Recent Topics on Steel Bridge Engineering in Japan -design and maintenance

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Abstract
Current status on bridge engineering in Japan is briefly summarized. Design and maintenance related research topics are explained. It is design method and activity towards LSD for hybrid girder bridges is also introduced. As maintenance related issues, SHMS, application of new material of CFRP, redundancy analysis and weathering steel are introduced.

Keywords: LSD, maintenance, hybrid bridges, SHMS, CFRP, weathering steel, redundancy

1. Introduction
In this paper, two topics related to recent bridge engineering in Japan are dealt with. One is design issue and another is maintenance issue. First, current situation on steel bridge engineering and business is briefly explained, in which it is stated that new construction business has been decreasing, on the contrary, deteriorated old bridges have been increasing. With respect to the design issue, the activity on making Performance-based Limit State Design (PB-LSD) in Japan Society of Civil Engineers (JSCE) is explained, and importance of introduction of LSD for steel-concrete hybrid girder bridges is emphasized. Regarding maintenance issue, SHM technology, redundancy analysis, weathering steel, CFRP bonding to corroded steel members are introduced.

2. Current situation on steel bridge engineering
In Japan, the steel volume consumption for new bridge construction has been decreasing. On the contrary, old bridges have been increasing, and it has been reported, in 20 years, that a half or more of existing bridges with a span exceeding 15 meters is predicted to be over 50 years old. Fig.1 shows the construction number of bridge per year in USA and Japan. In USA, where the development of highway infrastructure started 30 years earlier than Japan, improper management and maintenance of them caused serious condition called “America in Ruins” in 1980’s. If highway infrastructures are not maintained properly, Japan will also face similar situation to USA. In order to maintain the old bridges and to keep sound performance, the development of new technology such as SHM (inspection and detection of damage), introduction of new materials (for repair) and so on are very important.

Due to shrinkage of new bridge construction business and strong request on construction cost cut, competition between steel and concrete alternatives is becoming severe and severe. To cope with this subject, the development and proposition of steel-concrete hybrid bridges is active. The details will be explained in the next chapter.

Fig.2 shows examples of corroded steel members. After the collapse of I-35W interstate bridge in Minnesota, USA, Ministry of Land, Infrastructure, Transportation and Tourism (MLITT) belonging to a central government of Japan decided to change policy. It is from remedial repairing or rebuilding to preventive inspection and repair work, which will be carried out cyclically. Bridges owned and managed by MLITT have been inspected per 5-year, and the repair has been carried out, if judged to be necessary. However, almost all bridges owned by local or regional government have not been inspected due to lack of financial budget. Hence, as seen in Fig.2, a lot of damaged members mainly due to corrosion has been observed and has been predicted to increase. Hence, MLITT decided to finance the local bridge inspection and repair. In Japan, normally, a life span is considered to be 50 to 60 years. If we estimate the cost for replacement of 50 or 60 years old bridge,

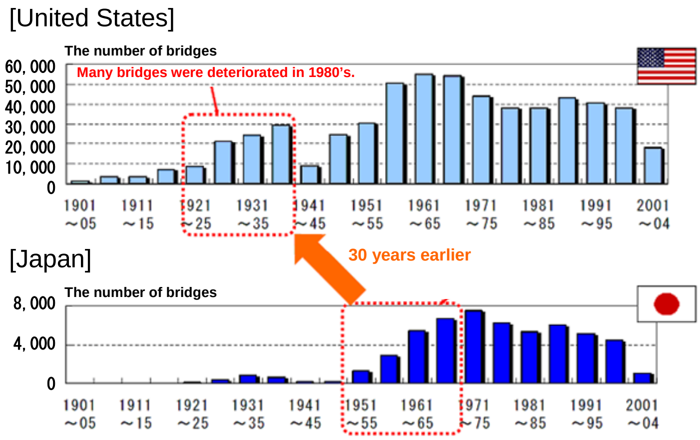


Figure 1. Comparison of bridge construction number between USA and JPN (by MLITT)

Figure 2. Examples of corroded steel members (from booklet issued by MLITT)

huge budget has to be prepared. On the contrary, if we continue preventive procedure such as cyclic inspection and, if necessary, repair work, it is natural to think that the maintenance cost will be saved. This is the reason why MLITT changed policy and started the preventive maintenance strategy.

3. Design issues

3.1. General

As explained in chapter 2, since new bridge construction business is now shrinking, the competition between steel and concrete alternatives is becoming severe and severe. To cope with subject, the development and proposition of steel-concrete hybrid (composite or mixed) bridges in both steel and concrete sides is active.

Fig.3 and Fig.4 show steel I-girder and box girder bridges with a very simplified transverse stiffening system, which are alternatives proposed by bridge engineers belonging to Nippon Expressway Company Ltd. (NEXCO; former Japan Highway Public Corporation). Among these, composite or non-composite two I-girder bridges are evaluated to be the most competitive alternative for bridges with a span from 40 to 60 meters. Fig.5 shows the total construction number of these types of bridge. From this figure, it is seen that the increase of I-girder bridges is prominent. Fig.6 is PC box girder bridges with steel corrugated web or steel pipe truss web. When the span length less than 40 meters, and exceeds 60 or 70 meters, PC bridges are very competitive. In order to compete with long-span concrete bridges, steel bridge engineers are now proposing a double-composite girder bridge and cable-stayed composite girder bridges, however, they have not been realized so far.

Our proposition to enhance competitiveness is to establish a happy collaboration of the following three items, such as 1) Introduction of LSD for steel-concrete hybrid bridges, 2) In addition to newly developed conventional two I-girder bridges (see Fig.3), introduction of new structural type such as double-composite girder (see Fig.7) and hybrid girders (see Fig.8) and 3) Utilization of HPS for thick plate such as yield point constant steel, low pre-heating steel and so on. Since my colleagues are going to talk about hybrid bridges in detail, in this chapter, I focus on the design code making activity at the Committee of Steel

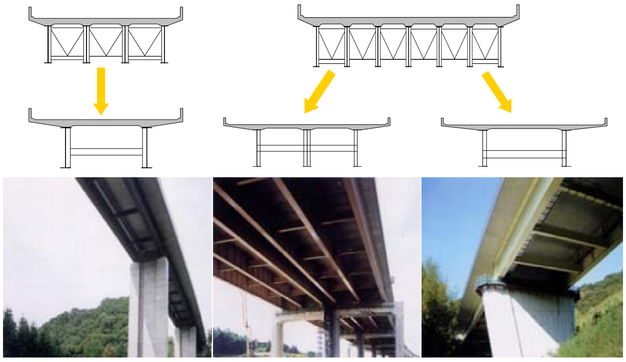


Figure 3. Newly developed steel I-girder bridges

Figure 4. Newly developed narrow-width box box girder bridges

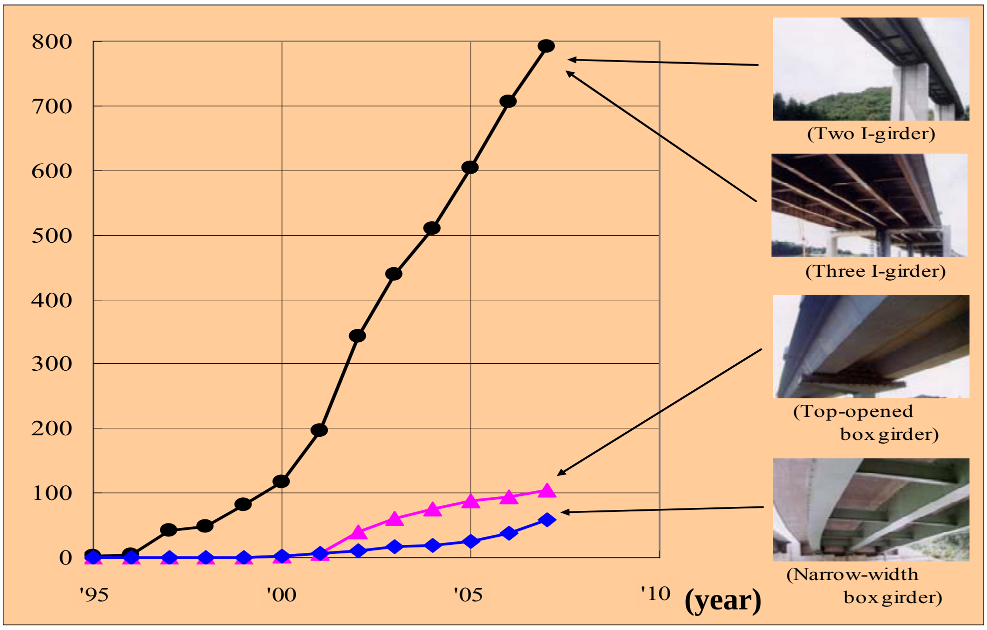


Figure 5. Construction number of newly developed steel girder bridges

a) Steel corrugate web b) Steel truss web

Figure 6. PC box girder bridges with steel webs

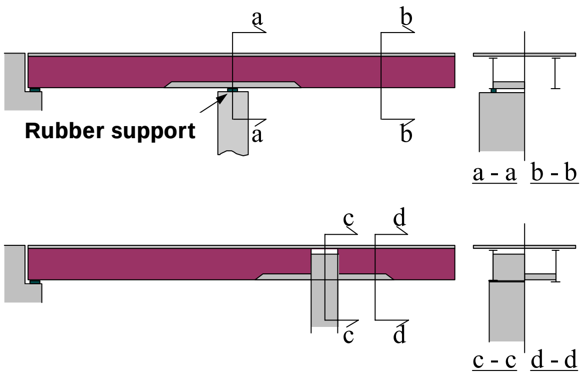


Figure 7. Double-composite girder bridges

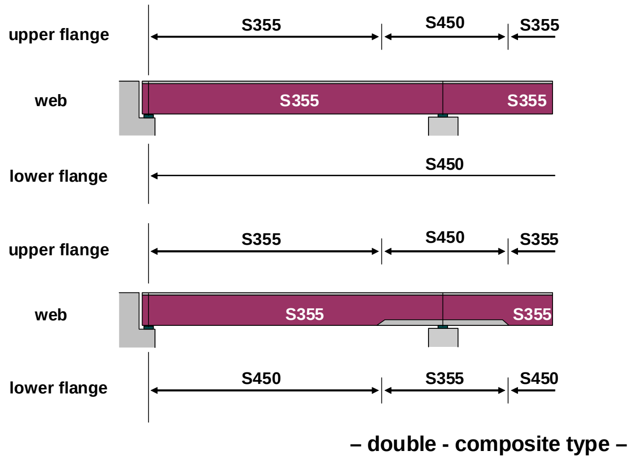


Figure 8. Hybrid (section) girder bridges

Structures, one of technical investigation committee organized in Japan Society of Civil Engineers (JSCE) and the benefit from LSD.

3.2. Code making activity in JSCE

In Japan, we have a legal design code (Japan Road Association, 2003) for steel bridges issued from Japan Road Association, which has been based on Allowable Stress Design (ASD) method for more than 35 years. The top cover of it is given in Fig.9. The revising work of it is now under way, and it had been announced that LRFD method will be employed. However, regardless of composite or non-composite girders, maximum flexure strength will be limited less than yield stress or yield moment.

Fig.10 shows top cover of two volumes issued from the Committee of Steel Structures of JSCE. One is the design guideline for steel structures in general (PART-A) and the other is that for composite structures (Part-B). Both were issued in 1987, and revised versions were issued in 1997 (The Committee of Steel Structures, 1997a, 1997b). In 1997 version, LSD format were introduced. Unfortunately, the provisions prepared in PART-A has not been used at the practical design stage. The provisions in PART-B, which are related to concrete slab design has been utilized at the practical design stage.

The committee of Steel Structures organized the sub-committee on Standard Specifications for Steel and Composite Structures in 2004 (current chairman is Prof. Nagai), and issued first volume (The Committee of Steel Structures, 2007) including "General provision", "Basic Planning" and "Design" in 2007. This is the first time publication on Performance–based LSD in civil steel structural engineering in Japan. Its English version is now being prepared and it will be released soon. In 2008, volume of "Seismic design" was issued (The Committee of Steel Structures, 2008), and Fig.11 shows top covers of two volumes. The volume of "Construction" was just issued in July, 2009 (The Committee of Steel Structures, 2009), and final volume of "Maintenance" is expected to issue in this financial year of 2009.

3.3. Consideration on benefit from introduction of LSD for hybrid bridges

EC (CEN, 2003, 2004) employed LSD and it is thought that most of major work for bridge design code was finished. In EC, the required or specified performance such as safety and serviceability is checked and ensured by employing partial factor format. AASHTO LRFD (AASHTO, 2005) is still partly under improvement. The required performance at SLS (Serviceability Limit State) and ULS (Ultimate Limit State) is checked and ensured by LRF (Load and Resistance Factor) format, so that it has been called AASHTO LRFD.

The following is typical ASD format ensuring safety against instability.

$$\sum\sigma \le h\left(\sigma_{ult}/\gamma[=1.7]\right) \tag{1}$$

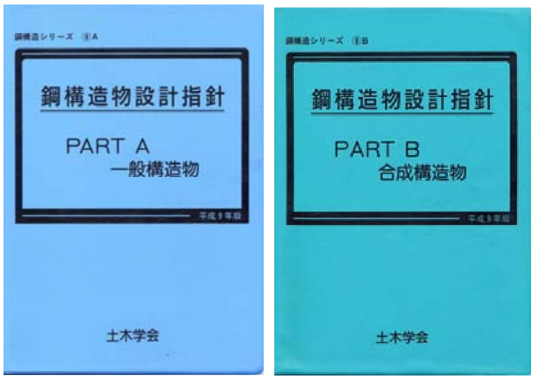
$$\sigma_{ult} = \min.\left(\sigma_y, \sigma_{buckling}\left[\le \sigma_y\right]\right) \tag{2}$$

Where, $\sum\sigma$ is the sum of stress under specified combination of actions such as loading, settlement, imposed deformation and so on, $h(\ge 1.0)$ is a coefficient taking into account of occurrence probability of combined actions and γ is basic safety factor of 1.7.

From eq.(2), it is noticed that maximum strength is σ_y (yield strength of material), because the buckling strength ($\sigma_{buckling}$: column buckling, beam lateral or lateral torsional buckling, plate local buckling and so on) has been defined not to exceed σ_y.

In EC and AASHTO LRFD, beam sections have been classified into 3 or 4 categories depending on thier flexural strength. In both codes, the section whose flexural strength is equal to or less than yield moment (M_y) is classified as "slender section". In this case, the ultimate flexural strength defined in ASD and LSD is the same. If the partial factor is assumed to be the same (the same safety factor is assumed), no drastic change of the cross-sectional size or dimension is predicted depending on the design method (ASD or LSD).

Figure 9. Top cover of Japan Highway Bridges Specifications (JHBS)

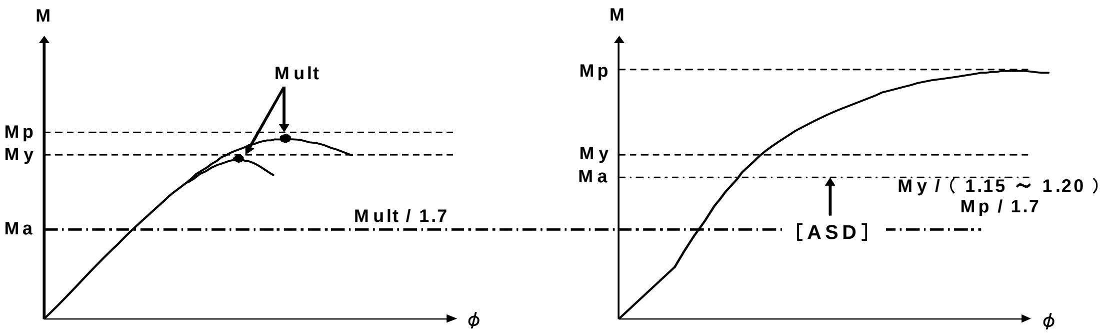


a) PART-A b) PART-B

Figure 10. Top covers of LSD codes issued from JSCE

a) General provision, Basic planning and Design b) Seismic design

Figure 11. Top covers of Standards Specification for Steel and Composite Structures issued from JSCE



a) Steel girder b) Composite girder

Figure 12. Schematic expression on the difference between ASD and LSD

The following is typical formulae checking whether or not the required performance is satisfied in AASHTO LRFD.

At SLS

$$\sigma_{1.0D+1.3L} \le \sigma_y \quad (3)$$

$$\text{or} \quad M_{1.0D+1.3L} \le M_y \quad (3)'$$

At ULS

$$M_{1.25D'+1.50DW+1.75L} \le M_{ult.} \quad (4)$$

Where, D is the dead load, L is the live load including impact effect, D' is the dead load excluding pavement (surface layer of concrete slab), DW is the load from pavement.

In eq.(4), when $M_{ult.}$ is defined to be equal to or less than M_y (classified into "slender section"), since the load factor in eq.(4) is larger than that in eq.(3)', it is easily understood that the cross-sectional size will be controlled by eq.(4) (at ULS).

Since non-composite welded-type plate girder has normally thin web, in order to attain plastic moment, appropriate web stiffening is inevitable to prevent local buckling of the web after yielding. The employment of thick web is one choice. However, too thick web will result in less competitiveness (uneconomical). Even though the labor cost increases, welding many horizontal stiffeners to thin web subjected to compressive force is another choice. However, due to thin web, contribution of plastic moment from web will be minor, so that the difference between M_y and M_P of the girder will of be minor. From this, it is easily understood that plastic design for thin-walled and welding-type plate girder bridges will not make sense. In case that plate girder consisting of rolled section (web with small depth-to-thickness ratio, and the section is classified into "compact section"), plastic design will make sense. Furthermore, the ultimate flexural strength of composite girder under sagging bending moment is plastic moment (M_P), which is around 1.5 times yield moment (M_y), so that plastic design also makes sense for composite girder under positive bending moment, and benefit from LSD will be promising.

In eq.(4), if DW is assumed to be 20% of total dead load (D) and $M_{ult.}$ is set to be 1.3M_y (maximum $M_{ult.}$ is limited to be less than 1.3M_y according to AASHTO LRFD (AASHTO, 2005)), it can be rewritten as follows,

$$M_{1.25(\times 0.8D)+1.50(\times 0.2D)+1.75L} \le 1.3M_y$$

$$M_{1.3D+1.75L} \le 1.3M_y$$

$$M_{1.0D+1.35L} \le M_y \qquad (5)$$

Comparing eq.(3)' and eq.(5), it is known that both give nearly the same criteria.

If we rewrite eq.(1) considering that the bridge is subjected to dead and live loads (h = 1.0), the followings are given.

$$\sigma_{1.0D+1.0L} \le \left(\sigma_y/1.7\right) \qquad (6)$$

$$\text{or} \quad M_{1.7D+1.7L} \le M_y \qquad (6)'$$

Comparing eq.(3)' or (5) (AASHTO LRFD) and eq.(6)' (ASD), in case that the composite girder is subjected to sagging bending moment, it is known that the larger cross-sectional size is necessary by eq.(1) (ASD), resulting in less competitiveness. In Fig.12, schematic expression on the above explained difference is given.

4. Maintenance issues

4.1. General

As explained in Chapter 1, instead of new construction business, maintenance business is expected to increase, and the development of new technology for maintenance work is now strongly requested. Herein, among maintenance related matters, SHMS, redundancy analysis, weathering steel, and CFRP bonding are explained, which is research topics being dealt with in my laboratory.

4.2. SHM

The collapse of the I-35W interstate bridge in Minnesota, United States in August 2007 shocked our communities. Recently, also in Japan, steel member fractures were found in the Kisogawa Ohashi Bridge (Fig.13(a)), Mie Prefecture, in June 2007 and in the the Honjo Ohashi Bridge (Fig.13(b)), Akita Prefecture, in August 2007. Although these incidents did not lead to collapse, it is pointed out not to have conducted enough frequent inspection as a problem.

Before local governments plan highway roads repairs, they are needed to understand the integrity of existing bridges. Although central government offer technical supports such as lectures or guidelines to local governments not having inspection manuals, herein, as one way to catching the current state of bridges, advanced structural health monitoring systems using laser devices are introduced.

The device introduced here is a Laser Doppler Vibrometer (LDV). This is an optical instrument employing laser technology to measure velocity (Fig.14). In comparison with conventional transducers such as accelerometers, LDV makes possible to conduct non-contact and long distance measurement without adding mass or stiffness to an object. And also, resolution of velocity is very high, and frequency bandwidth is broad. Furthermore, by attaching a scanning unit on a laser sensor head, measurement on multiple points can be realized.

4.2.1. Monitoring of a Shinkansen Steel Box Girder Bridge

Fatigue cracks were observed on several steel box girder bridges that support the high-speed train networks in Japan. These cracks appeared at the webs of the bottom end of the vertical welded stiffeners. In order to investigate dynamic characteristics of the bridges, field measurement of a selected bridge using LDVs was conducted (Miyashita T. et. al., 2005).

Measurement system consists of three scanning type LDVs and one single point type LDV (Fig.15). The single point type LDV acquires reference signal that is used to calculate phases between measurement points in order to identify mode shapes. Ambient and train-induced vibrations were measured before and after retrofit measures at the bottom part of the vertical stiffeners. As compared with conventional measurement works (Fig.16), this system can greatly reduce time consuming task such as wiring cables.

Fig.17 shows identified mode shapes from the ambient vibration measurement. Figs.17(a), (c) and (b), (d) show the mode shapes before and after retrofit respectively. It is found that mode shapes are greatly different such as shown in Figs.17(a), (b) although natural frequencies of both modes are almost same.

4.2.2. Monitoring of Shinkansen Concrete Viaducts

Integrity evaluation of Shikansen RC viaducts has been conducted by the impact test using 30 kg mass. Global

a) The Kisogawa Ohashi Bridge

b) The Honjo Ohashi Bridge

Figure 13. Recent steel member fractures in Japan

a) Single point type

b) Scanning type

Figure 14. Laser Doppler Vibrometer (LDV)

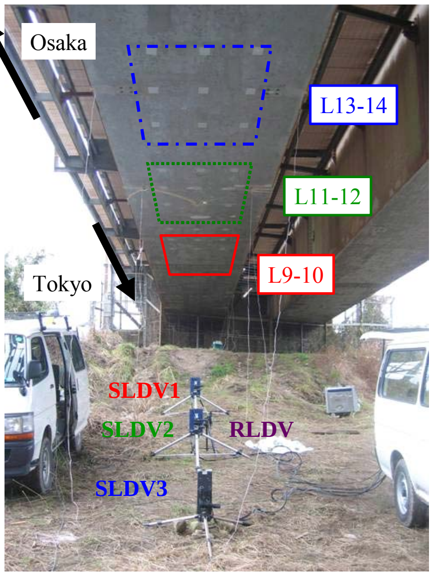


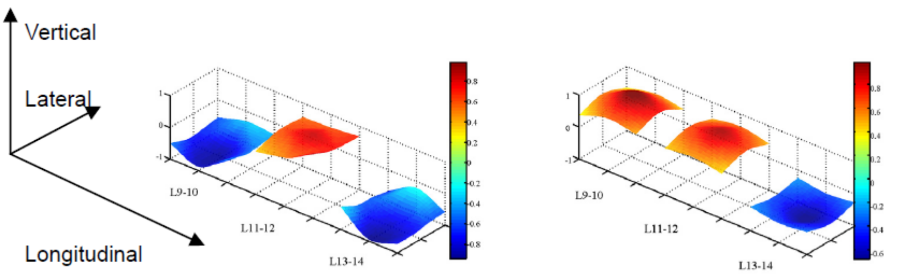
Figure 15. Measurement system using LDVs

Figure 16. Conventional measurement works

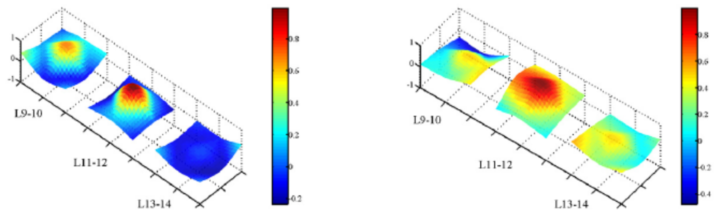
1st natural frequency of the viaduct and local one of a column are utilized for the evaluation. However, there are several demerits in this method. It requires time consuming dangerous works at high level. And also, it is sometimes difficult to access the viaduct itself after disaster such as earthquake. Therefore, a new monitoring technique using LDVs in order to replace conventional technique was investigated (Hernandez J. Jr., et. al., 2006).

Due to large stiffness of RC viaduct, amplitude level of ambient vibration becomes very low. Moreover, vibration of a LDV's tripod resulting from external disturbances such as ground vibration affects measurement accuracy. In order to eliminate external disturbances, a high accurate servo type velocimeter was put on the LDV. The velocimeter measures the vibration of the LDV itself. In analysis, velocity measured by the velocimeter is subtracted from LDV results.

During on-site measurements, a total of 11 points on the surface of an RC viaduct were selected: two per column and three on the main beam. Fig. 18 shows measurement system on the field. The system was set up at about 30 m from the viaducts. Once all measurement point coordinates were measured by scanning system, vibration measurements were carried out continuously. After eliminating the effect of external disturbances, global mode shapes of a viaduct were identified using peak-picking method. Identified natural frequencies of the viaduct were 2.833Hz, 2.933Hz and 3.033Hz. Fig.19 shows identified mode shapes coinciding with each natural frequency. Although the mode shape at 3.033Hz corresponds to the one in lateral direction, it is difficult to interpret the mode shapes at 2.833Hz and 2.933Hz. Detailed investigation revealed that they were a combination of torsional and lateral modes. This can be explained by dynamic structural interaction between adjoining viaducts due to the presence of the continuous rails.



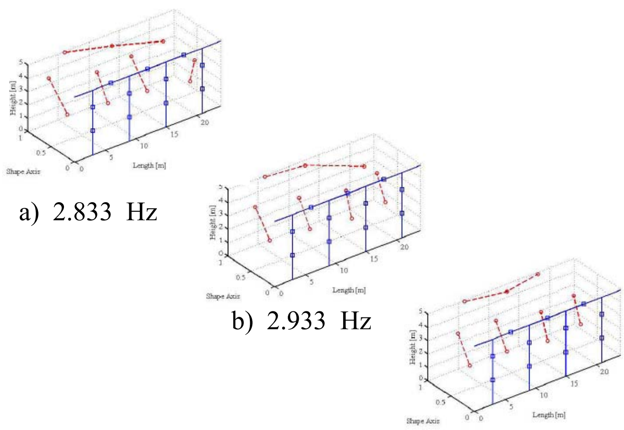
(a) Before retrofitting (29.35 Hz) (b) After retrofitting (29.35 Hz)



(c) Before retrofitting (32.75 Hz) (d) After retrofitting (33.35 Hz)

Figure 17. Mode shapes of three lower flanges

Figure 18. Vibration measurement of concrete viaducts



a) 2.833 Hz

b) 2.933 Hz

c) 3.033 Hz

Figure 19. Global mode shapes of a viaduct (□: Measurement points, ○: Mode shapes)

4.2.3. Remote Non-contact Monitoring System for Long-span Bridges

When a measurement point is far away from a LDV, it is difficult to hit laser beam to the point on-site. In order to improve this difficulty, a remote non-contact monitoring system (Fig.20), which is the combination of a LDV and a total station (TS) for surveying, is under development (Miyashita T., et. al., 2007).

This system makes possible to conduct automated remote vibration measurement continuously due to internal storage of the coordinates of measurement points and mechanical rotation of the LDV together with the TS. Previous investigation revealed that the utilization of prisms for surveying at measurement points extended measurement distance drastically. One of the effective applications of this system is to measure tensile forces of many cables in a long-span bridge since tensile forces of cables are usually identified from their natural frequencies by vibration measurement, which requires time and cost for setting up of equipments.

In order to check the effectiveness, measurement test was carried out at the Tatara Bridge. The outline of the test is shown in Fig.21 and Table 1. The maximum measurement distance was about 970m. The test started at 15:00 PM, and was repeated 13 times at each measurement point until 9:00 AM on the next day.

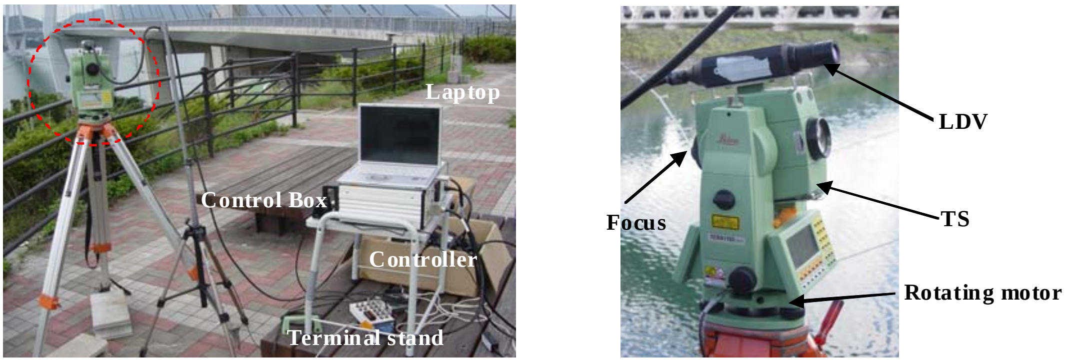


Figure 20. Remote non-contact monitoring system

Table 1. Measurement points and distances

	Point	Distance [m]
1	C1	50.844
2	C11	124.865
3	C21	278.383
4	P	302.288
5	G1	390.968
6	G2	525.719
7	C32	529.526
8	C42	725.841
9	G3	800.252
10	G4	970.857

Table 2. Identified 1st natural frequencies

Point	Member	1st Natural Frequency [Hz]	
		Previous result (Accelerometer)	2006.9 (LDV)
C1	cable	0.371	0.373
C11	cable	0.496	0.495
C32	cable	0.415	0.417
G3	girder	0.102	0.103

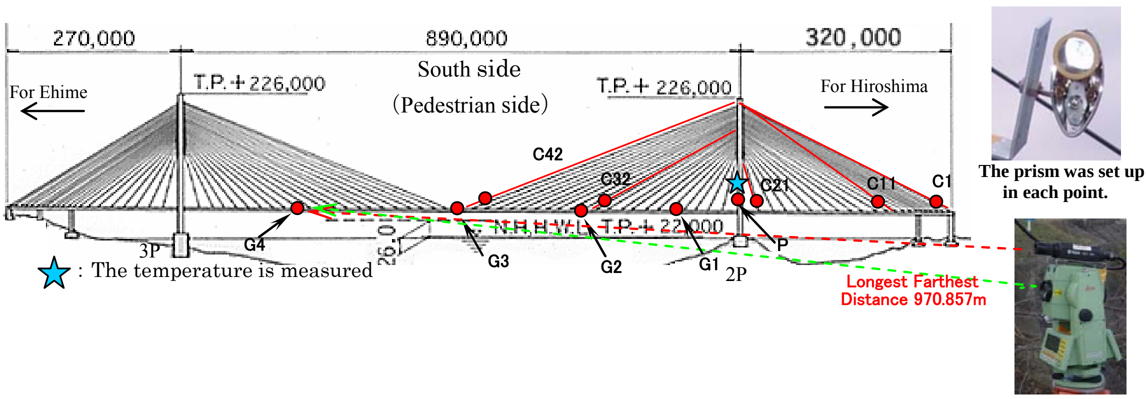


Figure 21. Measurement at the Tatara Bridge

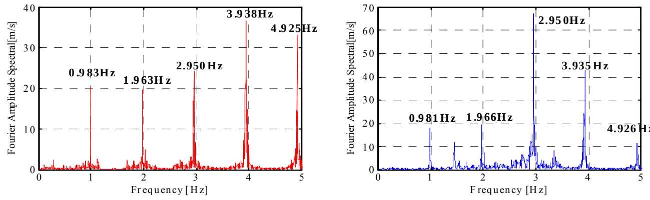
Identified 1st natural frequencies from ambient vibration by this system are compared with previous identified results as shown in Table 2. Fig.22 shows Fourier spectral amplitudes of LDV and servo-velocimeter at a cable (C21). Both frequency components show good agreement.

Then, in order to ascertain the maximum measurement distance, vibration of a steel plate located 2 km away from the system was measured (Fig.23). A prism for reflection of laser beam and a servo-velocimeter for validation were attached on the surface of the plate. Since LDV result is in good agreement with servo-velocimeter one, this system is expected to apply continuous vibration monitoring of the whole long-span bridges.

4.3. Redundancy analysis

In Japan, the number of old bridges is increasing, and a half of the existing bridges will exceed 50 year old in twenty years or so. However, since the lack of financial budget, most of bridges except ones belonging to MLITT, which is a central government, and expressway company Ltd., no inspection has been made.

After the collapse of a truss bridge in I-35W interstate bridge in Minnesota, USA, two trends happened in Japan. One is redundancy analysis and another is, instead of replacement, the preventive maintenance (cyclic inspection, repair and strengthening if necessary)



(a) LDV (b) Servo-velocitimeter

Figure 22. Fourier spectral amplitudes at C21

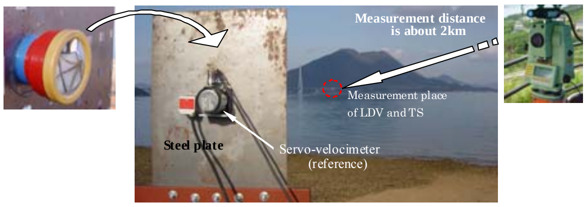


Figure 23. Long distance measurement test

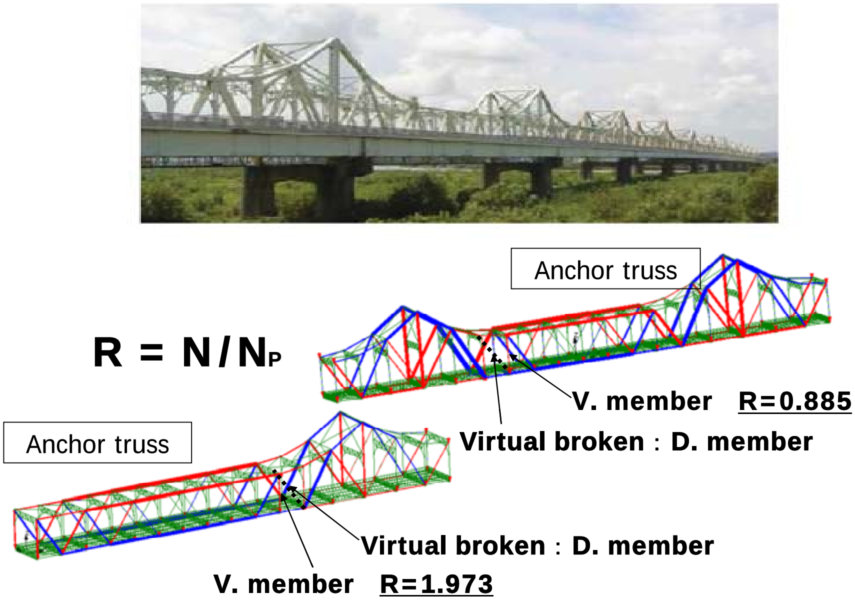


Figure 24. Result of redundancy analysis using an existing truss bridge

towards longer life. Replacement after 50 or 60 year life span will be very much expensive. In order to save money, MLITT decided to change policy and to carry out preventive inspection and repair cyclically. MLITT ordered local government to inspect and check the performance (behavior) all of existing bridges, and to prepare repair plan and to estimate cost for the purpose. In several years, such procedures will be finished.

Another topic is the redundancy analysis of a truss bridge. The influence of removing of one member on global stability is analyzed based on FEA. Fig.24 shows one result of analysis of the existing 73-year old bridge. By removing one member, whether or not the remaining members exceed the ultimate strength is checked. After carrying out the removal of all members, the most influential member is identified, and it is utilized to determine the priority for monitoring.

4.4. Weathering steel

In the competition of bridge alternatives, LCC evaluation has been made. In case of steel bridges, the cost for painting is counted, and the cycle is assumed to be around 10 years. It means, when the 100-year life span is set, 10 times repainting is made. In order to cope with this subject, long-life painting system, such as 30-year repainting system, has been developed. However, Even though it sounds strange, in reality, since the maintenance cost of concrete bridges is evaluated to be zero, the concrete bridge is competitive from a viewpoint of LCC evaluation. As a countermeasure, the weathering steel is highlighted.

Stabilized corrosion layer delays the further corrosion of steel plate. Hence, whether or not such layer is produced is the crucial problem, and it strongly depends on climate or atmosphere at construction site. In Japan, normally, when the flying salt volume is less than 0.02mmd/year, the weathering steel can be adopted. In order to enhance the reliability of evaluation on performance of weathering steel, the study is being continued in Japan. Fig.25 is measurement of the salt density. Fig.26 is analytical approach, which is FE mesh division to predict the salt distribution in steel plate girder bridges

4.5. CFRP bonding for repair of corroded steel members

Carbon Fiber Polymer Plastic (CFRP), one of new materials, has been drawn attention in the field of bridge engineering, in particular, for the repair of damaged members and for keeping ductility of concrete piers. Herein, the repair of corroded steel members by CFRP

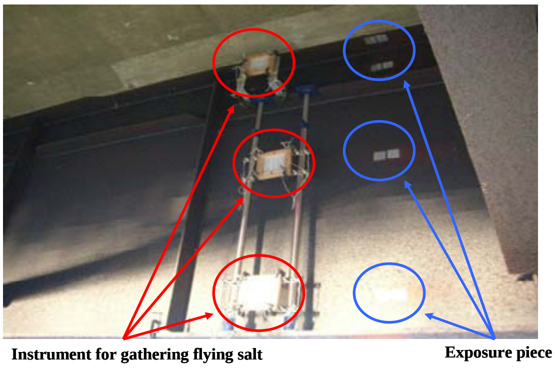


Figure 25. Measurement of flying salt

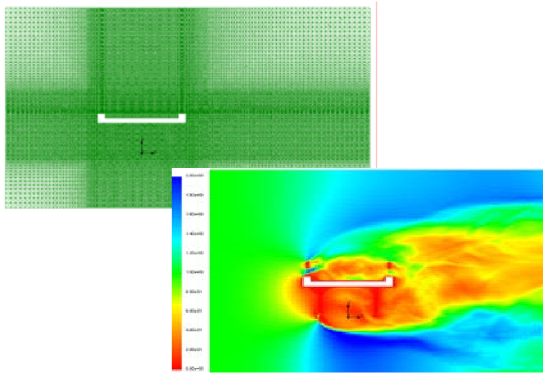


Figure 26. FEA under flying salt

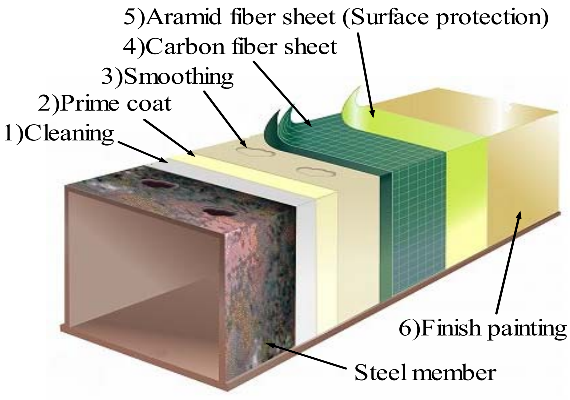


Figure 27. Proposition of CFRP bonding to corroded steel plates

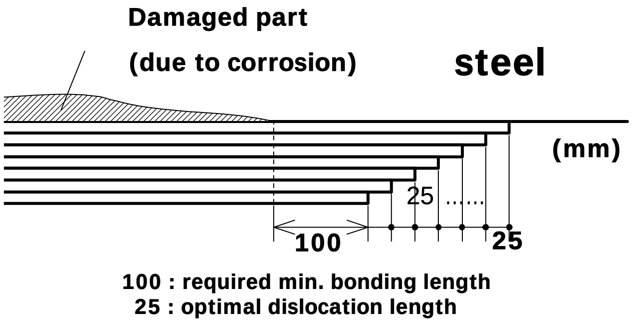


Figure 28. Proposed adhesion manner

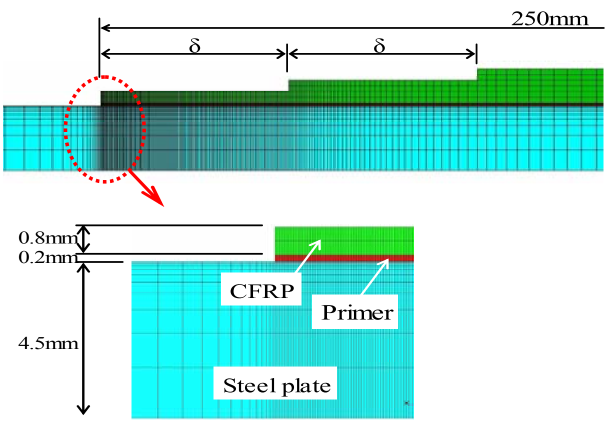


Figure 29. FEA of steel plate with CFRP bonding

a) Hand working

b) Finished

Figure 30. CFRP bonding at actual truss bridges

bonding is introduced.

Based on our extensive analytical and experimental studies, we proposed design method of CFRP bonding. Fig.27 shows working procedure proposed. The required CFRP effective cross-sectional area (required number of sheet), required bonding length and optimal dislocation length of each sheet were also presented.

One of important subjects is to detect correctly and also to increase CFRP peeling load. Fig.28 is our adhesion manner of CFRP sheet. The minimum required bonding length is 100 millimeters and the optimal dislocation length of each sheet is 25 millimeters. 25-millimeter is obtained in order to mitigate most efficiently the shear stress concentration, and is based on FEA (Sugiura H., et. al., 2008) (see Fig.29). We are now continuing the test to detect and also enhance the peeling

Figure 31. Asarigawa bridge on the route of Chuo Highway

load.

Fig.30 shows CFRP bonding to an actual truss bridge given in Fig.31, which has corroded members, located on the expressway route connecting Tokyo and Yamanashi-city with heavy traffic. The effectiveness of CFRP bonding was checked through at-site measurement, and the strain reduction due to bonding was surely confirmed (Sugiura H., et. al., 2009).

5. Concluding remarks

Two topics related to the design and maintenance issues were dealt with. In Japan, for more than 35 years, ASD has been employed, and this is completely different from the style of EC and AASHTO LRFD. Activity toward LSD in the committee of Steel Structures, one of technical investigation committees organized in JSCE, was introduced. At the design of steel-concrete hybrid bridges, the importance of design based on LSD was also emphasized. It is explained that new bridge construction business has been shrinking and, on the contrary, that deteriorated old bridges have been increasing. In this situation, new technological development for maintenance work is becoming important. Among new technologies, structural health monitoring (SHM) technology, redundancy analysis related to maintenance strategy, weathering steel and CFRP bonding to corroded steel members were explained.

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