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Special Feature

## Corrosion Protection for Steel Pipe Piles and Sheet Piles

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# Steel Sheet and Pipe Piles for Port Steel Structures

## — Corrosion Protection Technology: Today and Tomorrow —

By Dr. Hidenori Hamada (Kyushu University), Dr. Toru Yamaji (Port and Airport Research Institute), and Dr. Yoshikazu Akira (Port and Airport Research Institute)

**Hidenori Hamada:** Professor, Department of Civil and Structural Engineering, Kyushu University. He received a doctor of engineering degree from Kyushu University in 1998. He has been engaged in "concrete technology and corrosion engineering" for about 25 years.

**Toru Yamaji:** Group Leader, Materials Group, Structural Engineering Fields, Port and Airport Research Institute. He received a doctor of engineering degree from Kyushu University in 2010. He has been engaged in "concrete technology and corrosion engineering" for about 13 years.

**Yoshikazu Akira:** Researcher, Materials Group, Structural Engineering Fields, Port and Airport Research Institute. He received a doctor of engineering degree from Kagoshima University in 2004. He has been engaged in "corrosion engineering and corrosion-protection technologies" for about 8 years.

The history of corrosion-protection technologies in Japan extends more than 50 years. As a result, these technologies have developed to the remarkable level where protection against the corrosion of port and harbor steel structures is nearly complete. Also, the conditions affecting the application of corrosion protection technologies will differ from country to country, so the methods used will differ accordingly. However, the diversity of experience acquired in this field in Japan will also be of use in other countries. This article discusses corrosion-protection technologies applied to port and harbor steel structures in Japan. We believe that this article will contribute to the development of appropriate corrosion-protection technologies for steel structures in the world.

### Port and Harbor Steel Structures in Japan

#### • History

The oldest steel harbor structure built in Japan is a pier constructed using steel screw piles in the Port of Kobe in 1876, followed successively by the Ports of Yokohama, Nagoya, Osaka, and Tsuruga. In the last part of the Taisho era (1912~1926), steel sheet piles were imported to restore damage caused by the Great Kanto Earthquake. The first steel sheet pile-type mooring quay was constructed in 1926 at the Port of Osaka.

Entering the Showa era (1926~1989), imports of steel sheet piles increased, initially totaling 25,000~35,000 tons annually. Then in 1929, trial manufacture of steel sheet piles started at the government-run Yawata Steel Works, and full production started in 1930. In 1931 at the Port of Miyako, the domestically-produced steel sheet piles were first applied to port steel structure. Also, steel sheet pile mooring quays were constructed in the early Showa era in Osaka, Nagoya, Fushiki, Hakodate, and Rumoi. In the post-war period, steel pipe piles were extensively used for port and harbor facilities. The application of piles in pier foundation structures was expanded following the construction of Shiogama Port in 1954. The first cell-type mooring quay using flat steel sheet piles was constructed at the Port of Shiogama (1954~1959), followed by the Ports of Tobata, Nagoya, Naoetsu, Aomori, and Yokohama.

Entering the 1960s, steel pile piers were developed and were increasingly used to build large-capacity mooring quays in many ports. The Yamashita pier at the Port of Yokohama and the Maya pier at the Port of Kobe are typical structures.

Very recently, jacket-type steel structures are increasingly being adopted for port and harbor facilities. The Ooi container berth and the new D-Runway at Tokyo International Airport are typical structures.

#### • Features of Port and Harbor Steel Structures

Currently, nearly a half of the mooring quays in Japan are constructed using steel products. This is a feature in Japanese ports. A couple of major reasons for the greater use of steel products in Japan are the development of the Japanese steel industry as a core element in the country's rapid economic growth in the 1960s. Because Japan's rapid economic development beginning in the 1960s demanded that port and harbor facilities be quickly improved, the possibility of rapid construction was an additional reason for the enthusiastic adoption of steel structures. The total extent of mooring quays employing steel structures has reached 490 km. At the Port of Tokyo, the ratio of steel structures to total port and harbor facilities (including breakwaters) has rapidly grown. Of

Fig. 1 Typical Structure of Steel Sheet Pile-type Mooring Quay

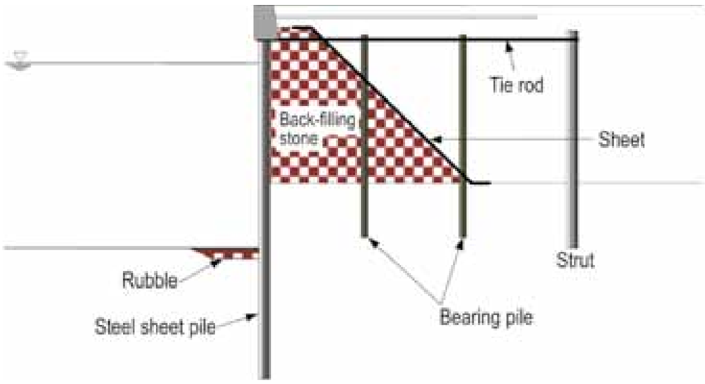



Fig. 2 Typical Structure of Steel Pipe Pile-type Mooring Quay

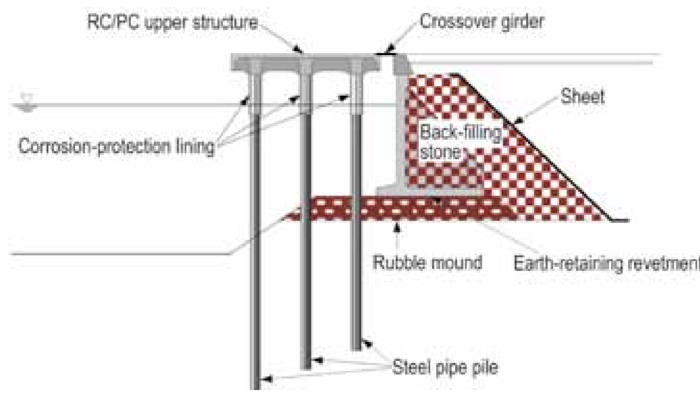
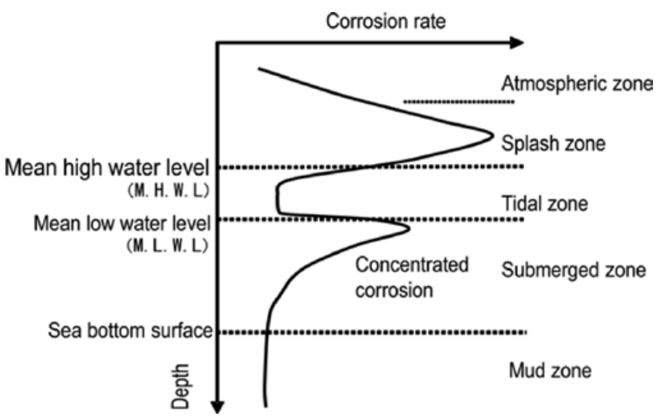



Fig. 3 Representative Examples for Vertical-direction Corrosion Rate of Steel Pipe and Sheet Piles




the total extension of more than 200 km of port and harbor facilities, steel facilities have surpassed 150 km.

## ● Typical Port and Harbor Steel Structures

### — Steel Sheet Pile-type Quays

Steel sheet pile-type mooring quays are constructed by driving steel sheet piles into the ground to form an earth-retaining wall (Fig. 1). The most common steel sheet pile-type mooring quay is formed using tie rods to connect the steel sheet pile wall to a strut structure (steel pipe, steel sheet pile, shape, etc.) installed behind the wall. Depending on the scale of the load to be supported, two kinds of piles are used for the pile wall—commonly adopted U-type steel sheet piles and steel pipe piles having connection joints. In cases when a small load is applied, as in ports with shallow water depth, a self-supporting wall structure is adopted without the use of a strut structure and tie rods. Even when using steel sheet piles or steel pipe piles, the front surface of the pile wall is subjected to harsh corrosive marine environments.

### — Pier-type Mooring Quays

Pier-type mooring quays are constructed by placing the upper structure on column members (Fig. 2). The pier-type quay consists of a pier located at the front of the quay and an earth-retaining structure to the rear, with steel products widely used for the front pier. Reinforced concrete or precast concrete beams and floor slabs are installed on the upper structure. Among port and harbor facilities, the RC (reinforced concrete) upper structure is the site where structural deterioration due to salt damage most frequently occurs.

## Features of Corrosion in Marine Environments

The environments where port and harbor steel structures are applied are roughly classified into five zones, namely, atmospheric zone, splash zone, tidal zone, submerged zone, and mud zone. In cases where long steel products such as steel sheet pile and steel pipe pile extend through multiple environments (tidal zone, submerged zone, and mud zone), macro-cell corrosion attributable to differences in these application environments occurs. The area where serious problem arises regarding the corrosion of steel structures without corrosion-protection measures is the splash zone and just beneath the mean low water level (M.L.W.L.). Corrosion tendencies by corrosive environment are introduced (Fig. 3).

### ● Atmospheric zone

In most cases, the corrosion rate (corrosion loss) for such structures is mostly around 0.1 mm/year.

### ● Splash zone

In the splash zone, the structure is constantly subjected to seawater splashing, and therefore a large amount of seawater and oxygen can be supplied to steel surface. Accordingly, the splash zone is the most corrosive environment. In general, corrosion rate in this zone reaches the level of 0.3 mm/year. According to surveys in the Okinawa area, there are examples of corrosion rate reaching 0.5~0.6 mm/year due to the effect of high temperature and high humidity.

### ● Tidal zone

The tidal zone is where structures undergo periodic immersion in seawater and exposure to the atmosphere due to tidal action. In this zone, corrosion rate in the vicinity of the mean seawater level (M.S.L.) is small, however, corrosion rate in the vicinity just beneath the M.L.W.L. is extremely large. The reason for this is the formation of a macro-cell with the cathode area around the M.S.L. (high dissolved oxygen concentration), and the anode area, the vicinity just beneath the M.L.W.L. (low dissolved oxygen concentration). There are cases, depending on the circumstances, when corrosion rate in the vicinity just beneath the M.L.W.L. exceeds that in the splash zone. This phenomenon is called "concentrated corrosion." The concentrated corrosion caused some example of structural collapse of port steel structures.

**● Submerged and mud zones**

Corrosion in the submerged zone is nearly uniform. Corrosion rate at depths below –1 m or more is about 0.1~0.2 mm/year. In the mud zone, because of reduced oxygen supply compared to the submerged zone, corrosion rate becomes smaller: about 0.03~0.05 mm/year.

## Corrosion-protection Technologies for Port and Harbor Steel Structures

**● History of Corrosion-protection Technologies**

The most prevalent concept of corrosion protection in old days was "corrosion allowance." Accordingly, the thickness of steel products was increased beforehand by a margin for corrosion loss. It was in 1953 that the cathodic protection was first applied on a harbor steel structure. This was the Port of Amagasaki in which an anodic system employing magnesium alloy anode was applied. The protection was an external current source system.

Entering the 1960s, attempts were made to use cathodic protection (external current source system) in various port and harbor structures. Around 1960~1970, oil paint and tar epoxy resin paint were developed and increasingly adopted for corrosion protection in the zones above the submerged zone. In coating/lining corrosion protection, zinc-rich paint was developed and used as an undercoat for tar-epoxy resin coating. Further, attempts were made to cover the upper section of pier steel pipe piles with concrete as a corrosion-protection method for structures above seawater level where the effect of cathodic protection could not be obtained. Around 1970, chlorinated rubber paint was developed, followed by the development of urethane paint in 1972. In cathodic protection, high-performance aluminum alloy anodes were developed, and the full-scale application of anodic corrosion protection started. Also around 1970, underwater welding technology was developed to reduce the work period and increase safety when attaching aluminum alloy anodes.

Starting in 1980 and for some years, diverse kinds of highly durable coating/lining corrosion-protection methods were developed, among which were the cement-mortar/FRP cover method, the petrolatum lining method, and the underwater hardening-type lining method. Around 1982, polyethylene lining and polyurethane lining (the so-called heavy-duty corrosion protection method) were developed. In the coating-type system, ultra heavy/thick type epoxy resin paint and fluorine resin paint having high weather resistance were developed.

However, even in those days, corrosion-protection systems were not necessarily applied for all port and harbor facilities, but the "corrosion allowance" system was still remained. As a result, in 1983, an accident occurred at the Port of Yokohama involving the subsidence of a harbor facility. Triggered by this accident, cathodic protection was established in 1984 as the standard method of corrosion protection for existing steel structures in the submerged and mud zones, and coating/lining protection as the standard method of corrosion protection for existing steel structures in the tidal, splash, and atmospheric zones.

During the same period, the practical application of titanium as a corrosion-protection material began in the form of titanium cladding for steel plates, as did the use of corrosion-resistant stainless steel linings. Titanium materials had already been adopted for actual structures such as the bridge piers of the Trans-Tokyo Bay Highway (for water depths ranging between –2 m and +3 m) and the Yumemai Bridge (floating, revolving-type). Seawater-resistant stainless steel linings were applied as a corrosion-protection measure for the jacket-type quays used to improve the Ooi Quay (for water depths of –1 m and above).

Further in the "Technical Standards for Port and Harbor Facilities" revised in April 1999, corrosion protection methods based on corrosion allowance was eliminated, and as a rule cathodic protection was stipulated for zones below the mean tidal level and the coating/lining protection method for all zones upwards from 1 m below the mean tidal level.

**● Concepts for Standard Corrosion-protection Methods**

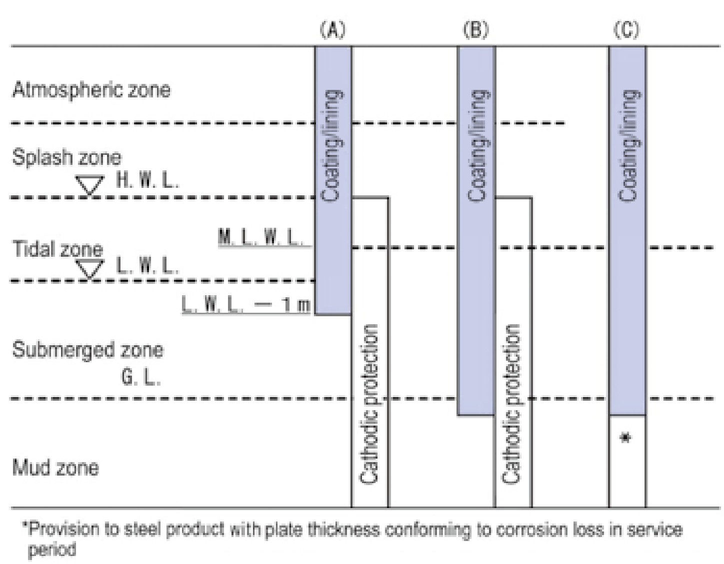
Regarding concentrated corrosion occurring in the vicinity just beneath the M.L.W.L., it is difficult to visually find it and, further, to repair it by the use of coatings, and thus it is necessary to implement appropriate countermeasures. Three standard corrosion-protection systems are established to treat concentrated corrosion (Fig. 4).

(A): This method applies coating/lining corrosion protection to the section above L.W.L. –1 m and cathodic protection to the section below the M.L.W.L. It is the most widely applied method.

(B): This method applies the coating/lining corrosion protection of method (A) to sections deeper toward the sea bottom. This method is most economical and effective in cases where the large corrosion-protection current density of cathodic protection is necessary in open seas and in areas subject to the flow of high tides. There are many examples of method (B) being applied to long-span bridges and floodgates.

(C): This method applies coating/lining corrosion protection to those parts of the splash zone where the severest corro-

**Fig. 4 Standard Corrosion-protection Methods for Port and Harbor Steel Structures**



sion occurs, tidal zone, submerged zone, and mud zone. In general, this method is applied to steel sheet pile revetments installed in areas of shallow water. In such applications, the coating/lining method should possess particularly excellent corrosion protection and durability. In most cases, polyethylene linings and urethane-elastomer linings are applied to newly-installed structures, and petrolatum linings and mortar linings to existing structures. In most cases, the limit application depth for the coating/lining method is up to G.L. –1 m. The corrosion-protection method is not applied to the mud zone at G.L. –1 m or below. In such an application, it is necessary to adopt a steel product having increased thickness sufficient for the corrosion loss expected in the corresponding sea area.

### ● Materials for Coating/Lining Corrosion Protection

Five major coating/lining corrosion-protection methods are applied to port and harbor steel structures—coating, organic lining, petrolatum lining, mortar lining, and metallic lining.

A representative coating system uses heavy/thick film-type zinc-rich paint plus epoxy resin paint. Organic lining has higher corrosion resistance. The organic lining applied to port and harbor steel structures is polyethylene lining, urethane elastomer lining, extra-heavy/thick film-type lining, and underwater lining. Underwater lining is available in two types—the putty type in which the lining material, in a putty-like state, is applied by manual cladding; and the painting type whereby the lining material is applied using rollers and brushes. One of the features of the underwater lining system is that the lining can be applied to complex-shaped structures such as the sections where steel sheet piles are joined.

Petrolatum lining has many recorded applications and is effective as a corrosion-protection method for port and harbor steel structures. In this system, petrolatum-type lining is tightly bonded to the steel product surface, which is protected by the use of plastic or reinforced plastic covers or corrosion-resistant metallic covers. There are cases in which a buffer material is inserted between the petrolatum material and the cover. The integrated system features underwater applicability, comparative ease in surface grinding, and no need for a curing period after lining.

Mortar lining is a method whereby corrosion protection is attained by forming a dense passive film on the surface of a steel product by fully utilizing the alkaline in cement. When lining is with concrete, the method is frequently called mortar lining. Mortar lining has long been applied in the corrosion protection of port and harbor steel structures. When deterioration in the form of cracking, peeling-off, neutralization of lining mortar occurs, the corrosion protection performance of the lining is lost. To remedy this, various countermeasures are adopted—increased lining thickness, mixing of organic polymers and steel fibers, surface coating, and the use of protectors that are also used as FRP and metal molds.

Metallic lining is particularly superior in impact resistance and abrasion resistance, and is high in corrosion resistance. Highly corrosion-resistant stainless steel and titanium are used as the metallic lining materials.

### ● Cathodic Protection

#### — Principle

In cathodic protection system, the direct current that overcomes the corrosion current flowing from the steel product into the electrolysis (seawater) is continuously flowed from an external source into the steel product so as to prevent ionization (corrosion) in steel products. There are two types of cathodic protection—the external current source system and the sacrificial anodic system. In the sacrificial anodic system, large/small and/or high/low trends toward the ionization of metallic materials is utilized in such a way that metallic materials such as aluminum, zinc, magnesium, etc. are connected to the steel and are ionized (corroded) instead of the steel so as to protect the steel product from corrosion.

#### — Application

The prescribed application range for cathodic protection is the section extending from M.L.W.L. downward. Cathodic protection is very effective in preventing concentrated corrosion from occurring in steel products located just beneath M.L.W.L. In cathodic protection as currently applied, the sacrificial anodic system using aluminum alloy anodes is almost exclusively adopted. The major reasons for this are many advantages offered by the sacrificial anodic system—no need to use a current source once the system is installed (in contrast to the external current source system), no need for power expenditures, and the possibility of inspection and maintenance by periodically measuring electric potential.

### New Topics on Corrosion-protection Technology — Tokyo International Airport (Haneda Airport) —

Fig. 5 shows a recent Haneda Airport with the new 4th Runway (in 2009, under construction). The feature of this runway is composed of reclaimed part (2,020 m in length) and wharf part (1,100 m in length). As previous experience on "Port and Harbor Steel Structure" described above, well established corrosion prevention technology is required on wharf structures for long term design service life, 100 years for the new 4th runway. For this very important structure, steel piles composing the steel jacket are all protected with stainless steel plate, 0.4 mm in thickness at the part of tidal and splash zones (Fig. 6). Steel beams composing the frame of the upper structure are protected with epoxy resin coating. The design service

**Fig. 5 New Haneda Airport with 4th Runway**

Fig. 6 Lower Structure of Wharf Type Runway


life of 100 years is a challenge for marine steel structures, exposing to very severe environmental condition. As just explained, highest level of corrosion protection technology is adopted for jacket-type steel structure. However, it is certain that proper maintenance is necessary to achieve the service life of 100 years.

## Future Insight in Performance-driven Design and Maintenance of Corrosion Prevention Technology

Roughly speaking, in history of corrosion-protection technology in Japan, in the 1980s, the "corrosion allowance theory" was diminished and "corrosion-prevention methods" such as cathodic protection and coating/lining were established. In the 2000s, the main part of infrastructure management changed from new construction to maintenance of existing structures. And, design system was gradually changed from the "specification" to the "performance-driven." Also, the design of corrosion protection system was gradually shifted to the performance-driven method. The definition of the performance of corrosion prevention system is "within the design service life, to prevent steel from corrosion (rusting)."

The design service life of general steel structures is mostly 50 years, except for the new runway of Haneda Airport (100 years). Table 1 presents the lining or coating method and expected service life. At present technological condition, the 50 year period is the highest durability. Normally, 20 years and 30 years are the expected level. This means the proper maintenance system is necessary to achieve the expected service life of more than 50 years for marine steel structures. In recent decade since 2000, major discussion was concentrated on the maintenance system of port and harbor structures, both of concrete structures (RC, PC, concrete-steel hybrid) and steel structures (steel sheet pile, steel pipe pile, jacket type).

Fig. 7 shows the performance degradation curve and maintenance effect. In this figure, three different maintenance levels are also shown. Maintenance level is defined as the "level I" as the highest grade, "level II" for middle grade, and "level III" for the lowest grade. The level is set for each structure considering several important factors such as "important level of the structure," "environmental condition," and "difficulty of the inspection/survey." Maintenance work should be based on the LCM concept of individual structure. A series of maintenance is composed of "periodic inspection," "required investigation," and "evaluation of deterioration or performance degradation," and if necessary, "repair and reinforcement" and "data base construction for high level and low cost maintenance system."

At present 2011, to some degree, corrosion-protection technology is well developed. However, in future, following upgrades, 1) performance-driven design system of corrosion protection system and 2) higher level maintenance system, are still necessary to be established.

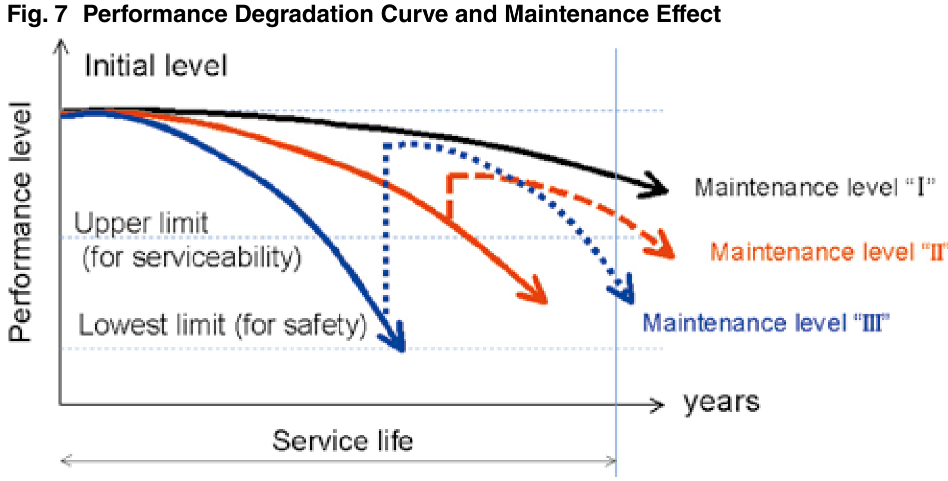
### *Concluding Remarks and Acknowledgements*

For this article, the Port and Airport Research Institute supplied many useful data and materials, for which we express our greatest thanks. While it is clear that corrosion-protection technologies have made great strides, the current technologies are not necessarily perfect. In Japan, new R&D efforts are being promoted in this area, and authors hope to contribute to these efforts in one way or another. In this regard, we would be very grateful if we could collaborate with engineers and researchers in all over the world, pertaining to corrosion protection in the construction of port and harbor steel structures.

Table 1 Lining or Coating Method and Expected Service Life

| Lining or coating | Materials and method | Expected service life |
|---|---|---|
| Precoated type | Thick epoxy resin type painting<br>Glass flake contained epoxy resin type | 20 years |
| | Polyethylene lining type<br>Polyurethane lining type<br>Thick epoxy resin type lining | 30 years |
| | Seawater resist type stainless steel lining<br>Thin titan plate clad type | 50 years |
| Coated on-site type | Underwater-hardened resin type | 20 years |
| | Resin covered petrolatum tape type<br>Metal covered petrolatum tape type | 30 years |
| | Reinforced concrete covered type<br>Cement mortal covered with protective cover | 30 years |

Fig. 7 Performance Degradation Curve and Maintenance Effect



# Steel Pipe Piles and Sheet Piles

## — Repair and Reinforcement Technology —

By Kazuhiro Masuda (Yoshikawa-Kaiji Co., Ltd.)
Research Group on Corrosion-protection and Repair Methods for Marine Steel Structures
Hiroshi Shiraishi (Nakabohtec Corrosion Protecting Co., Ltd.)
Research Group on Corrosion-protection and Repair Methods for Marine Steel Structures
Atsuo Moriwake (Toa Corporation)
Research Group on Corrosion-protection and Repair Methods for Marine Steel Structures

### Concerns about Concentrated Corrosion

Because port/harbor steel structures are exposed to severely corrosive environments, the inappropriate application of corrosion protection and maintenance is likely to cause fatal structural deterioration, such as a significant reduction of loading capacity. During the period of high economic growth in Japan from the latter part of the 1950s through the 1960s, numerous port/harbor facilities were constructed, as were many other steel structures. At that time, corrosion-protection technologies for such structures had not yet been established, in contrast to the current rapid progress of these technologies. Accordingly, among the structures constructed then, there were some that suffered severe deterioration due to concentrated corrosion.

Photo 1 Example of concentrated corrosion in underwater steel structure

Photo 2 Example of pitting corrosion in corrosion-protection coating

Photo 1 shows an example of concentrated corrosion occurring in a steel product installed underwater. Photo 2 shows an example of pitting corrosion in the corrosion-protection coating applied to the splash zone of a steel product. Under the conditions at work in these examples, if sufficient maintenance control is not applied in time, deterioration will develop further, inevitably leading to fatal structural damage.

The development of corrosion-protection technologies for port/harbor steel structures in Japan can be organized as shown in Fig. 1[1]. From the latter part of the 1950s through the 1960s when a number of port/harbor structures were constructed, cathodic protection relying on impressed current system was applied to submerged structures, and design methods employing corrosion allowance were applied to structures installed above the tidal zone.

In those days, because the importance of maintenance was not yet recognized to the extent that it is today, there were many cases of steel structures experiencing fatal deterioration due to concentrated corrosion and pitting corrosion, as described above. A typical example of such fatal deterioration occurred in 1981 when concentrated corrosion caused the buckling of steel pipe

**Fig. 1 Development of Corrosion-protection Technology for Port/Harbor Steel Structures in Japan[1]**

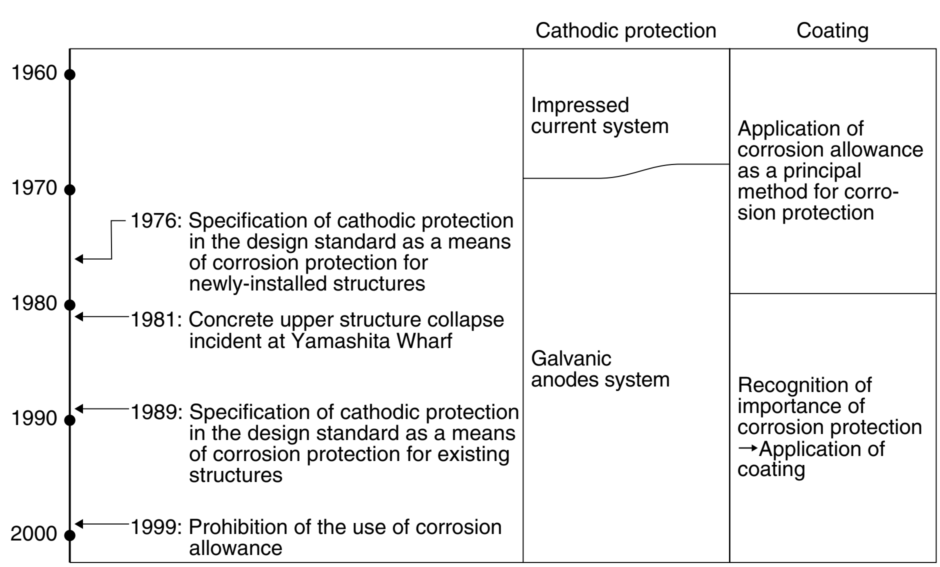


**Fig. 2 Conceptual Drawing of Severe Lowering of Cross-section Capacity due to Corrosion**

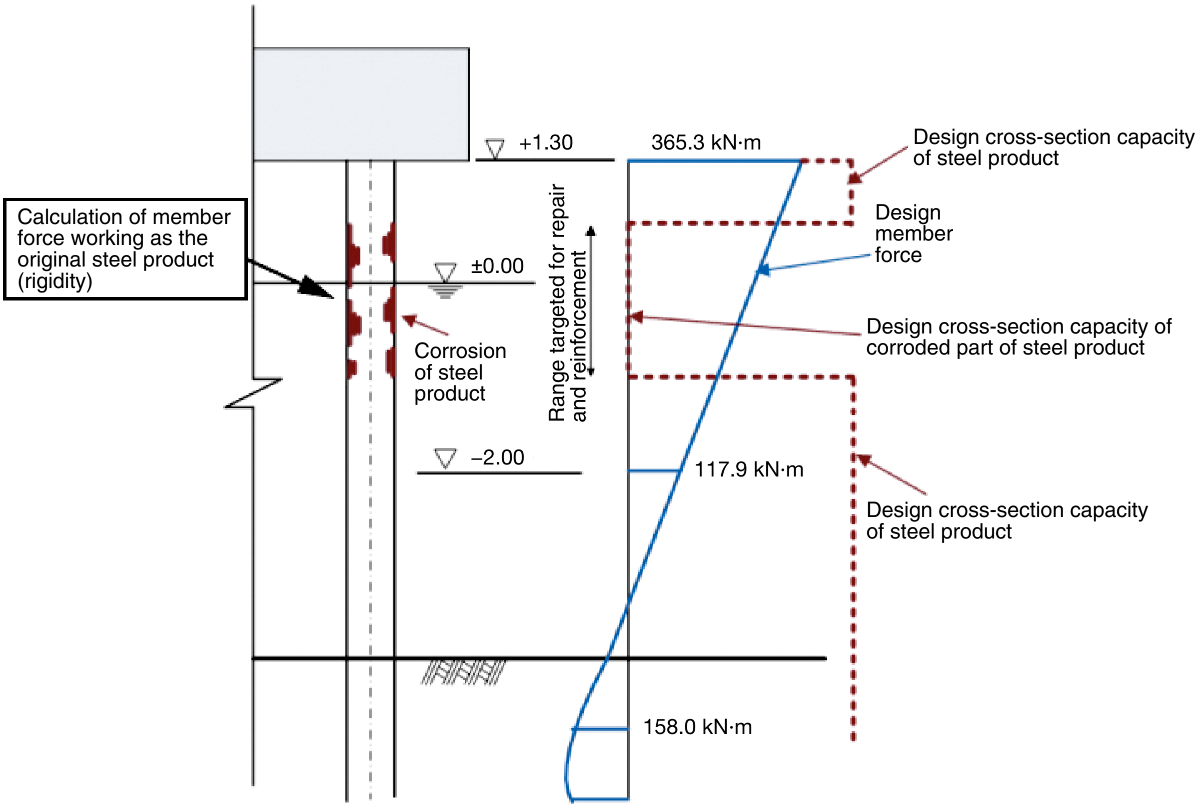
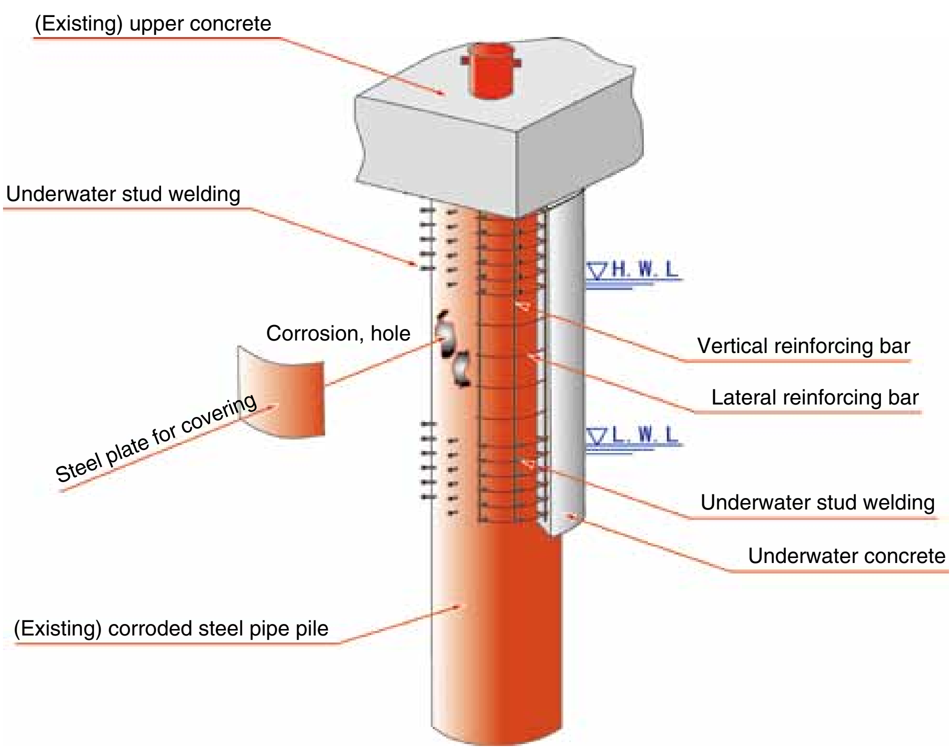



**Fig. 3 Conceptual Drawing of Repair and Reinforcement Employing Reinforced Concrete**




piles and the subsequent collapse of the superstructure at the Yamashita Wharf in the Port of Yokohama.

Triggered by this accident, the importance of corrosion-protection technology was recognized, and the development of advanced technologies was promoted and, at the same time, the *Corrosion-protection and Repair Manual for Port/Harbor Steel Structures*[2] was prepared. Currently, as a result of accidents like this, the provision of corrosion protection to newly installed steel structures is standard practice, and, further, the implementation of appropriate maintenance for such structures is mandatory.

Nevertheless, a number of steel structures designed using corrosion allowance criteria are in use, and further the corrosion of steel products still occurs due to insufficient maintenance practices previously applied. Accordingly, steel structures still exist that are likely to suffer serious deterioration. To this end, it is now necessary to repair and reinforce these structures capitalizing on the *Corrosion-protection and Repair Manual*.

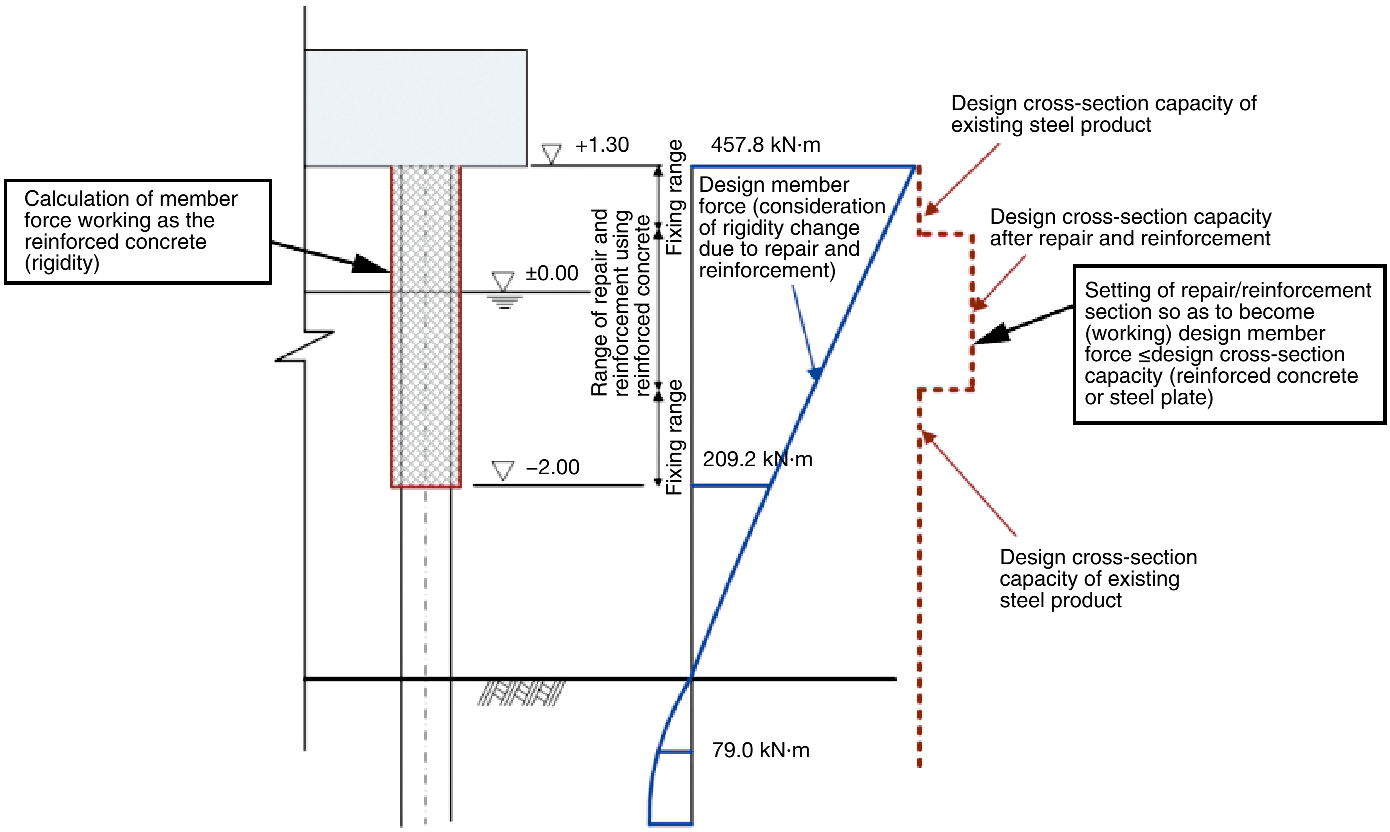
This article introduces the latest technologies for repairing and reinforcing the steel structures in which corrosion has already developed.

## Repair and Reinforcement Using Reinforced Concrete

### • Basic Practices in Design

The basic practice in this method is to use reinforced concrete members to repair and reinforce structural steel sections that have decreased in design cross-section capacity due to severe corrosion and that thus fall short of the design member force (refer to the dotted line in Fig. 2). In this case, repair and reinforcement are provided so that Equation (1) is satisfied.

**Fig. 4 Conceptual Drawing of Recovery of Cross-section Capacity of Steel Product after Repair and Reinforcement**




$$\gamma_i S_d / R_d \leq 1.0 \quad (1)$$

Where

$S_d$: Design member force

$R_d$: Design capacity of cross-section

$\gamma_i$: Coefficient of structure (in the case of adopting reinforced concrete)

Specifically, reinforced concrete is firmly affixed using underwater studs to the sound section of the steel pipe pile or sheet pile, the target member for repair and reinforcement, in order to integrate both the concrete and the pile so that the reinforced concrete, the repair and reinforcement member, can securely maintain the sectional strength.

Fig. 3 shows a conceptual drawing of the repair and reinforcement method using reinforced concrete. The underwater studs are weld-joined to both sides of the targeted repair and reinforcement member, to which the underwater reinforced concrete having the reinforcing bar arrangement to satisfy the equation (1) is affixed.

The conceptual cross-section capacity recovered by use of this method is shown by the dotted line in Fig. 4.

## ● Execution Outline

In executing the method, the marine organisms and loose rust that adhere to the target structure are removed using scraping bars, air chippers etc, and, in addition, surface preparation is conducted on the stud bolt joining section until the surface is suitable for welding (Photo 3).

Because the underwater studs are important members for integrating the pipe or sheet pile with the reinforced concrete, an underwater stud welding method that allows secure weld-joining underwater is adopted (Photo 4). To ensure quality control over the underwater stud welding, the method is adopted that confirms the weld quality using the waveform of the current flowing during welding.

Then, reinforcing bars are installed on the pile as necessary. And, when installing reinforcing bars on sheet piles, a frequently used method is to employ bars that were preassembled on land.

As regards the concrete, commonly applied underwater concrete is adopted, but in cases requiring that due attention be paid to the effect on water quality in the surrounding sea area, antiwashout underwater concrete with high separation resistance is commonly adopted.

Photo 3 Marking of stud welding position and surface preparation

Photo 4 Underwater stud welding

Photo 5 shows the condition after concrete form removal. As seen in the photo, the integration work can be carried out without damaging the configuration of the pipe pile or sheet pile, and, accordingly, when conducting subsequent maintenance, visual inspection can be done in the same

Photo 5 Reinforced concrete after completion

way as for the other installed piles.

## Repair and Reinforcement Using Steel Plates

### ● Basic Practice in Design

In this method, steel plate is used instead of concrete. Specifically, steel plate with the required thickness is attached to a section of steel pipe or sheet pile in which corrosion has caused a marginal drop below the design sectional strength. The plate is fixed with underwater welding to the sound sections at either end of the pipe pile or sheet pile in need of repair or replacement.

The basic conditions for sectional strength lowering and recovery by the use of steel plate are the same as for reinforced concrete. However, an important feature of the steel plate method is that, because the thickness of steel plate members is less than that of concrete members, the stress occurring in the pipe pile or sheet pile after repair and reinforcement is similar to that present before repair and reinforcement. Thus, fewer adverse effects are caused by repair and reinforcement.

### ● Execution Outline

After using air chippers etc. to remove the marine organisms and loose rust that adhere to the target steel product, the prefabricated steel plate used for repair and reinforcement is installed in the prescribed position. The reinforcing steel plate that is applied to the steel pipe pile is frequently fabricated as two pieces (Photo 6).

Then, the prefabricated steel plate is weld-joined to the pipe pile or sheet pile by

Photo 6 Installation of steel plate for repair and reinforcement

means of wet-type underwater welding (Photo 7). In weld joining, because the weld quality is governed by the welder's ability and the hydrographic conditions, it is important to secure a sufficient and safe welding length. For this purpose, the characteristic value for the yield stress of wet-type underwater welds is set at 70% that achieved in shop welding using a similar welding method. However, in cases of severe working environments marked by wave action or when the pipe pile or sheet pile is subjected to strict, repeated stress, a value of 70% is sometimes insufficient, and thus it is necessary to pay due attention to weld application.

Meanwhile, because the method in question premises the provision of a corrosion-protection method, it is necessary to apply cathodic protection to underwater steel piles, and corrosion-protection coating to steel piles installed in the tidal and atmospheric zones, referring the Corrosion-protection and Repair Manual.

## *Corrosion-protection and Repair Manual for Port / Harbor Steel Structures*

Currently, there are numerous steel pipe piles and sheet piles still in service that require repair and reinforcement. However, given the recent severe economic situation, it is difficult to provide repair and reinforcement to all these piles in a short period of time.

In this regard, the *Corrosion-protection and Repair Manual for Port/Harbor Steel Structures (2009)*[2], published in November 2009, informs us not only about the importance of maintaining port/harbor steel structures but also about practical ap-

Photo 7 Integrated repair/reinforcement steel plate and steel pile

proaches to maintenance. If the *Manual* is properly used to carry out repair and maintenance within the context of lifecycle costs related to steel structures that previously were insufficiently maintained, such endeavors will contribute to ensuring a safe social infrastructure. We hope this article will be helpful to such endeavors.

***Acknowledgements***

In preparing the current article, reference was made to results obtained from the revision of *Corrosion-protection and Repair Manual for Port/Harbor Steel Structures (2009)*, a joint work by the Research Group on Corrosion Protection and Repair Methods for Marine Steel Structures and the Coastal Development Institute of Technology. Similarly, reference was also made to achievements obtained from revision of the *Handbook on Practical Corrosion-protection and Maintenance*[3] (slated for publication in the fall of 2011 by the Research Group on Corrosion Protection and Repair Methods for Marine Steel Structures). We wish to deeply thank the individuals of those organizations for their generous cooperation.

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2) Coastal Development Institute of Technology: Corrosion-protection and Repair Manual for Port/Harbor Steel Structures (2009), Nov. 2009
3) Research Group on Corrosion Protection and Repair Methods for Marine Steel Structures: Handbook on Practical Diagnosis, Corrosion-protection and Maintenance for Port/Harbor Steel Structures

# Nhat Tan Bridge in Vietnam

## — Bridge Design and Substructure Construction —

By Hiroki Ikeda, Shigeyoshi Ando, Tsukasa Akiba and Harukazu Ohashi
Nippon Engineering Consultants Co., Ltd.

### Project Outline

The Nhat Tan Bridge project represents an 8.5-km highway construction project that crosses the Red River and extends south to north in Hanoi, the capital of Vietnam. The project commemorates the 1,000th anniversary (October 2010) of Hanoi being established as the nation's capital. The new bridge is a cable-stayed structure with five main towers and is expected to become a symbol of friendship between Vietnam and Japan.

Design and project execution control are undertaken by a joint venture of Chodai Co., Ltd. and Nippon Engineering Consultants Co., Ltd. Construction package 1, including construction of the main bridge, is being undertaken by a joint venture between IHI Corporation and Sumitomo Mitsui Construction Co., Ltd. and is financed as a yen-based STEP (Special Term for Economic Partnership) loan, a special Japanese government ODA loan. Construction work started in October 2009.

The following introduces mainly the design and construction of the steel pipe sheet pile well foundations adopted for the tower foundations of the main bridge.

### Plan and Design of the Nhat Tan Main Bridge

**● Bridge Outline**

—Project owner:
Ministry of Transport of Vietnam, Project Management Unit 85
—Bridge length: 1,500 m
—Length of Spans:
150 m + 4 × 300 m + 150 m
—Type
Superstructure:
Six-span continuous composite dual I-girder cable-stayed bridges
Slabs:
Precast reinforced concrete slabs
Main towers:
Reinforced concrete A-shaped towers
Foundations:
Steel pipe sheet pile well foundations
(Refer to Figs. 1~2)

**● Selection of Bridge Type**

The Red River flows west to east, and in the

**Fig. 1 Section of Superstructure**

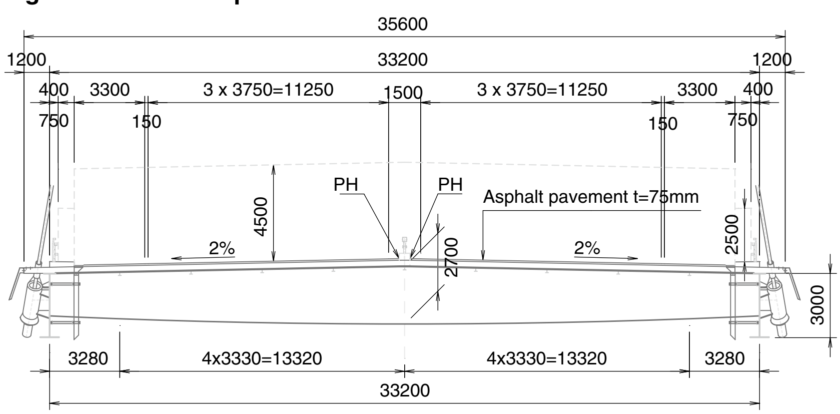



**Fig. 2 General Drawing of P12 Main Tower**

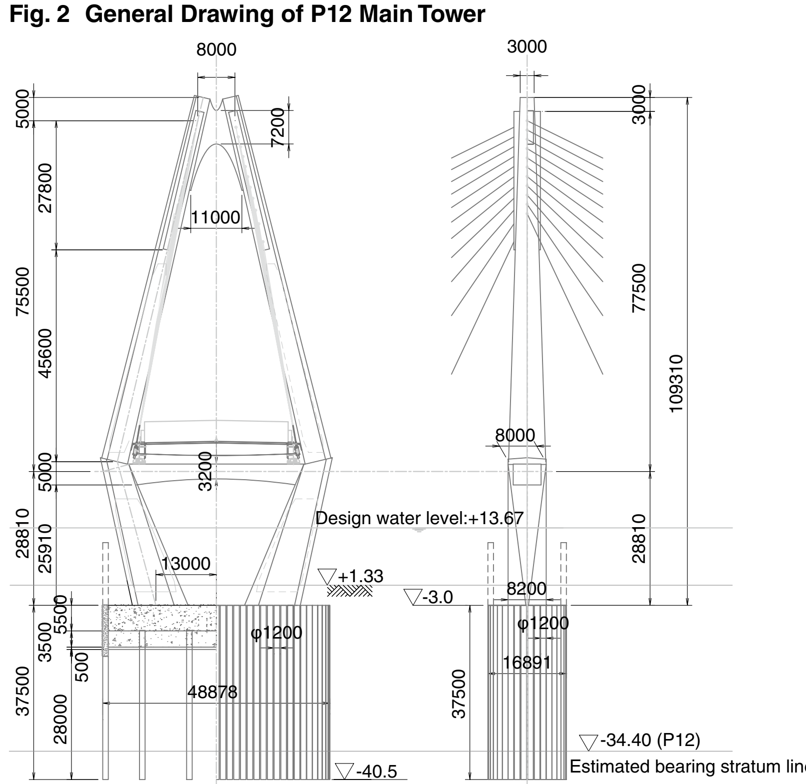


**Fig. 3  Elevation of Entire Bridge**

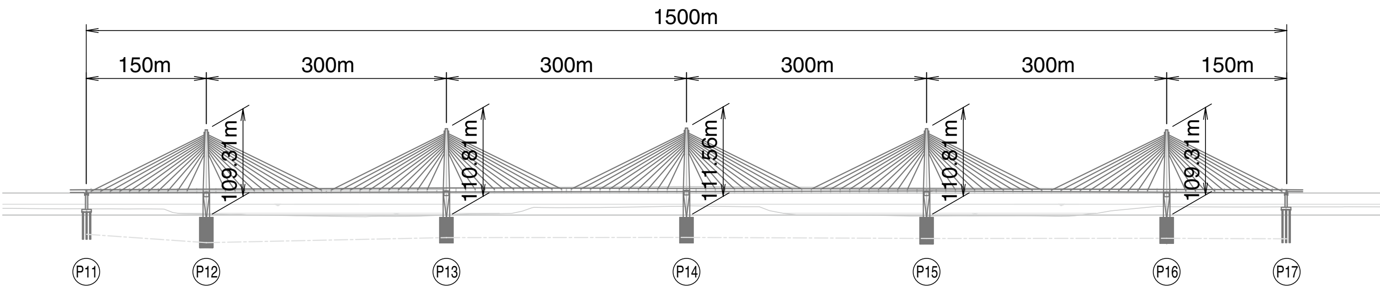



**Fig. 4  Perspective of Nhat Tan Bridge upon Completion (Computer Graphics)**

(Courtesy: Sumitomo Mitsui Construction Co., Ltd.)

vicinity of the bridge construction site it is separated into two channels that sandwich a sandbar. According to the historical record, the riverbed is known to change over time with regard to the location of the river channels and the sandbar. Because of this, future movement of the river channels and sandbar were taken into consideration in selecting a six-span continuous cable-stayed bridge plan having spans of equal length (Figs. 3, 4).

### ● Applied Standards

The bridge design conforms to the *Specification for Bridge Design 22TCN-272-05* of Vietnam, which is based on the AASHTO-LRFD of the U.S. Further, such items as steel pipe sheet pile well foundations and base-isolated bearings, which are not described in the Specification, followed specifications in the design standards of Japan.

### ● Major Structural Materials

The major structural materials are as follows:

—Steel products:
  SS400, SM400, SM490, SM490Y, SM520, SM570
—Cables:
  Parallel strands using 7 mm-diameter galvanized steel wire (tensile strength: 1,770 MPa)
—Steel pipe sheet piles: SKY400, SKY490
—Design standard strength of concrete:
  40 MPa (main tower, slab); 30 MPa (end piers, cast-in-place piles); 25 MPa (top slab of steel pipe sheet pile foundations)
—Reinforcing bars: SD390
—PC strands: SWPR7BL

### ● Road Composition

The road is composed from its center: two automobile lanes (3.75 m wide), a bus lane (3.75 m wide), a dual-use motorcycle-bicycle lane (3.3 m wide), and a walkway (0.75 m) along the outer edge.

### ● Superstructure

The superstructure is a continuous structure spanning 1,500 m in total length. The main structure consists of two main I-girders located along each edge of the roadway to which cross beams to support the slabs are arranged at 4-m intervals. The cables are anchored to the outside surface of the main girder webs. The main girders and cross beams form a composite girder structure to which precast slabs are connected using dowels. Fairings are installed on the outer edges of the slabs to improve wind stability.

The cables employing parallel wire strands are arranged in a fan-shaped form, and the girders are suspended in a double-plane format from the main towers.

### ● Main Towers

The reinforced-concrete main towers adopt an A-shaped structure to secure rigidity perpendicular to the bridge axis. Below the crossbeams that support the superstructure, the support columns of each tower draw steadily inward as they descend, thereby narrowing the space between the columns in order to decrease the plane shape of the foundation.

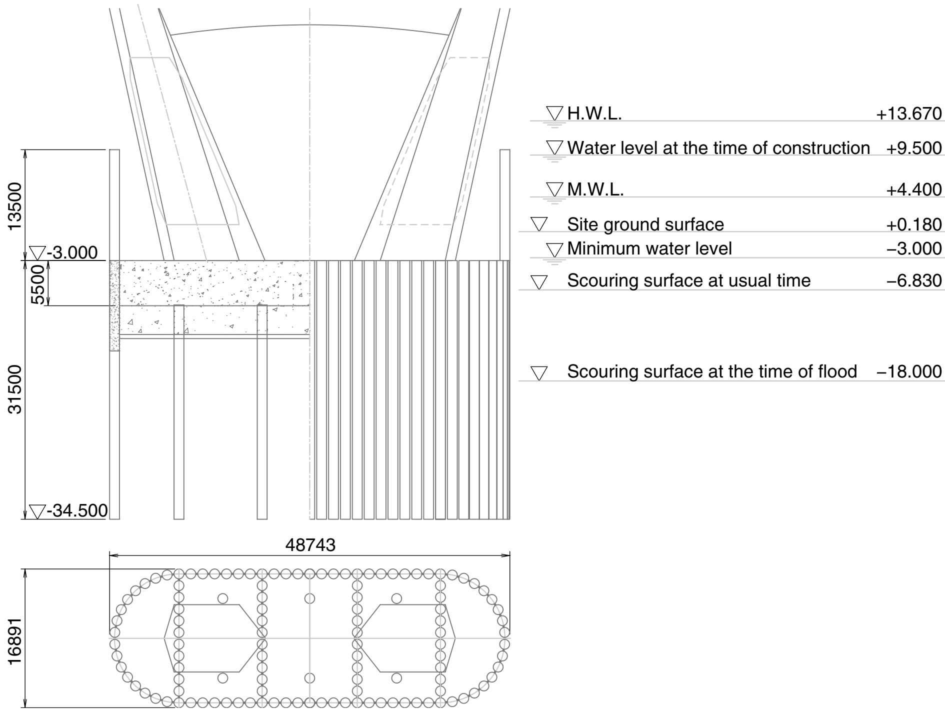
Because axial tension force works on the crossbeams, a prestressed concrete structure was adopted. Anchor boxes employing steel plates are embedded near the top of the tower, and the cables are anchored to the bearing brackets within the boxes.

### ● Steel Pipe Sheet Pile Well Foundations

With the aim of improving upon the construction quality of cast-in-place piles that have occasionally offered application concerns in Vietnam, steel pipe sheet pile well foundations were adopted for the first time in Vietnam. Because this type of foundation was developed in Japan, two Japanese specifications were adopted for design and construction—*Specifications for Highway Bridges IV* (2002) and *Design and Construction Manual for Steel Pipe Sheet Pile Foundations* (1997).

The scouring depth accounted for in the design was estimated to be, at maximum, 15 m from the riverbed. The driving method for pile installation was adopted with the aim of securing bearing capacity. The

**Fig. 5 Schematic Diagram of P13 Foundation**




piles were to be embedded in the bearing stratum, a gravel layer with N>50, to a depth more than five times the pile diameter. The top surface of the top slabs was set at a position equal to sea level –3 m, taking into account changes in the riverbed.

The adopted pile diameter was 1,200 mm, and the wall thickness of the piles is 16~21 mm. The well foundations have an oval plane shape with the dimensions of 48.7 m x 16.9 m. The maximum length of the steel pipe sheet pile well foundations, including temporary cofferdams, is 50 m. The number of pipe piles used, including the bulkhead and internal piles, amounted to 632. The reinforcing bar stud method was adopted for connecting the top slabs. (Refer to Fig. 5)

Photo 1 Full view of construction site

Photo 2 On-land construction at sandbar

### Progress in Construction

At the end of May 2011, construction of the main tower foundations was underway at P13~P15; and driving of the steel pipe sheet piles was completed and underwater excavation, installation of the lower slabs and upper slabs, etc. were underway at P12 and P16. (Photos 1, 2).

#### ● Execution Yard and Machinery

The sandbar in the Red River where P14 is being constructed is utilized as a structural material yard, reinforcing bar fabrication yard and emergency evacuation base. At P12, P13 and P15, which are located within the river, underwater construction is being conducted using a crane ship, material transport and other barges (Photo 3).

#### ● Driving of Steel Pipe Piles

In order to construct large-scale foundations and to drive and close steel pipe sheet piles with a maximum length of 50 m, it is important to execute precise vertical driving of the piles. At the construction site, the pipe piles were driven using hydraulic vibratory hammers in combination with the

Photo 3 Barges used for underwater construction

Photo 4 Water jetting nozzle

Photo 5 Final driving of steel pipe piles using diesel hammer

water jet method. This driving method was applied to drive piles up to a depth of 6D (D=pile diameter) above the pile tip, whereas final driving into the bearing stratum was undertaken using diesel hammers. (Refer to Photos 4, 5)

One test pile was driven for each foundation, and the bearing capacity was con-

Photo 6 Interior of well foundation

Photo 7 Discharge hole of underwater drilling pump

firmed by means of PDA (pile driving analyzer).

### ● Temporary Cofferdams

In the design stage, historical water level data for the Red River was relied upon to set the water level at the time of construction at sea level + 9.5 m, excluding the two months in summer when the water rises to its maximum level. Because of the large difference in water heads, various measures were taken to reduce the residual stress on the driven piles. These included studies and improvements for multi-step timbering, water level regulation, arrangement of the bottom slab placement period and other devices.

During the actual work, the water level rose to sea level + about 7 m due to water

Photo 8 Full-scale model prepared to examine installation of reinforcing bar cage for main tower bottom section

shortages in 2010. Because of this, construction work continued without interruption even during the summer season, moreover, it was possible to lower the height of the cofferdam by 1 m below the design level.

### ● Underwater Excavation

The method adopted to conduct underwater excavation inside the steel pipe sheet pile well was using pumps to discharge both water and riverbed sand (Photos 6, 7).

### ● Subsequent Processes

After excavation, underwater placement of the bottom concrete slabs, stud welding, reinforcing bar arrangement and concrete placement of the top slabs will be undertaken; following this, the main towers will be constructed. These subsequent processes will be introduced at the next opportunity. (Refer to Photo 8)

## High Expectations for Japanese Technologies

The next two and a half years will see the continuation of on-site, high precision construction work, such as construction of the main towers, installation of the anchor boxes at the top of the towers and cantilever erection of the main girders. We expect that, by capitalizing on Japan's advanced technological capabilities, the Nhat Tan Bridge will be safely completed and opened to traffic, and that the bridge will contribute to the development of the Vietnamese economy.

***Acknowledgments***

We wish to express our sincerest gratitude to Mr. Yamaji, Project Manager, and Mr. Mimura, Design Manager of the Nhat Tan Bridge Construction Site Office of Sumitomo Mitsui Construction Co., Ltd. for their generous cooperation during our inspection of the construction work on the steel pipe sheet pile well foundations.

# Tokyo Gate Bridge

## — Design and Construction of Steel Pipe Sheet Pile Foundation—

By Dr. Osamu Kiyomiya
Professor, Waseda University

**Osamu Kiyomiya :** Professor, Civil and Environmental Engineering, Waseda University. After graduating from the School of Engineering, Tokyo Institute of Technology, he entered the Ministry of Land, Infrastructure, Transport and Tourism in 1973. Then he filled key posts at the Ministry's Port and Harbor Research Institute. He became professor of Waseda University in 1997.

### Project Outline

At the Port of Tokyo, construction project has begun on an 8-km section of the Tokyo Port Coastal Highway to enhance the smooth distribution of international cargo between the Port of Tokyo and its coastal cities. Constituting a segment of this section of the highway, the Tokyo Gate Bridge is a large-scale structure composed of a main bridge (continuous 3-span truss-box composite bridge) that spans the No. 3 fairway of the Port of Tokyo and two offshore-onshore approach bridges (continuous multi-span steel slab box girder bridges) at both sides of the main bridge. The bridge has a total length of 2.9 km and is scheduled to be completed in fiscal 2011.

Because the construction area of the Tokyo Gate Bridge is located at a layer of alluvial clay (N value ≈ 0) that is 30 m or more thick, the foundation of the bridge rests on bearing strata of sand and gravel located at the deepest section of the No. 7 stratum. And since the foundation lies at least 65 m below the sea mud line, it is built as a great-depth type structure. Because the foundation is required to possess a seismic deformation capacity suitable for expected seismic motion between 534.7 Gal and –434.2 Gal and because economical structural sections must be built, it was decided that a large-scale steel pipe sheet pile well foundation (pipe pile diameter: 1,500 mm; checkered steel plates + high-strength mortar-filled interlocking joints) is adopted.

In this article, the large-scale steel pipe sheet pile well foundation that was adopted for construction of the Tokyo Gate Bridge is outlined and its construction method is discussed.

Two features should be cited regarding the construction of the Tokyo Gate Bridge. One is that the bridge spans the No. 3 fairway of the Port of Tokyo (crossing width: about 310 m; below-girder clearance: A.P. 54.6 m), and the other is that the construction work had to adapt to impediments posed by the restricted area surrounding Tokyo International Airport (A.P. +98.1 m). In order to satisfy these restrictions at both the design stage and the construction stage and to construct both a safer and higher-quality bridge through the pursuit of structural and economical rationality, it was necessary to adopt the latest in bridge technology: steel slabs utilizing large-size ribs, truss welds attained by eliminating the use of splice plates, BHS (bridge high-performance steel) products, large-size base-isolated shoes etc.

Design of the steel pipe well foundation was made in accordance with the *Specifications for Highway Bridges* (Japan Road Association) and the *Design and Construction of Steel Pipe Piles* (Japanese Association for Steel Pipe Piles). These two specifications state that the required performances of the foundation is constantly satisfied with regard to bearing force and overturning and that, in level 1 seismic motion, the structural materials remain within the allowable stress and the horizontal displacement is within 50 mm, and that, in level 2 seismic motion, the material remains within the yielding value and the yielding area of the foundation ground is within 40%.

### Foundation Structures

The substructure of the Tokyo Gate Bridge (total length: 2.9 km) is composed of two abutments and 21 piers (Fig. 1), of which 9 piers are located in the offshore section (extension: about 1.6 km) of the substructure. Of the offshore piers, the main piers (MP2, 3) are of the RC wall type (Photo 1) and the side-span piers (MP1, 4) are of the RC hollow type.

As shown in Fig. 2, a layer of soft alluvial clay (AC2 layer, N value ≈ 0) is thickly deposited over the foundation ground in the vicinity of the steel pipe well foundation. Further, the layers that serve as the bearing strata of the piers are located at great depth: the gravel layer (Dg1 layer) for the CP9~MP2 piers is located at A.P.–75 m or deeper, and the sand layer (Ds2 layer) for the MP3~WP6 piers at A.P.–50 m or deeper. The gravel layer at A.P.–75.5 m mostly serves as the bearing stratum of the CP9~MP2 piers, and the sand layer in the vicinity of A.P.–54.5 m~–50.5 m, shallower than the above, serves as the bearing stratum of the PM3~WP6 piers.

Photo 2 shows the driving of the pipe piles. Each pile is first driven into the bearing stratum using a vibratory hammer and then a hydraulic hammer. The hammers used are IHC-S280 and IHC-S200. The planned embedding length to the bearing stratum is 3.0D~3.2D. The outer diameter of the steel pipe piles used is 1,500 mm, and the wall thickness of the pile tip is 17 mm. Photo 3 shows the construction of the steel pipe sheet pile well foundation.

**Fig. 1  Entire Drawing of Tokyo Gate Bridge**

Bridge type: 3-span truss-box composite bridge (760 m)
Road classification: Type 4, Class 1
No. of lanes: 4 lanes (for both directions)
Total extension: 2,933 m (offshore section: 1,618 m)
Design speed: 50 km/hr
Planned completion: 2011 (project undertaking in 2001 by the Ministry of Land, Infrastructure, Transport and Tourism)
Total expenditure: ¥141,000 million

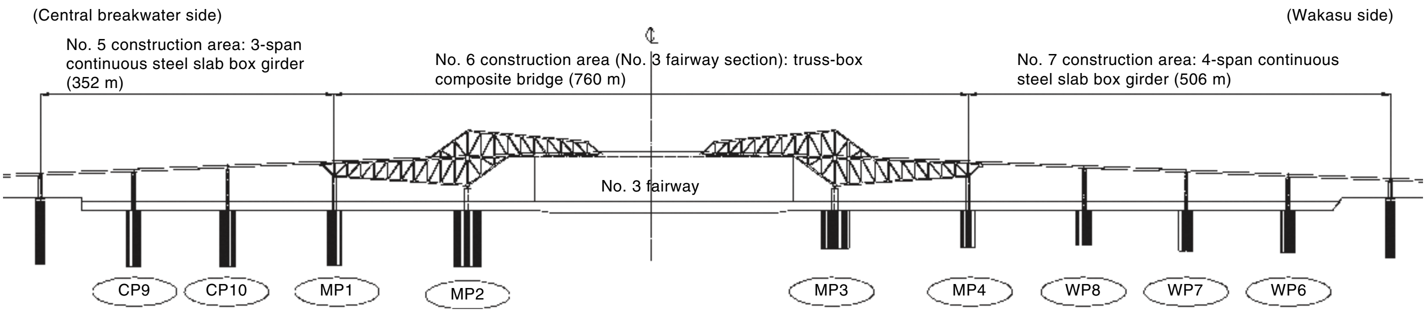


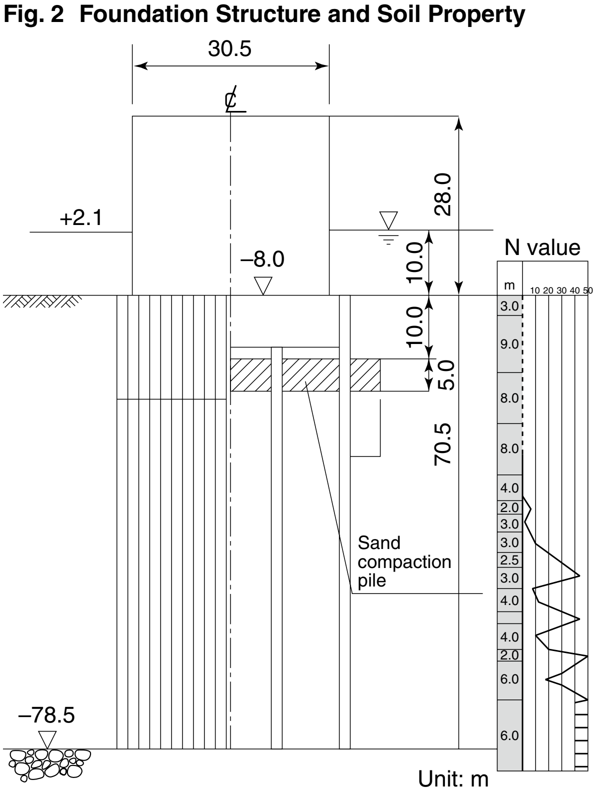


Photo 1  Construction of MP2


Photo 2  Driving of steel pipe piles


Photo 3  Construction of steel pipe sheet pile well foundation

**Fig. 2  Foundation Structure and Soil Property**




## Loading Tests for Vertical Bearing Capacity

In applying steel pipe sheet piles (diameter: 1,500 mm) in the construction of the Tokyo Gate Bridge, on-site loading tests were carried out in 2003 prior to the start of the construction work. The aim of the tests was to clarify the bearing mechanism of large-diameter steel pipe sheet piles that previously had incurred problems in similar work implemented in Tokyo Bay, and to modify conventional design and construction management of the foundation.

Three kinds of loading tests were carried out: dynamic loading tests (DLT), static loading tests (SLT) (horizontal and vertical) and rapid loading tests (STN), which led to the following results:

- Pile tip resistance by means of wave matching analysis from the dynamic test
- Skin friction resistance from the static loading test
- Set-up ratio (ratio of pile-tip static resistance during installation to tip resistance after ground recovery) of tip resistance from the dynamic and static loading tests
- Relation between the skin friction resistance and the N value from the rapid loading test

(Refer to Tables 1 and 2, Fig. 3)

In relation to the design of the steel pipe pile well foundation, the embedding length of the steel pipe piles into the foundation layer and the enclosure ratio of the pile tip are prescribed. Further, the ground reaction coefficient, vertical-direction ground reaction and other factors described in the *Specifications for Highway Bridges* are confirmed. Photo 4 shows the static loading test. Utilizing four pipe piles as the reaction pile, test pipe piles were pressed into the bearing stratum to a depth three times the pile diameter by means of a multi-cycle system employing hydraulic jacks. The hydraulic jack capacity was 48,000 kN for pile ④, and

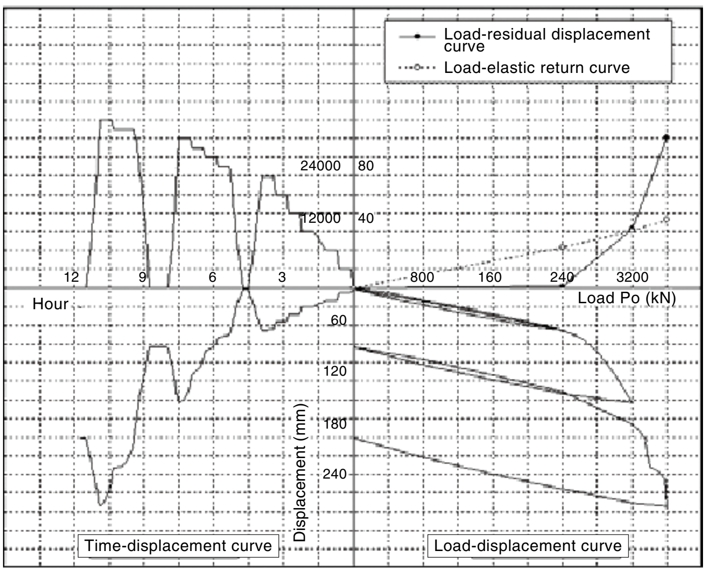
**Table 1 Test Piles and Kinds of Loading Tests**

| Test pile No. | Pile diameter D (mm) | Pile tip depth A.P. (m) | Test kind |
|---|---|---|---|
| ① | ϕ1500 | −86.0 | DLT |
| ② | ϕ1500 | −78.5 | DLT |
| ③ | ϕ1200 | −72.6 | DLT, STN, CPT |
| ④ | ϕ1500 | −73.5 | DLT, SLT, HLT, CPT |
| ⑤ | ϕ1500 | −86.0 | DLT, SLT, CPT |
| ⑥ | ϕ1500 | −86.0 | DLT |
| ⑦ | ϕ1500 | −86.0 | DLT |
| ⑧ | ϕ1800 | −89.0 | DLT |
| ⑨ | ϕ 800 | −42.0 | — |

**Table 2 Kinds and Objectives of Loading Tests**

| | |
|---|---|
| Kinds of loading tests | • Static (vertical) loading test<br>• Static (horizontal) loading test<br>• Dynamic loading test<br>• Rapid loading test |
| Test parameters | • Pile diameter<br>• Embedding length to bearing stratum<br>• Soil property of bearing stratum (sand or gravel) |
| Examination of design values | • Embedding length<br>• Tip bearing capacity<br>• Skin friction resistance<br>• Ground reaction coefficient (vertical and horizontal directions)<br>• Coefficient in bearing capacity calculation equation<br>• Enclosure ratio and correction coefficient of safety factor |

**Fig. 4 Results of Static Loading Tests**



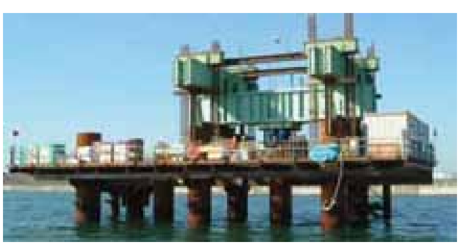



Photo 4 Static loading test

56,000 kN for pile ⑤.

Fig. 4 shows the relation between the applied load and the vertical displacement of pile ⑤. The maximum load was 36,000 kN, and the maximum displacement, 280 mm. Displacements up to about 60 mm were within the elastic range, and the yielding load was 2,000 kN. The strains were measured at 13 sections of the pile, and the axial force distribution and the skin friction resistance were calculated from the measurement results. Fig. 5 shows the distribution of the axial force of the pile. The axial force was calculated from strain meters set in the pipe pile. The difference in axial force between the respective measured sections can be converted as friction resistance. The skin friction resistance is somewhat low in the upper clay layer, but becomes larger in the lower layer. Fig. 6 shows the relation between the pile tip bearing capacity and the total load displacement, which were obtained by separating from the total force. It can be seen from the figure that the skin friction resistance is considerably larger than the tip bearing capacity for pile ⑤. Further, the friction resistance of both the inside and outside surfaces of the pile can be separated from the friction resistance at the pile tip. It was found in the test that the inside-surface friction resistance becomes considerably larger than the outside-surface friction resistance and that the sand and gravel inside the pile are compacted.

As shown in Fig. 7, in the rapid loading test, the 160-ton reaction mass affixed to the pile head was lifted upward at an acceleration of about 20G by the combustion pressure of the propellant, and the load was pseudo-dynamically applied to the pile head using the reaction thus generated by lifting with a loading time of about 0.1 second. This test offers advantages such as shorter test times and the elimination of reaction piles. Photo 5 shows the loading device. The axial force and acceleration distribution are calculated using a load cell, strain meter and accelerometer installed on the pile. Fig. 8

**Fig. 3 Relation between Bearing Stratum Thickness and N-value at Test Position**

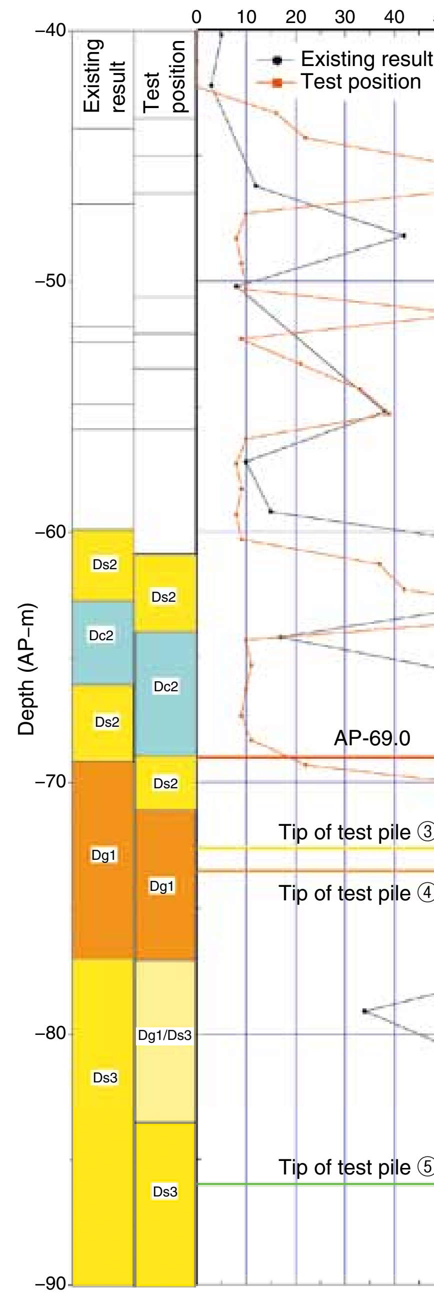


**Fig. 5 Vertical Distribution of Axial Force**

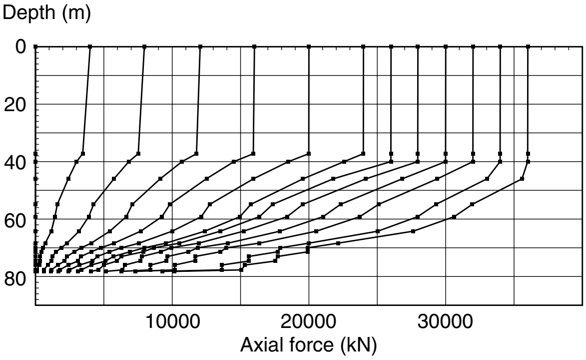
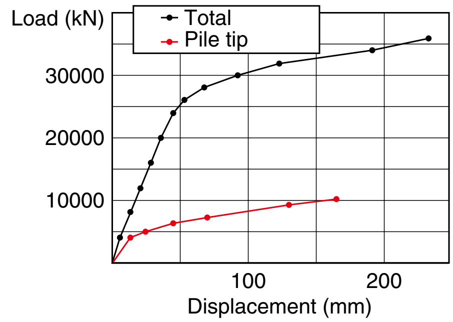



**Fig. 6 Tip Bearing Capacity and Total Bearing Capacity**




for the sand layer and 74% for the gravel layer. When a cross rib was attached to the pile tip, the pile tip resistance increased by about 30%.

- The skin friction resistance was estimated to be added to the pile-inside resistance, and the value shown in Table 3 was adopted.
- It was found in the horizontal loading test that the deformation coefficient of the ground was 2~3 times larger than that described in the *Specifications for Highway Bridges*.

**Fig. 7 Structure of Rapid Loading Test Device**

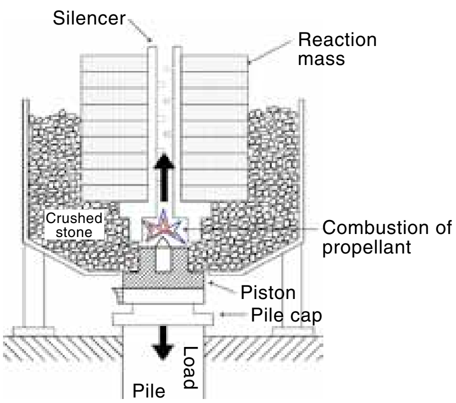



Photo 5 Loading device

shows the relation between the measured load and the displacement. Fig. 9 shows the model used to calculate the pile tip bearing capacity and the skin friction resistance. Calculation is made by setting the pipe pile as the elastic body and linking the peripheral ground, spring and dashpot while at the same time changing the ground constant by using the pile head input load so that the wave forms of both the measurement and the calculation agree each other. Fig. 8 shows the temporal changes of both the load and the displacement calculated together with the measurement results. Further, the input wave (hammer load) and the reflection wave (ground resistance) can be calculated from the axial force and acceleration. These value forms are shown in Fig. 10.

Fig. 11 shows the axial force and skin friction resistance of the piles as obtained from wave-form matching analysis. While the design of the steel pipe sheet pile well foundation was basically implemented according to the method described in the *Specifications for Highway Bridges*, various values applied in the design were modified based on the loading test results. Modification was made mainly of the following:

- The driving depth of pipe piles with an outer diameter of 1,500 mm (D) was set at 3D. The apparent enclosure ratio was 53%

## Loading Tests for High-strength Steel Pipe Sheet Pile Joints

Commonly, the deformation strength of a steel pipe sheet pile well is obtained from the bending rigidity of the steel pipe sheet pile structure and the shear resistance of the interlocking joints. In the case of adopting commonly applied pipe-to-pipe joints, the pipe piles are interlocked to each other by the use of steel pipes with a diameter of 165.2 mm, and mortar with a compression strength of about 20 MPa is used to fill the interlocked pipe joints. In the current Tokyo Gate Bridge project, it was necessary to increase the rigidity of the joints so that the horizontal displacement of the steel pipe sheet pile well foundation could remain within its allowable value during earthquakes.

In order to increase the rigidity of the steel pipe sheet piles, high-strength mortar was used to fill the joint openings (Fig. 12) to increase the adhesion between the mortar and the pipe joints. In the current project, the strength of the fill mortar was increased to more than double (40 MPa) the conventional level, and, further, checkered steel plate was used as the material of the pipe joints to increase the adhesion. As a result, it was confirmed that sufficiently large shear resistance could be obtained by the use of these two approaches.

As the results of the model loading test, a maximum shear resistance of about 1,640 kN/m was obtained as shown in Fig. 13. The values obtained in the test were about six times the upper limit (200 kN/m) of the

**Fig. 8 Matching Condition for Wave Form**

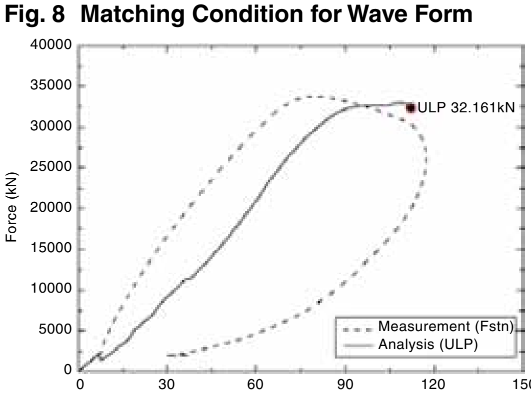



**Fig. 9 Calculation Model for Tip Bearing Capacity and Skin Friction Resistance**

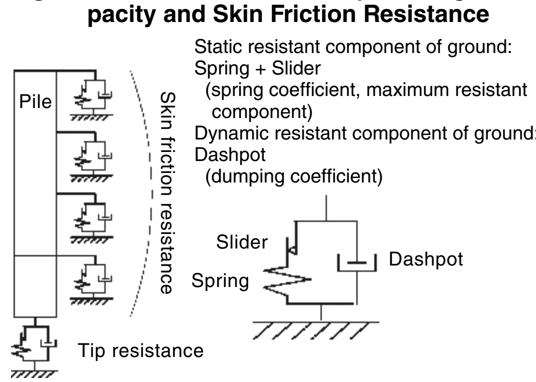


**Fig. 10 Input Wave Form Fd and Resistance Wave Form Fu**

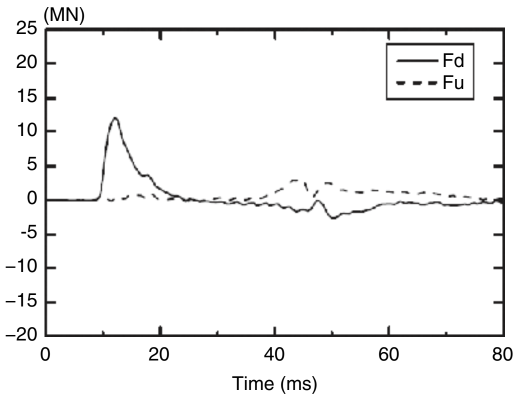



**Fig. 11 Results of Wave Form Matching Analysis**

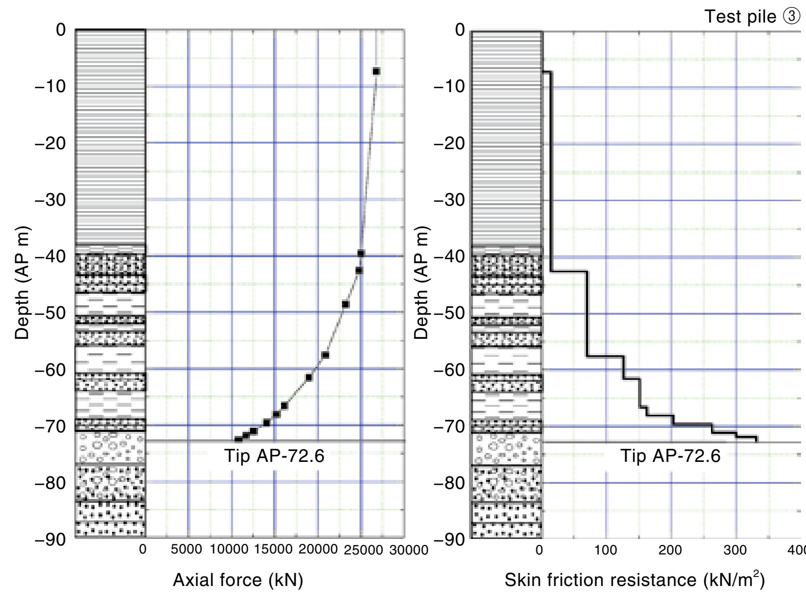



**Table 3 Fixing of Skin Friction Resistance**

| | Fixing by *Specifications for Highway Bridges* (kN/m²) | Fixing by loading test result (kN/m²) |
|---|---|---|
| Sandy soil | 2N (≤100) | 2N (≤100) |
| Clay soil | 10N (≤150) | 10N (≤150) |
| Gravel layer | 2N (≤100) | 3N (≤150) |

**Fig. 12 Interlocking Structure and Loading Test Device**

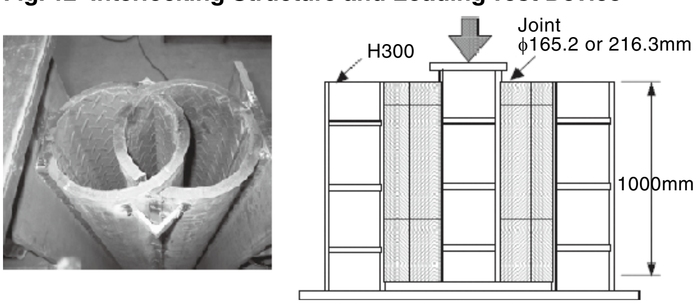
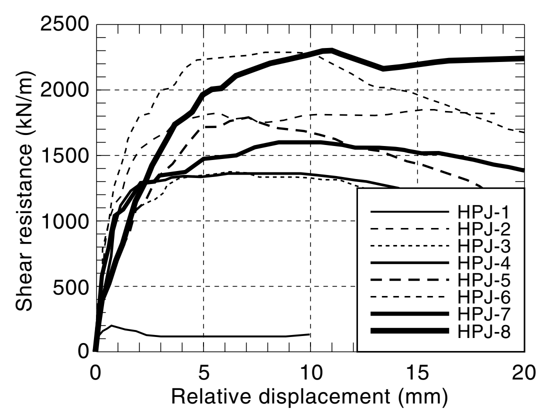



**Fig. 13 Relation between Load and Displacement**




shear resistance described in the common design specifications for steel pipe joint material (*Manual for Design and Construction of Steel Pipe Sheet Pile Foundations*, Japan Road Association).

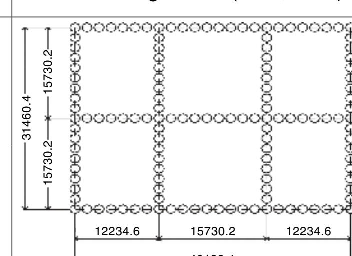
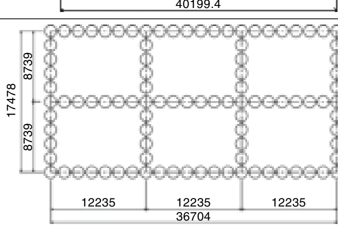
The configuration of the steel pipe sheet pile well foundation designed in the initial stage was greatly reduced through the appropriate use of the loading test results and a reexamination of the ground coefficients, which allowed for economical construction (Table 4).

## Reduced Construction Cost and Improved Quality

The steel pipe sheet pile well foundation of the Tokyo Gate Bridge was constructed on soft ground, and, further, large sectional force and deformation capability during earthquakes were required in its construction. Thus in the design stage, the foundation structure necessarily became quite large in terms of structural configuration.

After acquiring knowledge of the bearing mechanism at the construction site from the loading tests conducted on large-diameter steel pipe sheet piles, various design factors were reconsidered. Based on these, a more compact foundation configuration was successfully achieved by the combined use of large-diameter steel pipe piles and interlocking joints made of checkered steel plate filled with high-strength mortar. Similarly, the construction management procedures were also determined by the effective use of the loading test results. As a result of these considerations, the structural dimensions of the steel pipe sheet pile foundation were reduced, thereby simultaneously producing greater reductions in construction cost and improvements in structural quality.

**Table 4 Comparison of Designs of Steel Pipe Sheet Pile Well Foundation**

| | Planar configuration (MP2, MP3) | No. of pile / Pile length |
|---|---|---|
| Initial design |   | 136 / 84.5 m |
| Final design |   | 98 / 81.5 m |

*Special Report*

# Great East Japan Earthquake and Tsunami

Takeshi Oki
Chairman, Committee on Overseas Market Promotion
The Japan Iron and Steel Federation

The Great East Japan Earthquake that occurred at 14:46 on March 11, 2011, registered a magnitude of 9.0, making it the largest quake ever recorded in Japan. The resulting damage was unprecedented in its extent and ranged mainly along the Pacific coast of East Japan (Iwate, Miyagi, Fukushima and Ibaragi prefectures). The number of dead and missing together reached more than 27,000 and the number of completely destroyed buildings exceeded 65,000.

This massive earthquake was of the interplate type, with its epicenter at a point 24 km below sea level off the Sanriku Coast in the Pacific Ocean (approximately 130 km east-southeast of the Oshika Peninsula). Following the initial displacement of the fault at the epicenter off the Sanriku Coast, fault plane displacements subsequently occurred in the hypocentral region of the original earthquake, covering an extensive area off the coastline that runs from Iwate to Ibaraki and measures about 500 km south to north and 200 km east to west. Shortly after the main shock, a number of major aftershocks with magnitudes of 7.0 or over occurred in rapid succession in the region.

The devastating tsunami that followed the earthquake made the damage even more critical. Tsunami waves exceeding 3 m in height hit the main ports and harbors along the Pacific Ocean and caused considerable damage. A 9.5 m-high tsunami was observed sweeping far to the interior of Port of Ofunato. According to a schematic drawing of flooded areas prepared by the Geospatial Information Authority of Japan, the Ministry of Land, Infrastructure, Transport and Tourism, land areas extending inwards from the coast for 5 km or more were flooded, covering a vast expanse of land that extends from Ishinomaki in Miyagi Prefecture to the central part of Fukushima Prefecture.

A large vessel running onto the quay of Sendai Port due to the tsunami

View of a tsunami-struck area near the Natori River, Miyagi Prefecture

***Acknowledgements***

We wish to express our concern for all those citizens and their families who have suffered greatly as a result of the Great East Japan Earthquake. Much mail expressing sympathy and encouragement in overcoming the severe conditions has been received not only from people in Japan but from abroad. We would like to take this occasion to express our sincere gratitude to everyone, including our readers, who have extended extensive support and great kindness.

The Japan Iron and Steel Federation offers diverse steel construction methods and technologies in the civil engineering and building construction fields that will be helpful for the early restoration and reconstruction of the stricken areas. In the future, we will conduct research and surveys on the damage caused by the earthquake and tsunami, and will strive to develop and disperse steel construction as one link among many endeavors to promote better disaster-prevention measures.

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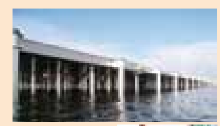
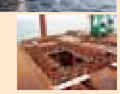
C O N T E N T S







**COVER**
(*top*) Seawater-resistant stainless steel lining of steel piles composing the steel jacket at the new No. 4 Runway of Tokyo International Airport (for details, see page 1); (*bottom*) Steel pipe sheet pile foundation used for the construction of Tokyo Gate Bridge (see page 14)

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**The Japan Iron and Steel Federation**
3-2-10, Nihonbashi Kayabacho, Chuo-ku, Tokyo 103-0025, Japan
Phone: 81-3-3669-4815 Fax: 81-3-3667-0245
Chairman: Eiji Hayashida
URL http://www.jisf.or.jp
**Japanese Society of Steel Construction**
Yotsuya Mitsubishi Bldg. 9th Fl., 3-2-1 Yotsuya, Shinjuku-ku, Tokyo 160-0004, Japan
Phone: 81-3-5919-1535 Fax: 81-3-5919-1536
President: Koichi Takanashi
URL http://www.jssc.or.jp
**Editorial Group**
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**The Japan Iron and Steel Federation**
**● Beijing Office**
Room 2206, Jingtai Tower, 24 Jianguomenwai-Street, Chaoyang-District, Beijing 100022, China
Phone: +86-10-6515-6678
Phone: +86-10-6515-6694

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