Scientific paper

New Renovation Method for Jetty Structure Using High Durability Concrete Form Cured with CO₂ Gas

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Abstract

Studies on reduction of life cycle cost and rationalized construction methods were carried out with emphasis on the renovation of slabs of jetty structures. The results of a comparison of various kinds of renovation methods with focus on slabs of jetty structures in service revealed that a construction method using pre-cast concrete form with exceptionally enhanced durability was ideal. Analytical and experimental studies were carried out on durability design related to renovation methods, and test construction using a new renovation method was implemented. Durability was verified after the studies over the years, and a rational renovation method was proposed.

1. Introduction

Maintenance of ports and harbors started in 1961 based on a Japanese government plan since port and harbor facilities were very important for trade especially in a country with little natural resources such as Japan (MLIT 2002). In the seventies, furthermore, large-scale construction of ports and harbor facilities starting with jetties commenced in cities such as Tokyo, Nagoya and Osaka, accompanying the increase in the gross national product.

On the other hand, it was confirmed that many of the concrete structures constructed in the seventies showed early degradation after about 10 years of use, as shown in **Fig. 1**, owing to inadequate knowledge related to the durability of reinforced concrete structures. To respond to these circumstances, substantial investigations and studies on measures were started (PWRI 1983, JRA 1984, CDIT 1987) from the latter half of the eighties, aiming for construction of structures with excellent durability, with the focus on concrete structures in ports and harbors. At the same time, technical investigations related to renovation methods of degraded concrete (JCI 1994, JCE 2000), technical development, accumulation of technologies related to maintenance, and concepts of maintenance were specified (JSCE 1995, JSCE 2001), and guidelines were issued for their application to general structures.

While repair and maintenance guidelines were being developed, many instances of degradation re-appeared in

structures within 10 years after renovation from the latter half of the eighties to the nineties. The mechanisms of such instances of degradation have been clarified in recent years. The degradation was due to macro-cell corrosion in renovation materials and existing concrete.

Table 1 shows an overview of the history of renovations carried out from the time of completion of Jetty A, as a detailed example. Jetty A was renovated at intervals of about 10 years. The items assumed to be the causes of degradation are listed below.

- At the time of completion of the structures in the sixties, the understanding on ensuring durability against chloride induced deterioration was inadequate, and the concrete cover on the structures was insufficient.
- Awareness of the necessity of preventive maintenance was lacking.
- The period of renovation was restricted by usage and economic conditions, so renovation and reinforcement to suit the level of degradation could not be implemented appropriately.
- Renovation was restricted to rehabilitation of the actual state so as to avoid changes in the form of the structure, such as the thickness of members, because of various issues related to applications and formalities; therefore, renovation and reinforcement could not be implemented rationally.
- Knowledge and know-how related to macro-cell

Fig. 1 Status of early degradation of slab of jetty structure.

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Table 1 Jetty A renovation history.

Year (years passed)	Jetty A	Overall trend
1968 (0 years)	Completed	1961: Substantial maintenance of ports and harbors Seventies: Signs of early degradation Latter half of eighties: Substantial investigations and studies on measures Latter half of eighties to nineties: Early degradation of structures found again
1981 (13 years)	Slabs and beams renovated	
1984 (16 years)	Substructure reinforced	
1988 (20 years)	Access jetties renovated	
1991 (23 years)	Slabs and beams renovated	
2004 (36 years)	Slabs and beams renovated	

corrosion did not exist at that time.

- Superior renovation materials of today did not exist then.
- Maintenance taking into account life cycle cost (LCC) has not been applied generally to reinforced concrete structures, and makeshift renovation has been carried out repetitively.

The above were the causes that led to re-degradation at an early stage. Renovation was carried out at intervals of 10 years; as a result, LCC was not minimized.

The authors have carried out various studies as part of the present research to establish a jetty renovation method using pre-cast concrete forms with high durability aiming to rationalize construction work and reduce LCC. The focus was on the floor slabs, selected from among items that constitute the jetty structure, such as

floor slabs, beams, and piles. Investigations were carried out on the design and renovation methods from construction aspects, structural capacity and durability of repair parts, with verification through analyses and experimental methods, and verification tests were carried out by test construction including work efficiency verification tests.

2. Conventional renovation method of slab of jetty structure

Table 2 shows an overview of a typical conventional renovation method used mainly for jetties. Although the method of using pre-cast concrete form in method (1) has the disadvantage of a slight increase in construction expenses by manufacturing and erection of pre-cast concrete forms, it has the advantages of "the right material in the right location," that is, high quality materials can be rationally used in members under conditions such as a severe environment, stabilized quality and short construction period. Although the patching repair method in (2) has the disadvantage of a high probability of renovation becoming necessary again after about 10 years because of macro-cell corrosion, it also has the advantages of a good track record of construction, being economical and there being no need for large special equipment. The chipping and replacing method (3) is at the intermediate level in terms of durability, workability, cost and construction period. Furthermore, method (4), which employs patching repair and cathodic protection, has the advantages of high durability and good track record, although it has the disadvantages of long construction period, high construction cost of cathodic pro-

Table 2 Overview of typical jetty renovation method.

	Method (1): Pre-cast concrete form + cast-in-place concrete	Method (2): Patching repair	Method (3): Chipping and replacing	Method (4): Patching repair + cathodic protection
Overview of method	Remove degraded parts of beams and slabs. Erect pre-cast concrete form, and cast in place the normal concrete.	Remove beams and concrete up to and near reinforcements of slab, and replace with resin mortar and so on.	Remove degraded parts of beams and slabs, and cast in place the normal concrete.	Remove all degraded beams and floor slabs, repair sections using resin mortar, and provide cathodic protection.
Section figure
Overview of construction	(1) Remove degraded parts of old slabs and beams (2) Treatment and arrangement of reinforcements (slabs, beams) (3) Erect pre-cast concrete form (slabs) (4) Cast in-place the concrete (slab)	(1) Remove concrete near reinforcement (2) Treatment and arrangement of reinforcements (3) Plastering of resin mortar and so on	(1) Remove degraded parts of old slabs and beams (2) Treatment and arrangement of reinforcements (3) Cast in place the concrete	(1) Remove degraded parts of old slabs and beams (2) Treatment and arrangement of reinforcements (3) Cathodic protection (beams, slabs) (4) Place resin mortar (slabs)

tection, and high running costs. Although not mentioned here, there are many other renovation methods; each has its own advantages and disadvantages, but the fact remains that a rational renovation method has not been established yet.

3. Investigations related to renovation of existing jetties

3.1 Outlines of existing jetties

Various investigations were carried out, taking an existing jetty in service as the model case. **Table 3** gives an introduction to the jetty under study. Jetty A was constructed in 1968 and has been in service for about 40 years. During this period, the slabs in jetty A have been renovated twice. The degradation stage of jetty A corresponds to the "degradation stage" (JSCE 2001), and urgent repair was required.

3.2 Renovation guidelines

Various conditions were assigned from the using situation, environmental conditions, and so on, to renovate jetty A. These conditions and the status of construction drawings are shown in **Table 3**. Chloride had penetrated through the overall depth of the 250 mm thick slabs in jetty A with its content exceeding the critical chloride ion concentration initiating steel corrosion of 1.2 kg/m^3 (JSCE 2007), and even visual inspection confirmed corrosion of the reinforcement and occurrence of cracks. For this reason, merely carrying out patching repair and providing surface coatings would have been inadequate, and the entire concrete in the floor slabs had to be renewed. The underside of the floor slabs was exposed to a particularly severe chloride damage environment. Since the thickness of the floor slabs could not be increased for the given conditions, it was necessary to ensure durability for over 30 years by using a concrete cover of 62 mm. Therefore, pre-cast concrete forms providing excellent durability to the underside of slabs even in the presence of large chloride supply were used, as shown in **Table 3**. All reinforcements were renewed, and normal concrete was placed again.

3.3 Design investigations

(1) Structural design of jetty A

Design studies were carried out for the sectional view shown in **Table 3**. Design investigations and durability design were carried out, and the performance required from each kind of material to ensure strength to resist service loads, stresses, and crack width, and durability of more than 30 years, was clarified. Due to restrictions in transportation by road, the size of one pre-cast concrete form was taken as approximately 2.5 m or less. Since the slab width of jetty A is about 12 m, it was decided to use a total of 8 pre-cast concrete forms, as shown in the plan view in **Table 3**.

Studies on loads were carried out for the section in which reinforcements and steel joints were arranged (see

Table 3 Overview of jetty A.

	Jetty A	
Completed in	1968 (41 years of service)	
Objectives	Loading and unloading of products	
Shape	Length 117 m/ slab thickness 250 mm	
Loading conditions	Dead load + live load (Dead load: 24.5 kN/m ³ , Live load: 5 kN/m ²)	
Concrete cover	62 mm	
Renovation history	1981: Part renovation by affixing steel plates 1991: Corrosion protection for reinforcements, patching repair, surface cover	
Condition
Overview of degradation	Reinforcement corrosion due to chloride induced deterioration and macro-cells (Critical chloride ion concentration exceeded in entire slab)	
Degradation stage	Degradation period	
Conditions related to renovation	Item	Contents
	Shape	No change in shape
	Loading equipment: Small	Can be chipped from the upper surface of slab
	Service life after renovation	30 years
	Cathodic protection	Cannot be used from the service conditions
	Environment	Chloride concentration on the lower surface of slab: 23 kg/m ³
	Crack	To be controlled to values below the crack width limit
	Surface appearance	Streaks of rust on the surface to be inhibited
Sectional View	[Before renovation]

Sectional View	[After renovation]

Plan figure	[After renovation]	
	Lower surface

Table 3 sectional view). The stress resultant generated by dead load and live load was calculated by 3-D FEM analysis taking the slabs of the jetty structure as shell elements. The model was as shown in **Fig. 2**; the boundary conditions were assumed as all four sides being fully fixed. A part of the pre-cast concrete form was assumed to be with concrete cover. The concrete in the entire section was taken as normal concrete. The elements to be studied were taken as Element 1 at the center of the slab and Element 2 located at the mid-point of the longer side. The loading conditions for design were taken as dead load of $W = 24.5 \text{ kN/m}^3$ and live load of $P = 5 \text{ kN/m}^2$. The results of the stress resultant are shown in **Table 4**. Moreover, the crack width was estimated by performing RC calculations from the stress resultant and calculating the stress in the reinforcement from the crack width equation (1) (JSCE 2002) and the section figure after renovation of **Table 3**.

$$w = 1.1k_1k_2k_3\{4c + 0.7(c_s - \phi)\}\left(\frac{\sigma_{se}}{E_s} + \varepsilon'_{csd}\right) \quad (1)$$

Here, $k_1 = 1.1$ (combination of modified reinforcements and flat bars), $k_2 = 1.041$, $k_3 = 0.909$, C : Concrete cover on steel ($= 62 \text{ mm}$); C_s : Spacing of steel members ($= 500 \text{ mm}$); ϕ : Diameter of steel member ($= 16 \text{ mm}$); σ_{se} : Increase in stress intensity of steel member; C : Young's modulus of steel ($= 200 \text{ kN/mm}^2$), ε'_{csd} : Effects of shrinkage and drying shrinkage of concrete are not considered.

Table 5 shows the calculated results of crack width. The crack width considered to have occurred at the center of the slab due to dead load and live load is 0.07 mm . This value is smaller than the allowable crack width of 0.105 mm ($0.0035c$; where c was taken as the minimum concrete cover on steel member of 30 mm) defined in the specifications (JSCE 2007), and was confirmed to satisfy the properties. On the other hand, the crack width occurring on the upper surface of the connection between beam and slab was assumed as 0.22 mm . Thus, it was decided to coat epoxy adhesive on the boundary surface of the beam, and water repellent agent on the upper surface.

(2) Durability design of jetty A

An item that specially needs verification related to durability of jetty A is durability against chloride induced deterioration. Verification of the lower surface exposed to severe environment was carried out in accordance with the Standard Specifications for Concrete Structures (JSCE 2007), and the concrete quality and required properties were determined. **Table 6** shows the verification results and the conditions used in the verification. The basis for verification of the lower surface was that the durability was to be satisfied only in the range of 30 mm corresponding to the pre-cast concrete form so that the chloride content becomes less than the critical chloride (1.2 kg/m^3) for initiation of steel corrosion

Fig. 2 Model of slab for 3-D FEM analysis.

Table 4 Results of analysis of stress resultant.

Load	Value	Element 1 lower surface		Element 2 upper surface	
		x	y	x	y
Dead load	24.5 kN/m ³	3.16	3.84	-1.02	-6.01
Live load	5 kN/m ²	2.58	3.13	-0.84	-4.91
Total stress resultant		5.74	6.97	-1.86	-10.92

(unit:kN)

Table 5 Calculated results of crack width.

	Load	Element 1 lower surface		Element 2 upper surface	
		x	y	x	y
Crack width (mm)	Dead load	0.04	0.04	0.02	0.12
	Live load	0.03	0.03	0.02	0.10
	Total	0.07	0.07	0.04	0.22

Table 6 Verification of penetration of chloride ion.

	Lower surface
Surface chloride ion concentration (kg/m ³)	23.0 (actually measured)
Concrete cover (mm)	30
Target service life (years)	30
Maximum effective diffusion coefficient required in concrete (cm ² /year)	0.04
W/C of concrete*	Less than 16%

*: Ordinary Portland cement assumed as the type of cement

during the target service life (30 years). Here, the surface chloride ion concentration is estimated from the measurement results of distribution of chloride ions concentration in drilled cores sampled from concrete slabs. As shown in the results of calculation of chloride content in **Table 6**, the effective diffusion coefficient required in concrete is $0.04 \text{ cm}^2/\text{year}$ or less. It was found that if this is calculated in accordance with the Standard Specifications for Concrete Structures (JSCE 2007), it would be necessary to take a W/C ratio of concrete of less than 16%. That is, such values cannot be achieved using

normal concrete; special concrete needs to be used.

4. Studies related to properties of pre-cast concrete form

4.1 Physical properties and durability of pre-cast concrete form

(1) Physical properties of concrete

The pre-cast concrete forms with excellent durability used in the study make use of special admixture and carbon fiber (hereafter called CF) as the reinforcing material; they are pre-cast concrete forms (Watanabe *et al.* 2006) made of new concrete with durability enhanced because of carbonation curing.

The basic condition for carbonation curing (Watanabe *et al.* 2006) was taken as a total of 28 days with wet air curing for 1 day after placement, standard water curing for 1 day after removal of the formwork, and later, carbonation curing for 26 days at the temperature of 50°C, humidity of 60% and carbon dioxide gas concentration of 20%. **Table 7** shows the mix proportion of concrete for high durability concrete (hereafter referred to as "HDC") for pre-cast concrete forms after carbonation curing, while the properties are listed in **Table 8**. Compared to the normal concrete (hereafter referred to as "NC") used in normal pre-cast concrete forms, HDC demonstrates properties such as approximately 2.3 times the compressive strength, and about two times the tensile strength of NC.

(2) Resistivity of chloride attack

One of the particularly important properties in the renovation of jetties is the resistivity of chloride attack, and the effective diffusion coefficient of chloride ion related to this property was obtained by electrophoresis (JSCE-G-571-2003).

The ideal cement type of HDC was selected taking carbonation curing for HDC. **Table 9** lists the properties of various cements, and **Table 10** lists the test results. The results reveal that the resistivity of chloride permeation of HDC-LPC is satisfactory; and that the ratio of effective diffusion coefficient of chloride ion to NC is extremely small at about 1/50. The diffusion coefficient became smaller in the sequence of LPC < HPC < OPC in HDC because when LPC composite with large belite content is cured with CO₂ gas, vaterite of small density is mainly generated. On the other hand, when HPC or OPC composite with large alite content is cured with CO₂ gas, calcite of large density is mainly generated; therefore, the

gap filling effect due to carbonation is high in the case of LPC (Sakai *et al.* 1999).

(3) Bending toughness of concrete form

When the application of pre-cast concrete form is specified for the slabs of a jetty structure, behavior against bending is required. Here, flexural loading tests were carried out to study the crack width generated and the flexural capacity of the panel unit. Furthermore, the effects of the volume fraction of carbon fiber, which is the reinforcing material, on the flexural capacity, were also studied.

Four cases were taken for fiber volume between 0.75 and 2.0 vol.% using chopped fibers of diameter 1.0 mm and fiber lengths of 20 mm in the tests. The dimensions of the test specimen were taken as 740 mm (length), 150 mm (width), 30 mm (thickness), as shown in **Fig. 3**. Long fibers of fiber diameter 2.0 mm were placed at a

Table 7 Concrete mix proportion.

Mix proportion type	Cement	Water-binder ratio (%)	Fine aggregate ratio (%)	Unit weight (kg/m ³)					
				W	C	B*	S	G	CF
NC	OPC	43	42.6	168	391	—	749	1009	—
HDC	—	23	46.1	150	—	650	745	871	24.5

*B: Binder

Table 8 Properties of HDC.

Test item	Unit	Values		Ratio HDC/NC	Test methods
		HDC	NC		
Compressive strength	N/mm ²	130	57	2.3	JIS A 1108
Young's modulus		4.0×10 ⁴	3.4×10 ⁴	1.2	JIS A 1149
Tensile strength		8.0	4.4	1.8	JIS A 1113
Flexural strength		11.5	6.4	1.8	JIS A 1106
Bending toughness coefficient		9.0	0.1	90	JCI- SFR2
Porosity	%	6.1	8.6	0.71	Mercury permeation method
Change in concrete length	μ	83	694	0.12	JIS A 1129-1993

Table 9 Properties of cement.

Notation	Cement	Density (g/cm³)	Chemical composition (mass %)							
			CaO	SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	MgO	SO ₃	Na ₂ O	K ₂ O
OPC	Ordinary Portland cement	3.16	64.2	21.1	4.8	2.8	1.6	1.7	0.3	0.5
HPC	High-early-strength Portland cement	3.13	65.3	21.2	4.5	2.8	1.4	2.9	0.3	0.4
LPC	Low-heat Portland cement	3.22	63.0	26.1	2.8	2.7	0.9	2.1	0.3	0.2

Table 10 Results of resistivity of chloride permeation tests.

Mix proportion type	Type of cement	W/B (%)	Effective diffusion coefficient of chloride ion (cm^2/year)	Verification for required property of $0.04 \text{ cm}^2/\text{year}$
HDC	LPC	23	0.012	O.K.
	OPC		0.742	N.G.
	HPC		0.079	N.G.
NC	OPC	43	0.782	N.G.

pitch of 60 mm at the concrete cover position of 10 mm from the tension side. Concrete was mixed and placed, and within one day after form removal, the test specimens were cured in water at 20°C for one day. Subsequently, the test specimens were cured to an age of 14 days in the concrete curing system with CO₂ gas at the temperature of 50°C, RH of 60% and CO₂ concentration of 20%. For loading tests, the equal-span loading method was used as shown in Fig. 4. The distance between support points was 540 mm. The equivalent bending moment span was taken as one-third the distance between support points. The loading speed up to the maximum load was taken as 0.2 mm/min, and the rate until the deflection at the center of the panel became 25 mm was taken as 0.5 mm/min., and subsequently, the loading rate was taken as 1.0 mm/min. until failure. The items measured were deflection, load, cracking load, crack width and displacement.

Figure 5 shows the relationship between deflection and load at various chopped fiber volumes. The load increased with increases in displacement even at the smallest chopped fiber volume (0.75 vol.%). Also, as the volume fraction of chopped fibers increased, the load also increased. Here, the sum of the product of load and displacement is defined as load-carrying energy (unit: kNmm), and its value is shown by numerals within brackets in Fig. 5. The load-carrying energy increased with increases in the chopped fiber volume; however, even when it was increased beyond 1.5 vol.%, that is to 2.0 vol.%, the load-carrying energy did not increase.

Figure 6 shows the relationship between chopped fiber volume, cracking load, and crack width. With increases in the chopped fiber volume, the cracking load increased very slightly. When the fiber volume increased beyond 1.0 vol.%, the width of the generated cracks became smaller; when it increased to 1.5 vol.%, it reduced significantly to a small value. This is attributed to the increase in the flexural strength and bending toughness coefficient of the base concrete accompanying the increase in the volume fraction of chopped fibers on the mutually reinforcing effects of long carbon fibers.

The carbon chopped fiber volume of pre-cast concrete form required to obtain the target allowable crack width of 0.105 mm was determined as 2.0 vol.%.

Fig. 3 Shape of bending test specimen.

Fig. 4 Loading conditions and position of measuring jig.

Fig. 5 Load-displacement relationship.

Fig. 6 Relationship between fiber volume and load, crack width.

4.2 Load-carrying properties of composite beam using pre-cast concrete form

Loading tests were conducted using the test specimen shown in Fig. 7 to verify that the strength after renovation was equivalent to the strength before renovation, that there is no noticeable peeling-off of the pre-cast concrete form, and that the construction joint did not

Fig. 7 Loading test specimen (jetty A).

Table 11 Mix proportion and properties of concrete (jetty A).

Cement	Water-binder ratio (%)	Fine aggregate ratio (%)	Unit weight (kg/m ³)				Strength during test (N/mm ²)		
			W	B	S	G	Com.	Tens.	Bend.
HPC	40	41.6	168	420	736	1011	57.0	3.8	5.2

become a structural flaw for the section after renovation. The test specimen used was a reinforced concrete test specimen arranged on the tension side of the pre-cast concrete form. Two pre-cast concrete forms of 1500 mm length, 200 mm width and 30 mm thickness were used. A joint of 7 mm width that was determined considering the condition of the construction on site, was placed at the connection in the central part of the test specimen and was filled with elastic epoxy resin. Concrete using high-early-strength Portland cement shown with the mix proportion in **Table 11** was used as the post-cast concrete, and SD345 was used as the reinforcement. The test specimen was sealed cured at room temperature, and the test age was taken as 14 days. At this stage, the maximum moment was calculated when the dead load and live load were applied on the slab, as shown in **Table 3**, and the crack width for the same bending moment was verified from the loading tests. Tests with four point loading method were conducted; the spacing between the supports was taken as 2600 mm and the equivalent bending moment zone was taken as 800 mm. During loading tests, measurements of beam deflection and joint opening of the pre-cast concrete form and crack observations were performed. Measurements and crack observations were performed at 5-kN intervals up to crack generating load, and at 50-kN intervals up to yield load. Unloading was performed at appropriate times, and the status of occurrence of crack was observed. Furthermore, to verify the integration of the pre-cast concrete form and post-cast concrete, the displacement of the construction joint was measured.

Figure 8 shows the results of measurement of load and displacement of the test specimen (pre-cast concrete form on the tension side). Firstly, a design load of 22 kN was applied and it was verified that reduced deflection occurred. This is probably because the bending strength increased due to the arrangement of high strength

Fig. 8 Loading test results (jetty A).

Fig. 9 Joint opening amount of concrete pre-cast form (jetty A).

pre-cast concrete form on the tension side.

Figure 9 shows the behavior of the joints between the pre-cast concrete forms arranged on the tension side. When the assumed load was applied on jetty A, the result was that the joint opened by about 0.12 mm. For this reason, it was concluded that a material with ductility greater than 2% should be used for a joint of 7 mm.

Figure 10 shows the crack pattern at yield of reinforcement, while **Table 12** lists the crack width measurement results for various loading conditions. The crack width generated at the assumed load for jetty A was less than 0.04 mm approximately, verifying that this width did not cause any problems for ensuring durability.

5 Verification tests

5.1 Overview of test construction

Test construction was implemented with the aim of evaluating workability and extracting further issues in the renovation method shown in **Table 3**. Eight pre-cast concrete forms of size approximately 11 m × 5 m and construction area of 55 m², and approximately 2.7 m × 2.5 m size were used for the front end part in jetty A.

Fig. 10 Crack pattern (jetty A).

Table 12 Results of measurement of crack width (jetty A).

Load (kN)	Crack number and width (mm)								
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
30	0.04	0.04	0.04	0.04	—	—	—	—	—
40	0.04	0.08	0.04	0.04	0.04	0.08	0.04	0.04	0.04
50	0.04	0.10	0.08	0.06	0.04	0.10	0.04	0.04	0.04

—: Indicates no crack

5.2 Test construction

Figure 11 shows the renovation flow chart. Firstly, beams were renovated by the common patching repair method. For floor slabs, firstly a plywood seat was erected as shown in Fig. 12, and the pre-cast concrete form was erected on top of it. Small sacrificial anodes were used at the ends of the renovated parts with the aim of preventing macro-cell corrosion. Furthermore, the pre-cast concrete form was transported using four ceramic inserts. Finally, these inserts were used as dowels, as shown in Fig. 13, and the construction was completed without problems. Since the working time and production rate, etc., are related to other repair work, their estimation is not easy. However, the typical work corresponding to a construction area of 55 m² from "4. Erect plywood seat" to "7. Place studs and reinforcements" in the flow chart shown in Fig. 11 was completed within 4 days. Issues such as the necessity of processing the pre-cast concrete form in the shape of the existing jetty, arrangement of pre-cast concrete form according to existing reinforcements, the time required for positioning and so on, were raised.

5.3 Measured chloride content

Figure 14 shows the results of measurement of chloride content after 3.5 years, taken from studies on general parts constructed using high-early-strength Portland cement concrete. While chloride content of about 4.0 kg/m³ was confirmed near the surface in general parts, practically no chloride had permeated the pre-cast concrete form. The high resistance of pre-cast concrete form to chloride permeability was thus verified.

5.4 Calculation of life cycle cost (LCC)

For jetty A, a renovation method applicable to the environment of each jetty was selected, and the LCC for each method was calculated with emphasis on slabs. Fig. 15 shows the results. For the LCC of jetty A, the initial renovation cost of the renovation method using pre-cast

Fig. 11 Renovation flow chart for jetty A.

Fig. 12 Erection of pre-cast concrete form (jetty A).

concrete form was counted as "1" and the ratio of renovation cost required up to N years after the renovation was defined as the life cycle cost point (hereafter referred to as "LCC pt."). The legend "PR" in the figure refers to

Fig. 13 Reinforcement and stud dowels (jetty A).

Fig. 14 Chloride content 3.5 years after renovation.

Fig. 15 Results of evaluation of LCC for jetty A.

"patching repair and surface protection" in general renovation method; "CR" refers to the method of chipping away the concrete, replacing new concrete and surface protection, while "PR+CP" refers to the method of cathodic protection in addition to patching repairs, "NR" refers to the new renovation method using pre-cast concrete form. Furthermore, re-renewal of "PR" and "CR," were assumed to be carried out once in 20 years, and only surface protection were renovated once in 10 years. The frequency of re-repair was determined based

on experience. Re-repairs of "CP" (cathodic protection) were assumed to be implemented once in 40 years. Regarding the renovation method "NR", it was decided to renovate at the frequency of once in 40 years, slightly exceeding the critical chloride ion concentration initiating corrosion in jetty A.

5.5 Result of test construction

Test construction was carried out and satisfactory workability of the renovation method using pre-cast concrete forms was verified. Moreover, high chloride shielding property was confirmed after 3.5 years. Henceforth, considering application to large sections and enhanced flexibility for existing structures, rationalized erection and arrangements will be pursued, so that a renovation method with more significant advantages may be realized.

6. Conclusions

The authors have carried out various studies as part of the present research to establish a jetty renovation method using pre-cast concrete form with high durability aiming to reduce the life cycle cost. The results of the studies clarified the following:

- (1) Cracks can be inhibited effectively by using long fibers and chopped fibers of carbon as the renovation material in slender pre-cast concrete forms with excellent durability.
- (2) For existing slabs of jetty structure, a renovation method using pre-cast concrete form was suggested. The results of verifying structural properties through analyses and tests showed that cracks that led to durability problems during service did not occur, and high durability was confirmed.
- (3) The results of conducting verification tests by test construction showed that the initial renovation cost incurred was rather higher than the conventional renovation method, and a rational jetty slab renovation method was developed that could be used to minimize life cycle cost.

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