Design of Joint Sections of a Steel-Concrete Composite Bridge

Toshihiro Iwai[1(✉)], Masaharu Akamine[2], and Nobuhiko Azetsu[2]

[1] P.S. Mitsubishi Construction Co., Ltd., 1-8-30 Tenmabashi, Kita-ku, Osaka 530-6027, Japan
t-iwai@psmic.co.jp

[2] West Nippon Expressway Co., Ltd., 1-13 Iwakura-cho, Ibaraki City, Osaka 567-0871, Japan
{m.akamine.aa,n.azetsu.aa}@w-nexco.co.jp

Abstract. The Arinogawa Bridge designed by the authors is one of the longest steel-concrete composite bridges on the Shin-Meishin Expressway. The dimensions of the bridge are as follows: 364 m for the inbound lane, 363 m for the outbound lane, and 448 m for the ramps. In the Arinogawa Bridge, there are seven joint sections of steel girders and PC girders, and some of these joint sections are located in junctions between the through lane of the bridge and the ramps. The authors considered that these joints would be placed under complex stress, and there was a possibility that the stress might exceed the expected levels. To solve this problem, the authors employed special countermeasures both in the design and the construction process.

The countermeasures related to design consisted of creating a full-bridge model and implementing FEM analysis. In the FEM analysis, a model of the entire bridge was created, and precise calculations were conducted of the complex stress distribution in the event that live loads act on the joint sections in the most adverse manner.

The countermeasures related to construction consisted of tests conducted to confirm that concrete is filled properly into full-scale specimens of joints, because the narrow steel cells placed in the joints must be tightly filled with concrete without compaction.

Meanwhile, since the project owner was delayed in acquisition of land for construction of the bridge, the construction process had to be curtailed significantly. All cast-in-place slabs to be installed on top of the steel girders were replaced with pre-cast slabs (a total of 183 slabs with an overall slab area of 3,560 m^2).

Keywords: A steel-concrete composite bridge · FEM analysis of a full bridge model · Full-scale test specimens of joint · Pre-cast floor slab

1 Introduction

Currently in Japan, projects to improve the expressway network are underway nationwide to provide congestion-free traffic flow and multiple road systems, with the goal of enhanced competitiveness in various industrial fields. A project that is

D.A. Hordijk and M. Luković (eds.), *High Tech Concrete: Where Technology and Engineering Meet*, DOI 10.1007/978-3-319-59471-2_152

representative of this effort is the Shin-Meishin Expressway (Fig. 1), 174 km in total length, planned as an alternative route for the Meishin Expressway where traffic congestion and aging of equipment pose serious problems. Some segments of the new expressway are now at the peak of construction. For this expressway, new bridge techniques are used extensively to provide a better bridge durability with reduced costs. The Arinogawa Bridge designed by the authors is a steel-concrete composite bridge that is a relatively new type.

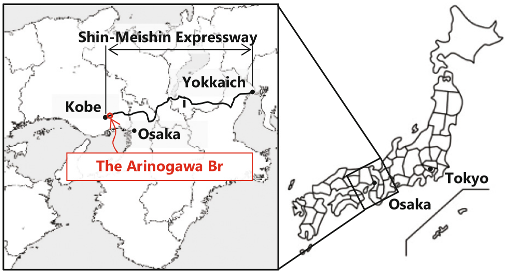

Fig. 1. Location of the Arinogawa Bridge

The Arinogawa Bridge is one of the longest continuous steel-concrete composite bridges on the Shin-Meishin Expressway. It is 1,175 m in total length consisting of 364 m for the inbound lane, 363 m for the outbound lane, and 448 m for the ramps (Figs. 2 and 3). The viaduct portion between spans A1 and P9 is built with PC slab girders and PC box girders, whereas in the segment between P9 and P10 over the river, steel girders are used, since a longer effective span is necessary. For better traveling comfort and more reasonable maintenance, the steel girder and PC girder are joined without an expansion device. There are seven such joints. Since this bridge is located in the vicinity of junctions, both inbound and outbound lanes have complicated alignments with ramp junctions. Some joint sections are located in junctions between a through lane and a ramp. In the joint sections of the junctions, a complicated stress distribution was anticipated to occur under occasional live loads, generating stresses greater than expected. Hence the authors decided to take special measures to ensure satisfactory quality of the joint sections from the viewpoint of both design and construction. Since, in this project, there was a delay on the owner's side in acquisition of

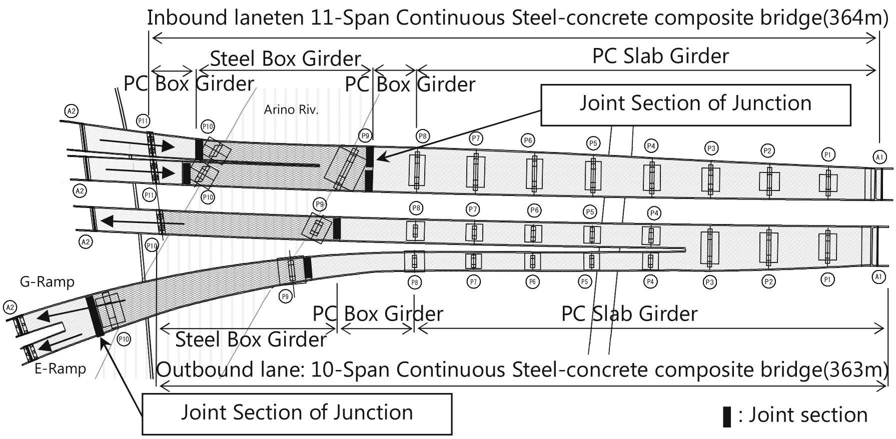

Fig. 2. Plan view of the Arinogawa Bridge

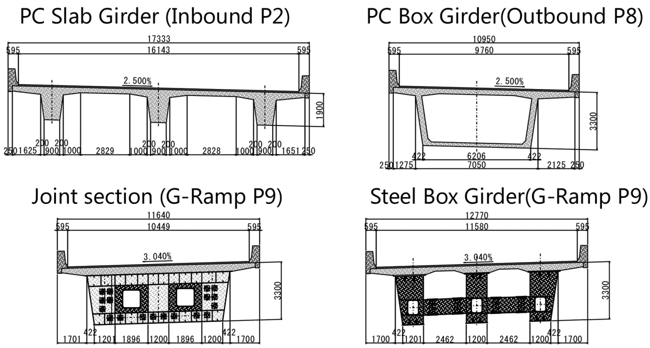

Fig. 3. Cross sectional view of the Arinogawa Bridge

the land for construction, significantly curtailing the construction process and length of time for the project was required. One way to accomplish this reduction was to replace all cast-in-place slabs on top of the steel girders with pre-cast slabs.

2 Design Measures for Ensuring Satisfactory Joint Quality

2.1 Issues in the Joint Design

Figure 4 shows the structure of the joint used in the Arinogawa Bridge, which features steel cells provided with Perfobond Leiste ribs (PBL in the following) and back plates on the flanges of the steel joint blocks. Concrete is continuously placed into the steel cells from the PC girder side. This configuration enables smooth transmission of stresses from the steel girder to the PC girder through the steel cells filled with concrete. External cables installed through some steel cells are used to introduce prestress to keep the PC girder near the junction in compression, free of tensile stress.

The structure of this bridge is complicated, with different materials such as steel members and concrete in the same section of the joints. Another difficulty is that data of composite bridges constructed in the past are relatively limited. It is therefore essential to minutely analyze the joint structure with satisfactory precision. Another point to be

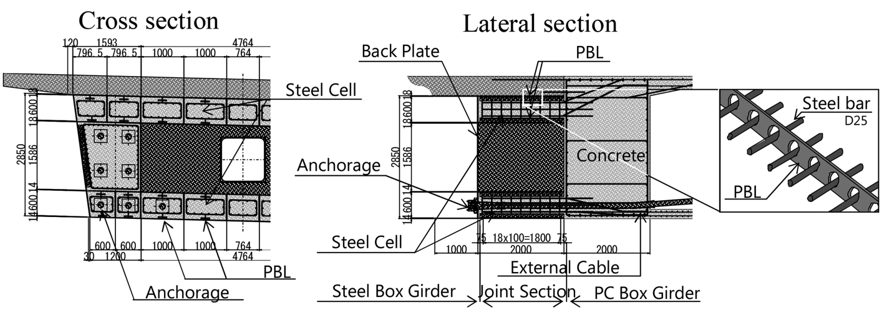

Fig. 4. Structure of joint section

fully considered was the stress distribution in the joints located in the junction of the through lane and ramp bridge. There was fear that the stress distribution at such locations could be more complex than in usual joints under an uneven load when live loads are applied only on one side, either on the through lane or on the ramp, generating stresses exceeding the expected level.

The authors determined that it is extremely important in the design of this bridge to conduct precise analysis simulating actual loads, support conditions and the structural configuration, for ensuring satisfactory quality of the joints located in the junctions.

2.2 FEM Analysis with a Model of the Entire Bridge

In the design of joint sections of the bridge, as usual design practice, some simple calculations and FEM analysis with a partial model of the joint section were conducted (Fig. 5). In addition, as a solution for the issues mentioned above, FEM analysis was performed with a model representing the entire bridge. The analysis dealt with the inbound lane. The PC slab girder was modeled with beam elements, the PC box girder with solid elements, and the steel box girder with shell elements (Fig. 6). To grasp the bridge behavior under live loads, referring to the influence line ordinates in grillage analysis, the locations were determined where the joint is in the most disadvantageous condition, and live loads were applied there.

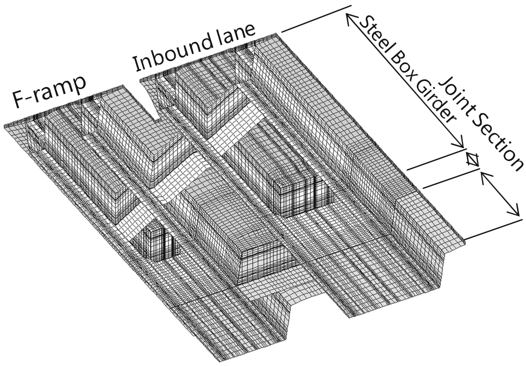

Fig. 5. Partial model of FEM analysis (View from below)

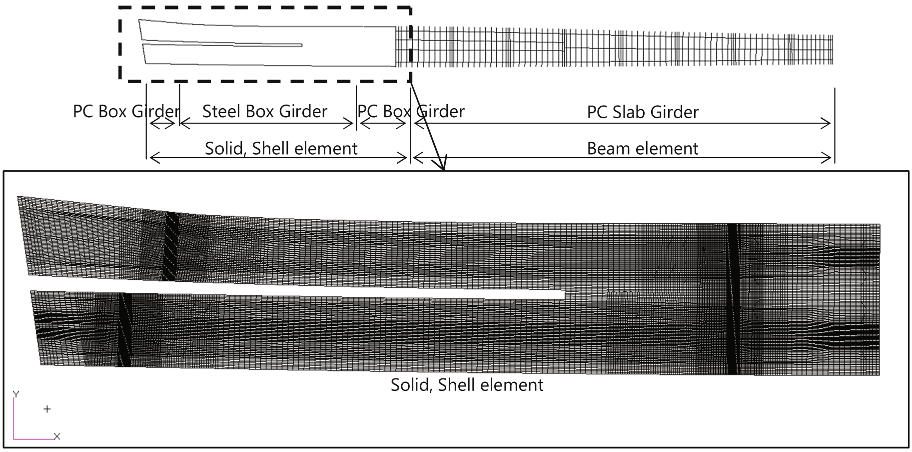

Fig. 6. Full-bridge model of FEM analysis

Figure 7 shows the results of the displacing shear force (axial force in the bridge axis direction) in the PBL given by the FEM analysis with the full model. The graphs also compare the results of the FEM analysis with the partial model of the joint section mentioned above (Fig. 5). The analysis was focused on the joint section on P9 side of the F ramp, assuming such loading conditions that negative bending moment acts there, producing tensile forces working on the upper flange and compressive forces on the lower flange.

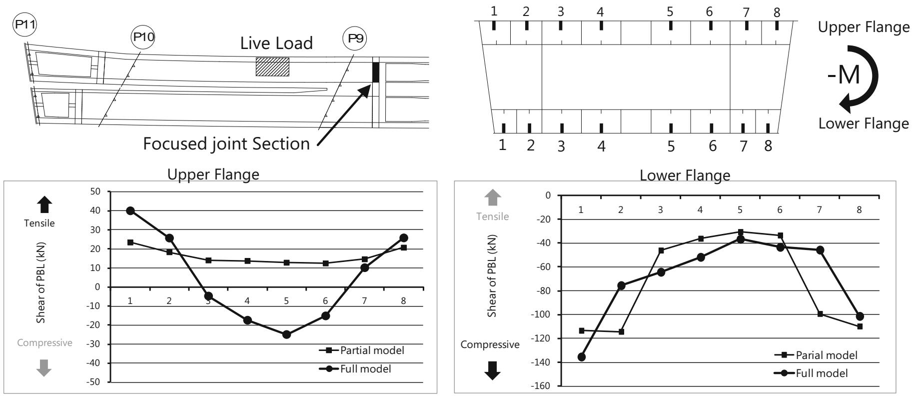

Fig. 7. Axial force of PBL on F-ramp P9 joint section

On the upper flange assumed to be subject to tensile force, the tension in the PBL installed on the outer side of the section tends to be larger in the full model than in the partial model. This result can be explained as follows. In the analysis with the partial model to which sectional forces were uniformly applied to its cut cross section, whereas, in the full model involving actual loading, sectional forces acting on the steel girders were transmitted through the joint block webs to the entire steel cells. In contrast, on the lower flange side where compressive forces are supposed to act, relatively large compressive force was induced in the same distribution both in the partial model and full model. This result can be attributable to the fact that, on the lower flange side, external cables were provided in some steel cells for prestressing, and compressive forces were transmitted mainly by the back plates. In some locations of PBL in the lower flange, a larger displacing shear force occurred in the full model than in the partial model. However, all the values obtained in the analyses with two types of models were under the allowable stress. Consequently, structural safety of the bridge design was validated, confidently confirmed by the full model analysis conducted with actual loads. In addition, with regard to the back plate, different members of the steel girder and PC girder, the stress induced in each section was proved to be under the allowable level.

3 Construction Techniques for Ensuing Satisfactory Quality of the Joint Sections

3.1 Essential Requirements for Construction of Joint Sections

The concrete placement technique selected for the joint section is as follows. Concrete is injected to form the crossbeam of the PC girder, and the concrete moves horizontally to fill the steel cells. The top plates are provided with air holes (Fig. 8). The space in the steel cells is narrow, with many obstacles such as PBL and reinforcing bars, making it difficult to compact the concrete. Therefore, high fluidity concrete with self-compacting capability is used. The steel cells should be completely filled with concrete to enable the joint section to work effectively. However, it is impossible to visually check concrete fill status in the cells which are surrounded by steel plates. For achieving complete concrete filling, the essential requirements were optimizing the properties of the high-fluidity concrete, the size of air holes of the steel cells and staff deployment schedule.

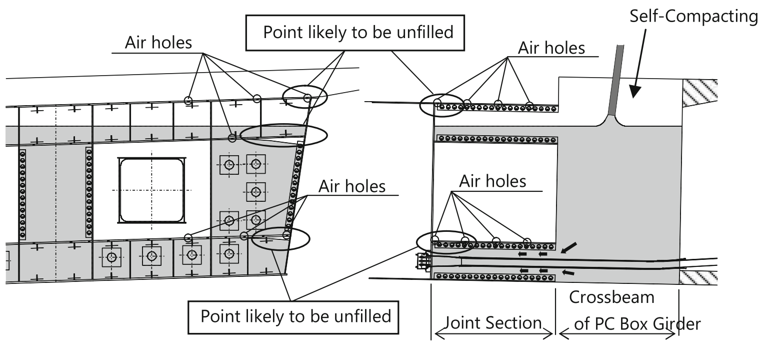

Fig. 8. Point likely to be unfilled of steel cell

3.2 Concrete Fill Test with a Full-Scale Specimen of the Joint Section

In order to satisfying the requirements mentioned above, a concrete fill test was conducted, using full-scale specimens of the joint section (Fig. 9). High fluidity concrete is moved horizontally from the PC girder, as discussed above. The larger the longitudinal or transverse gradient, the smaller the hydraulic head difference between the crossbeam of the PC box girder and steel cell top. Consequently, concrete may fail to entirely reach the upper portion of the steel cells on the upper flange side, leaving unfilled spots. To find a solution for this problem, the longitudinal and transverse gradients and the

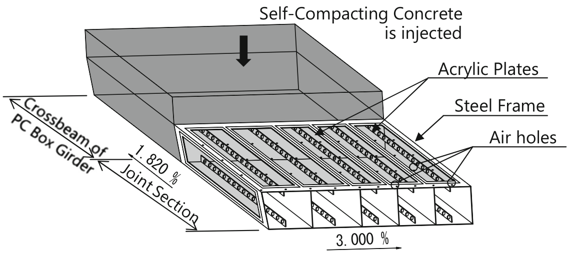

Fig. 9. Full-scale specimen of the joint section (upper flange)

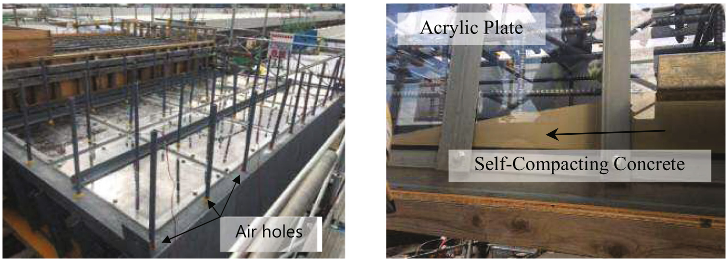

Photo 1. Full-scale specimens of joint section (upper flange)

Table 1. Requirements, items for confirmation, how to confirm and solutions

Requirements	Items for confirmation	How to confirm	Solutions
Complete filling of concrete into the narrow steel cell	Fill status in the steel cell	Check the filling through the acrylic plates during test concrete placement	Review the air hole size
		After test placement, remove the acrylic plates to check air bubble density, etc.	Review the mix proportion of concrete
	Arrangement of concrete fill monitors	Check the function of the monitors during test placement	Review the monitor arrangement
		Check the fill status at the spots provided with monitors after test placement	
	Deployment of personnel and workers	Conduct test placement simulating actual construction, to check the suitability of staff deployment	Review the staff deployment

(*continued*)

Table 1. (*continued*)

Requirements	Items for confirmation	How to confirm	Solutions
	Workable time of high fluidity concrete	Determine the slump flow loss, air volume loss at the test placement	Optimize the schedule of dispatch from the plant
Placement of high fluidity concrete featuring good self-leveling capability to form the upper slab with longitudinal and transverse gradients	Finishiability of the slab top surface	Disassemble the sloping concrete form at a certain time after the end of test placement, and check ability to finish	Set the finishing time

steel cell size were taken into consideration and specimens were fabricated, which represent the most disadvantageous conditions of the joint section. Photo 1 shows the steel cells of the specimen. At the test, verification was conducted on the items shown in Table 1 for the preparation of construction of an actual bridge.

4 Use of Precast PC Slab

At the time of detail design of the bridge, a portion of the land for construction was not yet acquired, leading to delay in the start of construction of the substructure and superstructure. As a consequence, it was required to shorten the period of superstructure construction. For this purpose, precast PC slab was selected instead of cast-in-place slab on the steel girder. Photo 2 shows the erection of the PC slab. Figures 10 and 11 respectively show the plan and section of the segment with the precast PC slabs. Use of the precast PC slabs shortened the construction period by one month. Furthermore, manpower reductions at the site during concurrent work of different trades curbed the risk of delay in construction.

Photo 2. Erection of precast floor slab (Outbound lane)

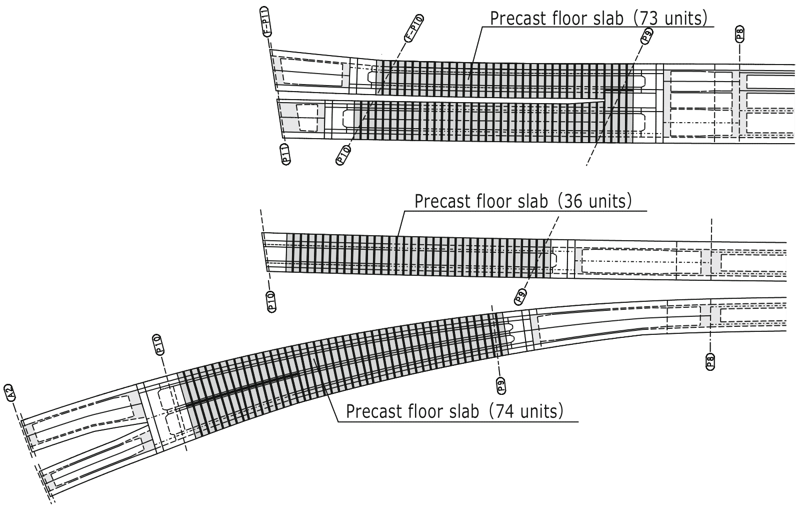

Fig. 10. Plan view of precast floor slabs

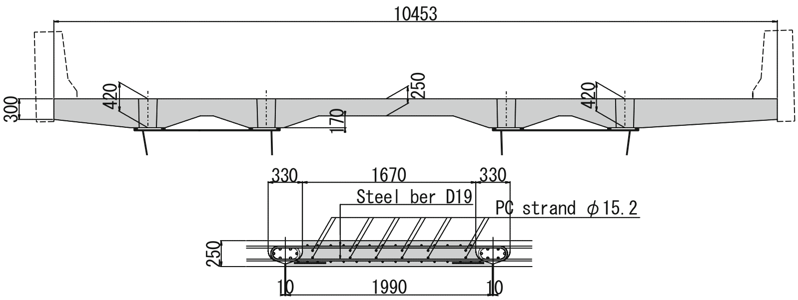

Fig. 11. Section view of precast floor slab (Outbound lane)

5 Conclusions

In this bridge project, the study emphasized satisfactory quality of the joint section between steel girder and PC girder.

The design involved FEM analysis with a model representing the entire bridge in addition to conventional design practices. This enabled validation of the safety of the joint sections with an enhanced accuracy.

During construction, the concrete status in the steel cells will be checked for satisfactory filling.

The construction period was significantly reduced by the effective use of precast floor slabs.

Reference

Expressway Technology Center: Takamatsu Expressway – Report on the Detailed Study of the Design and Construction of Steel-Concrete Composite Bridges (2000)