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Osman Gazi Bridge in Turkey

Prize-winner: IHI Infrastructure Systems Co., Ltd.

Project Overview

The Osman Gazi Bridge named in honor of Osman I (1259-1326, who founded the Ottoman Empire in 1299), locates in northwest Turkey and carries the Gebze-Orhangazi-Bursa-Izmir motorway across the Sea of Marmara at the Bay of Izmit between the Diliskelesi Peninsula on the north and the Hersek Peninsula on the south.

The bridge, contracted to IHI Infrastructure Systems Co., Ltd. (IIS) on an EPC (Engineering, Procurement and Construction) basis, was constructed with a 1,550 m main span making it the fourth longest suspension bridge in the world. The scale of the bridge and the tight construction schedule in the EPC contract required the state-of-the-art and sustainable design, well-proven construction methods adapted to the technical challenges and financial success for the project. (Refer to Fig. 1)

Construction was started in January 2013 and the bridge opened to traffic on July 1st, 2016 only within 3.5 years. More than 83,000 tons of steel has been used and 200,000 m^3 of concrete has been cast in total. (See Photos 1 and 2)

Engineering Challenges

Since the bridge locates in a high seismic zone where magnitude 7.4 Kocaeli Earthquake took place in 1999 along the North Anatolian Fault (NAF), high durability against huge seismic events is required with achieving very fast and safe construction. To realize this requirement, IHI has managed and collaborated with a number of subcontractors in more than 10 countries and overcome many engineering challenges.

The tower foundations are concrete caissons placed 40 m below sea level on a gravel bed over soil strengthened by 195 nos. of the steel inclusion piles per tower (Fig. 2). The tower foundation is allowed to move by releasing friction, functioning as an isolation system during a huge earthquake (return period of 2,500 years). The NAF is running close to the bridge site, around 2 km away from the south anchorage area, and the south anchorage is in the secondary fault zone. It is the first case in the world in which this isolation system is applied to the suspension bridges.

Fig. 1 General Arrangement of Osman Gazi Bridge

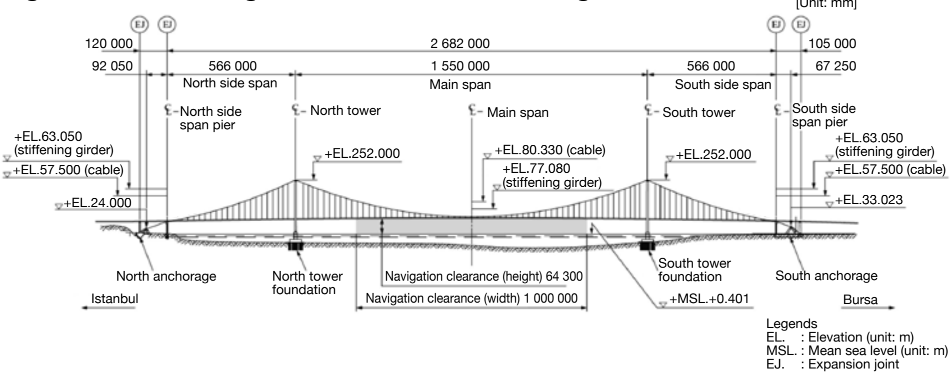


Photo 1 Aerial view of Osman Gazi Bridge

Photo 2 South anchorage area and the bridge

The two towers are 252 m high, each consisting of two single-celled box legs with two cross beams. The steel towers have been chosen to minimize the whole construction period by overlapping the foundation work at the seabed, the caisson fabrication work and the tower fabrication work, although the concrete towers are common for the long-span bridges in the world except for in Japan where technology of the steel towers has been developed. Each tower leg is divided in-

Fig. 2 Schematic Image of Tower Foundation

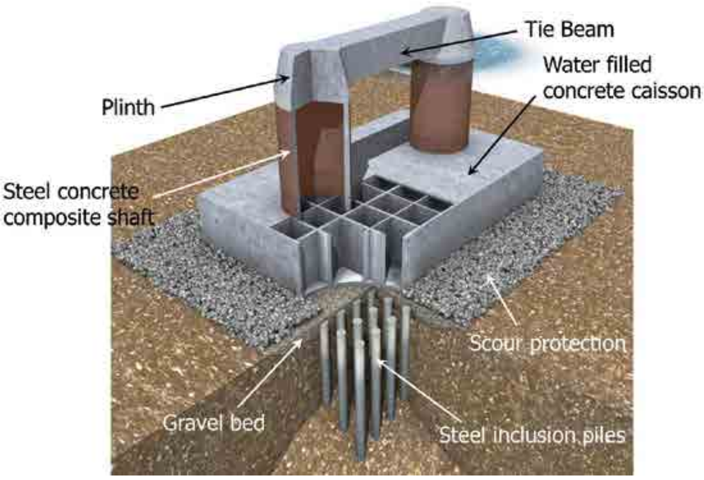


Photo 4 Main cable erection by PPWS method

Photo 3 Tower block erection by floating crane

Photo 5 Deck erection by lifting device

to 22 blocks, erected by floating crane for the lower half and by jib-climbing crane for the upper half and connected each other by combined method of welding and HSFG bolt connection (Photo 3).

The main cables are made of a prefabricated parallel wire strand (PPWS), each consisting of 127 high strength steel wires of 5.91 mm in diameter and having a breaking strength of 1,760 MPa. 110 PPWS per one main cable are spanned between the cable anchorages and 2 extra PPWS are placed between the tower and the cable anchorage on both sides. It is the biggest diameter of the wires used in the PPWS method in the past suspension bridges and it makes the anchorages at both ends smaller for less concrete volume and shorter construction period. (See Photo 4)

The deck is a hexagonal closed steel box girder with a width of 30.1 m and a depth of 4.75 m and is carrying three lanes of highway traffic in each direction. The walkway for maintenance vehicles with a width of 2.9 m is at both sides of the steel deck similar to the 1st and the 2nd Bosporus Bridges. The deck is divided into 117 segments of 25 m length typically due to the capacity of lifting devices for erection (Photo 5). To achieve further shortening of the construction period, the typical 2 blocks in the main span were connected into a 50 m long block by welding at the fabrication shop. This reduces the number of on-site welding joints (thus welding work) and the period between the deck closure to the traffic opening into only 2 months.

Contribution to Social Development

In the project, more than 100 Japanese engineers were stationed in Turkey at the fabrication shops or on site to supervise the fabrication work and superstructure construction works. The purpose of this supervision is not only to construct the bridge with high quality based on Japan's experiences but also to transfer their knowledge to Turkey and to contribute to Turkey's technology development.

Furthermore, the project welcomed many internship students from Turkey and abroad and site visits by local children who will play important roles in the future. (Photo 6)

Acknowledgement

The authors are deeply grateful to KGM (General Directorate of Highway, MOT Turkey) and OTOYOL YATIRIM VE ISLETME A.S for guiding us to a successful completion and their permission on publication of this paper. ■

Photo 6 View at tower top

Shinjuku Toho Building

Prize-winner: Takenaka Corporation

The Shinjuku Toho Building is a complex building that accommodates a movie theater, shops and a hotel. Featuring a slender façade, it has 30 stories above ground and a height of 130.25 m. The building was completed in 2015 in Shinjuku-ku, Tokyo. (Photo 1)

Response Control Using Extra High-strength CFT and New Diaphragm Plate Specifications

The ratio of height to width of the high-rise section of the building is quite large, 6.8 at maximum, and further the horizontal stiffness of the low-rise section differs greatly from that of the high-rise section. Because of this, the horizontal deformation of the high-rise section becomes large during earthquakes (Fig. 1).

To solve this problem, it was necessary to control the horizontal stiffness of the low-rise section so that it does not much exceed that of the high-rise section, although the columns of the low-rise section were required to have sufficient strength. In order to satisfy these two conflicting requirements, extra high-strength concrete-filled steel tube (CFT) columns were adopted that employ 780 N/mm^2-grade high-strength steel and Fc 100 N/mm^2-grade concrete. This facilitated the successful control of horizontal stiffness between the low-rise and high-rise sections during earthquakes (Fig. 1).

As the inner diaphragm plate had never been used for electro-slag welding of columns employing 780 N/mm^2-grade high-strength steel, there was some concern regarding the securement of welding quality. Then, full-scale welding tests were carried out on the electro-slag welds to establish a steel column manufacturing method that would secure the required Charpy value, 27J or higher (Photo 2). Consequently, specifications for electro-slag welding of the inner diaphragm plate were, for the first time ever, put into practice for 780 N/mm^2-grade high-strength steel columns.

Development of an Improved Non-scallop On-site Welding Method

In the construction of high-rise steel-frame buildings, it is essential to secure the soundness of column-beam joints. Concerning the design of the Shinjuku Toho Building, we developed an "improved non-scallop on-site welding method" as the new beam end detail (Fig. 2). With this method, the plastic deformation capacity of the column-beam joints can be improved by filling up on-site weld scallops by means of welding and without the expansion of beam ends.

After tests using full-scale T-shaped specimens were conducted on the newly-developed welding method in order to confirm the high deformation capacity of the column-beam joints, the method was put into application. The newly-developed method offers high applicability for common uses and is expected to be used in many new building construction projects in the future. ■

Photo 1 Symbolic façade of south-side surface

Fig. 1 Response Control by the Use of CFT Columns Employing 780 N/mm^2-grade High-strength Steel

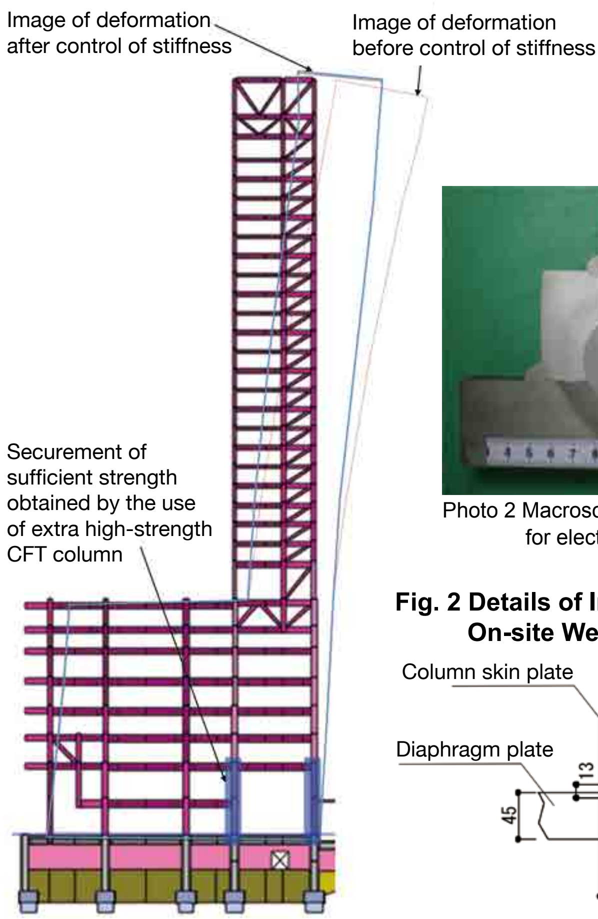
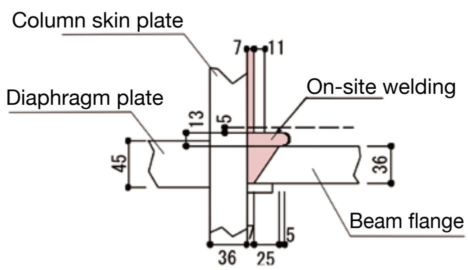


Photo 2 Macroscopic test results for electro-slag weld

Fig. 2 Details of Improved Non-scallop On-site Welding Method



[Tokyo Garden Terrace Kioicho] Kioi Tower

Prize-winners: Yuichi Koitabashi, Seiya Kimura, Nikken Sekkei Ltd.; and Kajima Corporation

In the plans for the [Tokyo Garden Terrace Kioicho] Kioi Tower, the following original ideas (A and B), devices and advanced measures were incorporated to handle the various tasks involved in the design and construction. These ideas led to an architecture that contributes to the development and diffusion of steel structures.(Photo 1)

Ultra High-rise Building with High Seismic Performance

A: Ensure high seismic performance in which the main structure will remain elastic and the floor response acceleration will be 250 gal or less during a major earthquake, including an inland earthquake in the Tokyo area, to enable the functions of the offices and hotel to be maintained continuously.

In this way, a high durability long life ultra high-rise building was realized in economizing on steel resources and minimizing damage to building materials from a long-term perspective. Therefore a new "high efficiency hybrid vibration control system" that combines the following 1) and 2), and other innovative measures were incorporated.

1) An innovative vibration control system was devised utilizing the properties of the building scheme that was capable of attaining high energy absorption efficiency and capable of reducing the seismic force (story shear force) on each story, using the deformation properties of a large structure consisting of the transfer truss structure and coupled steel plate walls. The energy absorption efficiency was increased by a maximum of about 50% compared with a normal ultra high-rise building, and the story shear forces were reduced to about 75% (Figs. 1~3).
2) Comprehensive monitoring was carried out such as measurement of floor response accelerations, displacements with the aim of achieving a long life steel structure building, to enable effective maintenance of the vibration control devices, which are indicative of the degree of damage in an earthquake.

High Efficiency Hybrid Vibration Control System

B: Scheme to reduce vibrations after a long period earthquake or a major earthquake

For this purpose an active mass damper (AMD) was installed on the roof. The AMD has an additional mechanism capable of reducing the amplitude of the vibrations after an earthquake by about 30 to 50% and shortening the time period of oscillation by about 3 minutes compared with no vibration control, with the aim of enabling rapid evacuation activities.

Fig. 1 Overview of Vibration Control System

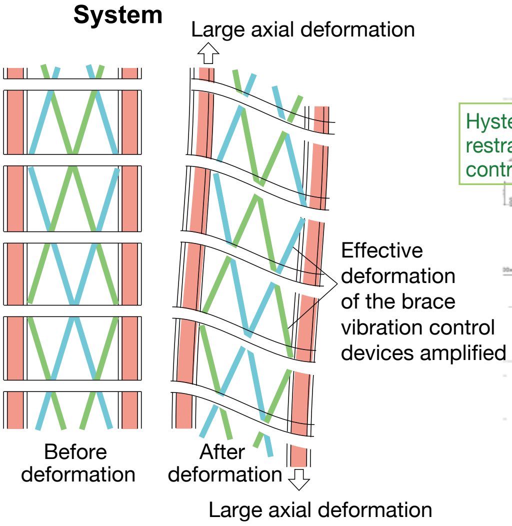


Fig. 2 Vibration Control Device Layout Diagram

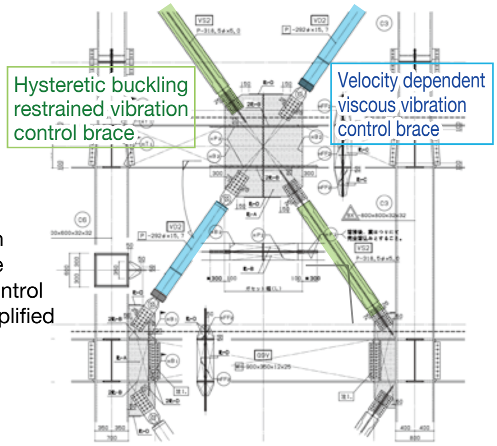


Fig. 3 Time History of Axial Forces Generated in the Vibration Control System (1 Unit)

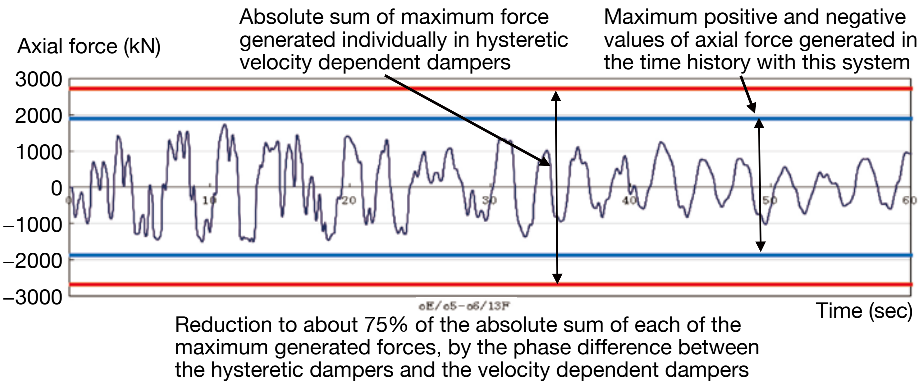


Photo 1 Full view of [Tokyo Garden Terrace Kioicho] Kioi Tower

Relationship of Macro Stress and Maximum Principal Stress on Electro-slag Weld Zone between Interior Diaphragm and Box Column

Prize-winner: Takumi Ishii, JFE Techno-Research Corporation

Takumi Ishii
1991: Graduated from Graduate School of Engineering, Chiba University; Entered Kawasaki Steel Corporation
2003: Civil Engineering Dept., JFE R&D Corporation
2009: Steel Research Laboratory, JFE Steel Corporation
2017: Structures Performance Center, JFE Techno-Research Corporation

Higher Toughness for ESW

The built-up box section column is manufactured by weld assembling four steel plates using electro-slag welding. In the electro-slag weld (hereinafter referred to as ESW) of the interior diaphragm and column skin plate of a column, it is worried that cracking will break out from the slit tip occurring between the backing metal and the column skin plate after welding (Fig. 1).

While there are no reports of actual damage caused by such cracking, it is clear from recent experiments that there is the possibility of brittle fractures occurring in the interior diaphragm ESW under specified conditions. Triggered by such experimental results, higher toughness is required of ESW.

Fig. 1 Outline of Column-Beam Connection of Box Column and Anxiety over Brittle Fracture Occurrence

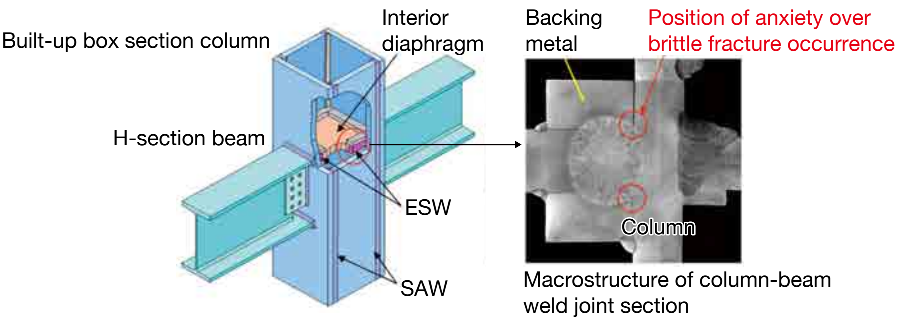


Fig. 2 Relationship between Equivalent Value of Maximum Principal Stress at Fracture Initiation Point and ESW Toughness Level

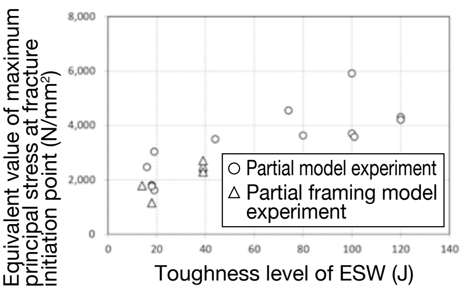
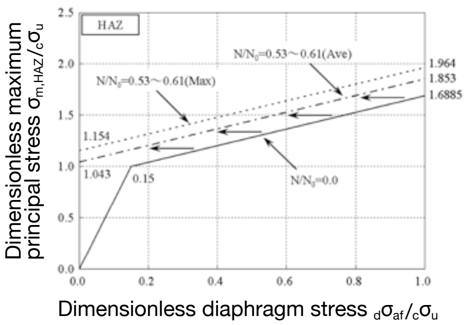


Fig. 3 Relationship between Dimensionless Maximum Principal Stress and Dimensionless Diaphragm Stress that Takes Penetration into Account (Conversion Curve)



Studies on ESW Fracture Toughness

Two studies on ESW fracture toughness were reported by us in past issues of *Steel Construction Engineering* of the Japanese Society of Steel Construction: “Study on Fracture Behaviors of Electro-slag Welds of Column-Beam Partial Models” (Vol. 16 (2009), No. 64) and “Study on Fracture Behaviors of Electro-slag Welds of Column-Beam Partial Framing Models” (Vol. 17 (2010), No. 68). In these studies, we noted the fracture toughness of the weld metal, fusion zone and heat-affected zone of ESW.

These two studies suggest that there is a positive mutual relationship between the toughness (that denotes the Charpy impact energy here) of the weld metal, fusion zone and heat-affected zone and the interior diaphragm stress. In addition, analytical examinations were made of the results of the experiments mentioned above, which find that the maximum principal stress is effective as the fracture toughness parameter that relates to the Charpy impact energy (Fig. 2).

Estimation of Toughness Required for ESW

Given this situation, the current study proposes a method that connects the critical maximum principal stress with the interior diaphragm stress, which is considered important in the process of calculating the toughness value required for ESW (Fig. 3). Concurrently, a method is proposed that simply calculates the stress working on the interior diaphragm of the column-beam connection.

By making the most of these proposals, the maximum principal stress is found from the stress working on the interior diaphragm of ESW without the use of the finite element analysis, and as a result it is now possible to estimate the required toughness in terms of the stress level of ESW. ■

Fatigue Crack Growth Rate of Steel under Large Cyclic Strain and Its Application to Crack Growth Prediction in Welded Joints

Prize-winners: Takeshi Hanji, Associate Professor, Nagoya University; Nao Terao, Graduate student, Nagoya University; Kazuo Tateishi, Professor, Nagoya University; and Masaru Shimizu, Assistant Professor, Nagoya University

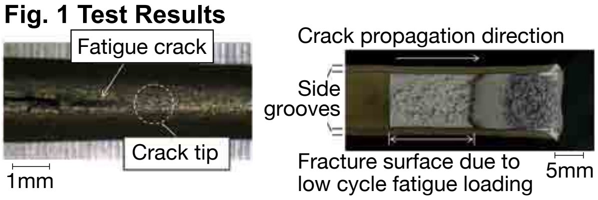
Takeshi Hanji
2001: Graduated from School of Engineering, Nagoya University
2006-2009: Researcher, University of California, San Diego; EcoTopia Science Institute, Nagoya University; Center for Urban Earthquake Engineering, Tokyo Institute of Technology
2010-: Associate Professor, Graduate School of Engineering, Nagoya University

Low cycle fatigue is one of the failure modes in steel structures during earthquakes. With a focus on crack growth in the low cycle fatigue region, this study developed a fatigue crack growth curve, and verified its applicability to crack growth prediction in welded joints.

Fatigue Crack Growth Tests

Fatigue crack growth tests under highly plastic conditions were performed using compact tension specimens with side grooves. Three kinds of materials (structural steels of 400 N/mm^2 and 490 N/mm^2 classes, and deposit metal) were used. Each specimen was loaded under displacement control. The range of fluctuation of the displacement was controlled during the test. The crack length was measured at regular intervals using a microscope. Fig. 1 (a) shows an example of a crack tip observed from the side surface of the specimen. A photograph of one of the fracture surfaces after the test is shown in Fig. 1 (b). These figures reveal that the crack propagated straight along the side groove, and that the location of the crack front was almost same throughout the thickness direction because of the side groove.

Fig. 1 Test Results



(a) Crack tip observed from side surface

(b) Fracture surface

Fatigue Crack Growth Rate Formula

Elasto-plastic finite element analysis was used to calculate the cyclic J-integral range at the cracks in the specimens. The cyclic J-integral range ΔJ was defined as the range of fluctuation of the J-integral from the minimum to the maximum load points of the loading process. The cyclic J-integral range is calculated as in the following:

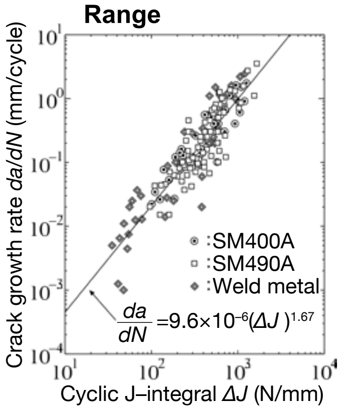
$$\Delta J = \int_{\Gamma}\left(W'dy - \Delta T \frac{\partial \Delta u}{\partial x} ds \right)$$

$$W' = \int_0^{\Delta\varepsilon_{ij}} \Delta\sigma_{ij} d\Delta\varepsilon_{ij}$$

where W' is the range of strain energy density, ΔT is the range of traction vector, Δu, Δσ and Δε are the ranges of displacement vector, stress and strain in the loading process, respectively.

Fig. 2 shows the relationship between the crack growth rate da/dN measured during the test and the cyclic J-integral range ΔJ calculated analytically. All plots were distributed in the same area, meaning that the cyclic J-integral range correlates with the crack growth rate regardless of the material. Based on the results, the crack growth rate formula can be derived.

Fig. 2 Crack Growth Rate vs Cyclic J-integral Range



Fatigue Crack Growth Prediction in Welded Joints

Low cycle fatigue tests were conducted using corner welded joints. The crack growth from the weld root in the joint was predicted to confirm the applicability of the proposed crack growth curve.

The crack growth in the test was estimated based on the cyclic J-integral range from the analysis and the proposed formula by the following steps:

(a) The initial crack length was defined and its cyclic J-integral was calculated based on the analysis.

(b) According to the proposed formula, crack growth length da due to one loading cycle (dN = 1) was calculated.

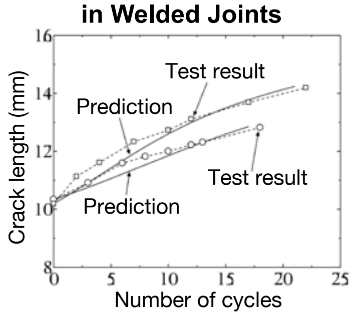
(c) The crack length a was updated to a+da, and again, its cyclic J-integral was calculated.

(d) The crack growth length da was recalculated using the formula.

The above steps were repeated.

Fig. 3 shows the prediction results. The predicted and observed crack growths were in good agreement. This demonstrates that the crack growth of the corner welded joints can be predicted using the cyclic J-integral range and the proposed crack growth formula in the low cycle fatigue region. ■

Fig. 3 Crack Growth Prediction in Welded Joints



Feature Articles: Japanese Steel Construction Technologies

In the following, examples are introduced in which Japanese steel construction technologies are applied in various countries, or examples of steel construction projects are introduced in which Japanese companies have contributed toward the successful completion of these projects.

Feature Articles: Japanese Steel Construction Technologies (1)

Passenger Terminal of New Doha International Airport

by Tsutomu Hirata, Taisei Corporation

Outline of Passenger Terminal Building

Location: Doha, Qatar
Total floor area: About 484,000 m^2
No. of stories: 5 stories aboveground
Type of structure: Steel structure (including concrete-filled box section columns)
Application: Airport facilities—passenger terminal and concourse
Project owner: New Doha International Airport Steering Committee
Construction management (CM): Overseas Bechtel Inc. (O.B.I.)
Design consulting: Hellmuth, Obata+Kassabaum (H.O.K.), Middlebrook+Louie (M+L, currently Louie International)
Construction: Sky Oryx Joint Venture (Taisei Corp. of Japan 65%, TAV of Turkey 35%)

Outline of New Doha International Airport

In Qatar, plans called for the construction of a new airport at a 29 km^2-spacious site (60% accounted for by reclaimed land) located south-east of Doha, the capital of Qatar. It is a leading airport in the Middle East that consists of a passenger terminal with an initial annual handling capacity of 24 million passengers, a cargo terminal with an annual handling capacity of 1.4 million tons of cargo, two runways each with a length exceeding 4,000 m, hangars, catering facilities, a control tower and a VIP terminal for the exclusive use of the royal family. The New Doha International Airport is a grand-scale national project composed of more than 100 contract packages.

The New Doha International Airport Terminal, among others, is the nucleus of the project (Photo 1). Its construction period ran about 64 months from March 2006 to July 2011. The airport was opened to the public in 2014, establishing it as a hub airport in the Middle East. The passenger terminal building was constructed by a joint consortium led by Taisei Corporation, a major general contractor in Japan, and a partner company from Turkey.

Outline of Construction of New Passenger Terminal

• Tendering Details of the Project

Taisei Corporation participated in the tender for the new airport terminal building construction by forming a joint venture (Sky Oryx Joint Venture) with TAV of Turkey, who are both experienced in airport construction and management. Pre-qualification started in April 2005, and in March 2006 the joint venture received a purchase order/construction start order, which led to the conclusion of a formal agreement between the New Doha International Airport Steering Committee and the Sky Oryx Joint Venture in June 2006.

• Outline of Terminal Building

The first-phase of the work covered the construction of the main terminal building composed of a main airport passenger terminal, a passenger concourse, elevated viaducts, pedestrian bridges and passenger boarding bridges. The main terminal building is a complex five-story steel structure with a wave-formed arch-shaped roof that is supported by part concrete-filled steel box section arch columns. The second-phase of the work consisted of an extended airport passenger concourse and girder framing for an indoor monorail that serves as a means

Photo 1 Full view of Passenger Terminal of New Doha International Airport in Qatar

of transportation from the main terminal building to the more distant departure gates. These facilities comprise a two-story steel-structure with a high vaulted roof.

The total floor area for the first to second-phase works is about 490,000 m^2 and the amount of steel used came to about 54,000 tons. While the construction work for the main terminal building had to overcome strict restrictions imposed by the harsh desert climate, logistic constraints and local practices, the main terminal building saw a full-scale opening on May 27, 2014.

• Steel Frame Construction

The main passenger terminal and its connecting concourse section have an approximately 1 km×1km square footprint with a total floor area of about 490,000 m^2. The arch columns, hybrid steel-composite structures, were adopted in constructing the passenger terminal and concourse section, where a large-span arch-shaped terminal building roof structure was built using part concrete-filled steel box section arch columns having span lengths of 160 m and 80 m. (Refer to Photo 2 and Fig. 1).

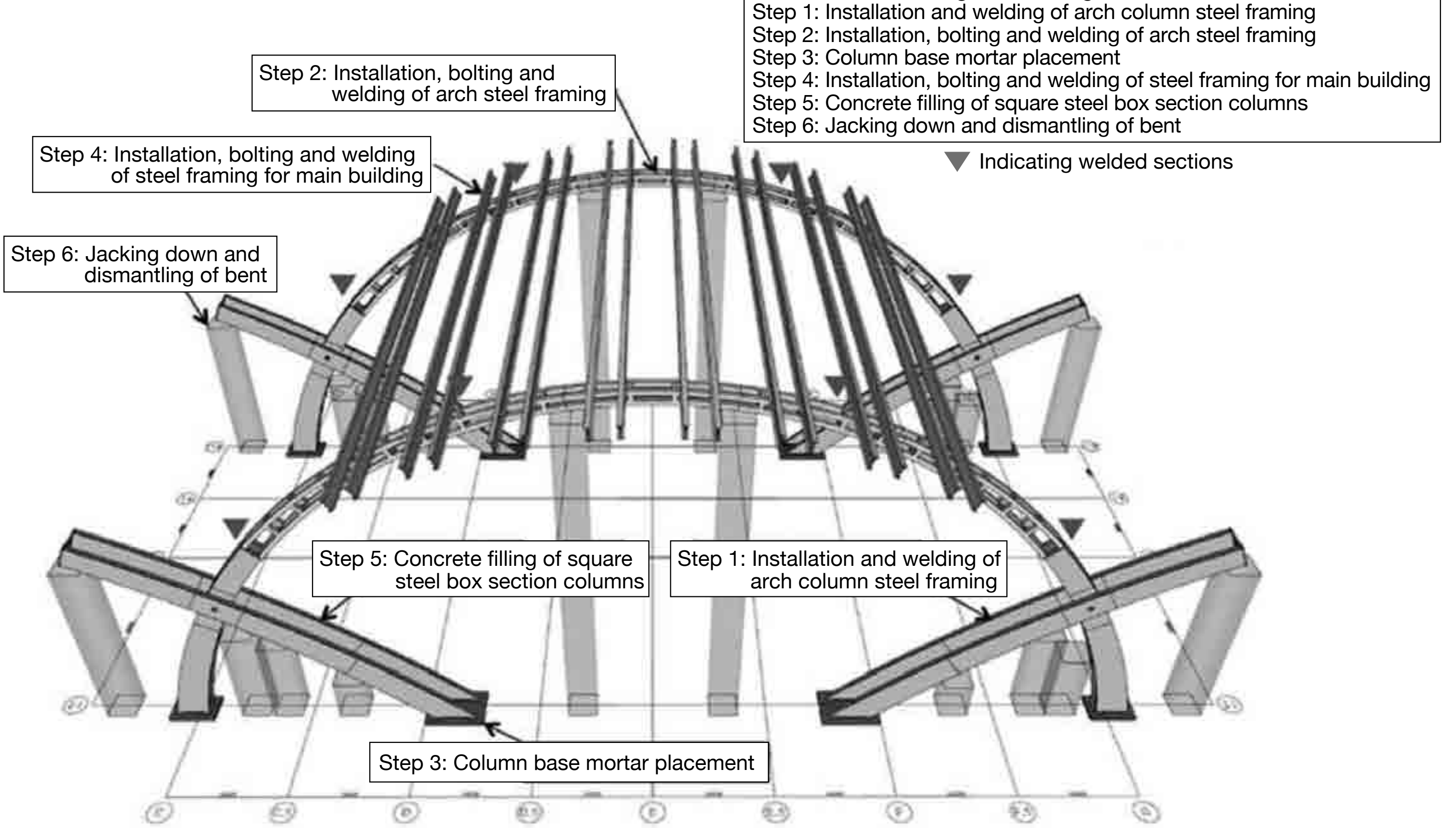
The upper arch-shaped hollow steel framing of the main building was installed on top of the lower concrete-filled box section arch columns. The length of steel-frame members for the main building measures from 40 m to 60 m in maximum. Three airport passenger concourses extend to the east, west and north sides of the terminal building.

All the arch-shaped steel framing was assembled on the ground, and the temporary support bent construction method was adopted for the erection of the steel framing.

During the construction of the huge terminal building, notable changes were made to the terminal design prior to and after the start of the project. To cope with this situation, it was required that the joint venture presented alternative proposals and construction plans applicable to the project. As a prime example, a proposal was submitted by the contractor prior to the start of the project that the original RC structure for the arch column lower sections be changed to a concrete-filled steel box column structure. In addition, it was required after the start of the project that an additional concourse be constructed. As it was necessary for the building contractor to be able to meet these design changes, the Sky Oryx Joint Venture successfully cleared all these requirements to complete the terminal building.

Photo 2 Installation of arch column steel framing

Fig. 1 Installation Plan for Arch Column Steel Framing



Collaborative System between Engineers from Many Countries

In this airport terminal project, as an outstandingly large-scale construction project to be undertaken in the Middle Eastern country of Qatar, it was considered necessary to establish a multinational collaborative system for those working on the project in order to build it efficiently.

About ten engineers and architects in charge of the contractor's design management were stationed on site to proceed with the project. They came from Turkey, the Philippines, Egypt and Japan. The persons working in the construction management (Overseas Bechtel, Inc.) and design (Hellmuth, Obata＋Kassabaum and Middlebrook+Louie) teams were mostly Americans, and the company working on the steel-frame manufacture (EVS Metal Inc.) was an Indian company. The sense of values of these engineers and architects and the languages they each used were different. One of the important roles played by the Sky Oryx Joint Venture was to guide the multinational engineers and architects towards one purpose, the successful completion of the project. (Refer to Photo 3)

Photo 3 On-site confirmation of arch column steel framing by engineers

Accurate Response to Technological Issues

Diverse kinds of technological issues occurred during the terminal construction. The steel products used for construction were imported from various nations, and thus there were many cases in which quality, the dimensions and other specifications could not be easily and correctly judged simply by examining different mill sheets. Further, an important role was imposed on the construction engineer who managed the entire construction project—to judge whether or not the specifications of the structural materials, prescribed in the structural drawings prepared by the structural design office, were appropriate. On top of these, because diverse collaborative design offices were used in the operations depending on the engineering capabilities and the specialized field, it was necessary to prudently undertake the choice of collaborative engineering offices at the site.

In spite of technological issues such as these, the terminal building was completed without major issues, supported by the technological strengths accumulated by the Sky Oryx Joint Venture. In this regard, in order to successfully promote construction projects overseas, it is considered important to implement design verification from multilateral viewpoints and to base them on the on-site conditions.

Overcoming Difficult Work in the Desert by the Optimum Use of Technological Strengths

The project of constructing a passenger terminal having an originally planned total floor area of 239,000 m^2 started with an initially planned construction period of three years. Finally, a terminal having a total floor area of 484,000 m^2 was completed in a construction period of more than five years (Photos 4 and 5).

Nearly 20,000 workers from 25 countries participated in the construction of the terminal building. The Sky Oryx Joint Venture dealt with complex process control by promoting communications among workers from different countries by respecting their various cultures, religions and habits. As regards the influx of construction materials supplied by various countries, the joint venture closely monitored the unsettled delivery terms. In addition, the contractor was faced with difficulties in accurately following the construction schedule of such a large-scale project and efficiently attaining the daily targets.

Even in the face of such hard conditions, the Sky Oryx Joint Venture joined forces to proceed with the new airport terminal project—transferring to the workers the basic knowledge accumulated by experienced Japanese engineers, training the workers in such basic knowledge, accurately coordinating the entire project, and implementing the basic practices involved in construction management. It is believed that these daily accumulated efforts produced the positive results that led to the successful completion of the terminal building of the New Doha International Airport in compliance with the high expectations of the people of Qatar, and the supervising design and CM teams. ■

Photo 4 Top light of main terminal building

Photo 5 The huge-scale terminal project was successfully completed by the joint efforts of workers and engineers from many countries.

Taipei Nanshan Plaza Project in Taiwan

by Hiroshi Kawamura, Mitsubishi Jisho Sekkei Inc.

Outline of the Project

The Taipei Nanshan Plaza is a large-scale project composed of complex facilities that are located in the central area of Taipei, Taiwan. The project is located at a vacant lot formerly used as an exhibition site and adjoins the Taipei 101, a 101-story high-rise building with a height of 509 m, the tallest in Taiwan. After completion of the current project, the pair consisting of Taipei Nanshan Plaza and Taipei 101 will constitute a new landmark in Taipei (Fig. 1).

Mitsubishi Jisho Sekkei Inc. jointly with its client, Nan Shan Life Insurance Company Ltd., participated in the project competition held by the Taiwan City Government in 2012, in which our proposal was adopted after winning first place in the competition. Nan Shan Life Insurance Company, a major Taiwanese life insurance company, is the owner and developer of the project. The company leases the land from the Taiwan Government and also develops and operates the Taipei Nanshan Plaza.

The Taipei Nanshan Plaza project is composed of three buildings: an office tower with a height of 272 m with restaurants in the three topmost floors, a retail building housing top-brand shops, and a cultural building where various events and exhibitions are to be held. Bus terminals are located in the underground floor of the cultural building. These three buildings with a total floor area of about 200,000 m^2 are designed and constructed as one unified building complex.

Construction started at the end of 2013. Currently, the project is underway with completion scheduled for the end of March 2018. The Taipei Nanshan Plaza is becoming the focus of public attention in Taipei. Table 1 shows an outline of the three buildings.

Design Approach

While Nan Shan Life Insurance was pro-Japanese and possessed a positive outlook in incorporating advanced technologies from Japan into the design of the project, Mitsubishi Jisho Sekkei did not particularly impose Japanese styles on its client, but rather respected local customs, laws, regulations and procurement procedures. In light of this, Mitsubishi Jisho Sekkei conducted advance exchanges of opinion with local engineers and authorities when examining the various proposals for the Taipei Nanshan Plaza project, and when explaining these proposals to the client, we stressed that the proposals were prepared after consulting with local engineers and authorities.

As a result of these processes, Mitsubishi Jisho Sekkei was able to successfully establish a highly reliable relationship with the client and the local engineers, which led to a structural design that imparts a unique configuration and a striking design concept to each building.

Fig. 1 Image of a Pair of Landmark Towers in Taipei: Taipei Nanshan Plaza (left) and Taipei 101

Table 1 Outline of Three Buildings in Taipei Nanshan Plaza

	Office tower	Retail building	Cultural building
Location	Sung Jen Road, Xinyi District, Taipei, Republic of China (Taiwan)		
Main applications	Office and carpark	Shop and carpark	Cultural facility and entrance to office tower
Site area	17,708.00 m^2		
Building area	10,271.41 m^2		
Total floor area	192,891.35 m^2		
No. of stories	5 stories underground, 48 stories aboveground, 2-story penthouse	5 stories underground, 9 stories aboveground	5 stories underground, 3 stories aboveground
Maximum height	272.00 m	56.75 m	24.61 m
Type of structure	Aboveground: Steel structure (~36 floors: CFT column) Underground: SRC structure	Aboveground: Steel structure (CFT column) Underground: SRC structure	Aboveground: Steel structure (partially CFT column) Underground: SRC structure
Grade of steelwork	SN490B, SN490C, SM570C; Maximum plate thickness: 70 mm; Maximum column size: 2000 mm×1400 mm		
Project owner	Nan Shan Life Insurance Company Ltd.		
Scheme design/Supervision of design development and detail design	Mitsubishi Jisho Sekkei Inc.		
Design development and detail design	Architect: Archasia Design Group; Structural engineer: Evergreen Consulting Engineering, Inc.		
Contractor	Fu Tsu Construction Co., Ltd.; Steelwork fabricator: Chun Yuan Steel Industry Co., Ltd.		

Structural Design Concept

• Office Tower

The office tower has 48 stories aboveground and a height of 272 m, and the ratio of height to breadth is about 5.8 (Photo 1). The frame is a double-tube structure composed of a core in which braces (5-48 floors) and steel plate seismic-resistant walls (1-4 floors) are arranged in the short-side direction and an outer peripheral frame. With regard to the overall appearance of the office tower building, the building's frame is designed to promote the idea of two joined palms (Fig. 2), and with regard to the outer appearance of the low-rise section, the frame is designed to offer a unique configuration in which the outer wall surface becomes narrower toward the bottom (Fig. 3).

Meanwhile, in order to secure both horizontal and torsional rigidity for the building structure, truss floors were provided on the top and intermediate parts and the lower part where the perimeter columns are bent (Fig. 3). Intermediate machine rooms are located on these truss floors with belt trusses used for the outer periphery and outrigger trusses for the interior sections.

In the office tower building, the wind load is more predominant than the seismic load, and in order to suppress the vibration caused by the wind load, two tuned mass dampers (TMDs) weighing 250 tons each were installed on the upper truss floor.

At the topmost section of the office tower building, sky restaurants having a floor-to-floor height of more than 7.2 m were planned for each of the three topmost floors. Because the building exterior is designed based on the underlying concept of a vertical composition, it was required for the exterior of the topmost section to possess high transparency that would allow for a fine view. Accordingly, we proposed a framing structure in which diagonal kickers and horizontal trusses would be installed, thereby allowing the client's wishes to be successfully put into effect. (Refer to Photo 2 and Fig. 4).

Photo 1 Erection of office tower*

Fig. 2 Image of Joining Together of Palms to Express Thankfulness

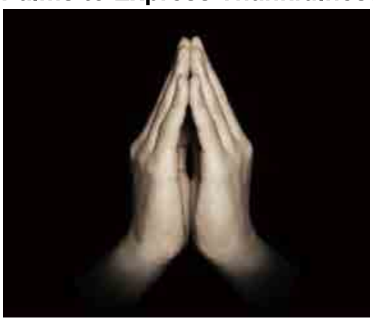
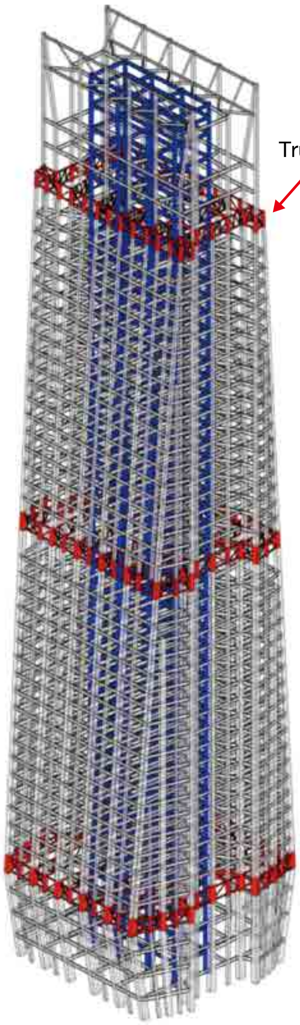


Fig. 3 Framing Model of Office Tower



• Retail Building

The retail building is a nine-story aboveground structure with a height of about 57 m. It presents an image in which jewel boxes are piled in shifted forms at each floor (Photo 3). The approximate size of the floor plate is 66 m×50 m, and a moment frame steel structure was applied that adopts a basic module measuring more or less 11 m×8 m in conformity with a floor-to-floor height of 6~7.2 m.

Photo 2 Erection of topmost section of office tower*

Fig. 4 Framing of Topmost Section of Office Tower

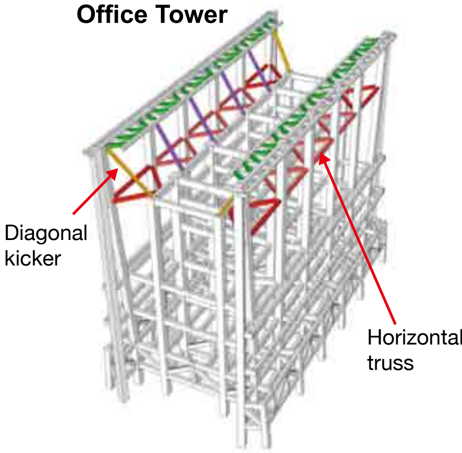


Photo 3 Erection of retail building

The overhanging lengths that occur due to the shifting of floor units at each floor is about 10 m in maximum, and diverse methods were effectively applied to install the suspended frame using triangle-shaped trusses, Vierendeel trusses or cantilevered beams depending on the respective overhanging lengths. (Refer to Fig. 5)

For the 1~3 floors of the retail building that connects with the office tower building, a structural form was devised in which these three floors are put onto the corbel provided in the office tower and bearings having a sliding mechanism were installed on the corbel. The plan for the exterior of the retail building called for a structural system in which the entire building was covered by a screen of Taiwan's national flower plum-blossom patterns (Photo 4), that was manufactured by three-dimensionally bending a stainless steel-formed material. As regards the framing that supports the complex exterior members, a stress analysis was made to examine its applicability.

• Cultural Building

The cultural building has three stories aboveground and a height of about 25 m. It is composed of unique polyhedrons having an octagonal plane shape. The building is used as both an entrance to the office tower and an event hall. The roof and exterior walls are composed of titanium panels. (Refer to Photo 5 and Fig. 6)

Users and visitors come up to the second floor by an escalator that is installed in the first-floor atrium hall. The second floor connects to the office tower and also serves as an access way to the event hall on the third floor.

The event hall on the third floor is a 30 m×28 m column-free space arranged on top of the first-floor atrium hall. It was examined whether or not the outer configuration with its unique architectural design could be fused with the building structure, and then we proposed a framing type in which the seismic load is dealt with by using an outer shell in the long-side direction and the installation of inner columns is reduced to a minimum. The upper chord of the triangle-shaped trusses on the third floor functions as a structural foundation of the outer shell, on which parallel arch members sit in 3-m spacing in the short-side direction (Fig. 7).

In the underground space beneath the cultural building are located a large-size bus parking lot and traffic lanes, and as a result, arrangement of supporting points for the superstructure were limited. Due to these needs, the complex structure discussed above was adopted for the framing of the cultural building. Discussions on the practical application of this complex structure were made between the local structural engineers and Mitsubishi Jisho Sekkei to reconcile proposals and plans.

Acknowledgments

I would like to express my sincere gratitude to our client, Nan Shan Life Insurance Company Ltd., who gave us extensive guidance regarding the current project, and also to Evergreen Consulting Engineering, Inc., who gave us kind support. ■

**Photos by courtesy of:*

Photo 1 by Fu Tsu Construction, Photos 2 and 6 by YKK-AP, Photo 4 by Kinzi

Note

The current article was prepared based on the progress of the project as of November 2017.

Photo 5 Erection of cultural building

Fig. 5 Framing Model of Retail Building

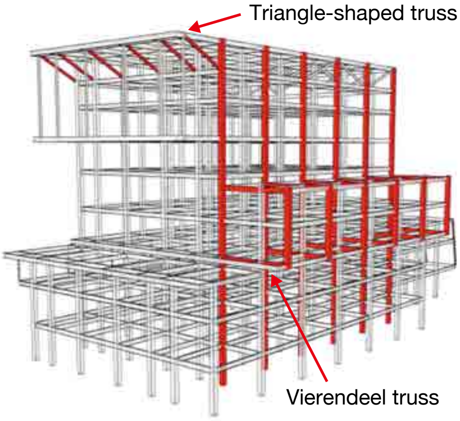


Fig. 6 Image of Cultural Building Serving also as Office Tower Entrance

Fig. 7 Framing Model of Cultural Building

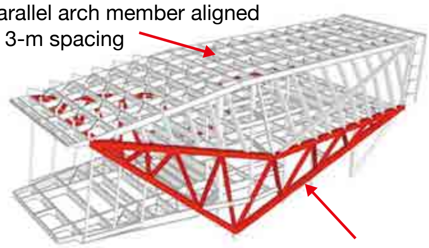


Photo 4 Screen having plum-blossom patterns*

Photo 6 Distant view of Taipei Nanshan Plaza from Elephant Mountain in Taipei*

Tsubasa Bridge in Cambodia

by Masahiro Nomoto, Chodai Co., Ltd.

Project Outline

National Road No.1 (NR1) in Cambodia is a part of the Asian Highway and Southern Economic Corridor which connects Ho Chi Minh City and Bangkok via Phnom Penh. As the economy of the surrounding areas developed, traffic jams due to increased traffic demand became a huge problem. Especially the Mekong River crossing area was a traffic bottleneck because of the low capacity of the ferries used to cross the river. Tsubasa Bridge was thus constructed as a Japanese grant aid project to alleviate this bottleneck. (Refer to Figs. 1 and 2, Photo 1, Table 1)

This article describes the design characteristics and construction method of the PC cable-stayed bridge, the main bridge of the project.

Design Phase

• Edge Girder Bridge

An edge girder was selected as the main girder type for the PC cable-stayed bridge in consideration of the scale of the bridge (13.5 m effective road width).

Edge girders are more cost-effective than box girders since they are easier to reduce the dead load resulting from the concrete volume (Fig. 3 and Photo 2).

They are also superior in assuring quality in countries with less developed technology, because they enable easier formwork and bar arrangement work through a simpler girder design and shape.

Edge girders can be affected by torsion flutter at low wind speeds due to the shape of the main girder because of their low torsion rigidity compared to box girders. Therefore, wind tunnel tests were conducted in order to determine a main girder cross-sectional shape with high wind resistance and stability.

• Wind Tunnel Tests

To select a cross-section for the main girder with high wind resistance and sta-

Fig. 3 Edge Girder Dimensions

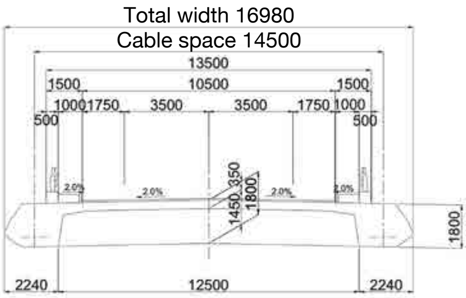


Photo 2 Edge girder

Photo 1 Full view of Tsubasa Bridge

Table 1 Project Outline	
Project length	5.500 km
Bridge type	3-span continuous PC cable-stayed bridge
Foundation type	Hypostyle foundation
Spans	160 m+330 m+160 m
Bridge width	13.500 m
Design velocity	80 km/h

Fig. 1 Location Map

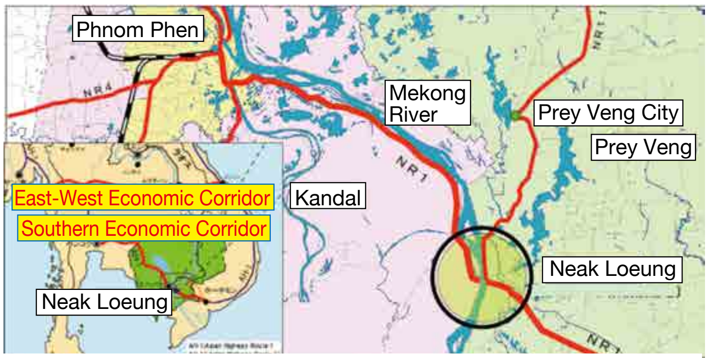
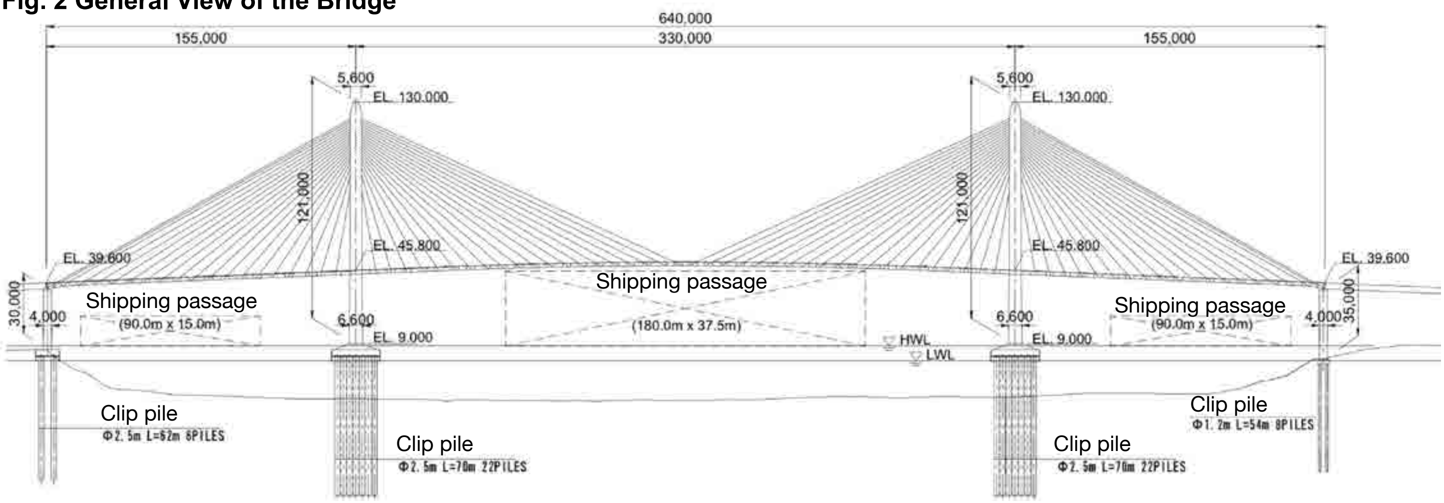


Fig. 2 General View of the Bridge



bility, wind tunnel tests were conducted not only for the original cross-section but also for 3 other cross-sections with different fairings using 1/60 scale models (Table 2 and Photo 3).

In the experiment, a uniform wind flow with −3, 0, and +3 degree angles of attack was applied to the spring supported partial model with 2 degrees of freedom of vertical deflection and torsion.

Deflection vortex excitation and torsion flutter occurred on the original cross-section with a −3 degree angle of attack (downward wind direction). Although the amplitude of deflection vortex excitation was less than allowed, torsion flutter occurred at a wind speed of 44.0 m/s which is smaller than the verifying wind speed of 52.9 m/s. Therefore, the same experiment was conducted with different shapes of fairings and a (small) fairing was adopted which did not suffer from flutter at an 80 m/s wind speed and showed little deflection vortex excitation. (Refer to Figs. 4, 5 and 6)

Table 2 Wind Tunnel Test Conditions

Structural damping ratio	0.02
Design velocity	U_{10}=30 m/s
Degree of roughness	0.16
Girder altitude	45.8 m
Target wind velocity	38.3 m/s

Photo 3 Wind tunnel test

• Stay Cables

Taking into account the benefit of wind-resistant stability, a parallel stay cable arrangement was adopted. Cables which are vertically arranged in the cross-section help mitigate the sense of confinement that pedestrians may feel and also ensure easier constructability during stay cable installation.

The vertical spacing of the stay cables is between 1.25 m and 2.3 m on the towers and 8 m on the concrete deck.

Non-grouting site assembly cables were selected considering their advantages which include easy transportation and installation and reliable rust-proofing.

• Towers

The towers are H-shaped to conform with the parallel out-plane arrangement of the stay cables (Fig. 7). An RC structure was adopted for the shafts which are always under compression, while a PC structure with a hollow cross-section was adopted for the cross beams which connect the shafts.

The shaft cross-section and the shape of the cross beams and the top of the towers were examined for their aesthetic qualities.

The stay cable anchorages are a separate type which makes maintenance work easier. U-shaped PC strands and PC bars were used to reinforce the anchorage zones (Fig. 8).

Construction Phase

• Unexploded Ordnance

Although about 4,200 unexploded ordnances (UXOs) were removed with the cooperation of the Cambodian military before the start of construction, an explosion still occurred during the construction of the foundation piles of the cable-stayed

Fig. 4 Fairings for Wind Tunnel Testing

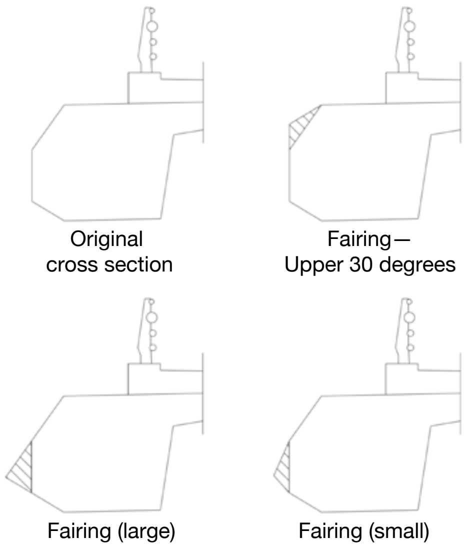


Fig. 7 Front View of H-shaped Tower

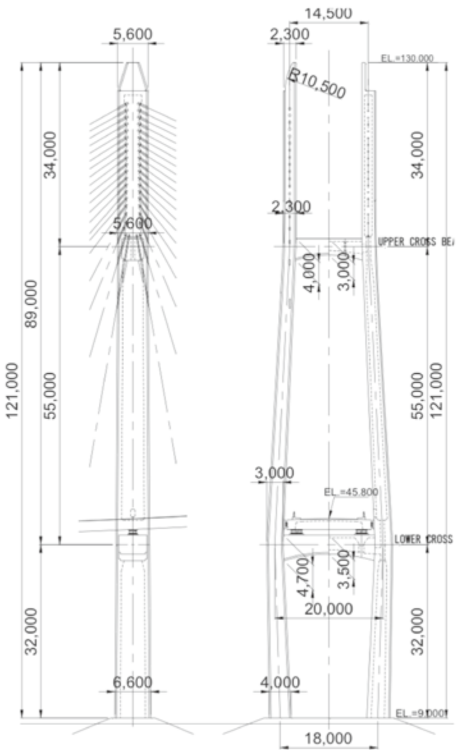


Fig. 5 Wind Speed-Amplitude Relation (Torsion −3 deg.)

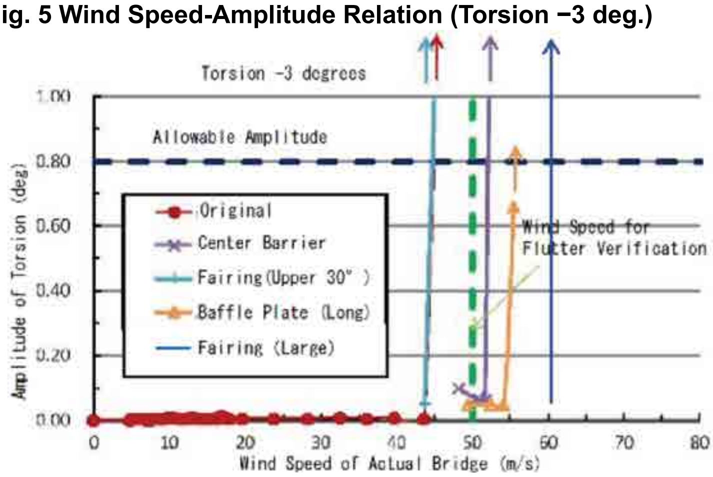
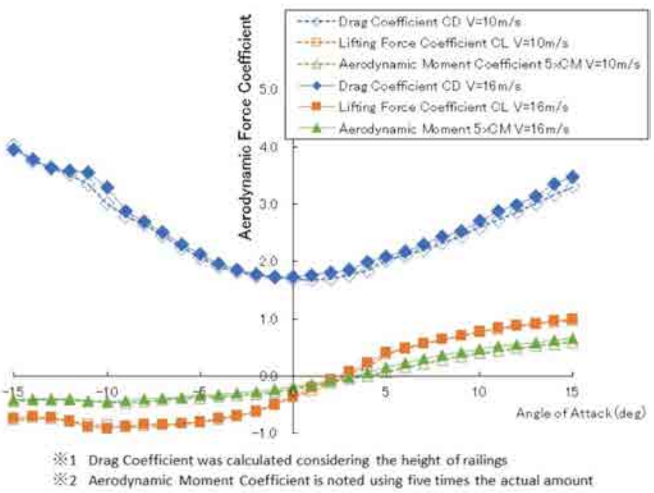


Fig. 6 Static Three-component Force Test Results



bridge. Luckily no one was hurt, but a re-investigation and the adoption of necessary countermeasures led to a 4-month suspension of construction work. In order to recover this delay and shorten the construction period, the methods described below were adopted which enabled construction to be completed on time.

• Sliding Form and Prefab Construction

For tower construction, a jumping form system with integrated formwork and scaffolding and prefabricated rebars was adopted in order to save work and shorten the construction period (Photos 4 and 5).

• Adoption of 8 m Blocks

8 m blocks were adopted with an underslung form traveler for the main girder cantilever construction which enabled a basic construction cycle of 10 days (Photo 6 and Fig. 9). For this method, countermeasures such as prefabricated rebar for the cross girders of the main girders were taken (Photo 7). Counter weights were also applied on the opposite end of the cantilever in order to prevent an unbalanced situation which was an adverse reaction from the large-scale form traveler.

Conclusion

The opening of Tsubasa Bridge shortened the time needed to cross the river from 7-8 hours in busy seasons to only a few minutes. The fact that the Asian Highway was connected by road had a big role in improving the economy and quality of life of surrounding areas. Thorough discussions were held with the residents around the river from the route selection stage and the project was carefully planned to ensure the full comprehension and acceptance of the various project stakeholders in line with Japan's official ODA policy. ■


Photo 4 Jumping form system


Photo 5 Prefab rebar for tower

Fig. 8 Stay Cable Anchorage Zone

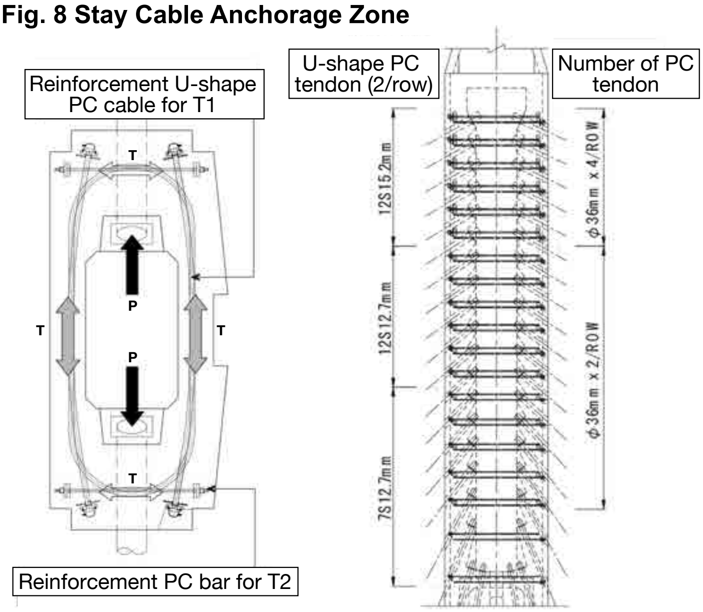



Photo 6 Form traveler for 8 m blocks

Fig. 9 Structural Drawing of Form Traveler

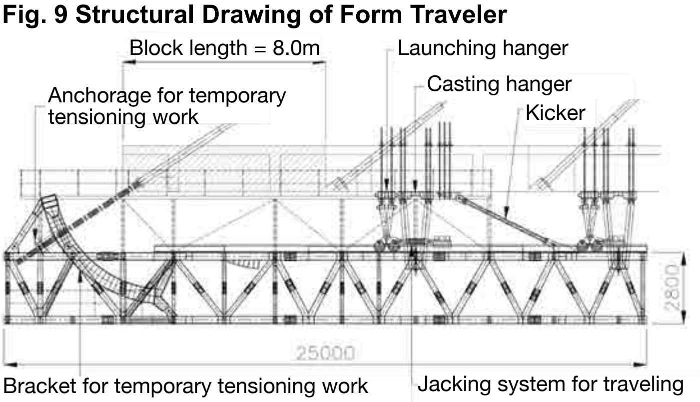



Photo 7 Prefab rebars for cross beam

Lean Duplex Stainless Steel for Selective Water Intake Facility

by Yasuhiro Tsuruga and Noriaki Fukushima, IHI Infrastructure Systems Co., Ltd.

Mitigation of Burdens and Costs

The Futase Dam is a multipurpose dam constructed in 1961 with the aim of controlling floods, securing agricultural irrigation water and operating a local government-owned hydropower station. In order to reduce the discharge of cold and turbid water from the dam into the river below, a selective water intake facility was additionally installed at the inflow section of the water conduit pipe for power generation.

Because of the additional installation at the existing gravity arch dam, a selective water intake facility of the multi-stage rubber membrane type was adopted with the aim of mitigating the burden placed on the existing dam body and reducing costs. Fig. 1 shows an entire dam body, and Fig. 2 an overview of the selective water intake facility.

Table 1 Outline of Selective Water Intake Facility of Futase Dam

Order placement	Kanto Regional Development Bureau, Ministry of Land, Infrastructure, Transport and Tourism
Project name	Installation of selective water intake facility at Futase Dam
Facility	Selective water intake facility of the multi-stage rubber membrane type×1 Water intake amount: 7.5 m³/s; number of stage: 8; water intake range: 40 m
Installation site	Ootaki, Chichibu, Saitama Pref.
Construction term	January 2014~June 2016
Structural materials applied	SUS304, SUS821L1

Fig. 1 Drawing of Entire Dam Structure (seen from upstream side)

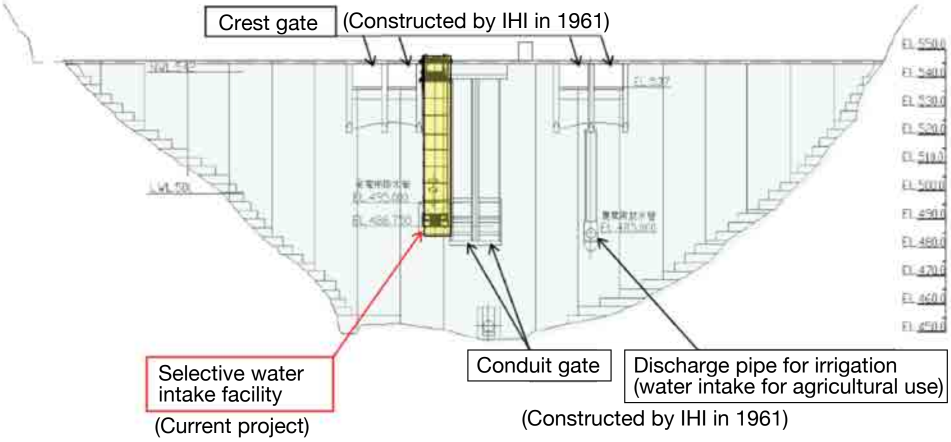
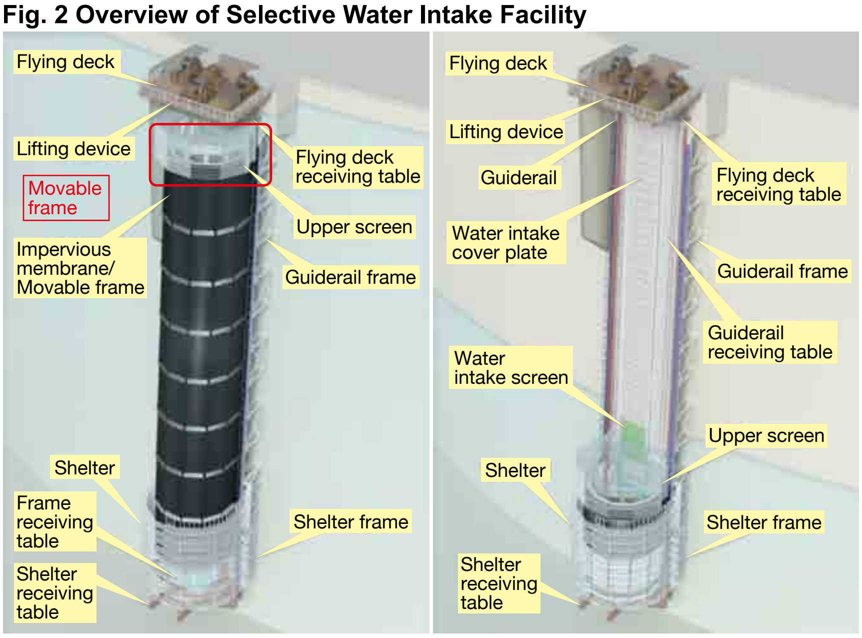


Fig. 2 Overview of Selective Water Intake Facility



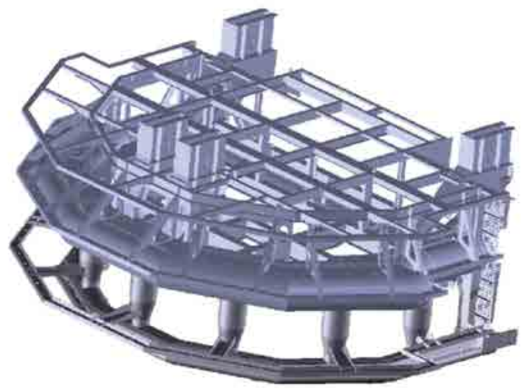
(Full expansion state) (Full contraction state)

Outline of the Selective Water Intake Facility

Table 1 shows an outline of the selective water intake facility at Futase Dam. The selective water intake facility is composed mainly of an expansion/contraction gate leaf (impervious membrane, movable frame), a guiderail serving as the lifting guide for the gate leaf, a wire rope-type lifting device, and a flying deck. The gate leaf is supported by the lifting device, and the lifting device is supported by the existing dam body via the flying deck.

The movable frame of the gate leaf is a welded structure. Because of restrictions imposed on its transportation, the movable frame was divided into several blocks, which were carried into the site and assembled by means of on-site welding. Fig. 3 shows a three-dimensional drawing of the movable frame.

Fig. 3 Three-dimensional Drawing of Movable Frame



Features of Lean Duplex Stainless Steel

The application of duplex stainless is growing worldwide. In order to incorporate the attainments obtained in the development of the new duplex stainless steel, lean duplex stainless steel was standardized in the Japanese Industrial Standards in 2015. Currently two steel grades, SUS821L1 and SUS323L, are registered as lean duplex stainless steel in the Standards.

In comparison with the conventional duplex stainless steel SUS329J4L, lean duplex stainless steel is manufactured by suppressing the addition of the expensive elements Ni and Mo and adding N to the extent that it does not affect weldability. The end result thus attained for lean duplex stainless steel is the realization of high strength, which is about twice that of SUS304, and economic advantages, corrosion resistance and weldability similar to those of SUS304. Table 2 shows the main composition and mechanical properties of lean duplex stainless steel.

Application of Lean Duplex Stainless Steel

The gate leaf of the selective water intake facility constantly remains in water. Accordingly, from the aspect of securing corrosion resistance and reducing the lifecycle cost of the selective water intake facility, it has recently become common practice to adopt stainless steel for such an application. In order to further reduce the burden placed on the existing dam body, lean duplex stainless steel SUS821L1 that has a strength about twice that of SUS304 and serves as a substitute for SUS304 was adopted for the movable frame of the gate leaf, which successfully led to a further reduction of the structural weight of the gate leaf.

In adopting SUS821L1 for the movable frame of the gate leaf, both factory and on-site welding tests were carried out to confirm its weldability and the validity of the welding conditions. In addition, corrosion resistance tests were implemented for welds to confirm that the corrosion resistance of the welds of SUS821L1 is similar to or higher than that of SUS304.

Photo 1 shows the trial assembly of gate leaf (movable frame) for which SUS821L1 is adopted, and Photo 2 the installation of gate leaf.

Low Burdens on and Reduced Expense of Civil Engineering Structures

The structural weight of the movable frame could be reduced by about 20% by adopting SUS821L1 lean duplex stainless steel, and the lightweight structure thus attained has led to a reduction of the burden placed on the existing dam body. Photo 3 shows the full view of the Futase Dam after completion of the selective water intake facility.

The development of lean duplex stainless steel, SUS821L1 and SUS323L, has allowed it to be used as a substitute for conventional stainless steel, SUS304 and SUS316, and in the installation of lightweight dam structures as well.

In the construction of gate facilities, because these lightweight gate leaves facilitate not only a reduction of the capacity of hoist and its downsizing but also mitigate the burden placed on dam bodies and other civil engineering structures, in recent years SUS821L1 and SUS323L are increasingly being applied in the construction of gate facilities on rivers and in dams. In the future, it is forecasted that the need will further grow for the effective utilization of existing infrastructures such as the redevelopment of existing dams, measures to cope with the superannuation of existing infrastructures and measures to enhance their seismic resistance.

Given these circumstances, expectations are high for the use of lean duplex stainless steel, SUS821L1 and SUS323L, as structural materials that will contribute towards the reduction of public works expenditures on such items as gate facilities and other civil engineering structures. ■

Table 2 Main Compositions and Mechanical Properties of Stainless Steel

Kind	Steel grade	Main compositions (mass %)	0.2% offset yield strength (N/mm²)	Tensile strength (N/mm²)
Lean duplex stainless steel	SUS821L1	21Cr-2Ni-0.17N	≧400	≧600
	SUS323L	23Cr-4Ni-0.15N	≧400	≧600
Duplex stainless steel	SUS329J4L	25Cr-7Ni-3Mo	≧450	≧620
Austenitic stainless steel	SUS304	18Cr-8Ni	≧205	≧520
	SUS316L	18Cr-12Ni-2Mo	≧175	≧480

Photo 1 Factory temporary assembly of gate leaf (movable frame) employing SUS821L1

Photo 2 Installation of gate leaf (movable frame)

Photo 3 Full view of Futase Dam after completion of selective water intake facility

PSSC2019 to Be Held in Japan

The Pacific Council of Structural Steel Associations (PCSSA) is an organization in which the Japanese Society of Steel Construction and other steel construction-related associations of the Pacific rim participate. PCSSA not only serves as a site for promoting exchanges among its participating organizations and solving common tasks but also functions as a nucleus for promoting the Pacific Structural Steel Conference (PSSC) that is hosted every three years on an alternating basis among the participating nations.

The first PSSC was held in New Zealand in 1986 followed by ten subsequent PSSCs in other countries (see the table at right). The previous 11th PSSC was held in Shanghai, China in October 2016, and on that occasion with representatives from various countries, it was decided to hold the 12th PSSC in Japan in 2019. In this connection, the ceremony to deliver the PSSC flag from China to Japan was held (see the photo). The 12th conference will be the first one held in Japan since the third one 27 years ago in 1992. According to the record, the participating counties numbered seven in 1992, but in recent conferences the number has increased to 11 (see the table at right).

In order to take all possible measures for a successful 12th PSSC, the Japanese Society of Steel Construction established the Specialized Committee for Arranging PSSC2019 which is chaired by JSSC President Yozo Fujino and has started preparations for the PSSC2019 that is planned for November 2019 in Tokyo. In 2020, the Tokyo Olympic and Paralympic Games will be held. In light of this, PSSC2019 is expected to present a good opportunity for the Japanese steel construction-related companies to disseminate their world-class technologies and for its young researchers and engineers to deepen exchanges with overseas researchers and engineers.

Outline of Pacific Structural Steel Conferences (PSSC)

Year	PSSC	Host country
1986	1st	New Zealand
1989	2nd	Australia
1992	3rd	Japan (Tokyo)
1995	4th	Singapore
1998	5th	Korea (Seoul)
2001	6th	China (Beijing)
2004	7th	USA (Long Beach)
2007	8th	New Zealand (Taupo)
2010	9th	China (Beijing)
2013	10th	Singapore
2016	11th	China (Shanghai)
2019	12th	Japan (Tokyo); Planned

Eleven Organizations in Eleven Participating Countries

- American Institute of Steel Construction (AISC)
- Australian Institute of Steel Construction (AISC)
- Canadian Institute of Steel Construction (CISC)
- Chilean Steel Institute
- China Steel Construction Society
- Japanese Society of Steel Construction (JSSC)
- Korean Society of Steel Construction (KSSC)
- Mexican Institute of Steel Construction
- Malaysian Structural Steel Association
- Steel Construction New Zealand
- Singapore Structural Steel Society (SSSS)

Ceremony to deliver the PSSC flag from China to Japan

Construction is proceeding of the New National Stadium for the Tokyo 2020 Olympic and Paralympic Games (as of February 2018). Courtesy of JSC

2017 China-Japan-Korea Tall Building Forum in Chongqing, China

The 2017 China-Japan-Korea Tall Building Forum was held on September 21 and 22, 2017 at The Westin Chongqing Liberation Square in Chongqing, China. The current forum was held jointly with other tall building-related symposiums under the common name of 2017 Super Tall Building Industry International Summit. The total participants numbered nearly 500 thereby resulting in a large-scale international conference.

In the day-and-a-half forum, diverse kinds of lectures concerning tall buildings were delivered by 28 lecturers such as land developers, architects and structural engineers from Japan, China, Korea, the US and Russia.

A total of 12 structural engineers from Japan, with Chairman Akira Wada of the CTBUH Japan Structures Committee of the Japanese Society of Steel Construction as their representative, participated in the forum. Of these engineers, four delivered lectures concerning the structural design of seismic-isolation and response-control tall buildings and a rapid

(continued overleaf)

diagnosis system of tall buildings during earthquakes to demonstrate advanced wind- and seismic-resistant technologies in the field of tall building construction in Japan.

In the afternoon of the second day of the forum, a Technical Tour was made to the site of Raffles City Chongqing that was under construction in Chaotianmen, Chongqing.

Raffles City Chongqing is a gigantic-scale development project that aims to construct complex facilities (hotel, residence, office and retail) with a total floor area of 1.13 million m^2. The project will include eight tall buildings composed of two north towers that are 350 m high and six south towers with a height of 250 m. On top of the four south towers, a 300 m-long observation deck will connect the four towers, which is seismically isolated via friction pendulum bearings. Diverse architecturally and structurally unique features are incorporated in the project. Currently, the building framing work is proceeding with a targeted completion date in 2019. More than 100 forum participants visited to the construction site.

(Prepared by Masayoshi Nakai, Director, CTBUH Japan Structures Committee of JSSC)

Lecture delivery from Japan

Raffles City Chongqing project underway

Message from International Committee Chairman

Towards Further Development of Steel Construction Technologies

Hiroshi Katsuchi, Chairman, International Committee of JSSC (Professor, Yokohama National University)

The Japanese Society of Steel Construction (JSSC) has conducted a wide range of activities in the form of surveys, research and technological development aimed at promoting the spread of steel construction and at improving associated technologies, and at the same time it has extended cooperation to related overseas organizations. Aimed at spreading the steel construction technologies of Japan to developing overseas markets, the International Committee of JSSC was responsible for editing Issue No. 53 of *Steel Construction Today & Tomorrow*, a special issue for JSSC.

Issue No. 53 firstly introduces the JSSC Commendations for Outstanding Achievements in 2017—JSSC Awards, Outstanding Achievement Awards and Thesis Awards. In addition, this issue features outstanding Japanese steel construction technologies utilized in overseas construction projects—Passenger Terminal of the New Doha International Airport, the Taipei Nanshan Plaza project in Taiwan and the Tsubasa Bridge in Cambodia. In addition, a report on the application of new lean duplex stainless steel to the water intake facility of the Futase Dam is introduced.

Regarding our international events in 2017, two reports are inserted—the establishment of the special committee for the 12th Pacific Structural Steel Conference (PSSC2019) planned for Tokyo, Japan in 2019, and the 2017 China-Japan-Korea Tall Building Forum in Chongqing, China held in September 2017.

Finally, we would like everyone to be continuously aware of the activities of JSSC and we would also like to hear your opinions at any time.

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