclusions

The aim of this study was to improve the quality of train rescheduling review work.

We proposed an evaluation metric that reflects passengers' convenience and can be applied to train rescheduling plans other than those that have actually been implemented. We also proposed an evaluation method using this metric. Specifically, we proposed the number of late-arriving passengers as an evaluation metric for train rescheduling, and demonstrated an evaluation method in which the smaller the number of late-arriving passengers, the better the train rescheduling plan. We also developed a train rescheduling simulator that can calculate a predicted timetable and an evaluation metric by inputting train rescheduling actions. By using these, we constructed a comparison and analysis method for judging the quality of train rescheduling based on the number of late-arriving passengers and arrival delay time. We also constructed a method for identifying train rescheduling actions and trains that cause differences in evaluation results.

As a case study, a train rescheduling simulator was used to create a reconstructed actual timetable and a virtual timetable in which some train rescheduling actions were changed to less desirable ones, and the number of late-arriving passengers was calculated and compared. As a result, it was confirmed that the number of late-arriving passengers was smaller in the reconstructed actual timetable, and this confirms that the evaluation metric was appropriate. In the case study, a line including both single-track and double-track sections was analyzed. However, the applicability of the

proposed method to double-track lines with connections between different types of trains has not been sufficiently verified. Therefore, further verification focusing on such conditions is required. Furthermore, in order to analyze the factors that caused the difference in the number of late-arriving passengers between the two timetables, passengers with large differences in arrival delay time were extracted by using a two-dimensional histogram. By visualizing the number of passengers on each train, it was confirmed that the method makes it possible to extract the train rescheduling actions that caused the difference in evaluation results.

In the future, we plan to verify whether the proposed method can be applied to double-track lines with connections between different types of trains. We also plan to develop an automatic generation algorithm for train rescheduling plans based on the proposed evaluation metric.

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Impact Tests on Railway Vehicle Aluminum Alloy Members Considering Buckling and Shear Fractures

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Two types of impact compression tests were conducted to investigate the impact characteristics of a double-skin aluminum alloy carbody structure used in a conventional railway vehicle. One of the two tests was an overall compression test in which the entire end face of the specimen was compressed. The other was a local compression test in which part of its end face was compressed. As a result of the tests, buckling was the main cause of failure in the overall compression test, whereas shear fractures were the main cause of failure in the local compression test. Finite element analyses were conducted in which several material fracture criteria were used to compare the analysis results with the experimental results. Comparison between the analyses and the tests showed that the fracture criterion based on the ESI-Wilkins-Kamoulakos model was applicable in the analyses of the compression tests.

Key words: carbody structure, aluminum alloy, compression test, FE simulation, fracture criterion

1. Introduction

In Japan, measures are continuously being implemented to improve safety. For example, grade separation to eliminate level crossings and the introduction of obstacle detection devices at level crossings has significantly reduced the number of level crossing accidents involving collisions with automobiles from about 700 in 1987 to about 100 in 2016. Nevertheless, level crossing accidents can still occur if cars accidentally enter crossings after the barriers have been activated. Therefore, assessing and improving the crash-worthiness of railway vehicles in view of possible accidents such as this, remains important.

Numerous collision scenarios between railway vehicles and automobiles, defined by the collision object, collision location, and impact velocity, can be envisaged. Since comprehensive investigation of these scenarios through full-scale testing is impractical, analytical approaches are a useful alternative. Accordingly, it is essential to develop analytical models capable of accurately simulating the impact behavior of carbody structures.

Conventional passenger railway vehicles in Japan predominantly use stainless steel carbody structures, followed by aluminum alloy structures. Because the structural configurations differ substantially between stainless-steel and aluminum alloy carbodies, analytical models must be developed to represent the specific structural characteristics of each type. Among analytical studies on the impact behavior of carbody structures for conventional railway vehicles in Japan, research has predominantly focused on stainless steel structures [1, 2]. In contrast, studies on the impact behavior of aluminum alloy carbody structures have been conducted mainly for export vehicles and have focused on energy-absorbing structures [3, 4]. These structures are designed to deform intentionally, and detailed investigations of the impact behavior of the carbody structure itself are limited to studies focusing on single-skin carbodies [5]. No studies have been reported on double-skin structures, which are predominantly used in recently manufactured vehicles.

When analyzing level crossing accidents, actual collision conditions vary widely. These include situations in which a railway vehicle experiences impact loads over a wide structural area, as well as situations involving localized loads, such as collisions with cargo carried by an automobile. There have been cases in which contact with cargo shaved off part of an aluminum alloy carbody structure, resulting in shear failure. Therefore, to investigate the impact behavior of carbody structures in level crossing accidents in detail, it is necessary to consider fracture characteristics associated not only with buckling deformation but also with shear failure modes. To handle such complex fractures of aluminum alloys in analysis, selection of appropriate fracture criteria is important. Examples of fracture criteria applied in analyses include a criterion based on a threshold of equivalent plastic strain [3, 5], and a criterion using Gurson–Tvergaard–Needleman model [6] (hereafter referred to as the GTN model) [4, 7], which represents void nucleation and growth during plastic deformation in ductile materials. However, only a limited number of studies have discussed which fracture criterion should be used for level crossing accidents, with supporting load–displacement curves and related evidence. In addition, fracture criteria effective for the shear failure modes considered in the present investigation have not been clearly identified.

This study aims to develop an analytical model capable of simulating the fracture behavior of aluminum alloy railway carbody structures in level crossing accidents. As a preliminary step, impact tests were conducted under multiple compression conditions using double-skin structural components (A6005CS-T5, roof members), in order to clarify both buckling deformation and shear-fracture behaviors. In addition, finite element (FE) simulation corresponding to these tests was conducted and compared with the experimental results. Several fracture criteria were compared, and their effects on the impact deformation and fracture behavior under each compression condition were clarified. The results demonstrate that judicious selection of fracture criteria yields excellent agreement with experiments for both buckling deformation and shear fractures. The con-

tent of this paper is based on Ref. 8, with partial additions and revisions.

2. Compression tests

2.1 Testing methodology

Figure 1 shows the specimen used in this study. It is a hollow extrusion made of aluminum alloy A6005CS-T5, and contains no welded joints. The specimen measures approximately 380 mm in width, 90 mm in height, and 300 mm in depth, and its mass is approximately 2 kg. Although details are omitted in this paper, coupon specimens were prepared from the same extrusion and subjected to uniaxial tensile tests to obtain fundamental material properties. Figure 2 shows a schematic of the test apparatus. The apparatus consists of a cart, a collision plate/block, and a rigid wall. By placing the specimen between the cart and the collision plate/block, impact tests can be conducted. Four load cells are installed between the rigid wall and the collision plate/block to measure the force during deformation. A laser displacement sensor installed inside the rigid wall measures the distance to the end face of the cart. Because the specimen remains in contact with the cart throughout the test, the cart displacement after impact is regarded as the deformation of the specimen.

Two types of impact tests were conducted: an overall compression test, in which the entire end face of the specimen was compressed, and a local compression test, in which only a portion of the cross-section was compressed. The local compression test was intended to identify damage when the aluminum alloy member is compressed locally. For the overall compression test, a narrower specimen was cut from the original member because the energy required to buckle the member exceeded the capacity of the test apparatus, approximately 7 kJ. On the other hand, for the local compression test, an aluminum alloy plate (A5052, 5 mm thick) was welded to one end of the specimen to enable the specimen to be attached to the cart, as described later in this section. In the overall compression test, one end of the specimen was placed in contact with the collision plate and remained at rest, and the other end was impacted by the cart at a velocity of approximately 10 m/s. In the local compression

test, the specimen was mounted on the cart and impacted against the collision block at an initial speed of approximately 10 m/s. Although mounting the specimen on the cart was initially planned for both tests, preliminary tests for the overall compression test indicated that heat input from welding a plate to one end of the specimen could influence the test results; therefore, mounting the specimen on the cart was not used for the overall compression test. The specimen setup prior to testing is shown in Fig. 3. The sampling frequency of the data recorder was 100 kHz. For each test type, three independent specimens were tested (test No. 1, 2 and 3).

2.2 Test results and discussion

The definitions of force, time, displacement, and velocity used in this paper are as follows. The force was defined as the sum of the outputs from four load cells, and the time at which the force signal began to rise was defined as $t = 0$. The cart position at $t = 0$ was taken as the origin of displacement. The velocity was calculated by differentiating the raw displacement signal. After data acquisition, the recorded force, displacement, and velocity signals were processed using a 1 kHz low-pass filter with phase-delay compensation. The impact velocity of the cart was defined as the mean value of the velocity over the 1 ms interval immediately prior to impact.

2.2.1 Overall compression test

Figure 4 shows the deformation process of the specimen in the overall compression test No. 2, recorded by a high-speed camera. The collision plate was located on the left side, and the cart impacted from the right side. In test No. 2, the major deformation initiated on the cart side (right side). Buckling deformation occurred, and fracture of the plates was observed. In contrast, in tests No. 1 and No. 3, the major deformation initiated on the collision-plate side (left side). The side on which buckling deformation initiated is considered to have varied among tests due to an initial gap between the



Fig. 1 Specimen

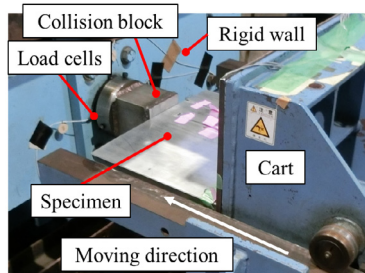


Fig. 2 Schematic diagram of cart, specimen, collision block, load cells, and rigid wall (local compression test)

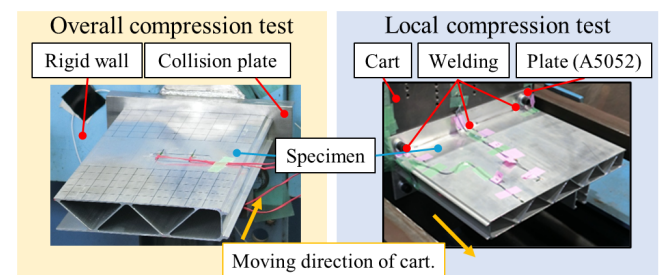


Fig. 3 Installation of specimen

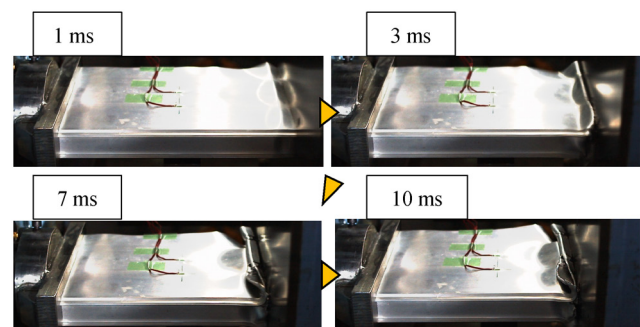


Fig. 4 Deformation process at 1, 3, 7 and 10 ms (overall compression test No. 2)

specimen and either the cart or the collision plate.

Figure 5 shows the specimen shape after compression. Fracture was observed in the plates (outer or inner plates) at the part of the buckled region (indicated by the yellow dashed lines in Fig. 5) and at the roots of ribs (indicated by the red dashed lines in Fig. 5). Figure 6 shows the force–displacement and deformation energy–displacement curves obtained from the tests. The deformation energy was calculated by integrating the force with respect to displacement. In test No. 1, noise was present in the force and displacement signals; therefore, only the intervals unaffected by the noise are shown. Table 1 summarizes the cart velocity, maximum force, maximum deformation energy, maximum displacement, and the measured specimen deformation with a ruler after tests. The maximum force was 294 kN in test No. 3, while the maximum displacement, maximum deformation energy, and maximum springback, which is defined as the difference between the maximum displacement and the measured length, were 50.1 mm, 7.37 kJ, and approximately 2 mm, respectively, all in test No. 2. The specimen deformation ranged from 47 mm to 48 mm, which was in close agreement with the displacement measured by the laser displacement sensor when

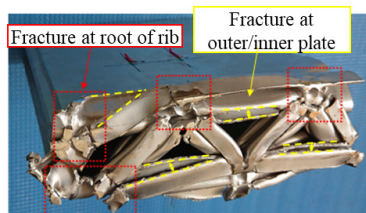


Fig. 5 Specimen shape after compression (overall compression test No. 2)

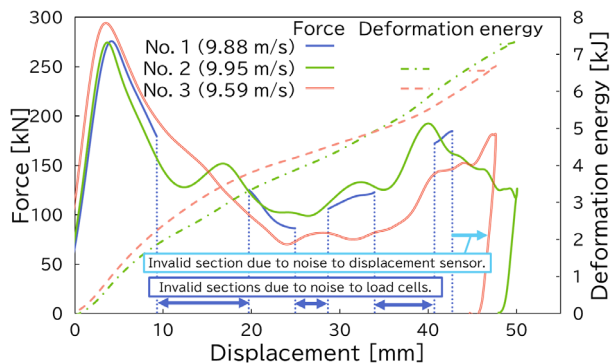


Fig. 6 Force, deformation energy – displacement curves (overall compression test)

Table 1 Overall compression test results

	No. 1	No. 2	No. 3
Cart velocity before collision, m/s	9.88	9.95	9.59
Max. force, kN	276	275	294
Max. deformation energy, kJ	—*	7.37	6.69
Max. displacement, mm	—*	50.1	47.7
Measured specimen deformation, mm	47	48	48

* Maximum deformation energy and maximum displacement are not shown in test No. 1 owing to the noise of the displacement sensor and load cells.

the force decreased to 0 kN. In test No. 3, the specimen deformation was larger than the maximum displacement because the displacement and deformation were measured using different methods. As shown in Fig. 6, multiple peaks were observed in the force history of test No. 2; these peaks are considered to result from the occurrence of additional buckling as deformation progressed.

2.2.2 Local compression test

Figure 7 shows the deformation process of the specimen in the local compression test No. 1, recorded by a high-speed camera. The collision block was located on the left side, and the cart and the specimen moved together from the right side. The plates and ribs near the region which came into contact with the collision block exhibited accordion-like buckling. In addition, in the region which came into contact with the side surface of the collision block, shear-fracture behavior was observed in accordance with the shape of the collision block. Due to the low fracture strain of the aluminum alloy material, this shear fracture occurred during penetration of the block, causing the buckling deformation to concentrate near the contact region. Figure 8 shows the specimen shape after compression. Figure 9 shows the force–displacement and deformation energy–displacement curves obtained from the local compression tests. Table 2 summarizes the cart velocity, maximum force, maximum deformation energy, maximum displacement, and the measured specimen deformation. The maximum force was 158 kN in test No. 3, and the maximum displacement was 78.0 mm in test No. 1. The springback of the specimen was approximately 1 mm. Although the final buckling shapes differed among the tests, the difference in maximum displacement was 8.0 mm, and the maximum force was

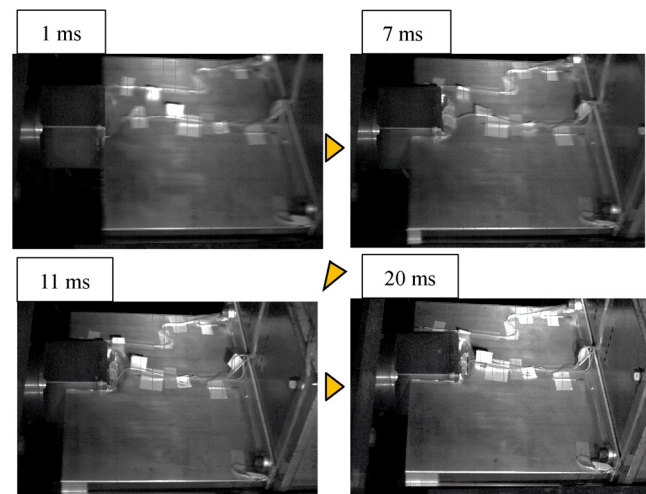


Fig. 7 Deformation process at 1, 7, 11 and 20 ms (local compression test No. 1)

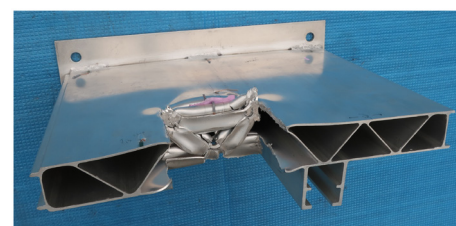


Fig. 8 Specimen shape after compression (local compression test No. 2)

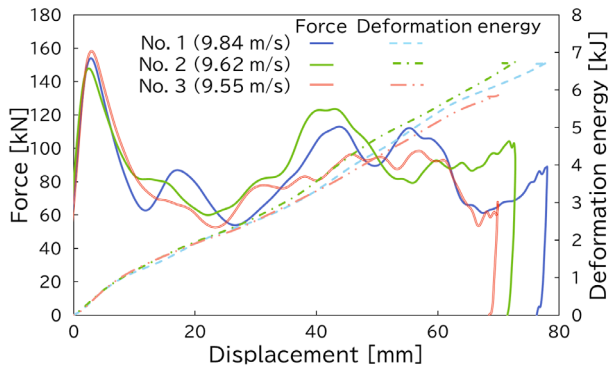


Fig. 9 Force, deformation energy – displacement curves (local compression test)

Table 2 Local compression test results

	No. 1	No. 2	No. 3
Cart velocity before collision, m/s	9.84	9.62	9.55
Max. force, kN	154	148	158
Max. deformation energy, kJ	6.74	6.75	5.86
Max. displacement, mm	78.0	72.8	70.0
Measured specimen deformation, mm	77	72	70

approximately the same for all tests. The maximum force was smaller compared with the overall compression test, which is attributed to the smaller contact area with the collision block and the resulting localized deformation. The post-peak force–displacement response differed among the tests, which is considered to result from differences in the buckling shapes.

3. FE simulations of compression test

3.1 Analysis methodology

FE simulations were performed to simulate the impact compression tests. Schematics of the FE models are illustrated in Fig. 10(a) for the overall compression test and in Fig. 10(b) for the local compression test. The specimen was modeled as elastoplastic shell elements, whereas the collision plate/block and the load cells were modeled as elastic solid elements. For the overall compression test, the end face of the cart was modeled as a rigid body. In contrast, for the local compression test, the nodes located on the cart-side end face of the specimen were connected via links to nodes assigned on the cart's mass, and the cart was modeled as a rigid body. The mesh size was set to a baseline value of 10 mm, considering future application to an aluminum alloy carbody structure model. Material fracture was considered in the specimen model; elements that reached the fracture criterion described later were deleted. Test No. 2 was selected as the baseline case for the overall compression test, while test No. 3—conducted after stable test operation was achieved—was selected for the local compression test. The deformation process shown in Fig. 7 for the local compression test corresponds to test No. 1 because high-speed camera images for test No. 3 could not be obtained due to a camera malfunction. For the overall compression test model, an initial geometric gap between the specimen and either the cart or the collision plate was considered to influence

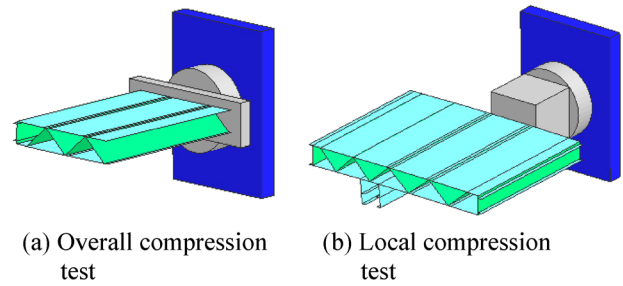


Fig. 10 Analysis model

the deformation behavior; therefore, the cart-side end-face geometry was adjusted to match the actual specimen shape. As a result, the large deformation observed on the cart side in the experiment was also simulated in the FE simulation.

When analyzing the behavior of aluminum alloy members under impact compression, selecting an appropriate fracture criterion is essential. As relatively simple approaches for determining material fracture in FE simulation, a method using a threshold of equivalent plastic strain at fracture (hereafter referred to as the “strain-based criterion”) and a method that judges fracture when the thickness reduction ratio of an element reaches a prescribed threshold (which accounts for fracture only under tensile deformation; hereafter referred to as the “thickness-based criterion”) can be used. More detailed approaches include, in addition to the GTN model described above, the Johnson–Cook model [9], which expresses the fracture strain as a function of strain rate and stress triaxiality. In previous studies [4, 7], the GTN model was applied to aluminum alloys, and its parameters were identified from uniaxial tensile test results. However, strict identification of the parameters requires uniaxial compression tests; therefore, reliable identification of the GTN model parameters was considered difficult in the present study. Moreover, introducing the Johnson–Cook model requires additional material tests to determine the fracture strain under multiple stress-triaxiality conditions. Accordingly, this study focuses on the ESI–Wilkins–Kamoulakos (EWK) model, for which parameter selection is relatively simple. This model was developed based on the Wilkins model proposed by Wilkins, who investigated crack initiation and propagation. At the element level, as expressed in (1), the stress is decomposed into a hydrostatic stress component (σ_m) and deviatoric stress components (s_i , with $s_1 > s_2 > s_3$), and the damage variable D is calculated by combining these terms.

$$D = \int \left(1 - \frac{\sigma_m}{P_{lim}} \right)^\alpha \left(2 - \max \left(\frac{s_2}{s_1}, \frac{s_3}{s_2} \right) \right)^\beta d\epsilon \quad (1)$$

P_{lim} , α , and β are EWK parameters, representing the threshold for the hydrostatic component and the exponents for the hydrostatic- and deviatoric-stress terms, respectively. The term $d\epsilon$ denotes plastic strain increment. In this study, the EWK model was adopted, and the input parameters (local normal strain and local shear strain) were determined through comparison with the results of the member tests. The EWK parameters used in the present analysis were $P_{lim} = 384$ MPa, $\alpha = -2.34$, and $\beta = 1.47$. In this analysis, in addition to the fracture criterion based on the EWK model (hereafter referred to as the “EWK criterion”), a thickness-based criterion (with the threshold identified from uniaxial tensile tests) and a strain-based criterion were considered, and the results were compared. For the strain-based criterion, three models were examined: one using the value identified from uniaxial tensile tests (0.162) and two using prescribed thresholds of 0.5 and 0.6.

3.2 Results and discussion on FE simulations

3.2.1 Overall compression test

Figure 11 shows the force–displacement curves, Fig. 12 shows the deformation energy–displacement curves, and Fig. 13 compares the deformed shapes obtained using each fracture criterion together with the experimental result (test No. 2). Hereafter, the number in parentheses following the strain-based criterion denotes the threshold value. In the simulation using the strain-based criterion (0.162), deformation progressed excessively. Therefore, Fig. 13(c) shows the shape at a displacement of 52 mm.

For the thickness-based criterion, the maximum displacement was in close agreement with the experimental result, and the deformation energy–displacement curve also agreed well with the experiment up to a displacement of 36 mm. Although fracture occurred in the plates and at the rib roots in the experiment, almost no element deletion was observed in the simulation using the thickness-based criterion. Despite this difference in fracture mode, the simulation and experiment agreed well in the overall compression test. This is because fracture occurred only in regions with severe buckling deformation, thus having a limited influence on the global compressive deformation.

For the EWK criterion, deletion of some elements led to more deformation than that observed in the experiment. Nevertheless, the EWK criterion is considered reasonable

because it partially simulated the fracture observed in the specimen, and the error in the maximum displacement was 11%, which is sufficient for practical use. For the strain-based criterion, using the threshold identified from the uniaxial tensile test (0.162) resulted

in an excessively large maximum displacement. Compared with the other fracture criteria, the strain-based criterion deleted elements more readily, resulting in lower deformation resistance and larger deformation. When the thresholds for the strain-based criterion were set to 0.5 and 0.6, the maximum displacement remained larger than in the experiment; however, fracture at the rib roots and buckling behavior observed in the experiment were simulated. To obtain deformation modes comparable to the experimental results using the strain-based criterion, the threshold must be set to a value at least approximately three times larger than that identified from the uniaxial tensile test. In the simulations using the strain-based criterion, large fluctuations in the force were observed at intervals of approximately 10 mm in displacement, regardless of the threshold value. This interval is considered to correspond to the mesh size of the analytical model (approximately 10 mm). That is, the force fluctuations are attributed to element deletion when the fracture criterion was met.

3.2.2 Local compression test

Figure 14 shows the force–displacement curves, Fig. 15 shows the deformation energy–displacement curves, and Fig. 16 compares the deformed shapes obtained using each fracture criterion with the experimental result (test No. 3). In the simulation using the strain-based criterion (0.162), deformation progressed excessively; therefore, the deformed shape at a displacement of 82 mm is shown in Fig. 16(c). In the simulation using the EWK criterion, shear fractures along the collision block and buckling deformation behind the impact surface were well simulated. The error in the maximum displacement was within 9%, and the force–displacement response was generally in agreement with the experiment. The deformation energy–displacement curve also agreed well with the experiment up to a displacement of 70 mm. In contrast, in the simulation using the

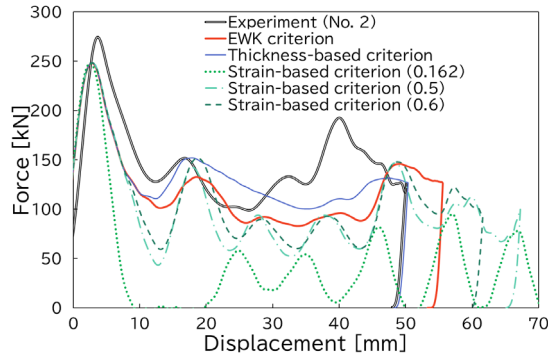


Fig. 11 Force – displacement curves (overall compression test and analyses)

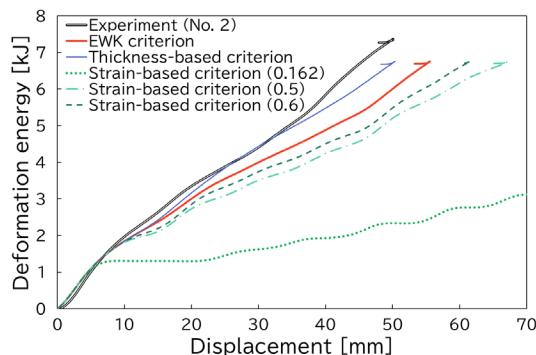


Fig. 12 Deformation energy – displacement curves (overall compression test and analyses)

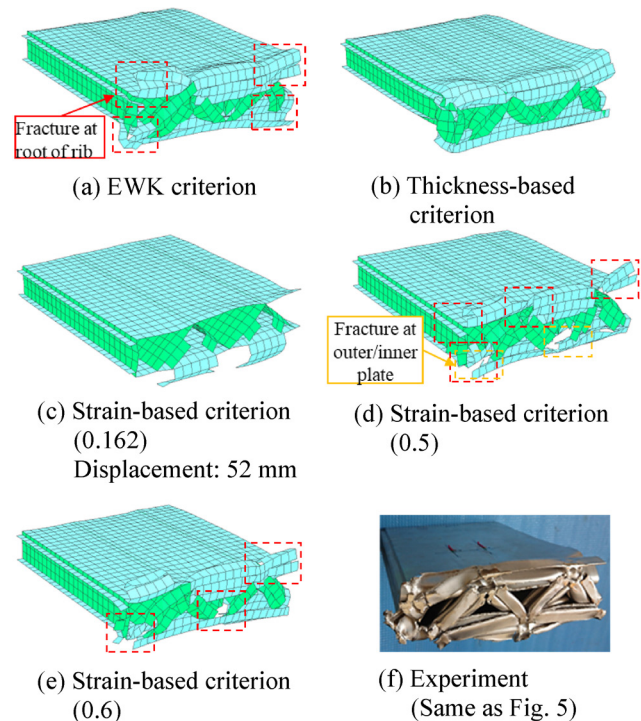


Fig. 13 Deformed shape (overall compression test and analyses)

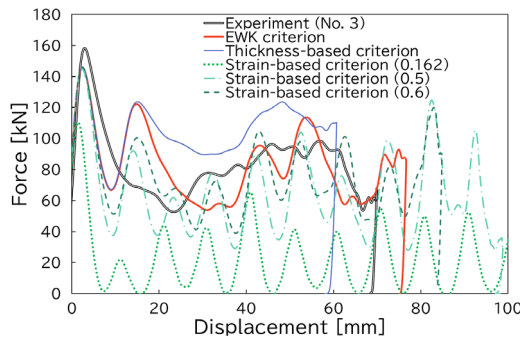


Fig. 14 Force – displacement curves (local compression test and analyses)

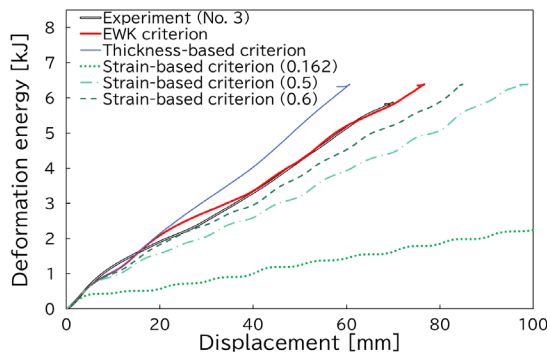


Fig. 15 Deformation energy – displacement curves (local compression test and analyses)

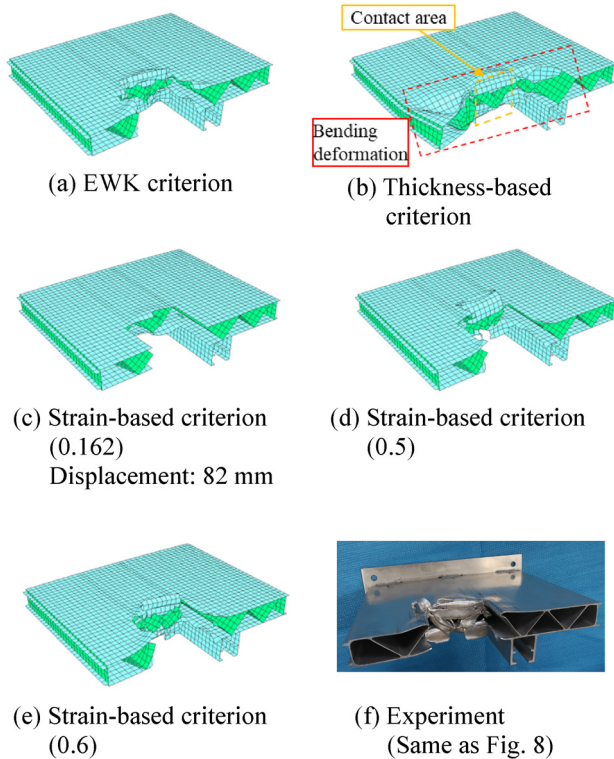


Fig. 16 Deformed shape (local compression test and analyses)

thickness-based criterion, bending deformation occurred in regions away from the contact area with the collision block, and shear fractures were not simulated. This is attributed to the fact that shear deformation does not contribute to a change in element thickness and thus does not reach the fracture criterion. For the strain-based criterion, when the threshold was set to 0.162, fracture occurred with little deformation resistance, whereas setting the threshold to 0.6 resulted in behavior similar to that obtained with the EWK criterion. Although not shown in detail, when the threshold was set to 0.7, bending deformation became dominant and the resulting shape was similar to that obtained with the thickness-based criterion. A second peak in the force at a displacement of 15 mm appeared in the simulations using the EWK and thickness-based criteria, although it was not observed in the experiment for test No. 3. This peak is considered to correspond to the peak observed in test No. 1 at a displacement of 17 mm (see Fig. 9).

3.2.3 Overview of the results

Table 3 and 4 summarize the experimental results and the corresponding FE simulation results obtained using each fracture criterion for the overall and the local compression tests. As indicated in Table 3 and 4, the results are organized in terms of four items: maximum displacement, maximum force, deformation energy, and specimen shape. To compare deformation energy at the same displacement, the deformation energy for the overall compression test is reported at the maximum displacement of the experiment (test No. 2), 50.1 mm. For the local compression test (test No.3), it is reported at 60.7 mm, which corresponds to the maximum displacement in the simulation using the thickness-based criterion. The percentage values shown in parentheses indicate relative errors divided by the experimental results. In Table 3 and 4, the item showing the smallest difference from the experimental result is highlighted with a red background. For the strain-based criterion (0.162), the displacement exceeded 100 mm in both the overall and local compression tests; therefore, the value is denoted as “>100”.

3.3 Discussion of the methodology to select fracture criteria

Section 3.2 examined how the results change with the fracture criterion used in FE simulation of the overall compression test and the local compression test. Based on these results, this section summarizes the fracture criteria that should be used when investigating the impact behavior of aluminum alloy members in numerical analyses. For the strain-based criterion, when the threshold was determined through trial and error based on the experimental results, the required value exceeded three times the actual equivalent fracture strain for the analytical model with a mesh size of 10 mm; as a result, the force in the force–displacement curve was smaller than that in the experiment. For the thickness-based criterion, appropriate results were obtained when deformation was dominated by bending modes, as in the overall compression test; however, it was confirmed that shear fractures cannot be addressed with this criterion. In contrast, the EWK criterion enabled discussion of fracture behavior associated with both shear and bending, and it is considered particularly effective for investigating impact behavior such as that observed in the local compression test. From these findings, simple modeling based on the thickness-based criterion is not appropriate when fracture is expected to occur in various modes; instead, a more detailed fracture representation is required. Among such approaches, the EWK model was shown to be capable of simulating impact behavior including material fracture with relatively simple param-

Table 3 Comparison between experiment and analyses (overall compression test)

		Max. displacement, mm	Max. force, kN	Deformation energy at displacement of 50.1 mm, kJ	Specimen shape
Experiment (Test No. 2)		50.1	275	7.37	—
EWK criterion		55.6 (+11%)	248 (−9.8%)	6.02 (−18.3%)	Agreed with experimental result.
Thickness-based criterion		50.4 (+0.6%)	248 (−9.8%)	6.72 (−8.8%)	Fracture was hardly observed.
Strain-based criterion	0.162	>100	248 (−9.8%)	2.34 (−68.2%)	(At displacement 52 mm) Buckling was not observed due to elements elimination.
	0.5	67.2 (+34.1%)	248 (−9.8%)	5.22 (−29.2%)	Agreed with experimental result.
	0.6	61.6 (+23.0%)	249 (−9.5%)	5.49 (−25.5%)	

Table 4 Comparison between experiment and analyses (local compression test)

		Max. displacement, mm	Max. force, kN	Deformation energy at displacement of 60.7 mm, kJ	Specimen shape
Experiment (Test No. 3)		70.0	158	5.21	—
EWK criterion		76.6 (+9.4%)	146 (−7.6%)	5.26 (+1%)	Agreed with experimental result.
Thickness-based criterion		60.7 (−13%)	146 (−7.6%)	6.30 (+20.9%)	Shear fracture was not observed. Buckling was widely observed near the side of collision block.
Strain-based criterion	0.162	>100	110 (−30%)	1.47 (−71.8%)	(At displacement 82 mm) Shear fracture was observed. Buckling was not observed.
	0.5	98.8 (+41.1%)	145 (−8.2%)	3.98 (−23.6%)	Shear fracture was observed. Buckling was partially observed.
	0.6	84.8 (+21.1%)	146 (−7.6%)	4.58 (−12%)	

ter settings. Conversely, the strain-based criterion did not provide reasonable results in either the overall or local compression test. When material fracture is not a dominant factor, as in the overall compression test, the thickness-based criterion is considered to be an effective option.

4. Conclusions

To characterize the impact compression behavior of double-skin members used in aluminum alloy carbody structures, impact tests were conducted under multiple compression configurations. The tests clarified not only buckling deformation but also shear-fracture behavior. In addition, FE simulation of the impact tests was performed. By applying multiple fracture criteria and comparing the results, a fracture criterion capable of addressing shear-fracture behavior was identified. Two types of impact tests were conducted: an overall compression test, in which the entire cross-section of the member was compressed, and a local compression test, in which only part of the cross-section was compressed. In the overall compression test, bending-dominated buckling deformation was predominant, and fractures occurred mainly in severely deformed regions of the plates and at the roots of ribs. In the local compression test, the plates and ribs exhibited accordion-like buckling near the contact region with the collision block, while shear fractures developed along the side surface of the collision block following its geometry. The localization of deformation around the

collision block, without propagation to other regions, is attributed to the small fracture strain of the aluminum alloy material. From the FE simulation results, for the overall compression test—where buckling deformation dominated—the influence of the fracture criterion on the global response was considered small, and reasonable results were obtained using both the thickness-based criterion and the EWK criterion. When the strain-based criterion was applied, the fracture strain identified from uniaxial tensile tests did not provide appropriate results, and a threshold value more than approximately three times larger was required. In contrast, for the local compression test accompanied by shear fracture, neither the strain-based criterion nor the thickness-based criterion was able to simulate the shear-fracture behavior. The simulation using the EWK criterion successfully simulated both shear fracture and buckling behavior, demonstrating its effectiveness for investigating impact behavior accompanied by fracture in various modes.

In future work, the EWK model will be applied to analytical models of aluminum alloy railway carbody structures to further clarify their impact behavior.

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Comparison of Driver Gaze Allocation Between GOA2.5 Automatic Operation and Manual Operation in Revenue Service

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A comparative study of driver gaze allocation during automatic and manual operation on a demonstration train was conducted to verify whether forward gaze allocation during automatic operation is equivalent to that during normal manual operation. Forward gaze allocation was measured for three drivers on test trains and 15 drivers on revenue service trains, and data from 11 drivers were analyzed. The results showed that 10 of the 11 drivers had the same or greater forward gaze allocation during automatic operation as during manual operation.

Key words: automatic operation, GOA2.5, gaze distribution

1. Introduction

GOA2.5 is a form of automatic train operation in which an automatic operation crew member rides in the cab with the driver. One of the duties of crew members is to perform an emergency stop when an abnormal situation is detected. This action must be carried out in the same manner as a driver operating under manual control (GOA1) would when an abnormality is recognized. This requirement is particularly important on lines where GOA2.5 is to be deployed, which have level crossings and some station platforms with no screen doors.

To detect abnormalities and promptly initiate an emergency stop, the crew member must direct their gaze forward, in the direction of travel, and maintain an appropriate level of alertness during driving operations.

This paper reports the results of a study that examined whether the allocation of forward gaze during automatic operation is equivalent to that during normal manual operation.

2. Preliminary study using test trains

To measure driver gaze during normal operation on revenue service trains, the measurement setup and sensor attachment must be simple and quick to set up. Previous studies of train drivers' gaze during operation have often used test trains because the equipment required for gaze measurement tends to be large and complex [1][2]. Although measurements were conducted on revenue service trains during the Japanese National Railways era [3], they were limited to a single run for each of two drivers.

The demonstration line examined in this study is operated by one driver. The range of gaze allocation therefore covers a wide area, including the onboard monitor above the driver's console used to confirm passenger boarding and alighting. At the time of this study, few commercially available systems could accurately and easily analyze gaze allocation over such a wide area.

For this reason, a preliminary study was conducted using test trains. Gaze allocation was examined for three drivers using a commercially available eye-tracking system.

2.1 Method

2.1.1 Survey section and participants

The survey was conducted over three days on a demonstration line for automatic train operation. A test train was scheduled to make two round trips a day on the same section. The target section consisted of 11 stations with an interstation travel time of approximately one to two minutes, and the one-way running time was about 35 minutes.

Since eyeglass-type sensors were used, the participants were drivers who did not wear glasses while on duty. One driver was measured per day, for a total of three drivers (hereafter referred to as Driver A, Driver B, and Driver C). All the drivers had over 10 years' driving experience.

2.1.2 Survey conditions

Each day, there was one round trip in the morning and one in the afternoon. There was a break time of approximately 45 minutes between the two round trips. The driving mode (automatic or manual operation) alternated between trips. Because the order of operation could influence the results, Driver A and Driver B performed the first round trip in manual operation, while Driver C performed the first round trip in automatic operation.

2.1.3 Measurement items

Gaze measurements were conducted using two types of devices: Smart Eye Pro (Smart Eye Co., Ltd.; hereafter referred to as the fixed sensor), in which sensors are installed in the cab, and Tobii Glasses 3 (Tobii Technology K.K.; hereafter referred to as the eyeglass-type sensor). The fixed sensor was used during outbound runs, and the eyeglass-type sensor was used during return runs. The measurement setup is shown in Fig. 1.

The fixed sensor uses multiple infrared cameras installed near the driver's console to measure the driver's gaze. By synchronizing with the forward-view camera installed in front of the driver (Fig. 1(a), "Forward-view camera"), the system automatically calculates the driver's gaze allocation within the field of view captured by the forward-view camera. For this reason, it was considered

suitable for investigating gaze allocation during driving. However, the system has some disadvantages. Installation requires a considerable amount of time. Due to the limited number of sensors that can be installed in the cab, areas such as the driving handle and the on-board monitor located above the driver's seat are outside the measurement range.

In contrast, the eyeglass-type sensor is portable and easy to use because it does not require installation in the cab. However, because the forward-view camera is built into the glasses, the captured scene continuously moves with head motion, and the system does not have a function for automatically analyzing gaze allocation.

In addition to gaze measurements, physiological measurements and subjective evaluations were conducted to examine differences in workload between automatic and manual operation. Since the investigator accompanied the drivers throughout the measurements, they may have experienced increased tension. Therefore, the experiment was not suitable for evaluating alertness (sleepiness), which is easily affected by such conditions. Nevertheless, subjective evaluations of sleepiness were obtained for use as reference information for the analysis.

Physiological measurements included heart rate (WHS3, Union Tool Co., Ltd.), pulse rate (OH1, Polar Inc.), and respiration (Zen-Tracker, Koto Co., Ltd.). In addition, at boarding and at the turnaround point, the flicker value and subjective evaluations reflecting fatigue and alertness were measured. For the subjective evaluation, drivers verbally reported their level of sleepiness using a nine-point

Karolinska Sleepiness Scale and their level of fatigue using a numerical rating from 0 to 100. The contents of the subjective evaluation are shown in Fig. 2.

2.2 Results

2.2.1 Gaze allocation measurement

Examples of gaze movement for each driver are shown in Fig. 3. In the figure, the blue line indicates the trajectory of gaze movement, and the circles indicate gaze fixations. The diameter of each circle represents the fixation duration.

As can be seen from this figure, Drivers A and C frequently looked at instruments such as the speedometer during manual operation. In contrast, Driver B showed no clear difference in gaze movement between the forward direction and the instruments during manual and automatic operation.

Because gaze was measured using two types of sensors, the results from each sensor are described separately.

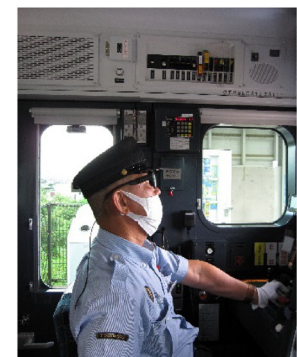
Three representative areas of gaze allocation were defined for the images captured by the forward-view camera of the fixed sensor, and the gaze allocation for each area was examined (Fig. 4(a)).

Target areas for gaze allocation (fixed sensor)

1. **Forward:** front window
2. **Instruments:** instruments on the driver's console, such as the speedometer and pressure gauges



(a) Fixed sensor (Smart Eye Pro)



(b) Eyeglass-type sensor (Tobii Glasses 3)

Fig. 1 Eye-tracking measurement setup

Question 1

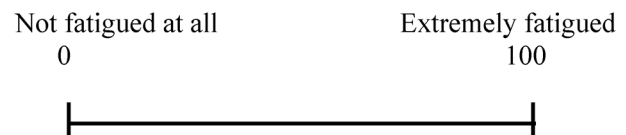
Select the number that best describes your **current level of sleepiness** from the scale below.

KSS	
1	Very alert
2	
3	Alert
4	
5	Neither alert nor sleepy
6	
7	Sleepy
8	
9	Very sleepy (fighting sleep)

(a) Sleepiness (Karolinska Sleepiness Scale; KSS)

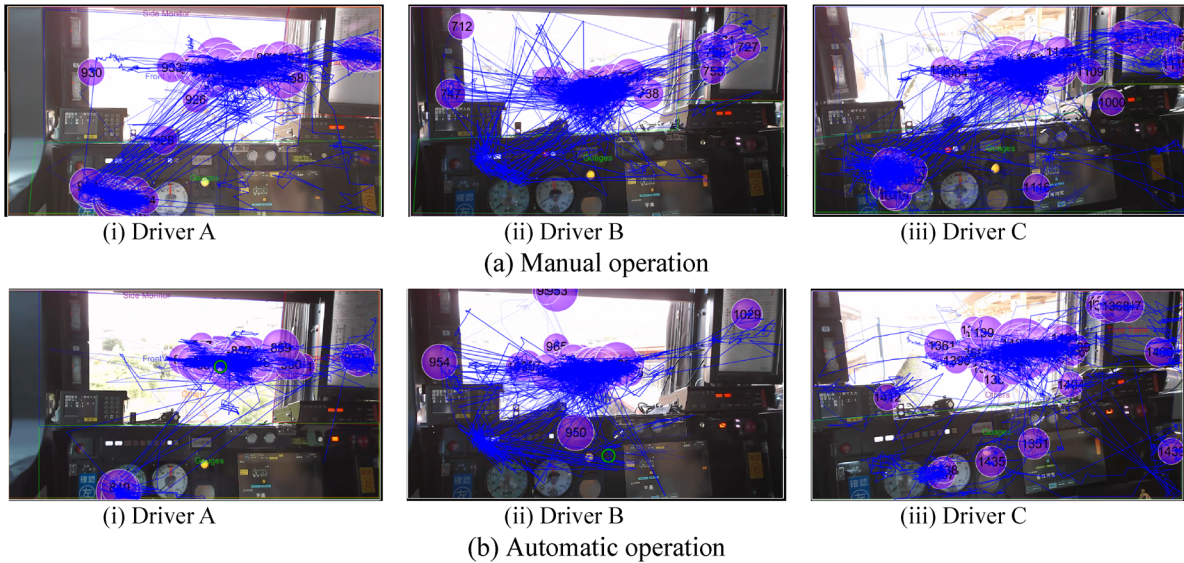
Question 2

Please indicate your **current level of fatigue** using a number from 0 to 100.



(b) Fatigue

Fig. 2 Subjective evaluation of sleepiness and fatigue



Note: The blue lines indicate gaze movement trajectories, and the circles indicate gaze fixations. The diameter of each circle represents the fixation duration.

Fig. 3 Examples of gaze movement and fixation of drivers measured using the fixed sensor (from left: Drivers A, B, and C)

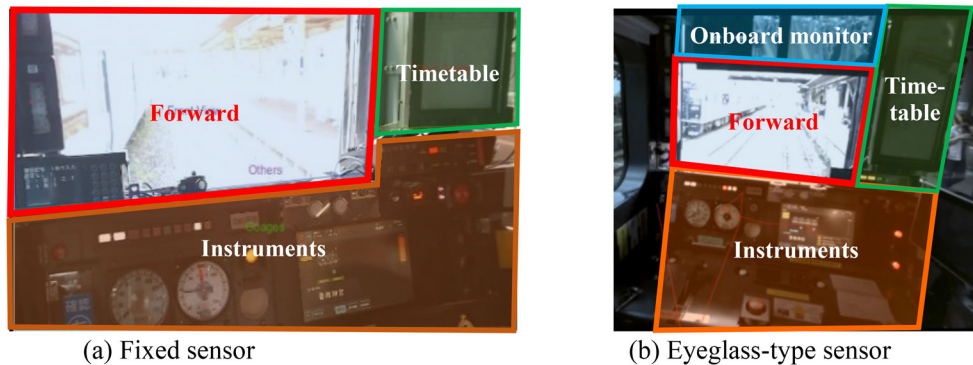


Fig. 4 Target areas for gaze allocation analysis

3. Timetable: timetable installed on the right side of the driver's console

In contrast, the eyeglass-type sensor has a built-in camera that can capture all areas which the driver looks at. Therefore, gaze allocation was examined using following four target areas, including the onboard monitor that the fixed sensor could not capture (Fig. 4(b)).

Target areas for gaze allocation (eyeglass-type sensor)

1. **Forward:** front window
2. **Instruments:** instruments on the driver's console, such as the speedometer and pressure gauges
3. **Timetable:** timetable installed on the right side of the driver's console
4. **Onboard monitor:** monitor located above the front window in front of the driver's seat

The analysis results for each driver obtained using the fixed sensor are shown in Fig. 5. The following analyses compare automatic with manual operation, considering a difference in gaze allocation within 5% (five percentage points) relative to manual operation as equivalent.

As can be seen from the figure, for Drivers A and C, the allocation of forward gaze increased by more than five percentage points during automatic operation compared with manual operation. However, no clear difference was observed for Driver B.

Next, the results measured using the eyeglass-type sensor are described. Since the eyeglass-type sensor does not have an automatic function for analyzing gaze allocation, image analysis was conducted using an analysis program developed for this purpose. The accuracy of the program was verified by comparing its classification results with visual judgments of randomly extracted images from the analyzed video. The agreement rate between the two methods was over approximately 90%.

The analysis results obtained using the eyeglass-type sensor are shown in Fig. 6. Due to a problem with the eyeglass-type sensor measurement for Driver C, only Drivers A and B are included in the results. For Driver A, the allocation of forward gaze increased by more than 5% during automatic operation compared with manual operation, whereas for Driver B no difference exceeding 5% was observed. This tendency is consistent with the results obtained using the fixed sensor. Figure 6 does not exactly correspond to Fig. 5 because the measured sections are different (Fig. 5 shows the out-

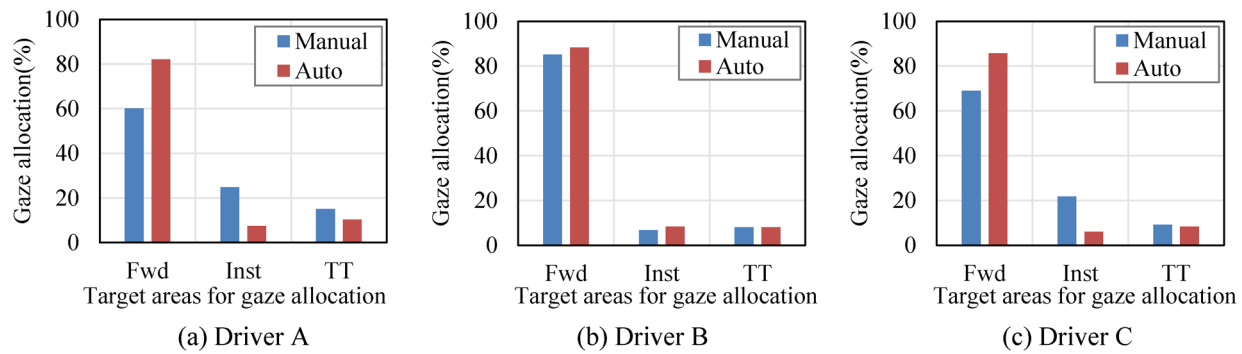


Fig. 5 Gaze allocation ratios for main target areas (fixed sensor)

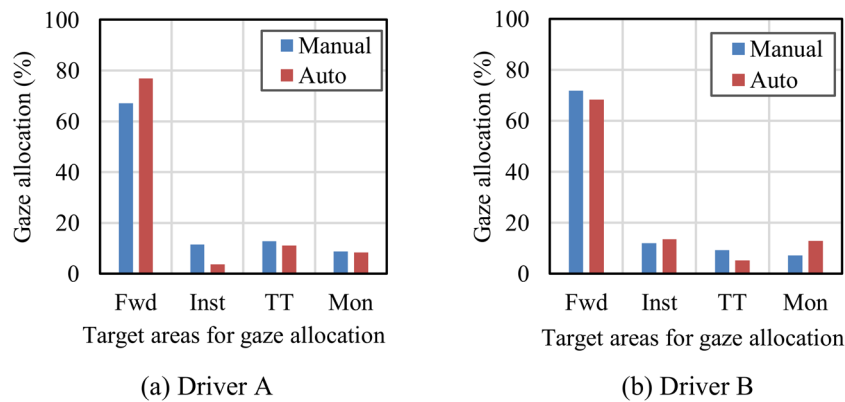


Fig. 6 Gaze allocation ratios for main target areas (eyeglass-type sensor)

bound run, whereas Fig. 6 shows the return run).

Finally, an examination of the results for the onboard monitor reveals that Driver B looked at the monitor nearly twice as often during automatic operation as during manual operation, as could only be measured using the eyeglass-type sensor. The onboard monitor is used immediately after departing from a station to confirm safety on the platform. From the perspective of detecting abnormalities, gaze directed to the onboard monitor is therefore as important as directed forward gaze.

2.2.2 Other measurement items

For the physiological indices, respiration tended to be slightly faster during manual operation overall. However, Driver C, whose operating order was reversed compared to Drivers A and B, had the fastest respiration during the outbound run of the first round trip under automatic operation. No clear difference was observed in heart rate between the two operating modes, but for all three drivers the highest values were recorded during the outbound run of the first round trip.

These results suggest that the physiological indices were more influenced by the order effect—namely the tension immediately after the start of the measurements—than by the difference in operating mode. No clear differences related to the operating mode were observed in the flicker value. Similarly, no clear differences related to operating mode were observed in the subjective evaluations of sleepiness and fatigue.

3. Main survey on revenue service trains

To verify whether gaze allocation during automatic operation is equivalent to that during current manual operation, it is desirable to examine gaze data from as many drivers as possible under normal working conditions on revenue service trains. The preliminary study described in Chapter 2 demonstrated the technical feasibility of analyzing gaze allocation using an eyeglass-type sensor suitable for measurements on revenue service trains. Therefore, in the present survey, driver gaze was measured on revenue service trains using the eyeglass-type sensor, and gaze allocation during automatic and manual operation was compared.

3.1 Method

3.1.1 Survey area and survey participants

The survey was conducted on revenue service trains operating on the same section used in the study described in Chapter 2. The participants were 15 drivers who did not wear glasses, had received an explanation of the study in advance, and had agreed to participate in the measurements. All the drivers had at least five years' driving experience, and 14 of them had more than 10 years' experience. However, four drivers experienced measurement problems; therefore, the analysis was conducted with 11 drivers. The driver age distribution was as follows: four drivers in their 30s, two in their 40s, two in their 50s, and three aged 60 or over.

3.1.2 Survey procedure

When the driver entered the cab, the researcher gave a brief

explanation and asked about the driver's level of sleepiness and fatigue. The sensors were then attached to the driver, and the measurement equipment was set up. This process took no longer than five minutes.

3.1.3 Measurement items, equipment, and attachment method

The measurement items were almost the same as those used in the survey described in Chapter 2. The measurement items and equipment are summarized in Table 1, and the appearance of the equipment and the measurement setup are shown in Fig. 7.

For physiological measurements, the CHET physiological measurement system (Fig. 7, center) was used, which employs wireless sensors modified to enable rapid initiation of measurement on revenue service trains [4]. Developed by RTRI, this system connects multiple commercially available wearable sensors via Bluetooth, enabling the data to be recorded on a measurement terminal (iPad mini, Apple Inc.) together with GPS information and vibration acceleration. The measurement items listed as No. 1–3 in Table 1 were recorded continuously, while subjective evaluations in No. 4–5 were obtained verbally at the start of the duty and at the turnaround point. Since the measurements were conducted on revenue service trains, flicker value measurement, which requires additional time, was not performed.

3.2 Results

3.2.1 Gaze allocation measurement

The analysis was conducted using data obtained while the train was in motion, excluding station stops. Figure 8 shows the comparison of forward gaze allocation between automatic and manual op-

eration for each driver. The vertical axis indicates the ratio of forward gaze allocation during automatic operation, assuming a 100% gaze allocation during manual operation. A value of 100 (red dashed line in the figure) indicates the same gaze allocation as during manual operation. A value greater than 100 indicates that forward gaze allocation is greater during automatic operation than during manual operation.

As a result, for 10 of the 11 drivers (approximately 91%), the value was either 100 or within ± 5 percentage points of 100. In other words, the forward gaze allocation during automatic operation was equivalent to or greater than that during manual operation. For the remaining driver (D04), the decrease was only 5.7 percentage points.

3.2.2 Other measurement items

For the physiological indices, individual driver differences were generally small for heart rate, and no clear tendency related to the operating mode was observed. For respiration interval, there were somewhat more cases in which the interval was longer (i.e., breathing was slower) during automatic operation, while only two of the eleven cases showed slower breathing during manual operation. This tendency was consistent with the results reported in Chapter 2.

No clear differences related to the operating mode were observed in the subjective evaluations of sleepiness and fatigue.

4. Conclusions

To verify whether the allocation of forward gaze by the crew member operating the train automatically during GOA2.5 is equiv-

Table 1 Measurement items, equipment, and methods

No	Measurement item	Equipment / method	Attachment to driver	Remarks (recording method, etc.)
1	Gaze	Tobii Glasses 3	Eyeglass-type sensor	Dedicated recorder
2	Heart rate (RRI)	CHET	Chest band	iPad
3	Respiration	CHET	Abdominal band	iPad
4	Sleepiness	Karolinska Sleepiness Scale	None (oral response)	Recorded by researcher
5	Fatigue	Numerical rating (0–100)	None (oral response)	Recorded by researcher



Fig. 7 Measurement equipment (left: eyeglass-type sensor, center: physiological measurement system CHET) and measurement during operation (right)

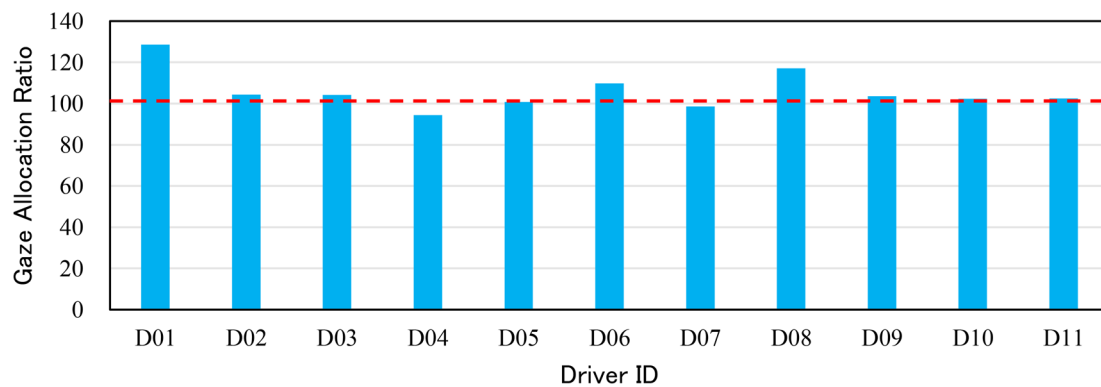


Fig. 8 Ratio of gaze allocation toward the forward direction during automated operation (manual operation = 100)

alent to that of drivers performing conventional manual operation, driver gaze was measured during both automatic and manual operations on a demonstration line for automatic operation. The results confirmed that gaze allocation toward the front of the train during automatic operation was equivalent to or slightly greater than that during manual operation.

However, the ability to detect obstacles or other hazards ahead depends not only on appropriate gaze allocation but also on maintaining attention. Automatic operation has the advantage of reducing the workload associated with driving tasks, which makes it easier to direct the driver's gaze toward the front. On the other hand, although this is not limited to automatic operation, some lines include conditions such as long straight sections or long distances between stations, under which it is inherently difficult for humans to maintain attention. When introducing automatic operation on such lines, it is therefore necessary to consider the possibility that crew members may experience drowsiness. Although not discussed in detail in this paper, we have also conducted preliminary studies on measures that are effective for maintaining alertness and compatible with operational tasks. These studies suggest that practices such as pointing and calling, voluntary verbal responses, and automatic responses to pointing and calling may help maintain alertness [5].

In the future, new working environments that differ from conventional ones are expected to emerge, as exemplified by GOA2.5 automatic operation, and the characteristics of personnel involved in railway operations are also likely to become more diverse. In order to maintain and further enhance railway safety under such conditions, it is important to continue multifaceted studies on the interac-

tion between new systems and humans, taking human characteristics into account.

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Summaries of Papers in RTRI REPORT (in Japanese)

Conventional Evaluation Method for Railway Embankment Settlement due to Damage and Water Infiltration to Small-diameter Pipe Under Track

Takaki MATSUMARU, Taketo SATO, Keisuke SHIMAMOTO, Hiroaki MORI

(Vol.40, No.5, 1-9, 2026.5)

Small-diameter pipes installed in railway embankments are often affected by seepage water when they are damaged, which causes soil outflow and leads to track settlement and cave-ins. In this study, we investigated pillars buried in railway embankments and conducted model experiments to understand the decrease in ground reaction force around the pipes. We also developed a numerical analysis method that takes into account the decrease in ground reaction force and calculated the degree of impact of pipe damage caused by seepage water on track settlement. The results are illustrated in a chart that classified the magnitude of the impact.

3D Reconstruction of Railway Track-side Environments Using Forward-Facing Smartphone Videos Captured on a Train

Kazuhiro KAJIHARA, Hirofumi TANAKA

(Vol.40, No.5, 10-19, 2026.5)

To address labor shortages and an aging workforce, this study investigates the use of train forward-facing videos captured by smartphones for efficient railway track maintenance. For this purpose, 3D spatial models of tracks and surrounding infrastructure were reconstructed using SfM-MVS techniques. The results suggest that these models have the potential to serve as low-cost, alternative or complementary tools for labor-intensive tasks, such as track alignment maintenance and construction surveying of railway facilities.

Development of a Method for Measuring Platform Height and Gap Using Forward-View Train Images

Kodai MATSUOKA, Yuki NAKADA, Sho KATO

(Vol.40, No.5, 20-28, 2026.5)

This study proposes a method for measuring platform height and gap using forward-facing images captured by commercial trains. The method employs ground markers to identify coplanar regions in the image depth direction. An algorithm was developed to automatically estimate platform height and gap from these images. Field tests were then conducted on a commercial railway line to evaluate its accuracy and practical applicability. The results indicated that while gap errors remained within 10 mm at five of the seven locations, height errors exceeded 10 mm at five. The sources of these errors were analyzed, and potential strategies for improvement are also proposed.

Evaluation of Seismic Running Safety of Railway Rolling Stock and Restorability of Continuous PC Girder Bridges with Seismic Isolation Bearings

Kenji NARITA, Munemasa TOKUNAGA, Manabu IKEDA

(Vol.40, No.5, 29-37, 2026.5)

The purpose of this paper is to clarify the running safety of railway rolling stock on continuous girder bridges with seismic isolation bearings in the traverse direction during earthquakes. Dynamic interaction analyses between vehicles and structures indicate that seismic isolation can significantly decrease derailment limit acceleration depending on the vibration frequencies of the piers and tracks. When the yield seismic intensity of the isolators is set to approximately 0.4 and the resulting dominant track frequency is reduced to 0.5 Hz or lower, derailment can be prevented even under

L2-class earthquakes. Moreover, the proposed isolation system effectively suppresses track displacement and reduces pier displacement by as much as 50%.

Sound Pressure and Frequency Characteristics of Whispered Voices Used in Hearing Tests for Railway Drivers

Kei HOSHINO, Noko KURATOMO, Tamaki UEDA, Hiroaki SUZUKI, Naotoshi YAMAMOTO

(Vol.40, No.5, 38-44, 2026.5)

In order to obtain a railway driver's license, drivers have to pass a physical examination. The hearing standard in the physical examination states that "each ear must be able to hear whispered words clearly at a distance of 5 meters", but the specific method of measurement is not clear, nor its relevance to the audiometer test method used by JR. A survey was conducted to understand the characteristics of whispered voices across different phrases and speakers. The results confirmed that, regardless of the phrase or speaker, the peak sound pressure generally occurred around 1000 Hz. Although the overall sound pressure level depended on background noise levels, these findings were consistent with trends observed in previous measurements conducted by railway companies.

A Continuous Measurement Method of Wheel/Rail Contact Position Using Wheel Bending Strain

Yoshinao NOGUCHI, Hiroyuki NISHIKORI

(Vol.40, No.6, 1-10, 2026.6)

This study proposes a method for continuously measuring wheel/rail contact position based on the measurement of the bending strain of wheel. The optimal placement of strain gauges was determined using finite element analysis. An instrumented wheelset was then fabricated with strain gauges attached at those positions, and a static load test was conducted on the wheelset. We have also developed a method for measuring contact positions by solving a nonlinear equation. This equation uses parameters derived from the relationship between load position and strain gauge bridge output obtained in the static load test. Running tests conducted with railway vehicles confirmed that the proposed method can measure the contact position continuously. Additionally, an evaluation using paint confirmed that the measurement accuracy in this test was ± 2 mm.

Estimation of Subgrade Settlement Using Cumulative Damage Theory in High-Speed Sections of the Shinkansen

Kazuki ITO, Yoshitsugu MOMOYA, Takahiro KAGEYAMA, Takahisa NAKAMURA, Hiroyuki KAWANAKAJIMA

(Vol.40, No.6, 11-21, 2026.6)

To ensure operational safety and maintainability of railway systems, subgrades supporting slab tracks are required to effectively mitigate settlement. Furthermore, a reliable model for estimating subgrade settlement is essential to enable higher train speeds. In this study, a calculation model for cumulative settlement was developed based on the cumulative damage theory. A full-scale loading test taking into account the effect of train speed was conducted, and the predicted cumulative settlement was compared with the observed results to validate the proposed approach.

Verification Method for Safety of Railway Rails Against Fatigue Failure Considering Application to Curved Tracks

Mitsuru HOSODA, Tadashi DESHIMARU
(Vol.40, No.6, 22-32, 2026.6)

This study examines the development of a safety verification method for rail fatigue failure in curved tracks. Specifically, we estimate rail stress using an FEM model simulating both rails on railway tracks and develop a method for estimating response values. In addition, we measure rail stress generated when a vehicle travels on a curve on an operating line and analyze the effect of lateral pressure on rail stress. We compare the measured results with the estimated rail stress to verify the validity of the estimation method.

Evaluation Method of Crack Propagation in Rocks Caused by Change in Weather Conditions

Tatsuya HISAKAWA, Takuya URAKOSHI, Shoichi KAWAMURA
(Vol.40, No.6, 33-47, 2026.6)

We have developed a model that evaluates the crack propagation susceptibility in rock by weather change. This model consists of two formulae: one characterizing rock deformation due to changes in temperature and humidity, and the other defining the crack growth rate due to stress changes caused by rock deformation. These formulae were derived from forced deterioration experiments on rock specimens subjected to variations in temperature and humidity. Furthermore, we validated the formulae by using the data on past rockfall occurrences.

Outline of Public Notice and Interpretative Standard of Technical Standard for Fuel Cell Trains

Yuta YASHIMA, Takashi YONEYAMA, Toshihiro MIZUNO, Yasunobu ENDO
(Vol.40, No.6, 48-55, 2026.6)

A public notice establishing the technical standards for fuel cell trains was issued in March 2025. In November of that year, an interpretative standard for these standards was issued. The public notice establishes technical standards for “compressed hydrogen gas” as a hydrogen carrier and “fuel cells” as power generation equipment. These standards were formulated based on safety verification results. The establishment of these technical standards is expected to accelerate the early introduction of fuel cell railway vehicles into practical use.

Development of VR-Based Training Materials for Fundamental Operational Procedures of Railway Conductors

Fumitoshi KIKUCHI
(Vol.40, No.6, 56-63, 2026.6)

This study developed two VR (virtual reality) training materials, — from a first-person and a third-person perspectives — to support railway conductors in learning fundamental safety procedures. The two materials were created using high-resolution 360° video and tested with trainee and senior conductors. Both perspectives were rated as helpful for understanding the required procedures, and VR-based training was generally well accepted. Participants indicated complementary benefits of the two perspectives. These findings suggest that VR materials can serve as a practical tool for facilitating the acquisition of fundamental operational procedures of railway conductors.

A Machine-Learning-Based Flaw Detection Method for Inspection Data of Bogie Parts

Kazunari MAKINO, Yu OGASAWARA
(Vol.40, No.7, 1-10, 2026.7)

Nondestructive inspections such as magnetic particle testing and ultrasonic testing are used to inspect railway vehicle bogie parts. However, identifying flaws from images or waveforms requires significant experience. Therefore, we developed a machine-learning-based method to automatically detect flaws. In magnetic particle testing, false indications caused by surface roughness require discrimination. Our model extracts suspected regions and identifies connected ones as flaws, achieving an accuracy of about 70%. In ultrasonic testing, where surface-shape echoes are confused with flaw echoes, a model trained on simulated waveforms detected internal flaws with an accuracy exceeding 95%. These methods enable uniform identification and de-skilling of flaw detection.

A Method for Detecting Vehicle Abnormality Using a Single-Link Traction Device

Shinichi SAGA, Tadanobu IIDA, Junichi NAKAHASHI
(Vol.40, No.7, 11-20, 2026.7)

To detect abnormalities in railway vehicles, many sensors (such as for acceleration and temperature) are installed on the car body and bogie. However, the growing number of these sensors escalate installation and operation costs. Therefore, there is a critical need to achieve abnormality detection with high accuracy. To address this issue, we investigated an abnormality detection method based on the forces acting on the traction device, with the aim of improving detection performance while minimizing the number of sensors. In this paper, we report the results of tests conducted using actual vehicles as well as dynamo bench tests using actual full-scale bogies.

Long-Term Deformation Prediction of PC Box Girder Bridges Using Strain Analysis Incorporating Moisture Diffusion

Ryu HASHIMOTO, Nobuaki TANAKA, Ken WATANABE
(Vol.40, No.7, 21-30, 2026.7)

In concrete structural design, shrinkage and creep are commonly evaluated using prediction formulas that predict these properties based on concrete mix proportions and climatic conditions, and these formulas are applied to analyses such as the long-term deflection of PC box girders. Although these methods can predict deflection with relatively high accuracy through fiber analysis, they rely on simplified assumptions for drying conditions and therefore cannot sufficiently account for detailed environmental conditions. The authors developed a strain analysis program that focuses on internal moisture behavior in concrete. By applying the analytical results to fiber analysis, it was shown that factors such as ambient relative humidity and water exposure on structural members significantly influence the long-term deformation of PC box girders.

Rapid Restoration Method Using Permanent Large Sandbag for Rainfall Induced Embankment Failures

Taketo SATO, Takaki MATSUMARU, Kazuki ITO
(Vol.40, No.7, 31-41, 2026.7)

Conventional restoration method for rainfall-induced railway embankments failures using large sandbags involves considerable construction costs and prolonged construction duration. This study proposes a permanent structure employing large sandbags and evaluates its seismic performance and cumulative settlement behavior under train loads. Based on these evaluations, an optimized restoration method is proposed. The proposed method was found to reduce construction costs and duration by approximately 80% and 60%, respectively, compared with the conventional approach.

ERRATA

Vol.67, No.2 (May 2026)

Paper title: Engine Condition Diagnosis Using Oil Condition Monitoring System

Page: 149

Added Table 3 (cited in Section 5.3 on page 149) below:

Table 3 Operating conditions of the vehicle running tests.

No.	Operating conditions
1	1N→Coasting
2	2N→Coasting
3	2N→Re-powering
4	3N→Coasting
5	2N→3N→Coasting
6	2N→2N re-powering
7	2N→Coasting
8	2N→Braking→2N re-powering, repeat
9	1N re-powering, repeat
10	2N re-powering, repeat
11	2N→2N re-powering

N: Notch

Re-powering: Coasting, then immediately enter the specified notch.

Editorial Board

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