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Doctoral Dissertation

Development of Performance Improvement Method of Existing Steel Railway Bridges by Installing Concrete Decks

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Masamichi Saito

Abstract

More than half of existing steel railway bridges have been in service over their design lifetime. Some of these bridges face problems such as decrease of load-carrying capacity due to corrosion, limited fatigue life due to stress increase, and large noise emission due to vibration of steel members. On the other hand, replacement of these bridges takes much cost and time. The purpose of this research is to develop the performance improvement method of existing steel railway bridges by installing concrete decks on the steel girders, and to investigate the applicability and the effectiveness of the proposed method, focusing on the improvement of load-carrying capacity, stress reduction considering fatigue life, noise and vibration reduction effect and workability for the application into actual bridges.

Considering application into actual bridges, an installation method using pre-cast concrete decks and girder-deck connection with filler mortar and steel fasteners was proposed. To evaluate the capacity of the fasteners, loading tests were carried out and it was found that the fasteners have enough capacity to vertical and horizontal load by train. To investigate the conditions of girder-deck connection for keeping the girder in composite state, loading tests of deck-installed girders were carried out. As a result, it was found that it is necessary to prevent the slip between steel girders and concrete decks and ensure load transfer between the pre-cast concrete decks.

To evaluate the structural performance of the deck-installed girder and failure mode of the girder-deck connection, loading tests of composite girder that was fabricated using proposed method were carried out. As a result, it was found that the bending stiffness and the yielding capacity of the girder were improved to those of the composite girder. At the girder-deck connection, the failure occurred at the steel-mortar interface at the girder end. Finally, shear strength of the girder-deck connection was obtained and it was found that the proposed method is applicable to actual railway steel bridges without failure of the girder-deck connection.

To figure out the noise reduction effect by installing concrete decks, impact hammer tests were carried out. As a result, it was found that the reduction of the on-girder vibration is dominant in reducing vibration of steel girder and structure-borne noise.

Finally, it was concluded that the proposed method has enough applicability on existing steel railway bridges in the viewpoint of the workability of the deck installation work in night time work. It was also concluded that the proposed method has the effect of performance improvement, such as increase of load-carrying capacity that can be predicted by assuming the composite effect of the concrete deck and the steel girder, extension of fatigue life by reducing stress, and reduction of structure-borne noise by reducing propagation of on-girder vibration into the steel girders.

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Chapter 1. Introduction

1.1 Background

1.1.1 Existing steel railway bridges

There are about 50,000 steel railway bridges in Japan. **Fig. 1-1** shows the number of steel railway bridges and the number of bridges in need of attention, with respect to age of bridges[1]. Although the data is a result of an old survey in 1986, the data shows the number of bridges in whole Japan National Railways. According to **Fig. 1-1**, more than half of existing railway steel bridges has been still in service beyond their designed lifetime, *i.e.* 60 years. **Fig. 1-2** shows the number of bridges in the area of JR East[2]. Until 70 years ago, steel bridges had been mainly constructed as railway bridges. On the other hand, concrete structures, *i.e.*, reinforced concrete (RC) and prestressed concrete (PC), have been constructed as railway bridges. As a result, there is a difference in its composition between older bridges (60 years ~) and younger bridges (~ 60 years). Most of these bridges are maintained appropriately in a systematized maintenance method[3]. However, still a large number of these bridges are in need of repair or replacement. These bridges have problems such as corrosion, fatigue, large noise and maintenance.

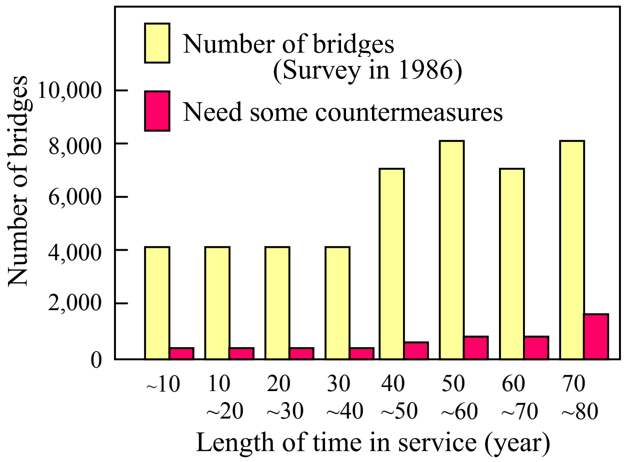


Fig. 1-1 Number of steel railway bridges in Japan Railways [1]

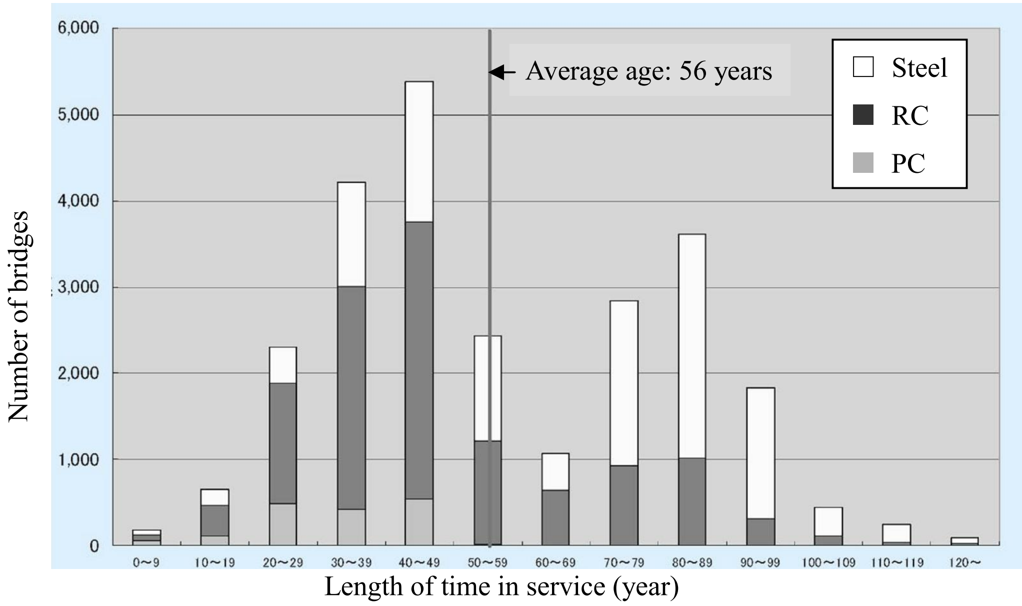


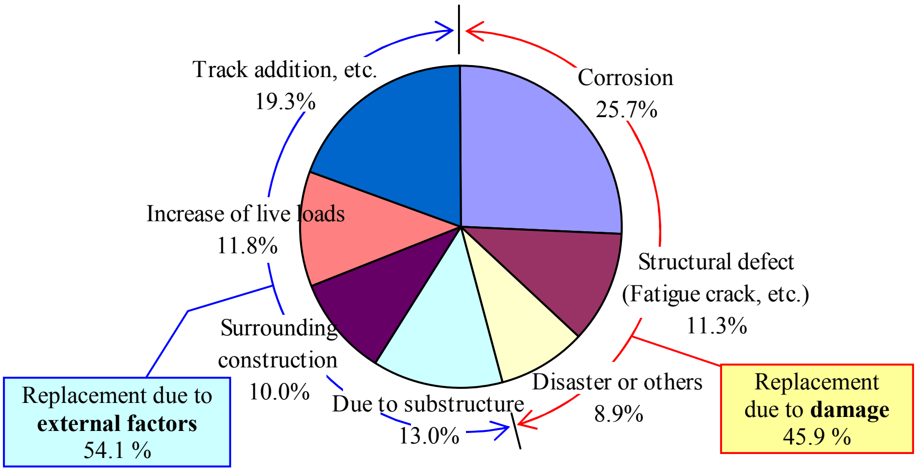
Fig. 1-2 Number of railway bridges in East Japan area[2)]

1.1.2 Corrosion

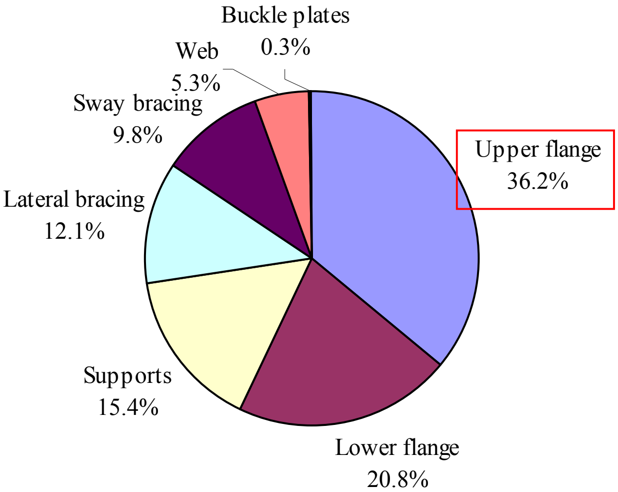
According to the result of the past investigations[4)], about half of the bridges were replaced due to their damage and another half of them were replaced due to external factors, as shown in **Fig. 1-3**(a). Among these reasons, corrosion accounts for 25.7 % and it is the maximum number. Furthermore, corrosion of the upper flange accounts for 36.2 % among the corroded members of deck girder bridges. Although these data are the result of old survey (in 1969), more than half of the existing steel railway bridges were constructed before this survey as mentioned in 1.1.1. Furthermore, the current situation of them is not totally different from the time of the survey, *i.e.* there are many cases of the upper flange corrosion[5)].

Many of existing railway steel bridges has sleepers directly set on the girders. Corrosion of upper flanges often occurs at the contacted areas with the sleepers[5)], as shown in **Fig. 1-4**. This is because ordinary paintings of the upper flanges under sleepers easily degrade by cyclic train load[6)7)8)]. In addition, the situation, that the re-painting of the upper flange is difficult because of the sleepers, also furthers the corrosion of the upper flange[9)10)]. **Fig. 1-5** shows the cross section of the corroded upper flange[11)]. In this case, most of the thickness of the cover-plate was decreased by corrosion.

The thickness reduction of the upper flanges due to corrosion leads to decrease of load-carrying capacity of the girders. Especially, the local buckling of the corroded section of the upper flange decreases the load-carrying capacity of the girder significantly comparing with the corrosion of the tensile members, such as the lower flange, as shown in **Fig. 1-6**[12)]. Many researches have been carried out on predicting the remaining load-carrying capacity in Japan[9)10)12)13)14)] and abroad[15)16)17)].

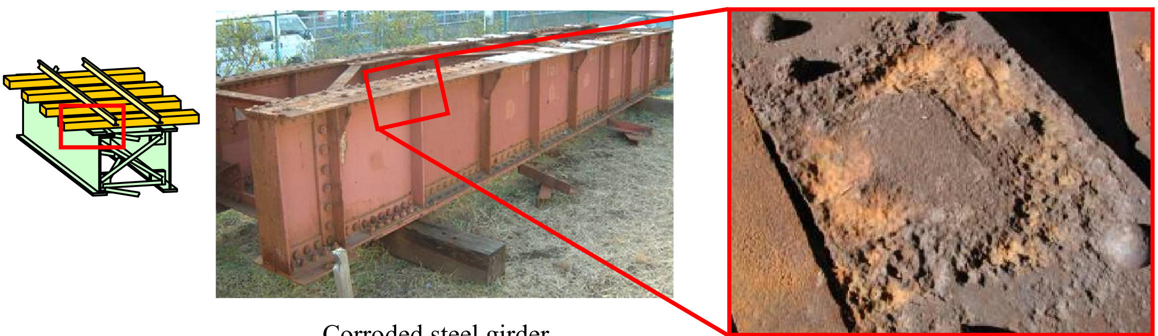


(a) Reason of replacement of steel railway bridges



(b) Corroded member in deck girder bridges replaced due to corrosion

Fig. 1-3 Reason of replacement and corroded member of steel railway bridges[4)]



Corroded steel girder

Local corrosion of
contacted area with sleeper

Fig. 1-4 Corrosion of upper flange[5)]

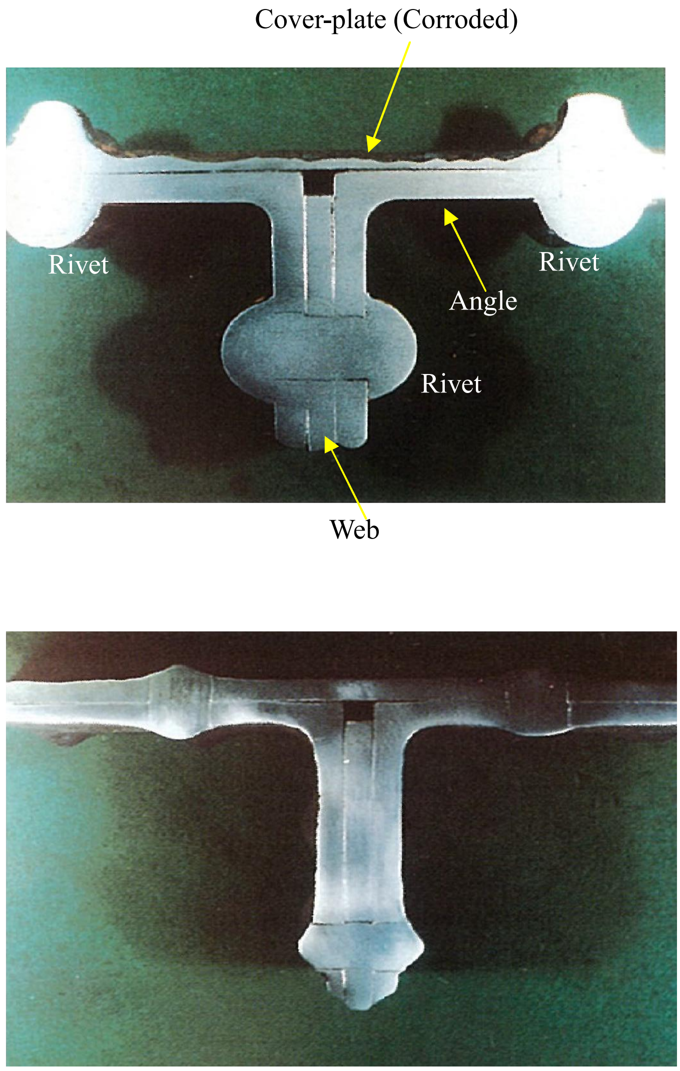


Fig. 1-5 Cross section of corroded upper flange of riveted steel girder[11)]

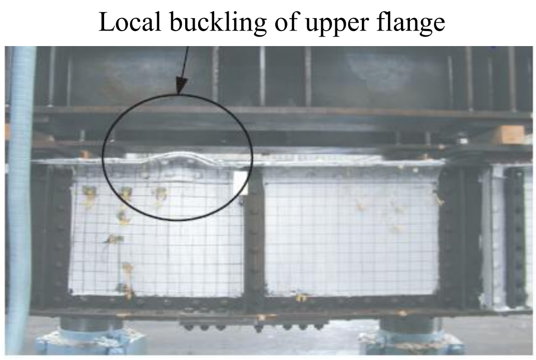


Fig. 1-6 Local buckling of upper flange[12)]

1.1.3 Existing countermeasures for corrosion

General application of counter measure against the corrosion includes re-painting, repair and retrofit, member replacement and bridge replacement, depending on the status of the members or the bridge.

(1) Re-painting of upper flange

If the damage is only in paintings, re-painting can be carried out. Several types of paintings with higher durability such as glass-flake painting[7] and polyurea paint and other new materials[18][19][20] have been developed and applied to actual bridges.

General procedure of re-paintings consists of "removal of existing paintings and rust" and "painting"[11][21]. However, the re-painting of the upper surface of the upper flange is different from that of the other members. In order to re-paint the upper flange, it is necessary to remove the track members by lifting-up prior to the re-painting works[23], as shown in **Fig. 1-7**.

(a) Lift-up of track members prior to re-painting

(b) Re-painting of the upper surface of the upper flange

Fig. 1-7 Example of re-painting work of upper flange[23]

(2) Retrofit of members

If the thickness decrease of the upper flange is significant, retrofit of the corroded member can be generally applied.

In general, the most widely used way of retrofit is “splice reinforcement”, a method to attach steel plates to the corroded steel members connecting with high-strength bolts[24]. By this method, the cross section of the corroded member can be recovered or increased further. While the splicing to the severely corroded members is difficult because of the surface irregularities, the splicing technique using adhesive material has been developed[25] and applied to actual structures[26]. It is also called “doubler plate” and the splicing plates are sometimes attached by welding[27]. However, the welding attachment is not recommended because the field welding to the existing structure can decrease its fatigue strength, as mentioned in 1.1.4.

Although the splice reinforcement is effective to recover the capacity of the corroded members, it is difficult to apply the splice reinforcement because there are track members such as sleepers on the upper flanges of the deck-girder type railway bridges.

For the retrofit of tensile members, “FRP attachment[27][28]” and “prestressing[22][24][27][29]” can be also effective methods to reduce the stress and to increase the load-carrying capacity. However, for the corroded compressive members such as the upper flange, these methods are not effective in preventing the buckling.

(3) Member replacement

If the thickness decrease of the upper flange due to corrosion is significant, replacement of upper flange can be carried out, as shown in **Fig. 1-8**[1)11)30)31)]. **Fig. 1-9** shows an example of the flange replacement. Prior to the night-work, it needs to remove existing rivets. At the beginning of the night-work, the track members, such as rails and sleepers, are removed out. Then, the upper flange can be removed and the new upper flange is installed. As shown in these photographs, the replacement work is not easy and it must be finished within the tight time schedule.

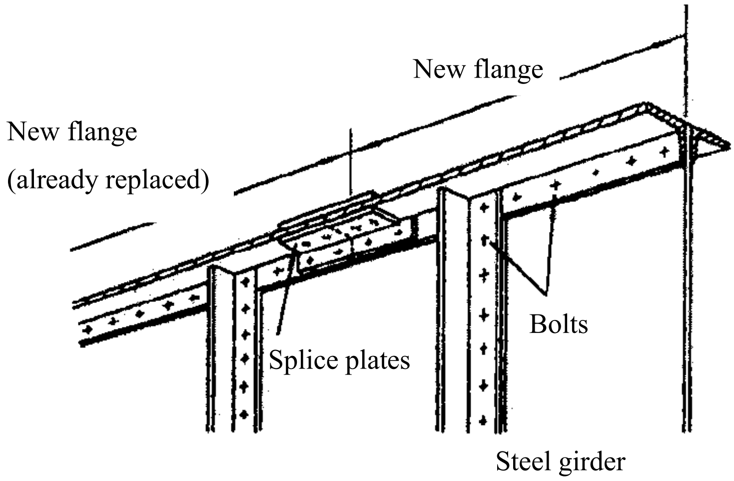


Fig. 1-8 Method of upper-flange replacement[11)]

(a) Replacement of rivets to high-strength bolts

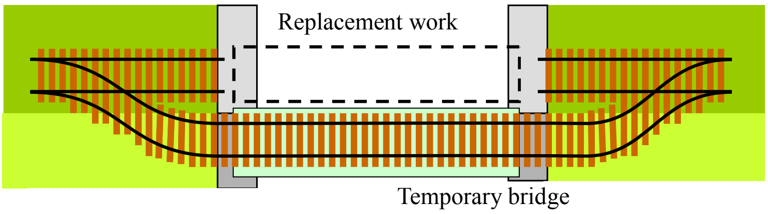
(b) Removal of old upper flange

(c) Installation of new upper flange

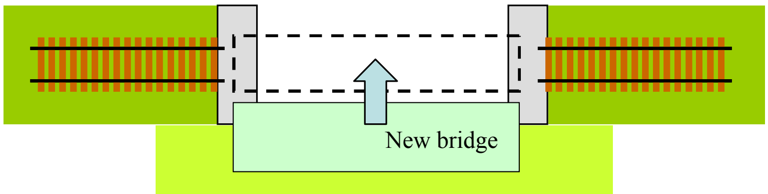
Fig. 1-9 Example of replacement of upper flange[11)]

(4) Bridge replacement

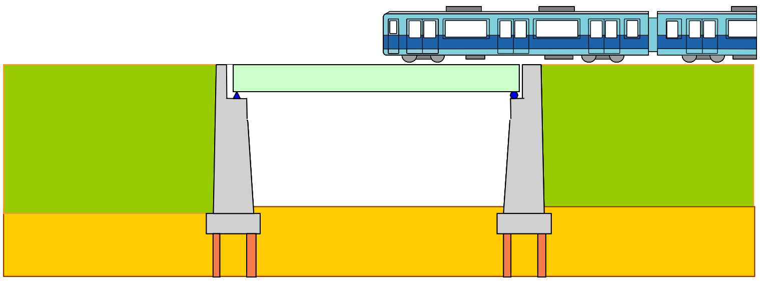
In previous times, severely damaged bridges were generally replaced with new bridges. However, the replacement requires high cost and much time during which the train operation has to be stopped. In railway system, train operation should not be stopped, especially in urban areas. To avoid the stop of the train operation, special methods are applied, such as replacement using temporary bridge and replacement by lateral transfer method, as shown in **Fig. 1-10**. In both methods, it takes much more costs for construction and removal of temporary equipment rather than the cost for new bridge.



(a) Replacement using temporary bridge



(b) Replacement by lateral transfer method



(c) Side view

Fig. 1-10 Methods for replacement of existing bridges

1.1.4 Fatigue

Generally, fatigue strength of riveted girders is comparatively higher than that of welded girders. However, the fatigue strength can be lowered when the cross section of the members decreases by corrosion or when the rivet loosens[32). Furthermore, the welded attachments, such as gussets and cover plates, on the riveted girders decrease the fatigue strength[33). In order to prevent fatigue cracks, reinforcement and retrofit, such as steel splice and attachment of additional members, are generally applied. As an effective countermeasure for fatigue, it is important to reduce the stress range under train passage.

1.1.5 Noise

Steel railway bridges tend to generate larger structure-borne noise than other structures, such as reinforced concrete structures[34). In recent railway bridges, the steel bridges tend to be avoided because of its cost and large noise. In the existing steel bridges, large noise emission to surrounding environment is one of big issues for railway operators.

1.1.6 Maintenance of track

As mentioned above, sleepers are directly set on the girders in many of the existing steel railway bridges. In such bridges, replacement of old sleepers takes a lot of time and effort. In addition, realignment of the track is also a difficult work.

1.2 Purpose and objectives

To avoid the replacement of existing steel railway bridges and to make it possible to keep the bridges in service in effective way, it is important to improve the performance of the bridges, such as load-carrying capacity, fatigue life and noise emission. In this research, an idea to install concrete decks on the steel girders instead of sleepers, as shown in **Fig. 1-11**, is proposed as a method to improve the performance.

This method is expected to change the structural system of a steel girder into a composite girder and increases its load carrying capacity. In addition, this method is also possible to mitigate accumulation of fatigue damage by reducing stress at the steel girders and to reduce noise due to vibration of the bridge members. Furthermore, the concrete deck prevents corrosion on the upper flange and thus the maintenance work of the steel girders can be reduced. The purpose of this research is to develop the performance improvement method of existing steel railway bridges by installing concrete decks on the steel girders.

However, these effects are not confirmed and the applicability to the actual bridges is necessary to be investigated. Considering this problem, the objectives of this research are as follows:

(1) To investigate the applicability of the proposed method in actual bridges, such as applicable range, installation procedure and workability. In order to enable the application of the proposed method, deck installation procedures and structural detail of concrete deck and connection between steel girder and concrete deck (hereinafter called as 'girder-deck connection') are proposed.

(2) To evaluate the effectiveness of the proposed method experimentally, such as increase of load-carrying capacity, reduction of stress, and reduction of vibration and noise.

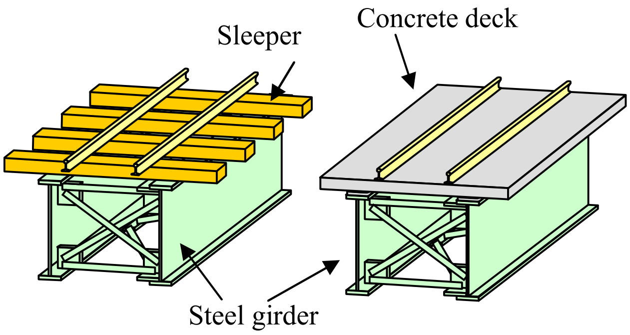


Fig. 1-11 Installation of concrete deck

1.3 Scope of research

The target of this research is steel railway bridges with single-track, single-spanned, deck-girder type bridges. These are the typical conditions of aged short span bridges. **Table 1-1** shows standard-designed deck girder bridges, constructed by Japan National Railways. Among these standard-designed girders, all of the deck-girder bridges are designed as single-track and single-spanned bridges. It means that most of the existing bridges, especially old bridges, are included in the target bridge type. From the table, the largest span of the deck-girder bridges depends on its constructed year and is about 25 to 47 m. For the spans larger than those limits, other structural type, such as truss bridges, was applied.

Table 1-2 shows the performance improvement discussed in this thesis. The improvement of the load-carrying capacity is discussed through the bending moment in Chapter 2 based on design calculation and in Chapter 4 based on loading test. The improvement of the fatigue life is discussed through the stress reduction in Chapter 2 based on design calculation and in Chapter 4 based on loading tests. The improvement of the noise emission property is discussed through the reduction of the vibration and the noise in Chapter 5. The workability of the method is discussed through work time, adjustment of decks and filler mortar in Chapter 2 and method of girder-deck connection in Chapter 3. Though the maintenance of the track members is also expected by applying the proposed method, it is not discussed in this thesis.

Table 1-1 Transition of standard-designed deck girder bridges in Japan

Type of bridge	Period (year)	Girder type	Smallest span (m)	Largest span (m)
Standard girders (Wrought iron type)	1885 ~ 1889	Riveted	3.7	21.3
Standard girders (Meiji 30 Type)	1897 ~	Riveted	4.6	24.4
Standard girders (Meiji 35 Type)	1902 ~	Riveted	6.1	24.4
Standard girders No.680	1909 ~	Riveted	6.1	24.4
Standard girders No.540	1919 ~	Riveted	5.5	24.4
Standard girders No.94	1920 ~	Riveted	6.1	24.4
Standard girders No.1084	1930 ~	Riveted	6.7	31.5
Standard girders	1957 ~	Riveted	8.2	46.8
Standard girders	1964 ~ 1967	Welded	8.2	46.8

*Data were obtained from existing design drawings of Japan National Railways

Table 1-2 Performance improvement of existing steel bridges discussed in this thesis

Performance	Criteria	Chapter
Load-carrying capacity	Bending moment	Chapter 2 Chapter 4
Fatigue life	Stress	Chapter 2 Chapter 4
Noise emission	Vibration, Noise	Chapter 5
Workability for application	Work time, adjustment, filling mortar Girder-deck connection	Chapter 2 Chapter 3
Maintenance of structural members and track members	(Out of scope of this thesis)	-

1.4 Flow and contents of research

This thesis consists of six chapters. Flow of this research is shown in **Fig. 1-12**. The contents of Chapter 2 to Chapter 6 are described as follows.

Chapter 2 Applicability of Performance Improvement Method by Installing Concrete Decks

This chapter describes an outline of the performance improvement method that is proposed in this research. Then, expected effects and influence by this method and the applicable range of the method are discussed based on design calculation. Considering application into actual bridges, deck installation procedure and detail of concrete deck and girder-deck connection are discussed. To obtain the time for installation work and positioning error of the decks, a test construction of deck installation was carried out in laboratory. Finally, to figure out the property of filler mortar, tests for property of fresh mortar and bonding property of mortar-concrete interface were carried out.

Chapter 3 Development of Girder-Deck Connection

In this chapter, details of girder-deck connection were proposed. To evaluate the capacity of the steel fasteners, the lateral loading test and the uplifting loading test of the fasteners were carried out. In order to examine the performance of the deck-installed girder and to clarify the condition for the composite effect of the girder and the deck, the bending tests of the girder were carried out.

Chapter 4 Structural Performance of Deck-installed Girder

To evaluate the structural performance of the deck-installed girder and to observe the failure mode of the connection, loading tests were carried out. From the results, the applicability to actual structures was discussed focusing on shear strength of the girder-deck connection.

Chapter 5 Noise Reduction Effect by Deck Installation

In this chapter, impact hammer tests of the steel girder before and after the concrete deck installation were carried out in order to clarify the reduction effect of vibration of structure and structure-borne noise. Through the tests, the vibration property of the web and the noise property near the girder were obtained.

Chapter 6 Conclusions

This chapter summarizes all the results in this dissertation and gives the recommendation of study for the future development.

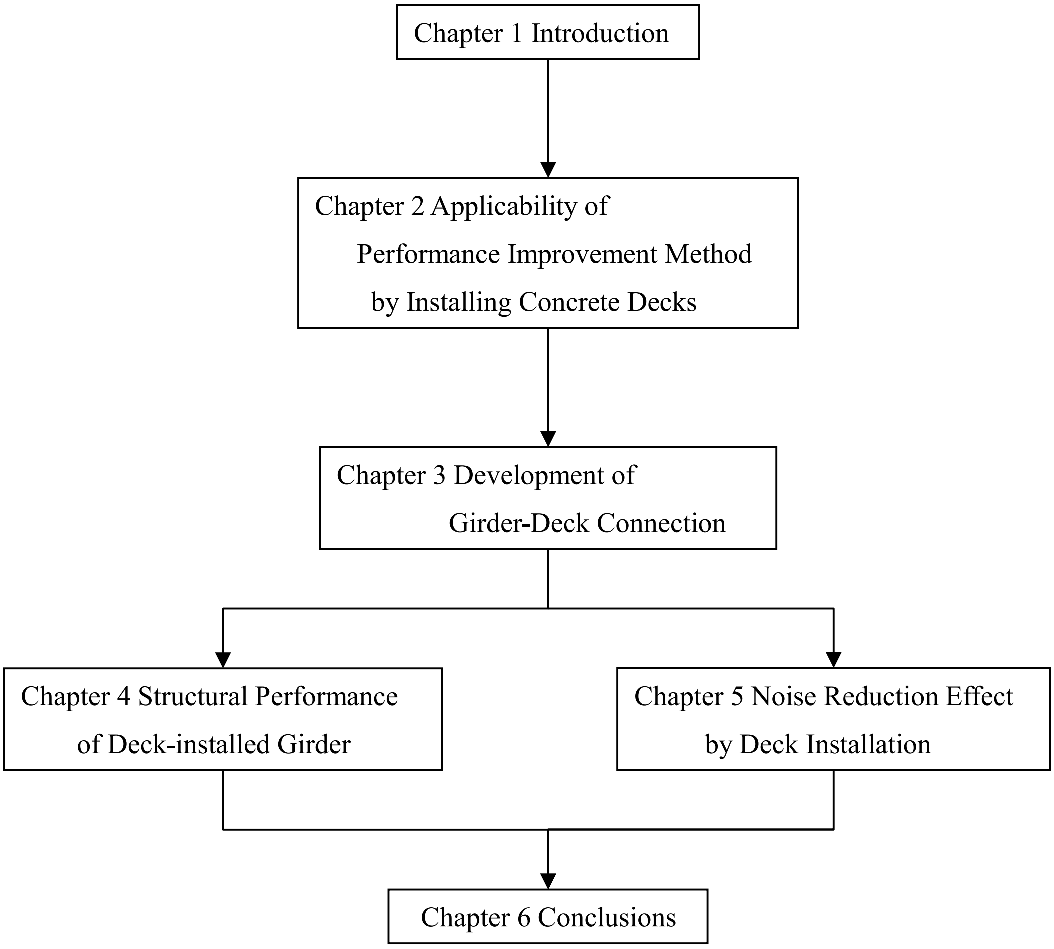


Fig. 1-12 Flow and contents of research

Chapter 2. Applicability of Performance Improvement Method by Installing Concrete Decks

2.1 Introduction

As shown in the last chapter, existing steel railway bridges are in need of improving their performance, such as load-carrying capacity, fatigue life and noise emission. In this research, a performance improvement method of existing steel railway bridges by installing concrete deck was proposed. To realize this method, it is necessary to discuss its applicability on existing structures. In this chapter, the applicability of the proposed method was discussed, focusing on applicable range of span considering increase of dead load by concrete decks, the effect of stress reduction at the existing steel girders and applicable type of concrete decks. As for the deck installation, the key points would be time required for installation and connection between girders and decks. To enable the installation of concrete decks in a short time work, an installation method using pre-cast concrete decks was proposed.

However, it is difficult to install the pre-cast concrete decks in normal way, *i.e.* putting shear connectors on upper flanges and pouring fresh concrete, because there are irregularities by rivet heads, *etc.* on the surface of the steel girders. To solve this problem and to enable the connection of the steel girders and the concrete decks, a connection method using filler mortar was proposed.

To confirm time and workability of the deck installation, the test construction of concrete deck were conducted. To obtain the property of the filler mortar, various tests were carried out.

2.2 Proposal of method

As shown in **Fig. 2-1**, typical existing steel bridge has sleepers directly set on the girders. In the proposed method, the sleepers are removed and a concrete deck is installed on the girder instead of the sleepers. The concrete deck is connected to the steel girder by shear connector or some other way that can transmit the shear force between the steel girders and the concrete deck.

By installing the concrete deck, the structural system of a steel girder can be changed into a composite girder and increases its load carrying capacity. In addition, this method is also possible to mitigate accumulation of fatigue damage by reducing stress at the steel girders and to reduce noise due to vibration of the bridge members. Furthermore, the concrete deck prevents corrosion on the upper flange and thus the maintenance work of the steel girders can be reduced. The purpose of this research is to develop the performance improvement method of existing steel railway bridges by installing concrete decks.

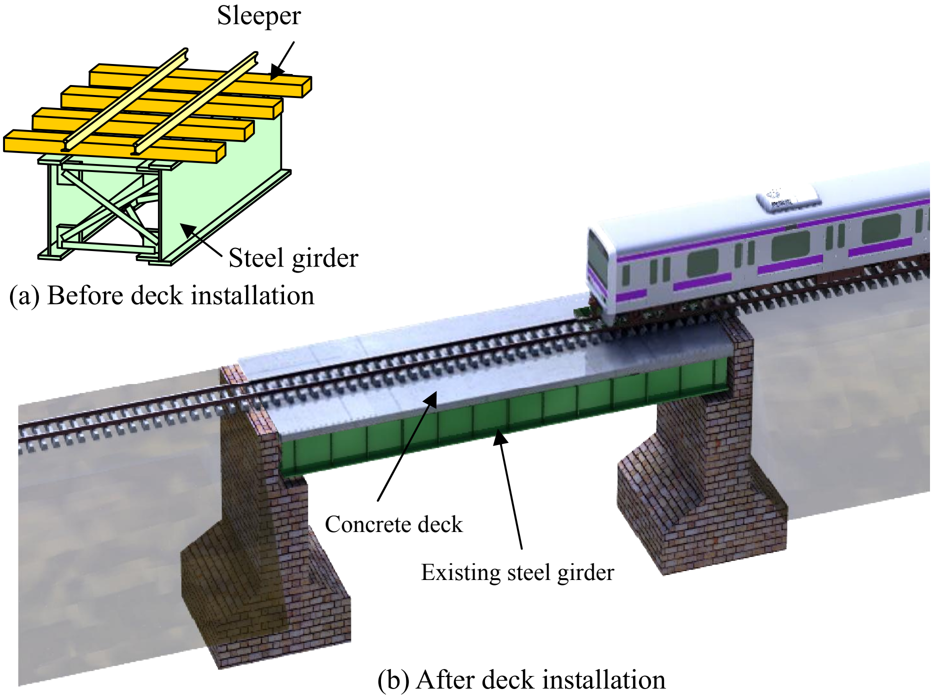


Fig. 2-1 Image of performance improvement method by installing concrete deck

2.3 Expected effects

2.3.1 Stress reduction

Through design calculation, the stress of the girder was estimated under the condition before and after the deck installation. The models for the calculation are typical existing railway bridges, which has simple girders with span of about 10 ~ 40 m. **Fig. 2-2** shows an example of the deck girder bridge with about 10 m span. The stress after the installation of concrete deck was calculated considering composite action of the deck and the steel girder. Here the thickness reduction of the girder is not taken into account. Depth of the deck must be less than 200 mm, conforming to that of the sleepers. The stress is calculated for 'live load', *i.e.* the train load and the impact load. For the train load, "E-17" load, shown in **Fig. 2-3,** was applied.

Fig. 2-4 shows an example of the result in the girder with 10 m span. Stress of upper flange can be reduced by approximately 80 % and stress of lower flange can be reduced by 20 %. In spite of the constraint of the depth of concrete deck, *i.e.* less than 200 mm, stress reduction effect is significant especially at the upper flanges, where the thickness reduction often occurs. This is because the neutral axis of the girder moves to the location near the upper flange.

Table 2-1 shows the results of the calculation, including other model bridges with 20 ~ 40 m span. The "stress ratio", calculated as the stress after the deck installation divided by the stress before the deck installed, are also shown in the table. As a result, stress at the upper flange can be decreased by 50 ~ 80 % and stress at the lower flange can be reduced by 10 ~ 20 %. The stress reduction effect is relatively large in shorter span bridges.

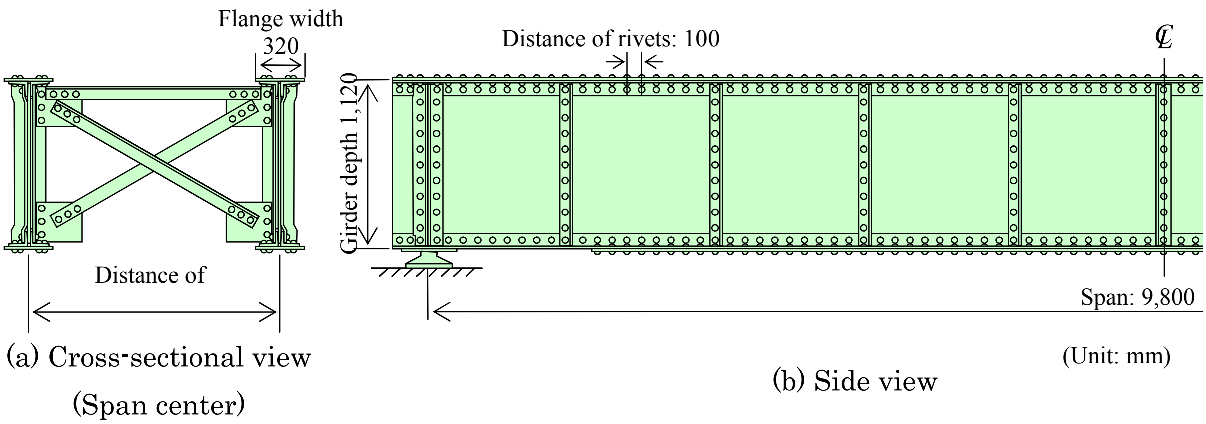


Fig. 2-2 Steel girder of example bridge

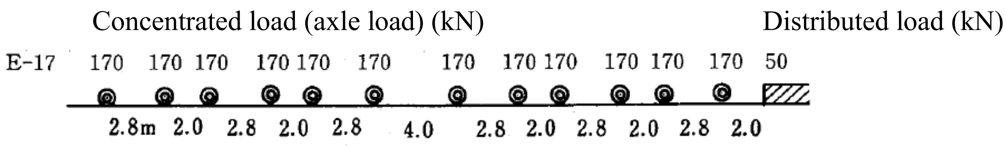


Fig. 2-3 Train load used for design calculation[37)]

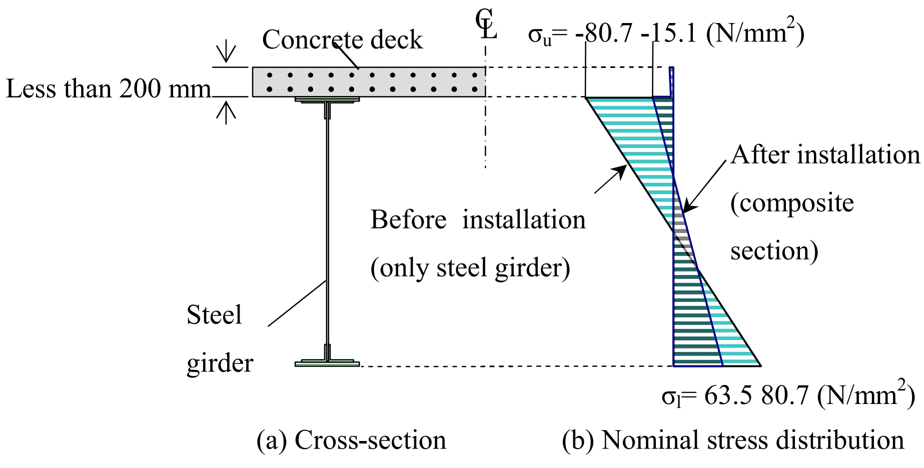


Fig. 2-4 Stress reduction by deck installation (9.8 m span girder)

Table 2-1 Nominal stress at flanges under live load and ratio of remaining fatigue life

Span (m)	Girder depth (m)	Upper flange			Lower flange			
		Stress σ_u (N/mm^2)			Stressσ_l (N/mm^2)			Fatigue life
		Before	After	Ratio	Before	After	Ratio	Ratio
9.8	1.12	-80.7	-15.1	0.18	80.7	63.5	0.79	2.0
19.2	1.67	-102.2	-35.7	0.35	102.2	87.4	0.85	1.6
31.5	2.34	-88.3	-43.7	0.49	88.3	79.5	0.90	1.4
40.0	2.69	-98.1	-46.3	0.47	98.1	74.8	0.76	2.3

2.3.2 Increase of fatigue life

In consequence of the stress reduction shown in 2.3.1, the remaining fatigue life of the bridge can be elongated. Although it is difficult to discuss the absolute value of the fatigue life, it is possible to discuss the change of the remaining fatigue life after the deck installation. In general, fatigue life is known to be inversely proportional to the cube of cyclic stress range. Focusing on the remaining fatigue life after the deck installation, the ratio of the remaining fatigue life is calculated by the stress reduction ratio of the flange of the lower flange, as shown in **Table 2-1**. For example, at the lower flange of the 9.8 span girder, the nominal stress can be decreased by 21 % , *i.e.* stress ratio is 0.79. Calculating the cube of the stress ratio, the remaining fatigue life ratio becomes 2.0 and it means the remaining fatigue life of the lower flange after the deck installation can be elongated to double of that before the deck installation. For the bridges with 10 ~ 40 m span, the remaining fatigue life of the lower flange can be increased to 1.4 ~ 2.3 times.

2.3.3 Increase of load carrying capacity

In consequence of the stress reduction shown in 2.3.1, the load-carrying capacity of the bridge can be increased. **Table 2-2** shows the bending capacity of girder before and after the deck installation. The bending capacity increases by 20 ~ 50 % by installing the concrete decks. Here, the bending capacity is determined as a moment when the stress at either of upper flange or lower flange of steel girder reaches the allowable stress, *e.g.* 132.4 N/mm^2 for the lower flange and -120.3 N/mm^2 for the upper flange in the case of 40 m span bridge.

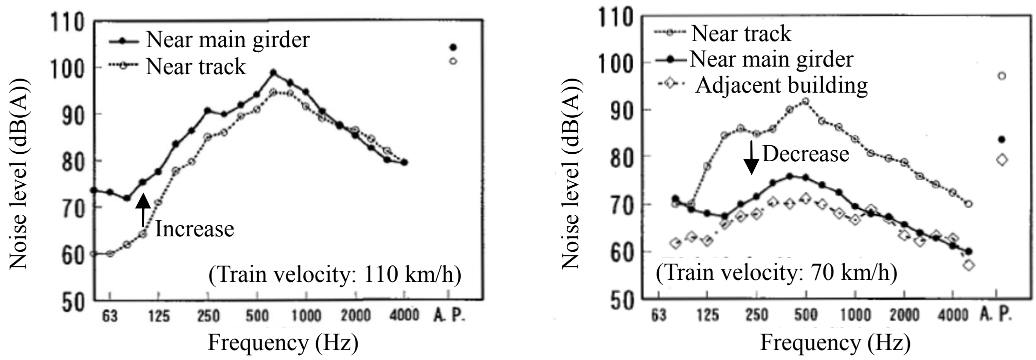
Furthermore, the upper surface of the upper flange can be covered with the filler mortar and it prevents from the progression of the corrosion at the corroded place. It also leads to the extension of the service life of the bridge.

Table 2-2 Bending capacity of girder

Span (m)	Girder depth (m)	Bending capacity (kN·m)		Ratio (After/Before)
		Before	After	
9.8	1.12	1.98×10^3	3.01×10^3	1.52
19.2	1.67	5.97×10^3	8.44×10^3	1.41
31.5	2.34	1.02×10^4	1.23×10^4	1.21
40.0	2.69	1.27×10^4	1.56×10^4	1.23

2.3.4 Noise reduction

Steel railway bridges tend to generate larger structure-borne noise than other structures, such as reinforced concrete structures[34]. According to the previous measurement result[35], the noise near the main girder is larger than the noise near the track, as shown in **Fig. 2-5**(a). This is because the noise is amplified by the vibration of the steel girder members. On the other hand, in the composite girder bridge, the noise near the main girder is smaller than that near the track, as shown in **Fig. 2-5**(b). It means that the noise from the steel girder in the composite girder bridge is comparatively small. From this fact, it is expected that the installation of concrete deck on the steel girder is effective to reduce the noise. This will be discussed in Chap. 5.



(a) Bridge A (Steel deck-girder bridge) (b) Bridge B (Composite girder bridge)

Fig. 2-5 Measured noise level in steel bridge and composite girder bridge[35]

2.4 Influence of installing concrete deck

2.4.1 Increase of dead load and stress of supports

When the concrete deck is installed onto the existing bridge, increased dead load acts directly on the supports of the bridge, such as steel bearings and support concrete shown in **Fig. 2-6**. Through the design calculation, the bearing stress of the support concrete was estimated as Load P divided by the supporting area A, under the condition before and after the deck installation. The models of calculation are typical existing railway bridges with structural type of deck girder bridges, simple-span and single-track. Here it was assumed applying the concrete deck with at most 200 mm depth, conforming to the depth of the existing sleepers, in order not to change the rail level. **Fig. 2-7** shows the results of the calculation. In the bridges with span of more than 40 m, bearing stress of the concrete of the supports exceeds the allowable stress for general existing structures[36). From the results, it was found that the proposed method is applicable to the bridges shorter than 40 m. For the bridges exceeding 40 m span, the actual strength of the support concrete should be obtained to judge the applicability.

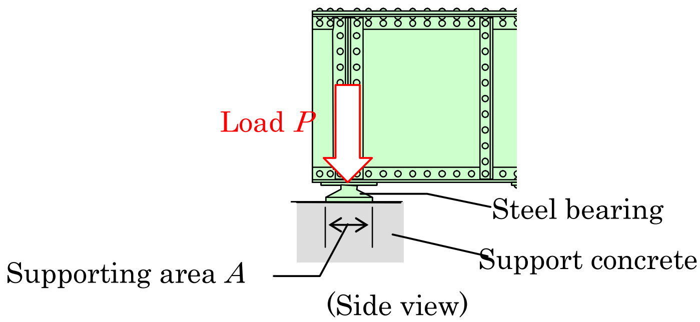


Fig. 2-6 Bridge support of existing steel bridge

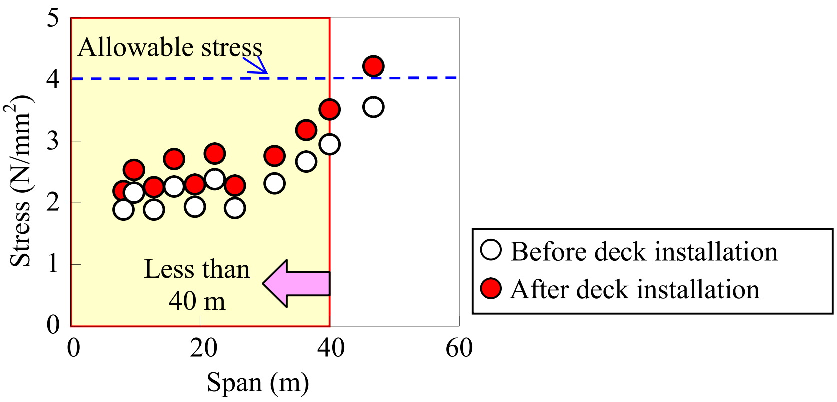


Fig. 2-7 Bering stress at support concrete and applicable range of span

2.4.2 Inspection of girder

Before the application of the proposed method, the sleeper-type steel girders are easier to be observed from the upper side of the track at the periodic walking patrol which is held several times in a year. This is because the girder can be seen through the gaps between the sleepers. After the concrete deck is installed, it becomes difficult to observe the girder from the upper side.

On the other hand, at the inspection of the bridge, temporary scaffolding can be installed and visual inspection at close range can be achieved, even for the sleeper-type steel girders.

Considering this situation, the installation of the concrete deck does not make it difficult to inspect the steel girder.

2.5 Applicability of concrete deck

2.5.1 Installation work and deck installation method

For the installation of concrete decks on steel girders, it is necessary to consider time constraint and connection between girders and decks. On the lines with heavy traffic, interruption of train operation is not acceptable. Thus, the installation should be finished within a given time of night work. Available installation time would be at most 4~5 hours. On the other hand, to reduce the stress and increase the load carrying capacity of the girders, decks should be sufficiently connected to the girders. In this study, three types of installation method were proposed, as shown in **Table 2-3**, and discussed their applicability to the existing bridges.

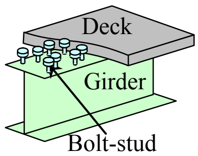
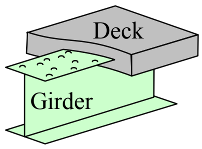
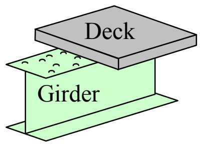
In the “bolt-stud method”, bolt-studs are installed on the upper flange as replacement for existing rivets. Then concrete is casted on the surface of the flange. The bolt-studs can retain shear connection between the girders and the decks.

In the “flange-encasing method”, the whole section of upper flanges of the girder is encased in the concrete. This method can increase more bending stiffness and capacity of the girder than the other methods.

However, the bolt-stud method and the flange-encasing method take several days or a few weeks for hardening of site-cast concrete. Considering the situation of the railway, interruption of train operation for such long time is not acceptable.

In the “pre-cast deck method”, multiple pre-cast concrete decks are installed on the girders and the precast decks can be installed in a short-time work at a site. Furthermore, the installation work for longer bridges can be carried out by separating the long work into several short time night works for each pre-cast deck, as shown in **Fig. 2-8**. For this reason, the pre-cast deck method is the most applicable to existing railway bridges on the view point of installation work. However, it is necessary to consider how to connect the girders and the pre-cast concrete decks.

Table 2-3 Comparison of installation method

Name	Bolt-stud method	Flange encasing method	Pre-cast deck method
Image			
Decks	Site-cast	Site-cast	Precast
Time of installation	Long (A few weeks)	Long (A few weeks)	Short (Several hours)
Connection	Well-connected	Well-connected	Need to be examined
Weight of decks	Light	Heavy	Light
Applicable situation	Shear connection required	Higher stiffness required	Short-time installation

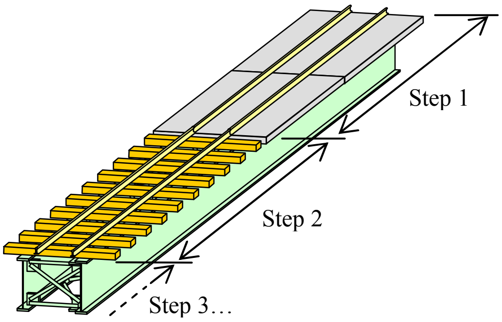


Fig. 2-8 Separation of deck installation work into several night works

2.5.2 Applicable type of concrete deck

When the concrete decks are installed, the rail level should not be changed because the change of the rail level causes shift of track and electronic devices at the site including the sections adjacent to the bridge. Then the depth of the concrete deck should not exceed the depth of existing sleepers.

The depth of existing sleepers is generally 200 mm. Besides, 50 mm of the depth is occupied by irregularity of the upper flange due to rivet heads, connecting layer between girders and decks, and track pad under the rail. Considering these constraints, the acceptable depth of the concrete deck is about 150 mm.

To confirm the applicability of 150 mm deck depth, the capacities and stress of the concrete deck were examined through design calculation. The standard distance between the main girders of existing steel bridges is 1500 to 1700 mm and it is longer than the distance between the rails, as shown in **Fig. 2-9**. Accordingly, the bending moment by the train loads acts on the concrete deck. Here, three types of concrete deck were examined, as shown in **Fig. 2-10**. The concrete deck was modeled of 1700 mm span and 750 mm width, considering the size of areas, which a train axle load affects. It was assumed the loads of 124 kN, considering train load and impact load.

Table 2-4 shows the results of the verification for the bending capacity and the shear capacity of the cross section and stress at the upper and lower edges of the concrete deck. The capacities were verified considering safety, and the stress of concrete was verified considering serviceability. The limit values were estimated according to the design standard[37]. From the results, it was found that the post-tension PC deck is applicable for the required deck depth. If the deck depth larger than assumed is acceptable or the distance between the main girders is smaller than assumed, RC deck or pre-tension PC deck is also applicable.

Considering the durability of the concrete deck, it is necessary to assess the fatigue of the concrete at the compressive side and the fatigue of the reinforcing bars or the prestressing tendons at the tensile side. In this study, the assessment for fatigue is considered to be satisfied because the stress of the concrete is limited as shown in **Table 2-4**.

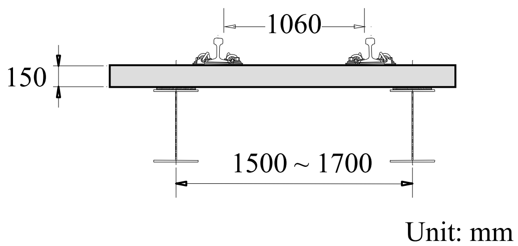


Fig. 2-9 Distance between girders

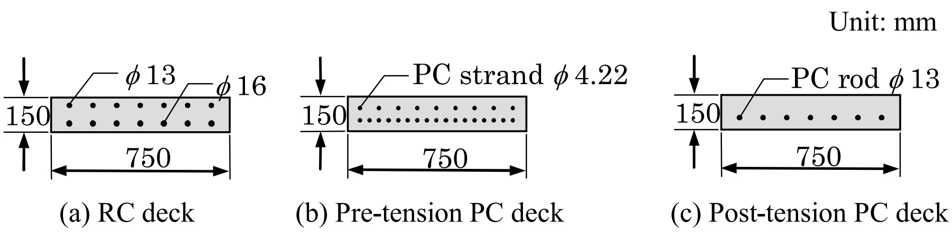


Fig. 2-10 Cross section of concrete decks

Table 2-4 Verification of concrete decks

Type	Safety (Capacity)		Serviceability (Stress, unit: N/mm²)	
	Bending $\gamma_i M/M_{ud}$	Shear $\gamma_i V/V_{yd}$	Upper edge (Compression)	Lower edge (Tension)
(a) RC	1.10>1.0	—	—	—
(b) Pre-tension PC	0.86≤1.0 Satisfied	0.75≤1.0 Satisfied	20.3 > 20.0	1.6 ≤ 2.0 Satisfied
(c) Post-tension PC	0.75≤1.0 Satisfied	0.86≤1.0 Satisfied	14.6 ≤ 20.0 Satisfied	2.0 ≤ 2.0 Satisfied

2.5.3 Use of filler mortar and deck installation procedure

As mentioned before, the use of pre-cast concrete decks is applicable to deck installation in a tight time schedule. However, most of the existing steel bridges are riveted girders, which were constructed before adopting welded girders. On the upper flange of the riveted girders, there are irregularities due to existence of rivet heads and cover plates, as shown in **Fig. 2-11**(a). In such situation, it is difficult to connect the pre-cast decks directly to the steel girders in a simple way.

To enable the connection, a method of connection using filler mortar and steel fasteners was proposed, as shown in **Fig. 2-11**(b). Filler mortar smoothes the irregularities on the surface of the steel girders and enables setting of the pre-cast concrete decks easily.

By applying the concrete deck and girder-deck connection proposed above, the deck installation procedure shown in **Fig. 2-12** is thought to be applicable to actual bridges in short time works. For the detail of girder-deck connection and the detailed installation procedure are discussed in the following chapters (Chap. 3 and Chap. 4).

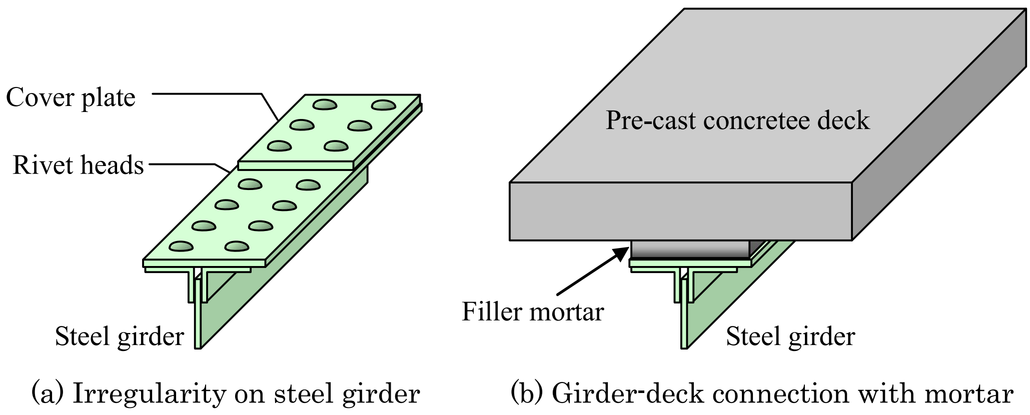


Fig. 2-11 Irregularity on steel girder and connection using filler mortar

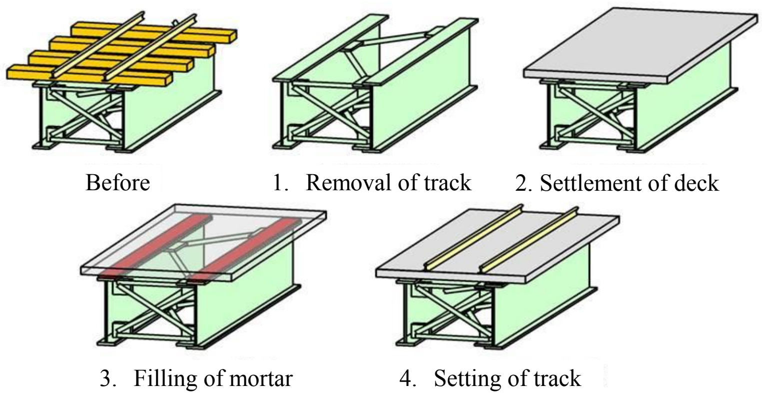


Fig. 2-12 Basic procedure of deck installation

2.6 Test construction of concrete deck

2.6.1 Objective and outline of test

At the stage of the deck installation work, it is necessary to finish the work in required time schedule. **Fig. 2-13** shows an example of time schedule of night work. From the last train to the first train in the next day, time frame available for the installation work is limited to 4~5 hours. Considering removing track before the installation and time for hardening of mortar and placement and adjustment of track after the installation, time for the settlement of pre-cast decks and mortar filling work should be less than 1 hour. To obtain work time for deck installation and setting error of the decks, a test construction of concrete deck was carried out. The specimen for the test, shown in **Fig. 2-14**, is the specimen Type 2 used in the loading test in Chapter 3. It has 8 m span and is the length for five pre-cast concrete decks.

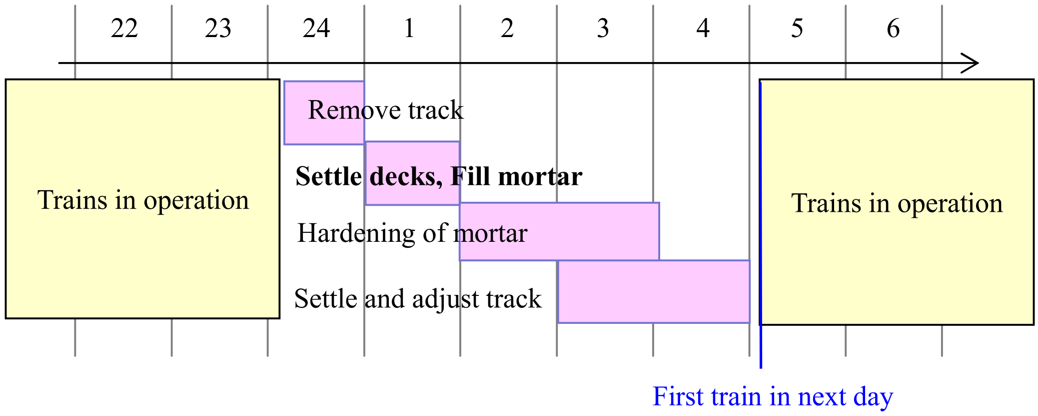


Fig. 2-13 Time constraint in night work

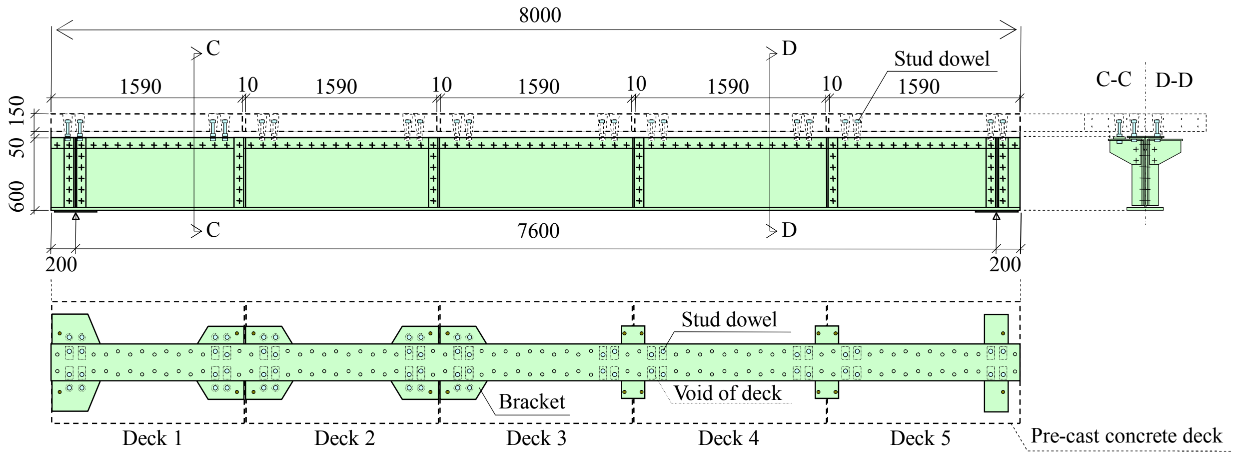


Fig. 2-14 Steel girder used in test construction

2.6.2 Methodology

The pre-cast deck was lifted by crane and settled on the steel girder, as shown in **Fig. 2-15**. After the crane work, the level error and the lateral error were adjusted by operating crowbars by hand. After the adjustment, the mortar was filled between the steel girder and the concrete deck.

(a) Settlement of pre-cast concrete deck

(b) Adjustment of pre-cast deck

(c) Measurement of error

(d) Filling mortar

Fig. 2-15 Test construction of concrete deck

2.6.3 Construction time

Table 2-5 shows time for each work measured in the test construction from pre-cast deck settlement to filling of the mortar. Total time until the mortar filling of all pre-cast panels took about 103 minutes. Summing up the maximum time took for each work, the maximum total time for one panel took 27 minutes. From these results, it is recommended to install one or two panels for each night work.

Table 2-5 Measured time in test construction

Deck	Settlement	Adjustment	Mortar filling
1	2’ 00”	8’ 00”	4’ 30”
2	2’ 50”	13’ 20”	3’ 20”
3	3’ 00”	12’ 30”	7’ 30”
4	3’ 40”	11’ 00”	7’ 00”
5	3’ 00”	11’ 30”	10’ 00”
Max.	**3’ 40”**	**13’ 20”**	**10’ 00”**

2.6.4 Accuracy of deck placement

Table 2-6 shows measured setting error of the pre-cast decks after the adjustment. From this result, it was found that the proposed method enables easy adjustment work by hand and accurate installation of the concrete decks.

Table 2-6 Measured setting error of pre-cast decks

Deck	Transverse error	Longitudinal error	Vertical error
Deck 1	1.0 mm	0	0
Deck 2	0	0	0
Deck 3	0	0.5 mm	0
Deck 4	0	1.0 mm	0
Deck 5	0	1.0 mm	0

2.7 Studies on property of filler mortar

2.7.1 Objective and outline

At the girder-deck connection, the mortar is filled into the gap between the steel girder and the pre-cast concrete decks. In the proposed method, the deck installation should be finished within the night work. In addition, the space for filling mortar should be as small as possible, so that the thickness of the pre-cast concrete deck can be maximized. In addition, the interface of the pre-cast decks and the mortar is inversely casted section and bleeding is concerned at the section. In this study, the property of the mortar was verified and the applicability was discussed, focusing on bleeding property, flowability and bonding strength of the mortar–concrete interface.

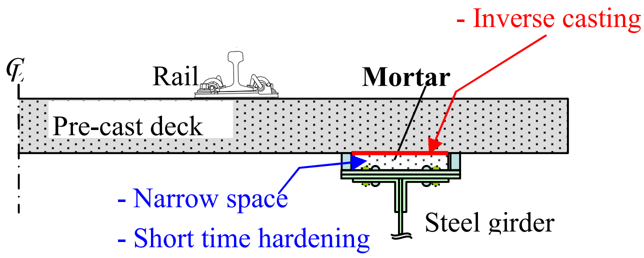


Fig. 2-16 Conditions at the filler mortar

2.7.2 Material

To enable filling work in narrow space and short time work, a pre-packed non-shrink mortar, with high early strength type, for narrow space, was applied.

2.7.3 Bleeding test of mortar

(1) Scope and methodology

As mentioned above, the mortar-concrete interface is inversely casted section. If the bleeding occurs at the mortar, bonding strength of the interface decreases and this is not desirable for the girder-deck connection of the proposed method. For this reason, a bleeding test of the mortar was carried out, in order to obtain the bleeding property of the mortar and to obtain appropriate water ratio for the mortar.

The bleeding property may depend on water ratio. For this reason, the mortar with three different water ratios, *i.e.* 'Specimen A' with 4.3ℓ, 'Specimen B' with 4.5ℓ and 'Specimen C' with 4.6ℓ per 25 kg pre-packed mortar material, were prepared. The test was carried out according to the standard test method by JSCE (JSCE-F 522-2007). Each specimen was prepared by pouring the fresh mortar into a polyethylene bag, as shown in **Fig. 2-17**.

Fig. 2-17 Bleeding test of mortar

(2) Result

As a result of the bleeding test, no bleeding water was seen in each specimen. However, at the top of the specimens, white powdery layer was seen, as shown in **Fig. 2-18**. In the specimen with higher water ratio, more powdery layer was produced. Considering this result, the water ratio of 17.2 % (4.3ℓ per 25 kg) is more desirable than the other water ratio.

(a) Specimen A
Water ratio 17.2 %
(4.3 ℓ/25 kg)

(b) Specimen B
Water ratio 18.0 %
(4.5 ℓ/25 kg)

(c) Specimen C
Water ratio 18.4 %
(4.6 ℓ/25 kg)

Fig. 2-18 White powdery layer at the top of specimens

2.7.4 Flowability test of mortar

(1) Scope and methodology

In the proposed method, the mortar should be filled in the narrow space between the steel girder and the concrete decks. On the other hand, from the result of the bleeding test, lower water ratio is desirable. Therefore, it is important to ensure flowability of the mortar even in a low water ratio. For this reason, flowability test of mortar was carried out.

The test was carried out according to the standard test method called 'J-14 wrought test' (JSCE-F 541-1999). Test material is identical to Specimen A in the bleeding test, *i.e.* the pre-packed mortar with 17.2 % water ratio (4.3 ℓ/25 kg). Considering influence of temperature, the material in two different water temperatures of 11 °C and 25 °C were prepared.

Fig. 2-19 Wrought for flowability test

(2) Result

Table 2-7 shows the flow time obtained in the flowability test. The flow time of the mortar with 25°C water is 7.2 seconds and it means higher flowability than that with 11°C water. From this result, it can be said that the high flowability can be enabled by high temperature water even in a construction in winter.

Table 2-7 Result of flowability test

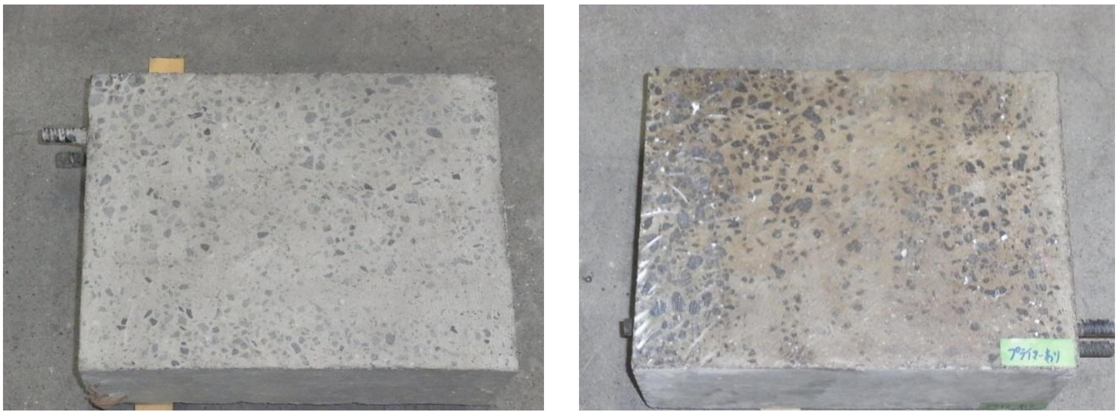
Water temperature	Flow time by J-14 wrought
11°C	9.2 s
25°C	7.2 s

2.7.5 Bonding test of mortar-concrete interface

(1) Scope and methodology

To obtain appropriate surface condition of the concrete deck, a bonding test of mortar-concrete interface was carried out. Focusing on the influence of surface treatment, specimens with two different surface treatment of the concrete deck was prepared, as shown in **Fig. 2-20**. Both of the specimens' surfaces were polished and cleaned in order to enable comparison of both specimens. Only Specimen 1 was painted by primer with acrylic polymer.

To evaluate the bonding strength quantitatively, the test was carried out according to JIS A 1171:2000. The test equipment of the bonding test is shown in **Fig. 2-21**. The mortar, with the flowability of 8.4 sec by J-14 wrought and average compressive strength 47.7 N/mm^2, was poured in inversely casted state. Thickness of the mortar was 10 mm. After the hardening of the mortar (nine days after the casting), the mortar layer was cut out in 40×40 mm square. The pulling out tests were carried out to five different cut squares, in 1500~2000 N per minute.



(a) Specimen 1: Without primer

(b) Specimen 2: With primer

Fig. 2-20 Surface of concrete deck

Fig. 2-21 Test equipment for bonding test

(2) Result

Table 2-8 shows the result of the bonding test. The bonding strength of the each specimen shows large scattering but some subcase of Specimen 1 shows low strength. This is thought because the concrete surface without primer absorbs the water in the fresh mortar and consequently the strength of the mortar was lowered in the area near the interface. Comparing the average strength, Specimen 2 (with primer) has higher bonding strength. From this result, it is recommended to put primer on the surface of the pre-cast concrete decks.

Table 2-8 Result of bonding test

Subcase	Specimen 1 (Without primer)		Specimen 2 (With primer)	
	Max. load (kN)	Strength (N/mm^2)	Max. load (kN)	Strength(N/mm^2)
1	2.1	1.31	2.0	1.25
2	0.9	0.56	1.5	0.94
3	1.3	0.81	2.0	1.25
4	1.9	1.19	3.4	2.13
5	0.9	0.56	1.5	0.94
Average	**1.42**	**0.89**	**2.1**	**1.30**

2.8 Summary

In this chapter, a performance improvement method by installing concrete deck was proposed. For the proposed method, expected effects and influences by installing concrete deck were discussed based on design calculation. Considering application into actual bridges, type of concrete deck and deck installation procedure were discussed. In order to confirm deck installation time and accuracy of installation, the test construction of concrete deck was carried out. Focusing on the filler mortar, the property of the fresh mortar and bonding strength of the mortar-concrete interface were tested. Through these studies, following results were obtained.

(1) Based on design calculations, stress at the upper flange can be decreased by 50 ~ 80 % and stress at the lower flange can be reduced by 10 ~ 20 %, by applying the proposed method. Especially at the upper flange, stress can be drastically reduced. The stress reduction effect is relatively large in shorter span bridges.

(2) In consequence of the stress reduction, the remaining fatigue life of the bridge can be elongated. Focusing on the lower flange, the remaining fatigue life can be increased to 1.4 ~ 2.3 times compared to the condition before the deck installation.

(3) In consequence of the stress reduction, the bending capacity of the bridges can be increased by 20 ~ 50 % after installing the concrete decks.

(4) Considering previous measurement result, reduction of noise is expected by installing concrete deck.

(5) The increase of dead load by installing concrete deck affects on bridge supports. Focusing on the stress of support concrete, the proposed method is applicable to the bridges shorter than 40 m span. For the bridges exceeding 40 m span, the actual strength of the support concrete should be obtained to judge the applicability.

(6) Applicable type of concrete deck was discussed based on design calculations. Under the conditions of maximum depth and distance between steel girders, post-tension pre-stressed concrete is applicable.

(7) To enable the installation of the concrete deck in limited work time schedule in night-time work, use of the pre-cast concrete decks was proposed. Considering the upper surface of the steel girders with irregularities, a girder-deck connection method using filler mortar was proposed.

(8) Through the test construction, it was found that the deck installation work, such as setting of the pre-cast concrete deck and adjustment and mortar filling, takes at most 27 minutes for each pre-cast panel. Considering time constraint at the night work, at least two pre-cast panels can be installed in each night work. The proposed method using pre-cast concrete decks enables easy adjustment by hand and the deck panels can be adjusted in good accuracy with about 1.0 mm error.

(9) Through the bleeding test of the filler mortar, it was found that no bleeding water occurred in the non-shrinkage mortar but white powdery layer can be generated at the top of the mortar if the water ratio is high. Thus, it is recommended to reduce the water ratio.

(10) Through the flowability test of the filler mortar, it was found that the mortar mixed with high temperature water shows high flowability even with low water ratio.

(11) Through the bonding test of the filler mortar, it was found that the mortar-concrete interface shows higher bonding strength if the surface of the concrete is painted by primer. To avoid dry out of the mortar, it is recommended to put primer on the concrete surface.

Chapter 3. Development of Girder-Deck Connection

3.1 Introduction

In the last chapter, the method of constructing the concrete deck using pre-cast concrete decks and filler mortar was proposed. In this chapter, the structural detail of the connection between the existing steel girders and the newly installed pre-cast concrete decks (hereinafter 'girder-deck connection') was proposed and the capacity of the connection was confirmed by loading tests. Furthermore, loading tests of deck-installed girders were carried out to clarify the condition that the composite effect of the steel girders and the concrete decks can be obtained.

3.2 Detail of girder-deck connection

While the train load acts on the bridge, the girder-deck connection is under vertical force, lateral force and shear force, as shown in **Fig. 3-1**. The vertical force is acted by the axle load of passing train. The lateral force is mainly acted by the horizontal load of train. The shear force is mainly acted by the bending behaviour of the composite girder. Considering these forces, two types of girder-deck connection were proposed, as shown in **Fig. 3-2**.

In type 1 connection, the horizontal fasteners resist against the lateral force. The vertical fasteners resist against the up-lifting force by the vertical force. The filler mortar smoothes the irregularities on the surface of the steel girders and enables setting of the pre-cast concrete decks easily. Furthermore, the filler mortar is expected to transmit the shear force between girders and decks by the irregularities by rivet heads on the steel surface.

In type 2 connection, the stud dowels are expected to resist against the lateral force, the up-lifting vertical force and the longitudinal shear force. For newly constructed composite bridges with pre-cast concrete decks, the decks are generally connected to girders with shear connectors. However, the filler mortar is also expected to resist against the longitudinal shear force.

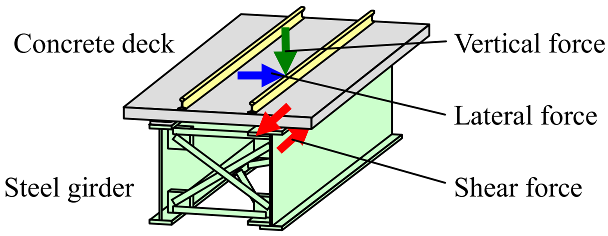


Fig. 3-1 Forces acting on girder-deck connection

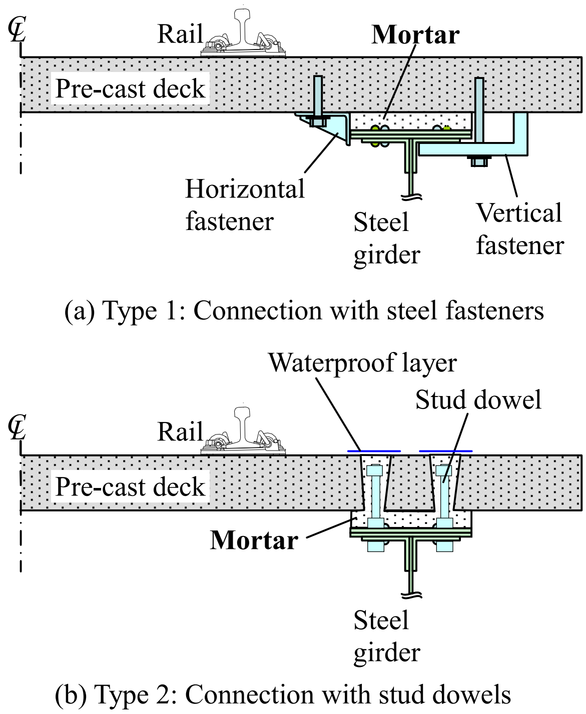


Fig. 3-2 Detail of girder-deck connection

3.3 Loading tests of girder-deck connection by steel fasteners

3.3.1 Objective and outline of experiment

In the Type-1 girder-deck connection, steel fasteners are expected to resist against the lateral force and the uplifting vertical force. However, such connection method is not generally applied for other composite structures. To confirm the capacity of the connection with fasteners, the lateral loading test and the uplifting loading test of the fasteners were carried out.

Fig. 3-4 shows the steel fasteners used in the loading test. Horizontal fasteners were installed at the inside of the steel girders and resist against the transverse force. It has a tapered washer in it and it is effective for adjustment of setting errors and it makes connection tighter when a transverse force is applied. Vertical fasteners were installed at the outside of the girder and resist against uplifting force. At each location of the fasteners, a steel insert screw thread was installed to the lower surface of the pre-cast concrete deck.

For the loading condition, design loads to each specimen for two cycles were applied. The design loads were calculated considering external forces by a train axle shown in **Fig. 3-3**, according to the design standard[38]. At the third cycle, increased load until the maximum loads of the specimens was applied.

Fig. 3-5 shows the condition of loading of the tests. Length of the specimen was set at 1 m considering a length of the area where one axle load is applied. Two fasteners were installed for each specimen. Here the filler mortar was not filled between the girder and the deck in order to evaluate the capacity of the fasteners. In order to evaluate the strength of the connection only by fasteners, rubber-banded steel plates were put between girder and deck instead of filling the mortar.

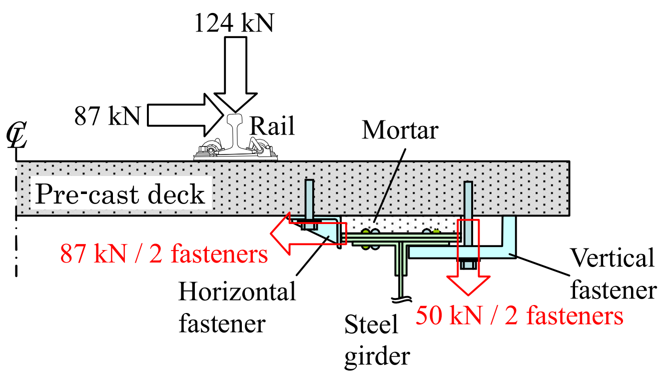
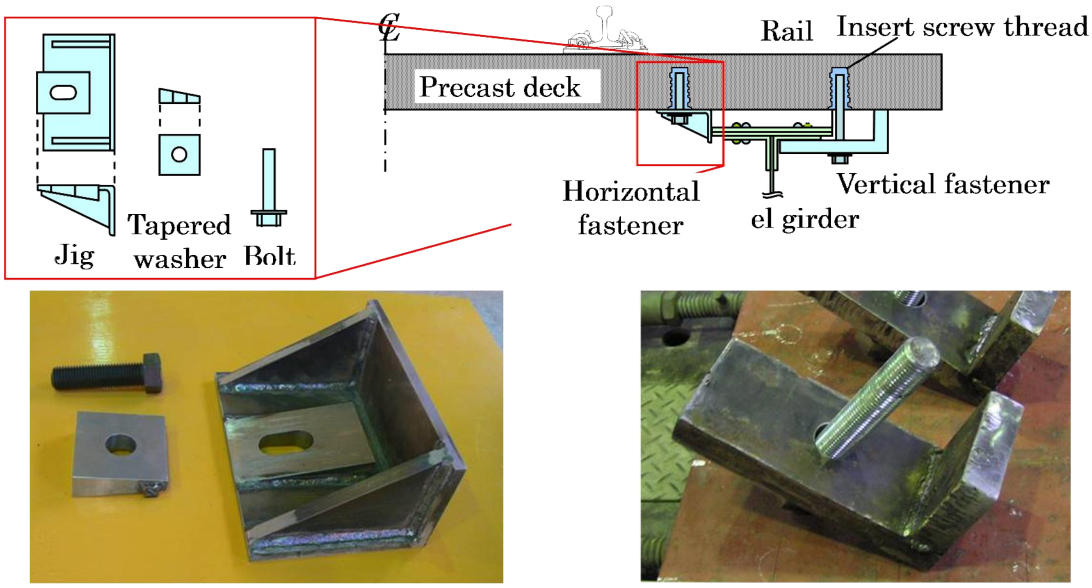


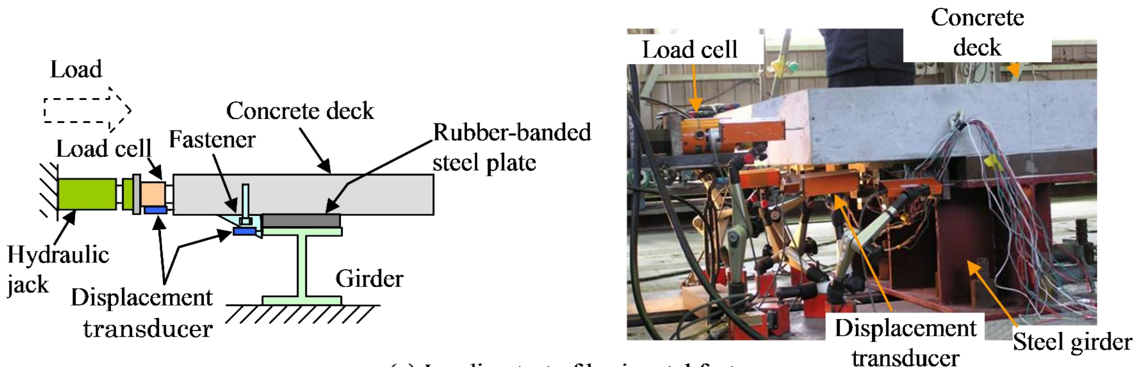
Fig. 3-3 External forces and design loads on steel fasteners



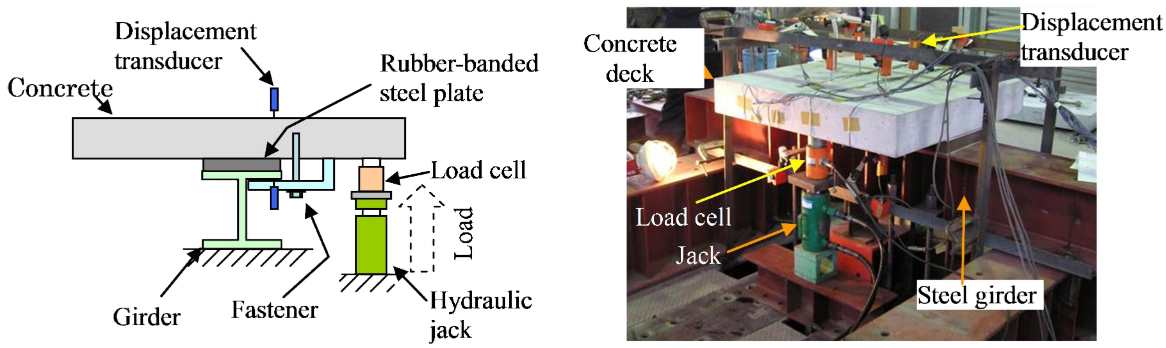
(a) Horizontal fastener

(b) Vertical fastener

Fig. 3-4 Steel fasteners for pre-cast concrete decks



(a) Loading test of horizontal fastener



(b) Loading test of vertical fastener

Fig. 3-5 Loading tests of steel fasteners

3.3.2 Test results

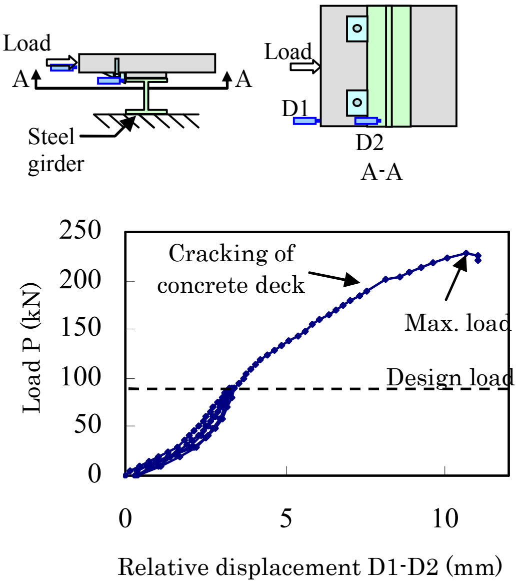
Fig. 3-6 shows the load-displacement relationship of each test. **Table 3-1** shows the maximum load at each loading test. From the results, it was found that the connection by fasteners has enough capacity against uplifting and transverse loads. After the loading test, the concrete deck was observed. **Fig. 3-7** shows the lower surface of the tested concrete deck. Around the insert screw thread, where the fasters were settled, cracks were observed in the concrete deck. From this result, it is considered that the concrete around the insert screw thread failed at the test.

Through the loading tests, it was found that the steel fasteners have enough capacity for designed vertical and transverse load. However, it is necessary to examine the capacity of the connection in the longitudinal direction. The capacity of the connection to the longitudinal shear force is discussed in Chap. 4.

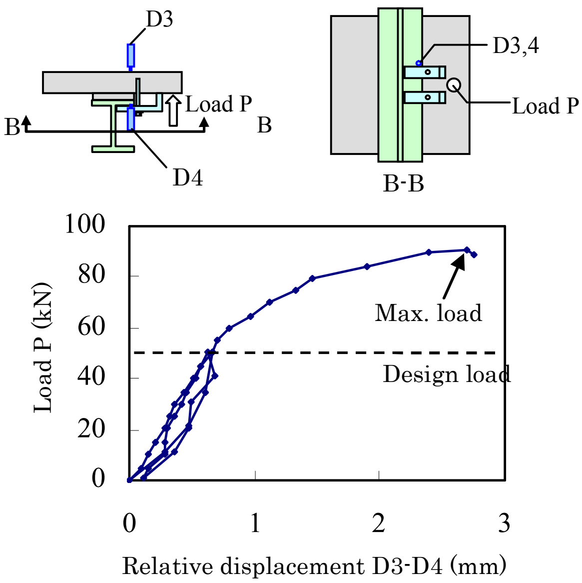
Table 3-1 Capacity of connection by fasteners

Type	Capacity of specimen	Capacity per fastener
Vertical	90.3	45.2
Horizontal	229.0	114.5

Unit: kN



(a) Loading test of horizontal fastener



(b) Loading test of vertical fastener

Fig. 3-6 Load-displacement relationship

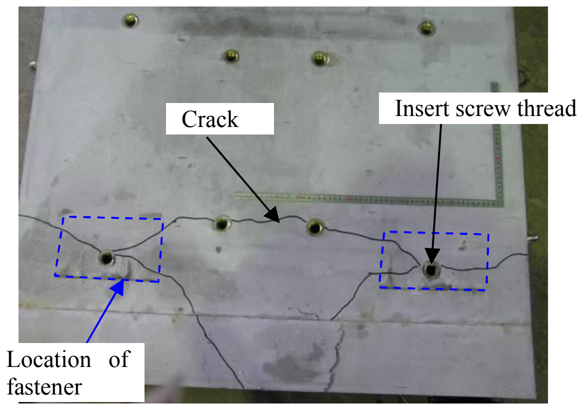


Fig. 3-7 Crack of concrete deck (after loading test of horizontal fastener)

3.4 Loading tests of steel girder with pre-cast concrete decks

3.4.1 Objective and outline of experiment

Considering the behavior of girder in bending condition, bending tests of the deck-installed girder were carried out. The objectives of the tests are as follows:

- To figure out the behavior of the girder to which the proposed methods are applied.
- To obtain necessary condition of the connections, i.e. girder-deck connections and deck connections (mentioned in 3.4.2).

The tests were carried out as static four-point bending tests. The load was gradually increased up to 200 kN, considering maximum train axle load is 180 kN. **Fig. 3-8** shows the specimen under loading test.

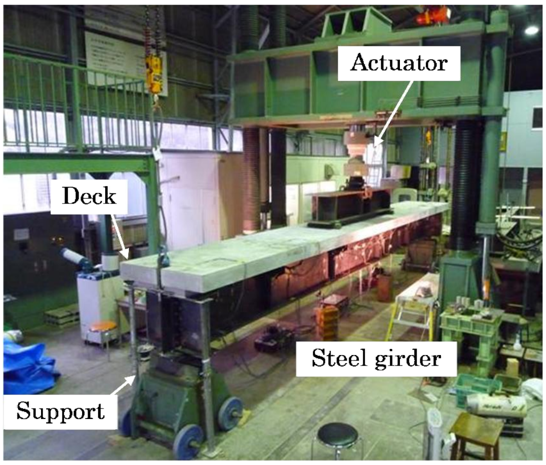


Fig. 3-8 Specimen under loading test

3.4.2 Parameters

Table 3-2 shows the test parameters. Considering these parameters, two specimens were tested. In order to compare the following parameter, each specimen has two different parameters in its sides (left hand side and right hand side, described below).

The first parameter is the type of girder-deck connection. As shown in 3.2, Specimen 1 has steel fasteners and it has no stud dowels on the upper flange. Specimen 2 has no steel fasteners but stud dowels on the upper flange. Comparing these specimens, the effect of stud dowel on the behavior of girder can be discussed.

The second parameter is the type of connections between the pre-cast decks. As mentioned before, installation work of the concrete decks is under tight time constraint. Therefore, it is desirable to reduce process of works. To clarify the effect of filling mortar between the pre-cast decks, mortar at the gap between the decks on Specimen 2 (Type 2A and 2B) and on the right side of Specimen 1 (Type 1B) was omitted.

Table 3-2 Test parameters for loading test

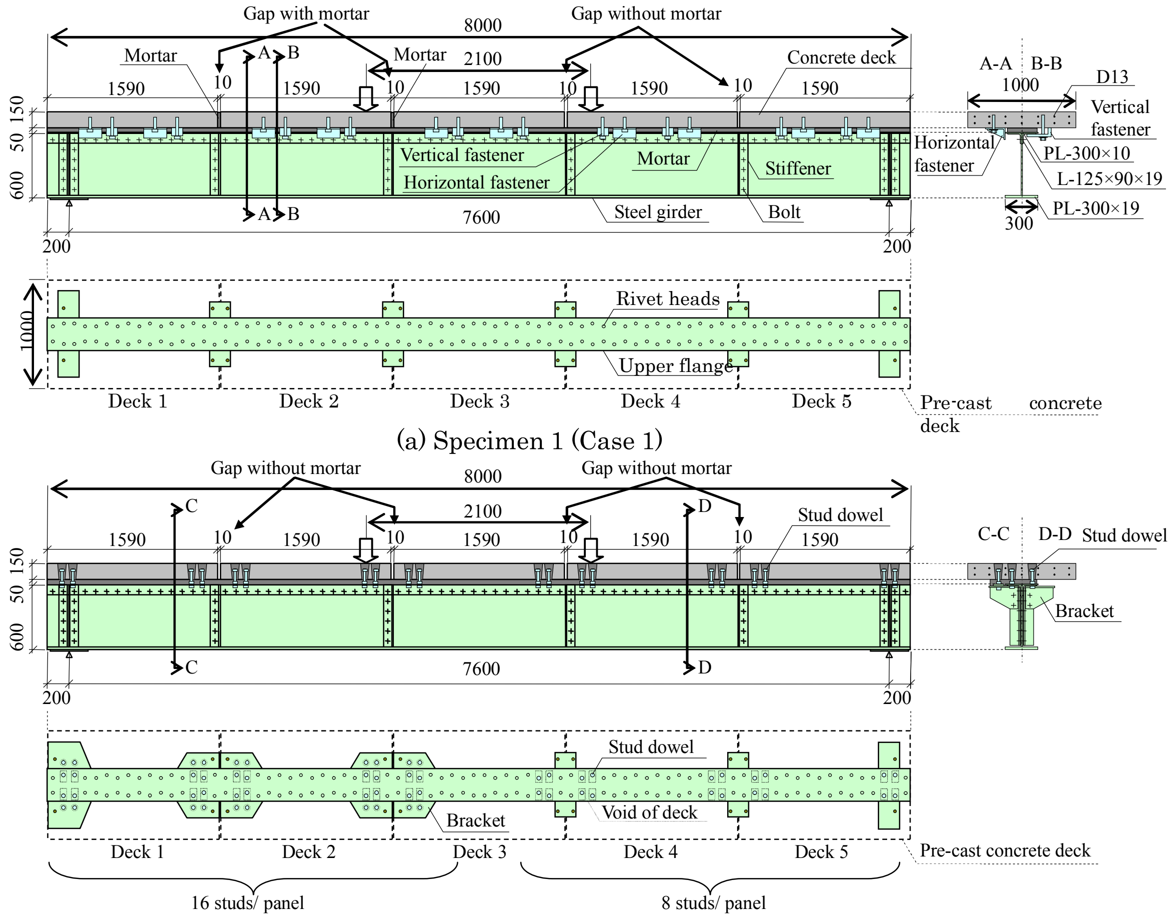
Parameter	Specimen 1		Specimen 2	
	Left side (Type 1A)	Right side (Type 1B)	Left side (Type 2A)	Right side (Type 2B)
Shear connector	No shear connector		Stud dowel (Bolt type)	
Steel fastener	With fasteners		No fastener	
Gap between pre-cast decks	Filler mortar (10 mm)	No joint (10 mm gap)	No joint (10 mm gap)	No joint (10 mm gap)

3.4.3 Test specimens

Fig. 3-9 shows the dimensions of the specimens. The specimens consist of a steel girder, pre-cast concrete decks, mortar, and connecting devices. The steel girder with 8 m length was fabricated by high-strength bolts and angle steels for the upper flange, in order to reproduce the existing riveted girder. To represent the rivet heads on the upper flange of the steel girder, torque-shear type high-strength bolts were installed on the upper flange. To represent the existing painting, the upper flange has its surface with mill scale. The lower flange was welded to the web. Although the girder depth of 600 mm is smaller than that of actual bridge, lower flange thicker than usual were applied to keep the neutral axis of the composite girder lower than the upper flange, considering height of neutral axis in actual bridges.

As mentioned in 3.4.2, there are differences in parameters between Specimen 1 and 2 and between the left hand side and the right hand side of each specimen. In Specimen 1, no shear connectors but steel fasteners were installed. In Specimen 2 no fasteners but stud dowels as shear connectors were installed. The stud dowel consists of a bolt and nuts and its height is shorter than the depth of the pre-cast deck, so that it can be installed prior to the deck-installing night work. On the left side of Specimen 2, 16 studs are installed for each panel (Type 2A). On the right side of Specimen 2, 8 studs are installed for each panel (Type 2B).

There are gaps between the pre-cast panels. On the left side of Specimen 1, the gap between Decks 1-2 and the gap between Decks 2-3 are filled with mortar (Type 1A). On the other gap has no filler mortar (Deck 3-4, 4-5 in Specimen 1 (Type 1B) and all gaps in Specimen 2 (Type 2A and 2B)).



(a) Specimen 1 (Case 1)

(b) Specimen 2 (Case 2)

Fig. 3-9 Test specimens for loading test and dimensions

3.4.4 Test results

(1) Stiffness of girder

Fig. 3-10 shows load-displacement relationship of each specimen. The vertical displacement u_1 and u_2 of the girder were measured at the sections of loading points. The graphs also show the calculated results under the following conditions:

Model 1: A girder assuming fully composite state

Model 2: A girder whose left hand side is assumed to be composite state and right hand side is assumed to be non-composite state

Model 3: A girder assuming non-composite state

Consequently, the deformation of Model 2 shows unsymmetrical distribution of the displacement, as shown in **Fig. 3-10**.

In Specimen 1, the test result of u_1 is similar to the calculated u_1 of Model 2 until 120 kN loading. Under the loading over 120 kN, the test result of u_1 become larger than the calculated u_1

of Model 2. On the other hand, the test result of u_2 is similar to u_2 of Model 2. In Specimen 2, both u_1 and u_2 are larger than those of Model 2. Specimen 2 also shows change in load-displacement relationship around 140 kN loading.

From these results, it is considered that the left side of Specimen 1 has composite section and the right side has non-composite section. However, the state was changed around 120 kN loading. On the other hand, neither of the sides of Specimen 2 has composite section.

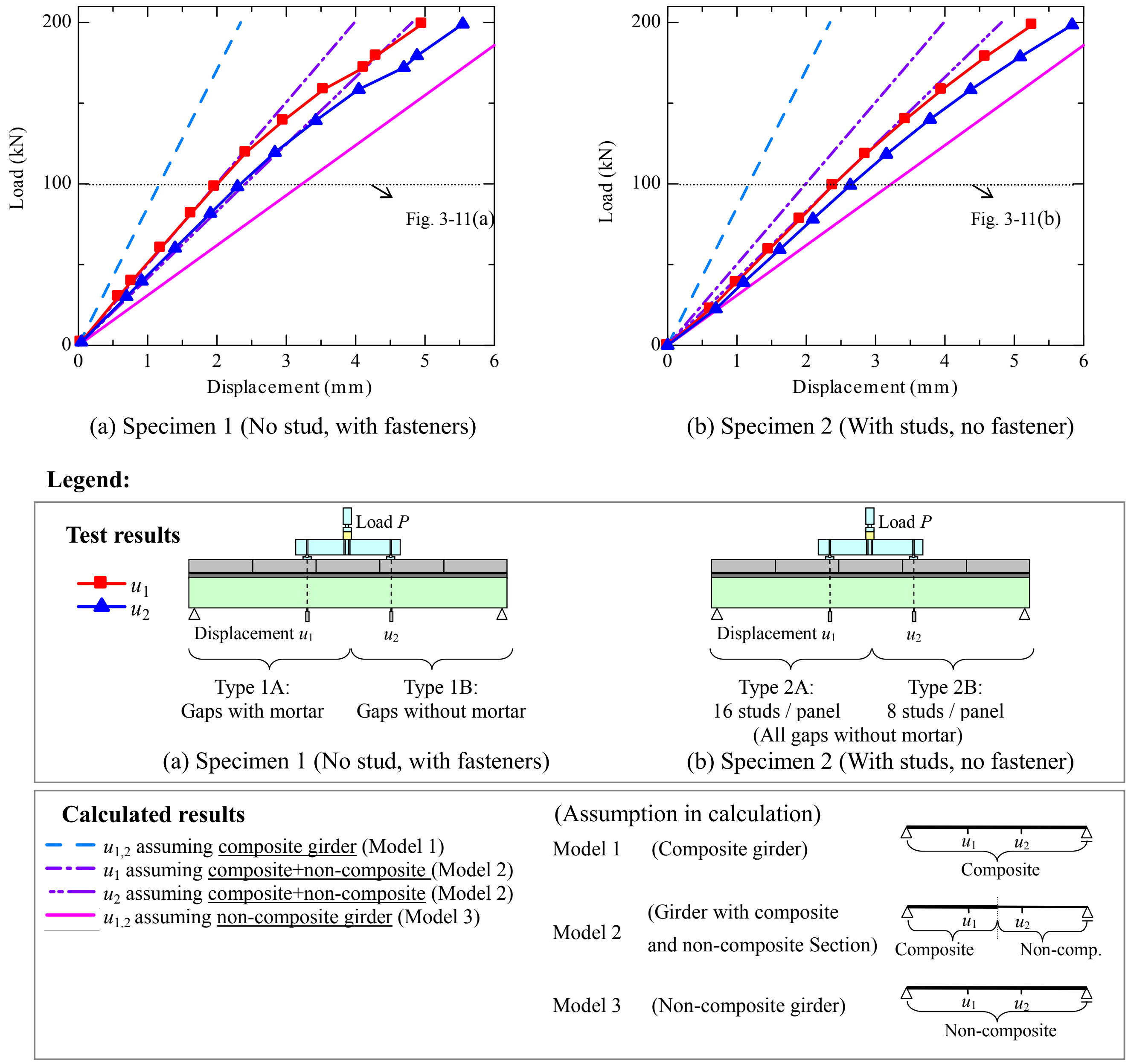
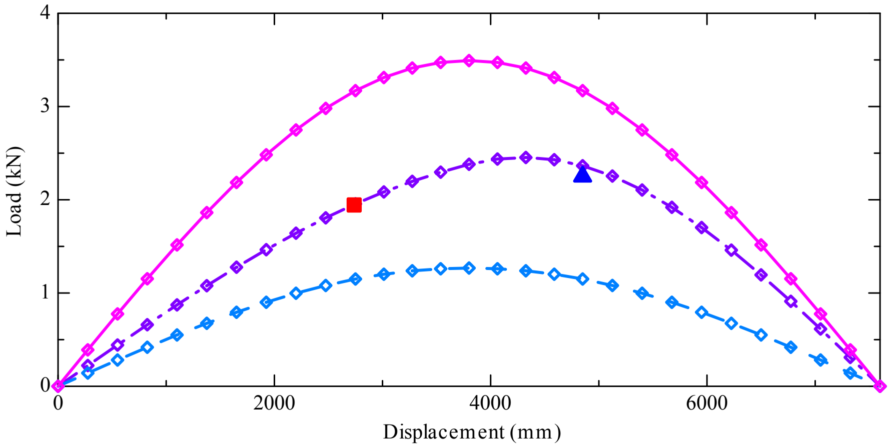
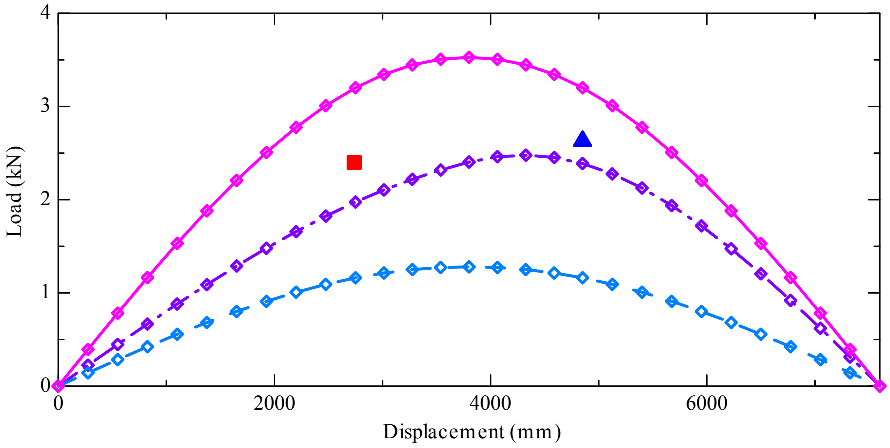


Fig. 3-10 Load-displacement relationship



(a) Specimen 1 (No stud, with fasteners, 100kN loading)



(b) Specimen 2 (With studs, no fastener, 100kN loading)

Legend:

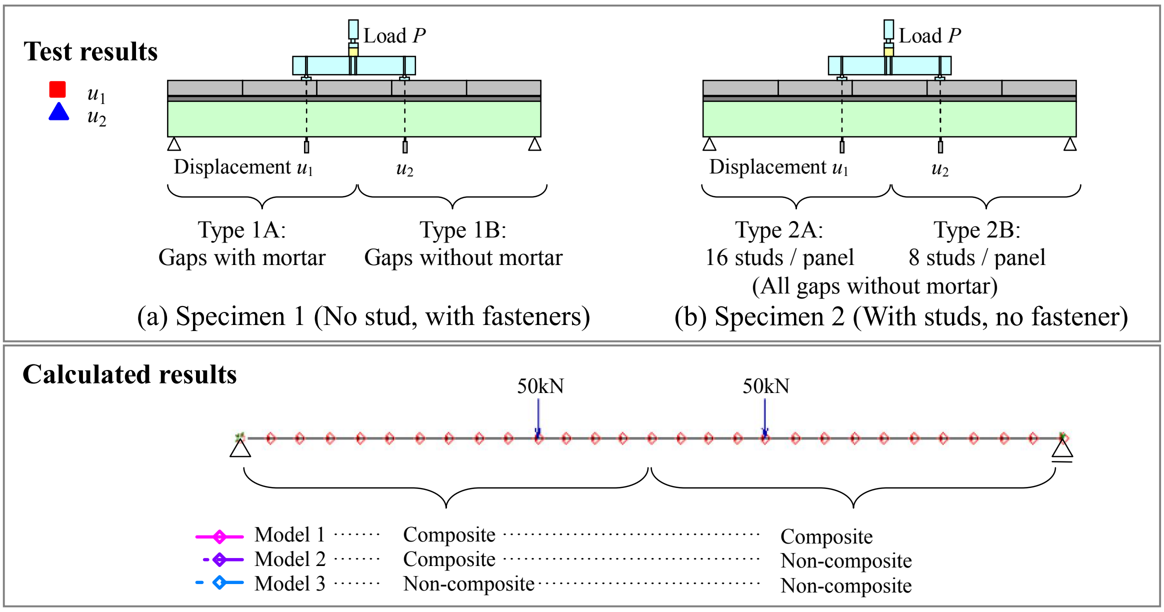
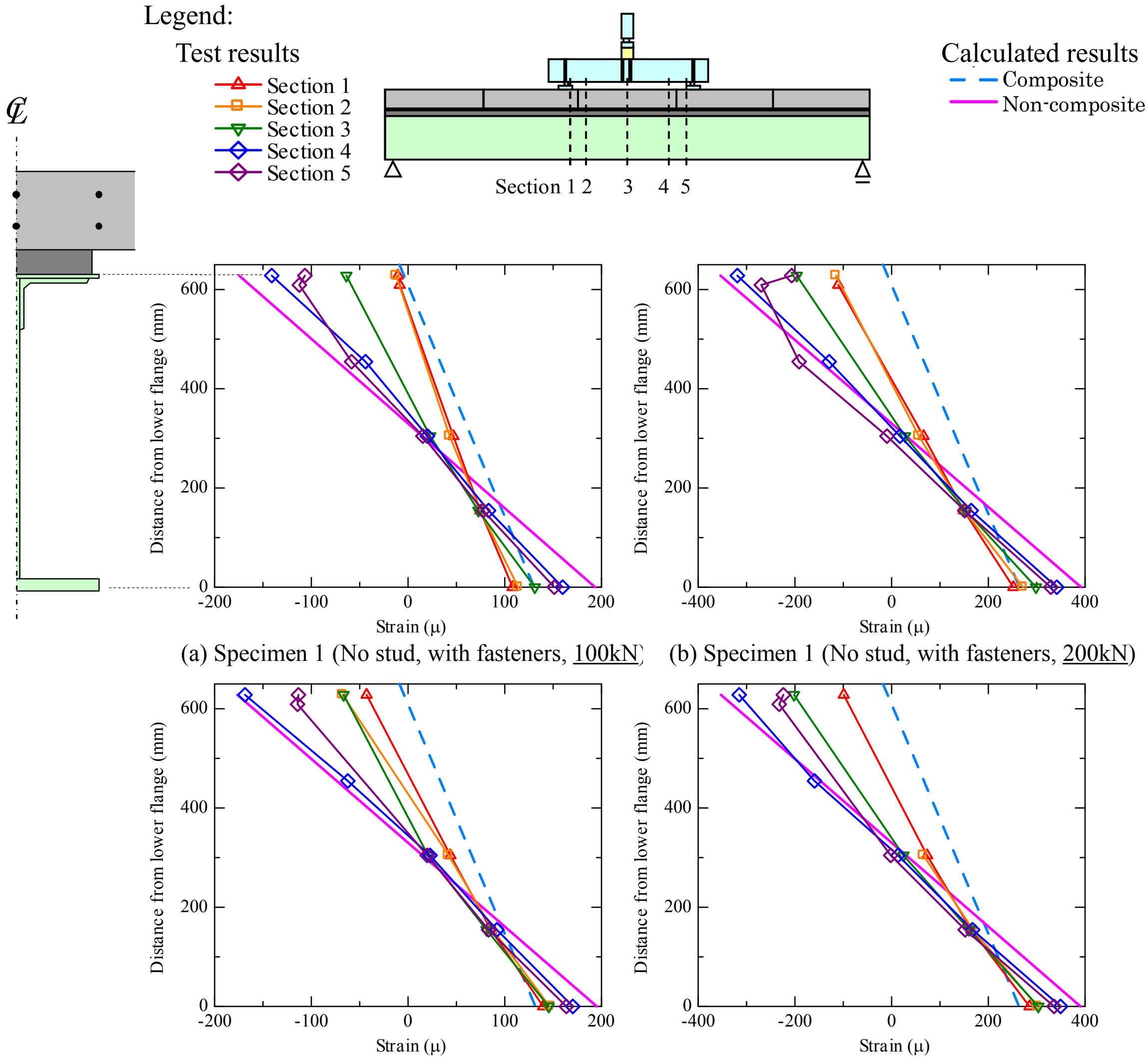


Fig. 3-11 Displacement of girder at 100 kN loading

(2) Strain distribution and state of section

To clarify the cause of state change in load-displacement relationship, strain distribution before and after the state change were compared, focusing in strain distribution of the steel girders. **Fig. 3-12** shows vertical distribution of the strain in the longitudinal direction at the sections in each specimen. The graphs also show the calculated results in composite and non-composite state, obtained through the beam theory calculation. The results of Case 1 at 100 kN loading are shown in **Fig. 3-12** (a), as a state before the change. At Section 1 and 2 in the left side of the specimen, the strain is similar to that of composite girder state. At Section 3, the strain state showed that the section has a middle state between composite and non-composite. At Section 4 and 5 in the right side, the strain is similar to that of non-composite girder state. The results at 200 kN loading are shown in **Fig. 3-12** (b), as a state after the change. The strain at Section 1 and 2 changed its state to the middle state. The test results of Case 2 at 100 kN loading are shown in **Fig. 3-12** (c). Section 1, 2 and 3 showed middle state and Section 4 and 5 showed non-composite state. At the 200 kN loading (**Fig. 3-12** (d)), the sections did not show much difference from those at 100 kN loading. These states of each specimen are summarized in **Table 3-3**.



(a) Specimen 1 (No stud, with fasteners, 100kN) (b) Specimen 1 (No stud, with fasteners, 200kN)

(c) Specimen 2 (With studs, no fastener, 100kN) (d) Specimen 2 (With studs, no fastener, 200kN)

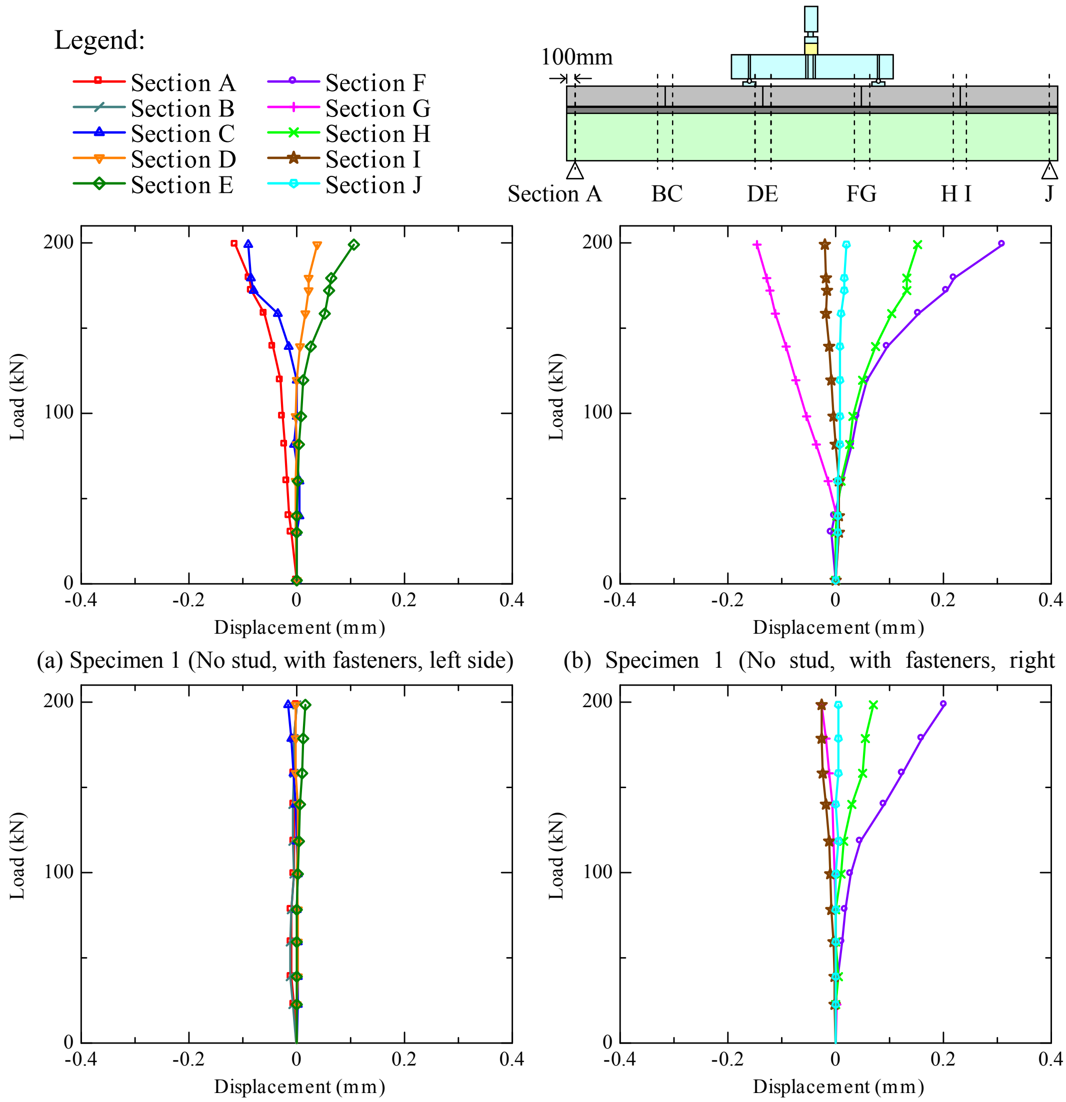
Fig. 3-12 Distribution of nominal strain in each section

Table 3-3 State of each section presumed from displacement and strain

Section	Case 1 (no studs)		Case 2 (with studs)	
	100 kN	200 kN	100 kN	200 kN
1	Composite	Middle	Middle	Middle
2	Composite	Middle	Middle	Middle
3	Middle	Middle	Middle	Middle
4	Non-composite	Non-composite	Non-composite	Non-composite
5	Non-composite	Non-composite	Non-composite	Non-composite

(3) Slip state of concrete decks

Fig. 3-13 shows relative displacement between the upper flange of the steel girder and the lower surface of the concrete decks in longitudinal direction. At Section A~F of Case 1, the relative displacement started to increase from 140 kN loading. On the other hand, at Section F~J of Case 1, the displacement started to increase from 60~80 kN. At Section A~F of Case 2, the displacement did not increase. On the other hand, at the section F~J, the displacement increased from 120 kN. **Table 3-4** shows the load when the slip displacement of each section started to increase.



(a) Specimen 1 (No stud, with fasteners, left side) (b) Specimen 1 (No stud, with fasteners, right

(c) Specimen 2 (With studs, no fastener, left side) (d) Specimen 2 (With studs, no fastener, right side)

Fig. 3-13 Relative displacement between steel girder and concrete deck

Table 3-4 Slip load of girder-deck connection

(a) Specimen 1 (No stud, with fasteners)

Deck	Deck 1		Deck 2		Deck 3		Deck 4		Deck 5	
Side	Left	Right	Left	Right	Left	Right	Left	Right	Left	Right
Slip load	140 kN	-	140 kN	140 kN	140 kN	60 kN	60 kN	80 kN	80 kN	80 kN

(b) Specimen 2 (With studs, no fastener)

Deck	Deck 1		Deck 2		Deck 3		Deck 4		Deck 5	
Side	Left	Right	Left	Right	Left	Right	Left	Right	Left	Right
Slip load	No slip						60 kN	100 kN	120 kN	80 kN

(4) Influence of slip state and condition of gaps

Table 3-5 shows the state of the specimens, from the view points of the slip state of the girder-deck connection and the condition of gaps between the pre-cast decks.

Type 1A (left side of Specimen 1) showed composite state until 120 kN loading. The gaps between the pre-cast decks were filled with mortar and the girder-deck connection had no slip until 120 kN. However, the section could not keep composite state because the connection started to slip. It means the slip occurs under the train axle load (180 kN).

On the other hand, Type 2A (left side of Specimen 2) had no slip even under the load over the train load. However, the section showed middle state between composite and non-composite state because the gaps between pre-cast decks were not filled with mortar.

From these results, it was found that it is necessary to prevent the slip of the girder-deck connection, *i.e.*, to install stud dowels onto the steel girders, and to fill the gaps between pre-cast decks with mortar in order to keep the girder composite state.

Table 3-5 Influence of slip state and condition of gaps on state of section

Gaps between pre-cast decks	Slip state of girder-deck connection	
	No slip	Slip
With filler mortar	**Composite state** (Type 1A (up to 120 kN))	**Middle state** (Type1A (from 140 kN))
Without filler mortar (gap)	**Middle state** (Type 2A)	**Non-composite state** (Type 1B and 2B)

3.5 Summary

In this chapter, the structural detail of the girder-deck connection was proposed. To evaluate the capacity of the steel fasteners, the lateral loading test and the uplifting loading test of fasteners were carried out. In order to clarify the condition for the composite effect of the steel girders and the concrete decks, the loading tests of the deck-installed girders were carried out. Through these studies, the following results were obtained.

(1) The steel fasteners have enough capacity for designed vertical and transverse load.

(2) The filler mortar shows resistance against the longitudinal shear force until the slip occurs at the steel-mortar interface. The slip state of the girder-deck connection affects the composite effect of the girder.

(3) The stud dowels are helpful to prevent the slip of the connection if there are enough stud dowels.

(4) If the gaps between pre-cast decks are not filled, the girder does not show composite behaviour even if the slip does not occur at the girder-deck connection.

(5) Considering these facts, it is necessary to prevent slip of the girder-deck connection and ensure the transmission of force between the pre-cast deck panels in order to obtain the composite effect.

To discuss the limit state of the girder-deck connection in longitudinal direction for longer span bridges, it is necessary to clarify the behavior of the connection in larger load range. It is discussed in the next chapter.

Chapter 4. Structural Performance of Deck-installed Girder

4.1 Introduction

Through the loading tests of the deck-installed girder shown in Chapter 3, the performance of the girder up to train load was confirmed. As a result, it was found that the slip state of the girder-deck connection has large influence on the performance of the girder. However, the structural performance of the deck-installed girder under the load exceeding the train load should be examined. Considering the redundancy of the girder, the girder-deck connection should not fail before reaching the yielding capacity of the original steel girder. Thus, it is important to ensure capacity of the girder-deck connection.

In the conventional design of composite bridges, bonding between steel girder and concrete deck is ignored and the shear resistance is ensured by putting many shear connectors on the steel girder [38]. There is also a method of making composite action of existing non-composite bridges by post-installed shear connectors[39]. However, on the existing steel bridges, it is not always possible to install many shear connectors, because of installation work at the site. On the other hand, according to the previous researches, it was found that the actual non-composite bridges, *i.e.* steel bridges with concrete deck in which the composite action is ignored, actually behave as composite girder [40]. In another research, it was found that the shear force is transmitted by bonding between steel and concrete even if there are shear connectors[41]. Furthermore, higher shear strength of the steel-concrete interface can be obtained if there are irregularities such as checker plates [42]. Considering these facts, it may be possible to transmit the shear force between the steel girders and the concrete decks can be obtained by interfacial bonding, without installing many shear connectors.

However, the mortar materials tend to have smaller elastic modulus than concrete and it is not sure whether the composite girder using mortar at the girder-deck connection and gaps between the pre-cast concrete decks affects the behavior of the girder or not. Thus, it is necessary to confirm the structural performance and failure mode of the girder, which is applied the proposed method. Furthermore, the thickness of the filler mortar at each connection is considered to affect not only the structural performance but also the workability of the mortar filling.

In this chapter, loading tests of composite girder specimen with steel girder and pre-cast concrete deck, connected by filler mortar, were carried out in order to figure out the structural performance of the composite girder.

4.2 Loading test of deck-installed girder

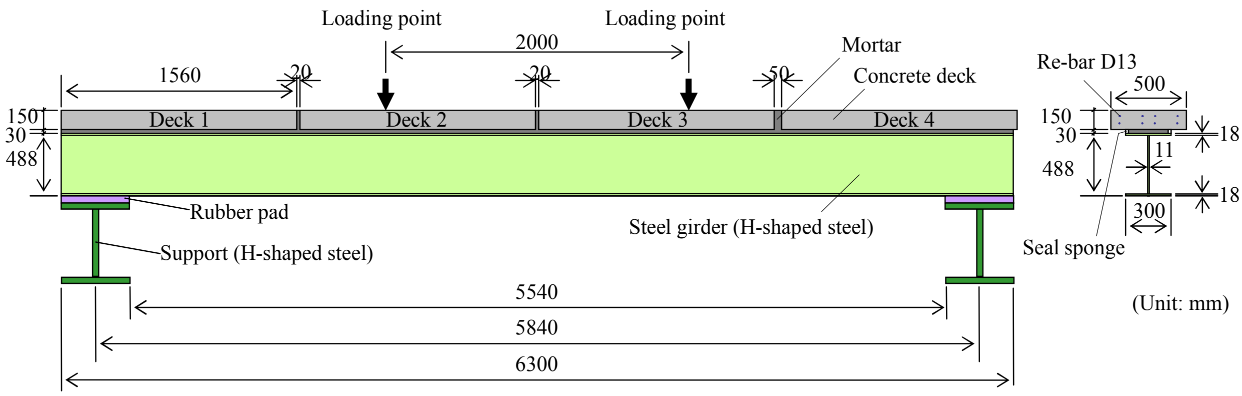
4.2.1 Test specimens

(1) Size of specimens

Fig. 4-1 shows the dimensions of the specimens. H-shaped steel girders with length of 6.3 m and girder depth of 488 mm were used for the specimen. The width of the flange was set to 300 mm considering the size of the flanges in existing steel girder bridges. For the concrete deck and filler mortar, actual thickness was applied in order to avoid size effect. Thickness of the pre-cast concrete was set to 150 mm considering the depth of the existing sleepers (about 200 mm), mortar layer and track equipment.　Considering the height of the neutral axis in the composite section, the width of the concrete deck was set to 500 mm, so that the neutral axis locates under the upper flange. The length of the pre-cast concrete panel set to 1560 mm considering actual panel size of 1~2 m for ease of handling in construction sites.

(2)　Thickness of mortar layer

To investigate the influence of mortar thickness on workability and structural performance, different mortar thickness at the girder-deck connection and the gap between pre-cast panels were set in each specimen.　The mortar thickness at the girder-deck connection was set to 30 mm in Specimen 1 and 20 mm in Specimen 2.　The thickness of the gap between the pre-cast panels was set to 20 mm at the gap between Deck 1 and Deck 2 (hereinafter called 'left gap') and 50 mm at the gap between Deck 3 and Deck 4 (hereinafter called 'right gap').



(a) Side view　　(b) Sectional view

Fig. 4-1 Dimensions of specimen (Specimen 1)

(3)　Fabrication of specimens

Fabrication of specimens was carried out according to the installation procedure proposed in the last chapter as shown in **Fig. 4-2**, in order to confirm the workability of mortar filling.

The upper surface of the steel girder contacts to the filler mortar.　To ensure bonding of the steel-mortar interface, the surface was shot-blasted before the deck installation.　At the upper flange, torque-shear type bolt (S10T, M22) was installed to represent the irregularity by the rivet heads, as shown in **Fig. 4-2** (a).

After the surface treatment of the steel girder, seal sponge was put on the girder as a formwork of mortar filling, as shown in **Fig. 4-2** (b).　The lower surface of the pre-cast concrete deck was roughened and primer was painted in order to ensure bonding of mortar-concrete interface.

After the settlement and adjustment of the pre-cast decks, mortar was filled from the holes of the pre-cast decks.　To ensure flowability of the mortar, the water with temperature of 25 °C was used. To extract the air out of the girder-deck connection and ensure contact of mortar-concrete interface, the mortar was poured using compressor.

Applying above-mentioned method for mortar filling, the mortar layer with good quality was obtained in both of the specimens. From these results, it was found that there is no problem in filling mortar in the range of the thickness of the girder-deck connection in 20~30 mm and the range of the thickness of the gap between pre-cast panels in 20~50 mm.

(a) Upper surface of steel girder

(b) Shot blast and putting seal sponge

(c) Deck settlement and mortar filling

Fig. 4-2 Fabrication of specimens

(4) Material properties

Table 4-1 shows material property of the concrete deck, the filler mortar and the steel girder. Rapid strength concrete with design compressive strength 40 N/mm^2 was applied for the pre-cast deck. For the filler mortar, high early strength type pre-packed non-shrinkage mortar was applied. For the steel girder, H-shaped steel from SS400 was applied.

Table 4-1 Material properties of specimens

(a) Concrete deck

Specimen	Specimen 1	Specimen 2
Days	17	24
Comp. Strength	50.7N/mm^2	51.6N/mm^2
Elastic modulus	36.7 kN/mm^2	36.9 kN/mm^2

(b) Filler mortar

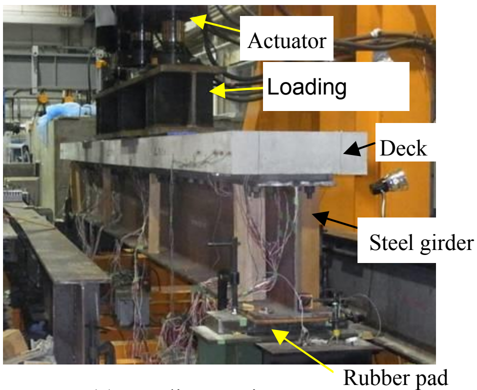
Specimen	Specimen 1	Specimen 2
Days	6	9
J-14 wrought	7.1 sec	7.6 sec
Comp. Strength	47.5 N/mm^2	47.7 N/mm^2
Elastic modulus	23.2 kN/mm^2	23.3 kN/mm^2

(c) Steel girder

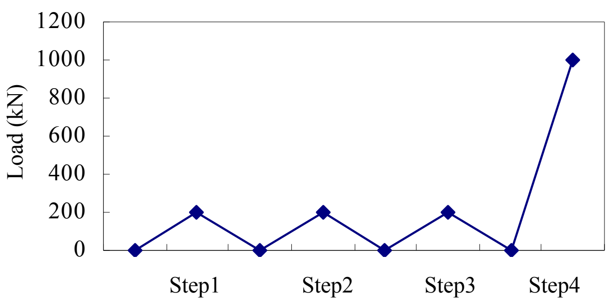
Specimen	Specimen 1, 2
Steel	SS400
Size	H-488×300×11×18
Yield strength	362 N/mm^2
Tensile strength	493 N/mm^2

4.2.2 Loading condition

The loading test was carried out in 4-point bending test. The loading points are shown in **Fig. 4-1**. The load was acted to each loading points through the loading beam as shown in **Fig. 4-3** (a). **Fig. 4-3** (b) shows the loading steps. The load was increased gradually up to 200 kN at Step 1~3, considering train load. In the next step (Step 4), the load was increased up to about 1,000 kN to confirm the limit state of the girder. In addition, wooden stiffeners were installed at the girder end and the loading points for safety during the test. The girder was supported by H-shape steels, through rubber pads between them.



(a) Loading equipments

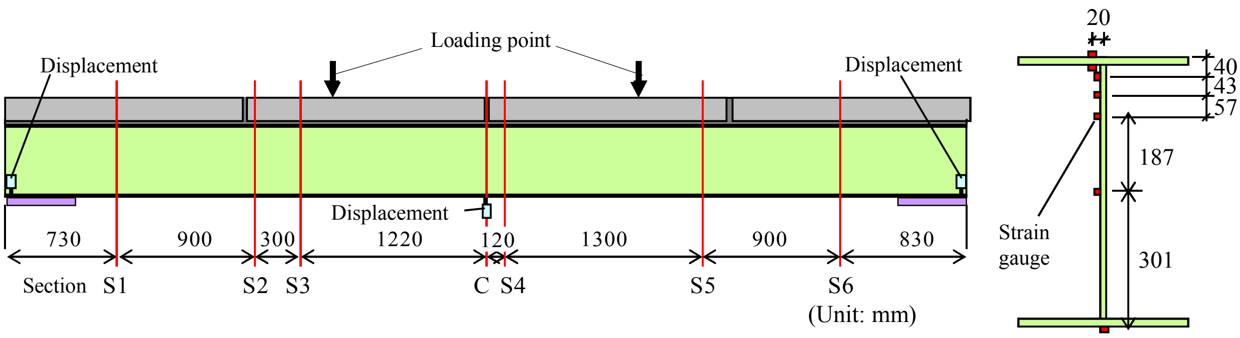


(b) Step and load

Fig. 4-3 Loading condition of the loading tests

4.2.3 Measurement

Fig. 4-4 shows the points of measurement of displacement and strain. In order to obtain stiffness and bending capacity of the girder, displacements were measured at the span center. In order to confirm stress reduction effect by concrete deck, the strain of the steel girder was measured at each point in each section S1~S6.



(a) Side view

(b) Sectional view of steel girder

Fig. 4-4 Measurement location of displacement and strain

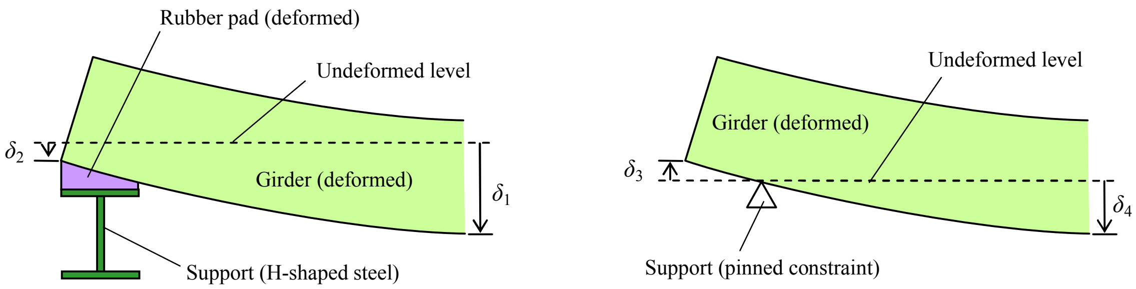
4.3 Performance of deck-installed girder

4.3.1 Behavior of girder

To investigate the behavior of the deck-installed girder under the bending load, load-displacement relationship is discussed. The displacement of the girder is calculated considering the support condition of the specimen.

a) Calculation of measured displacement

As mentioned before, the girder was supported by the H-shape steel through the rubber pads. The deflection, *i.e.* bending displacement of girder, should be calculated considering the deformation of the rubber pads. In the test, the displacement of the girder end was measured as shown in **Fig. 4-4**. The deflection of the girder is calculated using the measured displacement data, as shown in **Fig. 4-5** (a). For the calculated values of the deflection, displacement of the girder was calculated using the finite element analysis shown in **Fig. 4-6**. The analysis code is ABAQUS and the linear elastic solid elements were applied.



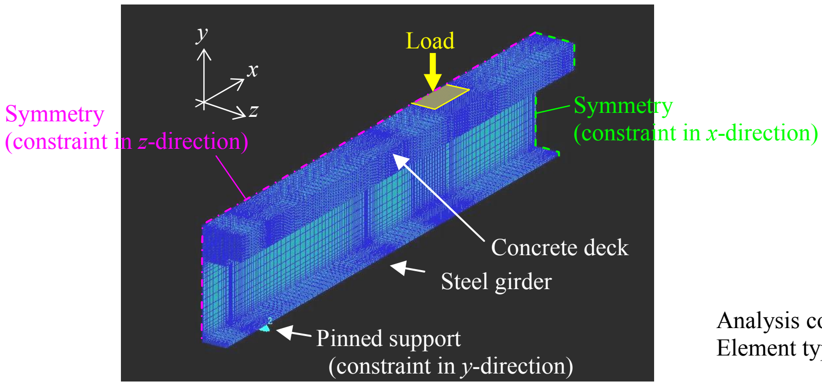
Girder deflection $\delta_{\mathrm{exp}} = \delta_1 - \delta_2$

(a) Deflection in experiment δ_{exp}

Girder deflection $\delta_{\mathrm{cal}} = \delta_3 + \delta_4$

(b) Deflection in analysis δ_{cal}

Fig. 4-5 Calculation of girder deflection



Analysis code: ABAQUS
Element type: Linear elastic solid element

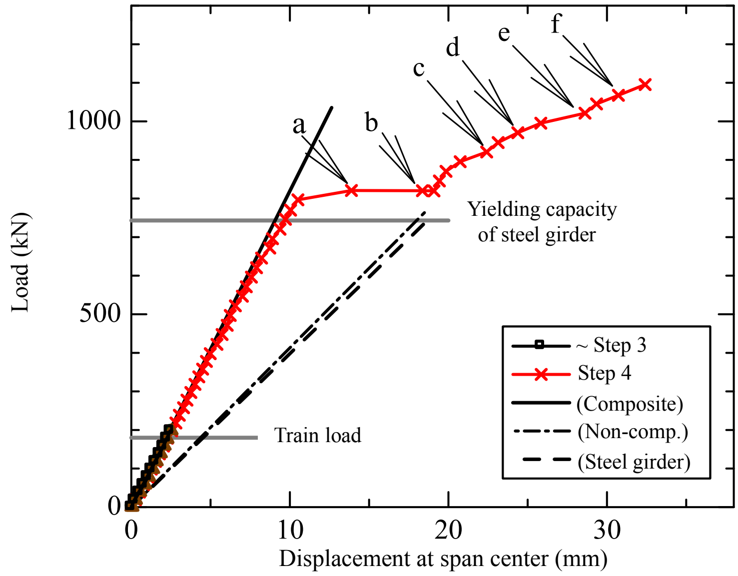
Fig. 4-6 Finite-element model of specimen (quarter model)

b) Load-displacement relationship and behavior of girder

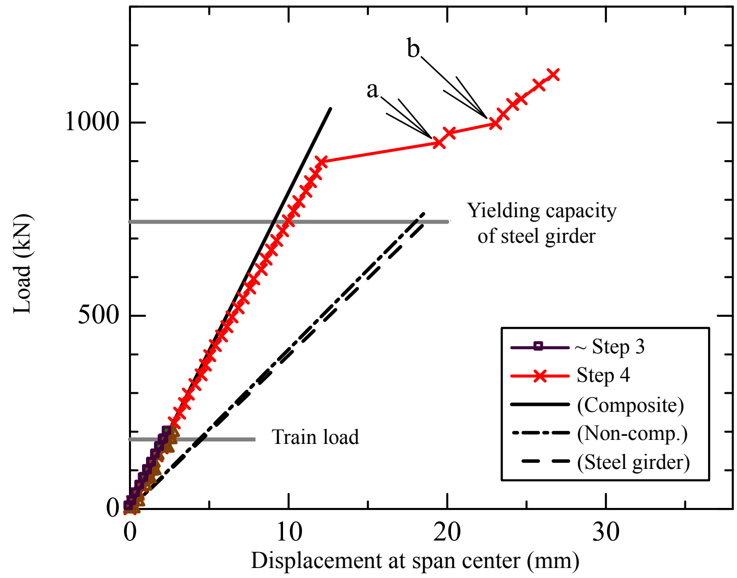
Fig. 4-7 shows load-displacement relationship of the girder. Observed phenomena during the test is described in the figure and explained in **Table 4-2**. In Step 1~3 (within train load), load and displacement shows linear relationship both in up-loading and down-loading stage. In Step4, load and displacement keeps almost linear relationship until following failure loads.

In Specimen 1 (mortar thickness 30 mm), at 821 kN loading, the failure of the girder-deck connection was observed at the right-hand-side girder end. At the time, the deck and the steel girder were separated off and the displacement at the span center suddenly increased (phase a). After that, without increase of load, the girder-deck connection at the left-hand-side girder end also failed and the displacement increased (phase b). After a while, load started to increase again. After the yielding of the lower flange, the slope of the load-displacement relationship started to decrease (phase c~f).

In Specimen 2 (mortar thickness 20 mm), at 948 kN loading, the girder-deck connection failed at the right-hand-side girder end. At the same time, the lower flange of the steel girder started to yield. (phase a). After increasing the load, the girder-deck connection at the left-hand-side girder end at the loading of 998 kN (phase b).



(a) Specimen 1 (mortar thickness 30 mm)



(b) Specimen 2 (mortar thickness 20 mm)

Fig. 4-7 Load-displacement relationship and state of specimens

Table 4-2 State of specimens in each step

(a) Specimen 1 (mortar thickness 30 mm)

-	Load	Situation
a	821 kN	Failure of girder-deck connection at right-hand girder end
b	821 kN	Failure of girder-deck connection at left-hand girder end
c	920 kN	Yielding of lower flange at section S5
d	970 kN	Yielding of lower flange at section S4
e	1,021 kN	Yielding of lower flange at section S3
f	1,045 kN	Yielding of lower flange at section S2

(b) Specimen 2 (mortar thickness 20 mm)

-	Load	Situation
a	948 kN	Failure of girder-deck connection at right-hand girder end Yielding of lower flange at section C
b	998 kN	Failure of girder-deck connection at left-hand girder end

4.3.2 Composite effect by deck installation

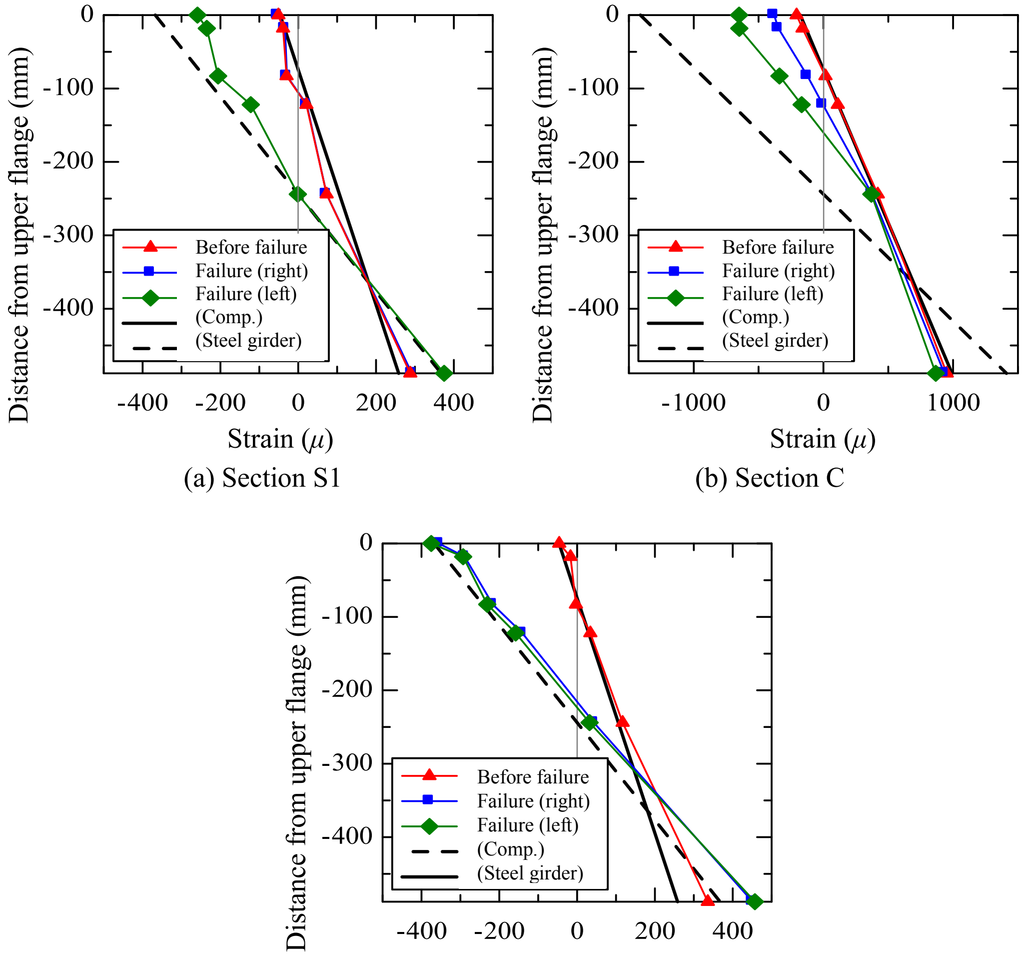
(1) Bending stiffness and capacity

To obtain the bending stiffness of the girder, the load-displacement relationship is shown in **Fig. 4-7**. The straight lines in the figure indicate the calculation results in composite state, non-composite state and only steel girder. Until the girder-deck connections failed, both specimens showed the bending stiffness near the calculation result in composite state. From this result, it was found that the steel girder and the concrete deck were connected each other and acted as a composite girder even the concrete decks were connected by mortar, which has relatively low elastic modulus. The failure load of the girder-deck connection was higher than the bending capacity of the original steel girder, as shown in **Fig. 4-7**.

(2) Strain distribution in steel girder

Fig. 4-8 shows the strain distribution in each section in Specimen 1. In all the sections, the strain distribution showed composite state before the failure of the girder-deck connection (red line).

After the failure of the girder-deck connection at the right-hand-side girder end (blue line, phase a), the strain did not change at Section S1 near the left-hand-side girder end while the strain showed the non-composite state at Section S6 near the right-hand-side girder end. After the failure of the girder-deck connection in the left-hand-side girder end (green line, phase b), the strain moved to non-composite state at Section S1. From these results, it can be said that the specimen kept composite state until the girder-deck connection failed and the state suddenly changed to non-composite state at the time of the failure. In addition, at Section C (span center), change of the strain distribution was comparatively small. This is because the girder-deck connection remained after the failure at the girder ends.



(a) Section S1

(b) Section C

(c) Section S6

Fig. 4-8 Longitudinal Strain distribution in steel girder (Specimen 1)

(3) Influence of mortar thickness

In this test, different thickness of the girder-deck connection and different thickness of the gap between pre-cast panes were set as test parameters. However, there was no difference in workability of the mortar, as mentioned before.

Before the failure of the girder-deck connection, both of the specimens showed almost the same slope of the load-displacement relationship. Although the failure load of the girder-deck connection was different in Specimen 1 and 2, both specimens showed the capacity higher than the yielding capacity of the original steel girder. The shear strength of the girder-deck connection calculated from the section force is 2.22 N/mm^2 in Specimen 1 and 2.27 N/mm^2 in Specimen2. It means that the mortar thickness do not affect the structural performance of the girder.

Furthermore, the failure of the girder-deck connection occurred at almost the same load in left-hand-side and right-hand-side girder end. Form this result, it was found that the thickness of gap between pre-cast panels does not affect the structural performance of the girder.

4.4 Failure mode of girder-deck connection

As shown in the last section, the deck-installed girder had enough structural performance as a railway bridge.　However, the limit state of the girder is the failure of the girder-deck connection. In this section, the failure mode of the girder-deck connection was observed and the mechanism of the shear resistance of the girder-deck connection was discussed. Considering the shear strength of the girder-deck connection, the applicability to actual bridges was discussed.

4.4.1　Fracture point in girder-deck connection

The red lines in **Fig. 4-9** indicate the area of failure.　These failure occurred at the phase a~b shown in **Fig. 4-7** and **Table 4-2**.　In both girder ends in both of the specimens, the failure was observed in the following three places:

Mode i): Debonding of steel-mortar interface at the girder end

Mode ii): Failure of filler mortar layer in skewed direction at distantly-positioned points from the girder end

Mode iii): Debonding of mortar-concrete interface at the area at inner side from the location of failure ii)

Fig. 4-10 shows the example of failure at the right-hand-side girder end in Specimen 1.　These failure modes i)~iii) seemed to occur at the same time.　Furthermore, these failed surfaces of i)~iii) were continuous each other.　From these results, it is considered that a failure, occurred at one point, propagated and lead to the other failure modes.

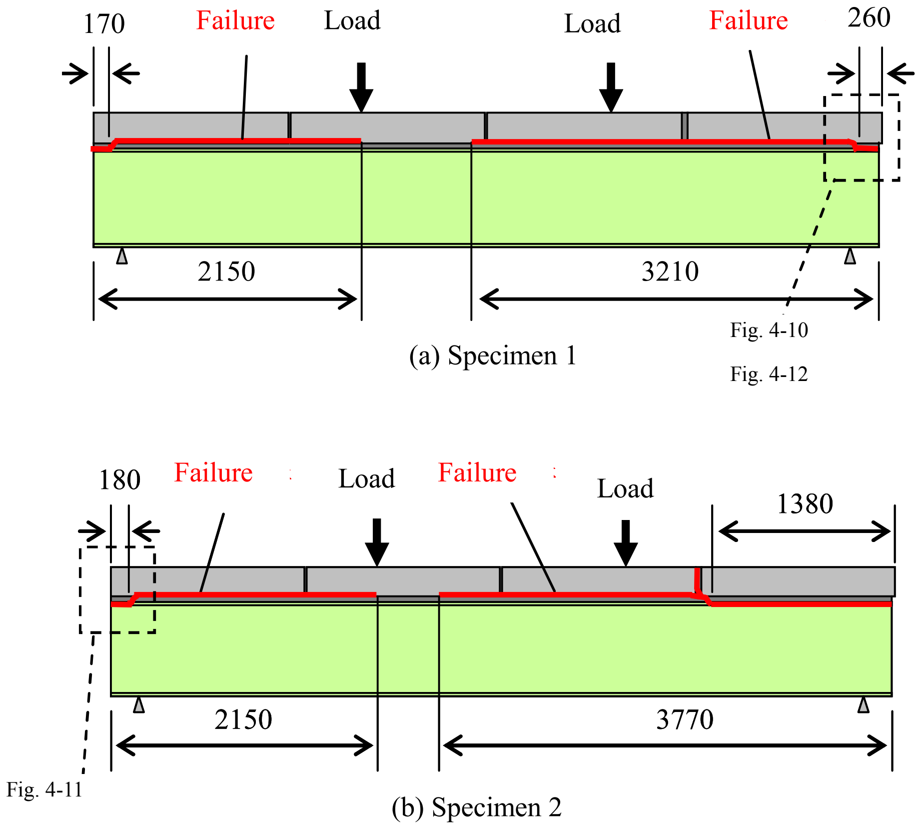


Fig. 4-9 Failure of girder-deck connection

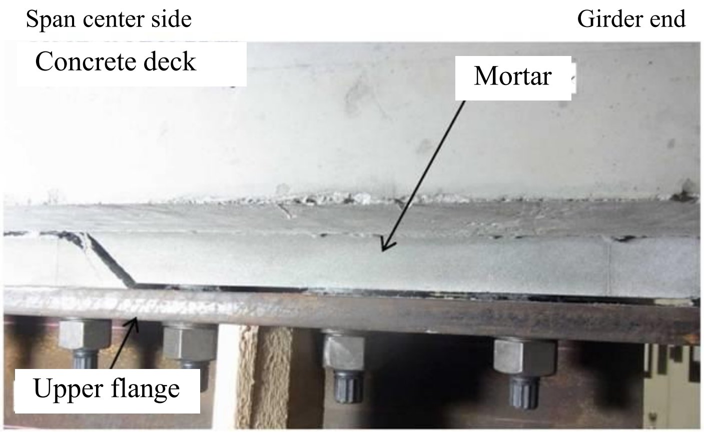


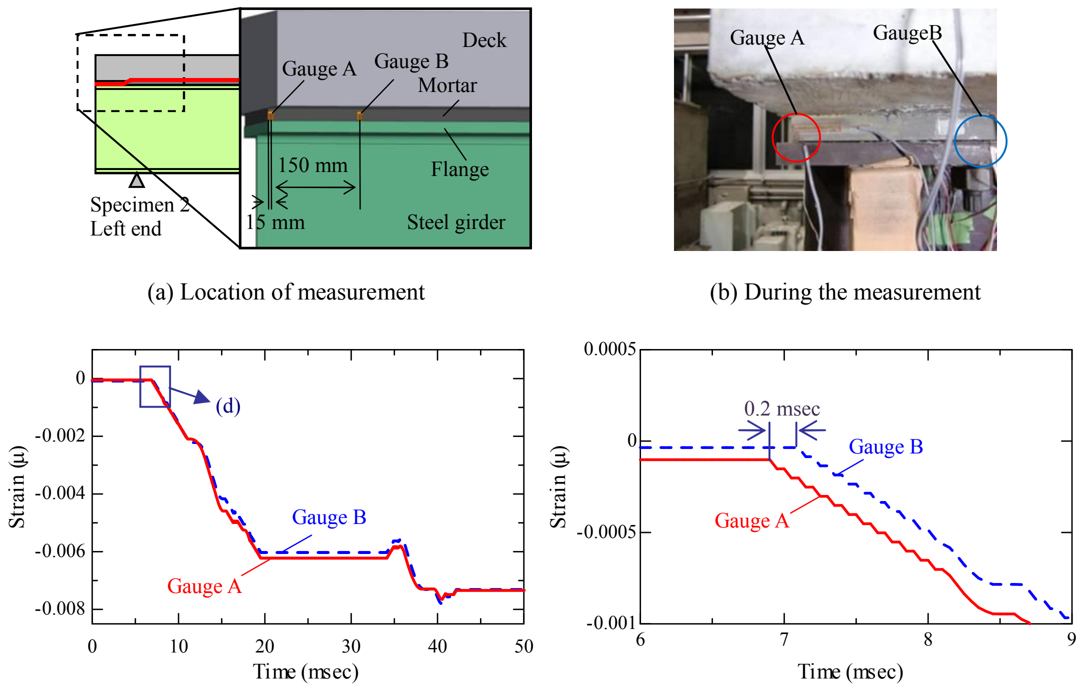
Fig. 4-10 Girder-deck connection after failure (Specimen 1, right end of girder)

4.4.2 Direction of failure propagation

Focusing on above mentioned failure modes i)~iii), the initiation point of the failure was estimated by a dynamic strain measurement in Specimen 2.

Fig. 4-11 (a) shows the left-hand-side girder end of Specimen 2. Dynamic strain at the steel-mortar interface was measured at the time of the failure of the girder-deck connection. The measurement points were set at the point 15 mm from the girder end (Gauge A) and the point 165 mm from the girder end (Gauge B). The strain gauges were placed between the steel and mortar, as shown in **Fig. 4-11** (b). The interval of the measurement was set to 0.04 msec.

Fig. 4-11 (c) shows the result of the measurement. The strain showed a change at the time of the failure. The strain change at Gauge A occurred 0.2 msec earlier than that of Gauge B. From this result, it is considered that the initiation of the failure occurred from the girder end and propagated in the direction of the span center. Thus, among the failure modes, Mode i) occurred prior to Mode ii). Although Mode iii) was not measured directly, it is thought to occur after Mode i) and ii).



(a) Location of measurement

(b) During the measurement

(c) Result of measurement

(d) Result of measurement (Enlarged)

Fig. 4-11 Measurement of steel-mortar interface (Specimen2, left end of girder)

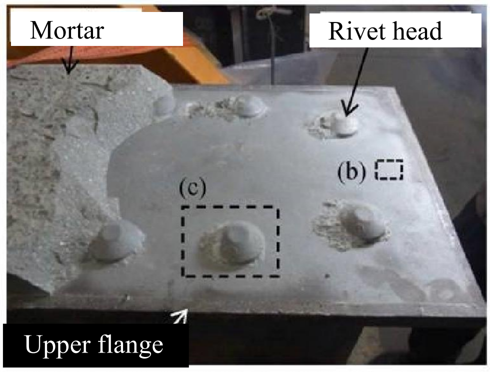
4.4.3 Failure mode of steel-mortar interface

From the last subsection, it is considered that the failure of the steel-mortar interface occurred firstly. To clarify the failure mode of the steel-mortar interface, the condition of the surface of the steel girder was observed after the loading test. **Fig. 4-12** (a) shows the example of the surface of the steel girder at the place shown in **Fig. 4-10**. The surface of the steel girder shows that almost all area of the failure is debonded out of the steel surface, except for areas around the rivet heads. From the figure, it can also be seen that the transition point from Mode i) to Mode ii) is at the location of rivet heads.

Fig. 4-12 (b) shows the zoomed surface. At the surface of the steel girder, a lot of scratching can be seen. Furthermore, as shown in **Fig. 4-12** (c), the mortar remains on the steel surface around the rivet heads in the side of span center. This is because the mortar around the rivet heads resists against the shear force in compressive state. By these results, it can be said that the failure of the steel-mortar interface occurred in the situation that the shear mode is dominant. At the time, as shown in **Fig. 4-13** (a), it is considered that the bonding of the interface and the compressive bearing by the rivet heads resist against the shear force at the same time.

From these results, it is considered that the failure occurred and propagated as shown in **Fig. 4-13** (b). The failure initiated from the debonding of steel-mortar interface at the girder end. The failure then propagated into the direction of span center. After that, at the place of rivet head, mortar layer started to fail and the failure propagated further.

In addition, at the place of Mode ii), starting point of mortar failure is different in the area around the rivet heads and area there is no rivet heads, as shown in **Fig. 4-12**(a). This is thought because the shear resistance was higher near the rivet heads. At the other girder ends, the same thing can be said.

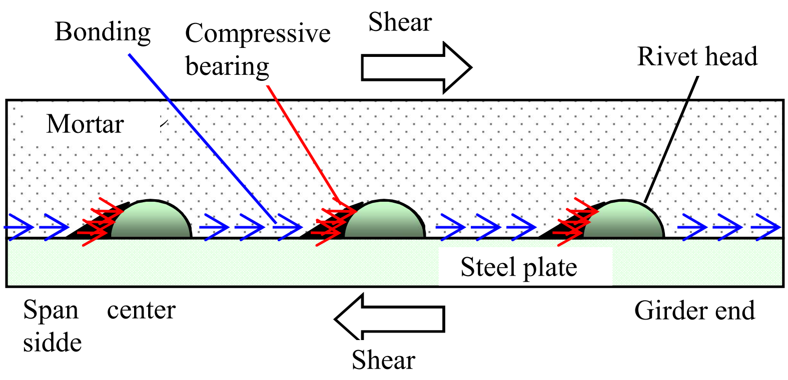


(a) Specimen 1, right end

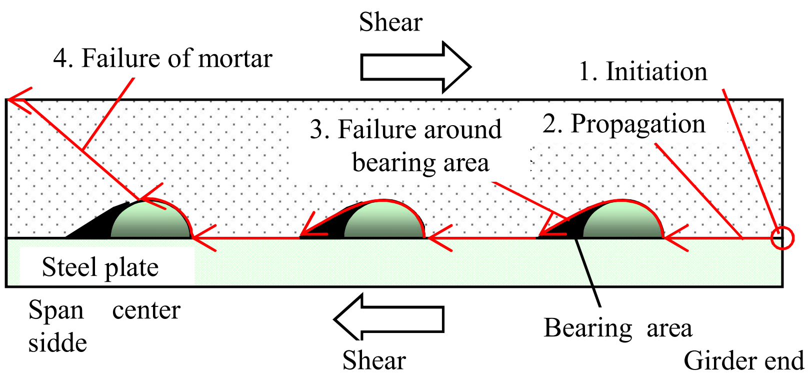
(b) Scratching on steel surface

(c) Remaining mortar around rivet head

Fig. 4-12 Upper surface of steel girder after failure



(a) Shear resistance



(b) Failure and propagation

Fig. 4-13 Presumed of shear resistance and failure at steel-mortar interface

4.4.4 Shear strength of girder-deck connection and applicability on actual structure

As mentioned above, it is considered that the failure occurred at the steel-mortar interface in shear mode. Here, the shear strength of the steel-mortar interface was evaluated using the test results. Considering the results, the application to actual structures was discussed.

(1) Evaluation of shear strength of steel-mortar interface

The shear stress acting on the girder-deck connection can be calculated as:

$$\tau = \frac{V \cdot G_c}{I_v \cdot b} \quad \cdots\cdots (1)$$

Here,

τ: Shear stress the on girder-deck connection (N/mm^2).

V: Sectional shear force (N).

G_c: Moment of area of the concrete deck about the neutral axis of the composite girder (mm^3).

$$G_c = \frac{A_c}{n} \cdot y_c$$

n: Ratio of elastic modulus of steel and concrete.

$n = E_s / E_c$

E_s: Elastic modulus of steel.

E_c: Elastic modulus of concrete.

A_c: Cross-sectional area of the concrete deck (mm^2).

y_c: Distance between the centroid of the concrete deck and the neutral axis of the composite girder (mm).

I_v: Moment of inertia of the composite girder (mm^4).

b: Width of the contact area between the upper flange and the mortar (mm).

Substituting the sectional shear force at the failure of the girder-deck connection into V in the Eq. (1), the shear strength of the steel-mortar interface can be obtained. **Table 4-3** shows the calculation of the shear strength. The obtained strength is much higher than that in the previous study[43)].

Table 4-3 Calculation of shear strength of girder-deck connection

Specimen	Specimen 3	Specimen 2	(Unit)
Sectional shear force V at the failure	411	449	kN
Distance of centroid y_c	179.5	174.3	mm
Converted cross sectional area of concrete deck A_c / n	1.38×10^4	1.38×10^4	mm^2
Moment of area of concrete deck G_c	2.67×10^6	2.41×10^6	mm^3
Moment of inertia of composite girder I_c	1.64×10^9	1.59×10^9	mm^4
Width of contact area b	260	260	Mm
Shear stress at steel-mortar interface τ **(Shear strength)**	2.57	2.62	N/mm^2

(2) Application to actual structures

To discuss the applicability of the proposed method, a trial design calculation was carried out focusing on standard steel girder, which was designed in 1930.

Table 4-4 shows the result of the trial calculation. When the "E-17 train load" is applied on the girder, the calculated shear stress at the girder-deck connection is much smaller than the shear strength obtained in the loading test. For general design of composite girder, the safety factor for shear capacity is about 3.0[38)]. Considering the safety factor of the shear stress shown in

Table 4-4, it can be said that the girder-deck connection has enough shear strength under train loads. Furthermore, the shear stress at the bending capacity of the deck-installed girders is smaller than the shear strength of the connection. Thus, the stress reduction of the girder shown in 2.3.1 and the load-carrying capacity increase shown in 2.3.3 can be achieved by using the proposed installation method.

Table 4-4 Calculation of shear stress in standard-designed deck-girder bridges

Bridge Span (m)	9.8	19.2	31.5	40.0
Sectional shear force by train load E-17* (kN)	224	359	554	653
Shear stress of connection by train load (N/mm^2)	0.37	0.37	0.43	0.29
Shear strength of connection (N/mm^2)	2.57	2.57	2.57	2.57
Safety factor (Shear strength / Shear stress) under train load	6.87	7.02	6.04	8.80
Bending moment by train load E-17* (kN・m)	429	1,547	3,876	5,907
Bending capacity (kN・m)	1,825	5,172	8,096	12,727
Shear stress of connection at bending capacity (N/mm^2)	1.59	1.22	0.89	0.62

* Live load was calculated as E-17 train load (**Fig. 2-3**).

4.5 Summary

In this chapter, loading tests of the deck-installed girder were carried out in order to evaluate the structural performance of the deck-installed girder and to observe the failure mode and the strength of the connection. From the results, the applicability of the proposed method to actual structures was discussed focusing on shear strength of the girder-deck connection. Through these studies, the following results were obtained.

(1) As a result of installing concrete decks, the bending stiffness and the yielding capacity of the girder were increased comparing with steel girders.

(2) Before the failure of the girder-deck connection, stress at the steel girders can be predicted assuming composite state of the girder.

(3) At the girder-deck connection, the failure occurred at the steel-mortar interface at the girder end and propagated into the direction of span centre.

(4) Shear strength of the girder-deck connection were obtained through the loading tests. Applying it into the design calculation of the standard bridges, it was found that the proposed method is applicable to actual railway steel bridges without failure of the girder-deck connection.

Chapter 5. Noise Reduction Effect by Deck Installation

5.1 Introduction

Steel railway bridges tend to generate larger structure-borne noise than other structures, such as reinforced concrete structures[34)]. It is possible to reduce the noise of the steel structures by reducing the vibration of steel members, *e.g.*, installing rubber vibration dumpers on the web[44)]. However, it is also expected to reduce the noise by preventing the on-girder vibration from propagating into the steel members.

By the performance improvement method proposed in this research, the stress of the steel girder can be reduced and the load-carrying capacity can be increased. In addition, the method is expected to reduce the structure-borne noise under the train passage. However, the effect and the mechanism of noise reduction by concrete deck have not been clarified.

In the previous study, Hansaka measured and compared the noise of a steel girder bridge and a composite girder bridge under train passage. As a result, it was found that the noise of the composite girder bridge is smaller than that of the steel girder bridge[35)]. However, the conditions such as the size of bridge members, the existence of noise barriers, the velocity of trains, *etc.* were different between the compared bridges.

To figure out the difference of the response of the girder under the same condition, impact hammer test is generally more suitable. Lin, *et al.* carried out impact hammer tests of a small-span steel girder and the girder after casting concrete around it. Consequently, it was found that the vibration and the noise can be reduced by casting concrete[45)]. However, the cause of the noise reduction was still unclear. To estimate the noise reduction effect more precisely, it is necessary to investigate how the vibration and the noise are reduced by installing the concrete deck.

In this chapter, the impact hammer tests of a steel girder were carried out in the conditions, before and after installing concrete decks, in order to evaluate the effect and the mechanism of reduction of the vibration and the structure-borne noise by the concrete deck.

5.2 Impact hammer tests of deck-installed girder

5.2.1 Aim and scope

Noise of railway structures consists of vehicle noise and structure-borne noise[46)]. As shown in **Fig. 5-1,** the structure-borne noise is caused by vibration of the steel members. The vibration of the steel members is caused by rolling vibration of rail-wheel contacted area (hereinafter called ‘on-girder vibration’) and the on-girder vibration propagates into the steel girders.

This study aims to clarify which effect contributes to the reduction of the vibration and the structure-borne noise. As possible effects of the noise reduction by the concrete deck, the following effects can be considered:

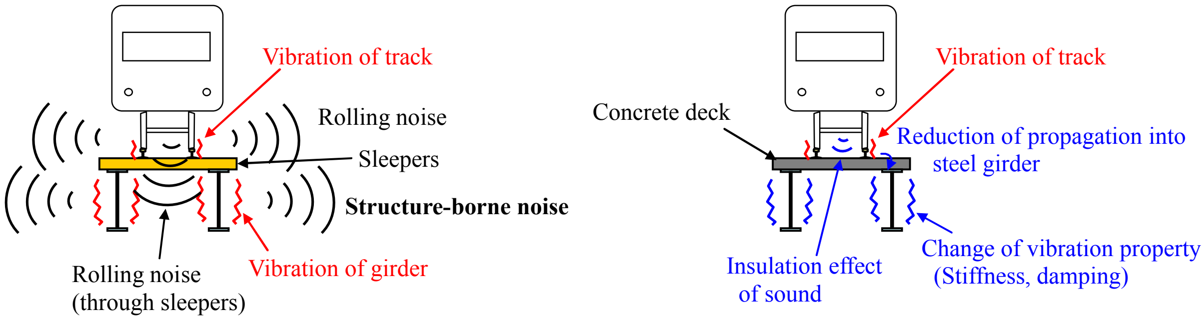
Effect (i): Reduction of propagation of on-girder vibration into the steel girder by concrete deck

Effect (ii): Reduction of vibration of steel member itself by increase of structural stiffness by concrete deck

Effect (iii): Reduction of vibration of steel member itself by increase of damping by concrete deck

Effect (iv): Insulation effect of sound (vehicle noise) by concrete deck

This study focuses on the vibration of structural members and the structure-borne noise. Effect (i) to Effect (iii) are related to the vibration of the girder. On the other hand, Effect (iv) may occur as a side effect. However, Effect (iv) is not considered in this study because it is not directly related to the structure-borne noise.



(a) Before deck installation　　　(b) After deck installation (expected)

Fig. 5-1 Noise and vibration of steel railway bridge

5.2.2 Outline of tests

To clarify the effect of noise reduction, the following two kinds of impact hammer test, “on-girder impact test” and “web impact test”, were carried out.

(1) On-girder impact test by large hammer

In order to grasp the effect of propagation reduction (above mentioned Effect (i)), the on-girder impact test was carried out. In this test, an impact was induced on the girder by a large sledge impact hammer with the hammer mass of 5.5 kg, which has a force sensor with sensitivity of 0.24 mV/N (hereinafter called ‘large hammer’), as shown in **Fig. 5-1**. By this large hammer, impact force was measured in time domain. The web acceleration was measured by an accelerometer with sensitivity of 6.42 pC/(m/s^2) and response frequency of 1~7,000 Hz. The accelerometer was placed at the center of the web. The sound level was measured by the microphone at the distance of 2.5 m from the web. The sound level was measured with the time weighting ‘Fast’, the frequency weighting ‘A’ and the measurement time of 5 sec (hereinafter called ‘Noise level’).

To compare the vibration property and the noise property, the test was carried out in two conditions: “before the deck installation (Case 1)” and “after the deck installation (Case 2)”. In Case 1, the specimen has no concrete deck (only a steel girder) and a wooden piece with 200 mm depth was settled on the steel girder considering a wooden sleeper. The impact was induced on the wooden piece, as shown in **Fig. 5-1**(a). In Case 2, the impact was induced directly on the concrete deck, as shown in **Fig. 5-1**(b). **Fig. 5-3** shows the photograph during Case 2 test.

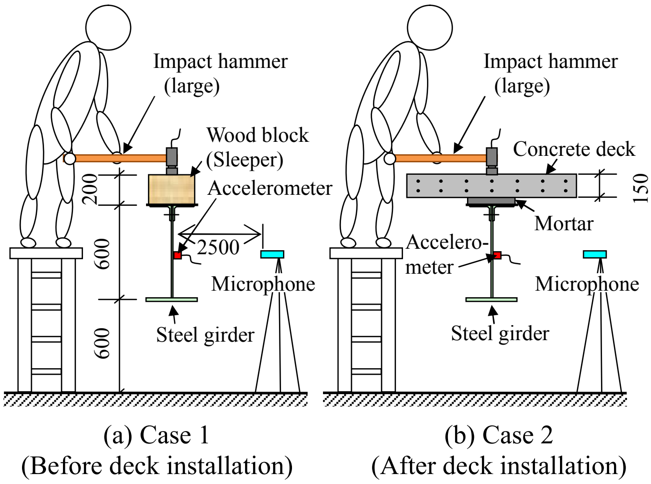


Fig. 5-2 Method of on-girder impact test

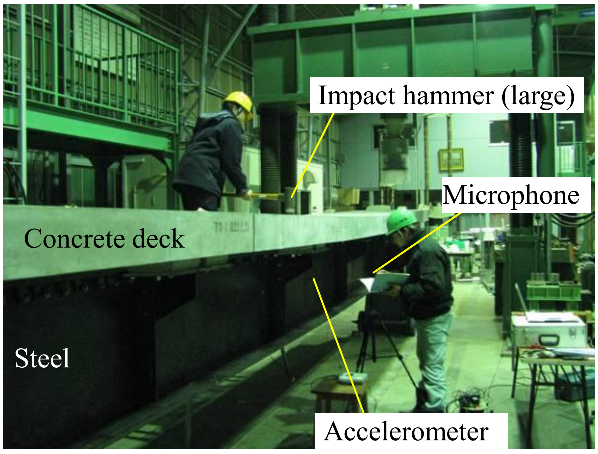


Fig. 5-3 On-girder impact test (Case 2)

(2) Web impact test by small hammer

In order to grasp the effect of the stiffness increase (Effect (ii)) and the damping increase (Effect (iii)), the web impact test by a small hammer was carried out. To compare the response of the web against the impacts under the same condition, the impact was induced on the center of the web directly, as shown in **Fig. 5-4**. As well as the on-girder impact test, the web impact test was carried out in two cases: "before the deck installation (Case 3)" and "after the deck installation (Case 4)". The web acceleration and the sound level were measured during the test in the same location as the on-girder impact test. **Fig. 5-5** shows the photograph during the test (Case 4). For the web impact test, an impulse hammer with hammer mass of 0.32 kg, which has a force sensor with the sensitivity of 0.23 mV/N (hereinafter called 'small hammer') was applied.

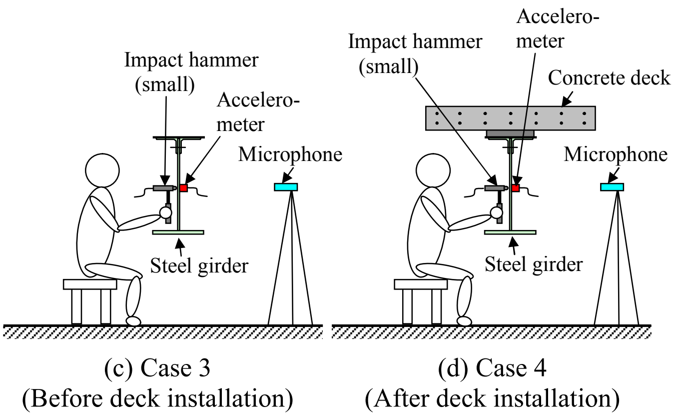


Fig. 5-4 Method of web impact test

Fig. 5-5 Web impact test (Case 4)

(3)　Test cases

Table 5-1 shows the test cases.　As mentioned above, Case 1 and 2 focus on the Effect (i) (reduction of propagation into steel girder).　On the other hand, Case 3 and 4 focus on the Effect (ii) (stiffness change) and the Effect (iii) (damping change).　In each case, inducing impact and measurement were repeated for 5 times and the results were averaged as shown in 5.2.3**(1)**.

Table 5-1 Test case of impact hammer test

Case	Girder	Inducing point	Hammer	Focus
Case 1	Before deck installation	On-girder (Sleeper)	Large hammer	Effect (i)
Case 2	After deck installation	On-girder (Deck)	Large hammer	
Case 3	Before deck installation	Web	Small hammer	Effect (ii) Effect (iii)
Case 4	After deck installation	Web	Small hammer	

(4)　Specimen

Fig. 5-6 shows the test specimen.　The specimen consists of a steel girder with 7.6 m span and five pre-cast concrete decks with filler mortar between the deck and the girder.　Simplifying the relationship between on-girder vibration and girder vibration, a one-girder specimen was applied to this test.

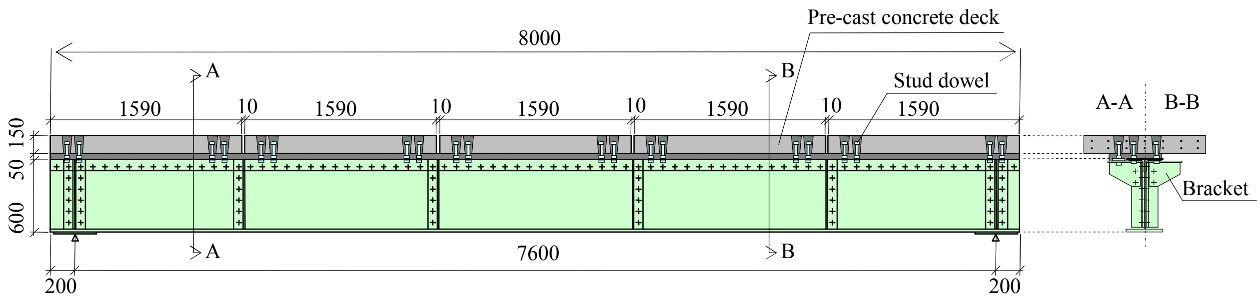


Fig. 5-6 Test specimen for impact hammer test

5.2.3 Test results

(1) Evaluation method of measured acceleration and noise data

The results of the tests are the web acceleration and the noise level near the web. These results were evaluated in a form of response function. In addition, the impact and the measurement were carried out for 5 times in each case and the response function was evaluated in a power-averaged state. The response function and the power-average can be calculated by following method.

From the measured web acceleration and the impact force in time domain, the power spectra were calculated. Focusing on structure-borne noise, the spectra were obtained from 20 to 2000 Hz, known as a range of general structure-borne noise. The power spectrum of the response function A_i of the web acceleration was calculated by Eq. (1).

$$A_i^2(\omega) = P_{ai}(\omega) / P_{fi}(\omega) \quad (i=1, 2, \ldots 5) \cdots\cdots (1)$$

Here, $P_{ai}(\omega)$: Power spectrum of web acceleration (m^2/s^4).

$P_{fi}(\omega)$: Power spectrum of impact force (N^2).

Then, the measured results were power-averaged in order to remove noise.

$$A_{ave}^2(\omega) = \{A_1^2(\omega)+A_2^2(\omega)+\ldots+A_n^2(\omega)\}/n \cdots\cdots (2)$$

Here, n: repeated number of each case (n=5).

Using the power-averaged value, the accelerance level of each case was calculated.

$$L_a=10log_{10}(A_{ave}^2 / A_0^2) \cdots\cdots (3)$$

Here, $A_{ave}(\omega)$: Power average of accelerance ($m/s^2/N$).

A_0: Reference value of accelerance ($=2.0\times10^{-5}$ $m/s^2/N$).

In a similar way, the response function of the noise level was calculated. The power spectrum of the response function Ni of the sound pressure was calculated as:

$$N_i^2(\omega) = P_{ni}(\omega) / P_{fi}(\omega) \quad (i=1, 2, \ldots 5) \cdots\cdots (4)$$

Here, $P_{ai}(\omega)$: Power spectrum of sound pressure (A-weighted) (Pa^2).

$P_{fi}(\omega)$: Power spectrum of impact force (N^2).

Measured results were power-averaged to remove noise.

$$N_{ave}^2(\omega) = (N_1^2(\omega)+N_2^2(\omega)+\ldots+N_n^2(\omega))/n \cdots\cdots (5)$$

Then, the noise level response function was calculated.

$$L_n=10log_{10}(N_{ave}^2 / N_0^2) \cdots\cdots (6)$$

Here, $N_{ave}(\omega)$: Power average of noise response function (Pa/N)

N_0: Reference value of noise response function (=1.0 Pa/N)

In the web impact tests (Case 3 and 4), the damping property of the web acceleration was also calculated. Firstly, logarithmic decrement δ was calculated as:

$$\delta = \frac{1}{n}\ln\frac{a_1}{a_{n+1}} \cdots\cdots (7)$$

Here, a_1: Maximum acceleration in first wave.

a_{n+1}: Maximum acceleration in $(n+1)$th wave

n: number of wave

Using the logarithmic decrement δ, loss factor η was calculated as:

$$\eta = \frac{\delta}{\pi} \quad \cdots\cdots (8)$$

(2) On-girder impact test

a) Impact force and response vibration of steel girder web

Fig. 5-7 shows examples of measured data of the impact force and the web acceleration in time domain and in frequency domain (Fourier power spectra), as standard data among the five-time test, obtained in Case1 (before deck installation, on-girder impact) and Case 2 (after deck installation, on-girder impact). The impact force has a form of pulse wave, as shown in **Fig. 5-7** (a). The web acceleration has a form of damped free vibration, as shown in **Fig. 5-7** (b). The spectrum of the impact force has a flat form, as shown in **Fig. 5-7** (c). The spectrum of the web acceleration has large value between 70 and 400 Hz and has its peak at 175Hz.

Using the obtained spectra, the power-average of the response function of the web acceleration (hereinafter called 'accelerance') was calculated according to Eq. (1) ~ (3). **Fig. 5-8** shows the calculated accelerance in 1/3 octave band. The vertical axis shows accelerance level of Eq. (3). As shown in the figure, the accelerance level of Case 2 (after deck installation, on-girder impact) is reduced by more than 10 dB comparing with Case 1 (before deck installation, on-girder impact) through the most of the frequency between 20 and 2000 Hz, the general range of structure-borne noise. From the obtained response functions, the all path levels were obtained as shown in **Table 3**. The all path levels were calculated as summation of power spectrum values over the frequency domain of 20~2000Hz. In Case 2, the all path level of the accelerance reduced by 12.9 dB and that of the noise response function reduced by 8.4 dB from that in Case 1. Comparing with the measured result of the web acceleration level by installing rubber dampers[48)], the deck installation can reduce the web vibration as much as the rubber dampers. It is thought that this is because the propagation of the on-girder vibration into the steel girder was reduced by the concrete deck (Effect (i)).

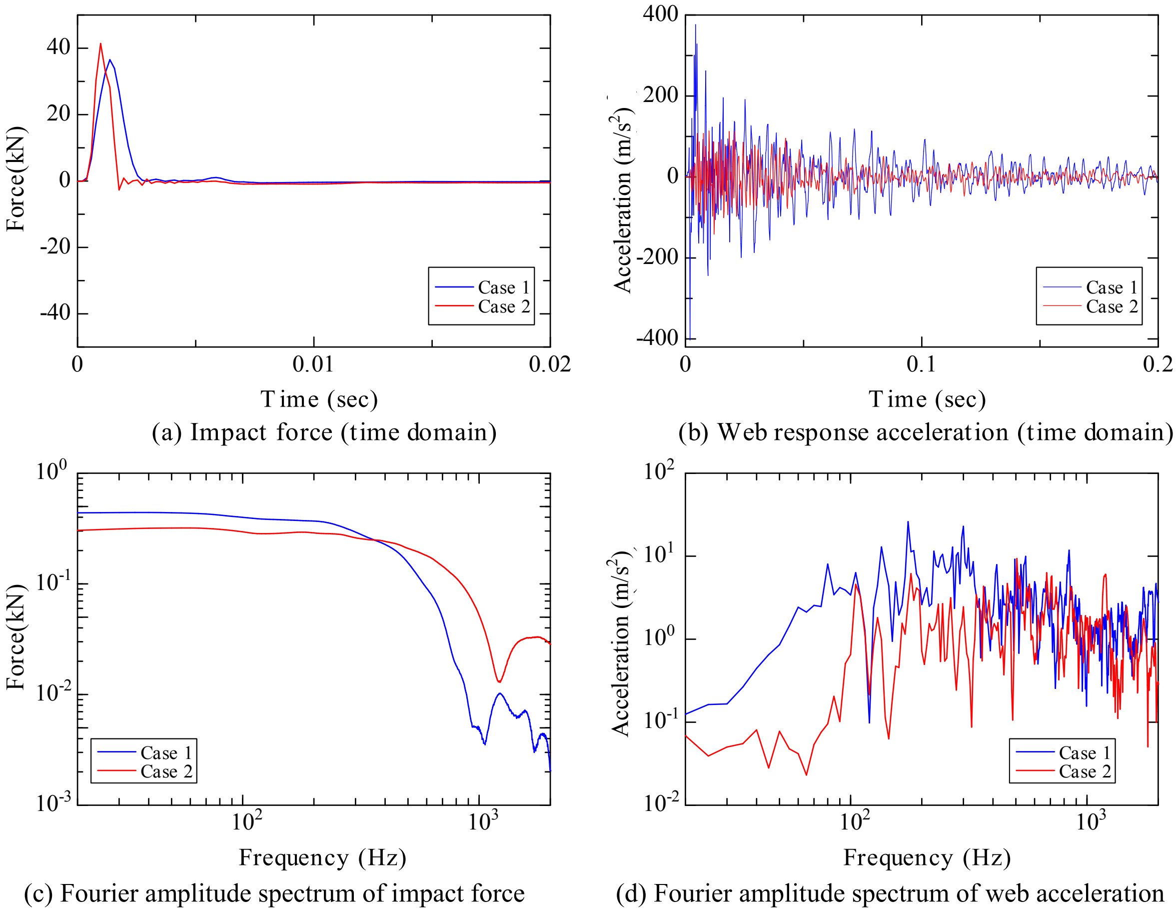


Fig. 5-7 Measured data of impact force and web acceleration

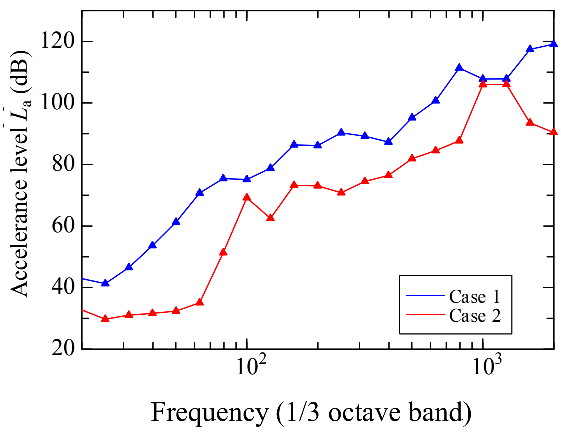


Fig. 5-8 Acceleration response function (power average)

Table 5-2 All path level of accelerance (On-girder impact test)

Case	Inducing point	Girder	Accelerance	Difference of accelerance
Case 1	On-girder	Before deck installation	122.1 dB	
Case 2	On-girder	After deck installation	109.2 dB	-12.9 dB

(All path calculation domain: 20 ~ 2000 Hz)

b) Noise response property

Fig. 5-9 shows examples of measured data of noise level at the 2.5 m distance from the web in frequency domain and power-averaged noise response function, obtained in Case1 (before deck installation, on-girder impact) and Case2 (after deck installation, on-girder impact). The spectrum shows large value between 100 and 1000 Hz. **Fig. 5-9** (b) shows the level of noise response function, calculated according to Eq. (4)~(6). The noise response function is reduced in the range from 200 Hz but it is not reduced in the range from 20 to 200 Hz. As shown in **Table 5-3**, the all path level over 20~2000Hz can be reduced by 8.4 dB.

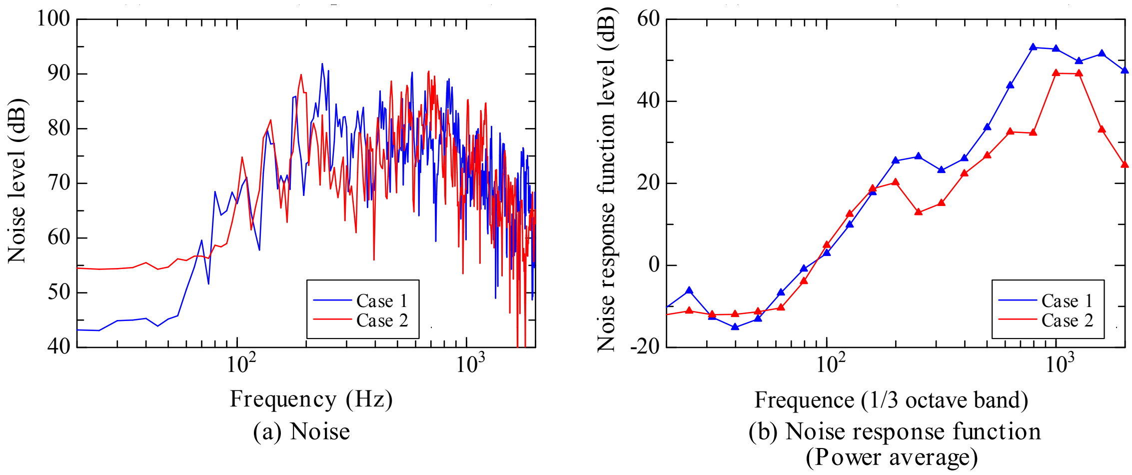


Fig. 5-9 Noise level and noise response function

Table 5-3 All path level of noise response function (On-girder impact test)

Case	Inducing point	Girder	Noise Res. Func.	Difference of N. Res. Func.
Case 1	On-girder	Before deck installation	58.5 dB	
Case 2	On-girder	After deck installation	50.1 dB	-8.4 dB

(All path calculation domain: 20 ~ 2000 Hz)

(3) Web impact test

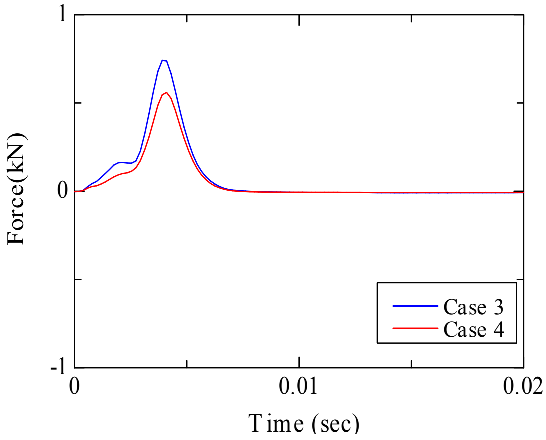
a) Impact force and response vibration of steel girder web

Fig. 5-10 shows examples of measured impact force and web acceleration in Case3 (before deck installation, web impact) and Case4 (after deck installation, web impact). As shown in **Fig. 5-10** (d), the spectrum of the web acceleration shows a shift of its peak frequencies into higher frequencies. For example, one of the peaks of the spectrum at 80 Hz in Case 3 can be seen at 110 Hz in Case 4. Such frequency change can be seen in the range from 80 to 200 Hz. It is because the stiffness of the upper end of the web was increased by the concrete deck.

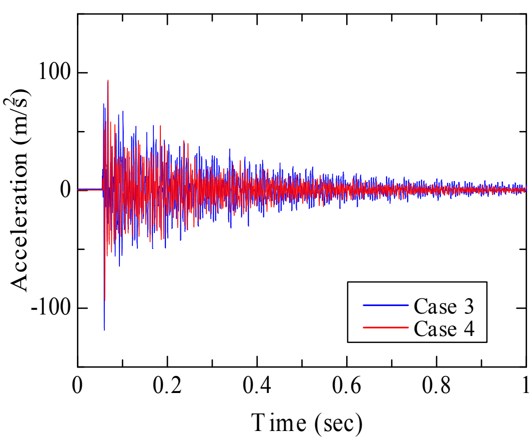
Table 5-4 shows the damping coefficients of the web acceleration, calculated according to Eq.(7)~(8) from the web acceleration in time domain shown in **Fig. 5-10** (b). From the results, loss factor η of the web acceleration can be increased from 0.00427 to 0.00749 by the deck installation. However, comparing with the loss factor η of the rubber damper on the web is 0.05~0.4[47), 48)], the damping effect of the concrete deck is much smaller.

Fig. 5-11 shows the acceleration response function (accelerance) of the web. **Table 5-5** shows the all path level of accelerance over 20 to 2,000 Hz. The accelerance of the web cannot be reduced by installing concrete deck.

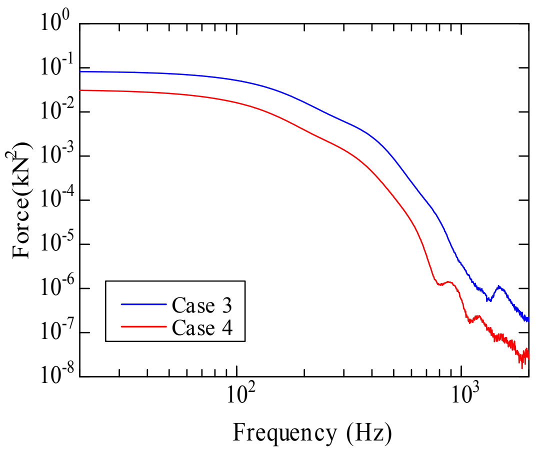
Considering these results, it can be said that the deck installation changes the response frequency of the web vibration and increases the damping property but neither of the effects does not lead to decrease of the web vibration. In other words, the above mentioned Effect (ii) and Effect (iii) are negligibly small.



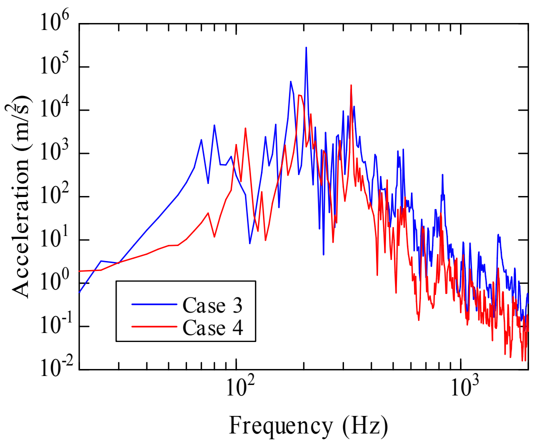
(a) Impact force (time domain)



(b) Web response acceleration (time domain)



(c) Fourier amplitude spectrum of impact force



(d) Fourier amplitude spectrum of web acceleration

Fig. 5-10 Measured data of impact force and web acceleration

Table 5-4 Damping property of web acceleration

Case	Logarithmic decrement δ	Damping ratio ζ	Loss factor η
Case3	0.0134	0.00214	0.00427
Case4	0.0235	0.00374	0.00749

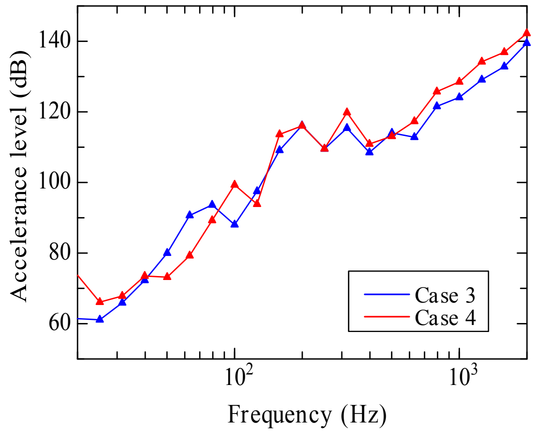


Fig. 5-11 Acceleration response function (power average)

Table 5-5 All path level of accelerance (Web impact test)

Case	Inducing point	Girder	Accelerance	Difference of accelerance
Case 3	Web center	Before deck installation	140.8 dB	+3.4 dB
Case 4	Web center	After deck installation	144.2 dB	

(All path calculation domain: 20 ~ 2000 Hz)

b) Noise response property

Fig. 5-12 shows the noise level near the web and the noise response function of Case 3 and 4. **Table 6** shows the all path level of the noise response function. As shown in **Fig. 5-12** (a), the spectra of the noise level show change of response frequency. However, as shown in **Fig. 5-12** (b), the noise response function was not decreased.

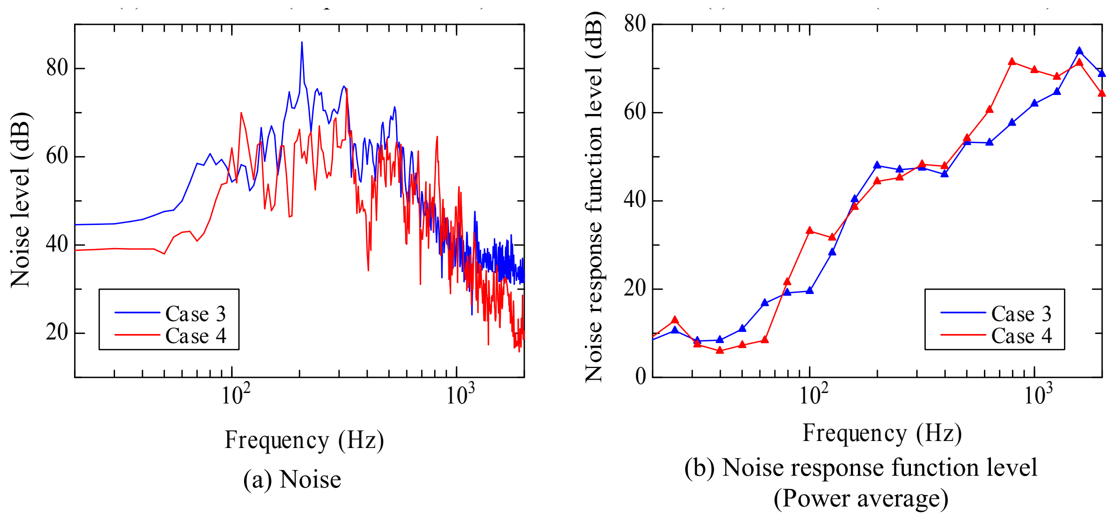


Fig. 5-12 Noise level and noise response function

Table 5-6 All path level of noise response function (Web impact test)

Case	Inducing point	Girder	Noise Res. Func.	Difference of N. Res. Func.
Case 3	Web center	Before deck installation	75.8 dB	+0.9 dB
Case 4	Web center	After deck installation	76.7 dB	

(All path calculation domain: 20 ~ 2000 Hz)

5.3 Summary

In this chapter, the impact hammer tests of the steel girder and the concrete-deck-installed girder were carried out, considering the conditions 'before' and 'after' the concrete deck installation, in order to clarify the reduction effect of vibration of structure and structure-borne noise. Through the tests, the vibration property of the web and the noise property near the web were obtained. Findings in this chapter are as follows:

(1) From the on-girder impact test, it was clarified that the installation of concrete deck is effective to mitigate the propagation of on-girder vibration into the steel girder (Effect(i)). Consequently, the structure-borne noise can be reduced. In the range from 20 to 2000 Hz frequency, the all path response acceleration of the web can be reduced by 12.9 dB. This reduction is as much as that of the rubber dampers set on the web in the previous studies.

(2) In the web impact tests focusing on web vibration itself, it was clarified that the deck installation makes change of vibration spectrum through the change of stiffness (Effect (ii)) and the damping coefficient can be increased by the deck installation (Effect (ii)). However, these changes are not significant enough to reduce the vibration of the web itself.

(3) From these results, the deck installation is as effective as the rubber dampers in reducing web vibration and its effect is mainly mitigating the propagation of on-girder vibration into steel girders (Effect(i)). Considering this property, the vibration and the noise would be reduced more if the proposed method is applied in combination with the installation of the rubber dampers on the web.

Chapter 6. Conclusions

In this research, a new method of performance improvement of existing steel railway bridges by installing concrete decks was proposed. Focusing on application of the proposed method to actual structures, the applicability of the method and the performance of the deck-installed girders were investigated. Finally, following conclusions were obtained:

Chapter 2 Applicability of Performance Improvement Method by Installing Concrete Decks

In this chapter, a performance improvement method by installing concrete deck was proposed. For the proposed method, expected effects and influences by installing concrete deck were discussed based on design calculation. Considering application into actual bridges, type of concrete deck and deck installation procedure were discussed. In order to confirm deck installation time and accuracy of installation, the test construction of concrete deck was carried out. Focusing on the filler mortar, the property of the fresh mortar and bonding strength of the mortar-concrete interface were tested. Through these studies, following results were obtained.

(1) Based on design calculations, stress at the upper flange can be decreased by 50 ~ 80 % and stress at the lower flange can be reduced by 10 ~ 20 %, by applying the proposed method. Especially at the upper flange, stress can be drastically reduced. The stress reduction effect is relatively large in shorter span bridges.

(2) In consequence of the stress reduction, the remaining fatigue life of the bridge can be elongated. Focusing on the lower flange, the remaining fatigue life can be increased to 1.4 ~ 2.3 times compared to the condition before the deck installation.

(3) In consequence of the stress reduction, the bending capacity of the bridges can be increased by 20 ~ 50 % after installing the concrete decks.

(4) Considering previous measurement result, reduction of noise is expected by installing concrete deck.

(5) The increase of dead load by installing concrete deck affects on bridge supports. Focusing on the stress of support concrete, the proposed method is applicable to the bridges shorter than 40 m span. For the bridges exceeding 40 m span, the actual strength of the support concrete should be obtained to judge the applicability.

(6) Applicable type of concrete deck was discussed based on design calculations. Under the conditions of maximum depth and distance between steel girders, post-tension pre-stressed concrete is applicable.

(7) To enable the installation of the concrete deck in limited work time schedule in night-time work, use of the pre-cast concrete decks was proposed. Considering the upper surface of the steel girders with irregularities, a girder-deck connection method using filler mortar was proposed.

(8) Through the test construction, it was found that the deck installation work, such as setting of

the pre-cast concrete deck and adjustment and mortar filling, takes at most 27 minutes for each pre-cast panel. Considering time constraint at the night work, at least two pre-cast panels can be installed in each night work. The proposed method using pre-cast concrete decks enables easy adjustment by hand and the deck panels can be adjusted in good accuracy with about 1.0 mm error.

(9) Through the bleeding test of the filler mortar, it was found that no bleeding water occurred in the non-shrinkage mortar but white powdery layer can be generated at the top of the mortar if the water ratio is high. Thus, it is recommended to reduce the water ratio.

(10) Through the flowability test of the filler mortar, it was found that the mortar mixed with high temperature water shows high flowability even with low water ratio.

(11) Through the bonding test of the filler mortar, it was found that the mortar-concrete interface shows higher bonding strength if the surface of the concrete is painted by primer. To avoid dry out of the mortar, it is recommended to put primer on the concrete surface.

Chapter 3 Development of Girder-Deck Connection

In this chapter, the structural detail of the girder-deck connection was proposed. To evaluate the capacity of the steel fasteners, the lateral loading test and the uplifting loading test of fasteners were carried out. In order to clarify the condition for the composite effect of the steel girders and the concrete decks, the loading tests of the deck-installed girders were carried out. Through these studies, the following results were obtained.

(1) The steel fasteners have enough capacity for designed vertical and transverse load.

(2) The filler mortar shows resistance against the longitudinal shear force until the slip occurs at the steel-mortar interface. The slip state of the girder-deck connection affects the composite effect of the girder.

(3) The stud dowels are helpful to prevent the slip of the connection if there are enough stud dowels.

(4) If the gaps between pre-cast decks are not filled, the girder does not show composite behaviour even if the slip does not occur at the girder-deck connection.

(5) Considering these facts, it is necessary to prevent slip of the girder-deck connection and ensure the transmission of force between the pre-cast deck panels in order to obtain the composite effect.

Chapter 4　Structural Performance of Deck-installed Girder

In this chapter, loading tests of the deck-installed girder were carried out in order to evaluate the structural performance of the deck-installed girder and to observe the failure mode and the strength of the connection.　From the results, the applicability of the proposed method to actual structures was discussed focusing on shear strength of the girder-deck connection.　Through these studies, the following results were obtained.

(1) As a result of installing concrete decks, the bending stiffness and the yielding capacity of the girder were increased comparing with steel girders.

(2) Before the failure of the girder-deck connection, stress at the steel girders can be predicted assuming composite state of the girder.

(3) At the girder-deck connection, the failure occurred at the steel-mortar interface at the girder end and propagated into the direction of span centre.

(4) Shear strength of the girder-deck connection were obtained through the loading tests. Applying it into the design calculation of the standard bridges, it was found that the proposed method is applicable to actual railway steel bridges without failure of the girder-deck connection.

Chapter 5　Noise Reduction Effect by Deck Installation

In this chapter, the impact hammer tests of the steel girder and the concrete-deck-installed girder were carried out, considering the conditions ‘before’ and ‘after’ the concrete deck installation, in order to clarify the reduction effect of vibration of structure and structure-borne noise.　Through the tests, the vibration property of the web and the noise property near the web were obtained. Findings in this chapter are as follows:

(1) From the on-girder impact tests, it was clarified that the installation of concrete deck is effective to mitigate the propagation of on-girder vibration into the steel girder (Effect(i)). Consequently, the structure-borne noise can be reduced.　In the range from 20 to 2000 Hz frequency, the all path response acceleration of the web can be reduced by 12.9 dB.　This reduction is as much as that of the rubber dampers set on the web in the previous studies.

(2) In the web impact tests focusing on web vibration itself, it was clarified that the deck installation makes change of vibration spectrum through the change of stiffness (Effect (ii)) and the damping coefficient can be increased by the deck installation (Effect (ii)). However, these changes are not significant enough to reduce the vibration of the web itself.

(3) From these results, the deck installation is as effective as the rubber dampers in reducing web vibration and its effect is mainly mitigating the propagation of on-girder vibration into steel girders (Effect(i)).　Considering this property, the vibration and the noise would be reduced more if the proposed method is applied in combination with the installation of the

rubber dampers on the web.

Finally, it was found that the proposed method is effective on increasing load-carrying capacity, reducing stress, increasing bending stiffness, and reducing vibration and noise. It was also found that the proposed method is applicable to the actual bridges in a viewpoint of deck installation work.

In order to promote the application of the proposed method to actual structures and to extend the range of application, conducting the following studies is recommended:

- Test construction to actual steel railway bridges

  In this research, test construction of the pre-cast concrete deck on the steel girder was carried out inside of laboratory. To obtain the actual workability, it is recommended to apply the proposed method into an actual bridge.

- Application of new material for pre-cast decks and filler mortar

  In this research, various tests were carried out considering standard bridges used in Japan. In some cases, the condition would be different from that considered in this research. If the normal concrete pre-cast deck and normal mortar material is not suitable for such cases, it is recommended to apply new materials, such as fiber reinforced concrete and engineered cementitious composite materials[50)] (ECCs), to ensure the performance of the concrete decks and the filler mortar.

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