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Development of a Baseline Digital Twin Model as a Prerequisite for the Digital Twin Definition of a PSC-I Bridge with Model Updating Considering Member Stiffness

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Abstract: Structural health monitoring using various sensors has been widely employed to assess the structural conditions of bridges. In addition, the concept of a digital twin was introduced, which encompasses the life cycle information of a bridge and its real-time data acquisition and utilization. However, the obtained real-time data from sensors primarily reflect the global behavior of the system, making it challenging to identify the root causes of structural changes. For a highly reliable assessment of the global behavior of a bridge, previous history information, that is, a prerequisite model, is required. This study defines a baseline digital twin model (B-DTM) as the stage preceding real-time data utilization in digital twins. The B-DTM is structured into a pre-update phase, which involves the collection of members and system historical data, and a post-update phase, which focuses on model updating. For the case of model updating, due to the inherent complexity of bridge systems, identifying the global optimum for updating remains challenging. In the pre-update phase, a probabilistic approach to historical data such as member stiffness restricts the search domain for model updating, whereas, in the post-update phase, deflection, mode shapes, and natural frequencies derived from load test results representing the real bridge's behavior are utilized to explore the global optimum solution. The proposed B-DTM was validated using collected data and load test results from a PSC-I girder bridge decommissioned after 45 years of service.

Keywords: baseline digital twin model; member stiffness; load test; model updating; global solution; PSC-I bridge

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1. Introduction

For the assessment and maintenance of bridges in use, a structural health monitoring system is utilized to detect the condition of the structure by attaching sensors such as accelerometers, temperature sensors, and strain gauges to the bridge. To extend this concept, a bridge assessment method is proposed based on the definition of a digital twin model (DTM), which receives real-time data from the actual structure, updates the digital model, and evaluates or predicts the actual structure using the digital model.

The data output from a bridge in real time represents a response to the bridge system, and determining the cause of this response when operating a digital twin remains challenging. The components that make up the bridge and the resultant values arising from various issues, such as boundary conditions, connection conditions, and severe damage that cannot

be observed externally, are obtained through the sensors. In addition, because the model does not reflect the current state, it is difficult to predict the response of an actual structure. In Korea, a load-carrying capacity assessment was conducted to assess the condition of a bridge that had been in service for 45 years. The load-carrying capacity is evaluated based on the load test results, and the condition grade was determined through visual inspection. The overall grade was determined to be Grade C, and the final decision was to demolish the bridge because of its insufficient load-carrying capacity. After demolition, a coring test was conducted on the PSC girder and the concrete slab comprising the bridge and a flexural test was conducted on the PSC girder comprising the bridge [1,2]. After the flexural test, the strengths of the PSC girders were compared and were found to be approximately 20% higher than the bending strength evaluated at the design stage. This suggests that the actual behavior of the bridge obtained from the load test conducted for load-carrying capacity assessment is difficult to apply to bridge conditions. To assess the use of data obtained from a bridge, a prerequisite or basis should be prepared.

To define the baseline model of the bridge, it is necessary to first consider how the historical data of the bridge can be stored and managed in terms of individual members for use within the system. The information defined for each member was integrated into the bridge model. However, even if the information for a member is updated by incorporating data, such as actual responses or material properties, the behavior of the bridge may differ from reality because of secondary members, support conditions, modeling errors, material reliability, and other factors. An environment for model updates must be established to reflect the material uncertainty of the members, the actual stiffness of the boundary conditions, and other parameters based on the response data obtained through load tests and similar methods. With these considerations in mind, the baseline twin model is ready to accept real-time data. To provide insights to decision makers through the analysis or prediction of a bridge's current state, a method for "What-if" analysis or simulation has been proposed [3]. This method involves not only predicting potential problems that may arise from the current state through pre-analysis but also deriving results that could occur under assumed future scenarios. This infers the cause when similar results are observed. This approach suggests the possibility of expansion into a digital twin model.

When updating a bridge, it is necessary to update the model according to the behavior of the bridge by considering the characteristics of the actual members. However, owing to the high complexity of bridge systems, finding a global solution for actual bridge systems is extremely challenging. After conducting the load test, the response of the bridge was evaluated by calculating the deflection or acceleration values. The factors affecting these responses were classified as stiffness and mass. In particular, the uncertainty of stiffness is significant because it is influenced by the deterioration and damage that occurs during use and the reliability of material, making it essential to consider these factors. In addition, when updating the model, it is necessary to consider a method for generating an analysis model that can most closely represent the response of the actual physical bridge model. In particular, whether or not to consider the shell or grid analysis model of the slab and the analysis model of the support has a significant impact on the analysis results, which ultimately affects the stiffness of the member being updated.

The structure of this paper is as follows: Section 1 deals with the problem definition of the current DTM configuration and the outline of the main text. Section 2 investigates the research trends on the twin model system diagram and analysis model considered in this study and the objective function configuration of load tests and model updates. Section 3 presents the definition of the B-DTM and the applied methodology. Section 4 deals with the detailed specifications and load test information of a 45-year-old bridge that applied the proposed methodology. Section 5 presents the model update results according to the

objective function configuration, Section 6 contains conclusions, and Section 7 contains further studies.

2. Literature Review

2.1. Digital Twin Model

Tao et al. [4] classified the hierarchical structure of digital twin configuration into six categories by reviewing research on DTs. Model construction involves developing models in geometric, physical, behavioral, and rule-based dimensions. Model assembly involves combining unit-level models to build a system-level model and implement interactions between components. Model fusion is a structure that reflects the characteristics of complex systems more precisely by integrating models with multiple performances. Model verification evaluates the relationship between the model and actual system to ensure reliability, complements errors found during verification, optimizes performance, and systematically manages the generated model and related data to increase future use. According to the concept of the hierarchical structure of the digital twin workplace proposed by Tao et al. [5], digital twin models are classified into unit, system, and system of systems according to their structure and function. The definition of the DTM establishes the maturity level based on its purpose and components and effectively expands the definition of the configuration in line with the maturity of the twin model and its intended use [6].

2.2. DTMs in Bridge Assessment

Studies on information models and DTs, which are expressed using standard data and geometric models, have also been conducted for bridges. Shim et al. [7] defined the geometric model and data for a single building information modeling (BIM) object. Furthermore, other studies have defined models and data at the system level and presented a system [8] and utilization for expansion as a DT based on the composition of BIMs [9,10]. Dang et al. [11] and Nguyen et al. [12] proposed a standard data system for information interoperability between BIM and finite element analysis for expansion as twins, whereas Shim et al. [13] presented a BIM model that receives data for interoperability and reflects deformation. Lee [14] defined the DT for girders at the member level and presented a method for mapping and displaying the results in BIM. Jeon et al. [15,16] defined the process of integrating the data generated at the subsequent system stage based on member DTs and as a system DT. The twin model configuration of a bridge consists of an information model, an assessment model to analyze the actual structure, and a response, visualization, and utilization plan for the actual structure [17].

2.3. Bridge Analysis Model

Ko [18] estimated the live load distribution factor through a relationship analysis of the secondary members and live load distribution factor of the PSC-I bridge. A model that closely approximated the actual deflection was proposed through a comparative analysis of secondary members and structural analysis models. The crossbeam was considered, and the analysis model with the slab as a shell model and girder as a beam model, with each model constrained as a rigid body condition, was evaluated as the most accurate. Jung [19] determined that the method of considering the deflection occurrence as homogeneous in the existing boundary conditions and then setting the boundary conditions when analyzing the deflection of a structure through the spring coefficient produces a value closer to the actual deflection value than the method of setting the boundary conditions. In [20], the grid and finite element models were compared when creating an analysis model for a skew bridge. It was confirmed that when the mesh size was larger than the appropriate ratio, the finite element method tends to overestimate the bending moment and underestimate the

torsional moment compared with the grid model. The finite element model showed higher accuracy than the grid model when setting an appropriate mesh size.

2.4. Model Updating for System

The load test for bridges is divided into static and dynamic load tests, and values such as displacement, strain, and natural frequency are derived based on the measured values of each load test. Various studies have evaluated the conditions of bridges using these observed values. Studies have extracted static behavior data from dynamic behavior data to predict the lateral distribution of a bridge [21], identify the degree of bridge damage [22,23], and create a baseline model to predict long-term behavior [24]. Jung et al. [25] updated the model by configuring the natural frequency and three-mode information in static deflection and the dynamic behavior as an objective function, thereby reducing the discrepancies between the design finite element model and measured data. In addition, the elastomeric bearing of the bridge was considered to calculate the natural frequency more accurately when updating the model [26]. Malekghaini et al. [27] proposed a two-step FE model updating approach for damage identification in PSC-I bridges. The first step used modal-based updating to estimate linear parameters such as boundary conditions, stiffness, and material properties, whereas the second step refined nonlinear parameters using time-domain updating with acceleration data.

3. Methodology

This study proposes an extension method for utilizing a digital twin (DT) for operation by defining the baseline DT model of a bridge. To define the baseline model, we identified model updating, which reduces the gap between the actual behavior of the bridge and the information or analysis model. To minimize the complexity of the bridge system during the update process, we present a method for obtaining a solution that is close to the global solution of the system using two approaches. First, we limited the stiffness range of the members during the update by considering the stiffness information at the member level and the confidence range of the material properties. Second, we configured the objective function set for the update to include static deflection, dynamic eigenmode, and natural frequency. We applied the proposed baseline model by updating a bridge analysis model composed of member units using load test data and material information for each member during the public use of a prestressed concrete (PSC)-I bridge, demolished after 45 years of service.

3.1. Definition of Baseline Digital Twin Model

To assess a bridge, the definition of a DTM must be established. In addition, for expansion to a DTM, the system for the baseline digital twin model (B-DTM) must be considered first. A configuration diagram of the B-DTM is shown in Figure 1. The bridge is evaluated by conducting load tests before and during its use. If the expected behavior of the bridge analysis model is different from that of the actual bridge, the model must be updated by comparing it with the load test results. For model update, the bridge analysis model and the information model or data that constitute the bridge are required. The information model for each member that constitutes the bridge and the materials and the cross-sections that constitute the members serve as the starting point for configuring the bridge B-DTM. Based on this configuration system, the updated model is defined as the B-DTM for the cause and evaluation of the actual response.

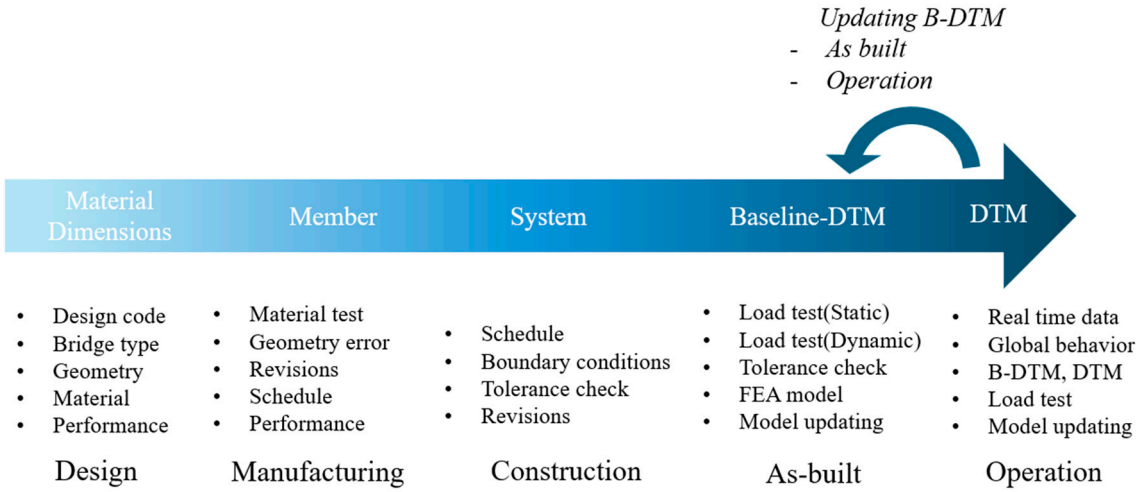


Figure 1. Definition of baseline digital twin model (B-DTM).

The B-DTM must be redefined whenever its behavior differs from the real-time data. To continuously utilize the DTM, the B-DTM must be updated through a continuous collection of historical data, suggesting model updating as a critical step in defining the B-DTM. The behavioral complexity of a bridge system increases depending on the number of spans and main members. Depending on the configuration of the objective function for the model update, whether to search for the area, including the global solution of the system, is determined. As the number of objective functions increases, the possibility of optimization in the section, including the global solution of the system, increases. Additionally, if the search factor for the update is not set to optimize the objective function, the amount of computation increases. As the system's complexity increases, the possibility of local optimization also increases. Accordingly, an efficient objective function configuration for system updates is required. The possibility of finding a global solution for the actual bridge is increased by reducing the search range within the system's domain to avoid local solutions. Figure 2 shows the characteristics according to the objective function and search domain.

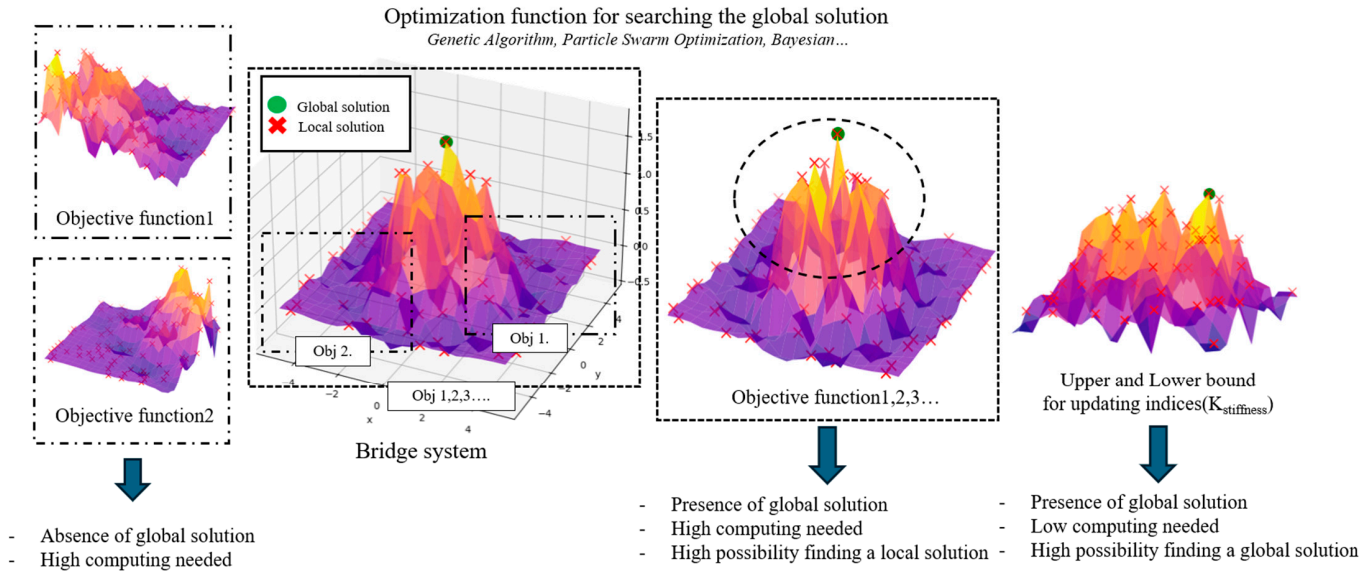


Figure 2. Optimization challenges of system model updating due to complexity.

3.2. Configuration of B-DTM for Bridge Member

To update the load test of the PSC-I bridge system and reflect it in the analysis model, it was necessary to define the stiffness and boundary conditions of the members. The bridge system information and data used in the analysis model were primarily covered to create an information model for each member. The superstructure of the bridge can be classified into structural and secondary members. The structural members are divided into girders and decks, and the secondary members into crossbeams, diaphragms, barriers, and curbs. In addition, the boundary condition affected the stiffness of the system, and bridge bearings were classified as structural members of the B-DTM configuration.

For the main members, such as PSC girders, and reinforcement concrete (RC) slabs, stiffness is influenced by the cross-sectional area and elastic modulus of the material. During the design stage, the elastic modulus of concrete is calculated through a formula from the design code using the concrete compressive strength or written as a function of flexural stiffness based on the deformation information according to the bending of the member. Bridge supports are designed as elastomeric bearings. Because compressive and shear stiffness can vary depending on the specifications of the rubber layer in the elastomeric bearing, the specifications related to the spring coefficient are additionally set. For secondary members, information on the elastic modulus and its management are defined according to the life cycle stage.

The following information was defined when creating an analysis model for the members: For PSC girders, the profile information is defined since the section modulus is determined according to the tendon profile. The type of analysis model for each member was also defined.

3.3. Configuration of B-DTM for Bridge System

The bridge B-DTM is developed based on the information of the member B-DTM, as shown in Figure 3. An information model of the system is created using the information model of the members. The created model acts as a digital model and includes a model update process that reflects the response of the actual bridge to expand to a DT. Model updates necessitate an analytical model of the bridge, based on data from the bridge's information model.

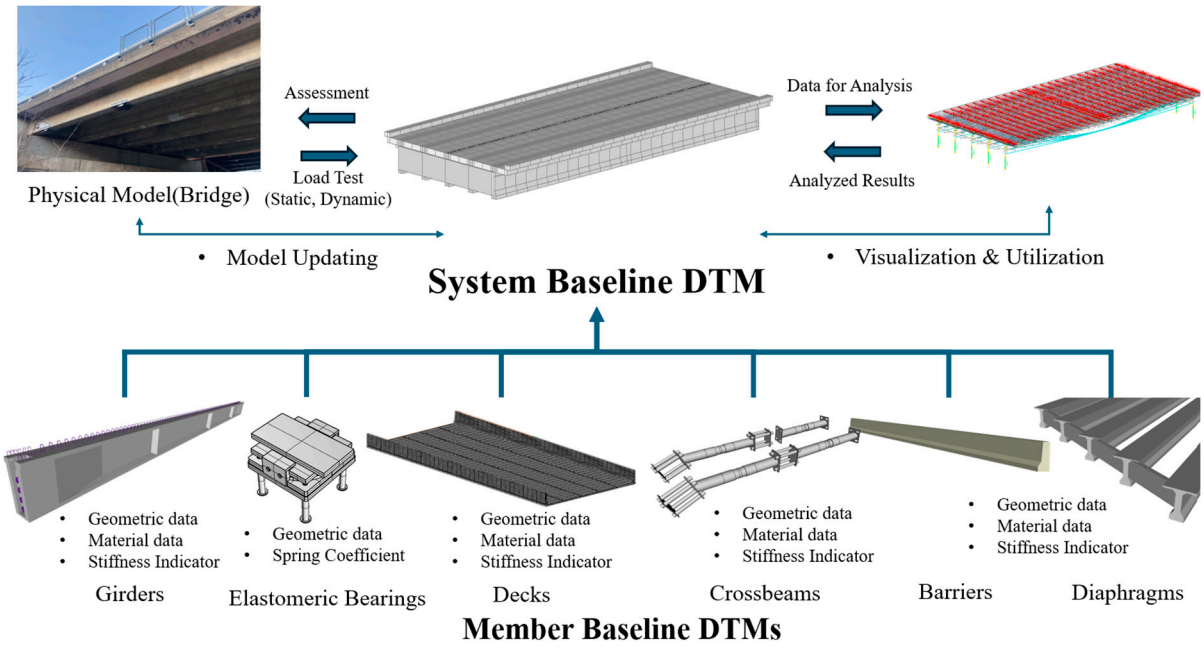


Figure 3. Configuration of B-DTM of bridge system.

3.4. Model Updating for Bridge

A model update for the bridge system integrated into the member units is required to construct a B-DTM that reflects the information measured in the bridge load test. To this end, defining a reference factor that minimizes the difference between the information obtained from the load test and analysis model is necessary. Static and dynamic load tests were conducted along with the bridge load test. In the static load test, the system's stiffness based on the load was confirmed, whereas the mode and impact factor of the bridge were confirmed in the dynamic load test. The factors corresponding to the stiffness information of each member were managed as elements that were updated with the model.

In the static load test, a dead load was applied to the bridge, and data on the deflection or strain of the bridge was obtained to assess the system. As the update factor, the central deflection value of each girder was set as the objective function in the optimization process. During dynamic load testing, the acceleration measurement of the bridge system was subjected to Fourier transformation to measure the natural frequency. The eigenmode was calculated through mode analysis together with the dynamic displacement data. To set the objective function for the eigenvector and natural frequency, a B-DTM system that reflects the dynamic behavior is required. The modal assurance criterion (MAC) index, which determines the similarity between the natural frequency (Equation (1)) and the eigenmode according to the displacement of the system (Equation (2)), is set as the objective function to determine convergence.

$$f_n = \frac{1}{2\pi}\sqrt{\frac{k}{m}}(\mathrm{Hz}), \tag{1}$$

k = stiffness matrix
m = mass matrix

$$\mathrm{MAC}(\varnothing_i, \varnothing_j) = \frac{\left|\varnothing_i^T \varnothing_j\right|^2}{\left(\varnothing_i^T \varnothing_i\right)\left(\varnothing_j^T \varnothing_j\right)}, \tag{2}$$

$\varnothing_i$ = Eigen vector from load test
$\varnothing_j$ = Eigen vector from analysis

When updating the model for static and dynamic load tests, the factors affecting the stiffness of the main members, such as girders, decks, and bridge supports, were targeted, and their values were determined. Secondary members such as barrier walls, crossbeams, and diaphragms were reflected in the analysis but were excluded from the update of the load test conducted immediately after completion. The variables for the elastic modulus of the members or, if necessary, the cross-sectional shape dimensions were reflected in the updating process to optimize the solution for the stiffness of the girders and decks. The shear and compression moduli were set as variables for the elastomeric, excluding the perpendicular and axial boundary conditions in the bridge axis direction.

4. Case Study

4.1. Bridge Properties

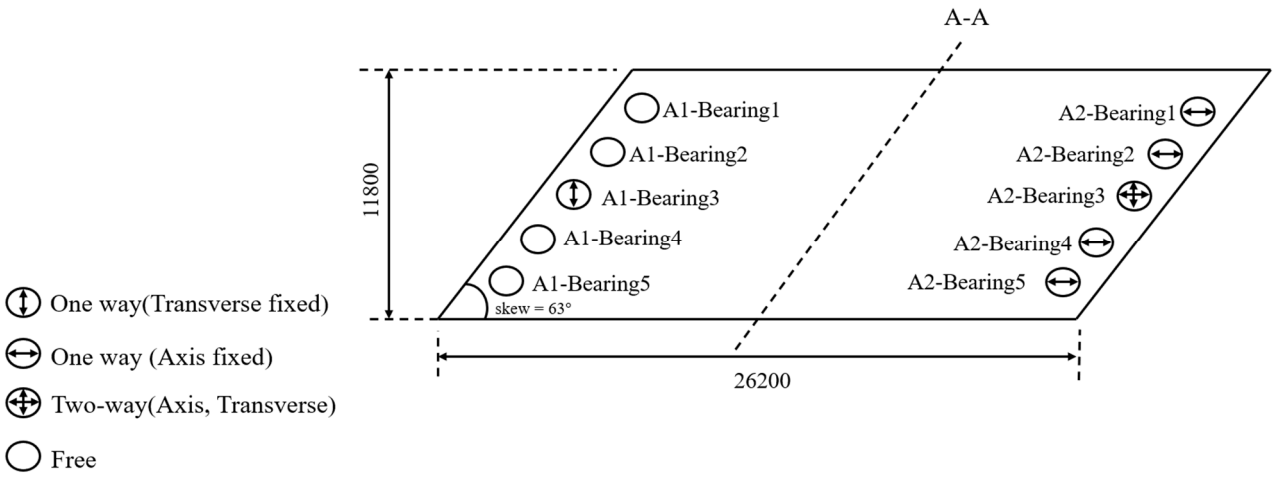
An application example is the PSC-I bridge, which has been in service for 45 years, which is mentioned in Section 1. It is a single-span bridge with a design length and width of 25.2 and 11.8 m, respectively. The bridge is skewed at a 63° angle. The bridge slab is made of RC slab. The girders are made of PSC girders, with five girders arranged at 2.3 m intervals within the span. Diaphragms exist at the ends, and curbs are located at both ends of the bridge width. The bridge conditions before the load test are shown in Figure 4.

The bridge was demolished because the condition grade derived from its safety diagnosis indicated the need for replacement.

Tables 1 and 2 present the specifications of the bridge and girders, respectively. The material properties of each bridge member are listed in Table 3. After demolishing the concrete girders and decks, a coring test was conducted on the members, and the average value and standard deviation of the material test and design values were presented. Additionally, design values for secondary members, such as the diaphragm and curb, are also included. Major maintenance issues reported include cracks and breaks in the concrete pavement, cracks in the PSC girders, corrosion of the supports, and efflorescence at many locations. The reinforcement history includes partial reinforcement of the decks (FRP and steel plate reinforcement) and external tendon reinforcement to cope with the insufficient load-bearing capacity of the outer girders. This study used the load test results with reinforcement removed as the subject.

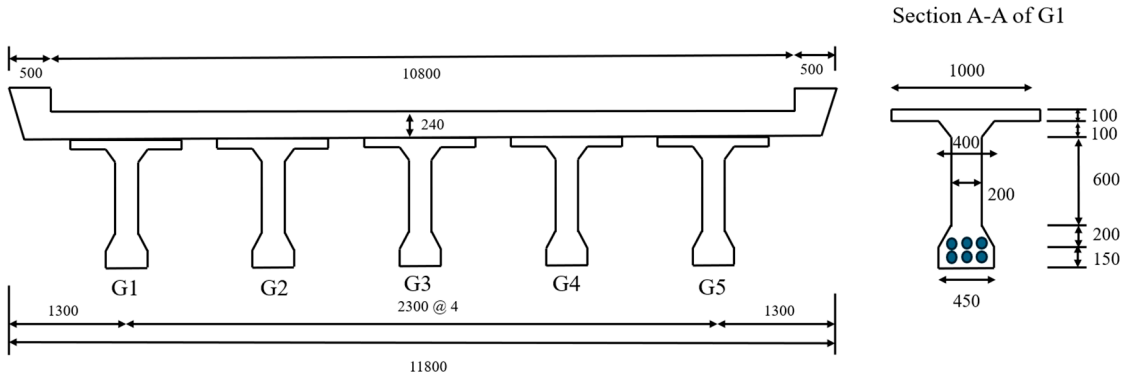


(**a**)

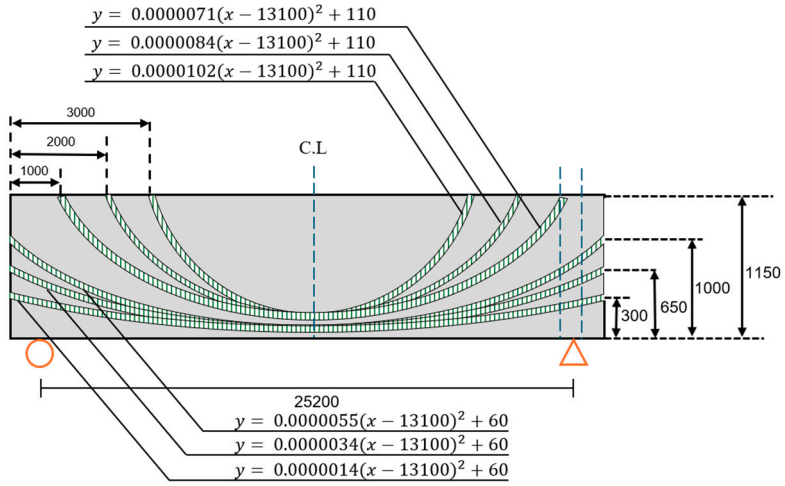


(**b**)

Figure 4. *Cont.*



(**c**)



(**d**)

Figure 4. Bridge description: (**a**) deterioration conditions of the bridge; (**b**) plan view and boundary conditions; (**c**) cross-section; (**d**) tendon profile.

Table 1. Properties of a PSC bridge.

Item	Parameter	Unit	Input Value
General	length	mm	26,200
	width	mm	11,800
	skew	°	63
	Girder number	–	5
	Girder spacing	mm	2300
	Deck thickness	mm	240
Curb	Height	mm	190
	Width	mm	500
Diaphragm	Height	mm	1000
	Thickness	mm	200

Table 2. Detail properties of PSC girder.

Item	Parameter	Unit	Input Value
General	Girder length	mm	25,200
	Girder height	mm	1150
Flange and Web	Upper flange width	mm	1000
	Upper flange thickness	mm	100

Table 2. *Cont.*

Item	Parameter	Unit	Input Value
	Upper haunch width	mm	300
	Upper haunch height	mm	100
	Web height	mm	600
	Web width	mm	200
	Lower haunch width	mm	450
	Lower haunch height	mm	200
	Lower flange width	mm	450
	Lower flange thickness	mm	150
Tendons	Number of tendons	-	6
	Number of strands in a tendon	-	12
	Strand diameter	mm	8
	Duct diameter	mm	45
	Vertical location of tendons at both ends	mm	300/650/1000/1150/1150/1150
	Vertical location of tendons at midspan	mm	60/60/60/110/110/110
	Longitudinal anchor offset at both ends	mm	0/0/0/1000/2000/3000
	Horizontal eccentricities at midspan	mm	−100/100/0/−100/100/0

Table 3. Compressive strength of concrete members.

	Girder (MPa)	Slab (MPa)	Secondary Member (MPa)
Design	43	32	32
Average	37.45	46.48	-
Standard deviation	8.09	12.72	-
Number of Sample	70	47	-

The bridge analysis model was created using OpenSeesPy [28], and the values for the analysis model were entered based on the geometric information of the bridge and its members, such as skew, girder spacing, and member properties. The models for each member comprised beam models for girders, curbs, and diaphragms, shell models for decks, and spring models for supports to model the elastomeric bearings used in bridges. The girders and diaphragms, girders and decks, girders and supports, and decks and curbs were constrained by rigid body conditions. The details of the analysis model for model updating are shown in Figure 5.

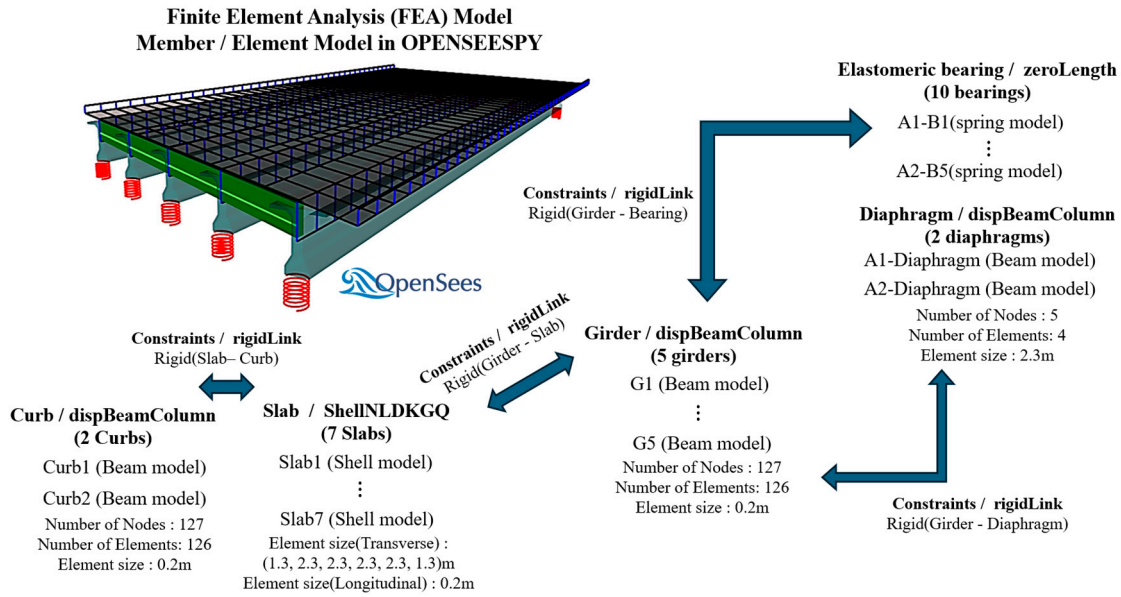


Figure 5. Properties of finite element analysis model for model updating.

4.2. Properties and Results of Load Test

To measure the response of the bridge, linear variable differential transformer (LVDT)s and strain gauges were installed on the underside of all girders in the center of the span for static and dynamic load tests, with accelerometers attached to the center of the span. The exact location of attached LVDTs, strain gauges, and accelerometers on the bridge are shown in Figure 6.

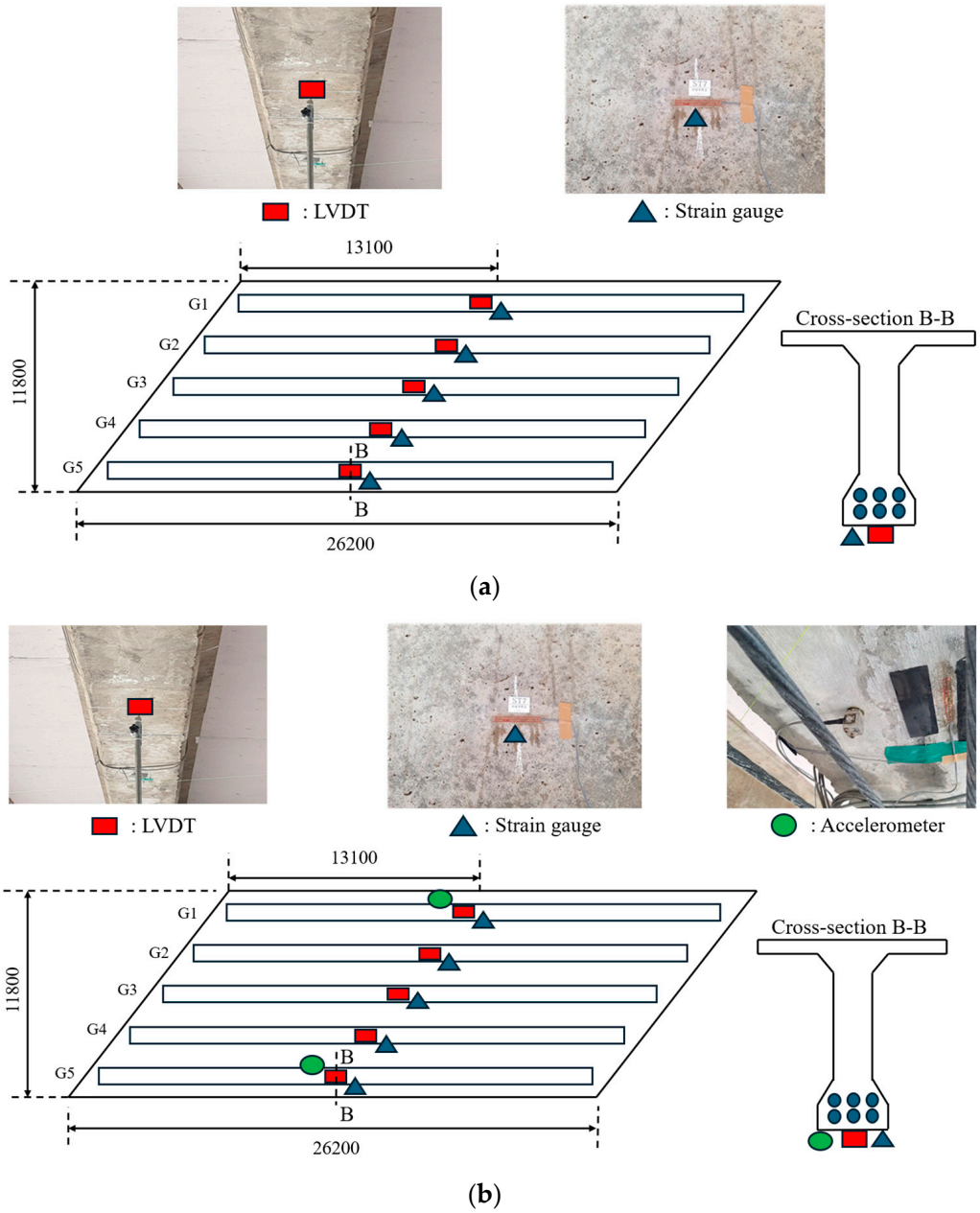


Figure 6. Location of attached sensors for the deflection and dynamic behaviors: (**a**) location of gauges for the static load test; (**b**) location of gauges for the dynamic load test.

The load was applied to a 6-axle truck loaded with soil. The loads on the front wheel, rear wheel 1 and rear wheel 2, were 72.89, 93.34, and 93.34 kN, respectively, totaling 259.77 kN. The truck and axle load locations according to the truck wheels are shown in Figure 7. For the static load test, the dead load was applied at the center of the span and bridge width with rear wheel 1 as the standard. The truck was positioned such that the truck's direction was toward the upbound lane. Data for the dynamic load test

were obtained by driving the same truck in the up and down directions at speeds of 20 to 50 km/h and using free vibration values with the truck load unloaded.

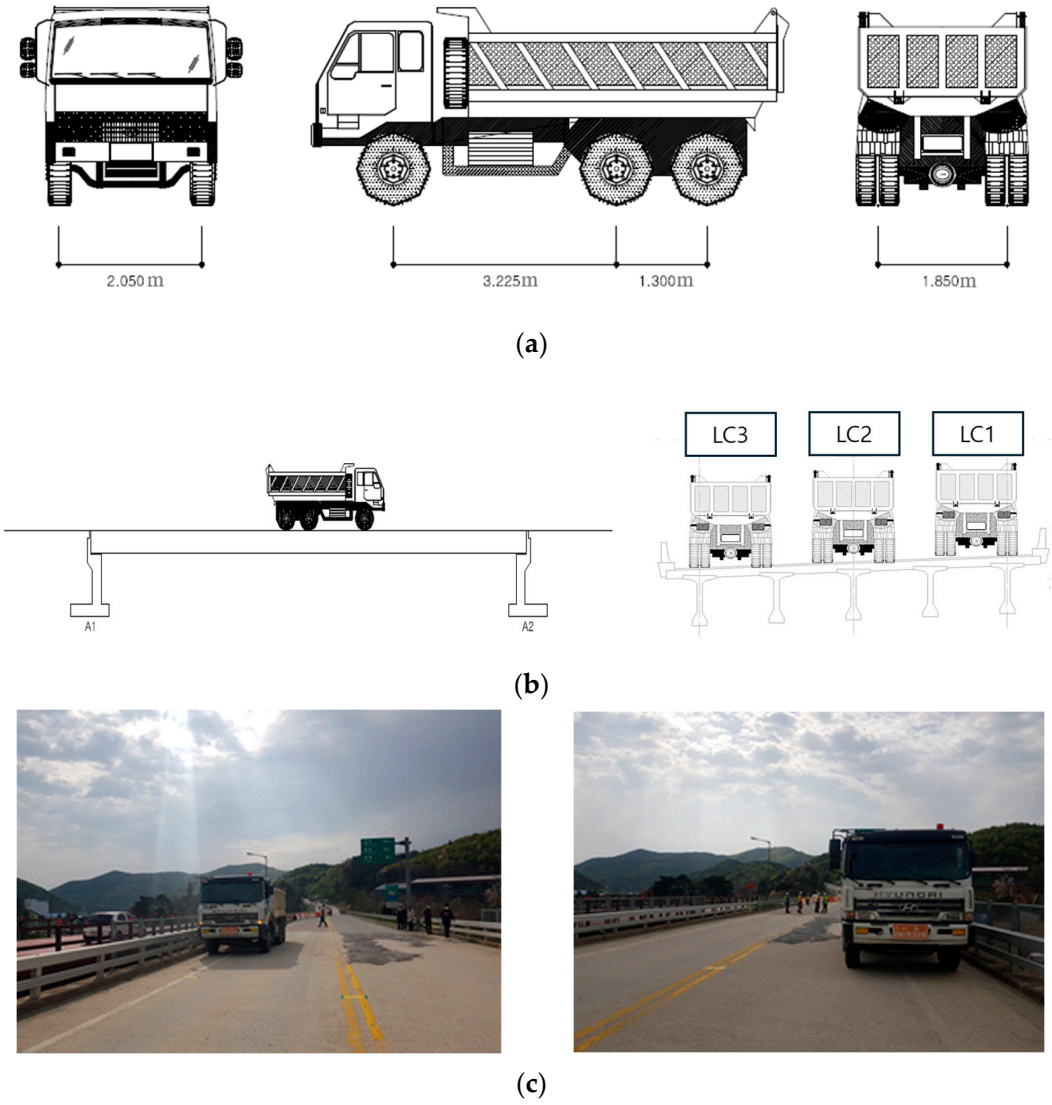


Figure 7. Properties of truckload for load test: (**a**) location of axle load within the truck; (**b**) location of truck on the bridge; (**c**) load test photo in real-site.

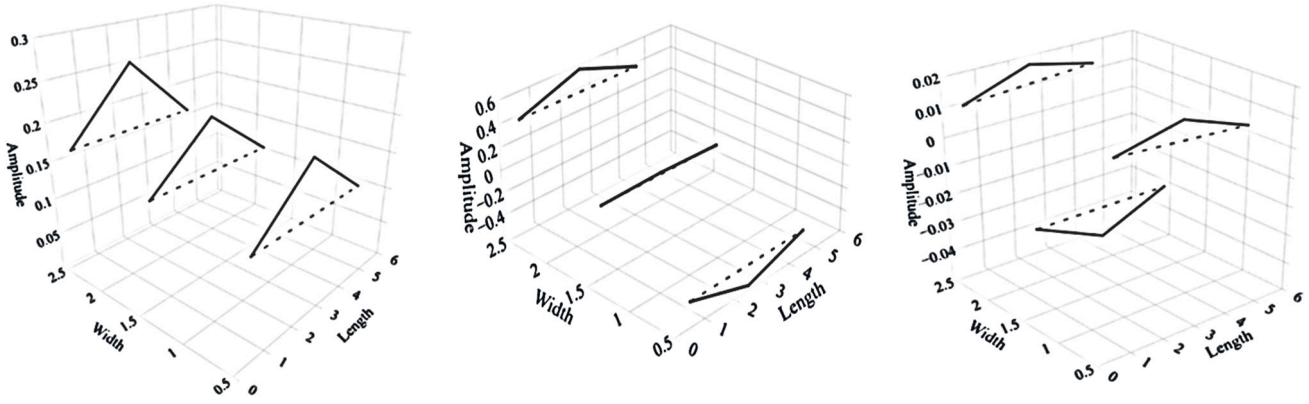
In the load test, the load test was conducted using the steel wires reinforced in the outer girders as a comparison group. For the load cases of LC1 to LC3, the data on deflection and strain were compared at steel wire removal ratios of 0%, 50%, and 100%. When the distribution ratio for the load was compared, the maximum deviation of the strain data between each outer girders (G1, G5) was approximately 50%, and the maximum deviation of the deflection data was 0.2%. Accordingly, the load test data on displacement was used. Among the displacement data, LC2, where the truck load is at the center of the span and width, was set as the reference value. The static deflection data for the load test (LC2) results on the bridge are listed in Table 4. The natural frequencies from the dynamic load test are listed in Table 5, and the modes of the eigenvectors are shown in Figure 8.

Table 4. Deflection of girders after the static load test.

Girder Number	G1	G2	G3	G4	G5
Deflection	1.749 mm	3.404 mm	4.353 mm	3.193 mm	1.822 mm

Table 5. Natural frequencies of the bridge after the dynamic load test.

Mode Number	Mode1	Mode2	Mode3
Natural Frequency	3.956 Hz	4.467 Hz	7.067 Hz



(**a**) (**b**) (**c**)

Figure 8. Eigenmodes of case study bridge: (**a**) 1st mode; (**b**) 2nd mode; (**c**) 3rd mode.

4.3. Results of Design Analysis Model

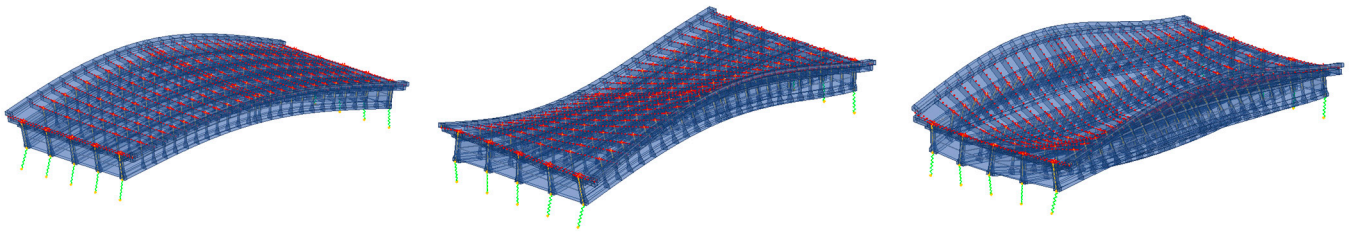
The deflection, eigenmode, and MAC index are derived for the analysis model created using the existing design data. The deflection values and natural frequencies are listed in Tables 6 and 7, respectively. The shapes of the eigenmodes are shown in Figure 9. The MAC index for the mode comparison between the load test and design analysis model is shown in Figure 10. The MAC index shows a similar pattern to the eigenmode in the load test; the eigenfrequency of mode 1 is measured to be slightly smaller, and the frequencies of Modes 2 and 3 are interpreted to be higher by approximately 1–2 Hz.

Table 6. Deflection of girders from design analysis model.

Girder Number	G1	G2	G3	G4	G5
Deflection	2.68 mm	3.34 mm	3.97 mm	3.06 mm	2.15 mm

Table 7. Natural frequencies of the bridge from design analysis model.

Mode Number	Mode 1	Mode 2	Mode 3
Natural Frequency	3.64 Hz	5.71 Hz	9.14 Hz



(**a**) (**b**) (**c**)

Figure 9. Eigenmode of case study bridge from the design model: (**a**) 1st mode; (**b**) 2nd mode; (**c**) 3rd mode.

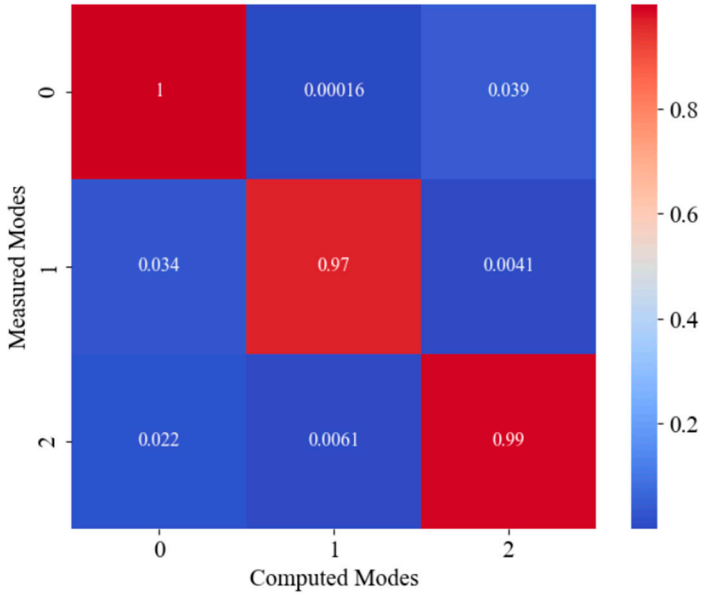


Figure 10. Comparison of MAC index between load test and design analysis model.

5. Model Updating

5.1. General

A linear variable differential transformer or strain gauge is attached to obtain a static response when conducting a static load test. After loading the dead load, data are acquired from the attached sensor. For the dynamic load test, a strain gauge, displacement transducer, and accelerometer sensor were attached, and the impact coefficient, natural frequency, and mode were calculated using the measured values according to the vehicle speed. A model update was performed to minimize the difference between the data acquired from the bridge load test and the bridge information and analysis model. To search for a solution, including a global one, within the area, deflection data were selected for the static load test, and the natural frequency and mode after the load test were selected for the dynamic load test and updated.

5.2. Model Updating Using Static Deflection

5.2.1. Configuration of the Static Model Updating

Model updating was performed using the OpenSeesPy analysis model to match the design data of the bridge with the actual behavior measured from the load test. The load test data from the static load test data collected before the bridge was demolished served as deflection data at the center of each girder (G1–G5).

The material information obtained during the experiment (Table 3) was used to update the stiffness indicators of the member. The optimization search range was calculated based on the elastic modulus of concrete strength. For the girder, the coring results were measured in accordance with the design value. Using the normal distribution of the concrete material, the 95% ($\pm 1.96\sigma$) search range was set at 23,350–32,054 MPa. For the slab, the compressive strength of the concrete was measured by actual coring and was 14.48 MPa higher on average than the design average compressive strength. Because the elastic modulus of the member was predicted to differ from the design value, it was selected as an updating variable. The upper and lower limit of the update was set to 27,472 and 35,348 MPa, respectively. The measured slab thickness ranged between 180 and 300 mm compared with the existing design of 240 mm for reasons such as construction errors; therefore, the corresponding variable was also selected as a characteristic to be updated. A girder member system has a bearing boundary condition, and the compression and shear stiffness values

vary depending on the detailed specifications of the bearing, allowable capacity, and type. The bridge has elastomeric bearings. The compression and shear spring coefficients were used as stiffness values with specifications of 454,343 and 1400 kN/m, respectively, to allow for dead load capacity. The condition assessment of the bridge revealed damage to the bearings and elastic rubber, likely contributing to an increase in the spring coefficient; thus, the corresponding characteristic was also updated. After accounting for deterioration, the range for reflecting the update on the elastomeric bearing was set to a maximum of 1.2 times the design value [29]. When updating the static model, the optimization of the central deflection value of each girder was set as the objective function, as in Equation (3). The elastic modulus of the girders and slabs, slab thickness, and spring coefficient satisfying the objective function were set as in Equation (4), and the values were extracted. Particle swarm optimization (PSO) [30] was used for updating.

$$Obj.Function: \frac{1}{5}\left|\sum_{i=1}^{5} Load\ test_deflection(\delta_i) - Analysis_deflection(\delta_i)\right| \quad (3)$$

$$\underset{E_{girder}, T_{deck}, E_{deck}, K_{spring},}{argmin} (Obj.Function) = E_{girder}, T_{deck}, E_{deck}, K_{spring} \quad (4)$$

5.2.2. Updating Results

Tables 8 and 9 present the factors after static model updating. In Table 9, all directions constrained as boundary conditions had spring coefficients set to 10,000,000,000 kN/m, and no update was performed on the factors. The analysis results based on the design data underestimate the stiffness of the outer girders and overestimate that of the inner girders, particularly that of the third girder. The thickness and stiffness of the slab and girders were adjusted to reduce the difference between the deflection data from the actual load test and the analysis model results. The deflection results of the updated model are listed in Table 10. For girder 1, there is a difference of 0.8 mm between the load test and the updated results; for girders 2 to 5, this difference is approximately 0.2 mm. The updated results exhibit values closer to the load test results than those based on the design data.

Table 8. Updated elastic modulus and thickness after static model updating.

Updating Factor	Description	Initial Value (Design Value)	Updated Value
$E_{c,girder1\sim5}$	Elastic modulus of girder	29,310 MPa	27,442 MPa/32,054 MPa 27,284 MPa/32,054 MPa/31,535 MPa
$E_{c.slab1\sim6}$	Elastic modulus of deck	26,986 MPa	32,286 MPa/29,407 MPa/28,677 MPa 33,122 MPa/27,479 MPa/30,060 MPa
$T_{c.slab1\sim7}$	Thickness of deck	240 mm	300 mm/214 mm/187 mm/180 mm 216 mm/196 mm/255 mm

Table 9. Updated spring coefficient after static model updating.

Bearing Number	Updated Factor (K_x)	Updated Factor (K_y)	Updated Factor (K_z)
A1–B1	1669 kN/m	1671 kN/m	545,211 kN/m
A1–B2	1406 kN/m	1461 kN/m	529,517 kN/m
A1–B3	1491 kN/m	Fixed	492,883 kN/m
A1–B4	1590 kN/m	1450 kN/m	478,750 kN/m
A1–B5	1617 kN/m	1425 kN/m	457,167 kN/m

Table 9. *Cont.*

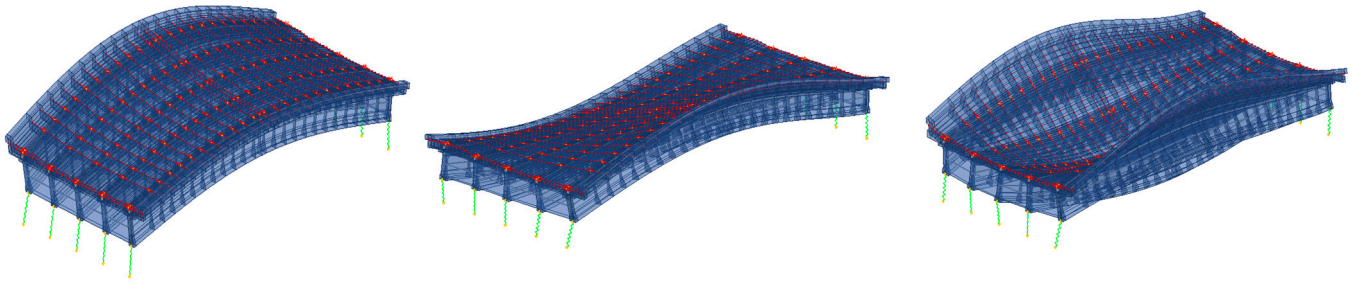
Bearing Number	Updated Factor (K_x)	Updated Factor (K_y)	Updated Factor (K_z)
A2–B1	Fixed	1400 kN/m	465,132 kN/m
A2–B2	Fixed	1629 kN/m	537,523 kN/m
A2–B3	Fixed	Fixed	500,581 kN/m
A2–B4	Fixed	1560 kN/m	545,200 kN/m
A2–B5	Fixed	1551 kN/m	491,477 kN/m

Table 10. Deflection comparison between design, load test, and updated models.

Girder Number	Design Analysis Model (A)	Load Test (B)	Updated Analysis Model (C)	Difference (B − C), (%)
G_1	2.68 mm	1.75 mm	2.62 mm	−0.87 mm
G_2	3.34 mm	3.40 mm	3.40 mm	0.00 mm
G_3	3.97 mm	4.35 mm	4.35 mm	0.00 mm
G_4	3.06 mm	3.19 mm	3.17 mm	0.01 mm
G_5	2.15 mm	1.82 mm	2.15 mm	−0.32 mm

5.2.3. Natural Frequencies and Eigenmodes of the Updated Model

For the bridge in the case study, an analysis was conducted on the natural frequencies and eigenvectors of the system, with the indices updated with the deflection data of each girder of the static load test. The analysis revealed differences from those of the actual bridge. Figure 11 presents the eigenvector shape analysis using the updated model based on the static load test data. The MAC indices are represented as a matrix in Figure 12 to compare the eigenvectors in the load test and the analysis results. The values of the natural frequencies are listed in Table 11. The MAC values for the natural modes are above 0.95, and the bending, torsional, and twisting mode shapes are shown in the order of the mode numbers. For mode 1, the natural frequency is consistent at 3.76 Hz; however, for modes 2 and 3, these are confirmed as torsional modes and differ from the natural frequencies evaluated during the load test.



(**a**) (**b**) (**c**)

Figure 11. Eigenmode after static model updating: (**a**) 1st mode; (**b**) 2nd mode; (**c**) 3rd mode.

Table 11. Natural frequencies of bridge after static model updating.

Mode Number	Mode 1	Mode 2	Mode 3
Natural Frequency (static model updating)	3.76 Hz	5.81 Hz	8.91 Hz
Natural Frequency (load test)	3.956 Hz	4.467 Hz	7.067 Hz

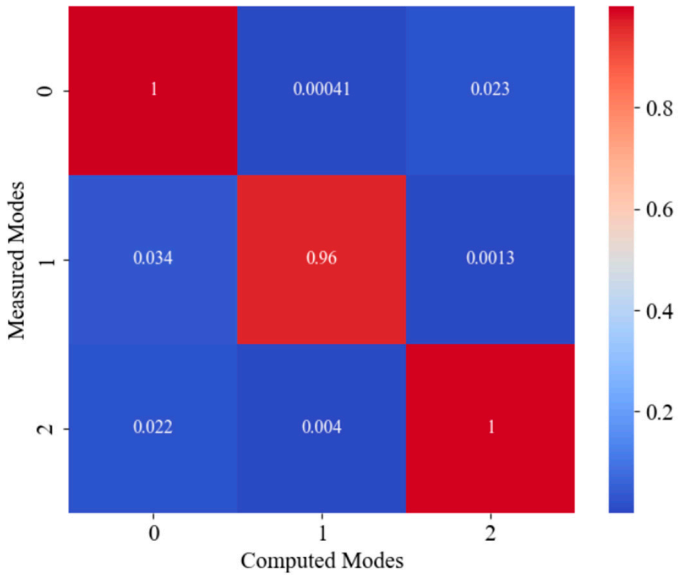


Figure 12. Comparison of MAC index between load test and static updated model.

5.3. Model Updating Using Static Deflection, Eigenmode, Natural Frequency

5.3.1. Configuration of the Static and Dynamic Model Updating

To find the stiffness value of the system that satisfies the static deflection information as well as the eigenvector and frequency, two objective functions (Equations (5) and (6)) were added to the existing objective function (Equation (3)), resulting in an updated model. The example bridge considered has been in service for 45 years and has judged many members as having low damage grades during visual inspections, suggesting that the natural frequency or mode is likely affected. Therefore, the update was performed differently from that reflecting the static deflection. The natural frequencies for the second and third modes, corresponding to the torsional modes, were calculated to be high. Additionally, the torsional stiffness was considered to reflect the extent to which the torsion affected the static deflection according to the bridge characteristics. The torsional stiffness was adjusted by adjusting the stiffness of the end diaphragm. In addition, diaphragm members were targeted for the update because of the high stiffness of the outer girder caused by the condition evaluation results of the diaphragm in the external survey and uneven load distribution. Thus, the range of the elastic modulus of the end diaphragm was 5000–28,000 MPa, and updating was performed using PSO. For dynamic updating, the elastic modulus of the diaphragm was added as an update factor along with the variables reflected in static updating. An analysis model was created for each diaphragm between the bridge girders. Two diaphragm models were included in the bridge, with one diaphragm created using four elements to update the elastic modulus.

$$Obj.Function2 : \frac{1}{3}\left|\sum_{i=1}^{3} Natural\ Frequency_i - Analysis_Frequency_i\right| \quad (5)$$

$$Obj.Function3 : \frac{1}{3}\{\sum|1 \quad 1 \quad 1| - Diagonal(MAC)\} \quad (6)$$

5.3.2. Updating Results

The results of factors updated by deflections, natural frequencies, and eigenmodes are presented in Tables 12 and 13. Table 14 compares the deflection results from the design, load test, static load updating, and static and dynamic load updating. The stiffness and thickness of the girder and slab exhibit different characteristics before and after static

updating. The diaphragm stiffness was updated to the minimum value, and other factors were evaluated differently. Except for the G5 girder deflection, consistent static analysis results were obtained.

Table 12. Elastic modulus and thickness after static and dynamic model updating.

Updating Factor	Description	Initial Value (Design Value)	Updated Value
$E_{c,girder1\sim5}$	Elastic modulus of Girder	29,310 MPa	31,099 MPa/29,746 MPa 32,054 MPa/24,705 MPa/30,500 MPa
$E_{c.slab1\sim6}$	Elastic modulus of Deck	26,986 MPa	34,522 MPa/35,348 MPa/27,946 MPa 27,472 MPa/32,044 MPa/30,014 MPa
$T_{c.slab1\sim7}$	Thickness of Deck	240 mm	246 mm/228 mm/266 mm/180 mm 180 mm/180 mm/291 mm
$E_{Diaphragm1}$	Elastic modulus of Diaphragm	25,000 MPa	5000/5000/5435/5000 MPa
$E_{Diaphragm2}$			5951/5988/5000/5015 MPa

Table 13. Updated spring coefficient after static and dynamic model updating.

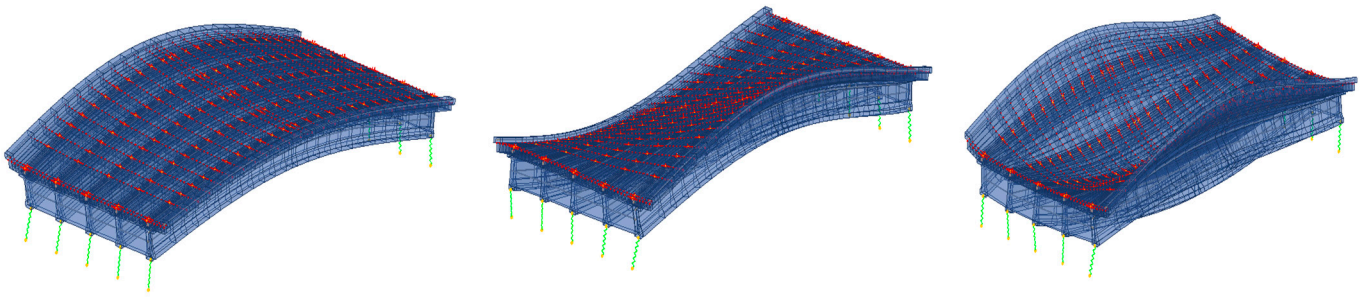
Bearing Number	Updated Factor (K_x)	Updated Factor (K_y)	Updated Factor (K_z)
A1–B1	1416 kN/m	1521 kN/m	454,343 kN/m
A1–B2	1467 kN/m	1484 kN/m	538,330 kN/m
A1–B3	1499 kN/m	Fixed	484,236 kN/m
A1–B4	1602 kN/m	1630 kN/m	523,664 kN/m
A1–B5	1478 kN/m	1665 kN/m	532,214 kN/m
A2–B1	Fixed	1575 kN/m	537,632 kN/m
A2–B2	Fixed	1400 kN/m	512,517 kN/m
A2–B3	Fixed	Fixed	505,096 kN/m
A2–B4	Fixed	1451 kN/m	512,353 kN/m
A2–B5	Fixed	1516 kN/m	523,500 kN/m

Table 14. Deflection comparison between design and load test for each updated model.

Girder Number	Load Test (A)	Design Analysis Model (B)	Static Updated Analysis Model (C)	Static and Dynamic Updated Analysis Model (D)
G_1	1.749 mm	2.68 mm	2.62 mm	2.57 mm
G_2	3.404 mm	3.34 mm	3.40 mm	3.36 mm
G_3	4.353 mm	3.97 mm	4.35 mm	4.32 mm
G_4	3.193 mm	3.06 mm	3.17 mm	3.18 mm
G_5	1.822 mm	2.15 mm	2.15 mm	2.06 mm

The eigenmodes derived from the analysis model, which incorporates updated factors based on static and dynamic data, are shown in Figure 13. Mode 1, which is the bending mode, and modes 2 and 3, similar to the torsional modes, were derived. A numerical comparison of the MAC index, presented in Figure 14, shows that all three modes have values greater than 0.95, indicating similar mode shapes. The natural frequencies after the update were compared with those of the load test, analysis model based on the design data, static update, and static and dynamic updates. The results are listed in Table 15. Compared to the load test, the design and static updated models slightly underestimated the natural frequencies for the bending mode and overestimated those for the torsional mode. The static and dynamic update model evaluated the bending mode similarly to the other models; however, the torsional stiffness was evaluated to be lower than that of the other models with lowered diaphragmatic torsional stiffness reflected in the second

and third modes. The difference between the natural frequencies for the second and third modes, calculated using the bridge analysis model, is approximately 1 Hz.



(**a**) (**b**) (**c**)

Figure 13. Eigenmodes derived from static and dynamic updated model: (**a**) 1st mode; (**b**) 2nd mode; (**c**) 3rd mode.

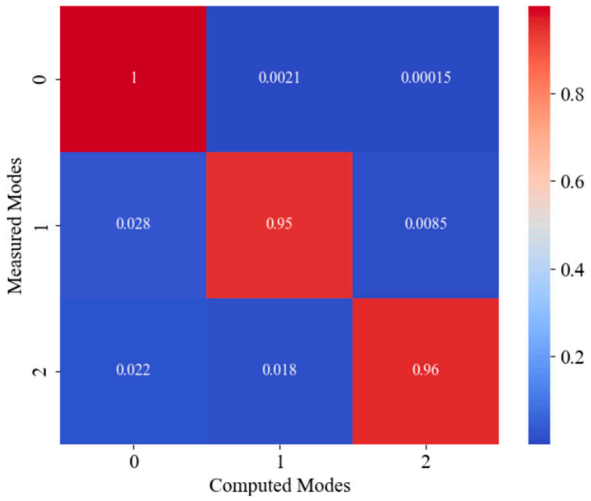


Figure 14. Comparison of MAC index between load test and static and dynamic updated model.

Table 15. Comparison of natural frequencies between design and load test, and two updated models.

Updating Variable	Load Test (A)	Design Analysis Model (B)	Static Updated Analysis Model (C)	Static and Dynamic Updated Analysis Model (D)
$Mode_1$	3.95 Hz	3.63 Hz	3.75 Hz	3.73 Hz
$Mode_2$	4.46 Hz	5.70 Hz	5.81 Hz	5.48 Hz
$Mode_3$	7.07 Hz	9.14 Hz	8.91 Hz	7.78 Hz

6. Conclusions

This study proposed a B-DTM for expanding the DTM in bridge systems during operation. The proposed B-DTM integrates a life cycle data-based information model with analysis models of bridge members, enabling seamless information exchange, data visualization, and accumulation for standardized management. The analysis model was updated using static deflection data from static load tests and natural frequency and mode data from dynamic load tests. Thus, the B-DTM is established as a foundational system capable of leveraging real-time bridge-behavior data. The meaningful conclusions from this study are as follows:

1. When updating the model for defining the B-DTM, the search range was limited by the elastic modulus values of the concrete members that constitute the superstructure

of the PSC bridge. In the case of the concrete members, considering the reliability of the material, the global solution was derived within the range of 95% (z-score: $\pm 1.96\sigma$) of the compressive strength or elastic modulus collected after the manufacturing stage. Through this approach, the search section in a complex bridge system was adjusted, thereby optimizing the amount of computation and improving the possibility of obtaining a global solution.

2. In order to update the bridge model, elastomeric bearings were also included as major members and were defined as update factors. The compression spring coefficient and shear spring coefficient of the bearings, which affect the static and dynamic behavior of the bridge, were selected as update factors. In addition, the spring coefficient range searched during the update was set by considering the deterioration of the rubber layer of the elastic bearing during use. Through this, the possibility of calculating the global solution of the system was increased. And the risk of underestimation or overestimation when updating the stiffness of girders and deck members based on the global behavior of the bridge is minimized.
3. The model update considering the deflection obtained from the bridge load test and the update results that take the deflection and dynamic eigenmode and natural frequency as objective functions together were compared. The model of the update factor that only reflects the static deflection did not accurately show the behavior of the natural frequency of the bridge, which verified that the update result was a local solution. The validity of the update method through the configuration of the objective function for these three things, static deflection, eigenmode, and natural frequency, was verified.
4. The B-DTM system is proposed through a model update system that sets the parameter range of the stiffness of the member and the spring coefficient of the support and uses the static and dynamic behavior of the bridge as the objective function. This is a model that provides a basis for real-time data of the bridge that occurs during operation and serves as a premise for the evaluation of the bridge.

These findings highlight the significance of comprehensive objective functions in model updates. Relying on limited objective functions risks local solutions, given the complexity of bridge systems. In particular, the spring coefficient of the elastomeric bearing considerably influences the dynamic behavior and deflection, making it a major factor in the model update. If constraints on the range and objective functions are disregarded, the stiffness evaluation of the main member may be inaccurate, and some objective functions could be improperly adjusted during optimization. When assessing the bridge using the updated analysis model, the results may deviate from the actual behavior, potentially leading to local solutions in the system. A probabilistic approach supported by narrowing the search range and leveraging bridge history from the manufacturing to the as-built stages is essential for efficient and accurate updates. Furthermore, deviations between baseline models and actual data should trigger model refinement. By enabling "what-if" simulations and system-level analyses, the B-DTM fulfills its role as a robust tool for bridge management and operational optimization.

7. Recommendation for Further Study

The problems and methods for improvement of the proposed B-DTM definition methodology are presented as follows:

- It is desirable to update the stiffness of girders and decks from the construction stage before completion. Since the uncertainty of secondary members is excluded and data before and after boundary conditions are reflected and can be obtained, more reliable stiffness data are produced. This can further narrow the range within the

95% reliability of the currently applied material, and this increases the possibility of reaching the global solution of the system.

- It is very difficult to make the difference between the results of the update and the results of the load test exactly match, and the reliability of using the calculated deterministic solution as the global solution of the bridge system is low. Accordingly, a method of expressing the reliability of the model update by applying a probabilistic model update method, such as Bayesian updating, is required.
- The shell model of the deck has a disadvantage in that it is difficult to adjust the torsional stiffness. Accordingly, an analysis model that can reflect the actual behavior of the bridge as much as possible through an improved model, such as Higher-Order Shell Elements or an improved grid model of the deck, is required.

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