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港

【 *m i n a t o* 】

"港(*minato*)" in Japanese, or port in English

Planned maintenance of port facilities and improvements to lifecycle cost assessment technology and to the reliability of port steel structures are becoming urgent tasks. This issue features methods to deal with these tasks.

In this issue.....

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Maintenance of Harbor Facilities

by Naoki Endo, Port Management Office, Engineering Planning Division, Ports and Harbors Bureau, Ministry of Land, Infrastructure, Transport and Tourism

Preface

Most of Japan's port and harbor facilities were constructed or underwent improvements in the 1970s and 1980s and are therefore expected to simultaneously enter superannuation in the near future. Like other social infrastructure, these port and harbor facilities need to receive appropriate measures to counter the effects of aging.

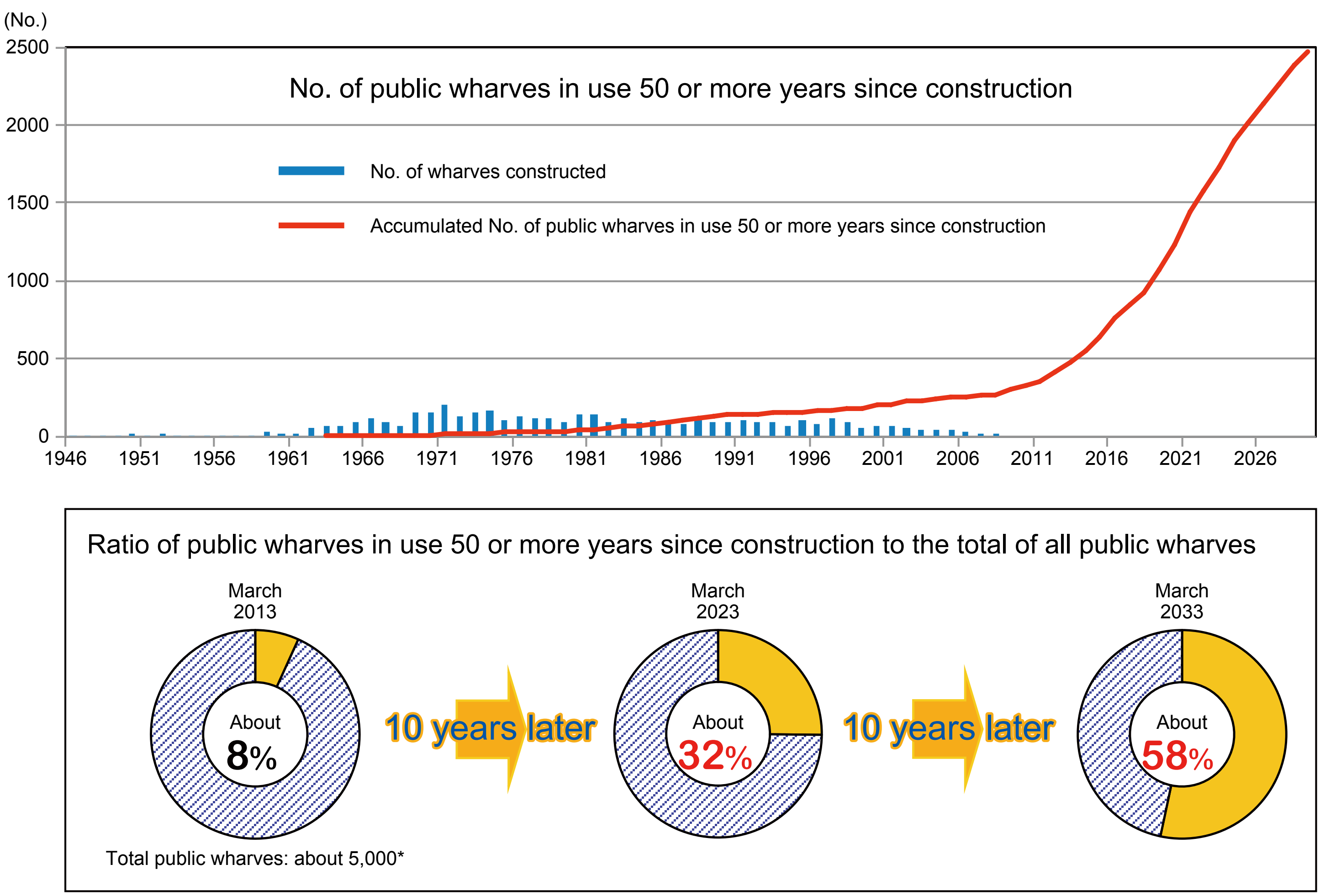
Japan, however, faces many other pressing concerns: severe fiscal limitations confronting both national and local governments, a declining population, a falling birthrate and an aging society. In light of these circumstances, it will be difficult to effectively deploy conventional countermeasures against the superannuation of the nation's social infrastructure. To cope with this, the Ministry of Land, Infrastructure, Transport and Tourism (MLIT) has established the Strategic Committee on Social Infrastructure Maintenance and numerous other committees mandated to study needed countermeasures against obsolescence. The following text introduces one of these countermeasures, a preventive maintenance plan for ports and harbor facilities.

From Posterior Maintenance to Preventive Maintenance

Public wharves serve a key role among port and harbor facilities. Of all the public wharves in service at the end of March 2013, about 8% had been in service for 50 or more years since they were constructed. However, this rate is forecasted to grow to about 58% after another 20 years at the end of March 2033 (Fig. 1). Because a growing number of these facilities are showing deterioration or damage caused by the aging process (Fig. 2), concern is rising with regard not only to functional deterioration but to the occurrence of accidents as well.

On-going superannuation poses two pressing problems: increasing lifecycle costs (the expense of future maintenance and improvements) and faltering public serviceability due to accidents and other adverse factors attributable to aging. To meet this situation, we consider it necessary to develop a new conceptual approach to maintenance: i.e., to transition from posterior maintenance, which does not implement needed countermeasures until drastic renewal work is judged necessary, to preventive maintenance that places primary importance on prolonging service life and minimizing lifecycle costs (refer to

Fig. 1 Construction of Public Wharves in Japan



*No. of public wharfs with water depth of 4.5 m or more (Source: MLIT)

Fig. 2 Examples of Superannuated Wharves

- Port and harbor facilities are exposed to salt damage and other severe application environments and they have many submerged sections that do not permit visual observation of deterioration and damage.
- Because of the above, the deterioration and damage that might occur to submerged steel sheet and pipe piles and at the rear of wharf floor slabs are overlooked, and this can lead to large accidents. Therefore, it is important to secure structural safety by means of appropriate maintenance.

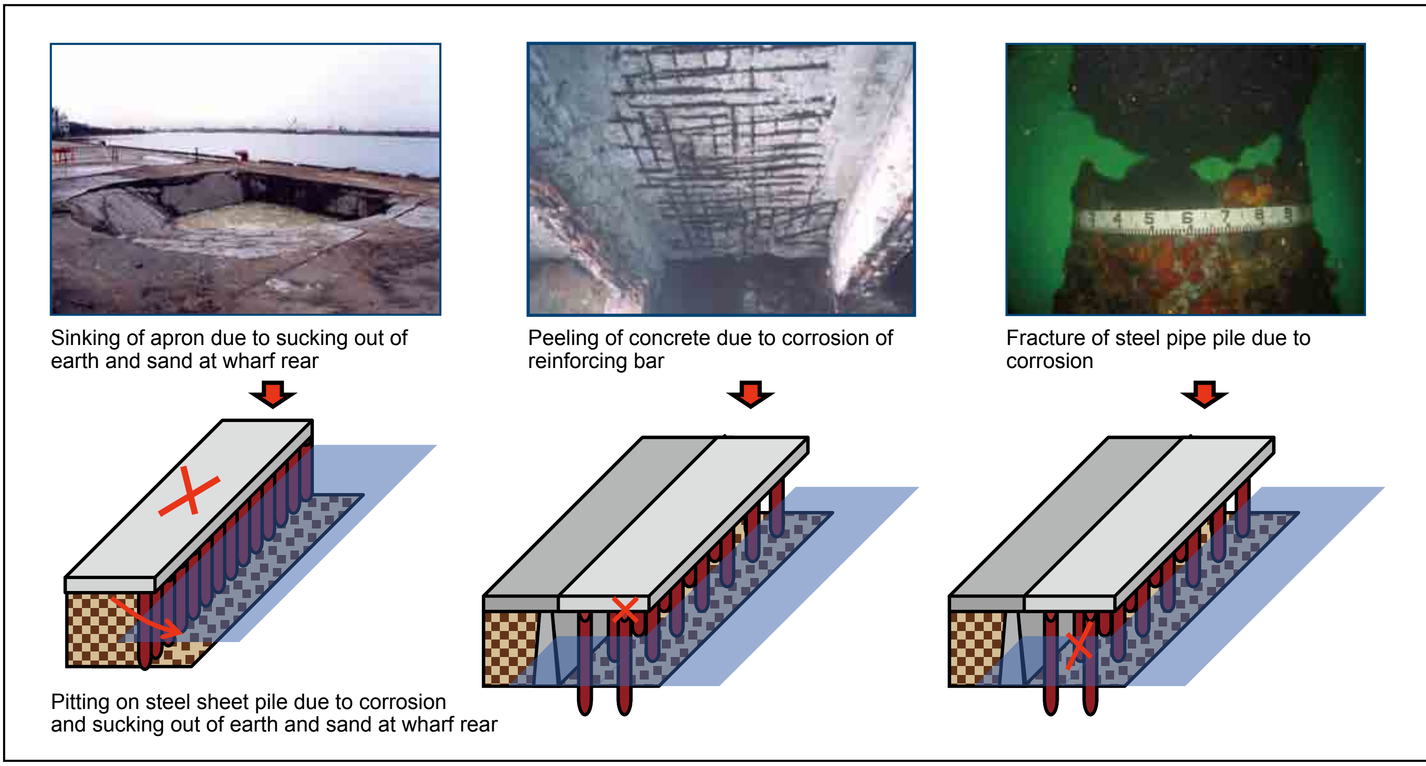


Fig. 3 Image of Preventive and Posterior Maintenance

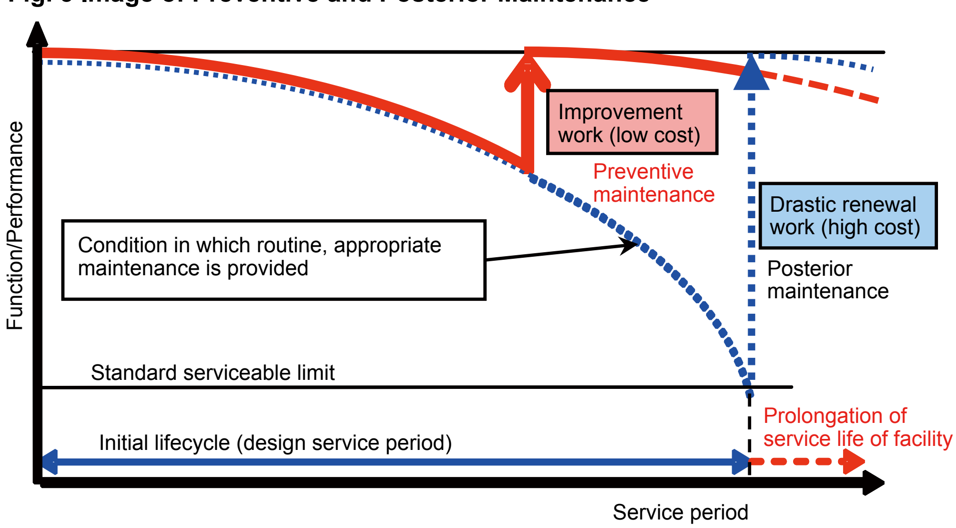


Fig. 3).

We also consider it indispensable that thoroughgoing maintenance should be put into effect by port and harbor administrating organizations, and that mechanisms should be worked out to prevent potential accidents and functional deterioration attributable to superannuation.

Specific Measures Promoted by the Ports and Harbors Bureau

In order to promote appropriately planned maintenance as a countermeasure to superannuation, the Ports and Harbors Bureau has striven to prepare laws and regulations and has implemented various measures related to technology and budgets aimed at preparing a maintenance plan and implementing appropriate maintenance based on that plan.

While the maintenance plan thus worked out has content suitable for maintaining individual port and harbor facilities, studies have yet to be made regarding project expense reductions in terms of all port and harbor facilities or regarding the leveling of annual project expenses. Given such a situation, the Bureau has introduced a "preventive maintenance plan" that allows comprehensive study on a port-by-port basis.

Preventive Maintenance Plans

The preventive maintenance plan targets various facilities within a port, and specifies countermeasures to the superannuation of facilities and it offers a nearly five-year project outline based on the countermeasures thus specified. The plan was basically developed on a port-by-port basis. The main entities involved in preparation of the plan were the national government (government organizations in charge of direct administration of ports and harbors) and port and harbor administrating organizations. The plan was worked out basically through necessary accommodations agreed to by both types of entities. The plan specifies in the "action guidelines" its basic policy determining whether or

Table 1 Preventive Maintenance Plan (Concept)—Preventive Maintenance Plan for Port A

Port name	Area name	Facility name	Completion year	Main applications	Relation with port application plan: Reexamination direction for current port application plan (whether or not reexamination has started on application conversion)	Obsolescence condition	Action guideline (draft)	Project outline (¥million): Approximate total project cost (¥million)	FY 2013	FY 2014	FY 2015	FY 2016	FY 2017
Port A	Area a	Wharf (1)	1959	Bringing in/out of gravel and sand	Reexmination has started on application conversion	Remarkable obsolescence such as pitting of pipe pile	Discontinuing of application as a wharf and conversion to service as a revetment				Dis-continued use		
Port A	Area b	Wharf (2)	1969	Use by regular container line	No plan for reexamination	Remarkable obsolescence such as cracking of superstructure	Because of imposition of application restriction, preferential implementation of preventive maintenance project	1,000					
Port A	Area c	Wharf (3): Seismic resistant	2001	Use by regular container line	No plan for reexamination	Sound	Implementation of continued inspection and survey						
Port A	Area d	Breakwater (1)	1965	Securement of calmness at Area d within Port A	No plan for reexamination	Remarkable obsolescence such as sinking of crest by about 1 m	Within a range of not affecting calmness inside port, implementation of continued inspection and survey						
Port A	Area e	Route/Anchorage	2002	Route/anchorage for vessels that use Area e	No plan for reexamination	Sound	Implementation of continued inspection and survey						
Port A	Area f	Road (1)	1965	Access road for vehicles that use Area f	No plan for reexamination	Rusting in bridge pier, showing marked progress of obsolescence	Planned implementation of preventive maintenance project	300					
Port A	Area g	Wharf (4)	1980	Export of completed automobiles	No plan for reexamination	Sound	Implementation of continued inspection and survey						
Port A	Area h	Wharf (5)	2000	Import of coal	No plan for reexamination	Sound	Implementation of continued inspection and survey						
Port A	Area i	Breakwater (2)	1995	Securement of calmness at Area i within Port A	No plan for reexamination	Sound	Implementation of continued inspection and survey						
Port A	Area j	Road (2)	1993	Access road for vehicles that use Area j	No plan for reexamination	Partial scattering of sinking section	Because of no obstruction in application, implementation of continued inspection and survey						

Hinode No. 4 wharf at Hinode area, Port of Shimizu (pier type, water depth of 12 m, completed in 2011)

not countermeasures against superannuation are to be implemented (see Table 1).

The determinations made using the "action guidelines" are based on a comprehensive overview that takes into account the social conditions surrounding a target facility (current application conditions, examination of application conversion plan or no examination of alternative conversion plan, assumption of emergency use or no assumption, requests from port users) and the physical conditions (level of superannuation, structural characteristics).

Further, in the decisions suggested by the "action guidelines," it is necessary to examine not only facilities in active implementation of superannuation countermeasures but also facilities that should be converted to other purposes or for which application should be discontinued. For example, preventive maintenance is not performed on facilities standing on sites planned for reclamation or on facilities to be converted to other uses because they can no longer serve as a mooring facility. In this way, it is important to pay due attention in planning preventive maintenance so that a rational plan can be developed that takes into account the minimization of lifecycle costs for all the facilities of the target port. In addition, the plan must be able to appropriately maintain the port facilities as a stock while at the same time securing all necessary functions, and it must have planned and strategic content.

Conclusion

The preventive maintenance plan is to be initiated in 2013. In order to better prepare the plan, we intend to reexamine the current plan by taking into account emerging changes in social trends and the various tasks involved in implementing the preventive maintenance projects. We at the Ports and Harbors Bureau are striving to promote effective countermeasures against the superannuation of port and harbor facilities. These efforts include: implementation of the current preventive maintenance plan that takes into account the operations of the examination council as it continues deliberations on the plan and the maintenance measures based on the new legal system; and cooperation with port administration organizations and other related entities. Our goal is to maintain good-quality public service. ■

Life-Cycle Management of Port and Harbor Steel Structures

by Hiroshi Yokota, Professor, Lifetime Engineering Laboratory, Hokkaido University

Introduction

A port and harbor structure has a long lifetime and must be expected to meet demands during its lifetime that cannot be foreseen. Steel and concrete as principle structural materials tend to deteriorate due to physical and chemical agents under harsh marine environments and loss in structural performance or even structural collapse may be consequences. At the initial design stage, designers make several assumptions, in which probably worst conditions are considered with certain safety margins, so that the structure can keep its structural performance over respective required levels. However, serious deterioration of structural members may be caused by insufficient durability design with optimistic assumptions and/or by lack of proper maintenance work.

To meet these facts, it is extremely important to pursue coordination between design and maintenance. The life-cycle management is an organized system to support engineering-based decision making for ensuring sufficient structural performance and long life of a structure at the design, maintenance, and all related work during the lifetime of a structure.

Fig. 1 Life Cycle of A Structure

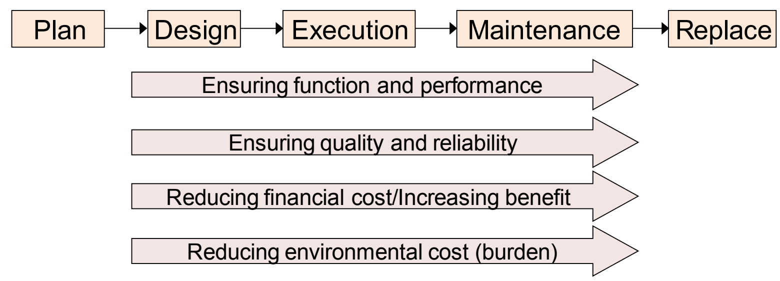
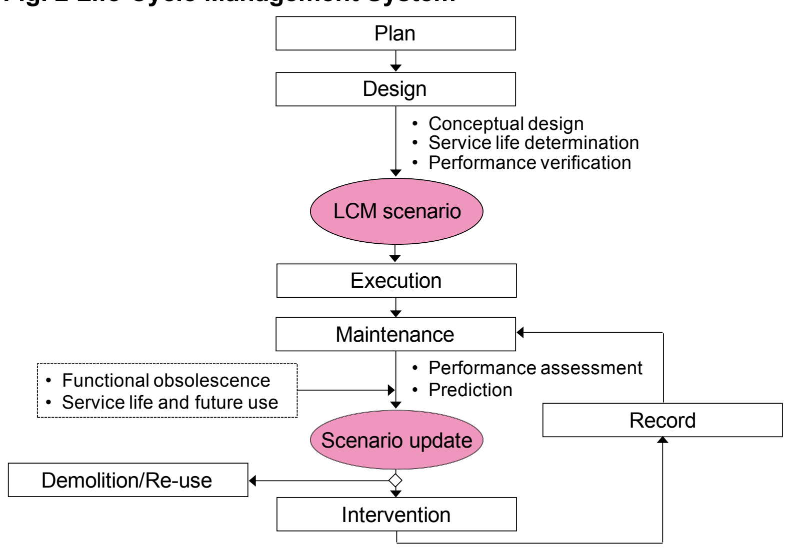


Fig. 2 Life-Cycle Management System



Life-Cycle Management

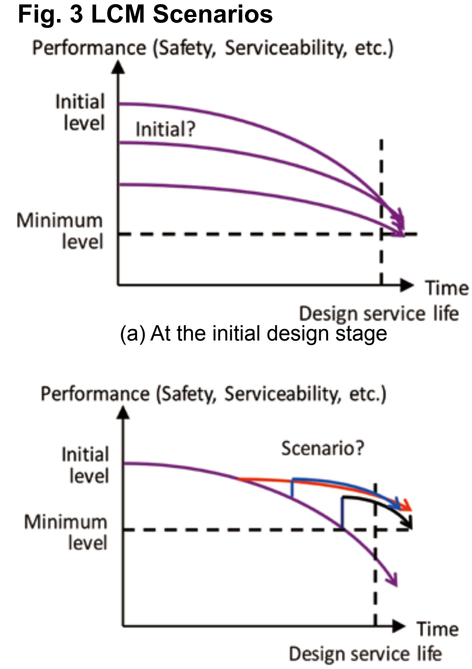
The service life of structure is made up of all the activities that go into planning, basic and detailed designs, execution including material selection, production and construction, maintenance including assessment and intervention, and decommissioning, as shown in Fig. 1. The life-cycle management is an integrated concept to assist all activities managing the total life-cycle of structures to realize sustainability. Sustainability is one of the very important keywords for port and harbor structures, not only them but for all civil infrastructure.

In the life-cycle management, as shown in Fig. 2, the most important work is to formulate and to update the life-cycle management scenario (LCM scenario). The scenario should be formulated in consideration of the following items:

- Environmental characterization;
- Assumption in designs;
- Result of verification;
- Specification;
- Initial cost estimation;
- Maintenance scenario and methodologies of the life-cycle management;
- Performance requirements;
- Service life estimation;
- Life-cycle cost;
- Environmental cost; and
- Obsolescence, demolition and reuse.

During the initial design stage, the service life design will be applied to predict the durability and performance degradation. As schematically shown in Fig. 3(a), while lots of alternatives can exist, the fundamental concept on how the structural performance should be ensured must be well considered based on conditions, design service life, structural characteristics, material properties, difficulties in assessment and intervention, social and economical importance, etc. For port and harbor steel structures, corrosion of steel

Fig. 3 LCM Scenarios



(a) At the initial design stage

(b) At the maintenance stage

is a principal cause of performance degradation to be considered at the initial design. Accordingly, corrosion of steel itself and/or deterioration of a corrosion protection system should be fully taken into account. In general, we do not have to worry about fatigue of steel for port and harbor structures.

Maintenance is the major strategy to counter the degradation, which is carried out to assess the present conditions of structure and to quantify the level of structural performance. In addition, by predicting future progress of structural performance degradation, the most appropriate method of intervention should be chosen for minimizing the life-cycle cost or maximizing structural performance recovery under budget capping, as shown in Fig. 3(b). During the maintenance stage, maintenance engineers will initially follow the scenario that had been assumed at the design stage. For realizing strategic maintenance work, as mentioned earlier, a maintenance strategy should be properly formulated as the LCM scenario. The service life design or durability design has been carried out based on lots of assumptions at the initial design. Accordingly the output of the design has to be verified with the maintenance work because progress of deterioration including corrosion would not follow the design assumptions. This is related to the LCM scenario update. The scenario should be updated with reflecting the actual situation of the structure and changes in conditions.

LCM Scenario

An example of the LCM scenario formulation will be presented[1]. An open-piled marginal wharf shown in Fig. 4 is taken up for a case study. The substructure of the wharf is made of steel pipe piles, of which diameter and thickness are 1500 mm and 19 mm

Fig. 4 Open-Piled Wharf for the Case Study

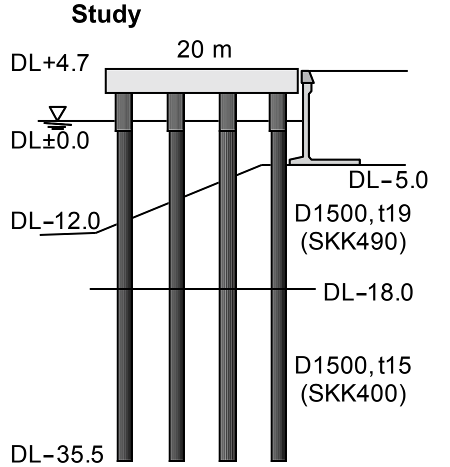
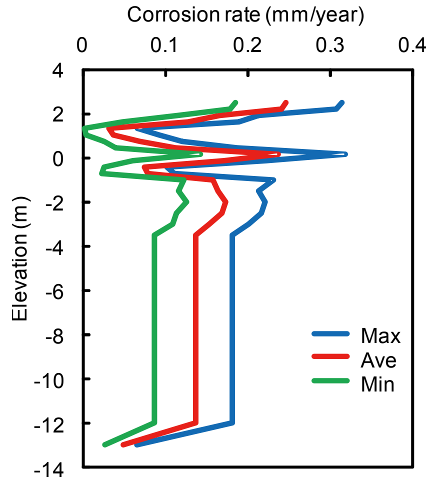


Fig. 5 Variation in Corrosion Rate



for the part above DL−18 m and 1500 mm and 15 mm below it. According to the corrosion rate of steel, the profile shown in Fig. 5 is used which had been measured in a neighboring wharf for 26 years. The progress of corrosion generally differs location by location. Thus, as shown in the figure, the maximum, the average, and the minimum values have to be considered to make probabilistic approach.

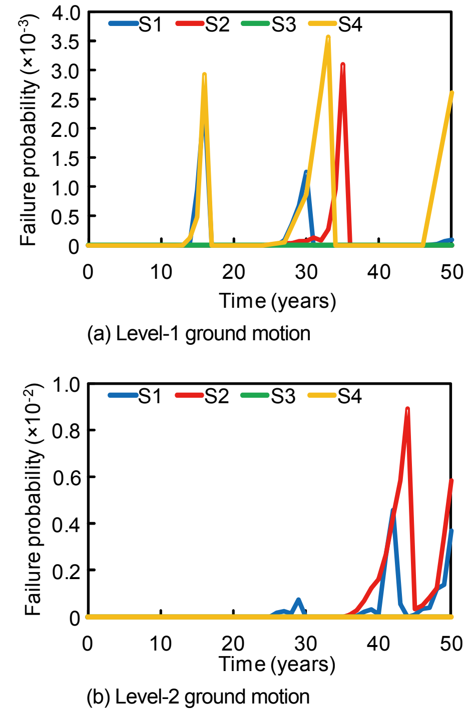
To counter such the corrosion risk, four LCM scenarios are formulated:

- S1: No corrosion prevention
- S2: Coating (DL+2.5~−1.0 m)
- S3: S2+S4
- S4: Cathodic protection (DL−1.0~−12 m)

The design life of the coating and cathodic protection systems is set at 50 years. In fact, many scenarios can be considered depending on the design life of the corrosion protection system. The corrosion protection efficiency of the cathodic protection is set at 90%[2].

The structural performance is verified by using the push-over analysis[3] and dynamic response analyses for level-1 and level-2 seismic ground motions[4]. The probability of failure of the wharf is increased due to corrosion of steel piles. Fig. 6 shows the failure probability over time against (a) level-1 ground motion and (b) level-2 ground motion. The allowable maximum values of the failure probability are set at 0.0038[2] and 0.01 for level-1 and level-2, respectively. Therefore, before reaching the values, strengthening is needed to recover the performance. Under those scenarios, S1 needs strengthening 3 times (16th and 30th years for level-1 and 44th year for level-2) during the design service life of 50 years. S2 needs strengthening 2 times at 36th year for level-1 and 45th year for level-2. While no strengthening is needed, S4 needs 2 times only for level-1 ground motion at 16th and 33rd years. For this calculation, steel plate welding is adopted for the strengthening. By the strengthening technique, load-carrying capacity can be perfectly recovered, but the ductility can be reduced by about 40% of the initial value due to the application of underwater wet welding[5].

Fig. 6 Increase in the Probability of Failure due to Corrosion of Steel Piles



Designers should select the best scenario among its alternatives. For the selection, the most appropriate indicator should be determined for objective judgment. The life-cycle cost will be one of the indicators, which will be mentioned later.

Assessment and Evaluation

Since performance degradation of a port and harbor steel structure is mainly caused by corrosion of steel, corrosion progress should be monitored during the service life. The progress of corrosion differs widely by its location because of inhomogeneous characteristics of materials and diversity of environmental conditions. By using the real corrosion data, the LCM scenario has to be updated.

Detection of age-related deterioration should be very important to understand the condition of a structure. The deterioration of coating and other corrosion protection systems can be found at first as damages on their surfaces. The level of inspection and investigation has an influence on the method of structural performance assessment. Assessment can be made by the condition-based concept and the performance-based concept[6]. The performance-based concept should be applied for performance assessment, but generally needs costs, advanced techniques, etc. Thus, the condition-based assessment is reluctantly accepted because of its feasibility. The grading system has been often applied, in which the state of deterioration is evaluated and judged using the deterioration grade.

Visual inspection has been most often applied to grade the condition of structure and make judgment whether further, detailed investigation is needed or not. For proper grading, inspection should be carried at with regular intervals. If the deterioration grade is associated with structural performance, the performance can be indirectly assessed. The visual inspection is only able to provide the change in appearance of structural member, but structural performance has to be evaluated as precisely as possible. If the relationship between structural performance (structural capacity) and the grade of deterioration could be found even tolerant margins of errors, the intervention could be discussed with using the deterioration grade[7].

When the visual inspection is insufficient to provide proper data for assessment, the detailed inspection is recommended to carry out. The detailed inspection or investigation includes quantification of the degree of deterioration with non-destructive or destructive techniques, etc. In case that the data of quality and amount is satisfactory to do numerical analysis, direct assessment can be made. As mentioned earlier, deterioration has considerable variation; thus, such variation is necessary to be precisely quantified.

Scenario Update

Some theoretical rules, simulation models, verification formulae, etc. are used for the prediction of deterioration progress. However, the trend that is observed through regular maintenance work, for example the measured corrosion rate, may have potentials for use in the future progress of deterioration and/or degradation during the maintenance stage. Based on the data and assessment results, the rule and process of deterioration and/or performance degradation have to be modified and the scenario be updated for further prediction.

Life-Cycle Cost

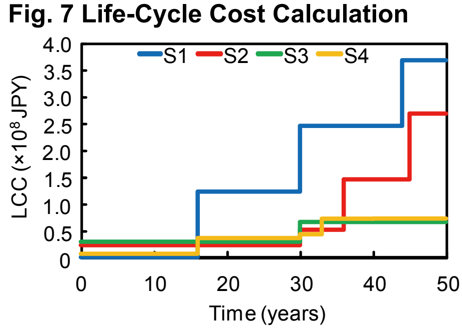
If member failures might cause hazards to safety, possible failures should be categorized by their consequences. To reduce the risk of failure occurring within the design life when the consequences of failure are judged to be critical, it may be necessary to require particularly long design lives for specific members or to strengthen the requirements for inspection and maintenance. To determine the LCM scenario including the appropriate timing and method of intervention, estimation of life-cycle cost is one of the best indicators[8].

The life-cycle cost is calculated based on various assumptions, but it provides important information needed for making decisions about the future direction of maintenance. In the calculation, the initial cost, maintenance cost including inspection cost, and the cost of planned intervention are totalled.

Life-cycle cost estimation enables comparative cost assessments to be made over a specified period of time. Being able to compare the costs of alternative LCM scenarios allows selection of the most economic overall strategy.

The life-cycle cost calculations of the four LCM scenarios are shown in Fig. 7. Since strengthening needs cost, just before reaching the allowable maximum value of failure probability, corresponding costs are increased. The figure indicates that preventive maintenance strategy is the least costly, which is about one-third of the most costly scenario. For 50 years, the most economical scenario is S3 followed by scenarios S4, S2, and S1. It means that the efficiency of corrosion prevention (preventive measure) should be the best scenario in terms of cost. However, if the intended service life is shorter than 50 years, the best solution might be varied depending on the service life.

Fig. 7 Life-Cycle Cost Calculation



Concluding Remarks

The life-cycle management system including evaluation of structural performance and prediction of the progress of deterioration would be one of the best tools for building and maintaining structures. The author expects that rational and effective maintenance is realized so that the life-cycle cost reduction and performance maximization can be attained. This may make it possible to realize sustainability of port and harbor structures. To further develop the system, research is necessary on accurate structural performance verification of existing structures including the methodology of inspection and investigation and precise prediction on future progress of deterioration and performance degradation. ■

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Influence of Defects and Exposure Conditions on Coating Degradation due to the Corrosion of Steel Piles and Plates in Marine Environments

by Nobuaki Otsuki, Professor, and Takahiro Nishida, Assistant Professor, Department of International Development Engineering, Graduate School of Engineering, Tokyo Institute of Technology

Corrosion of Offshore Steel Structures in Marine Environments

Because offshore steel structures are exposed to severe corrosion environments, it is important to understand how structural deterioration progresses due to corrosion in paint coated steel and to establish a method for forecasting declines in structural performance. Meanwhile, in order to assess the progress of long-term deterioration in a relatively short period of time, an acceleration test has been proposed[1].

However, the influence that exposure and acceleration conditions exert on the progression of quality deterioration has not yet been clarified. Accordingly, there is as yet no clarification of the "acceleration magnification," i.e. the correlation between the period of acceleration test and the exposure period in an actual corrosive environment. In particular, there are many examples in existing research that have noted the uniform corrosion (microcell corrosion) that occurs in steel products, but there are fewer examples of research that have examined the progress of deterioration targeting the localized corrosion (macrocell corrosion).

We have examined these issues from the perspective of coating degradation, the results of which are introduced below.

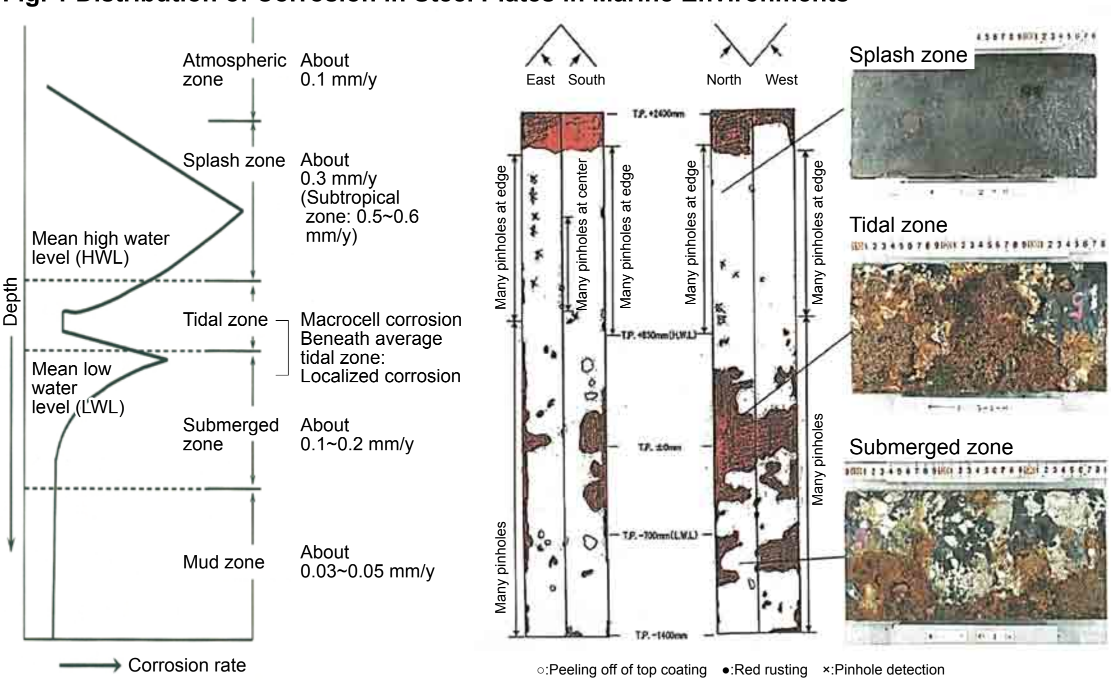
Corrosion of Steel Products in Marine Environments

Many examples exist of studies pertaining to the corrosion of unpainted steel products in marine environments. For example, the study results shown in Fig. 1 (a)[2] are obtained, through which it is known that the corrosion rate increases in the upper HWL section (splash zone) and in the zone below the LWL. In this way, the corrosion of unpainted steel products applied in marine environments progresses due to macrocell corrosion, which occurs mainly in (1) anodic areas where the corrosion rate increases and (2) cathodic areas where the corrosion rate decreases. In order to prevent such corrosion from occurring, it has become general practice to apply the corrosion-protection measures such as paint coating to offshore steel structures.

Among the major corrosion-protection mechanisms offered by painted coatings are: cutting off of corrosive substances such as Cl^-, O_2 and H_2O and suppressing the formation of reaction sites for corrosion products. It is conventionally thought that, when proper coatings are applied, coating degradation occurs with difficulty, and when a defect is created by collision by a ship or other structure with offshore steel structures, corrosion (mainly microcell corrosion) will occur in the area of the defect. However, it has been confirmed by the results of exposure tests conducted in an actual application environment (Bay of Suruga, 20-year exposure), shown in Fig. 1 (b)[1], that blistering, ripping and other types of coating degradation occur in actual application environments and that the area of corrosion expands.

We judged that the cause of such corrosion phenomena was attributable to macrocell corrosion, and conducted a study using special test specimens that enable the measurement of macrocell corrosion current using

Fig. 1 Distribution of Corrosion in Steel Plates in Marine Environments



(a) Without paint-coating[2]
(b) With paint coating[1]

divided steel products, as introduced in the following chapter.

Influence of the Corrosion of Steel Products on Coating Degradation and the Progress Mechanism

Fig. 2 shows a conceptual drawing of microcell and macrocell corrosion.

As shown in the figure, in the microcell corrosion, both anodic and cathodic reactions uniformly occur: the anodic reactions in which iron dissolution occurs and the cathodic reactions in which oxygen and water are consumed to form hydroxide ions. As a means to assess microcell corrosion in such cases, the commonly adopted method relies on the polarization resistance obtained using AC impedance and other methods. In this regard, the commonly used method is that proposed by M. Stern and A.L. Geary[3] to convert from polarization resistance to microcell current density.

In macrocell corrosion, locally occurring anodic reactions cause corrosion. In order to assess such macrocell corrosion, it is necessary to measure the current that flows from cathode to anode. However, it is difficult to directly measure current flowing through a steel product. To solve this problem, we prepared the divided test specimens shown in Fig. 3, and proposed a method to directly assess the current that flows from cathode to anode, which was used to assess the macrocell corrosion.

Fig. 4 shows the time-dependent change in the macrocell current density of specimen (a) and in the microcell current density of specimen (b), both of which were painted with a phthalic acid-type coating (thickness: 150 μm) and provided with defects. It has been confirmed from the figure that, while the macrocell current density in the defect section was pronounced at the initial stage of exposure, the negative current density (cathode current density) in the coated section steadily increased; and as the exposure period increased, the macrocell current density in the defect section continued to rise. Further,

Fig. 2 Outline of Microcell and Macrocell Corrosion

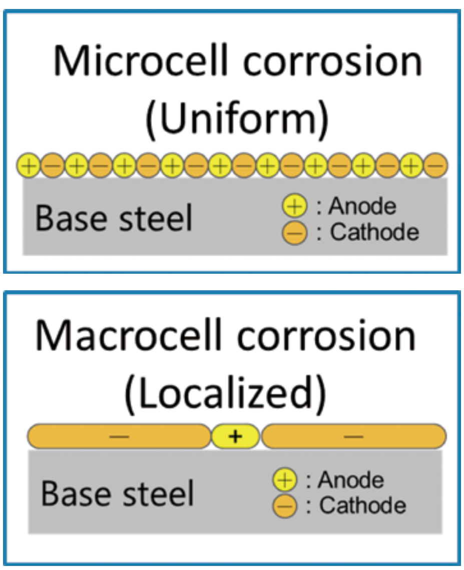
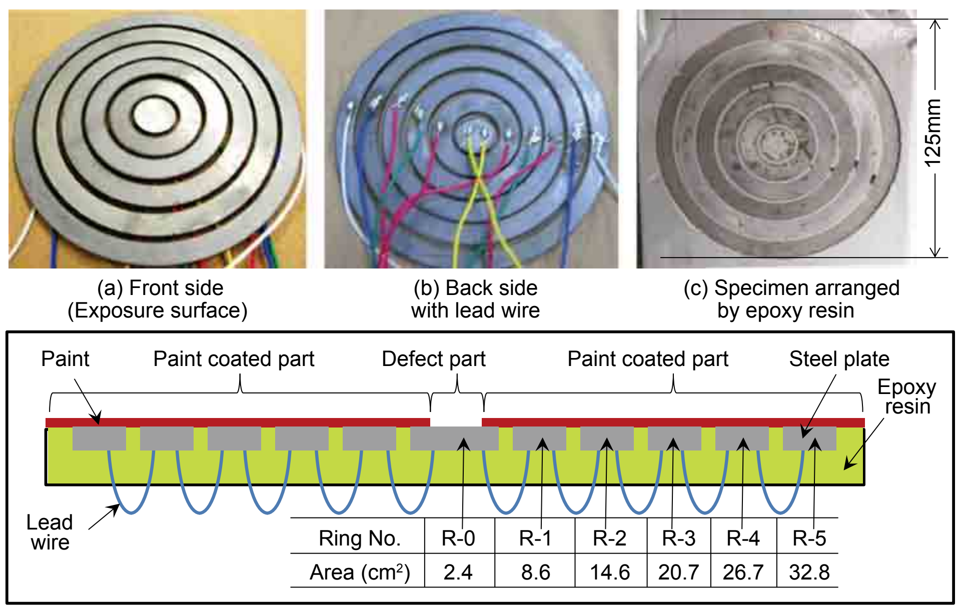
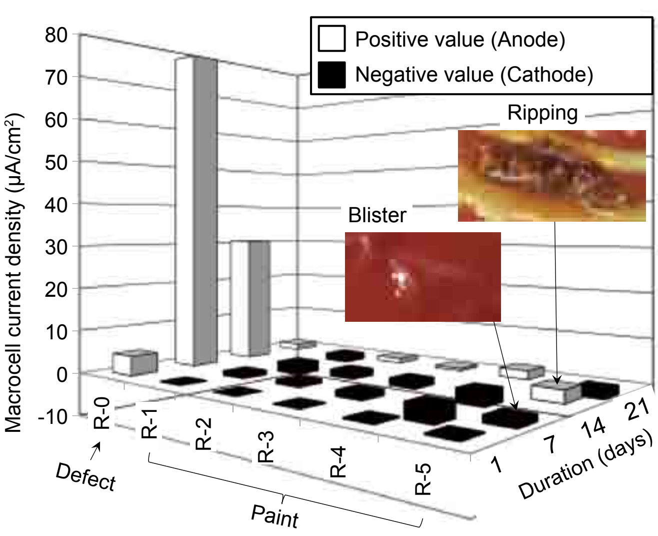


Fig. 3 Outline of Divided Steel Plate Specimens

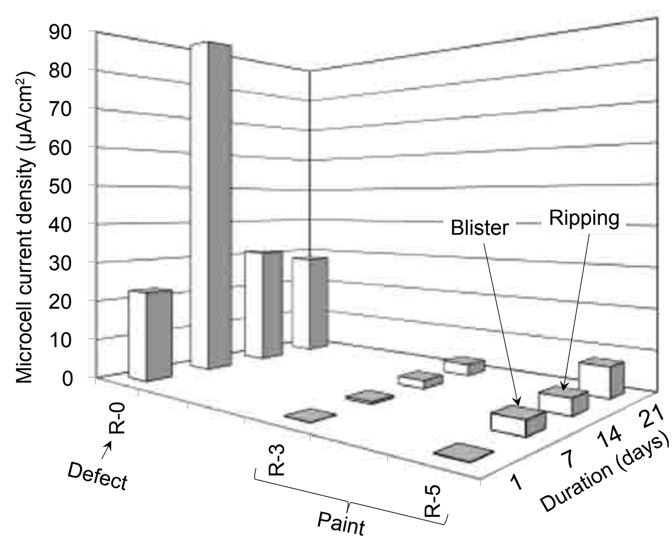


(d) Cross section of specimen

Fig. 4 Time-dependent Changes of Macrocell and Microcell Corrosion of Paint-coated Steel Specimens with Defects



(a) Macrocell



(b) Microcell

when the pH within the blisters was measured, it was confirmed that a high alkaline environment of pH 10~13 was generated, suggesting the possibility that OH is generated and accumulated under the painted-coated part due to cathodic reaction. Taking these results into account, it is believed that blistering, ripping and other types of coating degradation are strongly affected by the cathodic reaction of macrocell corrosion.

An examination of this behavior suggested that the deterioration of paint-coated steel products progresses as shown in Table 1. In particular, because the period when blistering occurs is largely related to further expansion of the area of corrosion, it is extremely important to specify the period when blistering occurs. Then, we tried to calculate the acceleration magnification from the acceleration test to the actual exposure environment by comparing the acceleration test results with actual exposure test results.

Verification of Validity in Converting from Acceleration Test Results to Actual Exposure Period

Table 2 summarizes the period up to the occurrence of blistering in the acceleration tests conducted on paint-coated steel products with defects (50°C, salt immersion) and in members exposed to the actual environment. As shown in the table, the values forecasted by the acceleration test results are consistent with the values obtained from the actual exposure test results. Therefore, our proposed approach as introduced above is considered valid for converting from acceleration test results to actual exposure test results.

Promotion of Research on the Corrosion Protection of Offshore Steel Structures

In this text we have introduced the results of our most current examination of steel product corrosion in a marine environment, specifically the influence of steel product defects and exposure conditions on coating degradation. But, as many of the other factors involved in corrosion remain unclear, the promotion of research into the corrosion of steel structures exposed to actual application environments is a pressing issue for engineers. We will be gratified if the results of our study and the proposal introduced prove helpful in structuring of future maintenance systems and the prolongation of social infrastructure service life. ■

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Table 1 Corrosion Progress around Defect of Paint-coated Steel

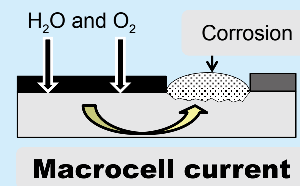
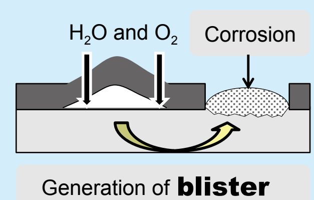
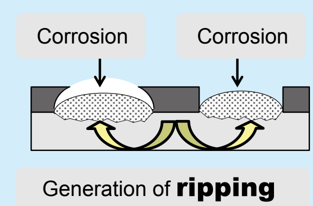
Steps	Corrosion progress	Schematic figure
Step 1: Incubation (Before blistering)	Before the generation of blister of paint, the macrocell corrosion between defect part and paint coated part occurs and hydroxide ion is accumulated between paint and steel due to cathodic reaction. In this term, the microcell and macrocell corrosion progress at defect part.	
Step 2: Propagation (Before ripping)	The blister of paint is generated by the osmotic pressure of water due to the high concentration of hydroxide ion and increase of diffusivity of paint. After that, the blister progresses to the ripping of paint and the paint coating is damaged.	
Step 3: Acceleration (After ripping)	The corrosion reaction around defect is accelerated by ripping of paint. Especially ripping part changes from cathode to anode of macrocell corrosion and more serious corrosion is generated.	

Table 2 Magnification of Each and Total Factors and Comparison between Estimated and Actual Values of Duration of Incubation Periods for Paint-coated Steel with Defect

Factors	Conditions for setting each magnification or duration	Laboratory	Futtsu (Outside exposure)		Miyako[1] (In-situ exposure)	
		Lab data	Site data	Magnification	Site data	Magnification
Temperature	$\frac{V_1}{V_2}=\exp\left(\frac{\Delta Ea}{R}\left(\frac{1}{T_2}-\frac{1}{T_1}\right)\right)$	50˚C (T_2)	21.8˚C (T_1)	×5.1	16.2˚C (T_1)	×7.4
Solution	3%NaCl: ×1.0; Seawater: ×2.3	NaCl3%	Seawater	×2.3	Seawater	×2.3
Paint thickness	$\frac{Y_1}{Y_2}=\frac{8.16\ln(X_1)-34.77}{8.16\ln(X_2)-34.77}$ Y:Duration;X:Thickness	150 μm (X_2)	150 μm (X_1)	×1.0	675 μm (X_1)	×5.3
Paint type	Phthalic: ×1.0; Tar epoxy: ×4.6	Phthalic	Phthalic	×1.0	Tar epoxy	×4.6
Dissolved oxygen	Y=0.28X + 0.24 Y:Duration; X: Dissolved oxygen	5.1 mg/L	5.0 mg/L	×1.0	7.0 mg/L	×0.7
Total magnification	Multiply all magnification	×1	×11.7		×290.5	
Estimated duration			23.4~58.3 days		1.6~4.0 years	
Actual duration	From observation	2~5 days	37 days		1~5 years	

Evaluation of the Remaining Load-carrying Capacity of Corrosion-damaged Steel Structures and the Prediction of Future Performance

by Katashi Fujii, Professor, Department of Civil and Environmental Engineering, Graduate School of Engineering, Hiroshima University

Because coastal and offshore steel structures in particular are exposed to severe corrosive environments, the loss of strength due to reduced wall thickness caused by corrosion poses problems in terms of structural safety. The following discusses how to secure the structural safety of corroded steel structures in marine environment through our research on an evaluation of the load-carrying capacity of corroded steel tubular piles and the prediction of their future performance.

Investigation of Corrosion Conditions

In order to accurately evaluate or predict the remaining load-carrying capacity of a structure, it is necessary to know its current corrosion conditions such as wall thickness and three-dimensional coordinates on the uneven surface caused by corrosion. In rating the remaining strength of a member, the statistical values of the corroded plate thicknesses, such as average thickness and standard deviation, are used as the representative indices, and as the measurement of the corrosion condition has become more minute and precise, these statistical values thus obtained become more accurate. For example, although the coordinates of uneven surface of the wall can be measured minutely and with high precision by means of three-dimensional laser measurements (Photo 1), the current practice frequently used to measure the wall thickness is the adoption of ultrasonic thickness gauges. However, the use of ultrasonic thickness gauges makes it difficult to perform multipoint measurements.

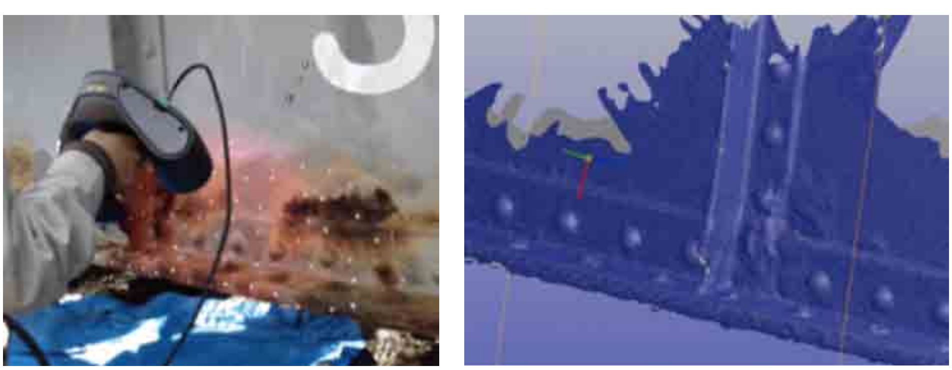
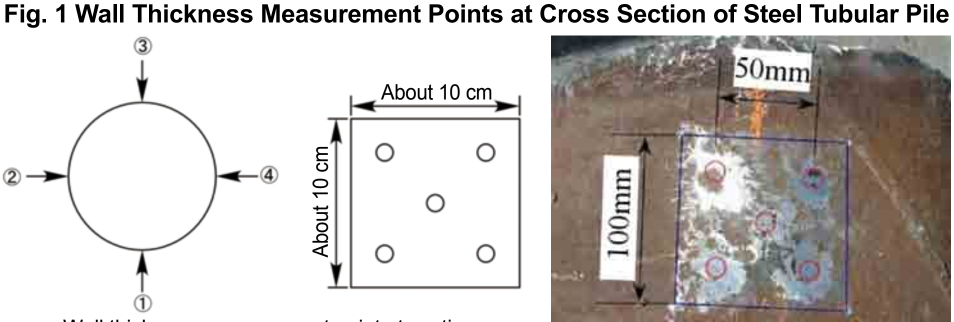


Photo 1 Surface coordinates obtained using a three-dimensional laser instrument
The photo on the left shows measuring conditions, and the photo on the right shows a three-dimensional image obtained from the measured results. (Courtesy: Keisoku Research Consultant Co.)

Fig. 1 Wall Thickness Measurement Points at Cross Section of Steel Tubular Pile



In the case of evaluating the average thickness at a cross section of steel tube, when individual measurements are made at a total of 20 points (4 sites × 5 points; see Fig. 1) per section, the average thickness t_r of the cross section can be rated by $t_r = t_{avg} - S$, which can be safe[1]. Here, t_{avg} and S are the average value and the standard deviation at each of the 20 points of wall thickness respectively.

Evaluation of Remaining Load-carrying Capacity

It can be said that the finite element method is the most effective and reliable means currently available for evaluating the remaining strength of corroded steel structures or members. Figs. 2 and 3 compare the analytical results and the experimental results pertaining to the remaining axial compressive strength of 6 steel tubular piles exposed to offshore conditions for 19 years. The analytical results were obtained using the elasto-plastic large deformation finite element analysis[2]. In this analysis, the thicknesses at the nodal points of each shell element were given by the da-

Fig. 2 Comparison of Remaining Axial Compressive Strength of Corroded Steel Tubular Piles between Finite Element Analysis and Experimental Results

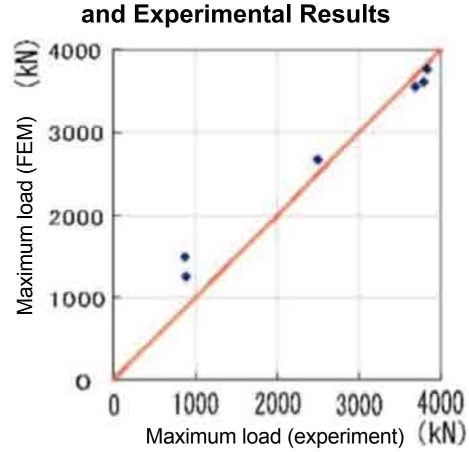
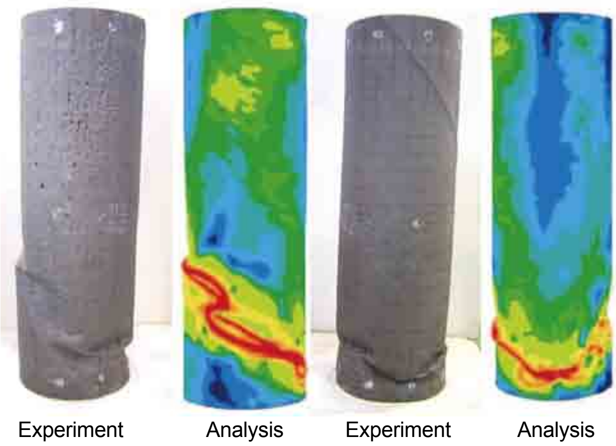


Fig. 3 Buckling Collapse Condition of Corroded Steel Tubular Piles (Splash zone)



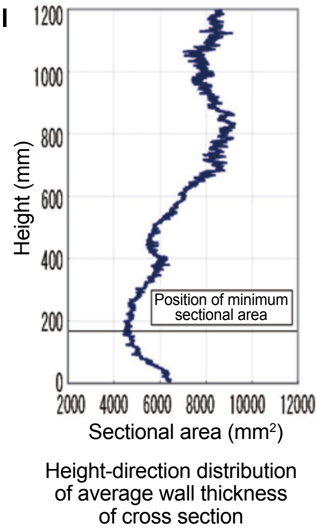


Fig. 4 Simple Prediction of Remaining Axial Compressive Strength Using Buckling Curve without Corrosion

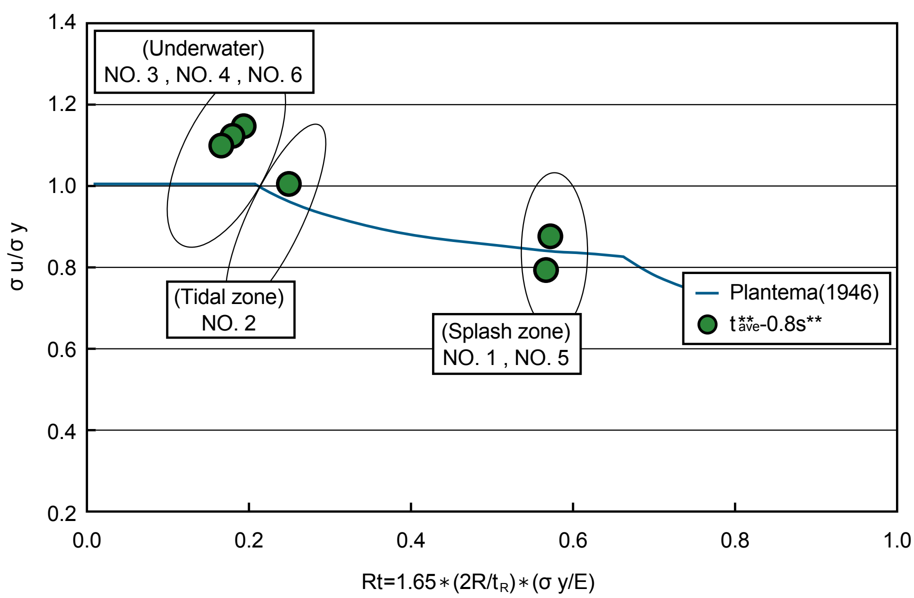
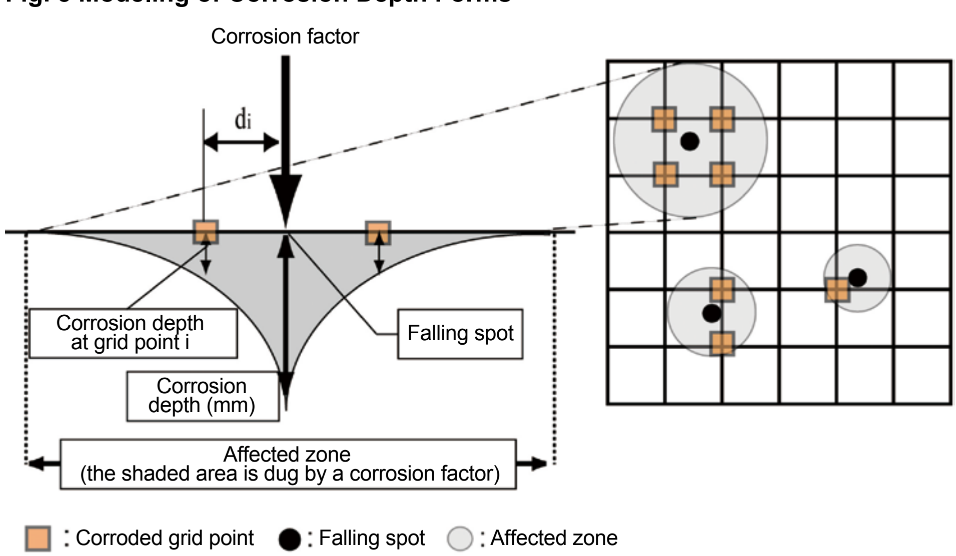


Fig. 5 Modeling of Corrosion Depth Forms



ta measured with 1-mm intervals. It can be seen from these figures that both the remaining strength and the collapse mode can be precisely evaluated in cases when the analysis is made using accurate corroded surface measurements.

Further, the remaining axial compressive strength can be evaluated easily by using the buckling strength curve in the case of no corrosion. In this case, the remaining strength can be obtained by substituting the statistical index (representative thickness) to the width-thickness parameter in the case of corrosion, in which the representative thickness can be obtained using the average thickness and the standard deviation obtained from measured thicknesses. Fig. 4 shows the remaining axial compressive strength of corroded tubular piles, obtained by means of above method, in which the representative thickness to be evaluated is set as (average value − 0.8 × standard deviation of measured thicknesses) and this is substituted to the width-to-thickness ratio parameter R_t, then the axial compressive stress σ_u can be obtained from the non-corroded buckling strength curve.

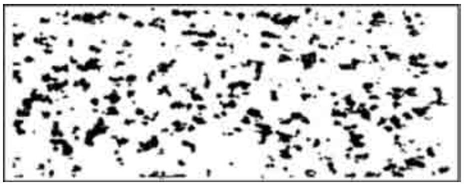
However, due attention should be paid to the following: The finite element analysis allows for the evaluation of remaining strength taking into account every buckling mode in the collapse, but when using a simple evaluation equation, the remaining strength only for the specified buckling mode can be evaluated. That is, in Fig. 4, only the local buckling strength of steel tubular piles is given, and the global buckling strength cannot be evaluated.

Prediction of Remaining Load-carrying Capacity

In order to realize minimum lifecycle costs for steel structures, it is necessary to predict secular changes in remaining load-carrying capacity and to draw up future maintenance plans. If simple models can be used to express secular changes in the surface unevenness shape of steel plate that have been caused by a deterioration in corrosion-protection function and subsequent corrosion, it will be possible to predict future declines in strength[3].

In the corroded surface generation model (Fig. 5), the surface of the steel plate is divided into a grid, and the corroded surface is expressed using the corrosion depth at each intersection in the grid. In this model, the following four factors are assumed: 1) two kinds of factors, deterioration factor and corrosion factor that have a certain strength and

Fig. 6 Examples of Analysis of Coating Deterioration Conditions (Dark areas: Exposed base metal)

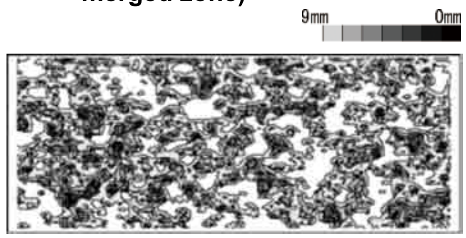


(a) Lapse of 50 years

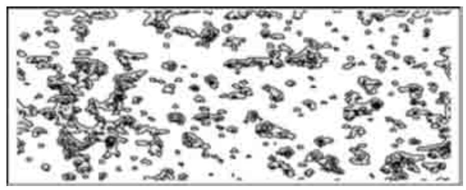
(b) Lapse of 100 years

a certain affected region respectively, will randomly fall on the surface; 2) the deterioration factor decreases the corrosion protection function; 3) when the corrosion protection function become lower than a threshold of corrosion prevention, the corrosion starts on the steel surface; and 4) consequently the surface is dug by corrosion factor, as shown in Fig. 5. The parameters for deterioration factor and corrosion factor (strength, affected radius, number of annual falls) can be decided from measurements of the actual corrosion conditions. It is now possible by use of this model to predict the corroded surface, and when the strength of the structure can be evaluated by means of finite element analysis considering the corroded surface shape thus predicted, it becomes possible to predict future remaining load-carrying capacity of the structure. Meanwhile, in cases when repainting is carried out, the values for corrosion-protection functions are to be renewed.

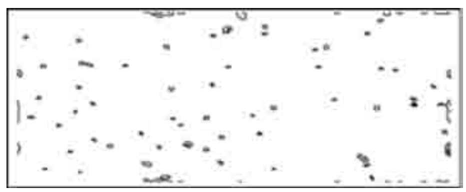
Fig. 7 Recoating Period and Simulation Results for Residual Wall Thickness after 100 Years (Submerged zone)



(a) Recoating every 50 years



(b) Recoating every 30 years

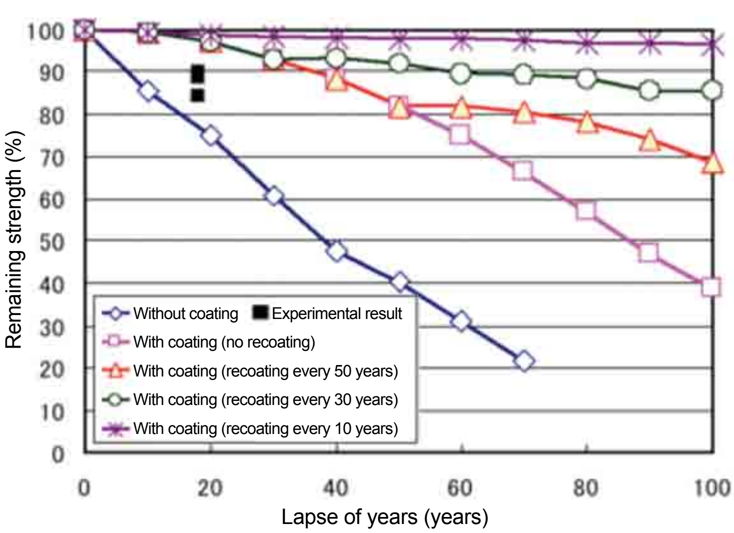


(c) Recoating every 10 years

Fig. 6 shows the surface condition in which certain years passed since the initial-stage coating and the base metal was exposed, and Fig. 7 shows the contour lines depicting remaining wall thickness obtained from simulations. Fig. 8 shows the results of axial compressive strength analysis conducted on steel tubular piles using the results of corroded surface prediction. As shown in Fig. 8, it is now possible to provide convincing prediction of remaining load-carrying capacity of steel structures by adopting the approach described above.

At the time of initial design of structure, the reliability of these predictions may be lowest because the predictions of corrosion progression are based entirely on assumed parameters. However, in cases when parameters of the model will be revised in accordance with the actual corrosion conditions measured at each stage of periodic inspection, more accurate and reliable updated strength of the structure will consequently be able to be predicted. We consider that, in cases when a maintenance plan is worked out based on the predictive approach described above, it will finally be possible to reduce the lifecycle costs of steel structures to a minimum.

Fig. 8 Predicted Results for Strength Loss Affected by Recoating Period (Submerged zone)



Acknowledgement

The current research was conducted with a specified research subsidy granted by the Japan Iron and Steel Federation and as a link in the activities of the Task Committee on Evaluation of Corrosion Protection and Durability for Steel and Composite Structures under Offshore Environments (chaired by E. Watanabe)[4], the Committee on Steel Structures of the Japan Society of Civil Engineers. We wish to thank the committee members and related organizations for their generous support and cooperation. ■

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Lifecycle Performance Assessment Technology for Port Steel Structures with a Focus on Post-repair Performance

by Yoshito Itoh, Professor, Yasuo Kitane, Associate Professor, and Mikihito Hirohata, Assistant Professor; Civil Engineering, Graduate School of Engineering, Nagoya University

Prolongation of the Service Life of Aging Port/Harbor Steel Structures

In order to prolong the service life of aging port/harbor steel structures by means of rationalized maintenance, it is important to precisely gauge performance before and after repair, to assess lifecycle performance during the designed service life, and to provide appropriate inspection and maintenance. Targeting steel pipe piles with deteriorating performance due to corrosion, we have studied the post-repair load carrying capacity and durability of these piles repaired by steel plate welding. An outline of our study follows.

Load Carrying Capacity of Steel Pipe Piles after Steel Plate Welding Repair

Fig. 1 shows the steel plate welding repair method[1]—a typical method used to repair and strengthen corroded steel pipe and sheet piles. In this method, the corrosion-damaged section is covered with steel patch plate, which is welded to the existing steel member using fillet welding. In cases when the corroded section is located in seawater, underwater wet welding is frequently adopted. Welding environments, dry or wet, can affect characteristic weld properties. It is reported that many weld defects occur during underwater welding and that whereas the hardness of welds in underwater welding increases when compared to that of open-air welding, ductility of underwater welds decreases[2]. Similar results were obtained from strength tests of fillet welds using steel pipe and steel sheet piles as the base metal[3]. As shown in Fig. 2, underwater fillet weld joints have larger static strength but much smaller ductility than open-air welds, and it was learned that these effects are more significant in sheet pile steel than in pipe pile steel.

Fig. 1 Repaired Steel Pipe Pile

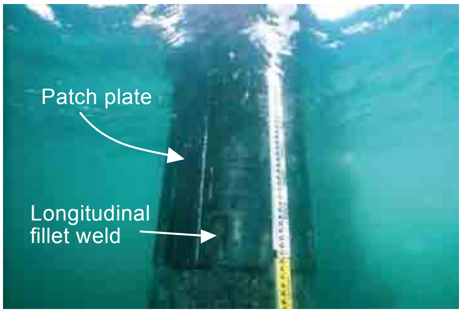
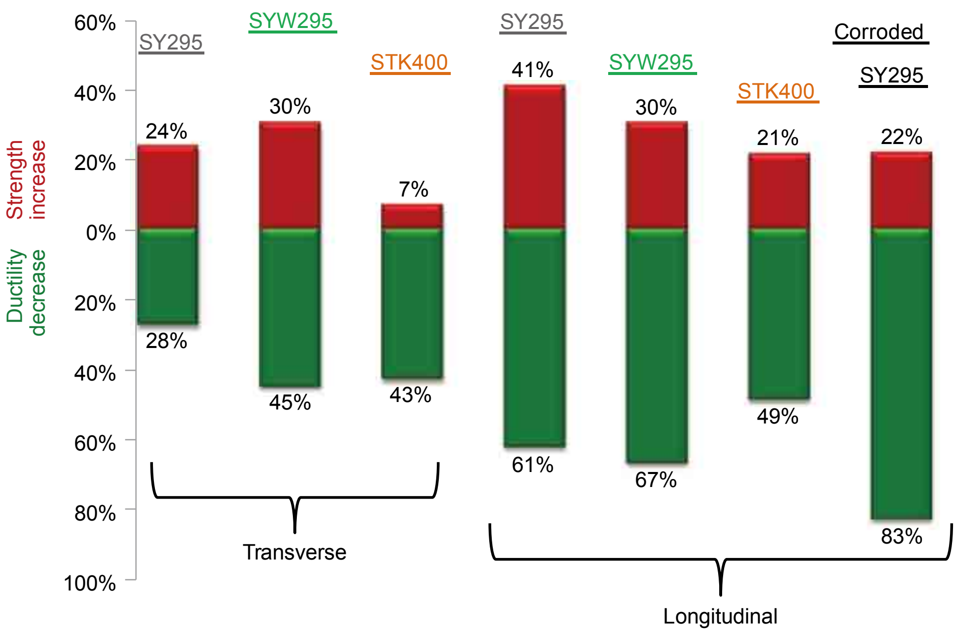


Fig. 2 Relative Changes of Strength and Ductility from Open-air Welds to Underwater Welds



Further, tests were conducted whereby steel plate welding repairs based on the current design method were applied to steel pipes whose wall thicknesses had been artificially reduced by cutting and whereby post-repair structural performance of the pipe was then examined by applying compression or flexural loading[4]. These tests clearly showed that, while stiffness and load carrying capacity of the repaired pipe against compression or flexural loading were recovered to nearly the same levels as before corrosion damage occurred, the corresponding ductility decreased from pre-corrosion levels. Fig. 3 shows the load-displacement curve obtained from the flexural loading tests.

As discussed above, the static strength of steel pipe after repair can be recovered to pre-corrosion levels, but the repaired steel pipe will then be subjected to cyclic loading during earthquakes, and thus an important question arises as to whether or not the repaired steel pipe will be able to show appropriate energy absorbing capacity.

Fig. 4 shows the load-displacement curve of four kinds of steel pipes that were subjected to cyclic flexural loading in the nonlinear finite element analysis: (a) intact pipe (216.3 mm in outside diameter, 12.7 mm in wall thickness); (b) steel pipe having a wall thickness that was uniformly reduced by 6 mm in a 150 mm-long section; (c) steel pipe having a wall thickness that was uniformly reduced

Fig. 3 Load-Displacement Relationships of Steel Pipes under Flexural Loading

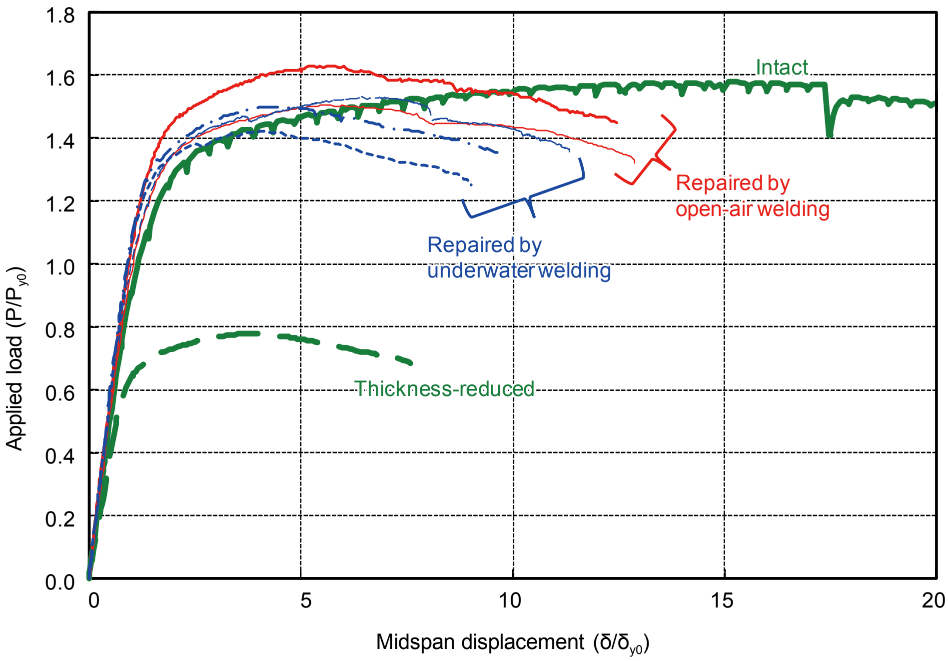
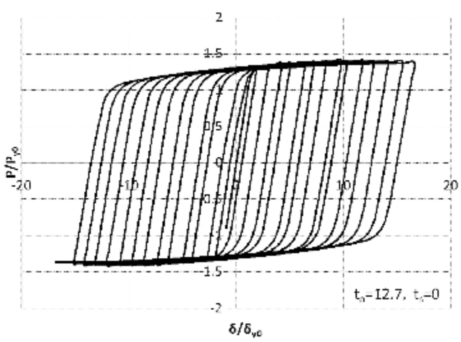
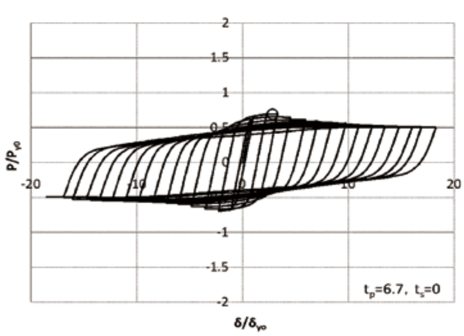


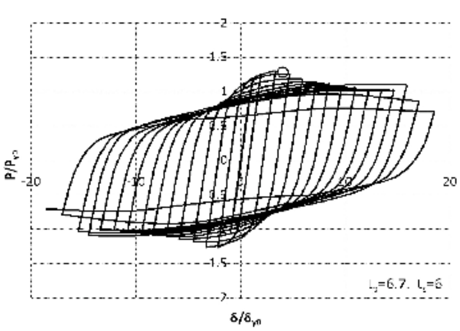
Fig. 4 Load-Displacement Curves of Steel Pipes under Cyclic Flexural Loading



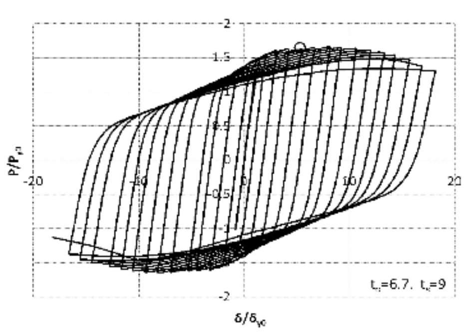
(a) Intact pipe



(b) Thickness-reduced pipe



(c) Repaired pipe with a 6 mm thick patch plate



(d) Repaired pipe with a 9 mm thick patch plate

by 6 mm and was then weld-repaired using 6 mm-thick patch plate; and (d) steel pipe having a wall thickness that was uniformly reduced by 6 mm and was weld-repaired using 9 mm-thick patch plate.

It is found from the figure that, in order to recover the energy-absorption capacity of corrosion damaged steel pipe to that of intact pipe, it is necessary to employ steel patch plate with a thickness greater than the reduction in wall thickness caused by corrosion[5].

Photo 1 Steel pipe pile repaired by patch plates with petrolatum coating and FRP cover

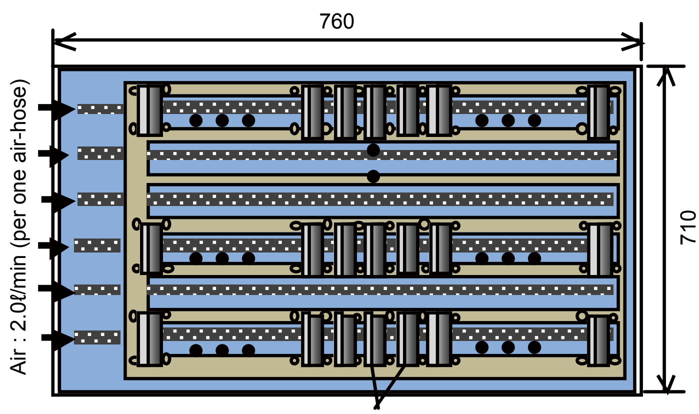
Examination of the Corrosion Characteristics of Welds Using a Seawater Bubbling Test

In order to show lifecycle performance, it is necessary to understand post-repair corrosion resistance. In the current practice, corrosion-protection coatings are applied to steel pipe piles that have been repaired by means of steel patch plate welding. Photo 1 shows an example of such steel pipe piles to which petrolatum coating was applied and an FRP cover over the coating had been attached. It is accepted that such a corrosion protection is effective for about 20 years; however, in cases when an FRP cover has been damaged by floating objects and a part of petrolatum coating has been gouged away, the repaired section is once again exposed to the corrosive environment. While many research results are available that pertain to the corrosion characteristics of steel members in seawater, less research has been conducted that compares general sections of steel members with welded joints in order to demonstrate differences in corrosion characteristics.

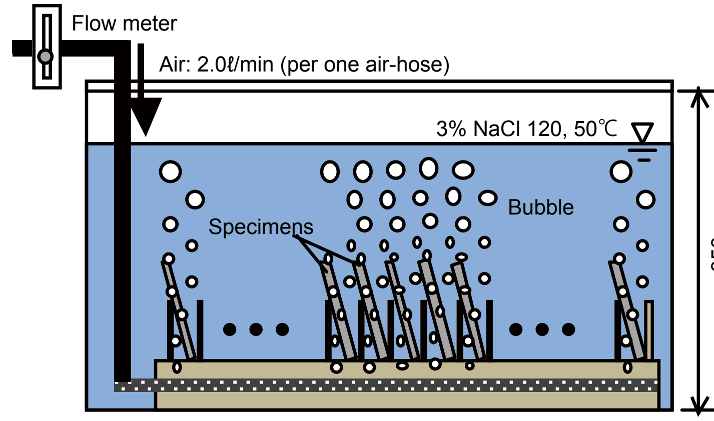
Given this lack of data, we used a bubbling corrosion acceleration test (3% NaCl solution, 50°C, 28 days)[6] to examine the corrosion characteristics of steel weld joints. Fig. 5 shows an outline of the testing apparatus used in the test. Two kinds of base plate were used for the test specimens, SY295 and SYW295, while SM490 steel plate was used for the patch plates. The patch plates and base plates were weld-joined by means of fillet welding using E4319 welding electrodes.

In the test, pre-test and post-test surface profiles were measured and compared using a laser displacement sensor to determine corrosion loss in the welds. Fig. 6 shows the corrosion loss in the weld of one specimen. As seen in the figure, the fillet weld joint shows uniform corrosion, and lacks any locally severe corrosion generated by uneven sections in the weld bead surface or by welding toes.

Fig. 5 Accelerated Exposure Test System for Underwater Corrosion

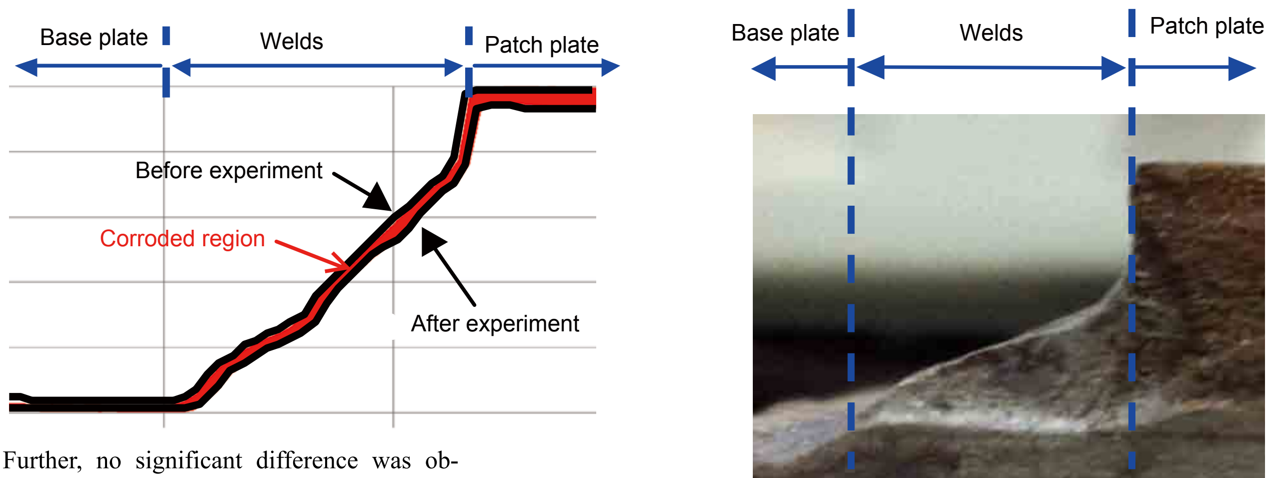


(a) Plan view



(b) Side view

Fig. 6 Change of Surface Profile of Weld due to Corrosion



Further, no significant difference was observed in the corrosion loss attributable to the direction of the welding (longitudinal or transverse fillet welding), the steel type of the base plates or the welding environment (open-air or underwater). Furthermore, it is found that corrosion loss in the weld joints is similar to that in the steel plate.

Lifecycle Performance Assessment of Port Steel Structures

With the goal of improving the lifecycle management of port/harbor steel structures, we examined the load carrying capacity of corrosion damaged steel pipe piles that had been repaired by means of steel plate welding in order to better understand the lifecycle performance of steel pipe piles. Our future goal is to assess the post-repair load carrying capacity of entire pier/quay structures in addition to single steel pipe piles. ■

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Assessment Method to Determine the Remaining Strength of Corrosion-deteriorated Port Steel Structures and Its Necessary On-site Measurement of Corrosion Profile

by Kunitomo Sugiura, Professor, Department of Civil and Earth Resources Engineering, Graduate School of Engineering, Kyoto University

Toward Longer Service Life of Infrastructure

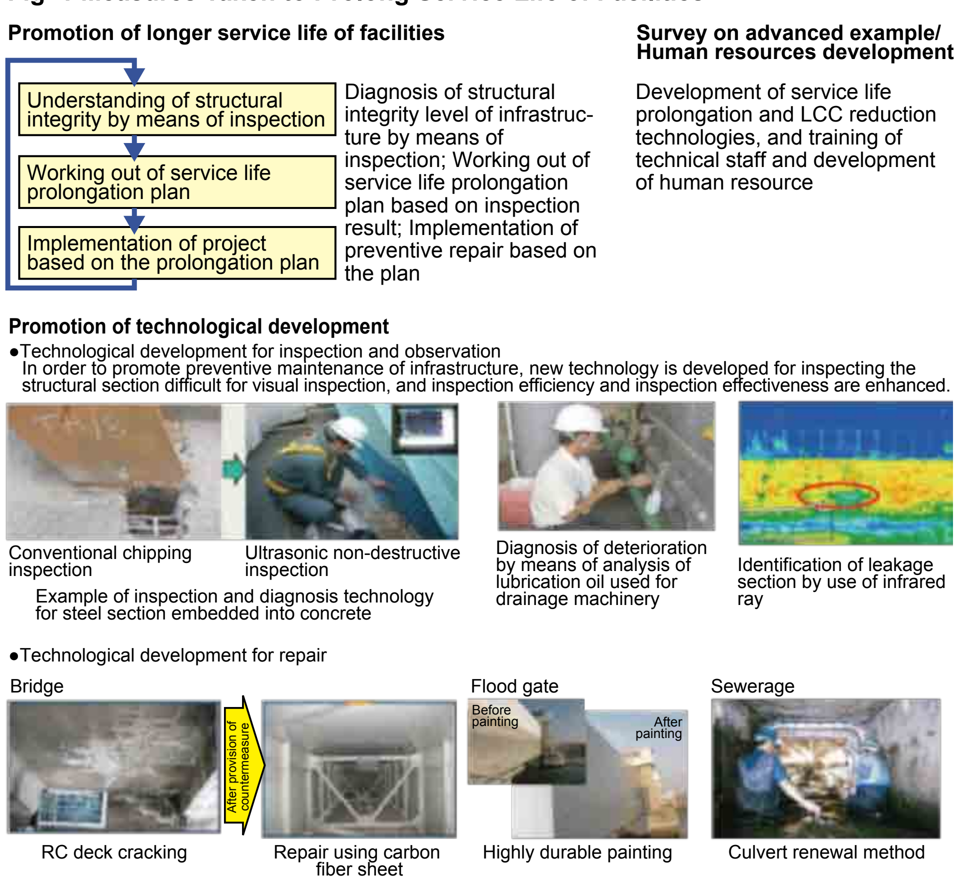
Surrounded by seas in every direction, Japan relies on port and harbor facilities to play a key role in promoting smooth economic and social development. As logistics facilities, these ports and harbors generally consist of wharves and other mooring facilities, of waterways and basins, such as fairways and anchorages, and of breakwaters and other protective facilities for securing safe harbor areas. While they have performed admirably through the individual and collective execution of their various functions, 30~50 years have passed since they were constructed. Given this situation, it is certain that maintenance costs will increase in the future, and it is therefore important to appropriately maintain these port and harbor facilities.

In Japan, the nation's social infrastructure, such as highways and river and harbor facilities, were intensively constructed during a period of high economic growth. In light of this, the Ministry of Infrastructure, Land, Transport and Tourism (MILT) has drawn up an estimate of required future maintenance and renewal costs based on past investment returns for infrastructure under MILT control (highways, ports/harbors, airports, public rental housing, sewage, urban parks, river control facilities, coastlines and coastal facilities). Trial calculations show that costs for the required maintenance/renewal will surpass the allocated funding in 2037, and that, of the approximately ¥190 trillion required for renewal over the 50 year period from FY2011 to 2060, about ¥30 trillion worth of renewals (16% of the total expense) will be impossible to implement[1]. This fact suggests that it is necessary to implement planned and strategic maintenance with the least possible financial burden and that, accordingly, it will be necessary to immediately establish lifecycle design/management technology that will effectively utilize existing facilities and prolong the service life by means of proper repairs.

To this end, it is important not only to design, construct and manage new facilities that are easy to inspect and observe, but also to implement "preventive maintenance" that will prolong the service life of entire facilities by means of early damage detection and minor repair of existing facilities. It will also be necessary to utilize materials and structures that offer high durability, to work out and implement longer service life measures and to prolong the service life of entire infrastructure systems. The final goal is to reduce the total cost. (Refer to Fig. 1)

In Japan, the application of steel sheet piles in the construction of wharves and other port facilities started in 1926 and that of steel

Fig. 1 Measures Taken to Prolong Service Life of Facilities



Source: MILT

Fig. 2 Example of Straight Pile-type Shore Bridge

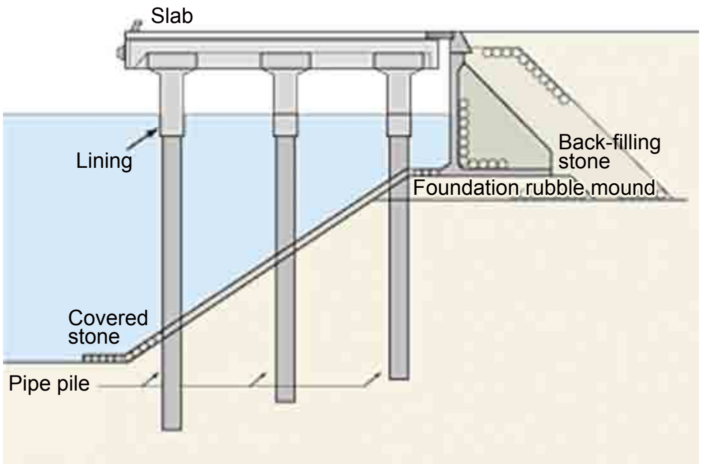


Fig. 4 Composition of Elements of Plane Frame Structure of Pier

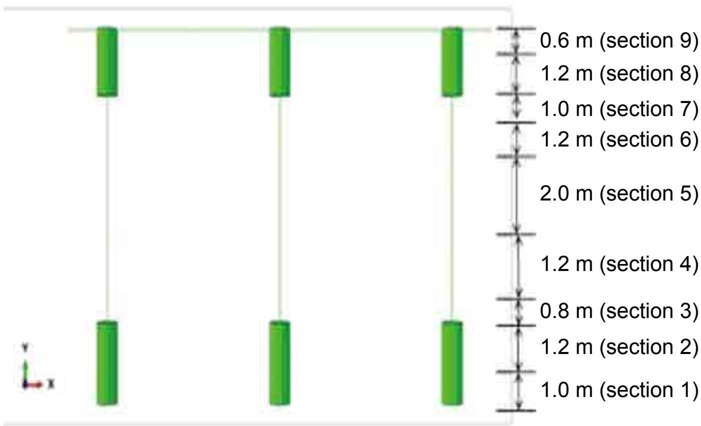
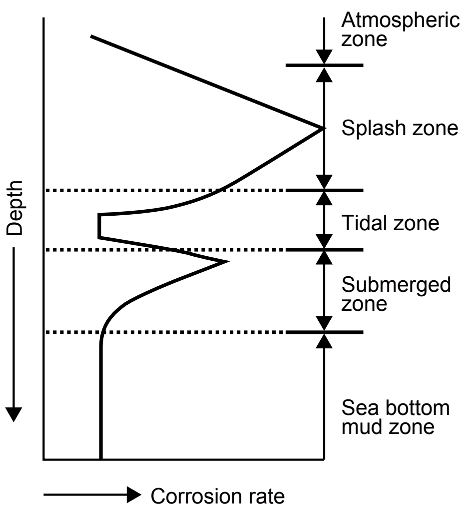


Fig. 3 Trend in Vertical-direction Corrosion of Offshore Structure[2)]



pipe piles in the latter part of the 1950s. Because of their applicability in rapid shoreline construction, these steel products have seen extensive use. Steel port facilities come in contact with seawater and are exposed to severe environmental conditions such as deviations in water level and direct tidal splashing, thereby showing corrosion behaviors that are different from those of land-based steel structures. Along with the increase in applications of steel products in port and harbor facilities, studies have been made to clarify corrosion mechanisms, develop diverse corrosion-protection methods and improve repair and strengthening methods. Prior to the establishment of corrosion-protection technologies, it was most common to build unprotected structures that in place of corrosion protection were designed with corrosion allowances that increased wall thicknesses rather than suppress corrosion. Today, newly constructed steel structures increasingly employ corrosion-protection systems such as cathodic protection or surface coating/painting. While these newly-constructed structures have a specified period of durability during which their corrosion-protection functions should remain viable, it is still necessary to implement periodic inspections and constantly monitor remaining performance so that the structure's required performance is not impaired by the corrosion of its steel members.

Meanwhile, it is well known that corrosion reduces the plate thickness of steel members applied in port steel structures and causes a decline in the load-carrying capacity of not only each structural member but of the entire structure as well. Accordingly, it is extremely important in terms of maintenance to appropriately evaluate the remaining structural performance of corrosion-deteriorated steel structures. In evaluating the remaining load-carrying capacity of port steel structures, various parameters have been proposed by means of analytical and experimental approaches to evaluate the load-carrying capacity of steel members subjected to various sectional forces[2)]. However, most of these experimental and analytical approaches target only some of the structural members, and thus evaluations are not very frequently made of an entire structure.

Then, in the following, an examination was conducted targeting a pier composed of steel pipe piles and RC slabs (a model having three piles for a plane frame, Fig. 2). The results of this assessment summarize the effect of corrosion-induced performance deterioration (Fig. 3) on the horizontal load-carrying capacity of the piles. In the assessment, the steel pipe specimens were extracted from corroded piles exposed to a marine environment for about 19 years. The corrosion profiles of the specimens were measured using a laser displacement transducer, and the corrosion profiles thus obtained were used as the typical corrosion characteristics of the steel pipe piles in the marine environment[3)4)].

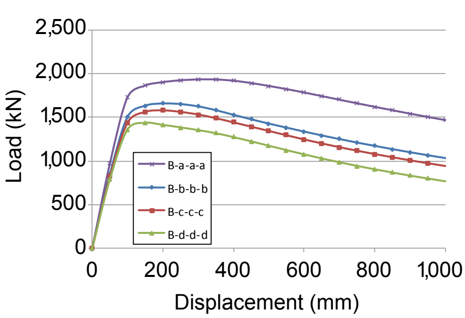
FEM Analysis of a Straight Pile-type Pier with Pipe Piles Subjected to Corrosion-induced Performance Deterioration

A plane frame structure (height: 10.2 m; pile intervals: 6.5 m), in which 3 pipe piles (outside diameter: 800 mm; wall thickness: 16 mm) are arranged parallel to each other at identical intervals, was adopted as a model of an entire pier having steel pipe piles whose performance had deteriorated due to corrosion, and the effect of the plane frame with corroded steel pipe piles on the horizontal load-carrying capacity was analyzed by changing the corrosion deterioration patterns. Meanwhile, analysis was made using the finite element analysis code ABAQUS (Ver. 6.10)[5)]. Fig. 4 shows the plane frame model that was structured as a pier model having corroded pipe piles and used in the FEM analysis. The pipe pile bases having a length 2.75 times the outside diameter of pipe and the pipe pile tops having a length 2.25 times the outside diameter were modeled using shell elements (4 node reduced integration shell elements), and the intermediate sections were modeled using beam elements. Meanwhile, the shell elements and the beam elements were to be rigidly joined at the nodes, the number of the shell elements in the steel pipe circumferential direction was set at 150, and meshing in the axial direction of the members was conducted using similar dimensions. The RC slab was to be elastic, and to be rigidly joined with the shell area of the pipe pile tops. The base edges of the pipe piles were completely fixed as the boundary conditions, and a maximum horizontal displacement of 1,000 mm was assigned to the entire slab in the in-plane horizontal direction. Steel grade SKK490 was adopted for the steel pipe; the wall thickness was input at every node in the shell elements; and in the beam elements, the cross-sectional properties used for the hollow circular cross-section with equal diameters were input for every element employing the average thickness.

Three cases were analyzed: Model A with one pile; Frame Model B-1 with a corrosion pattern in which three piles show same corrosion; and Frame Model B-2 in which only one pile shows large corrosion loss. As regards Model A, it was prepared using a pile similar to that used in the plane model, in which the top edge of the pile was set as the rigid surface, a rigid element was attached to the top edge, rotation restriction was applied to the rigid element, and then analysis was made until the maximum horizontal displacement reached 1,000 mm. In the analysis, the level of corrosion loss in each model was changed in increments of 0.0 (no corrosion), 1.0, 1.2, 1.4, 1.6 and 1.84 times the standard corrosion loss obtained in actual measurements of wall thickness; these corrosion levels were classified as a, b, c, d, e and f; and the analytical model cases are marked using B-[○]-[△]-[□] (○: left pile; △: center pile; □: right pile, and corrosion patterns for these three piles are expressed using the symbols a~f). By the way, because the corrosion conditions differ according to the circumferential direction of the pile, the horizontal strength of Model A was confirmed by changing the horizontal loading direction by a 45° pitch, and as a result the horizontal load-carrying capacity averaged 548 kN (559 kN max., 537 kN min.), and the deviation range was within about 4% of the average value.

Fig. 5 shows an example of the horizontal loading-horizontal displacement relation (B-1 Model). The load-carrying capacity was reduced by about 14.2% from the intact level (B-a-a-a) due to the progression of corrosion (B-b-b-b) caused by 19 years of exposure. Further, the load-carrying capacity fell by 18.3% and 25.7% respectively from intact level, which corresponded to increases in corrosion loss of 1.2 times (B-c-c-c) and 1.4 times (B-d-d-d). Also, displacement at the time of maximum loading fell below intact level as the corrosion loss increased. In addition, along with corrosion progress, various changes were observed in the axial force bearing of each pile, and significant differences in local buckling behavior and load-carrying capacity were found, from which it is known that the structural performance of entire structures must be evaluated.

Fig. 5 Example of Load-Displacement Relation



Meanwhile, as a result of selecting corrosion-damaged pipe piles configured as Model B-2, we see that even in cases where one of the piles shows remarkable corrosion, loading can be borne by the remaining piles, thereby demonstrating that the load-carrying capacity of an entire structure depends on the total corrosion loss of the entire structure.

Performance Evaluation of Corrosion-deteriorated Piers Based on On-site Measurements

The representative wall thickness of the corroded steel pipe piles used in the pier models targeted by the current study was determined assuming that on-site measurements of wall thickness were based on the currently-prevailing "Corrosion Prevention of Port and Harbor Steel Structures: Improvement Manual[6]." In the procedure for determining thickness, the cross-section having the least thickness was selected from among pile section in the splash, tidal and submerged zones, and 4- or 8-direction measurement center points were selected within the same cross-section with the point of minimum thickness as the starting point (Fig. 6). Further, within a 10-cm square patch around selected center point, 5 points including 4 points each separated by about 3.2 cm in longitudinal and circumferential directions from the starting point were extracted, and the average thickness thus obtained was set as the on-site measured thickness of piles for evaluation (Fig. 7).

The analysis targeted 1 pile (outside diameter: 406.4 mm; length: 10.5 m; original thickness: 9 mm; steel grade SKK490). Of the analytical models, Ave-[ ] in Fig. 8 denotes the model that adopts the average thickness in each section shown in the figure; 4d denotes the model for which 4 points of thickness measurement were selected in the circumferential direction at the time of selecting the center measurement point with the

Fig. 6 Example of Measurement Position in Height and Circumferential Direction[6]

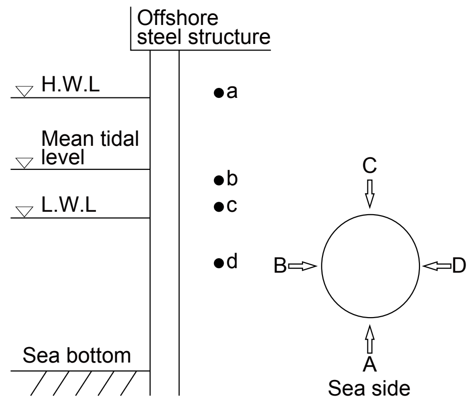
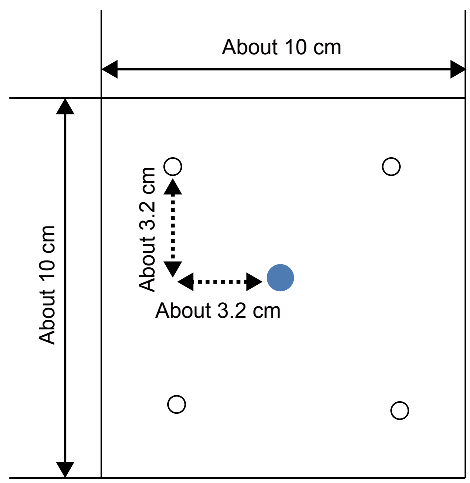
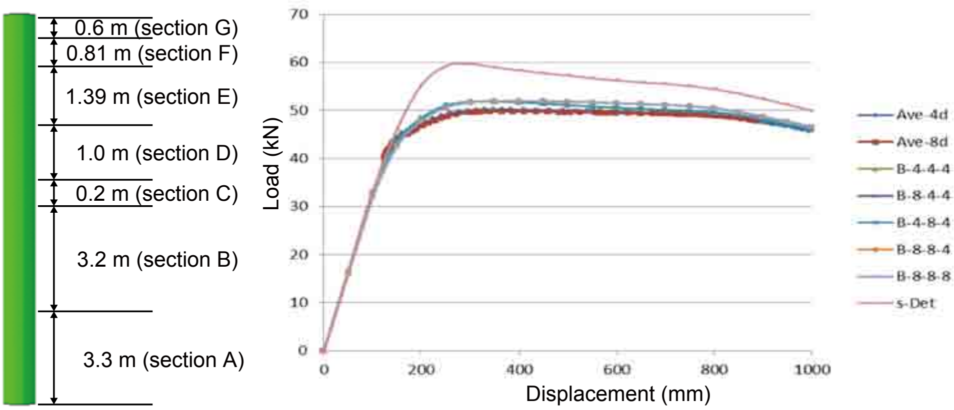


Fig. 7 Wall Thickness Measurement Points within 10 cm-square Area[6]



minimum thickness as the starting point; and 8d shows the model in which 8 points were selected. Further, B-[ ]-[ ]-[ ] (for example B-4-4-4 in the figure) denotes the model for which an average thickness was not adopted for each separated section but, rather, the section was separated in the longitudinal direction depending on the thickness measurement direction. The wall thickness calculated as the measurement value in each separated section was input in the analysis. [ ] shows the number of thickness measurement points in circumferential direction in the splash, tidal and submerged zones (from left to right). Meanwhile, s-Det shows the model in which the corrosion profile is reproduced in detail; and, while average thickness, linear interpolation and other assumptions are included in the section for which thickness measurement was not made, this analysis case is assumed to offer the greatest detail regarding thickness measurements and the highest accuracy with regard to the horizontal load-carrying

Fig. 8 Load-Displacement Relation of Pipe Pile Model Based on Virtual On-site Measurement



capacity of the pipe piles.

Fig. 8 shows the load-displacement relation of each analytical model. The margin of error in terms of horizontal load-carrying capacity obtained in comparison with s-Det amounted to 16.8% for Ave-4d and 13.1% for B-4-4-4. While 4 points per cross-section are specified for on-site measurement in the currently prevailing manual, it can be seen from the above that the load-carrying capacity of the model is given a comparatively low (safe-side) evaluation compared to the actual load-carrying capacity. This is because the thickness of the sectional area having the minimum thickness was input as the wall thickness of the piles in the entire splash zone, and thus the thickness applied in all zones was less than actual thickness. In order to precisely evaluate the load-carrying capacity, it is considered important to reproduce the local strength of the splash zone where it is strongly subjected to the effect of bending moment and in which the thickness variation is large.

Further, when examining the results in which the number of measured cross-sections in the splash and tidal zones is increased to 2 or 3, and when the analytical results are compared with those of s-Det, a load-displacement curve closer to that in s-Det can be obtained by increasing the number of measured cross-sections. This is considered to be so because the average thickness per cross-section can be calculated with a greater level of accuracy by increasing the number of measured sectional areas. Towards this end, while on-site measurement involves some difficulty, it is desirable to measure the thickness using multiple cross-sections for each zone where corrosion characteristics are judged to be similar, and, also, it is considered sufficient in measuring thicknesses to adopt 4 points set at 90° intervals within a cross-section.

Ideal Approach to On-site Measurements

As a result of plane frame analysis of a straight steel pipe pile pier, it was learned that the effect of the location of severely corrosion-damaged pipe piles on the load-carrying capacity is less, and that the load-carrying capacity of the entire structure depends on the corrosion loss of the entire structure. Also, as a result of evaluating the remaining load-carrying capacity by means of on-site pipe pile wall thickness measurements based on the currently-prevailing manual, it was learned that, while a margin of error of 13% or more is found in comparison with the load-carrying capacity in the case when corrosion is modeled in detail, a safe-side load-carrying capacity can be found.

Further, while thickness measured at 4 points within one cross-section is sufficient, particularly in the splash zone where the remaining wall thickness deviates greatly, it is more effective to modify the measured thickness using the standard deviation of measured thickness, and it is desirable to measure the thickness in multiple cross-sections. Detailed studies are being promoted to find the ideal approach to on-site measurements by which the remaining load-carrying capacity of pier structures can be precisely evaluated in terms of engineering that employs limited on-site measurement results ■

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