ACCELERATED REPLACEMENT OF STEEL PLATE RAILWAY BRIDGES USING MOVING CRANES ON RAILROAD

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Abstract The existing steel-plate-girder railway bridges in Korea have been aged and thus lost the integrity to obtain required riding quality during train passing. In addition, the existing bridges do not have blast, and thus impacts due to train passing deliver seriously to piers and abutments, which is one of main cause to produce cracks on pier concrete. To solve this problem, replacement of the existing bridges is required. However, they are in service and thus conventional construction method cannot be applied for the replacement. A new method is necessary to replace quickly the existing bridges not to disturb running train-operation.

In this study, a new accelerated replacing method was suggested using a newly developed crane-vehicle which was a train-vehicle with possessing cranes to lift the replacing bridge. This study also designed and manufactured a new bridge deck for the accelerated replacing construction. Finite element analyses and experimental tests have conducted to estimate the performance of the new bridge deck. The analyses included a static load case and dynamic analysis with varying train speed. The experimental tests consisted of modal tests, which captured the fundamental natural frequency and damping ration of the bridge deck, and an accumulated loading test. The results of this study can be used for practical replacement of the aged existing steel-plate-girder bridges and contribute to improve the integrity and riding quality of railway bridges in Korea.

1 Introduction

Open-steel-plate-girder (OSPG) bridges are one of the most common types of bridges on conventional Korean railway lines (as opposed to high-speed railway lines). According to inventory data [2], more than 35% of conventional railway bridges are OSPG bridges. Most of these bridges were constructed prior to the 1970s and were therefore designed without consideration for ride quality or vibration. OSPG railway bridges are characterized by a superstructure consisting of railway tracks sitting directly on steel plate girders without any ballast system. Hence, train axle loads are directly transferred to the steel girders and then to the piers and foundations without any absorption of the resultant vibration. This type of load transfer adversely affects the dynamics of bridge deck systems, bearings, and coping regions of the piers. Almost all the OSPG railway bridges in Korea have gravity-type concrete piers without reinforcement. The outdated characteristics of these OSPG bridges require their rehabilitation or replacement.

Rehabilitation and related studies of Korean OSPG bridges have been conducted continuously. Choi et al. (2010) introduced a method to replace the timber track of an OSPG bridge with a concrete slab track and analyzed the dynamic behavior of the bridge after replacement. Choi et al. (2011) and Rhee et al. (2011) introduced a seismic retrofitting method for strengthening a plain concrete pier and conducted an analysis of soil-structure interaction. These previous studies suggested methods of rehabilitation or replacement of parts of an OSPG bridge to improve dynamic response, absorb impacts, and improve seismic resistance. Recently, replacement of the entire superstructure has been suggested as a step that could address all the problems of OSPG bridges. However, the replacement method would need to be completed in a few hours since the railway bridges of interest are still in service. In addition, accelerated replacement is required to minimize direct and indirect costs related to stopping or diverting trains in motion.

The existing OSPG railway bridges have timber tracks connected by hook bolts and are seated directly on the steel plates as Choi et al. (2010) illustrated. The hook bolts are installed to tightly connect the timber tracks to the steel plates, but tend to fall off easily due to repeated impacts from running trains. Because the steel girders in OSPG bridges are of relatively light weight, a large vertical acceleration of the girder is induced. This deteriorates the ride-quality of trains. The loud noise from these bridges when trains pass over them has also become an increasingly important environmental problem. For all theses reasons, replacement of the whole superstructure of the bridge is required.

In this study, a new method for accelerated-replacement of the existing superstructure on OSPG bridges is proposed using a newly developed crane-train which moves on train tracks but has cranes adequate to lift such superstructures. This study also addresses design and manufacture of the new superstructure required for such accelerated replacement. The superstructure is tested with static and dynamic loading. Finally, this study reports the results of finite element analysis conducted to estimate the dynamic peak response of the superstructure resulting from passing trains running at various speeds.

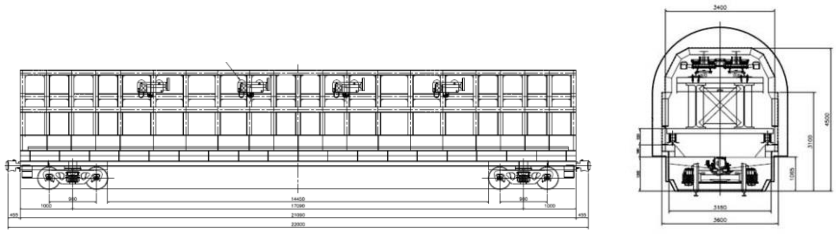
2 New accelerated replacement method using a crane-train

A crane-train consists of a crane-vehicle and two carrying-vehicles (Fig. 1). The crane-vehicle is located in the middle, with the two carrying-vehicles at either end.

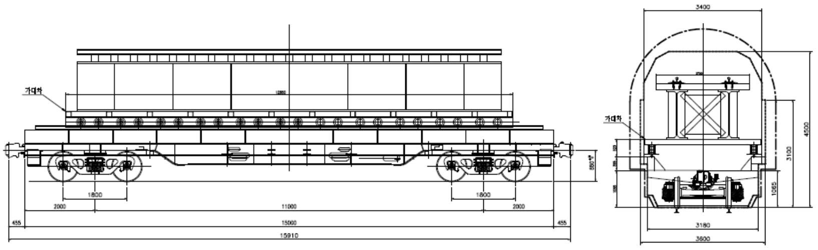
When the three vehicles are on the same track, one carrying-vehicle is ahead and the other is behind. The carrying-vehicle is devised to deliver existing and replacement superstructures between manufacturing and construction sites. The crane-vehicle is designed to lift and remove an existing superstructure; then set down a new replacement superstructure using cranes (Fig. 1a). Fig. 2 illustrates the procedure of replacing an existing superstructure. First, a crane-train with a replacement superstructure on the rear carrying-vehicle approaches and stops at the target span of the bridge. Then, the existing superstructure is lifted off and placed on the front carrying-vehicle. Sliding rails are used to move the old superstructure from the crane-vehicle to the carrying-vehicle. The replacement superstructure is slid onto the crane-vehicle and then raised to the replacement location. Finally, the crane-train is evacuated through the newly replaced superstructure. The replacement is expected to be completed in three hours.

The proposed method has several strong points: 1) It can be applied under any site situation, including rivers, valleys, or urban areas, since all the work is performed on the rail, 2) The method minimizes secondary costs for the replacement since the method does not block road traffic, and 3) The replacement superstructures can be of consistently high quality since they will all be manufactured in a factory.

In general, rails on the OSPG bridges are continuous. Thus, in the new method, the continuous rails should be cut and, after the replacement, connected again by welding on site. After complete replacement of the whole span of a bridge, the cut-and-welded rails would be replaced by new ones.



(a) Crane- vehicle



(b) Carrying-vehicle

Fig. 1. General and side view and a crane-train

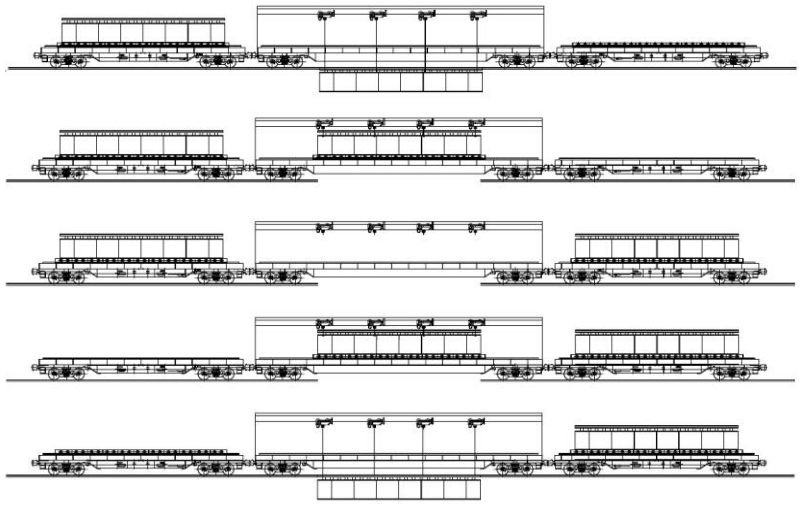
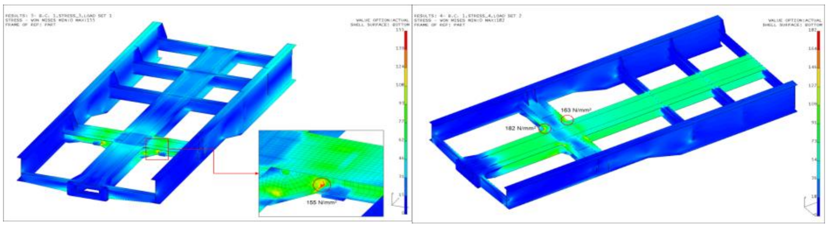


Fig. 2. Procedure for replacing the existing superstructure of an OSPG bridge

2 FE analysis of crane-and carrying-vehicle

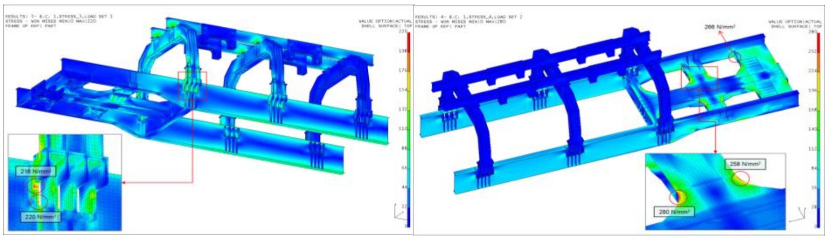
In this study, we performed FE (Finite Element) analyses of the crane and carrying vehicles using the IDEAS program [6]. The analytical models included only steel frames made from SMA490A, with a yield-strength of 325 MPa. Shell and plate elements were used in the analysis. In accordance with the railroad vehicle safety code, two load-cases were used (i.e., Load Case I: 1.3g x W loading and Load Case II: 2000 kN compressive loading). Load Case I represents a load that is 1.3 times the weight of a vehicle without an external load. For Load Case II, the weight of the vehicle is distributed on its surface and a 2000 kN lateral compressive force is applied at the coupler. Figures 8 and 9 show the analytical results for both vehicles.

The carrying-vehicle showed maximum stresses of 155 MPa and 182 MPa at the bolster for Load Cases I and II, respectively. The stresses developed in the carrying-vehicle were smaller than the yield strength (in railroad vehicles, the yield strength is allowable stress). For the crane-vehicle, stress of 220 MPa occurred at the bottom of a post for Load Case I, and stress of 280 MPa occurred at the upper plate of the bolster for Load Case II. For the crane-vehicle, the maximum stress of both load cases was smaller than the allowable stress. The FE analyses showed both of the newly designed vehicles were safe for the expected loading. However, the maximum stresses of both vehicles were observed at the bolster. This means that a fatigue assessment of the bolster will be required in an additional study.



(a) Case I (1.3g) (b) Case II (2000 kN)

Fig. 3. Analytical results of the carrying vehicle

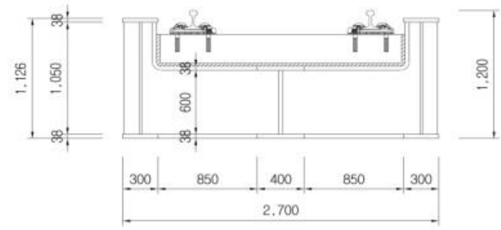


(a) Case I (1.3g) (b) Case II (2000 kN)

Fig. 4. Analytical results of the crane vehicle

3 Tests of the new superstructure and results

The superstructure designed in this study was a 12 m OSPG-type bridge; its total length was 13.4 m and the span between supports was 12.9 m. This is the largest superstructure that can be delivered by the crane-train. The superstructure consisted of three girders and a concrete slab, which contained rails and rail-fasteners (Fig. 5). Each girder was connected by cross-beams that also supported the concrete slabs. The superstructure was manufactured in a factory. The superstructure for the experimental tests did not include the rails and fasteners, and in this condition, its weight was approximately 420 kN. The experiment included static loading tests, tests of dynamic characteristics, and a dynamic magnification test. The superstructure was simply supported and an actuator of 250 kN was installed at the middle of the superstructure. In addition, displacement-transducers and accelerometers were installed on the bottom of girders (Fig. 6).



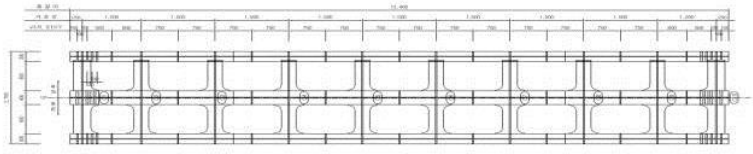
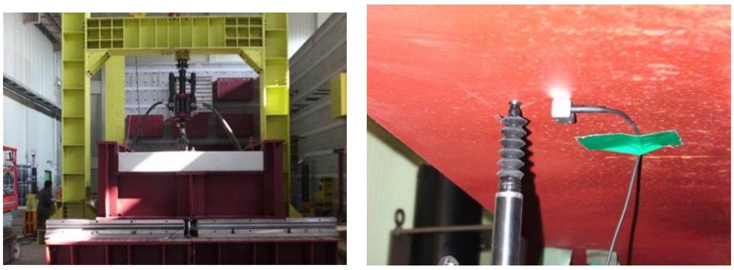


Fig. 5. Shape of a replacement superstructure



(a) Actuator (b) Displacement-transducer and accelerometer

Fig. 6. Test set up and measurement

4.1 Static tests and results

During the static tests, displacements were measured at the mid-point under the three girders as well as at the one-quater and three-quarter points under the central girder. A load of 2000 kN was applied monotonically. Fig. 7 shows the force-displacement relationship due to the static loadings. The three displacements at mid-span of the three girders showed almost exactly identical curves. The same thing was observed on the curves measured at one- and three-quarter points of the central girder. This indicated that torsional displacement did not occur and that the superstructure contained the same stiffness on both sides. In addition, the displacements at one-quarter were exactly consistent with those at the three-quarter point. This showed that the bending stiffness of the superstructure was symmetric.

The estimated bending rigidity (EI, where E and I are the Young's modulus of the material and second moment of cross-section, respectively) of the superstructure from the experimental data (5.04 x 106 kN·m2) was 1.76 times that of the existing superstructure (2.87 x 106 kN·m2). The new superstructure had an additional steel girder and the concrete slab showed composite behavior. Thus, it showed greater bending rigidity compared with the corresponding older superstructure.

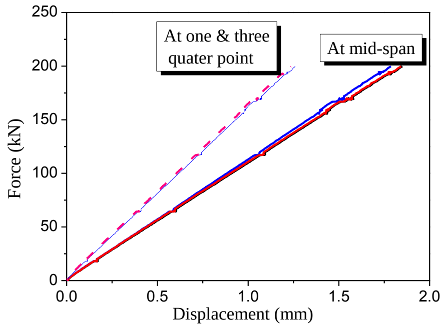


Fig. 7. Force-displacement relationship

4.2 Dynamic tests and results

Dynamic tests were conducted to assess the fundamental natural frequency of the superstructure and its damping ratio. To obtain these dynamic characteristics, a free vibration-response is necessary. Therefore, this study adapted quick-release and impact tests for this purpose. For the quick-release test, the superstructure was displaced downward by 1.0 or 2.0 mm and released abruptly to obtain the free vibration-response of the superstructure. In the impact test, the superstructure was hit at mid-span, or a quarter point, by an impact hammer to cause free vibration. In these tests, displacement and acceleration at mid-span of the superstructure were measured to estimate the natural frequency and damping of the fundamental mode. The estimated fundamental natural frequencies were 11.1 Hz, based on results from the impact and quick-release tests. According to UIC regulations, the lower and upper bounds of the fundamental natural frequency of railway bridges are 6.0 Hz and 14.0 Hz, respectively, for the span-length of 12.9 m [7]. Therefore, the UIC regulation was satisfied by the new superstructure, which is being provided to guarantee the ride quality of trains running over bridges. The damping ratio $\xi(\%)$ can be calculated using:

$$\xi(\%) = \frac{1}{2\pi n}\ln(\frac{u_1}{u_n})\times 100 \qquad (1)$$

where, u_1 and u_n are values at the first and nth peak in free vibration response. The average estimated damping ratio was 1.2% which was typical for steel structures.

The existing superstructure showed the fundamental natural frequency of 12.4 Hz which was similar to that of the new superstructure [7]. Both superstructures showed a similar fundamental natural frequency although the stiffness of the new superstructure was 1.7 times greater than that of the existing one. The reason for this is that the new superstructure was heavier than the existing one.

In this study, we also performed a dynamic magnification test which applied harmonic loading with varying loading frequency. The harmonic vibration had an amplitude of 20 kN and frequency-increment of 0.5 Hz starting from 1.0 Hz. The three curves showed peaks at 11.0 Hz because of resonance. A dynamic magnification factor (DMF) can be obtained by dividing the dynamic displacement ($D_{dymanic}$) by the static displacement (D_{static}).

$$DMF = \frac{D_{static}}{D_{dynamic}} \tag{2}$$

Fig. 8 shows the DMF of the superstructure, and the peak values were 1.73, 1.67, and 1.58 at 11.0 Hz for center, right, and left girder, respectively. Thus, the DMF was estimated to be 1.66 at resonance.

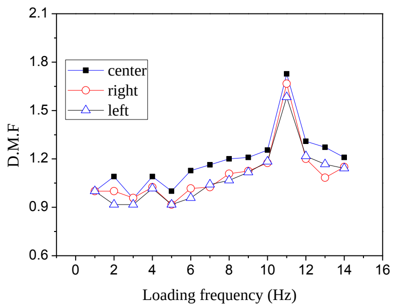


Fig. 8. Dynamic magnification factor

6 Dynamic analysis at various train speeds

Choi et al. (2012) provided measured displacement responses of an OSPG bridge, to a locomotive moving at various speeds. They compared dynamic displacement of OSPG bridges according to bridge bearing types. Kim et al. (2011) measured dynamic displacements of a high-speed railway bridge in Korea, in response to a moving high speed train, and compared them with those of an analytical model. They showed that the analytical results matched well with the measured ones.

This study performed FE analyses of the new and existing superstructures to assess and compare their dynamic responses for trains running at various speeds. For the analyses, a locomotive and two freight-trains were used. In the analysis, the forces were moving on the superstructures and the masses of the vehicles were not considered. A detailed explanation of how to apply moving forces was described in a previous study [1]. Multiple concentrated dynamic forces were applied at joints, considering the speed of the vehicles. The speed of the train varied from 10 to 150 km/h with an increment of 10 km/h. Displacement and acceleration at mid-span were calculated, and their maximum values were plotted according to speed (Fig. 9).

For the pseudo-static response at 10 km/h, the new superstructure showed 1.74 times less displacement than the existing one. This result corresponded to the estimates of bending rigidity of both superstructures made in the previous section. In the existing superstructure, sub-resonance was observed at a speed of 130 km/h. The displacement of the existing superstructure at 130 km/h was 9.2 mm. This was 12% larger than the response of the pseudo-static response. However, its acceleration at 130 km/h showed the very large response of 0.85g which was larger than the limitation of 0.5g. If acceleration exceeds that limit, the ride quality of trains may be diminished. For the new superstructure, sub-resonance was not observed and the maximum acceleration was 0.18g, which was much smaller than the 0.5g limit. Also, the displacement did not increase very much with increasing speed. Consequently, the new superstructure had decreased displacement and improved the ride-quality of trains compared with the existing superstructure.

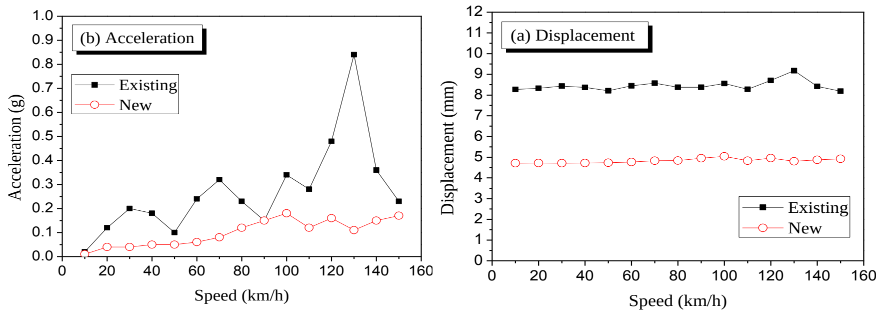


Fig. 9. Dynamic responses with various speeds

6 Conclusion

In this study, we proposed a new accelerated method for replacing the existing old superstructures of OSPG railway bridges with new ones. The method uses a crane-train consisting of a crane-vehicle and two carrying-vehicles. The proposed method had strong points: 1) it made replacement possible for bridges over water or valleys, and in urban areas; 2) it avoided additional construction costs; 3) it minimized secondary social costs resulting from blocking of road traffic; 4) it enabled consistently high quality of replacement superstructures. However, for the proposed method, a new superstructure must be delivered and installed from inside railway vehicles, which placed restrictions on dimensions and weight. During this study, we conducted static and dynamic tests of the new superstructure. The static test indicated that the bending rigidity of the new superstructure was 1.7 times greater than that of the existing one. In the dynamic test, the fundamental natural frequency and damping ratio of the new superstructure were estimated to be 11.1 Hz and 1.1%, respectively. Both of these values satisfied the limits of the UIC and Eurocode. The dynamic magnification factor of displacement was estimated to be 1.7 at resonance.

Finally, we conducted FE analyses of the new and existing superstructures using a locomotive and two freight-trains at various speed. For its pseudo-static response, the new superstructure showed approximately 1.74 times less displacement than the existing one. The new superstructure also did not show sub-resonance up to the speed of 150 km/h, which is the maximum speed on a normal railway. The existing superstructure showed sub-resonance at 130 km/h and the acceleration increased up to 0.82g, which was enough to disturb the ride-quality of trains. For the new superstructure, the maximum acceleration developed was 0.19g at 100 km/h. Therefore, it can be said that the new superstructure decreased displacement and acceleration and, thus, improved the safety and comfort of trains running over it.

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