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Seismic design and safety measures of the Otagirigawa (Phase II line) on the Joshin-Etsu expressway

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ABSTRACT: The Otagirigawa bridge is located between the Myokokogen Interchange and the Nakago Interchange along the Joshin-Etsu expressway. Structural type of this bridge is designed as a steel reversed Lhose bridge which is the same type as the existing Phase I line bridge crossing parallel, because this bridge spans over a steep 50m-deep valley and it is necessary to consider the workability taken into the impact on the existing Phase I roadway, economy, landscape etc. This bridge is designed towards increased seismic resilience utilizing buckling restraint bracing, and increased maintenance related performance. For erection of this bridge, the end spans were built using a lateral transfer single-operation erection method, while the central span was built using diagonal suspension method using temporary cables and traveler cranes. There is the existing Phase I roadway with offset distance of only 165cm to the side of the erection point, and there is a high-voltage transmission lines and underground gas pipes to the west. For this reason, Safety measures for the erection in confined yard was conducted under elaborate examination process, we completed safely this project.

1 INTRODUCTION

The Joshin-Etsu expressway is a national expressway with a total length of about 203 km, which branches off from the Kan-Etsu expressway in Fujioka City, Gunma Prefecture and connects to the Hokuriku expressway in Joetsu City, Niigata Prefecture. 37.5km between Shinanomachi Interchange (hereinafter, this is called IC) and Joetsu Junction (JCT) is tentative when the whole line is opened in 1999 (Opened in 1997 between MyokoKogen IC and Nakago IC where this bridge is located) It is open on two lanes and is the last tentative two-lane section on the Joshin-Etsu expressway.

The four-lane project in this section will strengthen the wide area network, prevent breakthroughs in the median strip, reduce traffic congestion during congestion periods, and secure smooth winter traffic on routes that pass through area of heavy snowfall. We have been conducting business since 2012 fiscal year. Among them, Otagirigawa bridge Phase II line (hereinafter, this is called this bridge) is located between the Myoko-Kogen IC ~ Nakago IC, face against the background of the Mt. Myoko is a Japanese one hundred famous mountains, one of the representative bridges on the route.

In this paper, we report mainly of design examination for vibration control in steel reversed Lohse bridge, as well as the safety measures on diagonal suspension method using temporary cables and traveler crane in close proximity the expressway.

2 BRIDGE OVERVIEW

Figures 1 and 2 shows a general view of the bridge. The upper line in the plan is the II period. The structure of this bridge is a fixed arched Lohse bridge with continuous girder stiffening girder. Since the bridge environment crosses the deep and steep Otagiri river with a height difference of about 50 m, the central span with a maximum span length of 167.0 m was installed by the diagonal suspension method using temporary cables and traveler crane. The construction weight is about 2,000t. The geology of the erection site is composed of pyroclastic flow deposits from Mt. Myoko and riverbed sediments caused by that deposit. It is an asymmetrical arch due to the arrangement of A-A2 substructure, which is an arch abutment. The abutments on both banks are inverted T-type abutments with spread footing foundations, the arch abutments are stepped footing, and eight caisson type piles of φ3.0m are used for A-A1 and six for A-A2, and that set up reinforced concrete pier with a leg height of 25.8m. After opening Phase I line, sand control dam was constructed downstream of intersecting Otagiri river. Because of that, it is difficult to access under the bridge. Sandwiched by the in-service Joshin-Etsu expressway to the east and high-voltage transmission lines and underground gas pipes to the west, fabrication took place in a confined construction yard.

3 FORMAT CONSIDERATION

The Otagirigawa bridge Phase I Line (hereinafter, this is called Phase I line bridge), which began operation in 1997, adopted a steel reversed Lohse bridge with a central span of 147.0m and a pier placed between the side spans A1 and A-A1 out of a debris flow of the Otagiri river that is soused from a headstream in the caldera at the summit of Mt. Myoko (Kita-Gigokudani Valley).

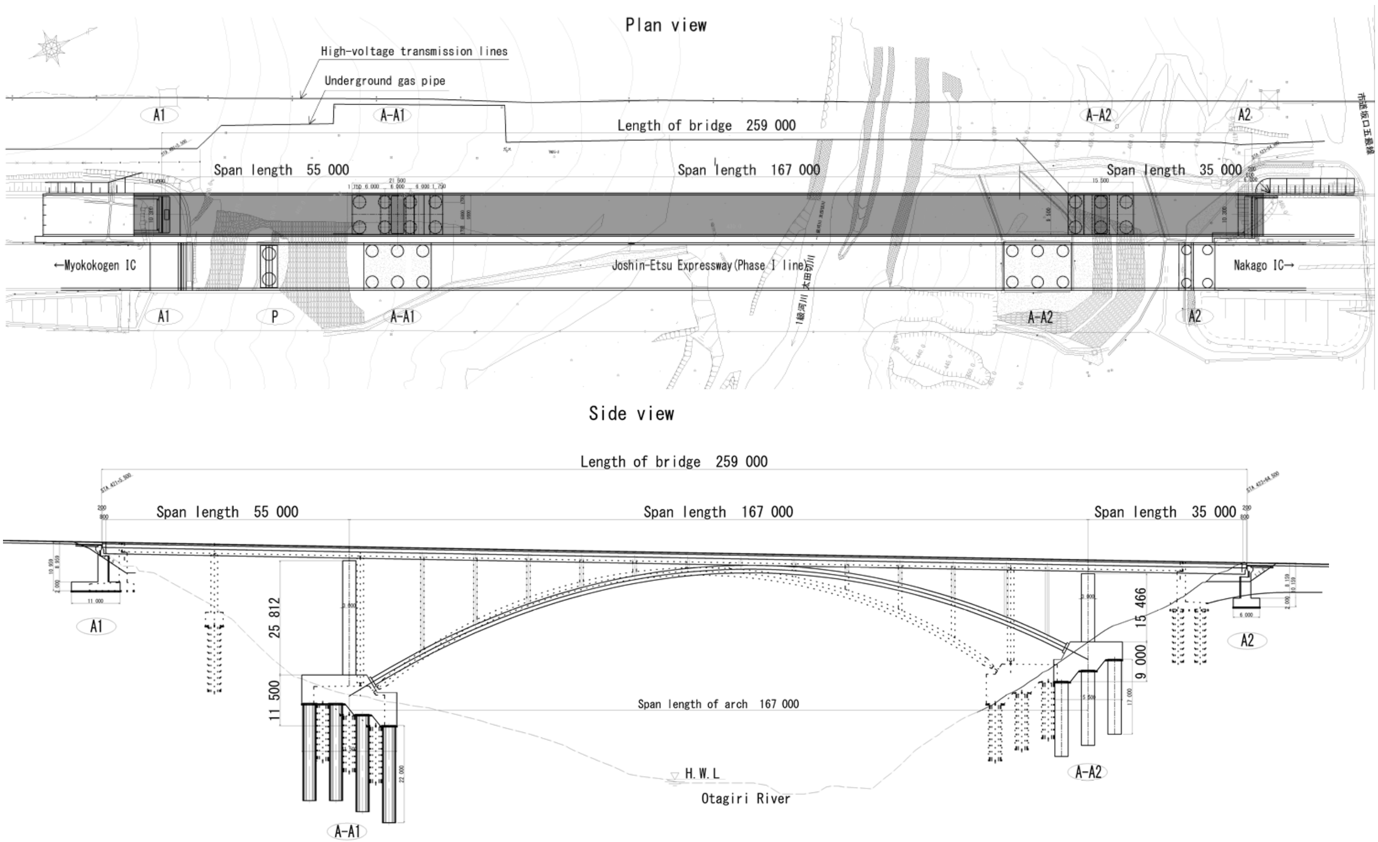

Figure 1. Figure of bridge public (plan view and side view).

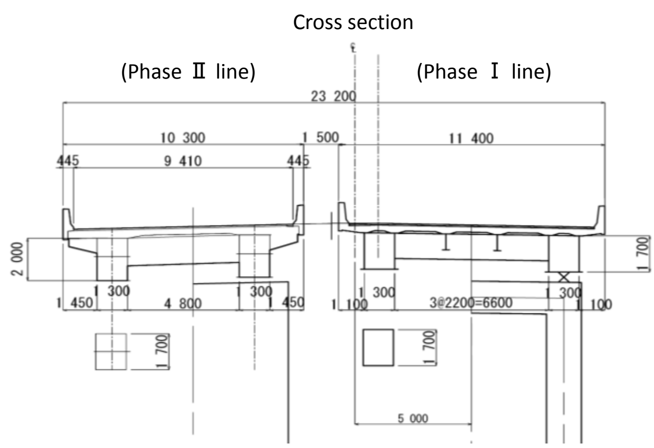

Figure 2. Figure of bridge public (sectional view).

Figure 3. View of Otagirigawa bridge from National Route 18.

In considering the format of this bridge, as a result of comparison with a three-span continuous PC box girder type, the plan was not to install a Phase II line's substructure footing deeper than Phase I line's substructure footing in order not to affect the A-A2 foundation of Phase I line bridge. Therefore, the center span length had to be about 20m longer than that of the Phase I line bridge. However, the PC 3 span continuous box girder bridge had a span ratio of 1:0.2 and a side span length shorter than the center span length. Since it becomes extremely short, the same steel Lohse type as the Phase I line bridge became economically advantageous. At this time, in the seismic study, the median strip was widened to secure the separation to avoid collision with Phase I line bridge, so the necessary space for installing the steel mast equipment when installing the cable crane could be secured. This also favored the adoption of the arch bridge. There is a concern that snow masses that have sticked on the cable crane equipment may fall on the service line, and due to safety reasons for the service line, the cable crane equipment cannot be maintained in winter. We considered that it is possible to install during the non-snow season from April to November.

In addition, we confirmed the silhouettes facing Mt. Myoko from the National Highway Route 18 running in parallel with the Joshin-Etsu expressway in several perspectives, and aligned the arch curves and vertical materials on the A-A1 side with the Phase I line bridge (Figure 3).

4 CONSIDERATION OF BUCKLING RESTRAINED BRACING

4.1 *Seismic Designs*

On seismic designs of this bridge, the response of the bridge-axial rectangular direction control becomes large response is, in order arch shape, as shown in Figure 1 is asymmetric, it is characteristic that the dynamic earthquake response is concentrated at the base of the arch on the A-A2 side the balance of the arch during dynamic earthquake. Therefore, in order to increase the rigidity of the entire bridge system from the time of the format consideration, a fixed arch type with a high degree of static instability was used. And increased rigidity in bridge-axial rectangular direction by adopting reinforced concrete wall-type pier in A-A1, A-A2 pier. Nevertheless, since the thickness of the arch ribs is 100 mm in excess of, a three-dimensional nonlinear dynamic analysis of the entire bridge system was performed in accordance with the specification for Highway Bridge at 2012, and the main steel members were in the elastic region. On the other hand, part of the lower lateral bracing, which is the secondary member, has a seismic control structure that allows plasticization. In this bridge, a buckling restraint bracing was adopted to control the earthquake force.

The buckling restraint bracing absorbs the energy of the earthquake to deform (plasticize) the low-yield point steel (LY225). It is a type of constrained elastic-plastic hysteretic damper from LY225 buckling for maximum utilization (Figure 4). Only a few domestic cases were adopted for the new bridge, and there was no track record for the expressway bridge. For this reason, in the detailed design of the superstructure, the specification and arrangement of this buckling restraint bracing were examined. The outline of the study is described below.

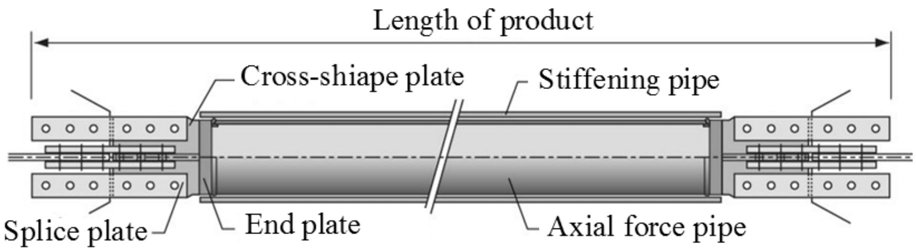

Figure 4. Structure of buckling restraint bracing.

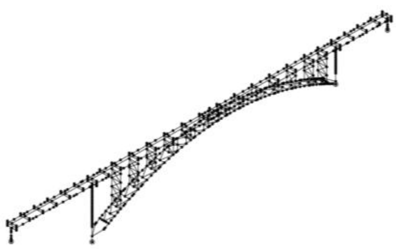

Figure 5. Analysis model.

4.2 *Arrangement of Buckling Restraint Bracing*

Initially, designs had plans to place buckling-restraint bracing on all panels below the arch ribs, but detailed designs were re-examined using the following flow of the optimal specification and arrangement.

Step 1: In order to understand the effect of the lower lateral bracing on bridge, we examined an extreme case where there was no lower lateral bracing at all. There of results, in order to satisfy the checking against the maximum thickness of specification from Highway Bridge provision thickness of arch rib near A-A2 was found to exceed significantly 100mm of provision both flange and web.

Step 2: A section steel was installed in the lower lateral bracing to satisfy the allowable axial compressive tress indicated in specification from Highway Bridge. Thereof a result, the thickness of arch rib was slightly improved compared to STEP1, the maximum thickness of specification for Highway Bridge provision could not be satisfied.

Step 3: When buckling restraint bracing were placed in the lower lateral bracing, the thickness of the arch ribs tended to be significantly improved. The buckling restraint bracing do not yield under the axial force generated by the level 1 earthquake and the wind, but must enter the plastic region and absorb sufficient energy in the level 2 earthquake. This bridge adopted a buckling restraint bracing with a yield axial force of 1,500 kN that satisfies this requirement.

Step 4: Figure 5 shows the analysis model of this bridge. The distribution of the maximum axial strain generated in the lower lateral bracing (buckling restraint bracing, Figure 7) shows that the deformation near the quarter of the arch span is dominant, while the deformation near the arch crown is small and the elasticity is low. It was confirmed that it stayed in the area. For this reason, the panel with the maximum axial strain of less than 1.0% the buckling restraint bracing was omitted, and the maximum thickness of the arch rib converged at 97mm by the arrangement shown in Figure 6.

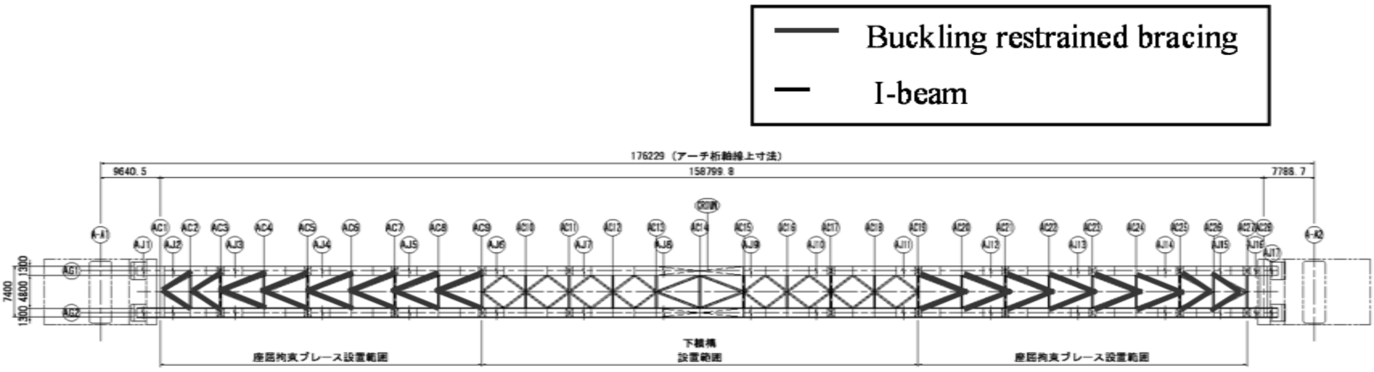

Figure 6. The location where I located buckling restraint bracing.

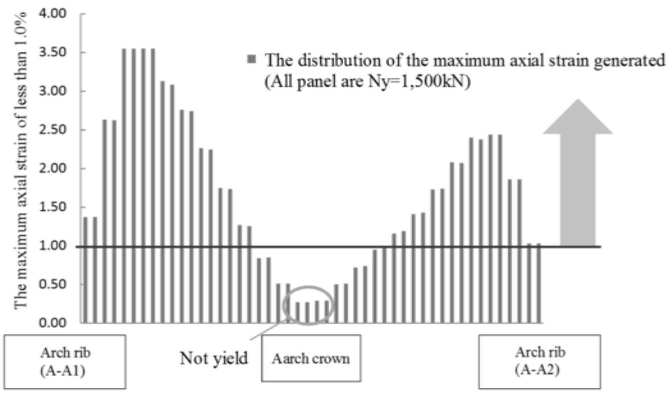

Figure 7. The distribution of the maximum axial strain generated.

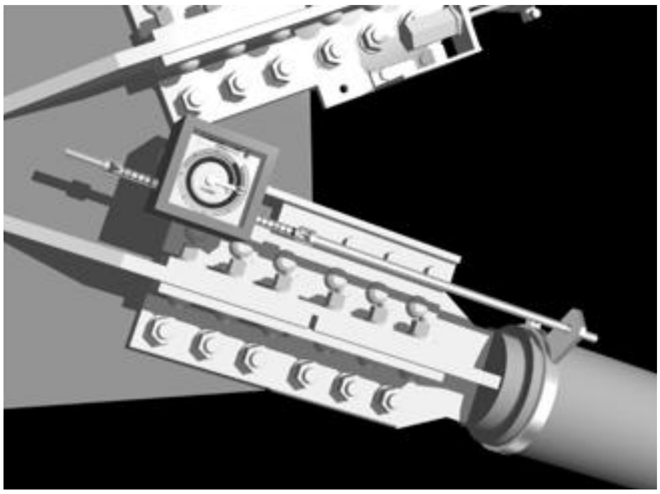

Figure 8. The measuring device of cumulative plastic deformation.

4.3 *Maintenance of Buckling Restraint Bracing*

Since the buckling restraint bracing is a member that is plasticized by earthquake, it is necessary to perform later inspection for the damaged area and award replacement. The residual performance of the buckling restraint bracing can be confirmed by the cumulative plastic deformation, but it is difficult to directly check the deformation because the plastically deformed part is covered by the restraining pipe. Therefore it was established a measuring device shown in Figure 8 to be able to see the cumulative plastic deformation from outside the restraining pipe. This measuring instrument is a mechanical type that requires no power and can be read. The structure is a ratchet type with a combination of gears, in which the displacement difference between the core material (movable side) and the restraining pipe (fixed side) is converted in one direction and accumulated and recorded. In consideration of durability, stainless steel was used for the mounting hardware, meter outer box, and internal gear material. Also, member installation position access to has been a problem in the maintenance such is not easy, earthquake immediately after generation and regular inspection to verify its identity at easily measuring instruments, inspection route was examined and inspection road was set up on the top of arch rib and maintenance was considered.

5 CONSREUCTION PLAN

5.1 *Preparatory Work*

At the time of construction of the Phase I bridge, access was possible from the downstream of the Otagiri River. After opening Phase I line bridge, sand control was constructed downstream of intersecting Otagiri River. Because of that, it is difficult to access under the bridge. It was also considered to cross the sand control dam with a temporary pier or the like, but a temporary pier parallel to the main bridge from both abutments was adopted which can be used for side span erection of superstructure.

5.2 *Construction of Substructure*

Although the influence on the Phase I line bridge basis was reduced and the chemical solution was grouted foe the hoke wall protection of earth retaining, it took time to process debris flow with boulders exceeding φ1.0m and pyroclastic flow deposits(Figure9) Furthermore, the hole wall for inserting the H steel of the retaining pile was not self-supporting due to the large amount of groundwater, and the construction was performed with a down-the-hole hammer that also used a casing. The process delay due to boulders processing became the performing the substructure construction in the snow season, cover the entire arch abutment around and piers in thermal insulation sheets, the snow fence were placed each two jet heater organaizing to cold weather construction (Figure 10).

5.3 *Construction of Superstructure*

5.3.1 *Side span erection*

At first, the construction of the side span was planned to install a truck crane vent using a temporary pier. However, the boulders delayed the construction period of the lower part, and the vent was to be installed on steep slopes where the construction period was expected to be longer. We decided to avoid it. In addition, since the construction of the girder was required to be completed by the time of the snowfall, a single-operation erection method with a multi-axle bogie was adopted from a temporary pier that can be constructed in a short period of time during the winter (remaining snowy season) (Figure 11).

At the work yard on the back of the abutment, the side span (girder length A1 side: 62.3 m, A2 side: 38.3 m) was constructed at ground level, transferred laterally to the tip of the temporary pier with a multi-axle bogie. In order to reduce the amount of main girder descent (3.5m in total) adjacent to Phase I line, a primary descent (0.8 m) was carried out on the temporary pier and replaced with pre-emption equipment (Figure 12).

Since this bridge is in close proximity to the Phase I line bridge in service, the laser barrier was operated at the boundary with the construction yard from when the side span was erected, so that cranes and equipment could be crossed on the service line same on the (transmission line side). During the term construction was constantly monitored and the construction proceeded so as not to cross the border.

5.3.2 *Central span erection*

The cable crane equipment for the installation of the central span is to install the steel mast above the road surface of the service line, so that the steel mast equipment should be as small as possible, such as to reduce the feeling of pressure on passing customers and to shorten the installation and dismantling time. In order to do this, a double-purposed mast was used for the construction cables (three: the truck saddle for the mast in Figure13) and the diagonal suspension cables (four: diagonal suspension suddle for the mast). Further, the modification of the load pickup yards from the A2 abutment back of the originally planned to the A2 side span girder, could be reduced to about 30m a mast height of originally planned 32m mast span length is shortened (Figure13). Since the anchor block behind the mast could only be installed in the direction parallel to the main line, a total of four large-diameter caisson type piles (φ6.5m) were organizing on the back of the abutment on the Phase II line side, and the same type of construction method in the past was used. In consideration of the accidents that occurred during the method in the past, the safety measures for the rear cable were strengthened by increasing the number of wire clips on the rear cable, and the safety monitoring system for the diagonal suspension equipment was organizing to constantly monitor the inclination of the mast and the cable tension.

Figure 9. The distribution of the maximum axial strain generated.

Figure 10. Installed snow fence.

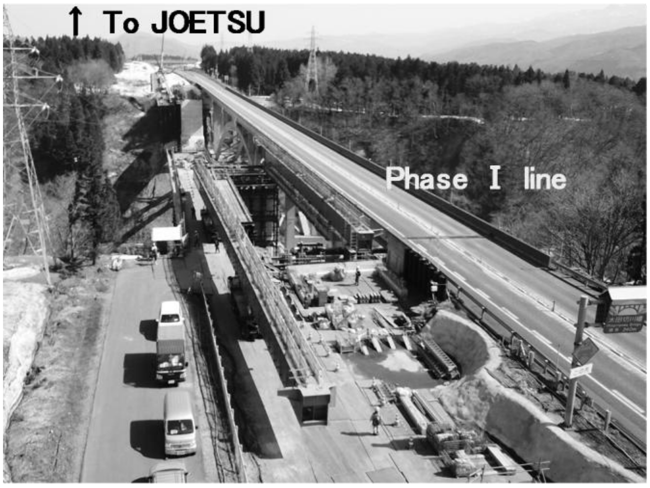

Figure 11. Transportation by multi-axle trolley.

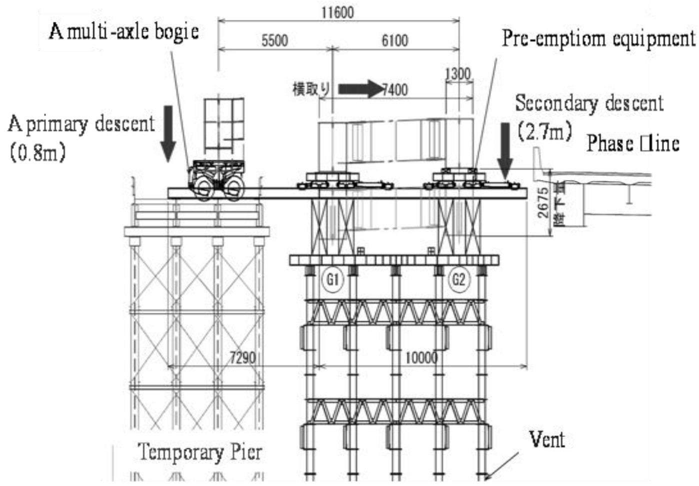

Figure 12. Plan of pre-emption equipment.

In addition, since the installation of the mast equipment will be performed for seven months from April to October, the mast will be connected to the substructure with wires to make it fail-safe so that the mast will not fall to Phase I line side due to large-scale earthquake (Figure 14).

For the installation of the main members, a 90-ton crawler crane (C/C) was placed on the temporary pier on the A2 side for loading because the trailer could not directly enter the loading yard. During erection to further reduce the influence on the Phase I line due to swinging of the load member, the depression angle 75 degrees Phase I line following scope was to perform outside (Figure 15).

Also, in the case of single-material erection, the possibility that the suspended load rotates and protrudes to Phase I line side increases. Therefore, the panel is assembled in the loading yard (on the span on the A2 side). It was planned to be suspended by the two erection cables (Figure16, 17).

At the base of the arch, in order to secure the erection shape of the arch, a temporary bearing was used as a pin bearing, and a fixed later 205 mm × 14 anchor bolts were tightened and fixed.

The accuracy of girder erection was managed by three-dimensional measurement using a total station, and measurements were taken at each erection step early in the morning, where the temperature was less affected, to ensure accuracy.

It was feared that the mast would have snowfall due to early snowfall during the construction, but fortunately there was no early snowfall and the construction was completed successfully (Figure 18).

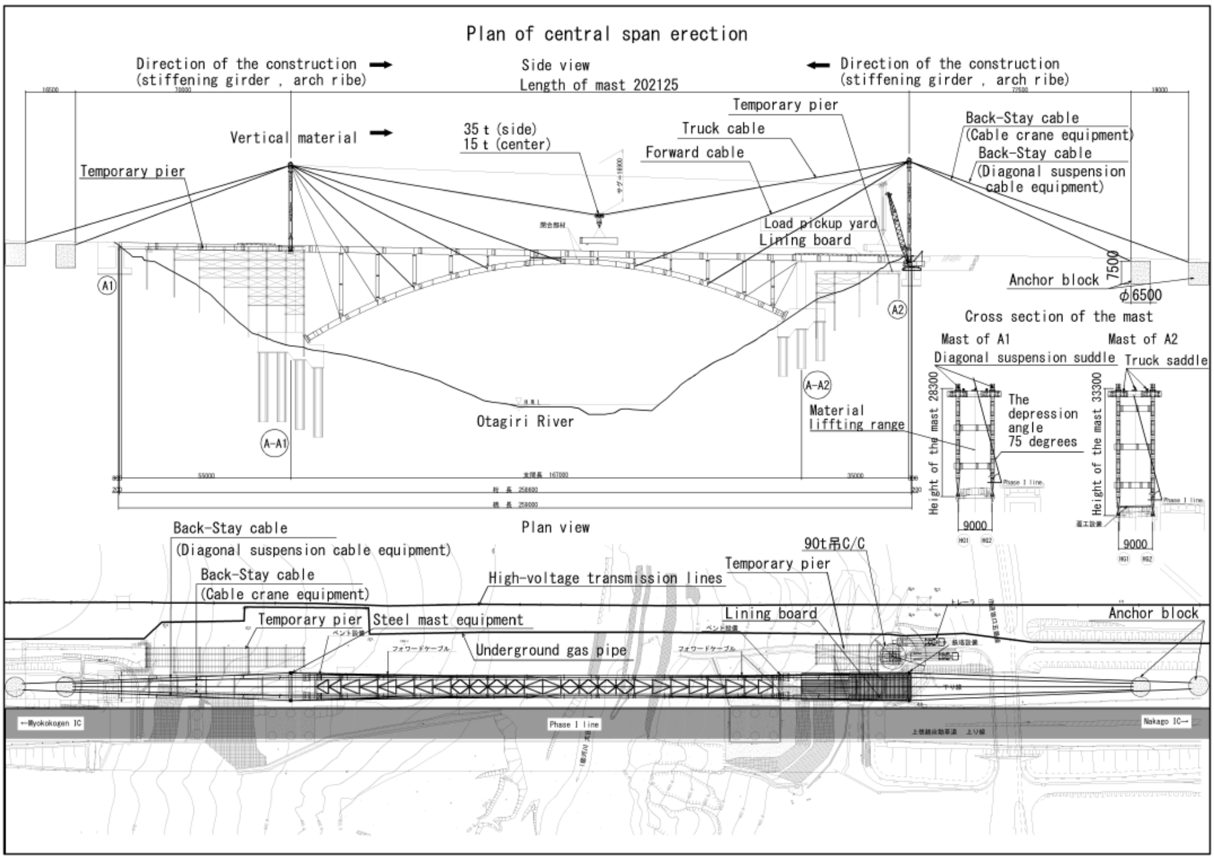

Figure 13. Plan of central span erection.

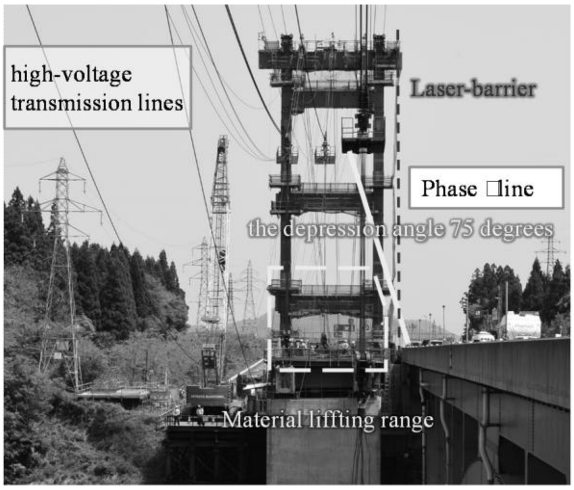

Figure 14. Setting of the material lifting range.

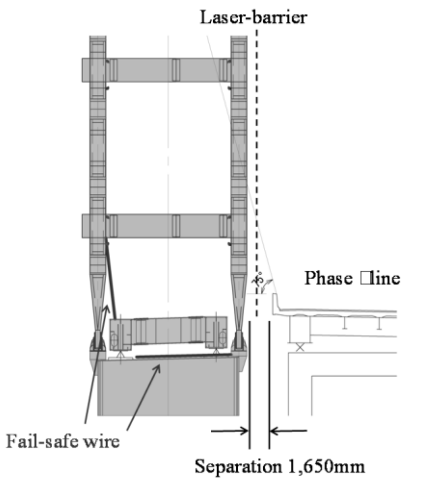

Figure 15. Fail-safe of the mast.

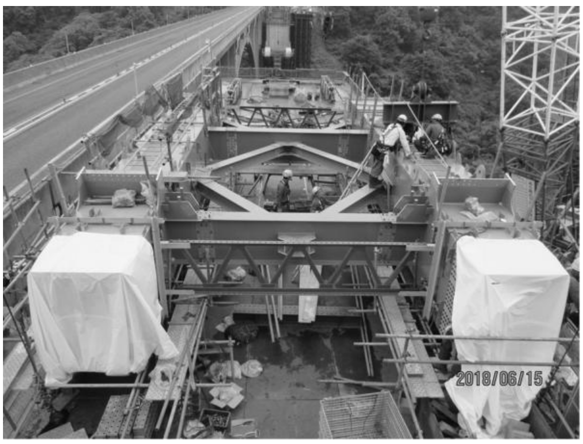

Figure 16. Suspended by the two erection cable.

Figure 17. Phase suspension erection.

Figure 18. Construction completed.

Figure 19. Floor slab erection.

Figure 20. Inspection road of superstructure.

Figure 21. Inspection road of arch crown.

Figure 22. The oil/water separator on the pier.

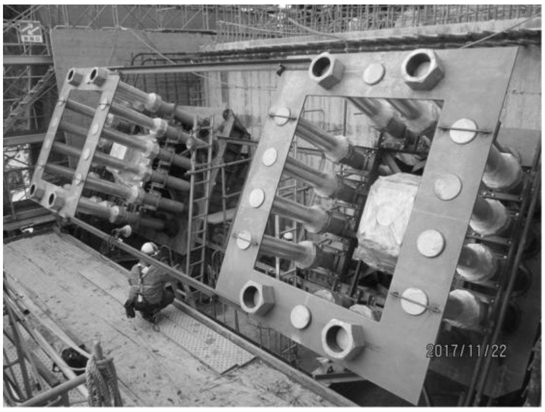

Figure 23. Arch-base anchor bolts with hot-dip galvanized.

Figure 24. Arch base for improved durability.

Figure 25. Arch base roof.

5.4 *Floor Slab Construction*

After erection of the arch girder complete, It was necessary to make effective ese of short period until the snow for the slab construction. The precast PC slab was adopted in the scope perfection capable of both sides side span from the temporary pier, shortening the construction period (Figure 19). The size of the precast PC slab was 2,000 mm in width in the bridge axial direction and 9,900 mm in width at rectangular direction. 31 slabs were used on the A1 side and 16 slabs were used on the A2 side. The joint structure between the PC versions was a jawless RC loop joint.

Construction was completed with the cast-in-place PC slab, wall railing, and bridge accessories for the center span, and handed over to pavement construction at the end of July, 2019.

6 CONSIDERATIONS FOR MAINTENANCE

Since this bridge is classified as a special bridge, various considerations were made in view of the future maintenance stage. Inspections road, zinc-aluminum alloy plating with high anticorrosion performance was adopted because corrosion was confirmed on the Inspections road of the Phase I line bridge. Important inspection points such as the points and attachments between the arch ribs and the vertical material, the locations where mechanical meters are installed, and the access to parts inside the arch ribs are wide inspections roads (Figure 20), and large bridge inspection vehicles are used. An Inspections path was also provided under the overhanging slab around the median strip before and after the arch crown where it would be difficult to inspection even if used (Figure 21).

In addition, when the oil spill from the accident vehicle or the like, oil-water separation basin to prevent the oil to a off highway flows out, access review the installation of on difficult steep slopes. Installing at the pier side substructure inspections road by installing directly underneath and making the sidewalk of the inspections road openable, the access from the road surface and the cleaning and inspection of the oil / water separator are facilitated (Figure 22).

The anchor bolts at the base of the arch are very difficult to replace. Therefore, antirust is applied to the objectives by hot-dip galvanizing (Figure 23), and the completed fixed later is covered with a protective cap. The durability was improved by filling with epoxy resin in order to make cap and bolt adhere (Figure 24). At the end, dead leaves and other deposits were confirmed at the base of the arch of the Phase I line bridge, so a roof was provided for protection (Figure 25).

The abutment girder clearance was 700 mm wide for inspection and girder cleaning work, and concrete coating was applied to the abutment and linig concrete to prevent salt damage due to water leakage from the expansion device.

7 CONCLUSIONS

Under strict constraints, the erection was carried out with careful construction planning and safety measures. As a result, the erection was completed without any accidents. In addition, the designs was able to be realized with an eye on the maintenance after operation.

Joshin-Etsu Expressway The four-lane project between Shinanomachi IC and Joetsu JCT completed approximately 80% of the section extension within 2018, and the entire four-lane system was completed in December of 2019. We hope that this report will serve as a reference for similar bridges and the increasing number of four-lane projects in the future.