MAINTENANCE MANUAL FOR PORT AND HARBOR FACILITIES
(13/June/2025)

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1. INTRODUCTION

In order to achieve stable economic development, it is very important to properly manage port facilities, which are the core of logistics, to maintain their sound condition and use them effectively for as long as possible. Compared to general land facilities, port facilities are exposed to harsh natural conditions and usage conditions, so their performance tends to deteriorate over the years of use after completion due to deterioration of materials, damage to components, subsidence of foundations, scouring and sedimentation in the surrounding area, etc. Therefore, it is extremely important to take appropriate considerations into account when designing and constructing port facilities so that they continue to meet the required performance during their service life, and to continue to maintain them in a planned and appropriate manner even after they begin service.

In Vietnam, TCVN13330 MARINE PORT FACILITIES: MAINTENANCE REQUIREMENT was issued in 2021 as a standard for the maintenance of port facilities with the aim of performing maintenance of port facilities appropriately and strategically. This TCVN13330 shows the principles for the maintenance of mooring facilities, outer facilities, and water facilities, and shows the basic ideas for a wide range of facility inspection and diagnosis techniques and countermeasure techniques based on the idea of life cycle management. However, when actually carrying out maintenance, practitioners need various information on specific investigation methods and investigation techniques when inspecting and diagnosing actual facilities. Furthermore, when countermeasures are required for confirmed abnormalities, specific methods and information on repair and reinforcement are essential.

This manual was compiled with the aim of providing such specific and standard information. In other words, this manual, together with the existing TCVN13330, plays the role of "two wheels of a car" in efficiently promoting the maintenance of port facilities.

Chapter 2 describes the investigation and inspection methods for port facilities. The targets are mainly water facilities such as sea routes and anchorages, concrete structures, and underground cavities, but pages are also devoted to steel structures, which are expected to become more necessary in Vietnam in the future.

Chapter 3 describes the methods to be used when the results of investigations and inspections indicate that certain measures are necessary, for each type of water facility, concrete structure, and steel structure.

Chapter 4 describes how to record, store, and utilize data that is essential for the life cycle management of port facilities, based on the results of maintenance.

Regarding the maintenance of port and harbour facilities, in addition to this manual, the following documents can be used as references.

・Maintenance Technical Manual for Port and Harbor Facilities, CDIT, 2018(revied)

・Technical Standards and Commentaries for Port and Harbour Facilities in Japan (2025 revied), OCDI (https://ocdi.or.jp/public/)

・Guidelines for Maintenance and Repair of Port and Harbor Facilities, CDIT, 2023 (https://www.cdit.or.jp/english/news/20230525.html)

(*CDIT: Coastal Development Institute of Technology)

(*OCDI: The Overseas Coastal Area Development Institute of Japan)

2. Inspection Technology for Port and Harbor Structures

2.1 General

Inspection is important activity in the maintenance and management of port and harbor facilities, and it is necessary to apply appropriate inspection techniques to understand the current state of the facilities. Currently, in addition to conventional techniques, new technologies using **ICT** technology are being utilized.

Inspection methods are divided into visual inspection and inspection using instruments. Furthermore, the location of the inspection is divided into above sea level (including land) and below sea level (underwater) (Table 2.1.1).

Table 2.1.1 Characteristics of visual and instrumental surveys

Method		Characteristics
Visual inspection	Above sea level	・Easy and low cost ・Depends on the researcher's subjectivity ・Limited locations and time available for survey
	Below sea level (Underwater Survey)	・Depends on the diver's subjectivity ・Results depend on environmental conditions
Inspection using instruments	Above sea level	・Obtaining objective and accurate data ・Costly to implement。 ・There are restrictions on methods, locations, time, etc.
	Below sea level	

2.2 Investigation of External Appearance

2.2.1 Visual Investigation

Visual investigation **is** a basic survey to understand the overall condition of a facility or structure. Visual surveys can roughly grasp abnormalities that have occurred on the surface of the structure, as well as deformation of the entire structure.
For structures where there is concern that deterioration is progressing, continuous visual surveys can also be performed to grasp the progress of the deterioration. If any particular abnormalities are found during a visual survey, a detailed survey and diagnosis using equipment etc. must be carried out.
Visual surveys can be divided into surveys above sea level (including land) and surveys below sea level, depending on the location of the survey target.

2.2.1.1 Visual Surveys Above Sea Level

Visual surveys of land areas above sea level should be conducted primarily through **close-up visual inspection**. Visual surveys should include photographs (or videos) and may also involve sketches. Displacement can occur when a structure, or its constituent parts, move from their designated positions. When visually inspecting the displacement of port structures, it is advisable to use simple measuring instruments such as tape measures, staffs, and inclinometers, or surveying instruments such as levels and transits. When inspecting the length and width of cracks in concrete structures, it is also advisable to use tape measures and crack scales (Photo 2.2.1).
An image of a visual inspection of a quay apron is shown in Fig.2.2.1.

Photo2.2.1 Example of Visual Survey using Simple Equipment

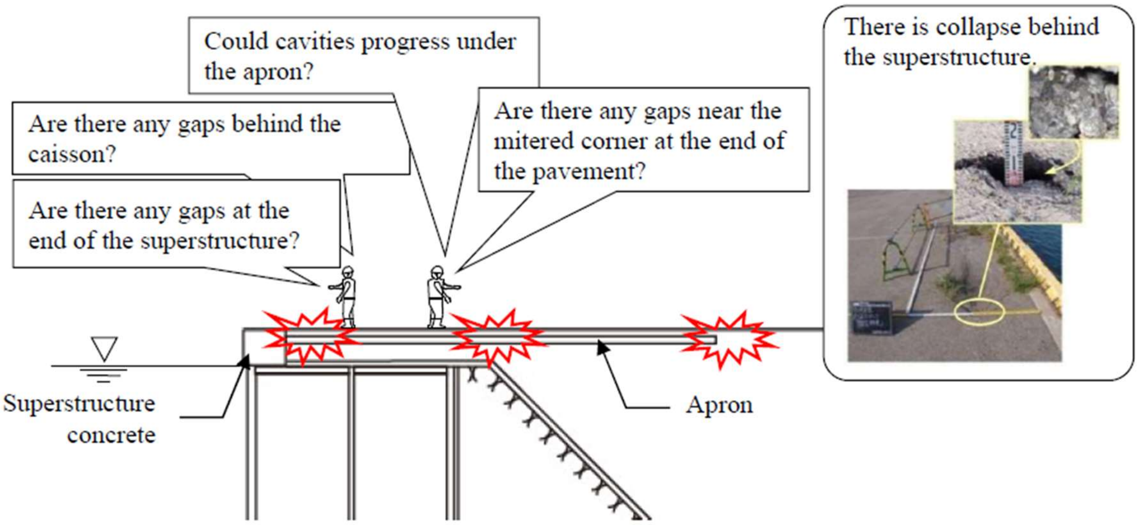


Fig.2.2.1 An image of a visual inspection of a quay apron

Surveys above sea level (relatively close to sea level) are generally conducted mainly from a **boat.** Depending on the part and location of the structure being surveyed, access is often poor and visual surveys are difficult. Or there may be cases where deformation or displacement needs to be quantitatively investigated. In such cases, it is **better** to use equipment as necessary to complement the survey. Examples include binoculars, scales, hammers, and crack scales.

When surveying tidal zones from a boat, it is effective to conduct the survey at a time when the area is more exposed to the air, such as at low tide during spring tides.

When carrying out a visual survey, it is advisable to get as close as possible to the target object, for example by using a small boat. For concrete structures, it is a good idea to also use a hammer to carry out a tapping inspection. Fig.2.2.2 and Photo2.2.1 show the image of a visual inspection of a wharf superstructure using a boat.

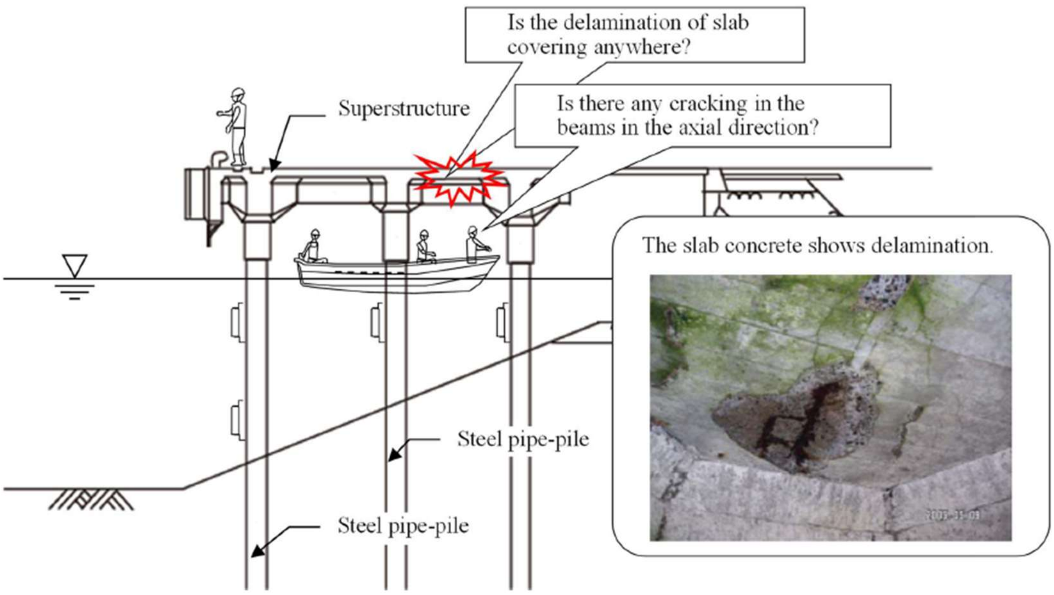


Fig.2.2.2 An image of a visual inspection of a wharf superstructure (bottom surface)

Photo 2.2.1 Visual inspection of a wharf superstructure

2.2.1.2 Visual Survey Below Sea Level

Visual surveys below the sea surface (underwater) are often conducted by divers, who generally take photographs of the target area with a camera or other device. Prior to the survey, it is necessary to confirm with the diver the target items, survey methods, and survey key points.

Depending on the location and conditions of the port, the water may be murky in the underwater area, making it difficult to see anything but the immediate vicinity, which can make the survey inefficient as it takes a long time. It is also important to note that for safety reasons, the time that divers can work is limited by the depth. Furthermore, it is necessary to ensure the safety of divers even when surveying in places with fast currents or narrow spaces. Photo 2.2.2 and Fig.2.2.3 show images of underwater sections inspected by divers.

Photo2.2.2 Visual Inspection of the Underwater of Steel Sheet Piles

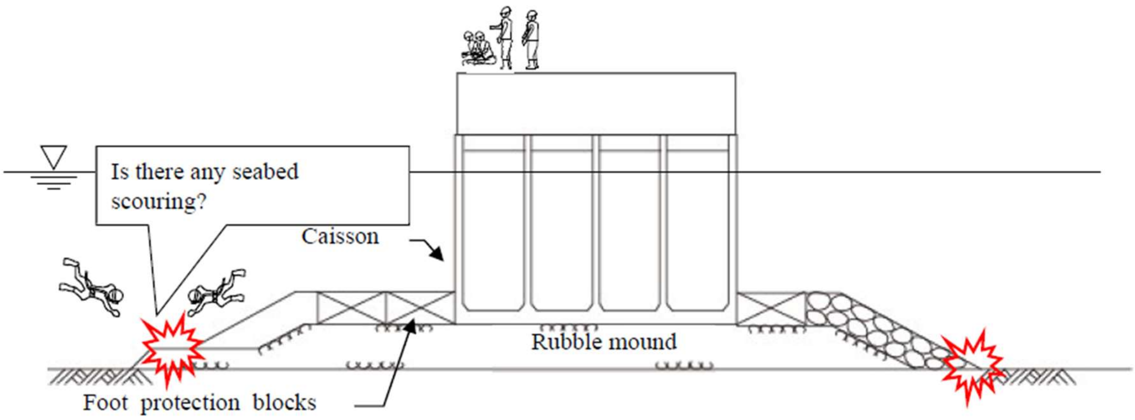


Fig.2.2.3 An image of a visual inspection of seabed (by divers)

2.2.2 Investigation using equipment

Optical equipment is a good choice when detecting changes in the overall shape of a facility, or when the area being surveyed is far from land or in a narrow space that prevents engineers from directly conducting a visual survey. In such cases, it is a good idea to use optical equipment. For example, by flying a UAV (Unmanned Aerial Vehicle), it is possible to take pictures of a wide area of the facility from a high place in a short time. It is also possible to take pictures of the condition of offshore facilities far from land using a UAV without using a boat.

Underwater image capture is also done by divers, but recently, photography has also been done using equipment such as underwater drones. These devices have the advantage of being able to take pictures safely even in places that are dangerous for divers to go.

Photo 2.2.3 Example of Underwater Drone

There are many models of underwater drones on the market, from hobby to commercial use, and most models are equipped with underwater cameras, so video can be easily taken. The images sent from the camera attached to the drone body can be viewed on the control panel (monitor), making it easy to operate the drone and camera. The clarity of the image is greatly affected not only by the performance of the camera but also by the turbidity of the water. In addition, in places affected by waves and tides, the drone will be carried away and attitude control will be difficult, making it difficult to take pictures in a nearly stationary state. Therefore, when using an underwater drone for a wide-area survey, it is necessary to consider the turbidity of the seawater, tides, and waves.

A simple and relatively inexpensive way to capture images is with a hanging camera. This method is particularly suitable for investigating areas that are inaccessible to the naked eye, but not accessible to divers or underwater drones. Figure 2.2.4 shows an image of the lower portion of a superstructure of open-type wharf surveyed using a hanging camera (rod attached). Figure 2.2.5 also shows how to use an underwater drone and a hanging camera.

Furthermore, the characteristics of underwater drones and hanging cameras are shown in Table 2.2.1.

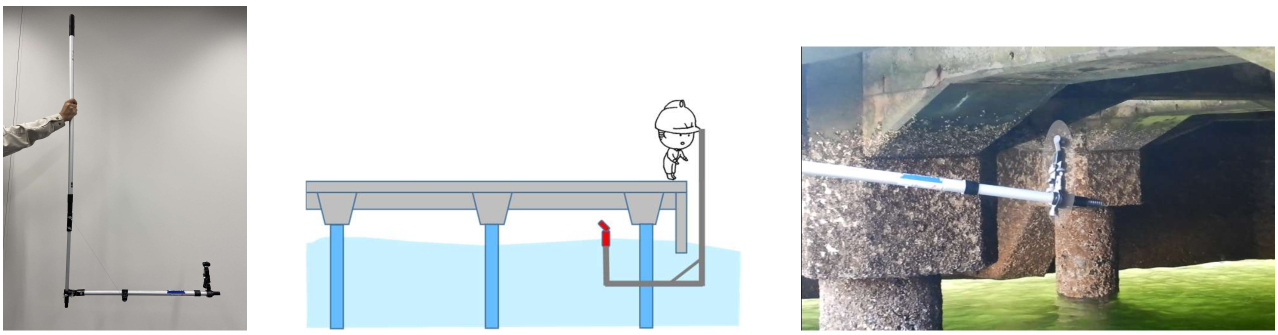


Fig.2.2.4 Examples of how to use a hanging camera

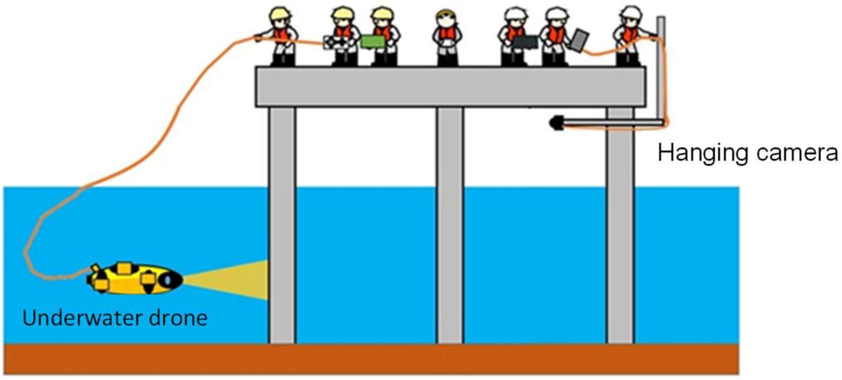


Fig.2.2.5 Image of a Pier Photographed by an Underwater Drone and a Hanging Camera

Table 2.2.1 Characteristics of Underwater Drones and Hanging Cameras

Technique		Overview	Scope of application	Merits	Demerits
Optical Instruments	Underwater drone	・ Taking images by operating an underwater drone from land	・ Applicable in environments with little effect from turbidity, waves, and tides	・ Easier to prepare and operate compared to audio equipment ・Shoots faster than a diver	・ Care must be taken to identify the location of the photo ・ Heavily affected by turbidity and currents
	hanging camera	・ Shooting with an underwater camera suspended from land	・Effective for use near water (in air and underwater)	・Very easy to shoot from land ・Shooting location is clear ・Inexpensive	・Not suitable for large areas or deep waters ・Easily affected by tides
Reference	Diving work	・ Visual observation by divers (Camera photography enable)	・Applicable to depths of 0 to 30m (Use proper equipment for safety)	・ Visual results can be adopted as the outcome. ・ Detailed measurements possible	・ Survey accuracy varies depending on the diver

The hanging camera itself is suitable for use in a small space, and it can take still or video images. It can be used in the air as well as underwater, so it can be used for a wide range of purposes. In order to take pictures of a target underwater without the camera being swayed by the current, it is difficult to simply hang the camera from a rope, and some ingenuity is required, such as attaching the hanging camera to the end of a rod to control the camera's attitude. In addition, like underwater drones, it is best to display the image on a monitor at hand to check the subject while taking pictures, but to do this, the camera and monitor must be connected by wire or wirelessly (Wi-Fi, etc.). However, wireless connections are not possible underwater, so special ingenuity is required.

Photo2.2.4 shows an example of a hanging camera (attached to a rod) in which a camera housed in an underwater photography housing is connected to a tablet device at hand via Wi-Fi, allowing you to take pictures while checking the camera image at hand even underwater.

Underwater drones and underwater cameras are relatively inexpensive to obtain and easy to operate, making them suitable for observing areas that are difficult for inspectors to access or observe on a daily basis. They can also be effectively used as a screening method to narrow down inspection locations and items prior to conducting a detailed investigation.

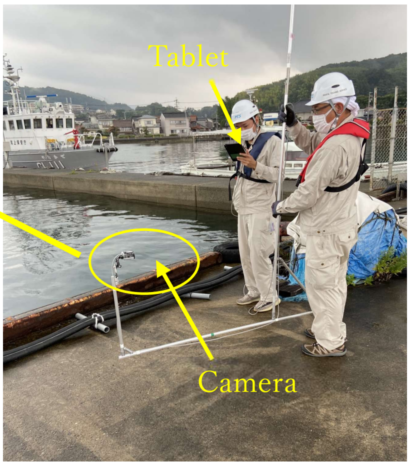


Photo 2.2.4 Example of a hanging camera capable of underwater photography

The underside of a pier has narrow clearance and is affected by waves and tides, so it is often difficult for people to reach the survey location. In addition, in the case of a structure with advanced deterioration, there is a risk that the surveyor will be injured if the deteriorated part falls, so in such cases it is effective to take images using a pier superstructure inspection ROV or a small radio-controlled boat etc..

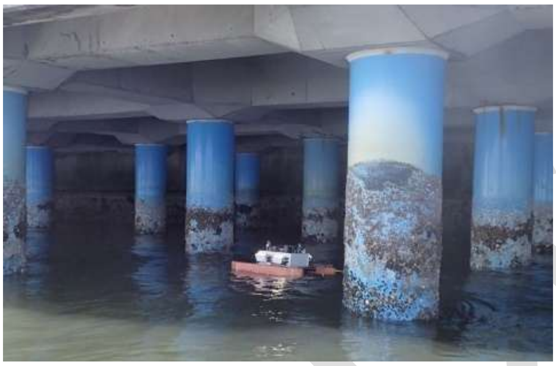


Photo 2.2.5 Survey of the underside of the pier using an ROV and a small radio boat

2.3 Investigation of Shape and deformation

2.3.1 Investigation above sea level

Surveying is a method to quantitatively measure the movement and displacement of port structures (Photo 2.3.1). Surveying methods include total station and level surveying, UAV (Unmanned Aerial Vehicle) surveying, and GNSS (Global Navigation Satellite System) surveying.

Photo 2.3.1 Status of surveying

Recently, aerial surveys are frequently conducted by UAV. UAV can be used to take pictures of structures from various altitudes and angles (Photo 2.3.2). It is also possible to create a 3D model from multiple 2D data using image processing software. The 3D model created by this process can be used for deterioration diagnosis, as it is easier to understand the status of deterioration damage. Fig. 2.3.1 shows an example of a 3D model created using multiple still images taken by UAV (Examples of software for creating 3D models from photographs are; http://www.kobeseiko.co.jp/uav_soft.html, https://www.bentley.com/).

Photo 2.3.2 UAV Survey Status

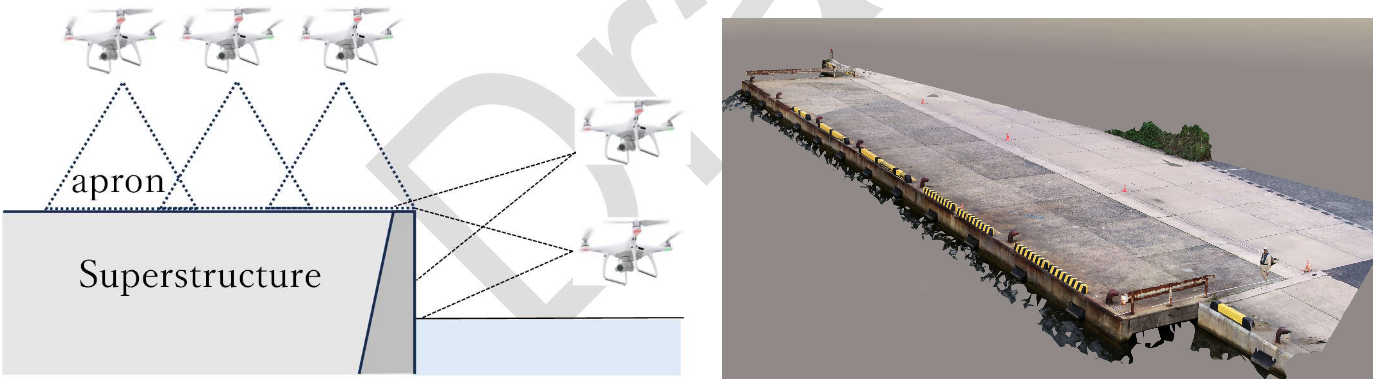


Fig. 2.3.1 Example of 3D Model of Wharf Created from UAV Photos

UAVs can be used to observe the entire facility in real time from the sky. The 3D point cloud data is a set of points in 3D coordinates, each point being represented by 3D Cartesian coordinates (x, y, z). Each point is an independent point, and there is no continuous relationship between adjacent points. 3D point cloud data on land can be obtained, for example, by 3D surveying using a laser scanner. Fig.2.3.2 shows an example of a 3D point cloud data representation of a wharf using a laser scanner mounted on a UAV. Furthermore, UAV flying must comply with each country's laws and regulations.

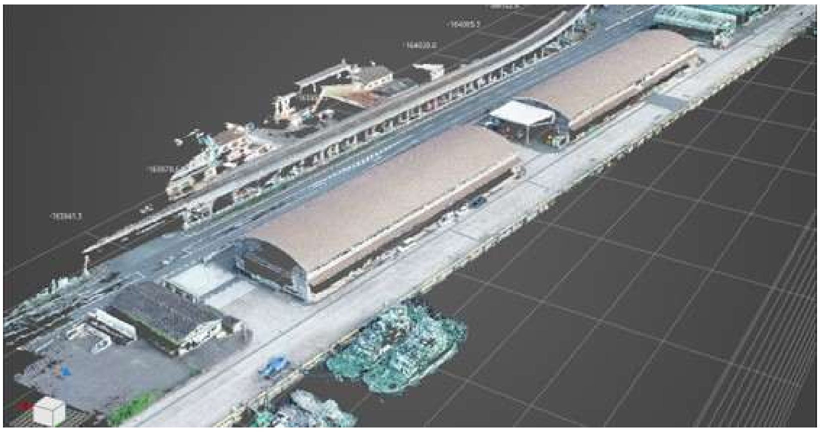


Fig. 2.3.2　Example of Wharf Modeling Using 3D Point Cloud Data

2.3.2 Investigation below sea level

2.3.2.1 Survey of water depth

When measuring water depth (bathymetry) of waterways, anchorages, ship basins, etc., most of the work is carried out at sea, so appropriate inspection and survey methods are selected taking into consideration the required bathymetry accuracy after considering the planning conditions of the facility (area, facility shape, etc.) and natural conditions water depth, tidal currents, seabed soil condition etc..

The following methods are mainly used for bathymetry. A fish finder can also be used as a simple echo sounder, and many inexpensive models, including handheld types, are commercially available, and can be used depending on the purpose.

1) Bathymetry Tape Method
2) Echo sounder method

2.3.2.2 Survey of seabed topography

Divers are often used to survey the shape of the seabed or the underwater parts of structures, but recently acoustic devices have been widely used to quickly survey the shape of the seabed over a wide area. There are also many technologies that treat objects as a collection of points, i.e., point clouds. Acoustic devices such as narrow multi-beam sonars and 3D scanners are often used to measure the shape of underwater parts (such as the shape of the seabed) and obtain 3D point cloud data. The characteristics of acoustic devices are shown in Table 2.3.1.

Table 2.3.1 Characteristics of the acoustic devices used for underwater shape survey

Technique		Overview	Scope of application	Merits	Demerits
Acoustic device	Narrow multi-beam・Underwater 3D Scanner	・Acquisition of structural geometry and topography as 3D point cloud data	・Capable of measuring from the water surface to a depth of 400m (Depends on machine performance)	・No influence of turbidity or dark areas in the water due to measurement by sound waves	・Cracks and defects in concrete, rusting of steel, and discoloration are difficult to detect.
Reference	Diving work	・Visual observation by divers (Camera photography is also possible)	・Depth of water about 0 to 30m (Requires appropriate equipment)	・Visual results can be adopted as they are ・Detailed measurements possible	・Survey accuracy varies depending on the diver

There are two methods for 3D scanners: one is to place the scanner body on the seabed and take measurements, and the other is to attach the scanner to a boat and take measurements while moving (Photo 2.3.3). Fig.2.3.3 shows an example of 3D point cloud data obtained near a sheet pile quaywall using a 3D scanner. The data acquired by a 3D scanner can contain three-dimensional coordinate values, so it can be color-coded according to depth, as shown in Figure 2.5, making it easier to understand the situation underwater.

【Outfitting of ships for measurement】 【Measurements taken on the seabed】

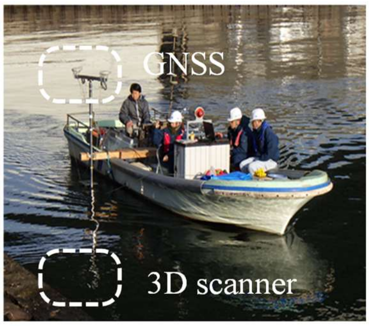


Photo 2.3.3 Measurement situation with 3D scanner

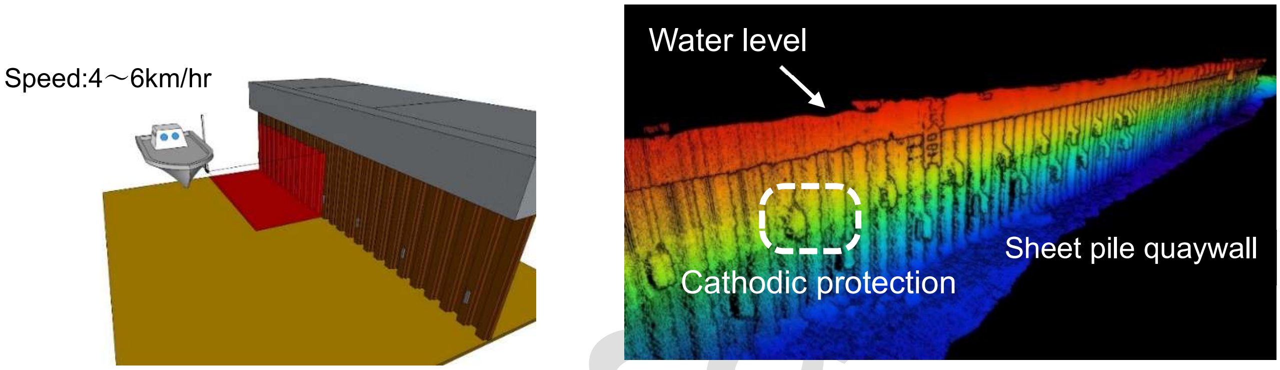
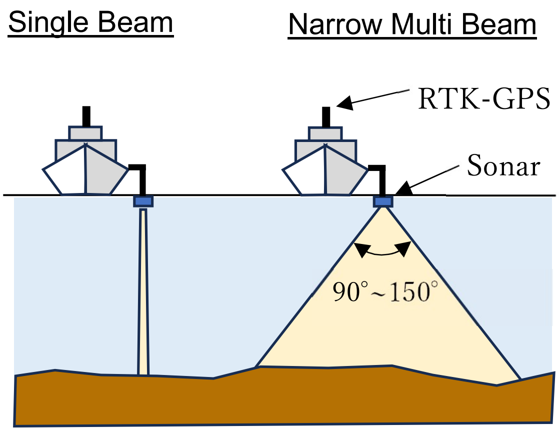


Fig.2.3.3 Example of 3D point cloud data acquisition of sheet pile quay wall by 3D sonar

There are two types of sonar methods: single beam sonar and narrow multi-beam sonar (Fig. 2.3.4). Narrow multi-beam sonar transmits and receives highly directional ultrasonic waves in a fan-shaped pattern, making it possible to survey an area of the seabed that is 2 to 7 times the water depth in one survey line at a time, making surveying work much more efficient than conventional methods (Fig.2.3.5). In terms of measurement range, the narrow multi-beam method is generally more suitable for measuring a wider area than the 3D scanner method.



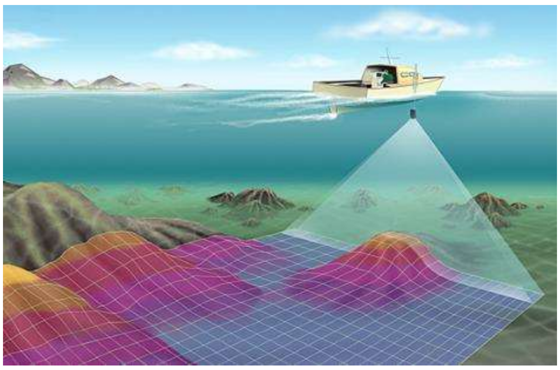


Fig.2.3.4 Two types of sonar methods

Fig.2.3.5 Image of depth survey with narrow multi-beam method

One method for measuring the seabed shape over a wide area is to use an autonomous AUV (Autonomous Underwater Vehicle), and technology has been developed that allows the AUV to perform measurements by automatically navigating along a navigation route pre-stored in the AUV itself (Fig. 2.3.6).

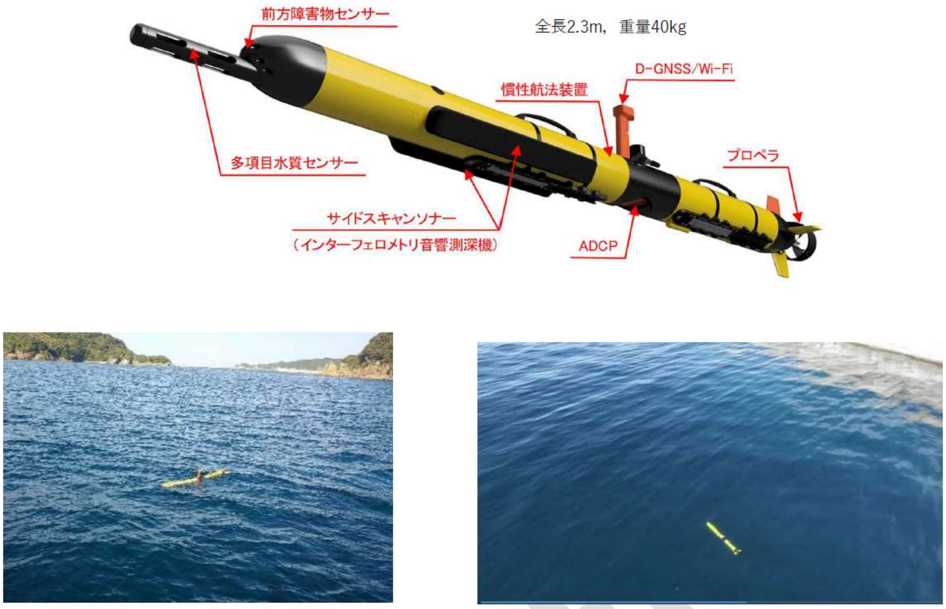


Fig. 2.3.6 Example of an autonomous AUV
(https://www.mlit.go.jp/kowan/content/001864027.pdf)

2.4 Cavity Investigation

2.4.1 General

Cavities such as those below the apron should be surveyed using a combination of appropriate methods, taking into account the type and structure of the facility.

Cavity investigation results should be appropriately recorded and preserved, considering future inspection and diagnosis programs for target structure.

2.4.2 Type of cavity investigation

The investigation of underground cavities and buried objects cannot be conducted by visual inspection alone, and requires an instrument-based investigation method. The following methods are mainly used to investigate cavities under the aprons, where subsidence is a particular problem in ports and harbors.

i) Hammer impact method
ii) Electromagnetic radar method
iii) Localized excavation method
iv) Cavity inspection hole method

i) Hammer impact method

This method involves striking objects such as an apron with a hammer and determining the presence of cavities based on differences in the resulting impact sounds. In this case, the extent of the cavity can be identified to some degree; however, the accuracy depends on the pavement thickness of the apron and it is not possible to determine the depth of the cavity.

ii) Electromagnetic wave radar method

The electromagnetic (EM) wave radar method is a non-destructive testing method that utilizes the property that electromagnetic waves emitted from a transmitting antenna are reflected at the boundary surface of materials with different electrical properties (dielectric constant, resistivity, etc.). This method is described in detail in “2.3.2 Cavity Investigation by Electromagnetic Wave Radar Method”.

iii) Localized excavation method

Various non-destructive testing methods can detect the presence and approximate shape of cavities, but accurately determining their three-dimensional shape and volume is challenging. This method involves locally excavating areas where cavity formation is suspected, for example, using electromagnetic radar, to directly determine the depth and scale of the cavity. For instance, core boring on structures such as an apron can be used to measure cavity depth, and by using a CCD camera in combination, it is possible to visually inspect the condition of the cavity near the excavated area.

In recent years, a method has been developed in which a small-diameter drill hole is made, and a specialized endoscope equipped with a measure that extends radially from its tip is inserted to directly measure the three-dimensional shape of the cavity.

iv) Cavity inspection hole method

The purpose of this method is to directly check the occurrence and progression of cavities by pre-installing inspection holes on the pavement. This method is expected to be effective in many general-purpose situations, as it can grasp the occurrence and progression of cavities directly and quantitatively, does not require advanced specialized knowledge, can be investigated during daily inspections, and can be applied to both new and existing pavement.

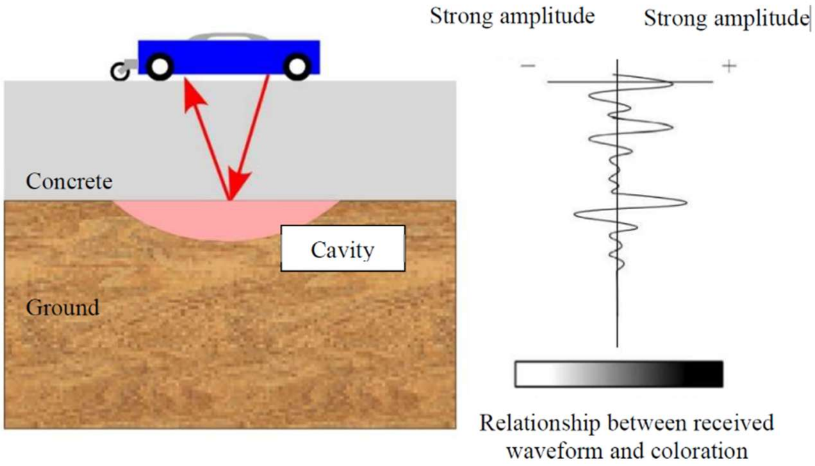
If such cavity inspection holes are installed to penetrate the superstructure of the caisson, they can also be used to investigate the loss of infill sand caused by damage to the caisson walls.

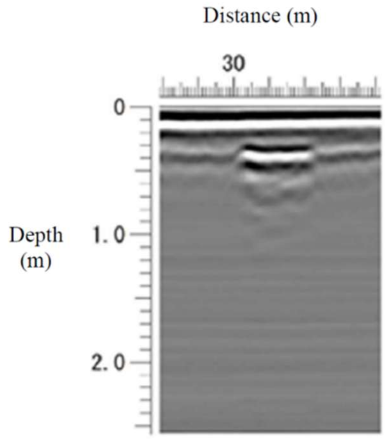
2.4.3 Cavity investigation using electromagnetic wave radar

At the quay wall, a cavity may be created on the underside of the apron due to loss of backfill soil from the sheet pile, resulting in a sinkhole in the apron (Photo 2.4.1).

Photo 2.4.1 Example of hole in apron caused by cavity

As shown in Fig.2.4.1, the electromagnetic (EM) wave radar method is a nondestructive testing method that utilize the property of EM waves emitted from a transmitter antenna reflecting at the boundary of materials with different electrical properties (such as electric permittivity or specific resistance). When an EM radar method is used to perform cavity investigation of under apron, for example, the received waves show regular shapes when no cavities exist at the inspection point. When cavities exist, the received waves include the reflections from not only the under apron but also the surfaces of the cavities. Hence, in theory, the waveforms are more complicated than when no cavities exist. In addition, focusing on changes in the waveforms (such as amplitude or phase) may allow for the estimation of the planar or spatial spread of cavities. Fig.2.4.2 shows an image obtained with the electromagnetic wave radar method.





Source：https://www.mlit.go.jp/kowan/kowan_tk5_000040.html

Fig.2.4.1 Conceptual diagram of measurement with the electromagnetic wave radar method

Fig.2.4.2 Example of an image from the electromagnetic wave radar method

The EM wave radar method currently utilized in cavity investigation is largely divided into several types, as listed in Table 2.4.1. An approximate guide for the relationship between the frequency and survey depth of EM waves is summarized in Table 2.4.2. The advantages of the higher frequencies of EM waves include greater resolution and the ability to inspect smaller cavities. However, a major disadvantage is a shallower inspection depth. Therefore, in the actual implementation of cavity investigation, it is necessary to utilize the appropriate instrument in considering the material of the object and the depth in the material.

In general, the impulse radar method is suitable for surveying cavities that occur on the underside of aprons. However, it should be noted that the corresponding depth and decomposition capability values will vary depending on the target soil type, groundwater level, and other conditions, as well as the type and thickness of the apron pavement. In addition, although the electromagnetic radar method can determine the presence or absence and location of cavities, it is difficult to determine the thickness (depth) of the cavity and cannot be applied to cavity exploration below sea level, so it is necessary to know the location of groundwater when conducting exploration below the apron.

Table 2.4.1 Types of EM Wave Radar methods used in cavity inspections

Methods		Characteristics
Impulse radar	For underground cavities	• General underground radar equipment • Used in cavity searches under roads
	For cavities behind concrete	• Possible to detect cavities at the back of reinforced concrete
Continuous wave radar		• Survey depth is approximately 5 times as much as that measured with ordinary equipment.
Chirp radar		• Survey depth is approximately 2 times as much as that measured with ordinary equipment.

The equipment used for cavity investigation is affected by steel and other conductive materials existing in the exploration area. Photo 2.4.2 shows an example of survey instruments. Since the influence of steel and other conductive materials is small in general road subsurface exploration, method (1), in which multiple exploration sensors are mounted on a vehicle for high-speed mobile exploration, can be used. On the other hand, if there are many steel materials located directly under the exploration area, method (2), in which the exploration sensor is moved at a low speed, is used.

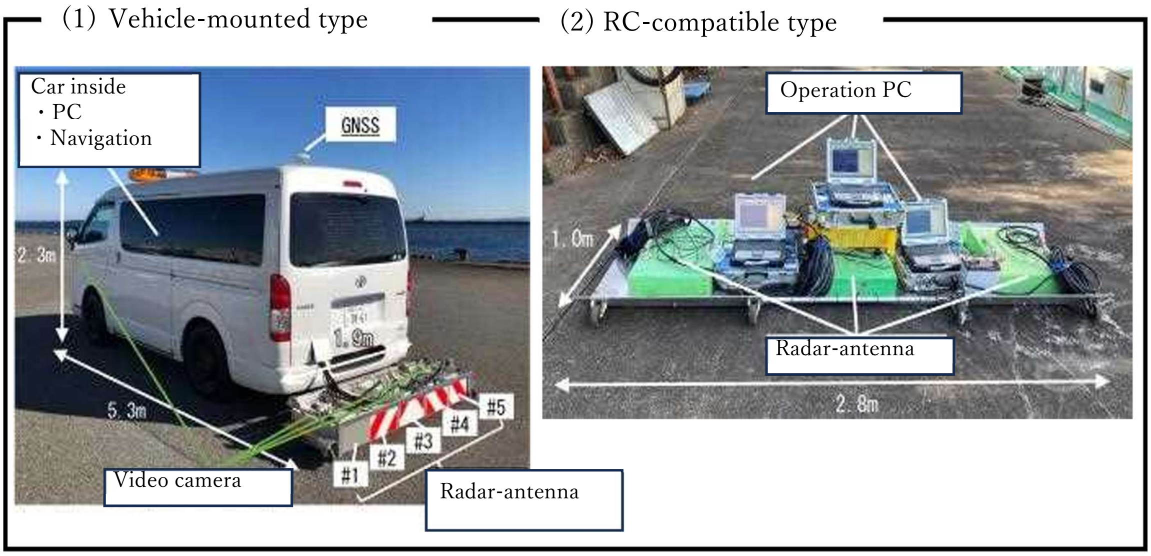


Photo 2.4.2 Example of cavity survey instruments

Table 2.4.2 Approximate guide for determining the relationship between the frequency and survey depth of EM radar methods

Target	Frequency used	Survey depth (m)
Underground piping	300 to 500 MHz	0.5 to 2
	80 to 120 MHz	2 to 10
Cavities under road	500 to 700 MHz	Just underneath the pavement surface
	200 to 500 MHz	0.5 to 3
Cavities in bedrock	500 MHz	1 to 2
	300 MHz	2 to 5
	80 MHz	5 to 10
Rebar in concrete	1000 MHz	0.05 to 3
	900 MHz	0.1 to 0.5
Thickness of pavement	1000 MHz	0.05 to 0.3
	900 MHz	0.1 to 0.5

If the presence of a cavity is suspected by the electromagnetic wave radar method, the location of the cavity is drilled and a fiberscope is inserted to investigate the presence and depth of the cavity (Photo2.4.3).

Scanning by electromagnetic wave radar

Drilling of apron where cavities are expected

Confirmation by fiberscope

Photo 2.4.3 Example of investigation of cavities

2.5 Investigation of Waterways and Basins

2.5.1 General

Inspection and diagnosis of waterways and basins should be appropriately performed to satisfy performance requirements throughout their service life.
The items for inspection and diagnosis of waterways and basins and their categorization should be specified by considering influence of deformation on performance of facilities.

The required functionality of waterway facilities such as sea routes and anchorages is to satisfy the prescribed water depth, and it is necessary to always ensure the water depth necessary for ship navigation. In waterway facilities, siltation is the main cause of performance degradation, so it is necessary to understand changes in water depth.

The causes of siltation in waterway facilities, such as sea routes and anchorages, that make it impossible to ensure the required water depth can be broadly divided into two types. One is the intrusion, precipitation, and deposition of drift sand, blown sand, and river sediment from outside the waterway facility. The other is the formation of sand waves on the bottom of the waterway facility or the movement of sediment within the waterway facility due to the collapse of the seaway slope. In addition to those that progress over time due to waves and currents, some progress rapidly due to high waves caused by typhoons or flooding from rivers caused by heavy rain.

When inspecting and diagnosing seaways and anchorages, it is necessary to select the inspection method and implementation time with full consideration of the expected siltation mechanism so that signs of insufficient water depth due to siltation can be detected as early as possible. It is also desirable to carry out inspections and diagnosis based on an appropriate inspection and diagnosis plan, with careful prior coordination to avoid interfering with the work of facility users.

The main purpose of inspections and diagnosis of waterway facilities is to confirm the current water depth of the aquatic facility using sounding equipment owned by the inspector, and to check whether floating obstacles impede the use of the facility. Specific methods include visual inspections by patrol, as well as interviews with relevant parties to confirm the condition of the facility and whether or not soil and sand have been dumped, and measuring the water depth using simple depth sounding devices or echo sounders owned by the inspector. Note that the accuracy of the water depth measurement results varies depending on the positioning and depth measurement methods used, so care must be taken when handling them.
The scope of the periodic inspection and diagnosis and the intervals between measurement lines should be appropriately determined taking into account the site conditions and burial tendency.

2.5.2 Investigation of water depth

Instruments used to measure bathymetry include acoustic bathymetry, lead, and staff (box scale), but acoustic bathymetry is preferable in deep water or in areas with fast currents. The use of narrow multibeam and other acoustic bathymetry equipment allows rapid and easy acquisition of 3D point cloud data on the seafloor profile over a wide area, greatly improving the efficiency of bathymetry operations. Acoustic devices are generally used to measure the seafloor while outfitting a ship with the device and navigating it (Photo2.5.1). Another method is to have an unmanned autonomous UAV navigate along a predetermined route to take measurements (see 2.2.1.2 Investigation below sea level).

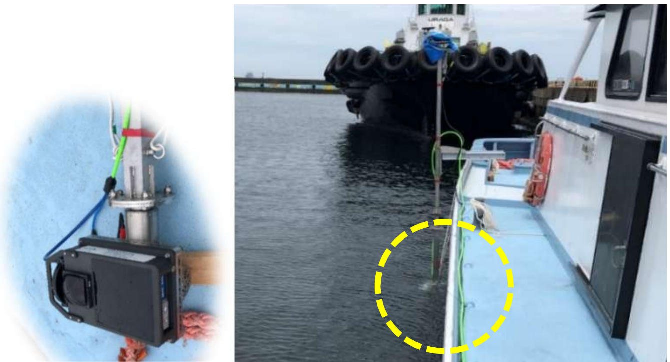


Photo2.5.1 Installation of acoustic equipment on ship

Narrow multi-beams are often used to survey the shape of the seabed, but if you want to survey the sides of structures (caissons, sheet piles, etc.) up to the water's edge and the shape of the seabed in front of the structures, a 3D scanner is generally more suitable than a narrow multi-beam. In addition, it is possible to obtain three-dimensional position information (topography and displacement data) from the sea to the sea surface by performing bathymetry and seabed shape surveys using a multi-beam bathymetry system and performing three-dimensional surveying of the sea surface using a UAV, etc., and correcting and integrating the data from both. This allows the facility to be represented in a single three-dimensional model from the sea to the land, making it easier to grasp the maintenance status of the entire facility and allowing for efficient maintenance.

For surveys of shallow water area or near the structures, bathymetry can be easily performed using a tread or staff. However, if the seabed ground is soft, it is necessary to select an appropriate method by taking into full consideration of natural conditions, such as using a tread and an acoustic device in combination and taking into account the accumulation of floating mud.

In addition, methods of positioning above sea level include optical methods, radio wave methods, and GPS methods

2.6 Investigation of Concrete Structures

2.6.1 General

The relationship between the deformations caused by various actions on port and harbor facilities and their effects on performance is called the deformation chain, and understanding this chain is very useful in selecting inspection and investigation items as well as candidates for repair and reinforcement.

Port and harbor concrete structures are exposed to more severe meteorological and oceanographic conditions than ordinary land structures. Port and harbor concrete structures are seriously affected by the physical and chemical actions of seawater, and materials steadily deteriorate over time. Chloride ions in particular penetrate the concrete, corroding the reinforcing bars. Consequently, various forms of deterioration, including cracks, delamination, and spalling of cover concrete, can occur, resulting in so-called chloride-induced corrosion. Many cases of alkali-silica reaction (ASR) caused by supplies of water and alkali in seawater in addition to the cement have been reported in certain regions.

2.6.2 Inspection/examination methods

The general inspection and examination methods for concrete structures are listed in Table2.6.1.
This section describes typical inspection and investigation methods, but these are standard methods. Even if a method is not described here, it should be applied in an appropriate manner for any inspection or investigation that is deemed necessary, taking into full consideration the scope of its application, etc.

Table2.6.1 General Inspection and Examination Methods for Concrete Structures

Inspection/examination item	Methods
Concrete strength	Compressive strength test using core, rebound hammer sounding, impact elastic wave method
Crack in concrete	Visual inspection, ultrasonic method, impact elastic wave method, infrared method, acoustic emission (AE) method
Depth of crack in concrete	Ultrasonic method, core sampling
Crack width in concrete	Crack scale method, contact gauge method

Spalling of concrete from internal cavity		Hammer sounding, ultrasonic method, impact elastic wave method, infrared method, electromagnetic wave radar method, X-ray transmission method
Conditions of corroded reinforcing bars		Measurement of half-cell potential
Rate of corrosion of reinforcing bars		Measurement of polarization resistance
Corrosive environment for reinforcing bars		Measurement of electrical resistivity (specific resistance) of concrete
Analysis of concrete	Chloride-induced corrosion and carbonation	Measurement of chloride ion concentration, measurement of carbonation depth
	Alkali-silica reaction (ASR)	Polarization microscope, scanning electron microscopy (SEM), powder X-ray diffraction
		Alkali-silica reactivity test

2.6.3 Investigation of Concrete Deterioration

2.6.3.1 Concrete Strength

The concrete strength of in-service facilities can be estimated and ascertained mainly by the following examination methods:

1) Test hammer sounding
2) Compressive test of sampled cores

1) Test hammer sounding

This method estimates the compressive strength (rebound number) of concrete based on the extent of rebounding. A weight is impacted against the concrete surface using a spring or natural gravity to determine the rebound number of the concrete. This simple method is commonly applied in various fields because it can be applied in a non-destructive way. Basically, the rebound numbers are modified based on the user manual supplied by the manufacturer to obtain the reference value. This reference value is used to estimate the compressive strength of concrete from the conversion formula. Any finishing layer or additional coating on the surface of the concrete should be removed in advance to expose the concrete. Any uneven area on the surface and organism adhesion or objects should be removed, and the surface should be polished with a suitable tool (e.g., grindstone) before starting the operation.

Photo2.6.1 Example of Test Hammer

Note that when using the rebound number, the test hammer strength can differ from the compressive strength of the standard cylindrical specimen made of the test piece's concrete by ±50% or more in certain cases. The rebound number can be affected by various factors, including the humidity of the concrete surface, the type of aggregate used and the type of test hammer. Hence, it is desirable, if possible, to sample the cores and compare the rebound number with the actual strength to verify the accuracy of the estimated results. This method is highly effective for comparing the strengths of various portions of concrete in the same structure that is constructed of the same concrete and for evaluating the uniformity of the concrete.

The following are Japanese standards for estimating compressive strength based on rebound number.

・Method of measurement for rebound number on surface of concrete (JIS A 1155:2012)

・Test Methods for Test Hammer Strength of Hardened Concrete（JSCE-G 504-2013 ）

・ASTM C805/805M Standard Test Method for Rebound Number of Hardened Concrete

2) Compressive test of sampled cores

If satisfactory data are not obtained from the aforementioned non-destructive test, or if more accurate data are required, damaging small portions of the structure will be unavoidable. A commonly used, partially damaging method is a load test of the sampled cores. This test will provide not only the compressive strength of the concrete but also the measured values of the tensile strength and the elastic modulus.

Before sampling the cores, an electromagnetic wave radar method or electromagnetic induction method should be applied to survey the reinforcing bars in the concrete to avoid cutting them while sampling (Fig.2.6.1). Photo 2.6.2 shows how the core was sampled. Care should be taken to avoid excessive torque, as this will reduce the strength of the core. Cores should preferably not be sampled at construction joints, at portions that were in contact with formwork or at damaged portions containing cracks and honeycombs, but the cores should be sampled at locations away from the reinforcing bars. After core sampling, the resulting holes are required to be repaired with an appropriate material, such as shrinkage-compensated mortar.

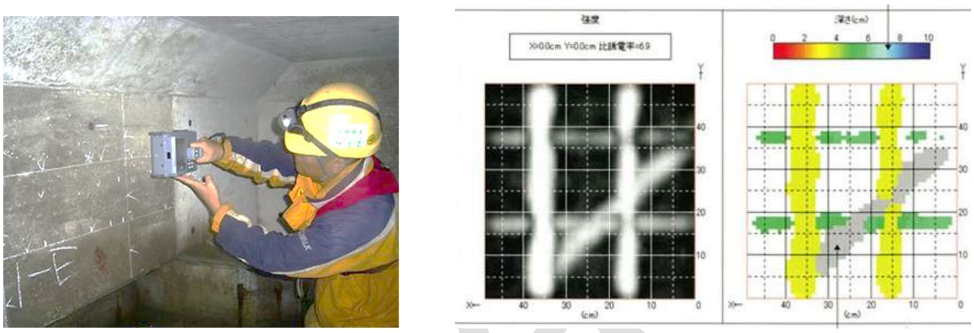


Fig.2.6.1 Exploration of internal rebar by electromagnetic induction method

Photo 2.6.2 Status of core sampling

In general, the cores to be subjected to a compressive strength test are required to have a diameter more than three times the maximum size of the coarse aggregate. The height-to-diameter ratio of the core should preferably be between 1.9 and 2.1.

The smaller the value of h/d, the larger the test result. h/d less than 1.90 should be corrected to obtain the strength, and the Japanese JIS A 1107-2012 gives the following correction factor (Table2.6.2). This correction factor is applicable when the strength is less than 100N/mm^2.

Table2.6.2 Correction factor according to h/d

Ratio of height to diameter (h/d)	2.00	1.75	1.50	1.25	1.00

Correction factor	1.00	0.98	0.96	0.93	0.87

The following Japanese standards are available for core sampling and compression testing of concrete structures.

- Method of sampling and testing for compressive strength of drilled cores of concrete （JIS A 1107:2012）
- Method of test for compressive strength of concrete（JIS A 1108:2018）
- Method of test for splitting tensile strength of concrete（JIS A 1113:2018）
- Method of test for static modulus of elasticity of concrete（JIS A 1149:2017）

In recent years, a method for estimating compressive strength using small-diameter cores has been developed to minimize the damage on the concrete structure itself (http://www.softcoring.jp/faq_kyoutu1.html).

2.6.3.2 Cracks, Spalling, and Internal Cavities

(1) Many types of inspection or examination methods can be applied to cracks, spalling, and internal cavities in concrete. The methods that are suitable for the purpose should be chosen depending on the defects to be identified and the required accuracy of inspection and examination.

(2) If deterioration develops in a concrete structure, deformation often becomes visible on its surfaces. A concrete structure damaged by chloride-induced corrosion often shows cracks on its surface, resulting from the corrosion of reinforcing bars. Rust fluid from the cracks discolors the concrete surface. If the corrosion progresses, the covering concrete will become cracked and delaminated from the main portion of the structure, exposing the reinforcing bars, and causing further corrosion to the bars. Thus, visual inspection of concrete surfaces is important for the early detection of chloride-induced corrosion. The typical symptoms of deterioration due to alkali-silica reaction (ASR) are cracks, discoloration, and exudation of gel. In general, the first step of inspection or examination is visual inspection of the concrete surfaces. If the inspector cannot access the concrete surfaces, tools such as binoculars are effective. In recent years, digital images taken with a digital camera have been used as an alternative to visual surveys.

For the spalling of concrete cover, in particular, combining a visual inspection with a hammer sounding is effective. To perform a hammer sounding, the concrete surfaces are impacted by a hammer. Based on the observed hammering sound and the impact of the hammer, the inspector estimates the locations of the concrete delamination and determines the presence of deterioration.

If any defects on the concrete surface are observed during visual inspection, and if they are to be quantitatively evaluated, it is desirable to use a simple tool, such as a scale, to determine the range of the defects. To measure crack widths, a crack scale is generally used because it is simple to operate. In general, the widest place of each crack is measured.

Below is a list of examination categories and specific phenomena to note during the observation of concrete surface conditions:

1) Discoloration and stains
- Exudation of corrosion products
- Exudation of white gel from concrete
- Efflorescence
- Discoloration of concrete

2) Cracks in concrete
- Orientations and patterns of cracks
- Number of cracks
- Width and length of representative crack
- Exudation of corrosion products from cracks

3) Delamination of concrete fragments
- Delamination of concrete fragments, number and range of locations of delamination
- Exposure and/or corrosion of reinforcing bars due to delamination, number and range of locations of exposure

and/or corrosion

4) Spalling of concrete fragments

□ Number and range of locations of spalled fragments

□ Exposure, corrosion and/or fracture of reinforcing bars due to spalling, number and range of locations of exposure, corrosion and/or fracture

(3) Non-destructive tests are highly effective when the internal conditions of the concrete are required to be identified or when more detailed data are required for estimating the deterioration mechanism and determining the degree of deterioration. For the inspection and examination using non-destructive testing apparatuses, the most suitable methods and apparatuses should be selected after establishing the purpose of inspection or examination, the scope of application and the required estimation accuracy.

Table2.6.3 lists common non-destructive test methods for cracks, spalling, and internal cavities in concrete. The elastic wave method refers to a broad category of methods for obtaining data on the internal zones of concrete by measuring the properties of elastic waves being propagated through the concrete. Included in this category are the ultrasonic wave method, the impact elastic wave method, and the acoustic emission (AE) method. The electromagnetic wave method also refers to a broad category of methods utilizing electromagnetic waves transmitted through or reflected onto the concrete mass. Included in this category are the electromagnetic wave radar method, the infrared method and the X-ray transmission method, and these methods are chosen depending on the type of electromagnetic wave.

Table 2.6.3 Types of non-destructive test methods for concrete

Condition to be evaluated	Type of non-destructive test method	Overarching category
Cracks	Ultrasonic method Acoustic emission (AE) method	Elastic wave method
	Infrared method X-ray transmission	Electromagnetic wave method
Delamination/internal cavities	Ultrasonic method Impact elastic wave method Hammering method	Elastic wave method
	Infrared method Electromagnetic wave radar method X-ray transmission method	Electromagnetic wave method

Below are examples of methods applied to the inspection/examination of locations and areas of cracks, delamination, and spalling in port and harbor concrete structures.

・Image analysis of cracks with a digital camera

Images taken with a digital camera are subjected to image processing to analyze the patterns (e.g., axial directions and honeycomb shape), the positions and the density (total extension of cracks/measured area) of the cracks. In many cases, the causes of cracks can be estimated from the identified crack patterns. The density or digitalization of cracks in concrete helps identify the nature of their development.

The performance of a digital camera depends greatly on the performance of the CCD (CMOS) and optical lens. The higher the pixel count of the CCD (CMOS), the higher the resolution, and the smaller (brighter) the F-number of the optical lens, the more faithfully the image information can be captured. On the other hand, the image information is affected by the dirt on the concrete surface to be photographed and the illumination at the time of shooting. Therefore, it is necessary to consider the width of the crack to be photographed and the shooting conditions (shooting range, brightness) when conducting a crack survey using a digital camera.

Fig.2.6.2 shows the front of the wharf photographed from the air by a UAV, with the image analyzed using special software and any abnormalities such as cracks written onto a CAD drawing.

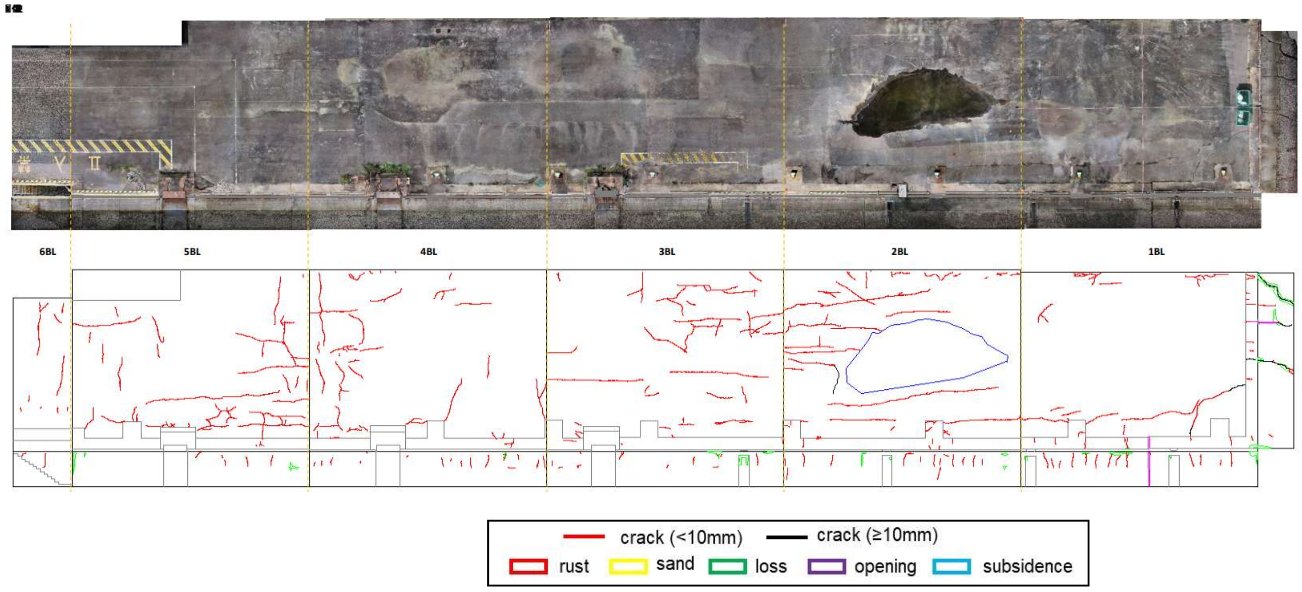


Fig.2.6.2 Deformation map of the top surface of the quay wall obtained from image analysis of a photograph taken by a UAV

・Direct measurement of widths and lengths of cracks

A crack scale, contact gauge and π-shaped displacement transducer are tools that can directly measure the width of a crack. A crack scale is placed on the crack, and the figure corresponding to the crack is visually read (Photo2.16). With a contact gauge, the length between two points is read on the gauge to determine the distance between the gauge points on both sides of the crack. With a π-shaped displacement transducer, the change in electric resistance on the gauge during loading is used to determine the change in crack width (Fig.2.6.3). To measure the crack length, a gauge is placed along the crack, and the length figure is read from the gauge.
Either method is chosen, measuring changes in crack width over time helps ascertain the future development of the crack.

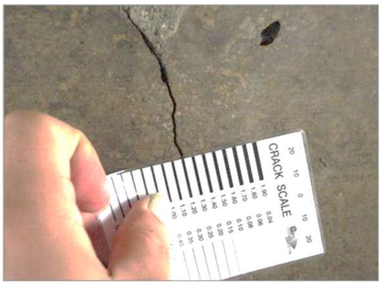


Photo2.6.3 Measurement of crack width using crack scale

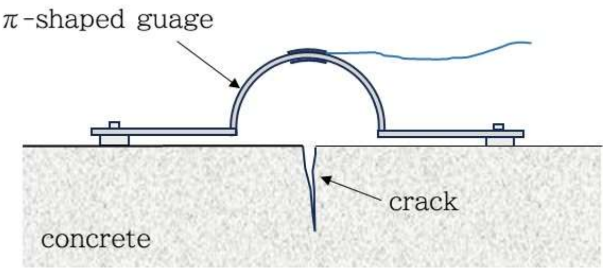


Fig.2.6.3 Measurement of crack width using π-shaped displacement transducer

・Measurement of the depth and width of a crack by core sampling

The depth and width of a crack observed on a concrete surface can be measured after sampling the core in the range containing the crack. If cracks have developed from the concrete surface into a deep zone, as in the case of concrete affected by an alkali-silica reaction (ASR), a core extending to a certain depth should be sampled.

・<u>Image analysis of delamination using a digital camera or thermography</u>

High-definition digital images are captured and modified to produce a synthetic image, based on which delamination is identified. The digital image method can be used to collect data from locations where they cannot be collected by visual inspection without approaching them. This method allows for the use of telephoto lenses that enable efficient operations without the need for scaffolding.
The infrared method (thermography) is a non-contact method for ascertaining the positions of delamination from the distribution of surface temperatures. This method is based on the principle that any delamination, cavity or crack in concrete transmits heat differently than the structurally sound portions of the same concrete
Whereas the non-contact method enables a wide range to be observed, the measurement results are affected by stains on the concrete surface, leakage of water and the degree of solar insolation. Additionally, the exploration depth limit of this method is 30 to 50 mm.

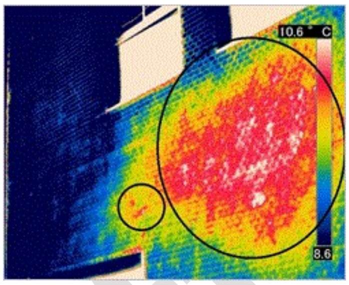


Fig.2.6.4 Example of delamination in building exterior walls using thermography

2.6.4 Corrosion of steel bars

The corrosion of reinforcing bars will not only damage the aesthetic appearance of the concrete structure with cracks and rust stains but also considerably affect its structural performance. For port and harbor concrete structures constructed in a more severe corrosive environment compared with ordinary land structures, identifying the development of corrosion of the reinforcing bars, and predicting the progress of deterioration are critical for maintaining and repairing the structures.
The methods of inspection/examination of reinforcing bars in concrete can be categorized as follows:

1) Estimating the corrosion of reinforcing bars using a non-destructive test
2) Observing the corrosion of reinforcing bars by local destruction of the mass

1) Estimating the corrosion of reinforcing bars using a non-destructive test

Non-destructive tests for the corrosion of reinforcing bars are generally based on electrochemical principles. These methods utilizing the corrosion of reinforcing bars as an electrochemical reaction allow the corrosion activity and the corrosion rate of reinforcing bars in concrete to be estimated. The main measurement items are the half-cell potential and polarization resistance. In addition, the electrical resistivity of concrete is considered an effective measurement item, as it considerably affects the development of corrosion of reinforcing bars in concrete.
A common issue with electrochemical methods is that the water content in the target concrete is liable to affect the results. If the concrete surface is completely dry or completely immersed in water, it is impossible to estimate corrosion conditions using an electrochemical method. Additionally, if epoxy resin-coated bars are used as reinforcement, an electrochemical method cannot be used.
To monitor the corrosion conditions of reinforcing bars over a long period, embedded sensors can be used in concrete. Various embedded sensors have been developed, such as sensors for measuring the half-cell potential or polarization resistance of reinforcing bars and sensors for measuring the electric current generated by macro cell corrosion.

(a) Half-cell potential
The half-cell potential is measured to estimate the possibility of corrosion in the reinforcing bars. The positive (+) terminal is connected to the reinforcing bar, and the negative (–) terminal is connected to the reference electrode. Then, the reference electrode is placed in contact with the concrete surface using a wet sponge at the point

immediately above the reinforcing bar. In most cases, the reinforcing bar is connected to the positive (+) terminal after a portion of the bar has been tapped off (see Fig.2.6.5).

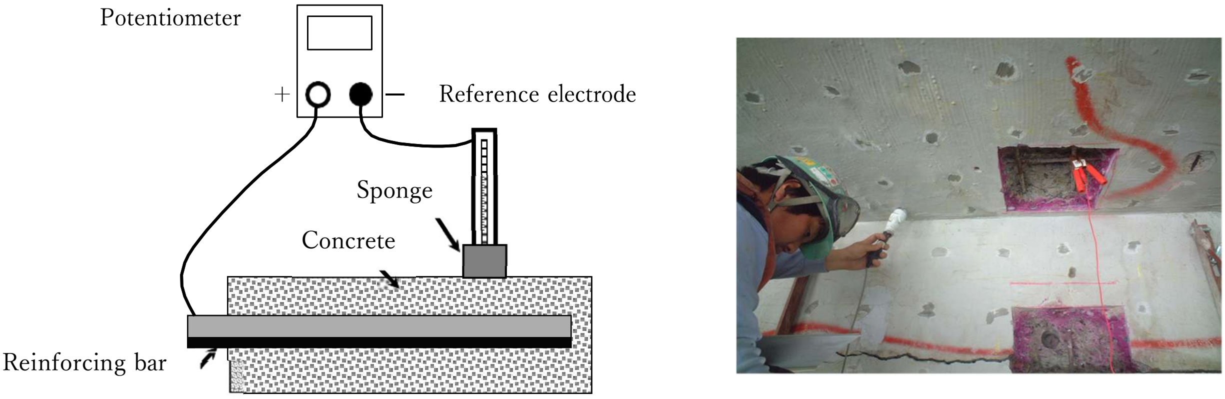


Fig.2.6.5 Measurement of half-cell potential

In general, if the half-cell potential is more negative than the criteria, corrosion has likely developed to a considerable extent in the reinforcing bar, whereas if it is more positive than the criteria, there is likely no or little corrosion. Table 2.6.4 gives examples of criteria for assessing the presence of corrosion in a reinforcing bar using measured values of half-cell potential. Notably, these criteria are not applicable without modifying port and harbor concrete structures. In such cases, the results of the assessment do not correspond to the actual corrosion conditions. These criteria should be applied very carefully after a review. It is difficult to define universal criteria for assessing the half-cell potential, since to a considerable extent, it depends on various environmental conditions, such as moisture in concrete, the chloride ion concentration, and the atmospheric temperature.

Thus, in general, half-cell potentials should be measured across the entire surfaces of the portions or members of the structure, and the distribution of half-cell potentials should be got. An effective method is to use operational tools to inspect visual factors. For example, a drawing of equipotential lines (contour mapping composed of equipotential points linked with lines) can be performed using the measured values of half-cell potentials in the examined area. Thus, the locations of the portions or members of the structure where the possible presence of corrosion is high can be visually estimated (Fig.2.6.6). Furthermore, in areas where the equipotential diagram indicates a high possibility of corrosion, pulling out the reinforcing bars and checking the corrosion status will help improve the accuracy of reinforcing bar corrosion evaluation over a wide area.

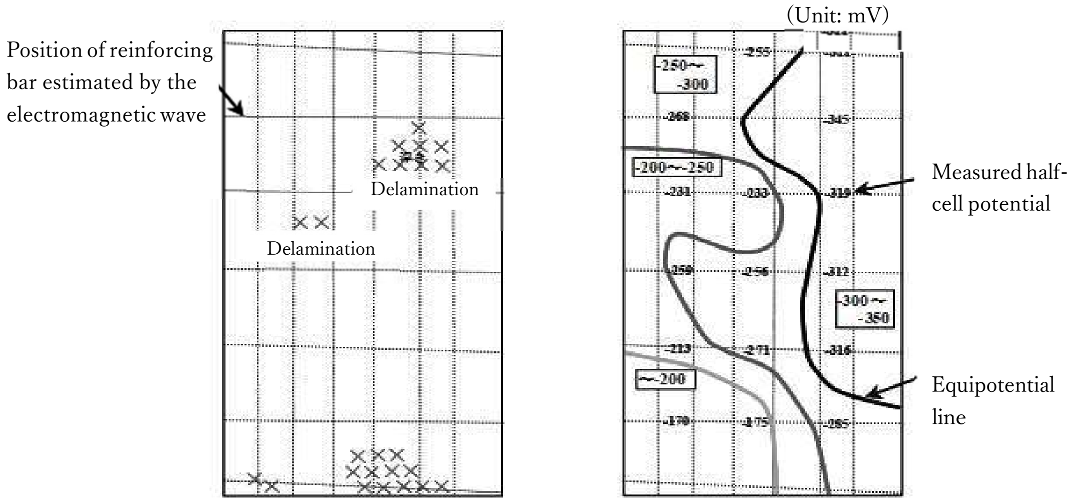
If a portion of the rebar is exposed at locations where corrosion is considered likely based on equipotential lines, and the corrosion status of the rebar is evaluated, the accuracy of the evaluation of a wide range of rebar corrosion will be enhanced.

Table 2.6.4 Criteria for assessing corrosion in a reinforcing bar using measured half-cell potentials (examples)

Half-cell potential E		Possibility of corrosion
When a saturated copper sulfate electrode is used*1	When saturated silver chloride electrode is used*2	
$-200\ \text{mV} < E$	$-80\ \text{mV} < E$	No presence of corrosion with a probability of 90% or greater
$-350\ \text{mV} < E \leq -200\ \text{mV}$	$-230\ \text{mV} < E \leq -80\ \text{mV}$	Uncertain
$E \leq -350\ \text{mV}$	$E \leq -230\ \text{mV}$	Presence of corrosion with a probability of 90% or greater

*1 ASTM C 876: Standard Test Method for Half-cell Potentials of Uncoated Reinforcing in Concrete

*2 Converted from the half-cell potential value using a 25° C saturated copper sulfate electrode



Defects on bottom of beam and positions on the reinforcing bar

Measurement results of half-cell potentials (vs Ag/AgCl)

Fig.2.6.6 Measured half-cell potentials in an RC beam of a pier superstructure

(b) Polarization resistance

To measure polarization resistance is a method to know the corrosion rate of a reinforcing bar. A part of the reinforcing bar in the concrete is exposed to secure electric conduction, and the sensor is placed in contact with the concrete surface at the point immediately above the bar. Various methods can be used for this measurement. A portable apparatus utilizing AC impedance can also be used.

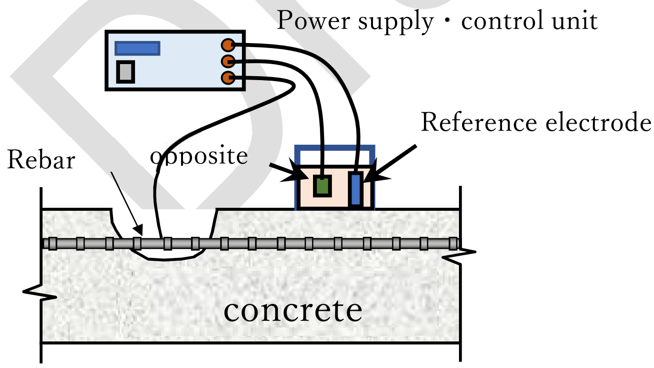


Fig.2.6.7 Measurement of polarization resistance

For a polarization resistance measurement, the electric current that is generated when a steel material's potential is slightly polarized from its half-cell potential (the potential is forcibly shifted) is measured. The polarization resistance is calculated using the following equation:

$$Rp = \Delta E / \Delta i \qquad (2.6.1)$$

where

Rp: Polarization resistance (Ω cm2)

ΔE: Amount of polarization (V)

Δi : Current generated (A/cm^2)

The corrosion current (corrosion rate) is calculated using the following equation:

$$Icorr = K / Rp \qquad (2.6.2)$$

where

Icorr : Corrosion current (A/cm^2)

K : Constant depending on type of steel material and environmental conditions (generally, 0.026 V)

Rp : Polarization resistance ($\Omega\ cm^2$)

Table 2.6.5 presents the example of the criteria for assessing corrosion rates of reinforcing bars using measured values of polarization resistance. These criteria can be used for reference purposes.

The polarization resistance method is used to estimate the corrosion rates. If the polarization resistance is successively measured, the extent of corrosion of a reinforcing bar can be estimated as an integral with time.

Table 2.6.5 Criteria for assessing the corrosion rate using polarization resistances

Polarization resistance R_p ($k\Omega\ cm^2$)	Corrosion current density I_{corr} ($\mu A/cm^2$)	Measured corrosion rate
$R_p \geq 130\text{-}260$	$I_{corr} < 0.1\text{-}0.2$	Passive (no corrosion)
$52 \leq R_p \leq 130$	$0.2 \leq I_{corr} \leq 0.5$	Low or medium corrosion rate
$26 \leq R_p \leq 52$	$0.5 \leq I_{corr} \leq 1.0$	Medium or high corrosion rate
$R_p < 26$	$I_{corr} > 1.0$	Very high corrosion rate

(c) Electrical resistivity of concrete (specific resistance)

The corrosion of a reinforcing bar in an internally dry concrete (with higher electrical resistivity) develops more slowly than that in an internally less dry (moist) concrete (with lower electrical resistivity). As there is a certain correlation between the water content and electrical resistivity of concrete, the measured values of electrical resistivity will, to a certain extent, indicate the corrosion activity rate of a reinforcing bar. Various methods can be used to measure the electrical resistivity, such as utilizing AC impedance to measure electrical resistivity and polarization resistance simultaneously and measuring electrical resistivity using four electrodes.

Fig.2.6.8 shows a conceptual diagram of the electrical resistance measurement method using the four-electrode method. ρ can be obtained from the voltage difference Δφ between C and D when a current I (DC or AC) flows between A and B, using Eq. (2.6.3).

$$\rho = 2\pi d \Delta\varphi / I \qquad (2.6.3)$$

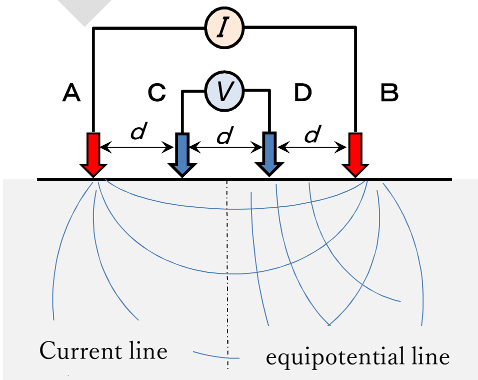


Fig.2.6.8 Method of measuring electrical resistance by four-electrode method

2) Observing the corrosion of reinforcing bars by local destruction

If satisfactory data cannot be obtained from the non-destructive test or if more accurate data are required, it is often effective to destroy local portions of the structure. If the structure is locally destroyed, the degree of corrosion of the reinforcing bars in concrete can directly be identified with the following procedure.

A part of the concrete is removed to expose the reinforcing bar, and the conditions of the bar are visually inspected. An important inspection item is the presence of pitting corrosion because if pitting corrosion is present, both the overall section of the reinforcing bar and the bearing capabilities of the portions and members are considerably reduced. Thus, the structure is in a dangerous condition.

In addition to visual inspection, if possible, the actual remaining diameter of the reinforcing bar should be measured with a caliper. When quantitative corrosion data are obtained, the corrosion rate up to the present can be estimated. Such data are beneficial for estimating the remaining bearing capabilities of the portions and members of the structure.

2.6.5 Chloride ion Distribution

2.6.5.1 Measurement of chloride ion content

Inspection and investigation of chloride ion concentration in concrete play an important role in determining the degree of deterioration and predicting the progress of deterioration of concrete structures due to salt damage.

The chloride ion concentration in concrete can be measured by potentiometric titration or ion chromatography using cores or pieces of concrete taken from concrete structures or concrete powder obtained by drilling. The inspection and investigation of chloride ion concentrations in concrete should include the following;

・Methods of test for chloride ion content in hardened concrete (JIS A 1154:2020)

・Methods for determining the distribution of total chloride ions in concrete in actual structures (draft) (JSCE-G 573-2013)

The total chloride ions are defined as the total chloride ions in the concrete. Here, total chloride ion refers to all chloride ions in the pore solution of hardened concrete, chlorine fixed in the hardened cement as salt, and adsorbed chloride ions.

Chloride ion concentrations in concrete are generally measured at the rebar location (depth of cover) and at several depths below the surface of the concrete.

The concentration of chloride ions at the rebar location is used to determine whether corrosion of the rebar has started or not. Normally, reinforcing bars in concrete are in a highly alkaline environment with a pH of 12 or higher, which makes them resistant to corrosion. This is due to the formation of a dense oxide layer called a passive film on the surface of the rebar. However, if chloride ions penetrate into the concrete and reach a concentration above a certain level up to the location of the rebar (this is called the corrosion threshold chloride ion concentration), the passive film of the rebar is destroyed and corrosion begins, even if the pH of the concrete is high. The lower limit of the chloride ion concentration for corrosion initiation is 2.0 kg/m3 in the Technical Standard for Port and Harbor Facilities and its Commentary. The value of 2.0 kg/m3 is the lower limit of the experimental results in Japan. The values may differ depending on the local environment and structural conditions.

By measuring the chloride ion concentration at each depth from the concrete surface, the apparent chloride ion diffusion coefficient in concrete can be calculated, and this apparent diffusion coefficient can be used to predict future chloride ion penetration.

When investigating the chloride ion concentration in concrete by taking cores, it is necessary to pay attention to the maximum size of the coarse aggregate, and the core diameter should be at least three times the maximum size of the coarse aggregate. In general, cores are often taken according to the following standards

・Method of sampling and testing for compressive strength of drilled cores of concrete（JIS A 1107:2012）.

It is important to record the location where the core was taken and which side of the concrete surface (exposed surface) was used. After the core is taken, the core should be stored in a sealed plastic bag to prevent the chloride ions from leaking out due to water exposure.

When using cores to determine the chloride ion concentration at each rebar location and at each depth from the concrete surface, it is advisable to use a dry concrete cutter to cut the concrete sample. This is to avoid leaching of chloride ions from the concrete due to the use of water. The thickness of the concrete pieces to be cut out should be 10 to 20 mm because the presence of coarse aggregate in a small sample can greatly affect the measurement results. Concrete pieces finely ground to 0.15 mm or less, including coarse aggregate, are used as the sample for analysis. An example of a core taken and cut in the depth direction is shown in the photo 2.6.4.

Photo2.6.4 Core sampling status and depth-sliced core samples

In the measurement of total chloride ion concentration, nitric acid is added to the sample for analysis and boiled to extract chloride ions. This is then filtered, and the chloride ion concentration in the filtrate is measured by potentiometric titration to determine the concentration of chloride ions in the concrete.

2.6.5.2 Prediction of chloride ion penetration in concrete

(1) Chloride-ion penetration and progress of reinforcing bar corrosion should be predicted to estimate deterioration of concrete structures caused by chloride-induced corrosion.

(2) In principle, deterioration should be predicted based on the results of inspection/investigation.

In chloride-induced corrosion, the reinforcing bars in concrete start corroding due to the presence of chloride ions. The expansion of the corrosion product causes cracks and spalling of the concrete, and the cross-sectional area of the reinforcing bars decreases, reducing the structure's performance. Therefore, to predict when the corrosion of a reinforcing bar will begin, it is necessary to predict the penetration of chloride ions. Predicting the progress of reinforcing bar corrosion requires knowing the reinforcing bar corrosion rate. The corrosion rate after the initiation of reinforcing bar corrosion mainly depends on the availability of water and oxygen, which are required for the corrosion reaction. Therefore, the corrosion rate varies depending on the environment in which the structure is located and on the quality of the concrete. The corrosion rate should be appropriately established by considering these influences. If the onset and rate of the reinforcing bar corrosion can be predicted in this manner, it becomes possible to predict decreases in the reinforcing bar cross-sectional area. This prediction, in turn makes it possible to predict the degradation in the structural performance of the reinforced concrete members.

Regarding the parameters required for predicting chloride ion penetration, it is preferable to use the results of chloride ion concentration from a core sampled from the actual structure. Based on the results of inspections and tests using electrochemical and other methods, it is possible to estimate the corrosion rate of the reinforcing bars, which is required to predict the progress of the corrosion.

As the movement of chloride ions in concrete can be considered a diffusion phenomenon, it is acceptable to use Eq.(2.6.4) or Fick's second law of diffusion, which is solved by using appropriate boundary conditions. Eq.(2.6.5) is widely known and is a solution to Eq.(2.6.4) that assumes the chloride ion concentration on the concrete surface is constant regardless of service life. Note that Dap in Eq.(2.6.5) is set as the "apparent diffusion coefficient" because C(x, t) represents the total concentration of chloride ions per unit volume of concrete, not in the liquid phase that is defined in Eq.(2.6.4). If the concentration of initially contaminated chloride ions Ci is unknown, it is acceptable to substitute the chloride ion concentration of a specimen sampled from a location that is considered to be unaffected by chloride ion penetration.

$$\frac{\partial C}{\partial t} = D_c \left(\frac{\partial^2 C}{\partial x^2} \right) \tag{2.6.4}$$

where

C: Chloride ion concentration in liquid phase (in solution)

D_c: Apparent chloride ion diffusion coefficient

x: Distance from concrete surface

t: Time

$$C(x,t) = C_0 \left(1 - erf\left(\frac{0.1\,x}{2\sqrt{D_{ap}\,t}} \right) \right) + \mathrm{Ci} \tag{2.6.5}$$

where

C(x, t) : chloride ion concentration (kg/m3) at a depth x (mm) from the concrete surface at elapsed time t (years)

C_0 : Chloride ion concentration at the concrete surface (kg/m3)

D_{ap} : Apparent diffusion coefficient of chloride ions (cm^2/year)

C_i : Concentration of initially contaminated chloride ions

erf : Error function

If Eq.(2.6.5) is used for prediction, the apparent diffusion coefficient Dap depends heavily on the quality of the concrete. Specifically, the apparent diffusion coefficient is greatly affected by the water-cement ratio W/C and cement type. In addition, the chloride ion concentration at the concrete surface C_0 depends greatly on the exposure environment (e.g., tidal zone and splash zone).

There are three feasible methods for setting these parameters (C_0, and Dap), as follows. Note that (iii) applies only to Dap.

ｉ) Using inspection results from the target structure

If the inspection/test results of the chloride ion concentration distribution have been obtained, C0 and Dap can be determined by conducting regression analysis of the chloride ion concentration profile according to Eq.(2.6.5).

When calculating the apparent diffusion coefficient of chloride ions using the chloride ion concentration distribution in the sampled concrete core, given the reliability of regression analysis, it is preferable to use values at five or more locations at different depths from the concrete surface. If neutralization has occurred on the concrete surface layer, since chloride ions in the concrete have moved to and concentrated at the neutralization front, the chloride ion concentration at the concrete surface layer should be excluded from calculation in such cases. Fig.2.6.9 shows an example of a chloride ion concentration profile and the result of regression analysis when the concrete surface layer is neutralized.

However, inspection and investigation results for C_0 and Dap have shown that there is a large variation even within the same structure, and therefore it is necessary to recognize that there is variation in the penetration properties of chloride ions into concrete.

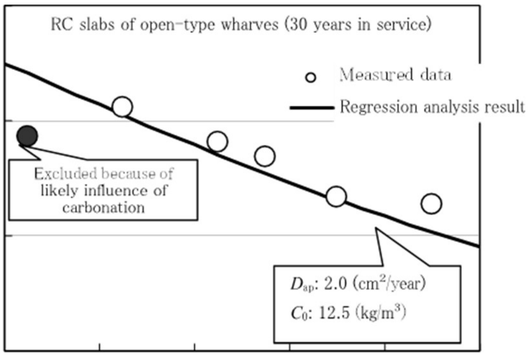


Fig.2.6.9 Chloride ion concentration profile in concrete whose surface layer is neutralized, and the result of regression analysis

ii) Using inspection results from a similar environment and structure and existing study records

If no inspection/investigation results exist, C_0 and *Dap* can be obtained by the following method:

①Apparent diffusion coefficient *Dap*

The design value of the diffusion coefficient *Dd* can be calculated from the water-cement ratio W/C using the following formula. Note that Dd can be considered as Dap. In addition, if there are no cracks in the structure or only a few cracks, the second term on the right hand side of Eq.(2.6.6) can be omitted.

$$D_d = \gamma_c D_k + \lambda \left(\frac{w}{\ell} \right) D_0 \qquad (2.6.6)$$

where,

γ_c : Material factor of concrete (= 1.0 generally)

D_k : Characteristic value of chloride ion diffusion coefficient in concrete (cm^2/year)

λ : Factor expressing the influence of existing cracks on the diffusion coefficient (= 1.5 generally)

D_0 : Constant expressing the influence of cracks on the movement of chloride ions in concrete (= 400 cm^2/y generally)

w/l : Ratio of crack width to crack interval ($w/l = (\sigma_{se}/ Es + \varepsilon'_{csd})$)

σ_{se} : Increase in the stress of the reinforcing bar (N/mm^2)

Es : Young's modulus of reinforcing bar (N/mm2)

ε'_{csd} : Constant for considering an increase in crack width due to concrete shrinkage, creep, and other factors

If the actual concrete that is used is known, the characteristic value of the chloride ion diffusion coefficient in concrete D_k used in Eq.(2.6) can be obtained from an experiment using a specimen produced from the concrete. In other cases, it is possible to obtain the value by assigning W/C to Eq.(2.6.7) and Eq.(2.6.8). In this case, however, the estimated accuracy of D_k is not high. These formulas were established based on the results of surveys conducted in Japan. These values may vary depending on the materials used (cement, etc.).

・When using ordinary Portland cement ($0.35 < W/C < 0.55$)

$$\log_{10} D_k = 3.4\,(W/C) - 1.9 \qquad (2.6.7)$$

・When using blast-furnace slag cement or silica fume ($0.40 < W/C < 0.55$)

$$\log_{10} D_k = 2.5\,(W/C) - 1.8 \qquad (2.6.8)$$

②Chloride ion concentration of concrete surface C_0

According to the Technical Standards and Commentaries for Port and Harbour Facilities in Japan, it is acceptable to obtain C_0 by Eq.(2.6.9). Note that this formula was determined based on the result of an investigation of superstructures of open-type wharves[6).

$$C_0 = -6.0\,x + 15.1 \qquad (2.6.9)$$

where

x: Distance between H.W.L. and the concrete surface (m). However, the application range of x must be approximately $0 \leq x \leq 2$, and the C0 does not fall below 6.0 kg/m3.

ⅲ) Using diffusion coefficients in concrete obtained from accelerated tests

An electrical migration test is a method to estimate the diffusion coefficient of chloride ions in concrete. With this method, even when chloride ions have not penetrated through a sampled concrete core, it is nevertheless possible to estimate the diffusion coefficient of chloride ions in the concrete. The electrical migration test measures the ease with which chloride ion migrate through porous concrete by using the electric potential gradient as the driving force of chloride ion movement. The coefficient representing this ease of movement is called effective diffusion coefficient. The effective diffusion coefficient differs from the apparent diffusion coefficient mentioned earlier, which is used for all the chloride ions in the concrete. Therefore, the effective diffusion coefficient estimated with the electrical migration test must be converted into apparent diffusion coefficient in concrete.

For the electrical migration test method and the conversion method of the effective diffusion coefficient to the apparent diffusion coefficient of chloride ions in concrete, it is recommended to refer to the following;

・Test Method for Effective Diffusion Coefficient of Chloride Ion in Concrete by Electrical Migration (JSCE-G 571-2007).

・Method for calculating the apparent diffusion coefficient using the effective diffusion coefficient by electrophoresis test (JSCE-G 571-2007 Annex).

The following Electrical Migration Test is not standardized in Japan. This test determines the diffusion coefficient from the depth of penetration of salts after the current is applied.

・Nordtest NT BUILD 492、 "Chloride Migration Coefficient from Non-steady State Migration Experiment"、Nordtest、Finland（1999）

2.6.6 Carbonation

Carbonation of concrete structures is a phenomenon in which the concrete gradually loses its alkalinity from the surface due to the penetration of carbon dioxide and various acids into the concrete. When the carbonation area progresses to the vicinity of the reinforcing bars, the passive film on the reinforcing bars is destroyed, and corrosion progresses with the supply of moisture and oxygen. Carbonation can also progress when the concrete is exposed to high temperatures due to a fire or other event.

Generally, in port structures, the concrete is in a wet state and carbonation progresses very slowly compared to salt damage, so inspections and investigations of carbonation are often not emphasized from the perspective of durability evaluation of port structures. However, when the concrete is located in the marine atmosphere or the cover is small, carbonation may progress to the corrosion of the reinforcing bars in the concrete, so it may be necessary to understand the degree of carbonation of the concrete. In addition, when carbonation occurs in the surface layer of the concrete, it may affect the distribution of chloride ion concentration in the concrete and accelerate the corrosion of the reinforcing bars. For this reason, it may be necessary to understand the degree of carbonation of the concrete even when predicting and evaluating the progress of salt damage.
The most common method for determining the degree of carbonation of concrete is to measure the depth of carbonation using cores taken from concrete structures, chipped concrete pieces and the chipped surfaces of components, or concrete powder obtained by drilling holes.

For inspection and investigation of carbonation of concrete, it is recommended to refer to the following;

- Method for measuring carbonation depth of concrete (JIS A 1152:2018)
- Carbonation test method for concrete structures using drilling powder (NDIS 3419:2011)

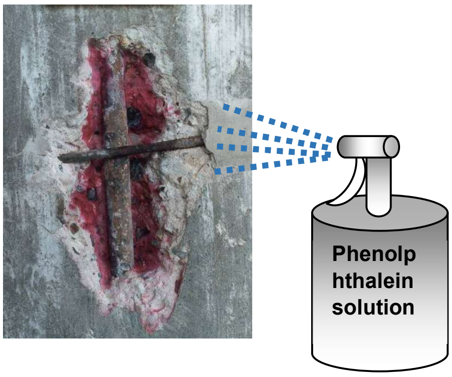


Fig.2.6.10 Test method of carbonation by Spraying 1%phenolphthalein solution

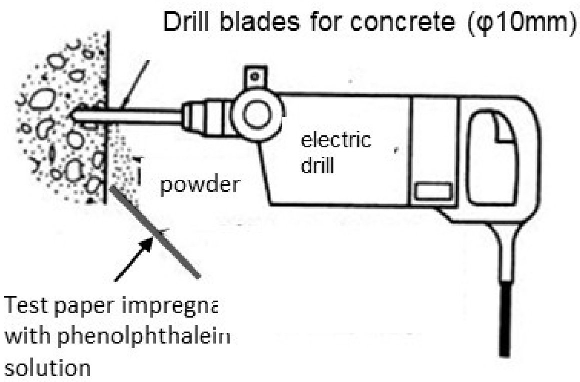


Fig.2.6.11 Test method of carbonation by drilling

2.6.7 Alkali-Silica Reaction (ASR)

The deterioration of concrete caused by ASR is characterized by the expansion of concrete due to ASR, and the occurrence and progression of cracks. The occurrence of deterioration due to ASR is mainly influenced by the reactivity of the aggregate used (type and content of reactive minerals), the concrete mix (amount of alkali in cement and unit amount of cement in concrete), and the environmental conditions to which the structure is exposed (water and alkali supply conditions, temperature, etc.), while the occurrence and progression of cracks is influenced by the amount of rebar in the structure and the restraining conditions of the concrete in addition to the above. Until now, in the maintenance and management of structures where ASR has occurred, attention has been paid to the occurrence of rebar corrosion due to cracks and consideration of aesthetics and scenery. However, in recent years, fractures of bent parts and pressure welds of rebar due to ASR have been confirmed, and the actual situation and mechanism have been investigated, and it has been pointed out that there is a possibility of serious damage that affects the structural performance of the structure.

Investigations of concrete structures where deterioration due to ASR is a concern generally consist of visual inspections, field investigations such as measuring crack widths and expansion amounts, and various laboratory tests using core samples. A list of inspection and investigation items for ASR is shown in Table2.6.6 . However, it has been reported that even if various tests are performed, the data obtained rarely provide more useful information than visual investigations. For this reason, the method of inspection and investigation of concrete structures where deterioration due to ASR is a concern must be appropriately selected based on the target deformation, the required information and its accuracy, and the reflection of the inspection and investigation results in future maintenance work. Some of the various tests using cores require a high level of expertise, and depending on the test method, data may be misinterpreted. In addition, the residual expansion of concrete obtained in accelerated tests is often smaller than that of the structure. Therefore, the amount of concrete expansion obtained in accelerated tests should not be used directly to evaluate the residual expansion of concrete structures. As a simple investigation of ASR, the Young's modulus of concrete is one indicator. According to previous knowledge, a characteristic of concrete deteriorated by ASR is that the Young's modulus decreases more significantly than the compressive strength (Fig.2.6.12).

Table 2.6.6 Inspection and investigation items for ASR

	Inspection and investigation items
Visual inspection	Cracking (in constrained direction, shape/pattern), delamination/peeling, pop-out, displacement/deformation, discoloration, step, gel exudation
On-site measurement	Crack propagation (contact gauge method, etc.) Expansion and displacement (displacement gauge, etc.) Non-destructive testing (rebound, ultrasonic pulse velocity, etc.)
Various tests with cores taken from structures	Mechanical properties (compressive strength, Young's modulus, etc.) Residual expansibility (e.g., amount of expansion) Aggregate rock types and reactive mineral types Alkali-silica reaction of aggregates Determination of alkali-silica gel Alkali content

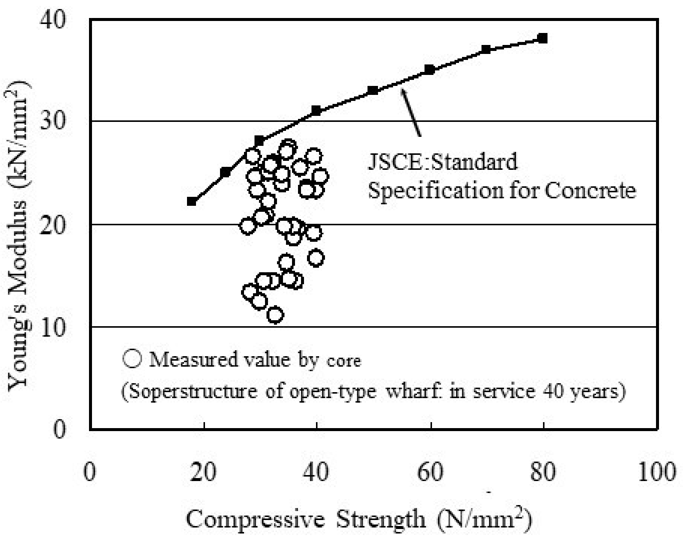


Fig.2.6.12 Relationship between Compressive Strength and Young's Modulus of Concrete

2.7 Investigation of Steel Structure

2.7.1 General

Port and harbor steel structures are exposed to the marine environment and are therefore in a more severe corrosive environment than onshore steel structures. For this reason, the steel materials are provided with protection against corrosion (corrosion prevention). Corrosion prevention methods for port and harbor t steel structures are classified as shown in the Table2.7.1.

Table2.7.1 Classification of corrosion protection methods for port and harbor steel structures

Type of Corrosion Protection		Method type and name		
Cathodic Protection Method		Galvanic Anode Method		
		Impressed Current Method		
Corrosion Protection Coating Method	Factory coating	Painting		Marine thick epoxy resin film
		Organic coating	Heavy duty anticorrosion coating	Polyethylene coating
				Urethane elastomer coating
			Super thick film coating	Super thick epoxy resin film coating
				Super thick polyurethane resin coating
		Inorganic coating	Corrosion resistant metal coating	Seawater-resistant stainless steel coating
				Thick titanium clad steel coating
				Thick clad steel coating
	On-site coating	Organic coating	Underwater coating	Painting
				Putty-type coating
				Wetting area-type coating
		Petrolatum coating		Resin protective cover
				Corrosion resistant metal protective coating
		Inorganic coating	Mortar coating	Resin protective cover
				Corrosion resistant metal protective coating
			Reinforced concrete coating	Removing form type
				Stay-in-place form type

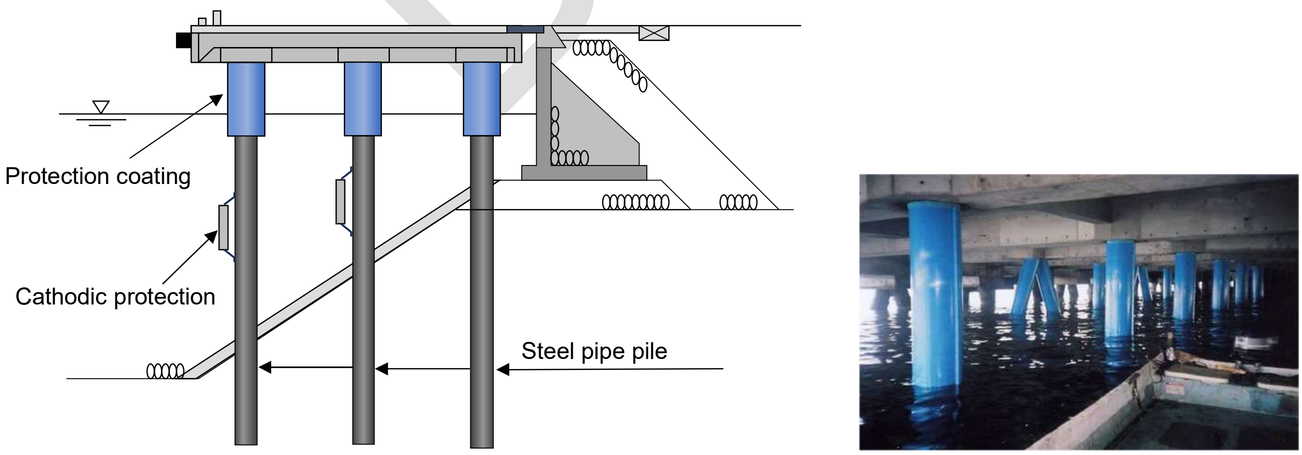


図 2.7.1 An Example of Corrosion Protection Method applied to Open-type Wharf

Generally, coating corrosion protection is applied from the tidal zone to the mid-air zone, and cathodic protection is applied in the underwater zone, but for some reason the corrosion protection function may be lost and corrosion may occur. In addition, some steels are still without corrosion protection due to designs based on the previous concept of corrosion allowance. Therefore, it is important to periodically investigate the presence or absence of corrosion of steel materials in these environments. In order to investigate the presence and extent of corrosion of steel and loss of backfill material in underwater areas, visual inspections are conducted by divers and underwater drones. Other methods are also used to confirm the corrosion protection status by measuring the electrical potential of the steel and investigating the amount of wear and tear on the anodes used for cathodic protection.

This chapter mainly covers port and harbor steel structures (quay walls, piers, revetments, etc.) using steel sheet piles, steel pipe sheet piles, and steel pipe piles, but it can be applied to other port and harbor steel structures by making judgments based on sufficient consideration of the conditions.

The inspection and investigation methods for steel materials and corrosion protection described in this chapter are standard.

This chapter describes the basic methods as standard methods for inspections and investigations of the following.

1) Inspection and investigation methods for cathodic protection (see 2.7.2)
2) Inspection and investigation methods for protective coating (see 2.7.3)
3) Measurement of steel plate thickness (see 2.7.4)

For matters not described in this chapter and basic ideas on corrosion protection, "Guidelines for Maintenance and Repair of Port and Harbor Facilities (2023), CDIT" and "CORROSION PROTECTION AND REPAIR MANUAL FOR PORT AND HARBOR STEEL STRUCTURES (1998), OCDI" can be referred to.

2.7.2 Investigation of Cathodic Protection System

There are two types of cathodic protection applied to port and harbor steel structures: 1) the galvanic anode method and 2) the impressed current method, as shown in Fig.2.7.2.

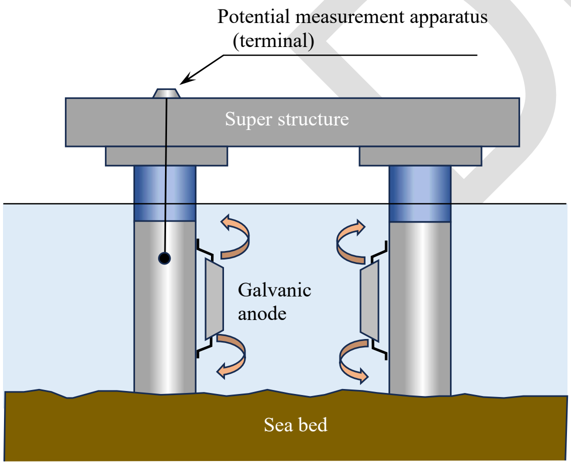
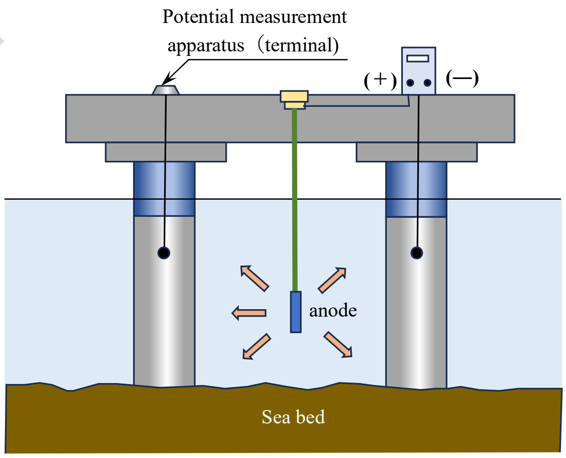
<table>
<tr><th>1) Galvanic Anode Method</th><th>2) Impressed Current Method</th></tr>
<tr><td>
</td><td>
</td></tr>
<tr><td>This method of corrosion protection is achieved by connecting a metal of lower potential than steel as an anode. In port steel structures, aluminum alloy anodes are generally used as anodes.</td><td>This method uses an external power supply device to forcibly pass a protection current from the anode to the steel material.</td></tr>
</table>

Fig.2.7.2 Types of Cathodic Protection

The selection of the cathodic protection method is determined by the efficiency of the electrolyte in the relevant sea area. Generally, the galvanic anode method using an aluminum alloy anode is adopted, but in an environment where river water is mixed in, it is necessary to select and apply the most suitable method, including an impressed current method. Fig.2.7.3 shows the classification of corrosive environments and applicable cathodic protection methods.

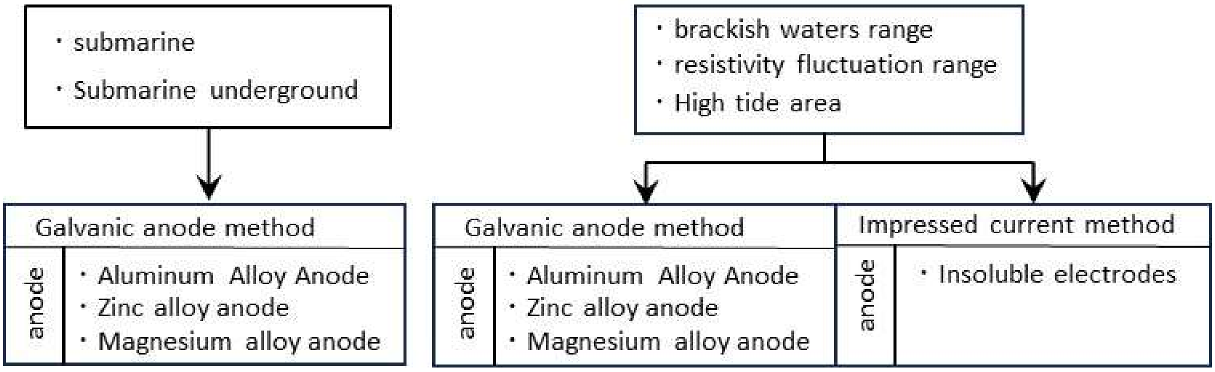


Fig.2.7.3 Corrosive Environments and Applicable Cathodic Protection Method

A site investigation of a cathodic protection system with galvanic anodes is generally carried out by;

(a) Measurement of the potential of the steel materials
(b) Investigating the installation status and consumption of anodes

Measuring the potential of steel structures can indicate the current corrosion state of a structure. The protection potential, which is the standard for determining whether steel immersed in seawater is in a state of anticorrosion, is -780 mV by an Ag/AgCl [sw] electrode. However, for maintenance and management purposes, the corrosion control management potential is set at -800 mV, which takes into account a margin in the corrosion protection potential (Fig.2.7.4). However, if the potential of steel is greater than the corrosion control management potential, the corrosion is out of control. As abnormalities such as exhaustion or anode drops occur, it is also necessary to examine the installation or consumption of the anode.

Whether using the galvanic anode method or the impressed current method, it is essential to ensure that the specified corrosion protection control potential is met. To do this, it is necessary to check the anode itself (damage or wear), the anode installation method, wiring, power supply, etc., as appropriate for any abnormalities, and to repair or reinforce any defective areas.

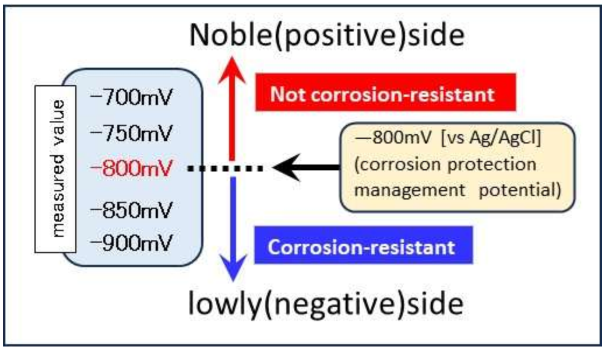


Fig.2.7.4 Example of determination of corrosion prevention effect

(a) Measurement of the potential of the steel materials
The potential of steel materials can typically be measured with a high resistance voltmeter, reference electrode and a

potential measurement apparatus installed at the target structure of corrosion protection.

In general, a DC voltmeter with an internal resistance of 1 MΩ/V or greater can be used as a high resistance voltmeter, and an Ag/AgCl[sw] electrode can be used as a reference electrode. However, if a reference electrode is used in a location such as on the seabed, where it is difficult to replace reference electrodes, and the same electrode is therefore continuously used for a long period, it is necessary to use a zinc electrode, whose composition maintains a stable and intrinsic potential. A long-term (approximately 10 years) onsite experiment has confirmed that a Zn (zinc) electrode possesses sufficient durability in a severe environment with large waves or sand movement.

The potential of corrosion protection target structures in seawater is measured at 1meter intervals in the depth direction of the structure, as shown in Fig.2.7.5. The same measurement points can be used each time. It is important to confirm the transition of the corrosion protection status with changes in potential. For potential measurements, the positive terminal of the high resistance voltmeter is connected to the potential measurement apparatus through a lead wire, while the negative terminal is connected to the reference electrode. For connection jigs, instruments, such as alligator clips, which have low contact resistance are generally used. It is also necessary to attach a weight to the reference electrode in advance so that the measurement position does not change due to conditions such as tides.

Potential measurement apparatuses (Photo 2.7.1) are installed every 20 to 50 m in the extending direction of the corrosion protection target structure. The potential is generally measured at the points where potential measurement apparatuses are installed and at the midway points.

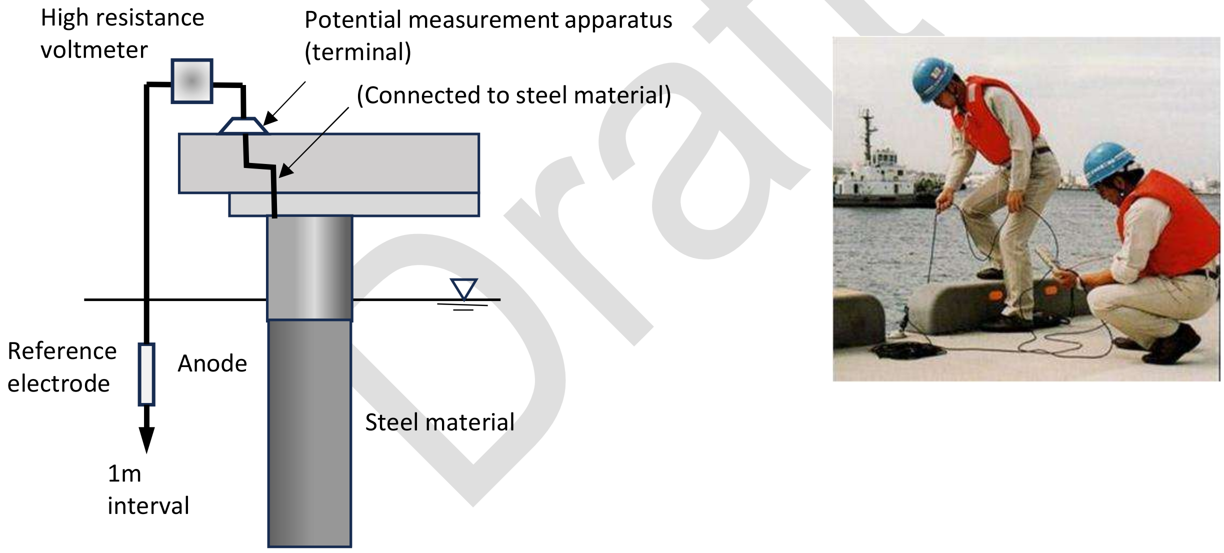


Fig.2.7.5 Potential measurement

Photo 2.7.1 Potential measurement apparatus (terminal)

(b) Investigating the installation status and consumption of anodes

Visual inspection of the anodes' installation status, the assessment of their consumption, and the measurement of the electric current generated at the anodes is necessary.

1) Visual inspection of installation status of anodes

Visual inspection is performed to confirm the number of installed anodes and that the status of the installed anodes is identical as that at the time of construction. If necessary, photos can be taken with an underwater camera.

2) Examination of consumption

By measuring the consumption of the anodes, the remaining life of the anodes and the density of protective current of the whole target structure can be determined. To measure consumption, 3 to 5% of the total number of installed anodes are randomly selected from arbitrary positions, and either a diver measures the forms and dimensions of the anodes, or the anodes are pulled up, and their weight is measured on land.

When selecting anodes to be measured, it is necessary to consider the installation position and depth, taking into account that the results of these measurements are representative of the corrosion protection state of the target structure. When selecting anodes, it is also necessary to confirm in advance the maintenance history of the anodes, such as the location and timing of anode renewal during the service period due to some cause. The following Photo2.7.2 show the state of anode wear and tear and other conditions surveyed by divers. Fig. 2.7.6 shows how to measure the anode shape and calculate the remaining mass of anode.

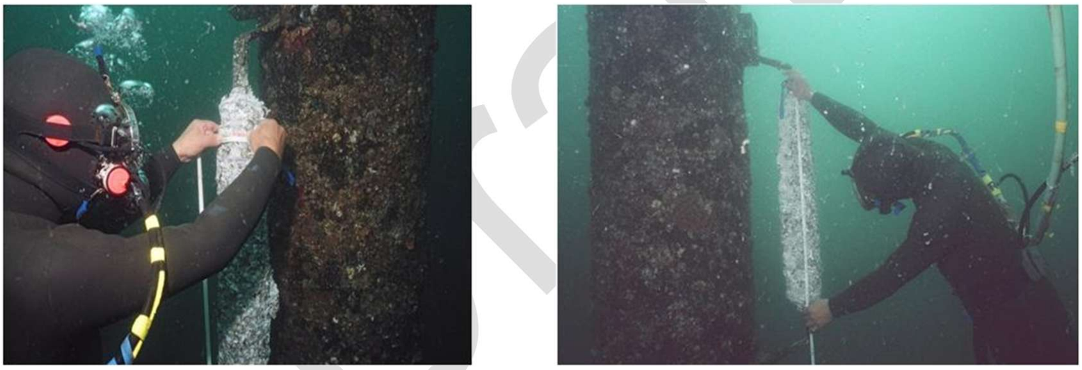
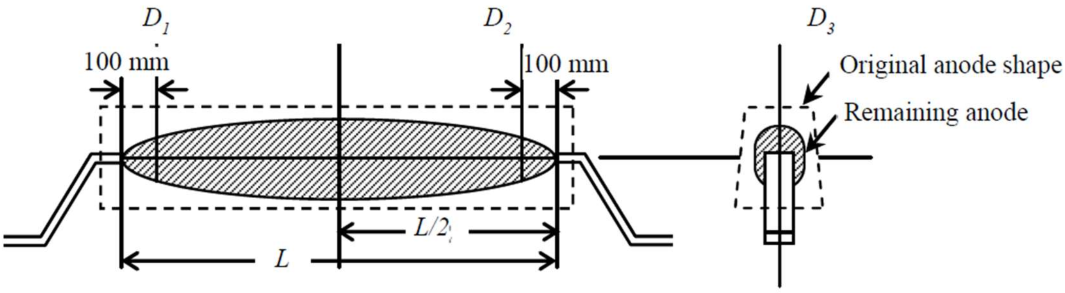


Photo2.7.2 Survey of anode wear and tear by divers



Remaining mass of anode = [$(D / 4)^2 \cdot L$ − volume of core metal] × density of anode

where

D: average perimeter = $(D_1 + D_2 + D_3) / 3$

D_1, D_3: Perimeter at a position approximately 100 mm from each end of the remaining anode

D_2: Perimeter at the center of the remaining anode

L: Length of remaining anode

Fig.2.7.6 Anode geometry measurement and calculation method of remaining mass of anode

3) Examination of generated current

By measuring the electric current generated by the anodes, it is possible to check the wear rate of the anode and estimate the approximate remaining life of the anode. The current generated by the anode is generally measured by the voltage drop method, which measures shunt voltage. For these measurements, the apparatus used to measure the generated current is required to be mounted on the anode in advance.

4) Investigation of corrosion rate by test piece

The corrosion protection effects of steel structures at ports and harbours may be confirmed using test pieces or through an environmental survey. The confirmation achieved using test pieces not only confirms the corrosion status of the surfaces of steel materials but also can quantitatively indicate the corrosion protection effects based on the weight reduction caused by corrosion.

When using test pieces, a pair of test pieces (one energized test piece and one non-energized test piece) is mounted at each depth of the target structure when the cathodic protection system is installed.

The surface condition is visually inspected, and the weight is measured. The weight reduction by corrosion, calculated from the mass reduction of test pieces, is used to calculate the corrosion rate and the corrosion protection efficiency of the target structure. The corrosion rate and corrosion protection efficiency are calculated from the test piece data using the following formulas:

Corrosion rate = corrosion weight reduction / test piece surface area / test period / test piece density (2.7.1)

Corrosion protection efficiency = (weight reduction of non-protected test piece – weight reduction of protected test piece) / weight reduction of non-protected test piece (2.7.2)

In Japan, a corrosion protection rate of 90% is generally used in the design of cathodic protection.

In addition to the above surveys, environmental surveys on water quality and bottom sediment in the surrounding sea area will provide effective data for understanding anode depletion trends. The survey items include pH, oxidation-reduction potential, sulfides, and sulfate-reducing bacteria. In addition, it is also possible to obtain water pollution indices by investigating the concentration of chloride ions, dissolved oxygen, and ammonium ions. By comparing the results of these investigations with the corrosion environment assumed at the time of design, changes in the corrosion environment of the target structure can be confirmed, and the results can be used as data for investigating the cause when the corrosion protection condition is not maintained within the service life of the anode.

(c) External Power Supply Unit

For external power supply units, it is necessary to check that they are in good working condition and are supplying the prescribed current. The following items should be checked;

・The corrosion protection control potential of the steel is continuously satisfied (if a potential monitoring device is

available).
・The operation and operating conditions of the power supply equipment meet the design conditions (output voltage, output current, etc.).
・Components have not reached their expected service life.
・Applied voltage and current values of the electrode device are appropriate.
・In the wiring circuit, there should be no wire breakage and insulation resistance should not deteriorate.

Photo 2.7.3 External power supply and inspection status

2.7.3 Investigation of Protective Coating System

Typical coating corrosion protection methods applied to port and harbor steel structures are classified as shown in Table 2.7.2 below.

Table 2.7.2 Typical coating corrosion protection methods applied to port and harbor steel structures

<table>
<tr><td rowspan="8">Corrosion Protection Coating Method</td><td rowspan="4">Factory coating</td><td colspan="2">Painting</td></tr>
<tr><td rowspan="2">Organic coating</td><td>Heavy duty anticorrosion coating</td></tr>
<tr><td>Super thick film coating</td></tr>
<tr><td>Inorganic coating</td><td>Corrosion resistant metal coating</td></tr>
<tr><td rowspan="4">On-site coating</td><td>Organic coating</td><td>Underwater coating</td></tr>
<tr><td colspan="2">Petrolatum coating</td></tr>
<tr><td rowspan="2">Inorganic coating</td><td>Mortar coating</td></tr>
<tr><td>Reinforced concrete coating</td></tr>
</table>

(1) The main objective of inspection and examination of the protective coating is to visually identify deformation in the coating materials that might have occurred due to aging or the impact of driftwood and vessels. This protection ensures that no abnormal condition exists. If any harmful deformation is observed, it is necessary to perform another specific examination to precisely identify the specific conditions and cause(s) of deformation and then take

appropriate measures.

Visual inspection of the protective coating is performed for the entire area of the target facility. It is recommended that all the steel materials be examined. If this is not possible, the most typically affected portions of the structure should be examined. Inspection and examination records should be consulted, and the portions of the structure in which abnormal conditions were observed during previous inspection should be added to the current inspection areas. Photos should be taken of these portions and deformations.

While examining the protective coating, defects, such as insufficient base material adjustment during construction and insufficient fastening of bolts/nuts, are often observed within one year of completion of construction.

(2) Examination to identify the precise conditions of any damage or deterioration should be performed in the splashing zone, tidal zone, and areas under the sea to which protective coating has been applied. In general, adhered organisms and objects below tidal zones should be removed before examining a pile at each inspection point or a concave sheet-pile and convex sheet-pile at each inspection point.

Deformation of the protective coating can take different forms, depending on the coating material. Visual inspections should focus on the following deformations based on the type of coating material:

- ①Painting: blistering, cracking, and separation of coating; signs of rusting on coated steel material
- ②Organic coating: separation, blistering and cracking of coating material
- ③Petrolatum coating: missing protective cover, corrosion and loosening of bolts or nuts
- ④Mortar coating: [Non-protective cover type] falling, cracks and separation of mortar
  [Protective cover type] falling, crack, deformation and separation of protective cover, corrosion and loosening of bolts and nuts
- ⑤Metal coating: corrosion, rusting, falling, crack, damage, abrasion, and flaws of the steel material

Inspection and examination methods and precautions for each type of coating material are specified as follows:

When any kind of deformation is observed in the coating protective, the degree of rusting of the base metal (steel) is important, and inspection and investigation should be conducted focusing on this.

The inspection and investigation methods and points to be considered are described below, depending on the type of coating material. In many cases, inspection, investigation, and evaluation require specialized knowledge on each type of cladding protection.

① Painting

Deterioration of the paint film can take the form of blistering, cracking, separation of the paint film and rusting of steel material under the paint film. If deterioration of the coating reaches the surface of the steel material, running rust will appear. Inspection and examination of the coating is mainly performed in detail visually, focusing on rust, blistering, cracking, and separation of the paint film. During visual inspection, a hammer sounding with a plastic hammer should be performed to determine if fragments of paint film with reduced adhesion remain on the steel material.

Several methods are available to identify the degrees of rust, blistering, cracking, and spalling. For example, these phenomena can be scored using standard pictures corresponding to the size and development density in several steps, or they can be quantified with an area ratio.

The area ratios of rust, blistering and spalling are visually estimated, using the criteria for the rust area ratio as a reference. The rust area is the area of the rusted surface of the steel material. The rust area does not include the area of the coating surface covered with running rust.

In areas where it is possible to conduct an instrumental survey, it is also possible to quantitatively determine the deterioration status of the coating by combining measurements of coating thickness, adhesion, impedance, and other parameters.

② Organic coating

Organic coating has a paint film thicker than that of an ordinary painting and has good durability. The items that are inspected and examined are essentially identical to those for ordinary painting, and a similar visual inspection is performed.

③ Petrolatum coating

Petrolatum coating consists of an anticorrosion petrolatum material on the surface of the steel material and a protective cover. The visual inspection mainly focuses on the protective cover. In the case of petrolatum coverings, an investigation 20 years after construction showed that if there were no internal voids, and the protective cover was sound, the petrolatum anti-corrosion coating inside was also sound, and no corrosion of the steel pipe piles was observed. Thus, for petrolatum coating in general, the deformation of the protective cover and any cavities beneath it should be inspected.

Inspection of the protective cover should focus on whether there is cracking and deformation of the cover and corrosion and loosening of the fastening bolts or nuts. Inspections of the cavity should be performed by a sound examination using a plastic hammer or other suitable means. In the case that pieces of particular band or stiffening plates have been applied to the joints of protective cover, and if there are any elements such as cracking or deformation, appropriate surveys should be required.

If the sound examination reveals internal cavities containing moving seawater, the protective cover should be removed, and any corrosion of the steel material should be examined.

④ Inorganic coating (mortar coating)

Mortar coating may or may not have a protective cover.

For mortar coating without a protective cover, deformation, such as cracks in the coating material or missing portions, should be visually inspected. If the coating material has deteriorated, cracks and/or a loss of a portion may be observed, and “rust strain” may exude from the corroded steel material at the end portion.

For mortar coating with a protective cover, the inner material is generally considered sound if the protective cover is intact, as in the case of petrolatum coating. Thus, crack examination on the cover should be performed. If there is a cavity between the protective cover and the mortar, and if seawater is flowing into this cavity, the cover has lost its protective function. In this case, sound examination should be performed to determine if there are any cavities. If a cavity is detected, the protective cover should be removed, and inspection should be performed as if it were a mortar coating without a protective cover.

The inspection and examination of mortar coatings should mainly be performed visually. The core of the material may be sampled to measure the chloride ion concentration as required because the concentration of chloride ions infiltrating the mortar material often provides valid information.

The inspection and examination of reinforced concrete coating should be analogously performed. It is desirable to also examine the reinforced concrete to detect any corrosion of the reinforcing bars.

⑤ Metal coating

Inspection of corrosion-resistant metal coatings is mainly carried out by visual inspection. When carrying out visual inspections, care must be taken as rust caused by metal powder such as iron powder may be mistaken for rust that has developed on the corrosion-resistant metal coating itself. Visual inspections below the sea surface are generally carried out by divers. If deterioration is found during the visual inspection, an ultrasonic thickness gauge or similar tool is used to measure the thickness of the corrosion-resistant metal at the deteriorated area.

In general, anticorrosive metals are used for metal coatings. Therefore, if the coating material is undamaged, the base material (steel) remains sound. However, once the coating is damaged, bimetallic corrosion occurs between the steel material and the coating metal. Such corrosion may develop considerably in a brief period. The examination should be carried out carefully to identify even minor damage. If the cathodic protection method applied in seawater is working correctly, bimetallic corrosion is prevented from occurring in the tidal zone. It is also necessary to evaluate the anticorrosive effect of the cathodic protection method during the inspection and examination of metal coatings.

2.7.4 Measurement of steel plate thickness

(1) Thickness measurement by ultrasonic thickness meter

Measurements of the steel plate thickness are performed to quantitatively ascertain the corrosion conditions of a steel structure. For this kind of measurement, appropriate examination locations, measuring positions and measurement points should be chosen depending on the purpose of the examination. Additionally, local corrosion should be measured, as necessary.

In general, an ultrasonic thickness meter is used to measure the steel plate thickness. This apparatus uses ultrasonic pulses. Pulses emitted by a probe placed on the steel material are reflected by the bottom of the steel material and returned to the probe. This apparatus utilizes the principle that the duration between emission and return is proportional to the transmission distance of the ultrasonic wave. Separate dedicated probes are used above and below sea level. An ultrasonic thickness meter capable of measurements both in the air and underwater using different probes is typically used. It is necessary to calibrate the ultrasonic thickness meter in advance. To obtain accurate thickness measurements, a reference steel plate whose thickness is known and nearly identical to that of the target steel material should be used.

Photo 2.7.4 shows the plate thickness measurements. Using an ultrasonic thickness meter, three rounds of each measurement are performed at the specified measurement points. The average of the resulting values is used as the definitive measured value. If an abnormal value is obtained, measures, including remeasurements, should be considered.

Photo 2.7.4 Plate thickness measurement

All operators of the plate thickness measurement devices should be briefed in advance on specific procedures, including the application of a probe to the surface of the steel material. It is desirable to take photos of the steel surfaces at the plate thickness measurement points while performing the measurements.

The ultrasonic thickness meter has the advantage of yielding accurate results in a simple way. Nonetheless, it is necessary to remove deteriorated areas of the coating material and organism adhesion in advance to expose the surface of the target steel material. This preparatory work requires manpower and takes a long time. The debris from the removal work is required to be disposed of elsewhere.

A non-contact thickness measurement technique using ultrasonic waves and electromagnetic induction has been developed as a new technique that can obtain a wide range of data more simply and efficiently than the method using a general ultrasonic thickness measurement (refer to https://www.mlit.go.jp/kowan/content/001597154.pdf, https://www.nipponkaiyo.co.jp/product/cygnus-dive/). However, when using these new technologies, it is necessary to understand the accuracy that can be obtained.

(2) Selection of examination points and measuring positions

The examination points are a group of measuring positions projected on a plane of a normal line of the structure. These points indicate the general conditions of corrosion. When performance assessment is performed for a facility, these points are used to evaluate the entire facility. Therefore, appropriate examination points should be selected. Based on the results of visual inspection of the structure, the conditions of opening and pitting corrosion, the conditions of corrosion (range of rust), the age of the facility, and the initial steel plate thickness should be considered, and considerably corroded portions requiring measures should be primarily selected.

If visual inspection suggests a nearly uniform distribution of corroded points along a long section of the normal line, the examination points may be selected for such sections using the following methods:

- Corrosion conditions (i): The pitting corrosions or a continuous group of orange-colored rust points are observed over a wide range from the mean low water level to the L.W.L. If the facility is at least five years old, one examination point should be selected approximately every twenty meters (20 m) along the normal line.
- Corrosion conditions (ii): Orange-colored rust points are partially observed from the mean low water level to the L.W.L. If the facility is at least ten years old, one examination point should be selected approximately every fifty meters (50 m) along the normal line.
- Corrosion conditions (iii): For conditions other than (i) and (ii), for example, if no or very few orange- colored rust points are observed in the range from the mean low water level to the L.W.L., one examination.

Photo2.7.5 Corrosion conditions（ⅰ）

Photo 2.7.6 Corrosion conditions （ⅰ）

Photo 2.7.7 Corrosion conditions（ⅱ）

Photo2.7.8 Corrosion conditions（ⅲ）

Photos 2.7.5 to 2.7.8 can be used as a reference. When the condition is unclear, the more serious corrosion condition should be selected.

For corrosion conditions (ii) and (iii), the distances between the examination points are wider than those for corrosion condition (i) because the possibility of the presence of serious corrosion requiring immediate measures is lower in corrosion conditions (ii) or (iii) than in corrosion conditions (i). Thus, the economic efficiency of the examination is considered in choosing among these options. Accordingly, if the steel plate thickness measurements suggest more severe corrosion requiring measures than was suggested by visual inspection, additional examination points should be selected to achieve a frequency of one examination point every twenty meters (20 m) along the normal line, as in the case of corrosion condition (i). Moreover, it is desirable to select at least two examination points for every facility. To determine the corrosion condition ((i), (ii), or (iii)), visual inspection should be performed.

When the performance of a substructure of an open-type wharf is evaluated, the plate thicknesses of all piles along a row perpendicular to the normal line should be measured at every examination point so that each pile can be evaluated depending on the corrosion conditions.

The measuring positions are the group of points representing the positions of the steel plate thickness examination points in the vertical direction. These points are selected to examine the distribution of the corroded spots of the portions and members of a structure in the vertical direction. The measuring positions should be selected by considering the functions of the corrosion protection after the corrosion conditions of the steel material are identified by visual inspection to ascertain the type of corrosion (e.g., concentrated corrosion). Moreover, it is desirable to select points that may correspond to the sites of structural problems (e.g., the points where the maximum stress is generated).

Fig.2.7.7 shows the standard measuring positions for piles and sheet piles.

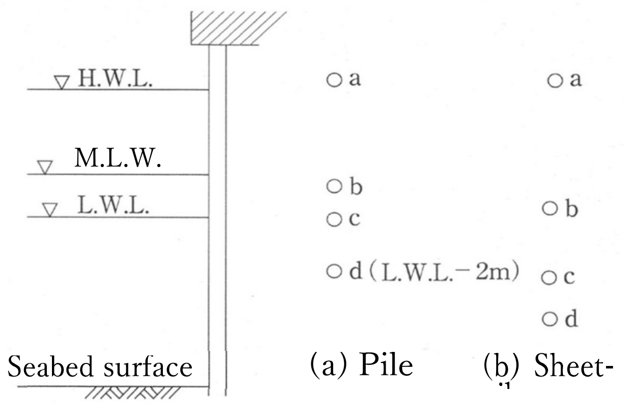


Fig.2.7.7 Standard measuring positions

【For piles】

Point “a” of a pile is located at the highest portion and is subject to high stresses and the most serious corrosion. Points “b” and “c” are subject to not only relatively high stresses but also concentrated corrosion. The results of a stress analysis for a structure largely depend on the section’s stiffness around these points. The maximum amount of corrosion should be determined as accurately as possible. Thus, positions that are expected to be most affected by corrosion should be selected as Points “b” and “c,” considering the corrosion conditions along the zone from the mean low water level to the L.W.L. If the tidal level difference or the range of serious corrosion is large, it is desirable to increase the number of measuring positions. If the tidal level difference is small, only one measuring position around this section is required to be selected. The position “L.W.L.–2 m” is acceptable as Point “d” in seawater, as the corrosion rate does not vary around this position below sea level even in the case of concentrated corrosion. Thus, this position will not significantly affect the results of a stress analysis.

【For sheet piles】

In the case of a sheet-pile, the stresses around the H.W.L. and the zone from the mean low water level (M.L.W.L.) to the L.W.L. are relatively low. However, if a hole is produced with the development of corrosion, the back-filling material will start to flow out. Therefore, the measurement points should be located around this zone. A position that is expected to be most affected by corrosion should be selected as Point "b," according to visual inspection of the zone from the M.L.W.L. to the L.W.L. As the maximum stresses are generated in seawater, two points should be selected in seawater to identify the maximum amount of corrosion. For sheet-pile structures, measuring positions should be selected around the zone where the maximum design bending moment is produced. A vertical distance of 0.5 m to 1.0 m should be secured between points.

(3) Selection of plate thickness measurement points

The plate thickness measurement points indicate the points at which the thickness of the steel material is to be measured. The measurements of thickness correspond to the actual section shapes of the steel materials at the measuring position. Fig.2.7.8 shows the standard thickness measurement points for differently shaped steel materials.

Organism adhesion and objects located in the 100 mm square should be removed with a tool, such as a hammer or scraper, and the exposed steel material surface should be further scraped with a wire brush or a grindstone to clear the surface of the steel material.

At each plate thickness measurement point, the probe of the thickness meter should be placed at five or three points (see Fig.2.7.9) to measure the steel plate thickness. The average of the five or three measurements should be adopted as the definitive measured value.

If the steel material has a hole, its location (the hole's center point) should be selected as one of the thickness measurement points, and the value "0" should be assigned to the thickness of the steel at this point. If the coating protection has partially deteriorated, the measurement points should be located in these portions. For steel pipe sheet piles, joints should be excluded from the selection of measurement points.

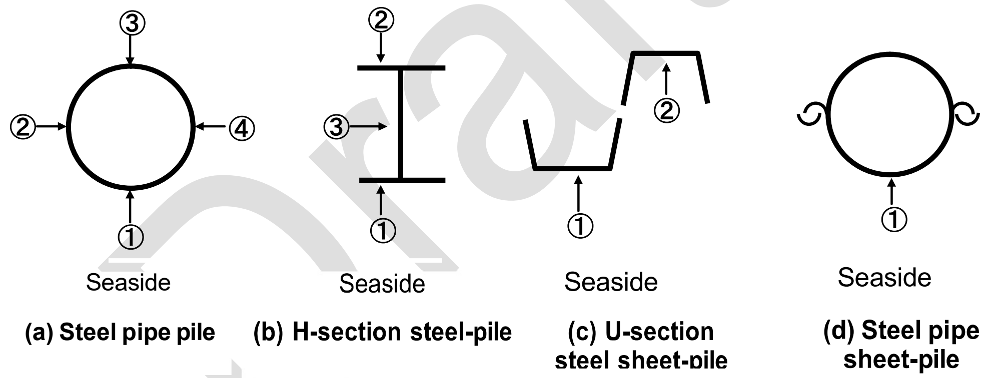
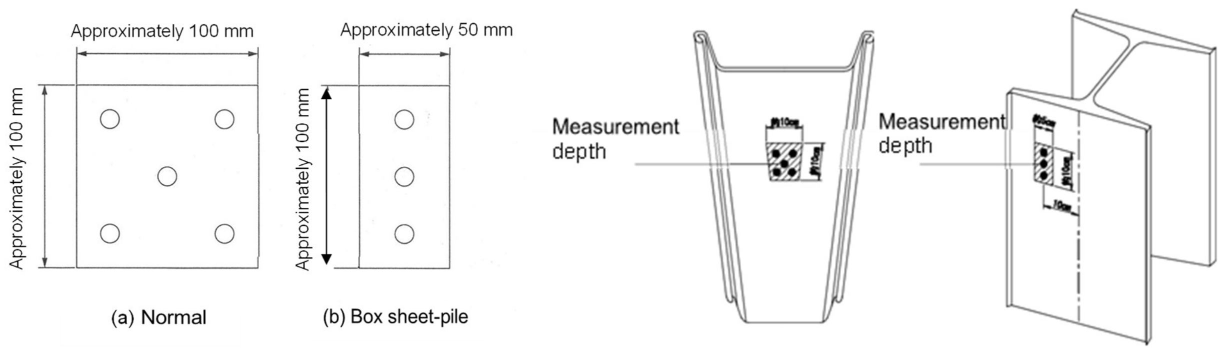


Fig.2.7.8 Standard thickness measurement points



▨ : Approximate zone where steel material surface is exposed

● : Points to which the probe is applied (three rounds of measurement at each point)

Fig.2.7.9 Points at which the thickness meter probe is placed

4) Measurement of local corrosion

The surface of corroded steel may have flat areas of uniformly developed corrosion and uneven areas with many scattered corrosion points (pitting corrosion) of different sizes. To enhance the accuracy of the performance assessment of steel materials, it is crucial to examine the conditions of the local corrosion areas and measure the depth of the corroded portions.

If any local corrosion (more than 3 mm deep) is observed, casts of these corroded portions should be taken, or their depths should be measured with a depth gauge to determine the current minimum steel thickness. Additionally, the corrosion shapes should be recorded. If several local corroded portions are observed, it is desirable to choose the five largest corroded portions and measure the depths of corrosion in these areas (see Fig.2.7.10).

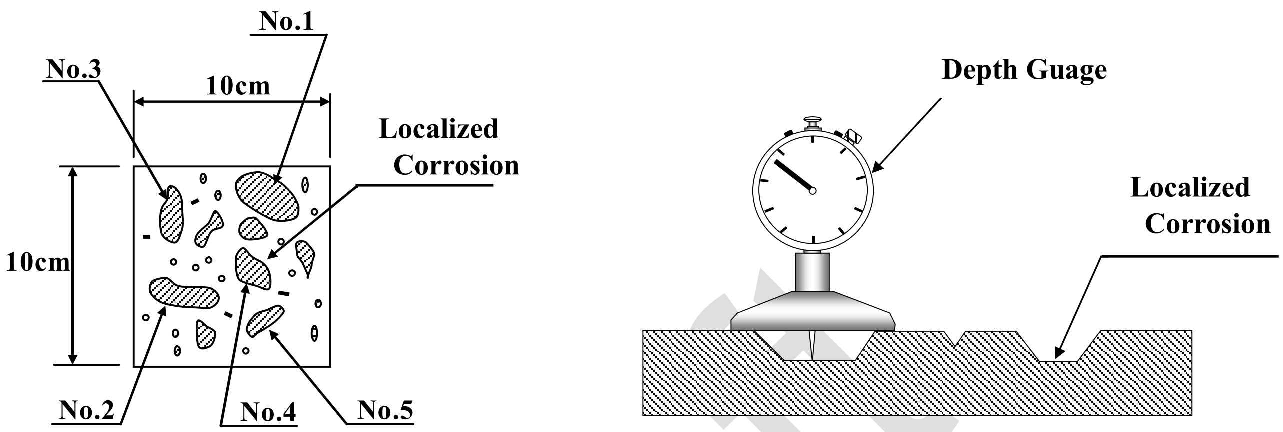


Fig.2.7.10 Example of local corrosion measurements with a depth gauge