No. 45 August 2015

# STEEL CONSTRUCTION TODAY & TOMORROW

http://www.jisf.or.jp/en/activity/sctt/index.html

構

【 *k o u* 】

"構 (*kou*)" in Japanese, or structure in English

The usefulness of steel structures is growing in the construction of bridge and other civil engineering structures. This issue, No. 45, covers the research attainments presented at the 19th Symposium on Research on Civil Engineering Steel Structures with a main theme of "Towards Stronger, Longer-lasting Steel Bridges."

## Special issue: 19th Symposium on Research on Civil Engineering Steel Structures

1 Towards Stronger, Longer-lasting Steel Bridges Conducive to Building National Resilience and Promoting Large-scale Retrofitting of Existing Bridges

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Published Jointly by

The Japan Iron and Steel Federation

Japanese Society of Steel Construction

## 19th Symposium on Research on Civil Engineering Steel Structure

# Towards Stronger, Longer-lasting Steel Bridges Conducive to Building National Resilience and Promoting Large-scale Retrofitting of Existing Bridges

The Japan Iron and Steel Federation (JISF) held its 19th Symposium on Research on Civil Engineering Steel Structures on March 10, 2015 in Tokyo. In 1995, JISF established a “subsidy system for steel-structure research and training” and since then has provided subsidies to researchers working in the field of steel structures. The symposium has been held every year with the aim of publicizing the results of research supported by the subsidy system while at the same time promoting wider applications of steel structures.

As a link related to the research activities promoted by the subsidy system, the research promotion organization was established in 2013 and 2014—Research Committee on Improvement of Structures and Durability of Steel Bridges and three affiliated ad-hoc committees, Working Group on Structures and Design Innovations of Steel Bridges, Working Group on Fatigue Strength of Steel Bridges and Working Group on Maintenance of Weathering Steel Bridges.

In recent years, the importance of technology that enhances the overall excellence of steel bridges in terms of cost, performance and quality has grown. Meanwhile, another bridge performance factor that is gaining importance is minimization of lifecycle costs (LCC), such as mitigation of maintenance costs and prolonging service life. In order to meet such requirements, the Research Committee extensively promotes related research activities and organizes the diverse attainments that are derived from these activities.

At this year’s 19th Symposium on Research on Civil Engineering Steel Structures, the attainments of this research were reported and related lectures were delivered to the approximately 380 engineers and researchers who attended. The titles of the lectures and reports and the names of the lecturers at the 19th symposium are shown in the table below.

This issue, No. 45, of *Steel Construction Today & Tomorrow* is published as a special issue spotlighting the symposium; an outline of the lectures and reports delivered at the symposium is introduced in the following pages. ■

**A List of Lectures and Reports of Research Attainments Presented at the 19th Symposium on Research on Civil Engineering Steel Structures**

| | |
|---|---|
| *Keynote lecture:*<br>Current State and Large-scale Renewal of Metropolitan Expressway | Kenichi Ando<br>Senior Executive Officer, Metropolitan Expressway Company Limited |
| *Report of research attainments:*<br>(1) Rational Performance Evaluation and Strength Design for Steel Bridges<br>(2) Fatigue Characteristics of Weld Joints Employing SBHS<br>(3) Maintenance of Weathering Steel Bridges | Yoshiaki Okui<br>Professor, Saitama University<br>Kazuo Tateishi<br>Professor, Nagoya University<br>Eiki Yamaguchi<br>Professor, Kyushu Institute of Technology |
| *Special lecture:*<br>“Building National Resilience” Initiative and Future Directions | Satoshi Fujii<br>Professor, Kyoto University; Special Adviser to the Cabinet, Cabinet Secretariat |


JISF held its 19th Symposium on Research on Civil Engineering Steel Structures in March 2015.

# Current State and Large-scale Renewal of Metropolitan Expressway

***by Kenichi Ando***
***Senior Executive Officer***
***Metropolitan Expressway Company Limited***

**Kenichi Ando**: After graduating from the School of Engineering, Waseda University, he entered Metropolitan Expressway Public Corporation in 1974. After serving as director of Planning and Environmental Dept. and Construction Dept. and director general of Tokyo Construction Bureau, he assumed his current post as senior executive officer in 2013.

### Outline of the Metropolitan Expressway Network

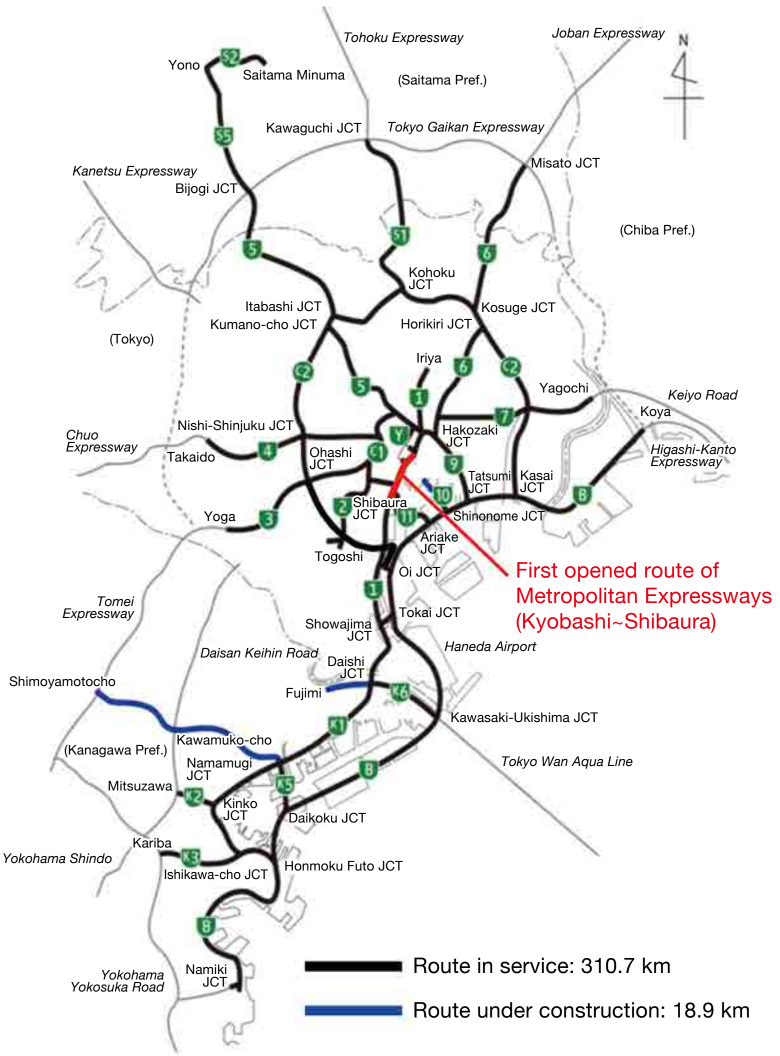
About 50 years have passed since the opening of Metropolitan Expressway, a network of arterial expressways linking metropolitan Tokyo and its surrounding areas. The first section of the expressway, extending 4.5 km between Kyobashi and Shibaura in downtown Tokyo, opened in December 1962 (Fig. 1). The daily traffic volume at that time was 11,000 vehicles, and the toll for ordinary vehicles was ¥100 (¥50 for the first year of operation).

Most of Japan's roads in the 1950s were unpaved and turned to mud when rain fell, sometimes requiring vehicles to be pushed through the mire. This situation demonstrated that road improvements in Japan were long overdue. To remedy this, two systems were initiated—a toll road system (1952) in which drivers were charged a toll fee to cover the cost of road construction and a revenue source earmarked for roads (1953).

The motorization of Japan progressed rapidly from around 1955 and brought with it deteriorating highway traffic conditions. In response, it was necessary to construct the expressway for exclusive use with automobiles. Then, it was decided that the network of expressways in metropolitan Tokyo be improved and, in June 1959, Metropolitan Expressway Public Corporation (former Metropolitan Expressway Company Limited) was established. In May of the same year, the International Olympic Committee selected Tokyo to host the 1964 Olympic Games.

Today, the daily traffic volume of Metropolitan Expressway is about 950,000 vehicles. The ratio of the total length of Metropolitan Expressway to that of national and Tokyo metropolitan roads in 23 wards is barely 15%. However, the ratio of Metropolitan Expressway is about 36% in terms of vehicle-kilometers and about 28% in terms of freight traffic volume—or more than twice all national and metropolitan roads in Tokyo in terms of vehicle-kilometers and freight traffic volume. This means that the Metropolitan Expressway is indispensable for the daily routines of the residents of Tokyo and its surrounding areas.

**Fig. 1 Network of Metropolitan Expressway**




### Network Development of and Improvement Plan for Metropolitan Expressway

On May 7, 2015, the final section of the Central Circular Route (Bay Shore Route~Route 3 Shibuya Line) was opened to traffic, extending the total length of routes in operation in the Metropolitan Expressway network to 310.7 km. Another 18.9-km route is currently under construction and, when completed, it will extend the total to about 330 km.

The Central Circular Route required more than a half century to go from initial route investigation to completion. The route's 18.2-km Yamate Tunnel is the longest road tunnel in Japan and it was built utilizing a variety of advanced construction technologies, among which were a method to expand shield tunnel width underground (Fig. 2) and a cutting shield tunnel construction method that could integrate two or more shields underground. Prior to that, ordinary roads were open-cut, traffic had to be closed and lanes regulated, and branch sections and exit/entry ramps were constructed by means of the open-cut method. However, use of these two underground methods has made it possible to suppress the effects on ordinary roads to a minimum.

The Ohashi Junction is a typical example of a junction constructed within a narrow space. In this junction structure, the lower section which begins about 35 m below ground connects with the upper section which ends about 35 m above ground. In the junction that connects the difference of elevation of about 70 m, the maximum grade was designated at 7%, and thus a 1,000-m distance was necessary for the vehicle to climb the 70-m difference of elevation. One effective solution that was devised to meet the requirement was to provide a 400-m track in order for the vehicle to reach the junction's upper section through 2.5 rotation running of track. Photo 1 depicts the Ohashi Junction.

As an environmental measure, a large cap-shaped structure was built over the junction and now serves as a park in Meguro Ward (Photo 1). Further, in order to meet the request of residents displaced by the junction to return to the site, the Tokyo Metropolitan Government built two high-rise buildings nearby as an urban redevelopment project. The displaced residents could move into these buildings after the junction was completed.

We believe that the Ohashi Junction project will serve as a model case for future road construction and urban development. The project has received the Good Design Award, a prominent award given for excellent design performance, as well as many other public recognitions.

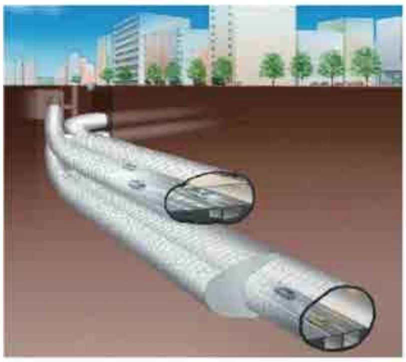
## Large-scale Renewal and Repair of Metropolitan Expressway

The structures of Metropolitan Expressway network are getting old. Of the network's total length of about 310 km, around 30% of the expressway structures have been in service more than 40 years and about 50% have been in service 30 years (Fig. 3). Further, the tunnels and viaducts that require minute maintenance account for about 95% of all expressway structures. On top of this, large-vehicle traffic volume is high.

Given this situation, the Metropolitan Expressway Company established the Research Committee Tasked with Reviewing Approaches to Large-scale Renewal of Metropolitan Expressway Structures (chaired by Professor Shiro Wakui of Tokyo City University). Based on the proposals submitted by this committee in January 2013, the company extended its deliberations and publicly announced the renewal plan in December 2013. Five sections of the expressway were determined to be in need of renewal and reconstruction: Higashi-Shinagawa Wharf~Samezu Landfill section, Daishibashi section, Ginza~Shintomicho canal section, Takebashi (incl. Nihonbashi)~Edobashi section, and Ikejiri~Sangenjaya section (Fig. 4). The renewal cost will amount to about ¥380 billion. Further, large-scale repairs are also planned for the 55-km section at a cost of about ¥250 billion. The total cost for the planned renewal and repair projects will be about ¥630 billion.

When the Tokyo OlympicGames were held in 1964, a 30-km section of what is today's expressway network was in service. That route is now in need of renewal. (Fig. 5)

**Fig. 2 Underground Shield Tunnel Width Expansion Technology**



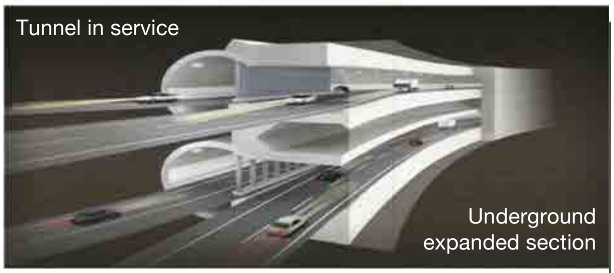
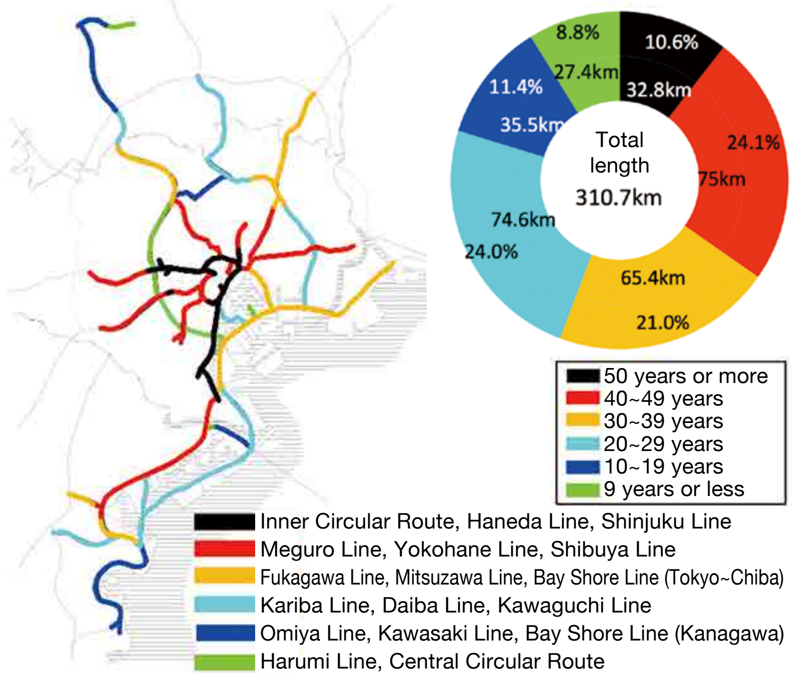



Photo 1 Full view of Ohashi Junction

**Fig. 3 Lapse of Years since Opening of Expressway Lines (as of April 2015)**



Of the sections slated for renewal, procedures for concluding the construction contract are underway for the Higashi-Shinagawa Wharf~Samezu Landfill section. This section is to be rebuilt to a viaduct structure higher from the sea level in order to improve durability and maintenance performance. A viaduct structure equipped with permanent scaffolding is being examined with the aim of allowing constant inspection work to be made. The current daily traffic volume of this section is about 80,000 vehicles, which makes traffic closures for renewal work unfeasible. Therefore, a new two-lane detour will be constructed for use during the renewal work. (Fig. 6)

Renewal of the Higashi-Shinagawa Wharf~Samezu Landfill section of the Route 1 Haneda Line will be underway during the 2020 Tokyo Olympic Games. In order to secure safe and smooth vehicle traffic during that period, Metropolitan Expressway Company is studying a construction procedure in which a temporary 2-lane detour and 2 lanes on the new viaduct will be put into service so that the damaged section need not be used at least (Fig. 6). Steady efforts are being made to start this renewal work within FY2015. ■

**Fig. 4 Expressway Sections subject to Large-scale Renewal**

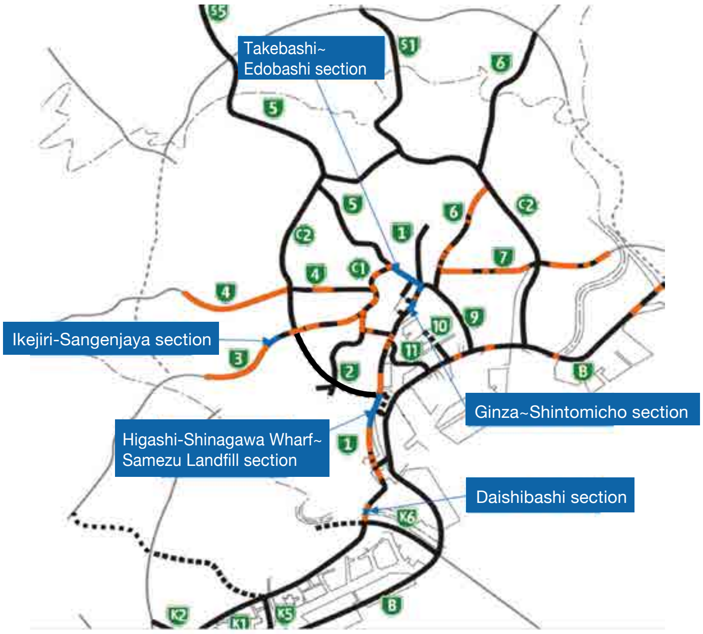



**Fig. 5 Expressway Sections Opened to Traffic in Tokyo Olympic Games 1964 and Location of Large-scale Renewal Sections**

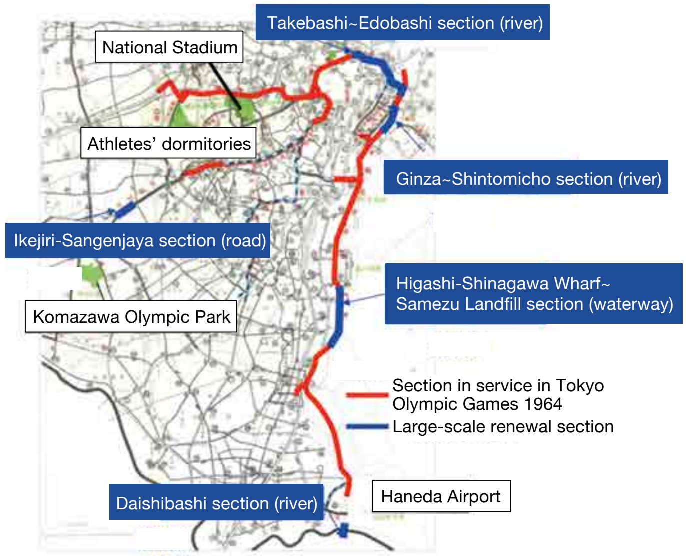
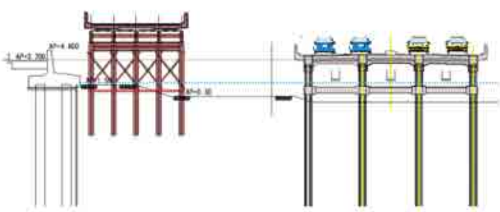



**Fig. 6 Image of Renewal of Higashi-Shinagawa~Samezu Landfill Section**

Present state

After large-scale renewal (image)

Temporary
detour



Detour under construction (image)

# Rational Performance Evaluation and Strength Design for Steel Bridges

**Yoshiaki Okui:** After finishing the master program of Graduate School of Science and Engineering, Saitama University in 1983, he entered Kawasaki Heavy Industries, Ltd. in 1985. He assumed a post as an assistant at the Faculty of Engineering, Saitama University in 1989 and his current position as a professor, School of Engineering, Saitama University in 2009.

***by Yoshiaki Okui***
***Professor, Saitama University***

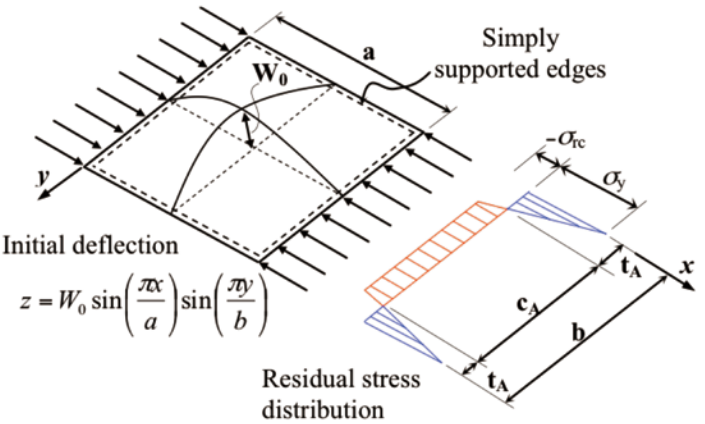
With the aim of further enhancing the competitiveness of steel bridges, the Working Group on Structures and Design Innovations of Steel Bridges, established in the Research Committee on Improvement of Structures and Durability of Steel Bridges, the Japanese Society of Steel Construction, conducted analyses and experiments to collect evidence for use both in examining new bridge design standards and in revising current bridge design standards.

The specific items examined were as follows:

- Rational load-bearing capacity of compression plates
- Bending and shear strength of steel I girders
- Performance-based design methods and advanced analytical approaches
- Rational friction joining of high-strength bolts
- Measurement of residual stress in SBHS (Steels for Bridge High Performance Structures) and the effect of residual stress on the strength of steel members

The following outlines the results for four of the items examined, excluding the final item.

**Fig. 1 Analytical Model for Non-stiffened Compression Plate (Four-side Simple Support)**




## Rational Load-bearing Capacity of Compression Plates

The compression plate strength design equation was reexamined[2]. Three major factors requiring reexamination are cited as follows:

- The regulation for standard load-bearing capacity that is found in the current *Specifications for Highway Bridges*[1] was prescribed based on the elastic buckling theory and data derived from experiments conducted in the 1970s; also, most of the data was based on experiments conducted on approximately 10 mm-thick plates. However, the maximum plate thickness in current steel bridge construction has grown to 100 mm, which has led to the increased use of heavy-gauge plates.
- While SBHS500 and SBHS700—Steels for Bridge High Performance Structures (SBHS)—were standardized in JIS in 2008, SBHS steels are not included in the *Specifications for Highway Bridges.*
- The practice of finding safety factors based on reliability design theory is the currently prevailing trend in the preparation of advanced

**Fig. 2 Probability Density Function of Initial Deflection Used for Monte Carlo Simulations**

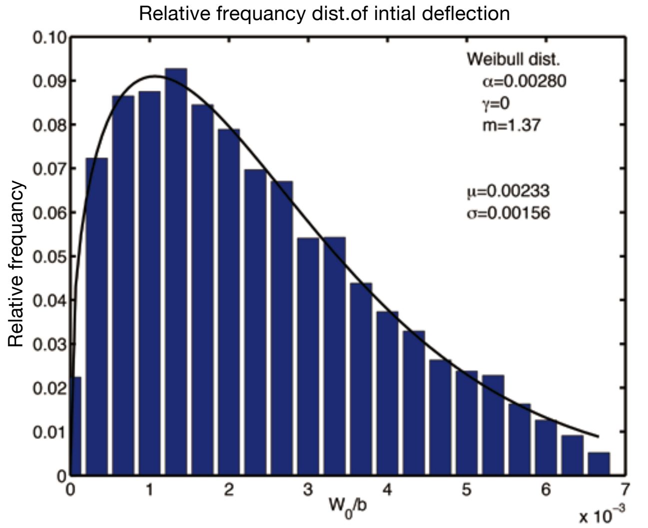
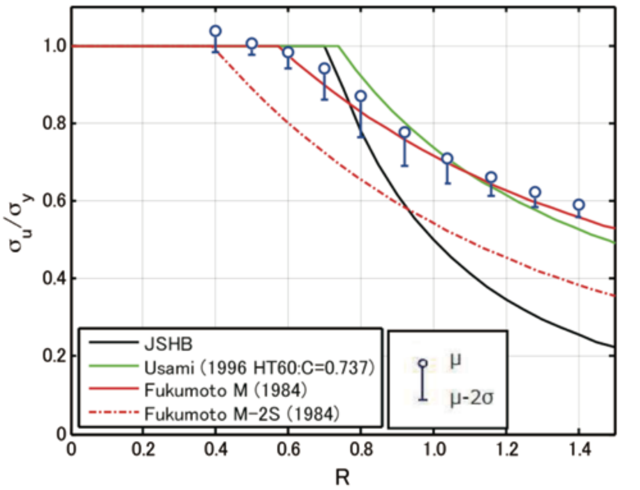



**Fig. 3 Comparison between Monte Carlo Simulation Results (*μ*=average value, *σ*=standard deviation) and Load-bearing Curve in *Specifications for Highway Bridges* (JSHB)**



design standards around the world. To cope with this trend, it has become necessary to obtain probability data such as mean values and standard deviations of critical strengths.

In order to meet such requirements, finite element analysis and Monte Carlo simulation were employed in this study. Fig. 1 shows the analytical model for non-stiffened compression plates (4-side simple support). The mode of initial out-of-plane displacement was assumed to be a sine waveform, and the residual stress was assumed to be the residual stress distribution configuration as shown in the figure. In the Monte Carlo simulation, the maximum value for initial deflection $W_0$ and the compressive residual stress $\sigma r_e$ are assumed to be random variables, and, they are randomly generated according to the probability density functions shown in Fig. 2 as an example. In the trial runs of the Monte Carlo simulation, because of the difficulty of conducting finite element analysis in every simulation, the response surface method was adopted to reduce the calculation time.

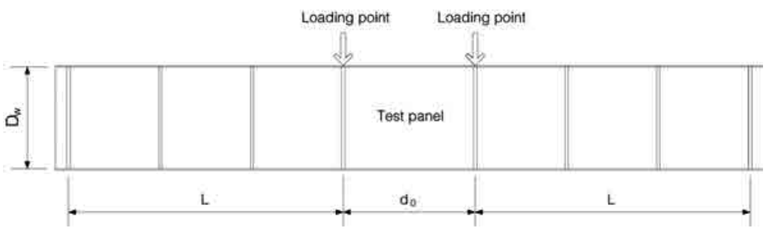
Fig. 3 shows the comparison between the Monte Carlo simulation results for the four-side simple supported stiffening compression plate and the load-bearing capacity curve in *Specifications for Highway Bridges* (JSHB), together with the existing study results[3),4)]. A similar investigation was made for projection panels[5)]. It is possible to rationally determine the partial safety factor by using these examination results. To that end, it is expected that these examination results will be incorporated in the next revision of *Specifications for Highway Bridges.*

### Rational Equation to Calculate I-girder Strength

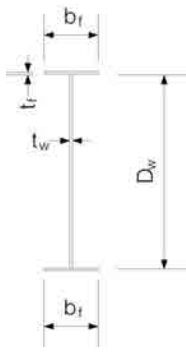
Load-bearing capacity tests were conducted for I-girders employing Steels for Bridge High Performance Structures (SBHS500 and SBHS700) in order to verify the validity of using SBHS in steel bridge girders and the validity of the design equation for SBHS steel girders. Fig. 4 shows the approximate configurations of the specimens used in the bending tests. Loading was applied by means of 4-point bending, and the panel located at the center of the specimen was used as the test panel. Table 1 shows the dimensions of the test specimens used in the bending test. Testing was conducted on two specimens named SBHS500M and SBHS700M.

Fig. 5 shows the load-deflection curve obtained in the tests. $P_n$ in the figure shows the load for nominal design bending strength specified in AASHTO LRFD, and $P_y$ the load for the flange yield bending moment in the tests. It was confirmed in the tests that the bending strength for both SBHS500M and SBHS700M specimens surpasses the nominal design bending strength specified in AASHTO LRFD. While the application range of steel products in AASHTO LRFD is for steel products within a yield strength of 485 N/mm$^2$, both SBHS500 and SBHS700 are out of the application range in AASHTO LRFD. However, it was verified that the bending strength evaluation equation that is developed for conventional steel can be applied to SBHS500 and SBHS700.

**Fig. 4 Bending Test Specimens Using SBHS (Steels for Bridge High Performance Structures)**




(a) Elevation

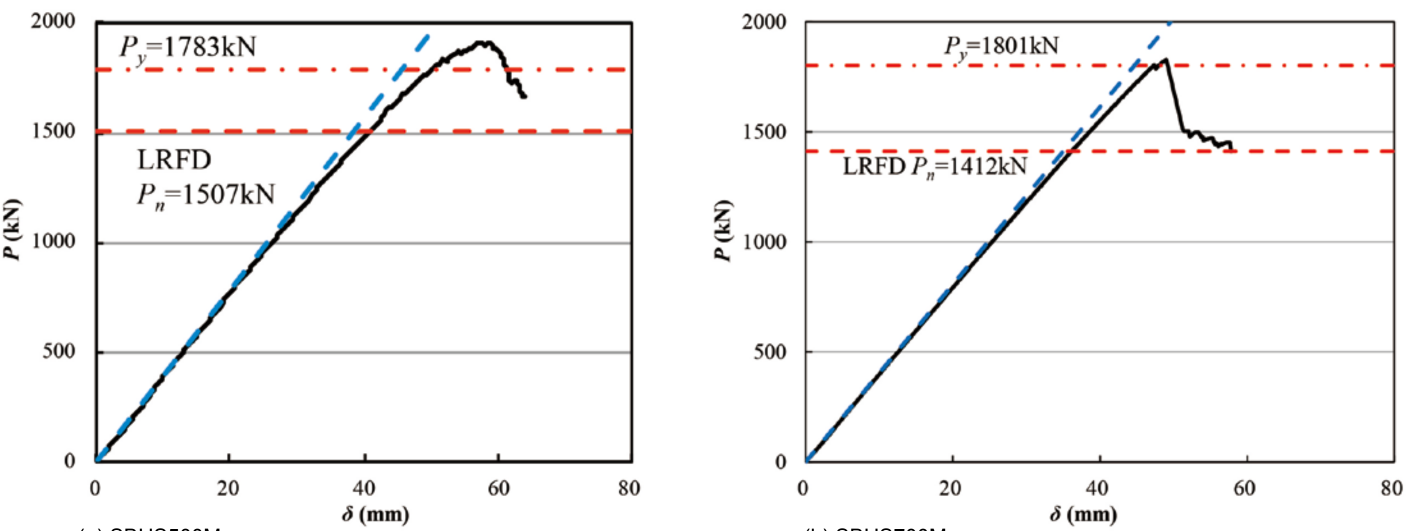



(b) Cross section

**Table 1 Dimensions of Test Specimens**

| Name of specimen | Steel grade of specimen | $b_f$ (mm) | $t_f$ (mm) | $D_w$ (mm) | $t_w$ (mm) | $d_0$ (mm) | $L$ (mm) |
|---|---|---|---|---|---|---|---|
| SBHS500M | SBHS500 | 250.4 | 12.2 | 900.8 | 9.0 | 1198 | 6199 |
| SBHS700M | SBHS700 | 250.2 | 12.6 | 901.4 | 9.0 | 1195 | 7788 |

**Fig. 5 Results of Bending Tests (Load-Deflection Curve)**




(a) SBHS500M (b) SBHS700M

Photo 1 shows the SBHS700M specimen after loading. The type of collapse was of the local buckling of the flange.

### Rational Bridge Design Employing Advanced Analytical Approaches

In steel bridge design, it is necessary to appropriately evaluate the stress occurring in the members. In conventional design, the sectional force of the members and the stress distribution inside the members are calculated by means of frame structure analysis based on beam theory. However, regulation of the current design environment allows for the relatively easy application of the finite element method and other advanced structural analytical methods; accordingly, the rational bridge design method was studied using these advanced analytical approaches.

The following introduces the results of studies directed to the corner sections of beam-column connection shown in Fig. 6. As regards the stress distribution in the corner sections, it is known that the stress distribution found based on beam theory differs greatly from the actual stress distribution due to the shear lag effect.

In this situation, comparative examinations of stress distribution were made using three analytical methods: (a) the finite element method using a shell element, (b) the analytical method proposed by Okumura and Ishizawa, and (c) the finite element method using a constant shear flow panel.

The method proposed by Okumura and Ishizawa examines the shear lag effect by use of the beam theory and the stress concentration factor, and has seen wide practical application in bridge design in Japan. Meanwhile, the constant shear flow panel method is an analytical approach that is composed of a plate element that resists only shear stress and a beam element that bears only vertical stress accompanied by bending along the outer periphery of the plate element. Compared to the finite element method with a shell element, the constant shear flow panel method offers the practical advantage of needing fewer calculations.

Fig. 7 shows a comparison of normal stress distribution in the beam lower flange among the various methods. In the figure, the red line shows the analytical results obtained using the constant shear flow panel method, the blue line shows the results obtained by using the shell method, the black line denotes the nominal stress obtained by using beam theory, and the Δ mark stands for the stress just above the web that was obtained by using the method proposed by Okumura and Ishizawa.

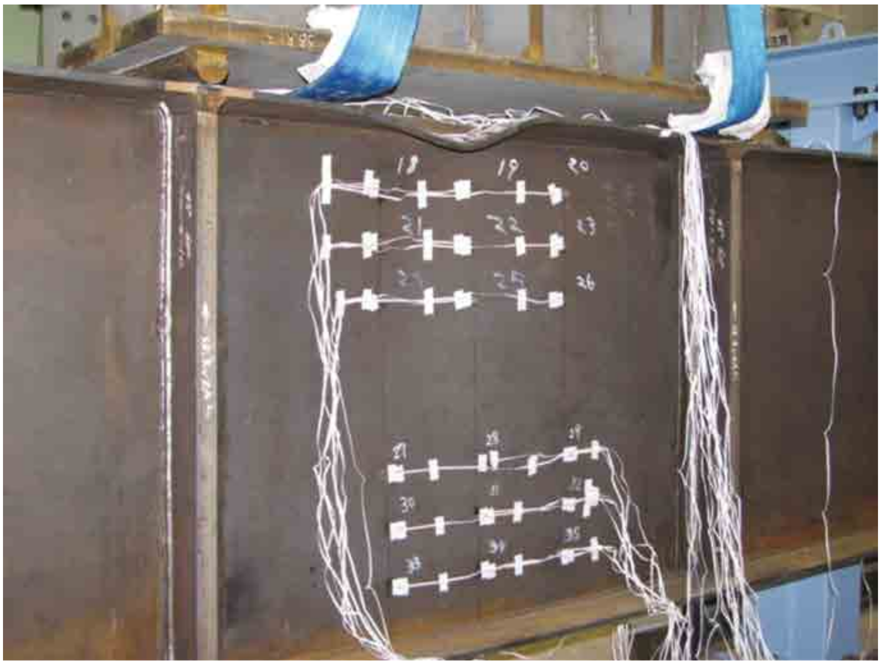


Photo 1 Deformation of SBHS700M specimen after loading

**Fig. 6 Numerical Model of Beam-Column Connection of Bridge Pier**

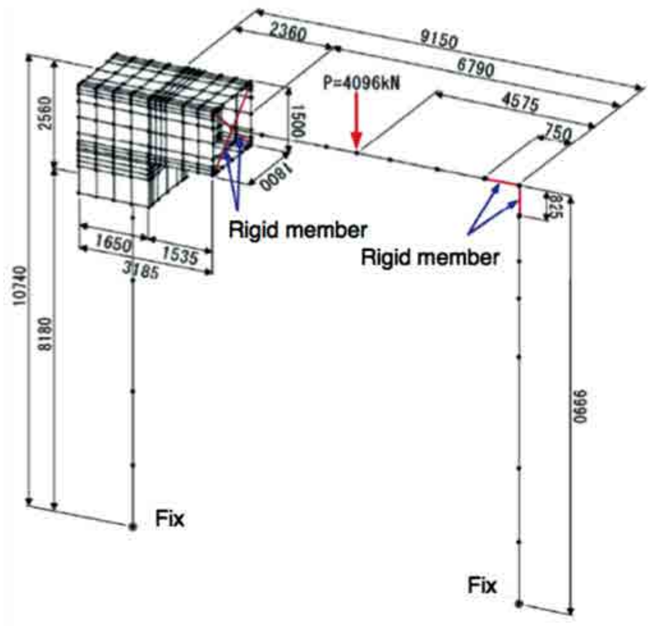
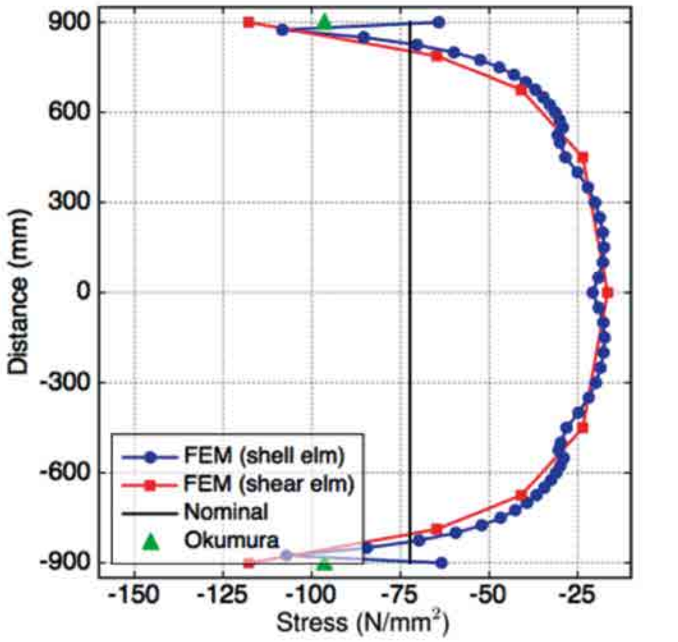



**Fig. 7 Comparison of Stress Distributions at Corner Section of Lower Flange**



While it is necessary, in the finite element method using shell elements, to finely divide the elements so that they are 25 mm or under in size in order to obtain the stress distribution with a certain level of accuracy; in the analytical method using a constant shear flow panel, it is possible to make a highly accurate analysis even when using comparatively rough element divisions as shown in Fig. 6.

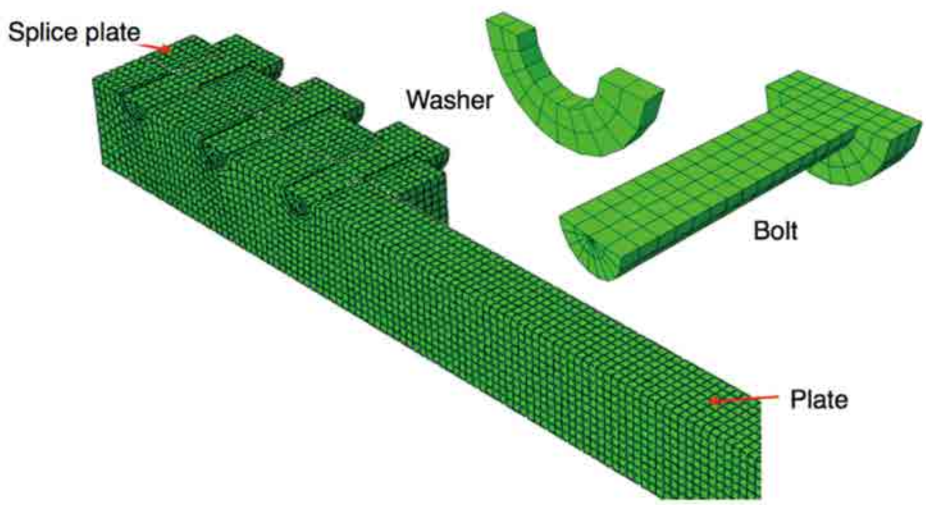
### Rational High-strength Bolt Friction Joining

In recent bridge construction, the size and plate thickness of the structural members are increasing; thus the number of rows of high strength bolts increases steadily for member joints. However, there is insufficient clarity regarding the effect that surface treatments, greater thickness of steel plates, and the multiple-row arrangement of bolts have on the load-carrying capacity of high-strength bolt friction joints. Therefore, experimental and analytical examinations were conducted to discover the extent to which the slip factor is effected by the heavier plate thickness of friction joints using high-strength bolts with contact surfaces coated with zinc-rich paint and by the multiple-row arrangement of high-strength bolts.

Photo 2 shows the experiment that was used to verify the sliding strength of the friction joints. The effect that the heavier plate thickness of friction joints and the multiple-row arrangement of high-strength bolts have on reducing the slip factor was examined by means of the experiment shown in the photo and by finite-element analysis (refer to Fig. 8) that takes frictional slippage into account. ■

Photo 2 Slip strength test for multiple-row high-strength bolt friction joint with heavier plate thickness

**Fig. 8 Finite Element Analysis Model of Bolt Joints**





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# Fatigue Characteristics of Weld Joints Employing SBHS

*by Kazuo Tateishi*
*Professor, Graduate School of Nagoya University*

**Kazuo Tateishi:** After graduating from the Faculty of Engineering, Tokyo Institute of Technology, he entered East Japan Railway Company. He then served as an associate professor of Tokyo Institute of Technology and University of Tokyo. He became a professor of the School of Engineering in 2003 and assumed his current position as a professor of Graduate School of Engineering, Nagoya University in 2010.

SBHS (Steels for Bridge High Performance Structures) are high yield-point steel plates for bridge construction that were specified in JIS in 2008. SBHS feature a high yield point and tensile strength (Table 1), and further because of their high weldability, preheating can be eliminated or preheating temperatures can be lowered. To that end, their application allows cost reductions in various aspects of bridge construction: fabrication, transport and erection.

On the other hand, in cases when SBHS are applied, there is a relatively large increase in the live-load stress because of the thin plate thickness of the members, thus leading to concerns about fatigue problems. However, as the fatigue characteristics of weld joints employing SBHS have yet to be satisfactorily clarified, almost no accumulated data are available, particularly for SBHS 700. Therefore, we decided to conduct a basic study of the fatigue characteristics of weld joints employing SBHS.

## Evaluation of Fatigue Crack Growth Characteristics

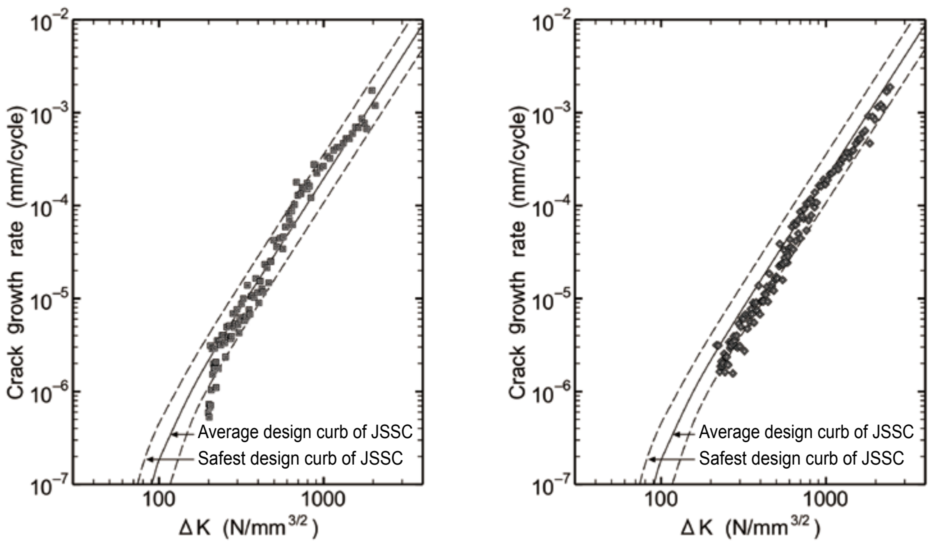
The crack growth characteristics of SBHS were investigated by means of a fatigue crack growth test for compact tension test specimens. The crack growth test was conducted conforming to ASTM.

Fig. 1 shows the relation between the crack growth rate and the stress intensity factor range ΔK of SBHS500 and SBHS700 that was obtained from the test. The curve in the figure shows the average values and the deviation range of conventional steel. As the test results for SBHS fall within the range of deviation of the test results for conventional steel, it can be said that the crack growth characteristics of SBHS are nearly identical to those of conventional steel.

**Table 1 Mechanical Properties of SBHS (Steels for Bridge High Performance Structures)**

| Grade | Plate thickness (mm) | Yield point or proof strength (N/mm²) | Tensile strength (N/mm²) |
|---|---|---|---|
| SBHS500<br>SBHS500W | 6≦t≦100 | 500 or more | 570~720 |
| SBHS700<br>SBHS700W | 6≦t≦75 | 700 or more | 780~930 |

**Fig. 1 Test Results for Crack Growth**




## Measurement of Residual Weld Stress

Using the cutting method and the x-ray diffraction method, measurements were made of the residual stress that occurs at a point 2 mm from the toe of out-of-plane gusset weld joints of small-size test specimens (SBHS500 and SBHS700, shown in Fig. 2) and of a girder test specimen (SBHS700, shown in Fig. 3).

Fig. 4 shows the measured results. In the small test specimens of both SBHS500 and SBHS700, the tension residual stress occurred that was equivalent to about 50~70% of the yield point. In the girder test specimen, the tension residual stress occurred that was equivalent to about 80~90% of the yield point, which was a higher value than that of the small-size test specimens. The reason for the higher value is thought to be due to the difference in the restraint level in the welding of both specimens.

## Fatigue Tests for Small-size Test Specimens

Fatigue tests were conducted on test specimens of small-size out-of-plane gusset weld joints prepared using SBHS. The configuration of the test specimens was as shown in Fig. 2. The stress ratio was set at about 0.

Fig. 5 shows the fatigue test results. The figure also shows the results of previous fatigue tests[1] conducted on out-of-plane gusset weld joints prepared using conventional steel and SBHS. The current test results show almost no difference in fatigue strength depending on the grade of the steel materials. The test results for the stress range of SBHS out-of-plane gusset as-welded joints are distributed in the vicinity of JSSC (Japanese Society of Steel Construction) Class E design curve, and thus satisfy the fatigue design guidelines currently in use.

### Fatigue Tests for Girder Test Specimens

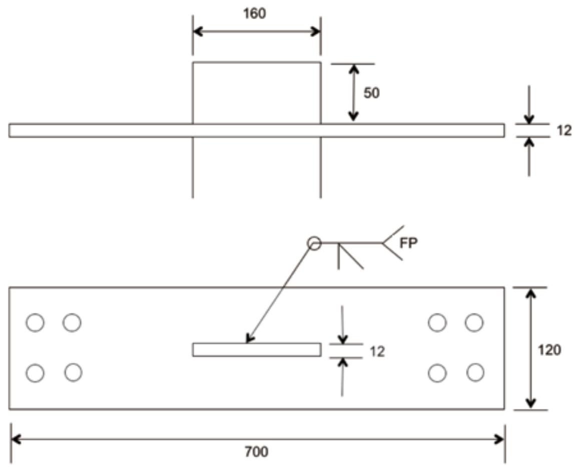
Fatigue tests were conducted on a large-size girder test specimen having a span of 6,000 mm shown in Fig. 3. The steel grade used for the girder specimen was SBHS700. Part of the welding toes were finished by means of grinding or peening.

Fig. 6 shows the fatigue test results. The number of testing cycles needed to reach the stage when a crack has grown to 10 mm (40~50 mm in entire length) from the weld bead to the base metal is indicated as the fatigue life. The figure also shows the existing fatigue test results for the girder test specimens (however the testing cycle is shown for a crack length of 20~40 mm). As a result of the test, the fatigue strength of the as-welded joints nearly satisfies the Class G requirements as specified by JSSC and is similar to that of conventional steel. Because the test was finished at a stage when no cracks had yet emanated from the finished weld toe, differences in fatigue strength depending on the method of toe finishing could not be confirmed. However, the fatigue strength of the finished toes obtained from the test results surpasses Class E specifications, and the fatigue strength of the finished weld joint is improved by two classes or more over that of non-finished (as-welded) weld joints in Class G.

### Effect of Grinder Finish on Fatigue Strength Improvement

The fatigue strength of steel materials improves as their static strength increases. It is

**Fig. 2 Weld Joint Test Specimens (SBHS500 and SBHS700)**




(unit: mm)

**Fig. 3 Girder Test Specimen (SBHS700)**

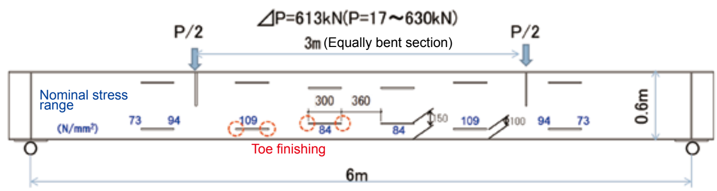



**Fig. 4 Measurement Results for Residual Stress**

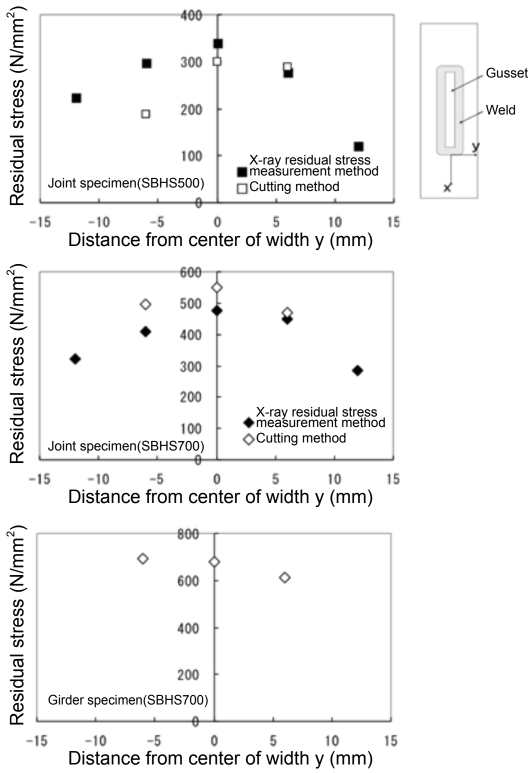



**Fig. 5 Fatigue Test Results for Small-size Weld Joint Specimens**

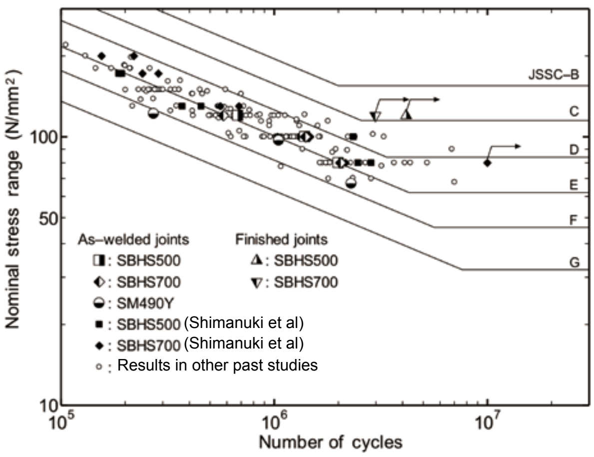
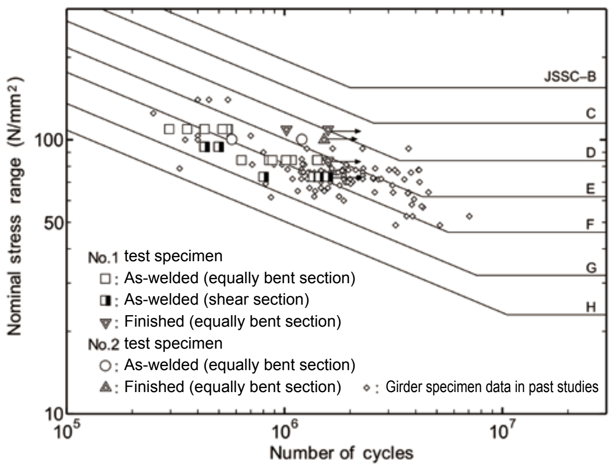



**Fig. 6 Fatigue Test Results for Girder Weld Joint Specimens**



possible that the fatigue strength of steel materials is improved by grinder-finishing the welding toe. To study this, out-of-plane gusset weld joint specimens were prepared and grinder-finished under identical finishing conditions. An examination was then made of the dependency of steel material strength on improvements in fatigue strength.

The steel grades applied were SM490, SBHS700 and SBHS500. Fig. 7 shows the configuration of the out-of-plane gusset test specimens. In gusset welding, full penetration welding was applied to a 50-mm section from the box weld zone in order to prevent the occurrence of fatigue cracks from the weld root section.

Grinder-finishing of toes was provided for the test specimens of the respective steel grades with a target grinding radius of 3 mm or more and a grinding depth of 0.5 mm. Fig. 8 shows the results of measurements of the radius and angle of the toe section (or finished toe section), and Fig. 9 the distribution of the grinding depth. It can be understood from the figures that the grinding treatment for the respective specimens was nearly identical in configuration.

The fatigue tests were conducted on these test specimens under the condition of out-of-plane bending. The stress ratio was set at 0.1 or lower. In the as-welded specimens, all fatigue cracks occurred from the weld toe section of the main plate member; in the grinder-finished specimens of SM490 and SBHS500, all cracks occurred from the grinder-finished section; and in specimens of SBHS700, all cracks occurred from the gusset-side toe section.

Fig. 10 shows the fatigue test results. Fatigue life is denoted as the number of testing cycles required for a crack to grow 10 mm from the weld bead to the base metal. In the as-welded specimens, there was no discernable difference in fatigue strength depending on the grade of the steel materials, and it can be confirmed that there is no dependency of steel product strength on the fatigue strength of the weld joint. On the other hand, the fatigue strength of grinder-finished specimens increases as the strength of the steel materials increases, and thus it is seen that the fatigue strength depends on the strength of the steel materials.

Particularly in SBHS700 specimens, fatigue cracks did not occur in the grinder-finished section but instead were seen in the gusset-side toe section. The fatigue test thus conducted shows grinder finishing greatly improve the fatigue strength of SBHS700 weld joints. ■


### Acknowledgement

The current research represents one aspect of the activities of the Working Group on Fatigue Strength of Steel Bridges of the Research Committee on Improvement of Structures and Durability of Steel Bridges. We offer thanks to those of the Japan Iron and Steel Federation and the Japanese Society of Steel Construction who support our current research activity.


### Reference


1) H. Shimanuki, T. Mori and M. Tanaka: Effect of Steel Product Strength on Improvement of Fatigue Strength by Means of UIT, Journal of Japan Society of Civil Engineers, I-144, 2011


**Fig. 7 Out-of-plane Gusset Test Specimens**

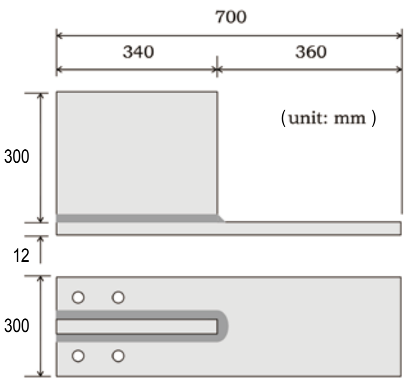



**Fig. 9 Grinding Depth**

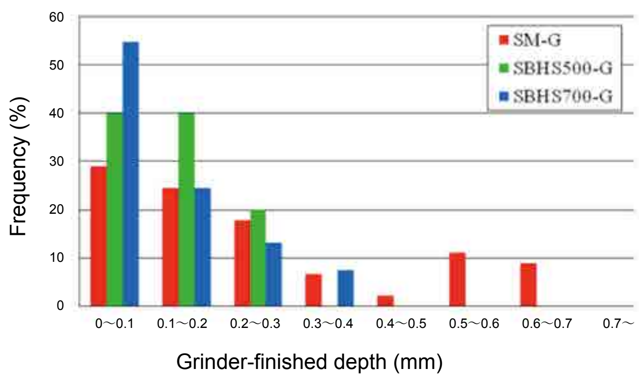



**Fig. 8 Radiuses and Angles of Toe or Finished Sections**

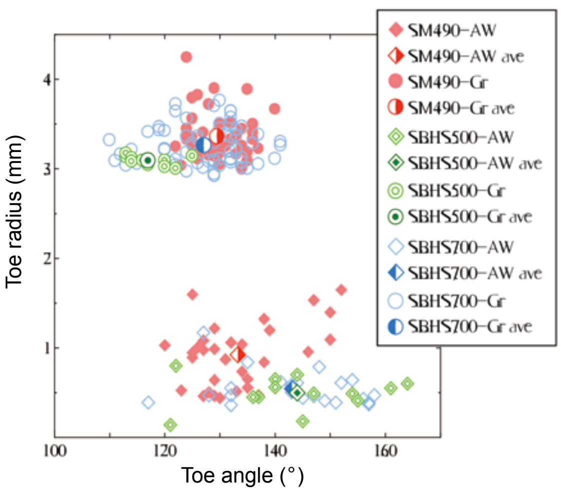
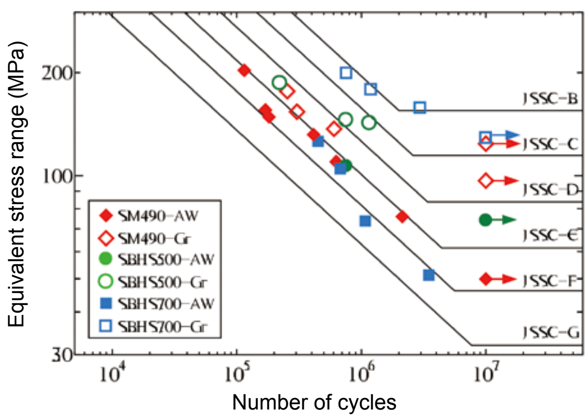



**Fig. 10 Fatigue Test Results for As-welded and Finished Weld Joints**



# Maintenance of Weathering Steel Bridge

**by Eiki Yamaguchi**
**Professor, Kyushu Institute of Technology**

**Eiki Yamaguchi:** Graduating from the University of Tokyo with a bachelor's degree in civil engineering, he went abroad for further study and earned his Ph.D. degree at Purdue University in 1987. After working in Kyushu University and the University of Tokyo, he joined Kyushu Institute of Technology and is Professor there since 2004.

It is crucial to prevent corrosion for steel bridges to have a long life. Painting is effective to that end, but quite costly, amounting to about 10% of the construction cost of a superstructure. Besides, repainting is required during the service life of a bridge. The conventional steel bridge cannot be very competitive especially from a viewpoint of life cycle cost.

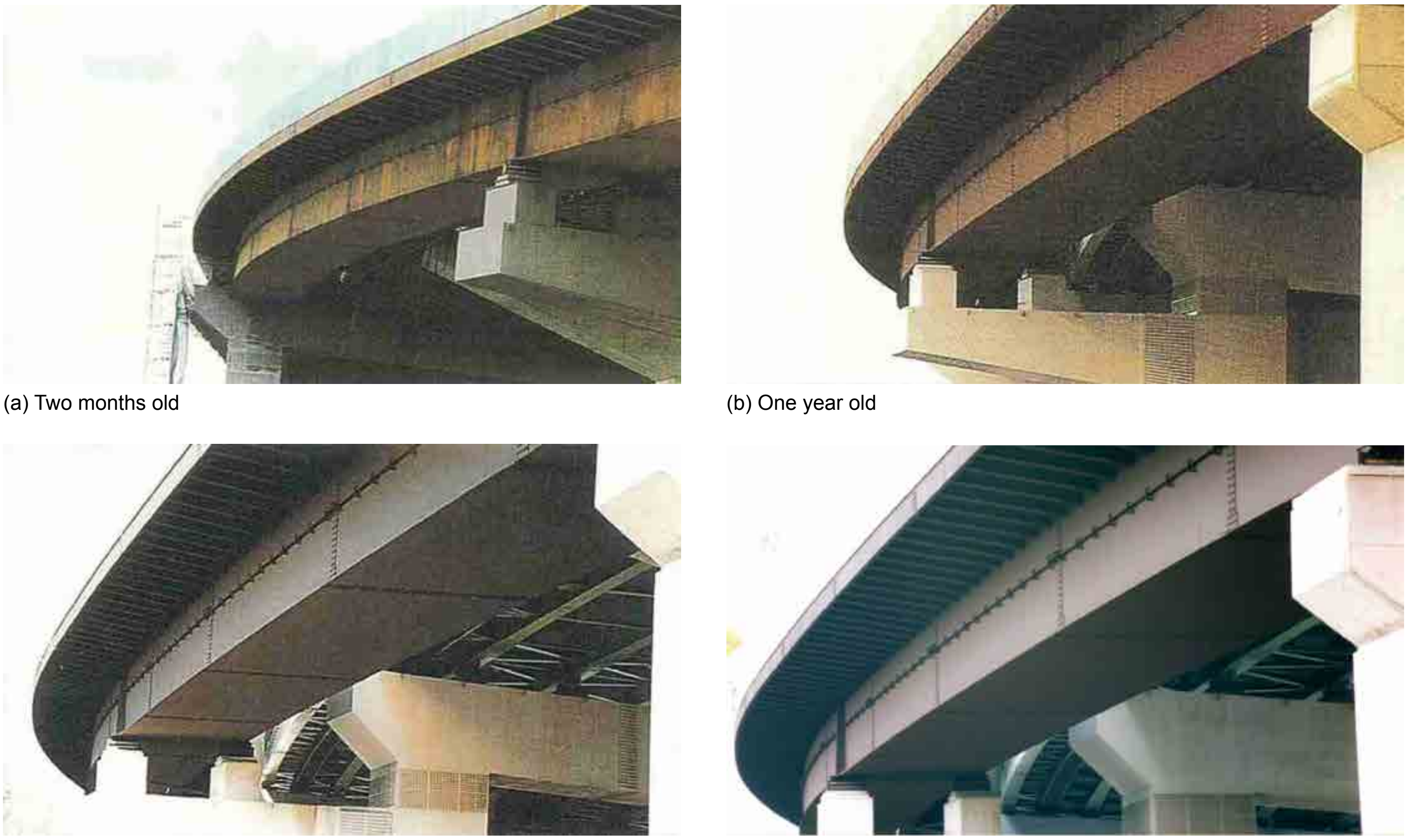
Weathering steel possesses a unique property of suppressing the development of corrosion by a layer of densely-formed fine rust on its own surface: the corrosion rate gradually reduces to the level that causes virtually no damage from an engineering viewpoint, as the layer of the rust grows. Photo 1 shows how the weathering steel bridge changes with time. The painting is not required in the weathering steel bridge, the cost of which can be much lower than that of the conventional steel bridge. The weathering steel bridge is therefore gaining popularity in recent years. As the number of weathering steel bridges increases, however, the reports of unexpected corrosion state is coming out.

In light of the current situation, the Working Group (WG) on Maintenance of Weathering Steel Bridges under the Research Committee on Improvement of Structures and Durability of Steel Bridges of the Japanese Society of Steel Construction was set up in 2012. The members of the group consist of a wide range of engineers including practitioners, bridge owners and academics. WG first reviewed the recent inspection results of weathering steel bridges. At the same time, it prepared questionnaires for bridge owners and bridge engineers as to the maintenance of the weathering steel bridge. Based on the information thus obtained, WG worked on the maintenance issues of the weathering steel bridge. The present article is to provide the outline of these activities.

## Performance of Weathering Steel Bridge

### • Judgement

Since the weathering steel bridge may not perform as expected, the performance is to be inspected. To that end, a state of corrosion is evaluated by classifying it into one of five levels. Brief description of the criteria for the five levels are shown in Table 1. While the performances of Levels 3 and higher are



(a) Two months old

(b) One year old

(c) 17 years old

(d) 22 years old

Photo 1 Weathering steel bridge

satisfactory, the corrosion states of Levels 1 and 2 are not expected from weathering steel and need attention if found. For Level 2, frequent observation is recommended. Level 1 requires further investigation to determine whether or not the bridge should be repaired.

Since the bridge is a large structure, a state of corrosion in a weathering steel bridge is not necessarily uniform. It can vary from portion to portion. In particular, a local corrosion state near the girder end is often found quite different from the corrosion state in a general portion.

The corrosion state is judged by conducting the so-called Scotch-tape test. In this test, the Scotch tape is pressed against the weathering steel. An example of the test result is given in Fig. 1. Examining rust particles thus taken, the corrosion state is classified into one of the five levels given in Table 1.

**• Performance**

The recent survey of Japan Bridge Association (JBA) reveals the performance of the weathering steel bridge. 86% of the inspected bridges are in good shape, carrying a corrosion state of Level 3 or higher, while the remaining bridges have a corrosion state of Level 1 or 2. An effort has been made to identify the cause for the poor performance. It turned out that 68% of those bridges with an unexpected state of corrosion had suffered from water leakage and 32% were located close to another bridge or sloped natural ground. The girder end is in a quite different situation: 31% of the bridges had girder ends in a state of Level 1 or 2. About 80% of those girder ends were found exposed to water leakage.

An expansion joint is one of the bridge elements most susceptible to damage due to traffic loads. The broken expansion joint causes water leakage, exposing the girder end to constant humidity. This accounts for the worse performance of the weathering steel at the girder end.

The survey also shows that de-icing agent increases the rate of poor performance of the weathering steel bridge by about 10%. This rate is the same at the general portion and the girder end. It is also found that with the presence of de-icing agent, water leakage tends to have a worse influence, accelerating the development of unexpected corrosion state.

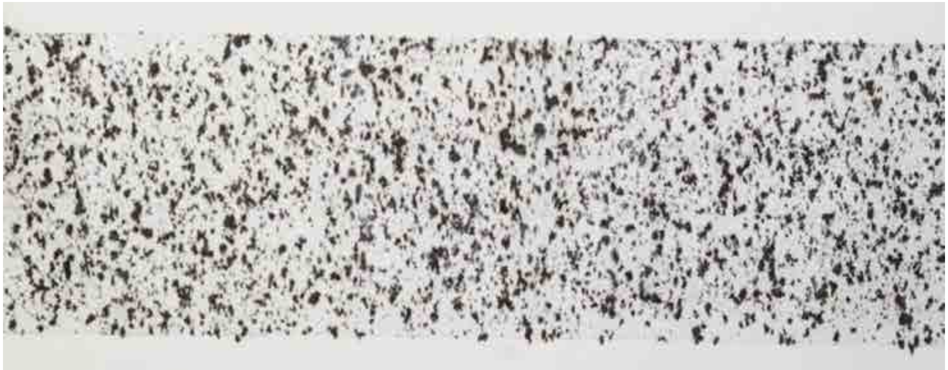
### Questionnaire Survey

The questionnaire survey was conducted so as to extract issues to be solved in dealing with weathering steel bridges. Some of the

**Table 1 Criteria for Corrosion-State Level**

| Level | Description of rust particle | Rust thickness |
|---|---|---|
| 5 | Fine but non-uniform | Less than about 200 μm |
| 4 | Average size of about 1 mm; fine and uniform | Less than about 400 μm |
| 3 | Average size of 1-5 mm | Less than about 400 μm |
| 2 | Average size of 5-25 mm | Less than about 800 μm |
| 1 | Formation of rust layer | Larger than about 800 μm |

**Fig. 1 Scotch Tape Test**



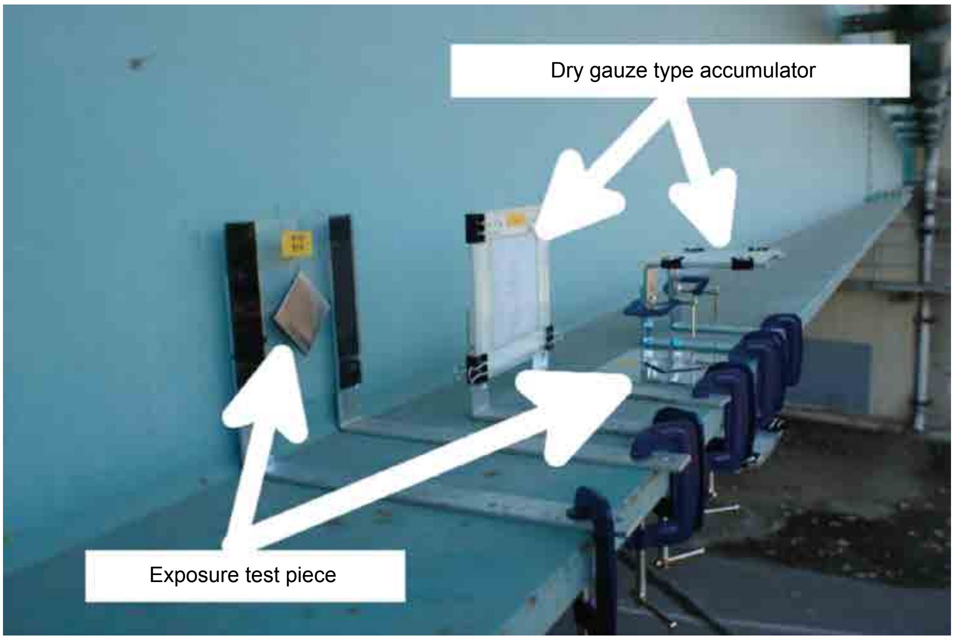
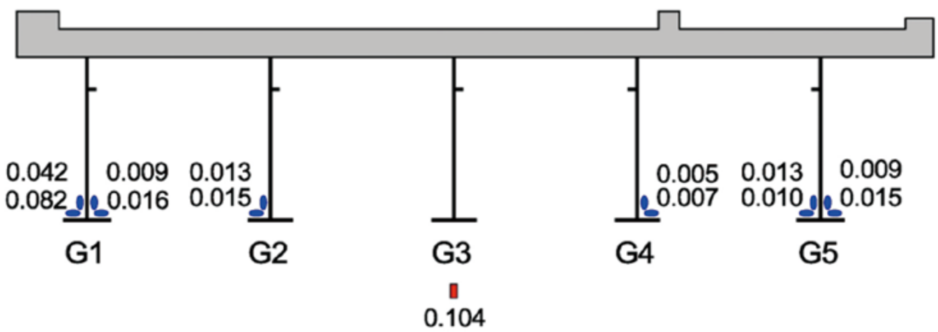



Photo 2 Survey setup for de-icing agent

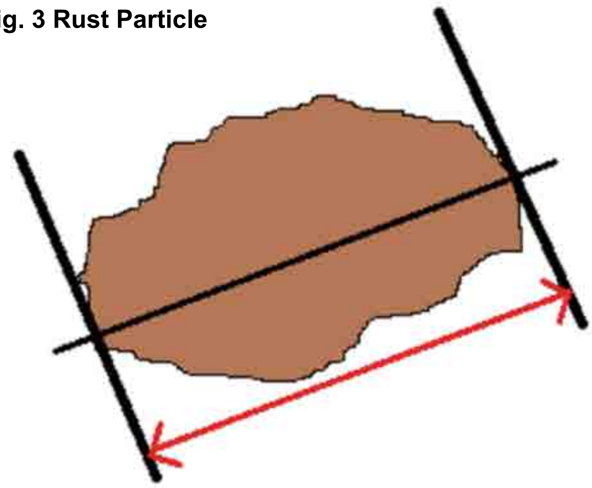
**Fig. 2 Deposition of De-Icing Agent**



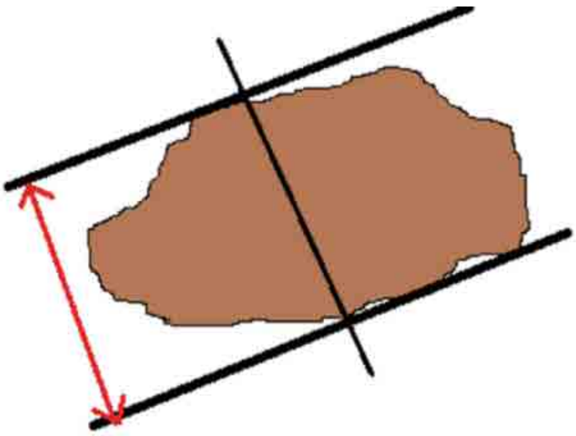
comments are given herein.

- There exist too many technical documents to study. They have to be combined and compiled, yielding an authorized document of some sort.
- There are structural details unique to weathering steel bridges. However, the effects of some of those details are not clear, making it difficult to employ them in design.
- It is not an easy task to evaluate the performance of weathering steel. A good set of sample photos of corrosion states is needed.
- It appears difficult to detect cracks in weathering steel.
- 1/3 of the local governments have experienced the repair of the weathering steel bridge. Many of those bridges were painted. Steel plates were installed in some bridges for reinforcement. Since the surface of those steels were corroded rather badly, they were not sure what would be a relevant base material control or adjustment. A manual for the repair of the weathering steel bridge is needed.

**Fig. 3 Rust Particle**



(a) Maximum size of rust particle



(b) Minimum size of rust particle

### Influence of De-Icing Agent

Survey of the influence of de-icing agent on nine weathering steel bridges that are different in volume of the agent consumption and traffic volume began in December, 2014. The locations of those bridges are far away from the coast, so that no influence of the air-born salt is expected. A typical survey setup is shown in Photo 2.

Although this is an ongoing project and not much result has come out yet, some observations including those from the pre-investigation are available: the influence of de-icing agent decreases rather rapidly in an exponential fashion, as one moves away from the site of the use of de-icing agent; the influence of the de-icing agent lasts about two and a half months even after the last use; the influence of de-icing agent is found large on the exterior surface of the outmost girder; and the amount of the deposition of de-icing agent is very large between closely-positioned parallel bridges. A result of the variation of de-icing-agent deposition is presented in Fig. 2.

### Quantification of Criteria for Corrosion States

A corrosion state is the indication of the performance of weathering steel. The evaluation of the corrosion state is therefore important for the maintenance of the weathering steel bridge. In a current practice, the evaluation is done by the criteria given in Table 1, which is based on the external rust appearance. While this practice is simple, it is quite difficult, if not impossible, to ensure the objectivity of the evaluation result. The criteria had better be more quantified.

Improvement was tried by making use of image analysis. Specifically, Scotch tapes with rust particles were analyzed and various quantities that characterized the state of corrosion were obtained. The quantities include the maximum size of a rust particle (Fig. 3 (a)), the maximum value of the minimum size of a rust particle (Fig. 3 (b)), the density of rust particles and so on. To this end, 27 pieces of Scotch tape with rust particles in various states of corrosion were picked out. The states of corrosion were also evaluated by bridge engineers with a good experience of weathering-steel bridge inspection.

From a viewpoint of maintenance practice, the most important information is whether or not a state of corrosion requires attention. Since the attention is needed for the corrosion state of Level 1 or 2, the target of the trial is to identify the corrosion state of Level 2 or worse. The classification of the corrosion states into the five levels is beyond the scope of this study.

The careful examination of the two sets of results by the image analysis and the experts' evaluation has led to a preliminary conclusion that a state of corrosion needs attention if the maximum value of the minimum length of a rust particle larger than 9 mm.

### Supplemental Document

Supplemental materials were prepared for helping the evaluation of a corrosion state. 28 bridges located in various places and having various states of corrosion were inspected, yielding 190 sets of data. Possible corrosion states are all covered. Each sample set includes many kinds of information: three-dimensional photo (anaglyph 3D) of the steel surface, a Scotch-tape test result, an overview photo, rust thickness and the other related data such the distance from the coast.

### Fatigue Crack

A fatigue crack in an existing bridge was reported during the WG activity and inspected.

Rust around the crack was brighter (Photo 3). Since the same observation had been made in the US, it seems that the rust color could be a clue for finding a fatigue crack.

### Load-Carrying Capacity of Corroded Girder

Unexpected corrosion is often observed at the girder end. The girder end is subjected to the concentrated load, the reaction at the bearing. Therefore, the corrosion in the girder end can threaten the safety of a bridge. Against this background, the deterioration of the load-carrying capacity due to corrosion was investigated numerically. The main-girder end and the end cross girder were studied with various patterns of corrosion in them. The computation indicates that not only the size and the depth of the corrosion but also the corrosion location is an influential factor for deterioration. For example, corrosion in the girder outside the bearing (Fig. 4) has a larger influence, thus requiring more attention if found.

### Repair by Painting

Difficulties have been pointed out as for the repair of the weathering steel bridge: it is not easy to remove the rust from weathering steel surface and the removal of salt deposit is not easy, either. Nevertheless, as those removals are very important, the repair work of the weathering steel bridge is said to take much more time.

WG had a chance to repair a corroded weathering steel bridge. With the consent of the bridge owner, valuable in-situ data on repair work were taken at this actual repair work as to the effectiveness of a surface-preparation method.

Based on the data, WG recommends the following method: the power tool cleaning is conducted first; the procedure of blasting followed by water-washing is repeated twice; and the finishing blast is carried out.

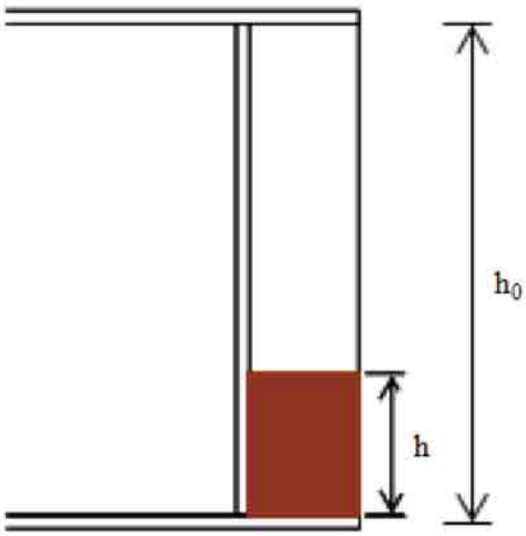
The data shows that even for the Level 1 corrosion state, a satisfactory result including the salt deposit of no more than 50 mg/m2 is achieved by this method. It is also confirmed that the number of blasts can be less for a corrosion state of Level 2.

### Concluding Remarks

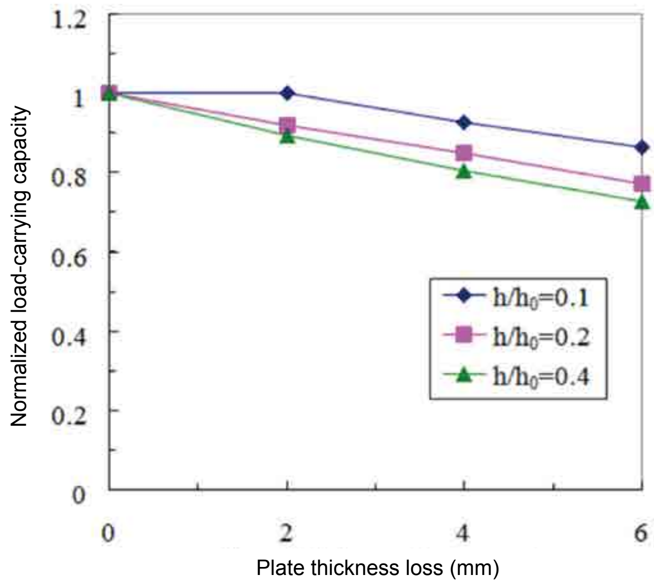
In general, most of the weathering steel bridges in Japan are in good shape. Yet there are some that do not meet the expectation. Maintenance work is therefore important. The WG activity described herein will help solve some issues associated with the maintenance work. The activity continues to provide further technologies that would help keep the civil infrastructure of the weathering steel bridge in good shape. ■

Photo 3 Fatigue crack

**Fig. 4 Load-Carrying Capacity of Corroded Main Girder**




(a) Location of corrosion




(b) Deterioration of load-carrying capacity

# "Building National Resilience" Initiative and Future Directions

***by Satoshi Fujii***
***Professor, Kyoto University;***
***Special Advisor to the Cabinet, Cabinet Secretariat***

**Satoshi Fujii:** After receiving the master's degree at Kyoto University in 1993, he became assistant professor at Graduate School of Engineering, Kyoto University. He became professor at Tokyo Institute of Technology in 2006, and assumed his current position as professor at Graduate School of Engineering, Kyoto University in 2009.

The risk of Japan suffering one or more unprecedented natural disasters is huge and growing quickly. To begin with, over the last half century, the country has built a huge civil society in which highly complicated and diversified elements are organically linked to each other. On top of this, the country has built a number of megalopolises in which the essential facilities of a modern society are excessively concentrated. Further, these megalopolises are located in areas that have been severely impacted in the past by the Kanto earthquakes, Nankai Trough earthquakes and Mt. Fuji eruptions, and that are apparently forecasted to suffer a succession of similarly large disasters in the near future. (Refer to Fig. 1)

### Outline of the "Building National Resilience" Initiative

In light of this background, the Japanese government began an initiative in 2015 known as "Building National Resilience—Creating a Strong and Flexible Country," with three major objectives:

- To break the current devastating situation of forecasted attack of unprecedented mega-disasters
- To minimize as much as possible the destructive effect of these mega-disasters on the future of Japan
- To avoid for the country to fall in a serious situation in which the nation could never return to its original state due to these disasters

The Abe Cabinet, led by Prime Minister Shinzo Abe, was inaugurated in December 2012. Following that, the cabinet position of Minister in Charge of Building National Resilience was created; concurrently, the National Resilience Promotion Office was established in the Cabinet Secretariat. Centered in this new office, the Building National Resilience initiative is being extensively promoted in all ministries and agencies.

Parallel to these endeavors, deliberations were held in the national assembly that resulted in the enactment of the Basic Act for National Resilience in December 2013. In this act, it was clearly stated that the National Resilience Promotion Office, led by the prime minister, would be established in the cabinet, and at the same time it was obliged to work out a Fundamental Plan for Building National Resilience that would rank highest among all administrative plans nationwide (Fig. 2).

### Fifteen Serious Situations and National Resilience

In working out the Fundamental Plan for Building National Resilience, the government identified 45 serious situations that might occur. In particular, 15 of these situations were assigned priority status requiring that immediate measures be implemented (see Table 1).

Each of these 15 serious situations were selected keeping in mind the likely occurrence of mega-thrust earthquakes and various other kinds of natural disaster. Among the major countermeasures for these situations are seawall improvements, seismic reinforcement, measures against land liquefaction and other hardware measures, and disaster-prevention training, close communication on

**Fig. 1 Megathrust Earthquakes Forecasted to Occur in Japan**

**Nankai Trough Earthquake**
(publicly announced in 2013)

M8~M9-class earthquake to occur with a probability of about 70% in coming 30 years

Predicted human casualties

- Casualties due to building collapse
  Fatalities: Approximately 17,000~82,000 persons
- Casualties due to tsunamis
  Fatalities: Approximately 13,000~230,000 persons
- Casualties due to fire
  Fatalities: Approximately 1,600~22,000 persons

Maximum damage:
Approximately 323,000 fatalities
(incl. other damages)

Predicted monetary damages

Damage to assets (disaster-stricken areas, land sides)
- Private sector ¥148,400 billion
- Semi-public sector ¥900 billion
  (Electricity, gas, communications, railways)
- Public sector ¥20,200 billion
  Total ¥169,500 billion

Effect on economic activities (nationwide)
- Loss due to degraded production and services ¥44,700 billion
- (Of the above, loss due to traffic disruption
  Disruption of highways and railways ¥6,100 billion)

Maximum loss:
Approximately ¥214,000 billion
(incl. other losses)

Source: "Forecasted Scale of Disasters Caused by Nankai Trough Earthquake"(Secondary Report) prepared by Cabinet Secretariat

**Inland Earthquake in Tokyo**
(publicly announced in 2013)

M7-class earthquake to occur with a probability of about 70% in coming 30 years

Predicted human casualties

- Casualties due to collapse of buildings
  Fatalities: Approximately 4,000~11,000 persons
- Casualties due to fire
  Fatalities: Approximately 500~16,000 persons

Maximum damage:
Approximately 23,000 fatalities

Predicted monetary damages

Damage to assets (disaster-stricken areas)
- Private sector ¥42,400 billion
- Semi-public sector ¥200 billion
  (Electricity, gas, communications, railway)
- Public sector
  (Lifelines, public civil engineering facilities, etc.) ¥4,700 billion
  Total ¥47,400 billion

Effect on economic activities (nationwide)
- Loss due to degraded production and services ¥47,900 billion
- (Of the above: Loss due to traffic disruption
  Suspension of functions of highways, railways, ports/harbors ¥12,200 billion)

Maximum loss:
Approximately ¥95,000 billion
(incl. other losses)

Source: "Forecasted Scale of Disasters Caused by Inland Earthquake in Tokyo and Its Countermeasures" (Final Report) prepared by Cabinet Secretariat risk assessment, promotion of business continuity plans and other software measures. A key for the successful promotion of these countermeasures is that they be jointly and comprehensively implemented by all governmental and private sectors from a variety of perspectives.

Furthermore, the most important measures that should be aggressively promoted in order to avoid these serious situations are the "easing of the intense concentration of diverse functions in the nation's capital Tokyo" and the "formation of nationally distributed functions." That is, the easing of function concentration and distribution of these functions nationwide not only can serve as an extremely effective measure for building national resilience but also will produce the following three benefits:

- Drastic mitigation of the impact of disasters caused by an inland earthquake that strikes the Tokyo capital region
- Post-disaster preservation of national vitality
- More vigorous and speedier implementation of rescue, restoration and reconstruction activities

In addition, another expected merit is:

- Greater contribution to building resilience in local areas through the re-activation of these areas due to the diffusion and distribution of functions

In recognition of this, the Fundamental Plan for Building National Resilience was decided in June 2014 at a cabinet meeting based on the Basic Act for National Resilience. It is clearly described in the Fundamental Plan that two major goals should be promoted—easing of the intense concentration of functions in the capital, Tokyo, and the nationwide formation of locally independent municipalities with dispersed essential functions and regional collaboration.

In order to steadily attain these two goals, it will be necessary to devise software measures conducive to accelerating the attainment of these two goals, such as reexamination of appropriate taxation, subsidy and support systems and conventional multifaceted mechanisms. At the same time, it goes without saying that hardware measures to accelerate the attainment of these two goals must be assigned greater importance. (Refer to Fig. 3)

**Table 1 Fifteen Serious Situations Cited by the Government as Requiring Rapid Measures**

- Collapse of high-rise and other buildings in urban areas and outbreak of large fires in such areas
- Prodigious loss of life due to tsunamis
- Prodigious loss of life because of inadequate transmission of information
- Absolute lack of rescue and emergency activities by self-defense forces, the police, fire fighters, etc.
- Increase in fatalities due to insufficient supply of food and other necessities to disaster-stricken areas
- Prolonged inability to use telephones and radios
- Suspension of energy supply
- Suspension of energy supply to domestic industries
- Significant reduction of national economic productivity
- Suspension of food supply
- Disruption of transportation arteries connecting eastern and western parts of the country
- Disaster-induced dysfunction of central government
- Prolonged inundation of towns due to extensive flooding
- Occurrence of terrible disasters due to large-scale volcanic eruptions, etc.
- Devastation of farmland and forests that account for 80% of national land area

**Fig. 2 Image of "Fundamental Plan for Building National Resilience" to Be Promoted as Priority Plan**

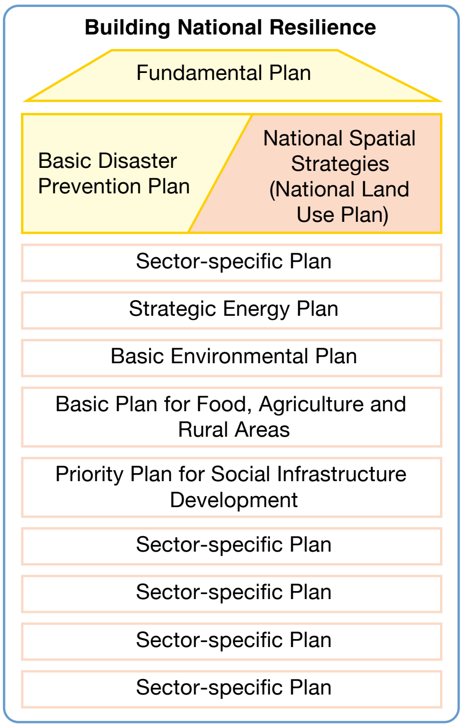
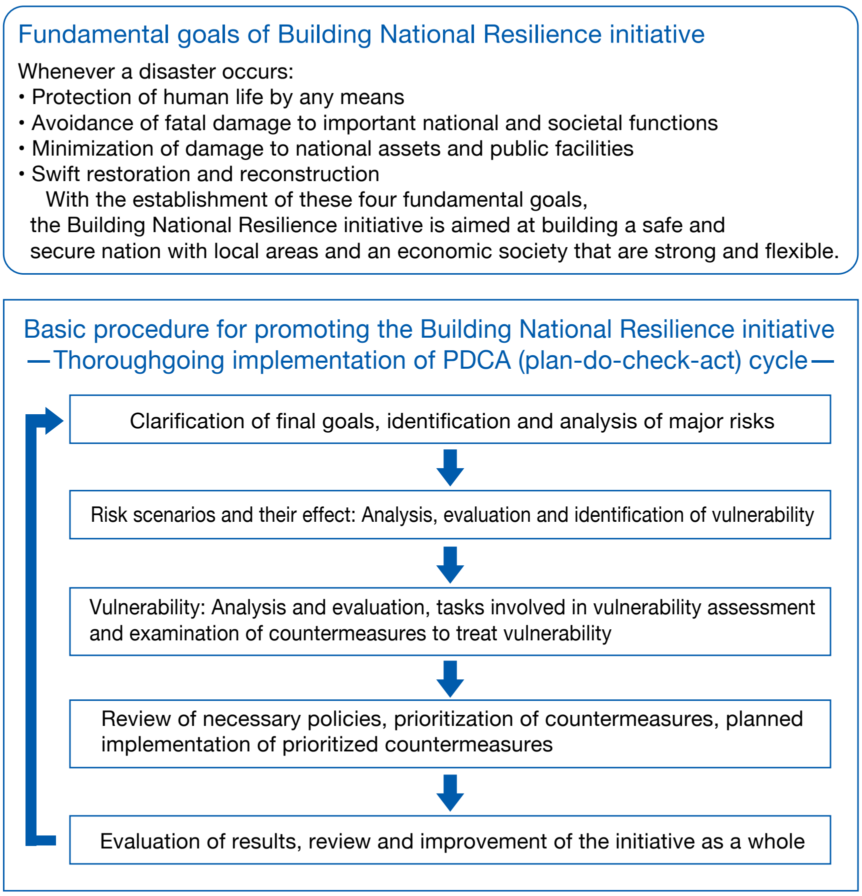



**Fig. 3 Fundamental Goals of the "Building National Resilience" Initiative and Related Promotion Efforts**



### Fundamental Plan for Regional Resilience

Following the central government's establishment of the Fundamental Plan for Building National Resilience that prescribes the direction for developing national resilience, the central government prepared guidelines for devising a "Fundamental Plan for Regional Resilience" and strongly encouraged the nation's local governments to work out their own fundamental plans. As a result, nearly half the nation's local governments are preparing specific plans of their own (refer to Fig. 4).

Concurrently, the central government is deliberating on the integration of both regional resilience and regional revitalization efforts by local governments. The main cause for integration is that, when regional revitalization is attained, regional resilience after suffering a disaster will naturally be improved; and, further, initiatives to secure regional resilience in response to disasters will bring about diverse advantages to the local economy during a normal period. Furthermore, it is needless to say that the reduction of the intense concentration of functions in Tokyo, a top priority in "Building National Resilience," will directly promote regional revitalization.

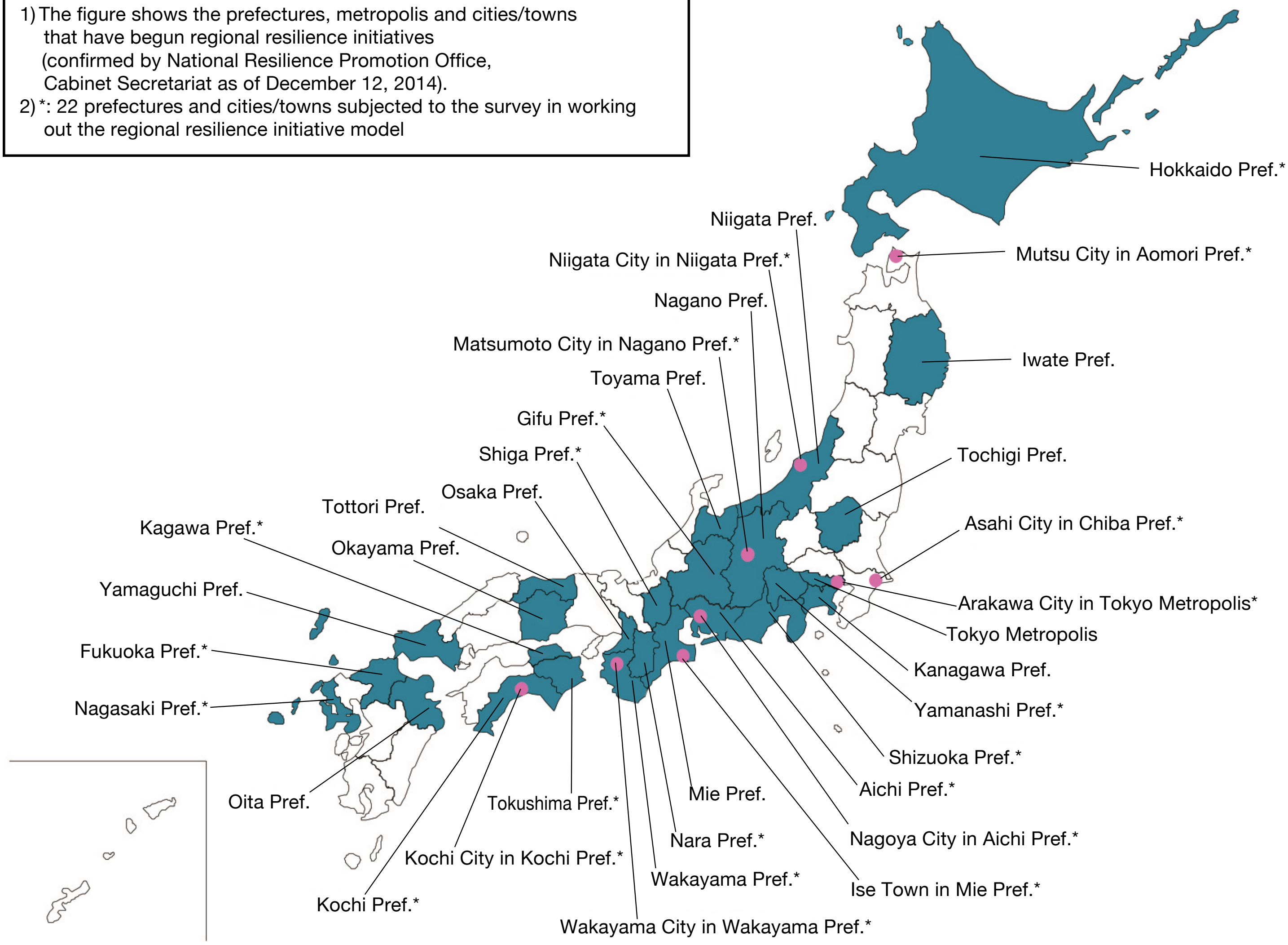
### "Building National Resilience" as a National Project

Currently, the Japanese government is discussing diverse measures to promote the Building National Resilience initiative. Among specific measures are the enhancement of private sector resilience, apart from central and local government initiatives; the promotion of school training programs aimed at educating children about the philosophy behind building national and regional resilience—"building of disaster-prevention nation and towns"; and the promotion of a framework for international cooperation.

The Building National Resilience initiative is being promoted as a national project under the direction of the central government and in cooperation with local governments, private sector organizations, individual citizens and related nations. ■

**Fig. 4 Local Governments to Publicly Announce Start of Effort to Work Out "Building Regional Resilience" Initiatives (incl. planning stage)**

As of December 12, 2014: 25 prefectures, 1 metropolis and 9 cities/towns
Notes
1) The figure shows the prefectures, metropolis and cities/towns that have begun regional resilience initiatives (confirmed by National Resilience Promotion Office, Cabinet Secretariat as of December 12, 2014).
2) *: 22 prefectures and cities/towns subjected to the survey in working out the regional resilience initiative model



# Steel Structure Conference in Cambodia

The Japan Iron and Steel Federation (JISF) held a conference titled "Recent Technologies for Steel Structures 2014" in Phnom Penh, Cambodia on December 4, 2014. It was held jointly with the Ministry of Public Works and Transport of Cambodia and the Institute of Technology of Cambodia.

Five lectures covering such topics as ports and harbors, bridges and building construction were delivered by experts from both Japan and Cambodia at the conference, to which approximately 120 engineers and university students attended. Concurrently with the conference, a Small Group Session of key persons from both nations was held for the exchange of opinions and information regarding current conditions and future tasks pertaining to the wider application of steel structures in Cambodia.

This conference is the second in the series and follows a conference held by JISF in 2012. JISF plans to hold a third conference in December 2015 in Cambodia.


Conference on "Recent Technologies for Steel Structures 2014" held in Phnom Penh

# Seminar on Seismic Resistance in Thailand

JISF in collaboration with the Iron and Steel Institute of Thailand held a seminar titled "Seismic Design Code and Earthquake Countermeasures Technology" on February 12 and 13, 2015 in Bangkok, Thailand. Triggered by an earthquake that occurred in May 2014 in Thailand and reflecting the resultant growing concern about seismic-resistant technologies for buildings in that country, the seminar was planned as one of the Steel Cooperation Programs stipulated in the Japan-Thailand Economic Partnership Agreement.

Three experts from Thailand and two from Japan delivered lectures on seismic design and seismic-resistant technologies at the seminar, to which 80-plus people attended. JISF and the Iron and Steel Institute of Thailand plan to jointly hold a "Seminar on Technology Transfer for Steel Construction Promotion" in Bangkok in September 2015.

Lecture delivery by General Manager Dr. Yasushi Ichikawa of Nippon Steel & Sumikin Engineering Co., Ltd.

Lecture delivery by Staff Manager Yukio Murakami, JFE Steel Corp.

Greeting by Acting President Pairojana Meethawee, Iron and Steel Institute of Thailand

Lecture delivery by Advisor Nuttapon Suttitam, Iron and Steel Institute of Thailand

# Opening of South East Asia Regional Office

JISF opened its South East Asia Regional Office in Kuala Lumpur, Malaysia and held an opening ceremony on April 28, 2015 to mark the start of regular operations. The Regional Office serves as a liaison that promotes collaborative operations between the Japanese steel industry and the ASEAN steel industry. Also, it collects information in a wide range of fields by developing human networks with the iron and steel organizations of the ASEAN region and, at the same time, conducts diverse activities to promote better understanding of the Japanese steel industry.

The office information is as follows:

**South East Asia Regional Office**
**The Japan Iron and Steel Federation**
Suite 8-1 & 8-2, Level 8, Menara CIMB, No.1, Jalan Stesen Sentral 2, 50470 Kuala Lumpur, Malaysia
Phone: +60-3-2298-8307 Fax: +60-3-2298-8201

Opening ceremony and greeting by Executive Director Yuzo Ichikawa of JISF at the opening ceremony


**STEEL CONSTRUCTION TODAY & TOMORROW**



*Published jointly by*
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*Edited by*
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Chairman (Editor): Kaoru Uehara

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