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Masanari Tominaga, Toyokazu Sakaki, Masahiro Idshida, Yoshimitsu Hosoya

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Masanari TOMINAGA** Toyokazu SAKAKI** Masahiro ISHIDA**
Yoshimitsu HOSOYA**

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1 Introduction

The Government of the Republic of the Philippines is pursuing the development of the Leyte Industrial Estate on the west coast of Leyte Island, as part of its industrialization policy. A copper smelter of 130 000 t annual capacity and a fertilizer plant of 900 000 t annual capacity are being built in this industrial area. Kawasaki Steel Corporation was awarded a turn key contract by the National Development Company (NDC) of the Philippines covering the site survey, design and supervision of construction of the port facilities exclusively for this industrial area. The complete project was executed between December 1981 and May 1984.

The Leyte Industrial Estate, as shown in **Fig. 1**, is located in the Isabel area on the west coast of Leyte Island. The Philippine Government designated the Isabel area as an industrial complex for the following three reasons:

(1) Ample power will be supplied at a lower cost from a geothermal power plant being built about 60 km east of the Isabel Area.
(2) Dupon Bay which is located to the west of this area is deep enough to provide a natural good harbor, and the port facilities can be constructed readily.
(3) The construction of the industrial complex will promote employment and provide the impetus for the development of the entire island including infrastructure.

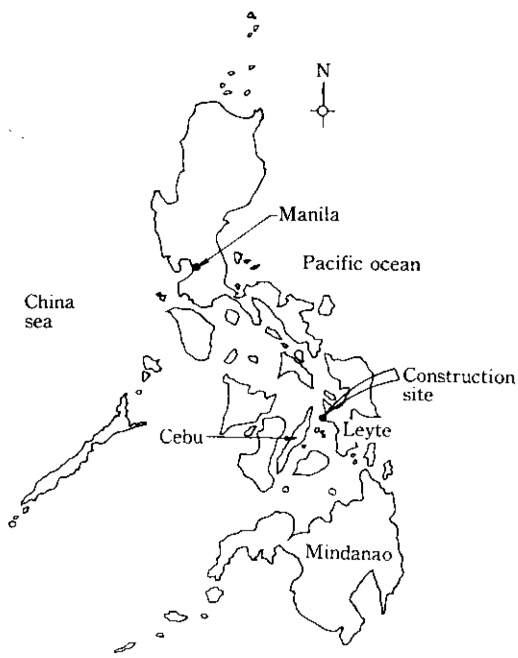

Fig. 1 Project site location

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** Engineering Division

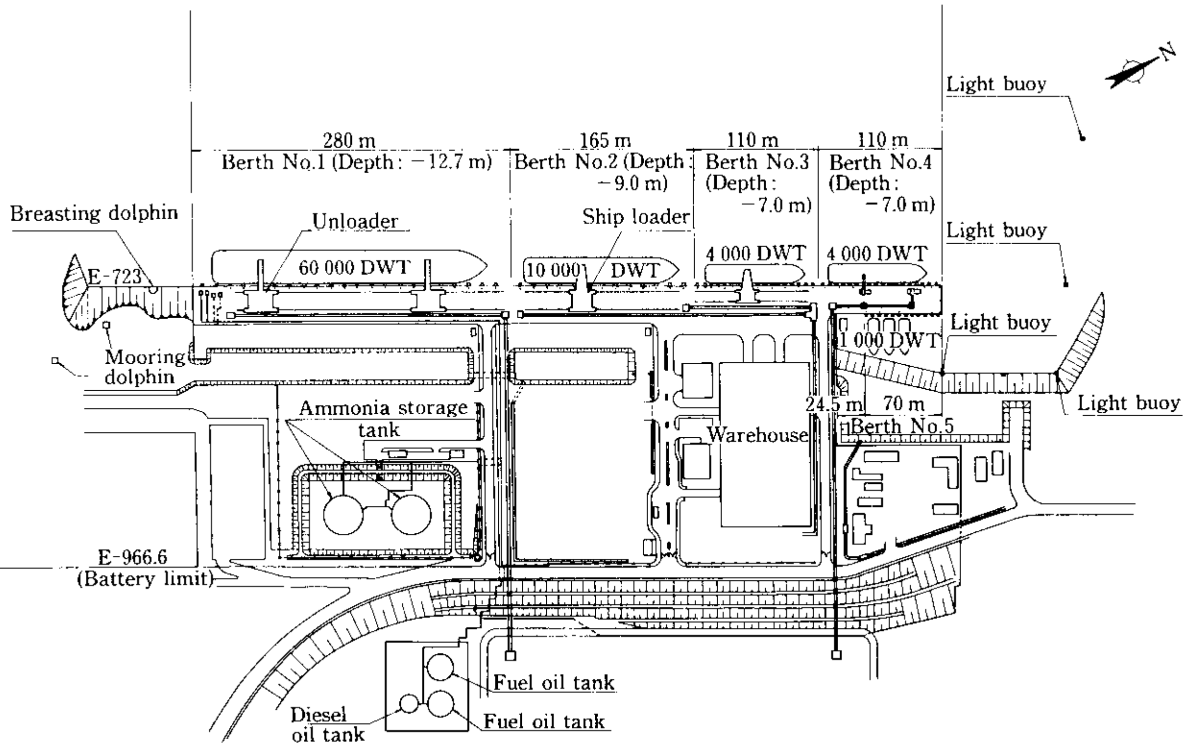


Fig. 2 General layout of port complex

Table 1 Summary of port facility

	Dimensions of berth (vessel to be accommodated)	Loading/unloading facility
Berth No. 1	Length: 280 m Depth: EL−12.7 m (max 60 000 DWT)	Bulk unloader(880t phosphate rock per hour×2 units) and belt conveyor, loading arms and pipelines for sulfuric acid, phosphoric acid, ammonia etc.
Berth No. 2	Length: 165 m Depth: EL−9.0 m (max 10 000 DWT)	Bulk/bagged fertilizer loader (300 t/2 000 bags per hour) and belt conveyor
Berth No. 3	Length: 110 m Depth: EL−7.0 m (max 4 000 DWT)	Bagged fertilizer loader (2 000 bags per hour) and belt conveyor
Berth No. 4	Length: 110 m Depth: EL−7.0 m (max 4 000 DWT)	40 t mobile cranes (2 units) and belt conveyor
Berth No. 5	Length: 70 m Depth: EL−4.0 m (max 1 000 DWT)	

Since the hinterland of the port facility includes a copper smelter and a fertilizer plant, a wide variety of materials from phosphate rock to cathode copper must be handled. The layout of the port complex as shown in **Fig. 2** was designed on the basis of the type of ships and the capacity of the materials handling system to accommodate these industrial facilities. The dimensions of the wharf and an outline of the loading/unloading system are summarized in **Table 1**.

The present report concerns planning, surveying, design and execution of construction for the port structures included in this port facility, as well as the characteristics of the wharf structures.

2 Wharf Structures and Site Selection

The owner of this project, NDC, originally designed the port facility on the basis of the wharf structure being supported on reinforced concrete caissons. This structure was adopted because the subsoil was compact enough with *N* value (result of standard penetration test) greater than 100, so that ordinary pile driving methods were not considered applicable. Based on KSC's work experience[1)] in the Philippines, the compact soil layer rarely exists at sandbank of such an island. In addition, the standard penetration test conducted by a local soil survey often gives a higher value than the actual one. Since inaccurate information about the soil density often results in difficulties during construction, KSC decided to conduct a preliminary soil investigation at this stage. Consequently, it was confirmed that the soil consisted mainly of sandy soil of *N* value 20-40, as anticipated, and considered suitable for wharf construction using interlocked steel pipe piles and steel sheet piles.

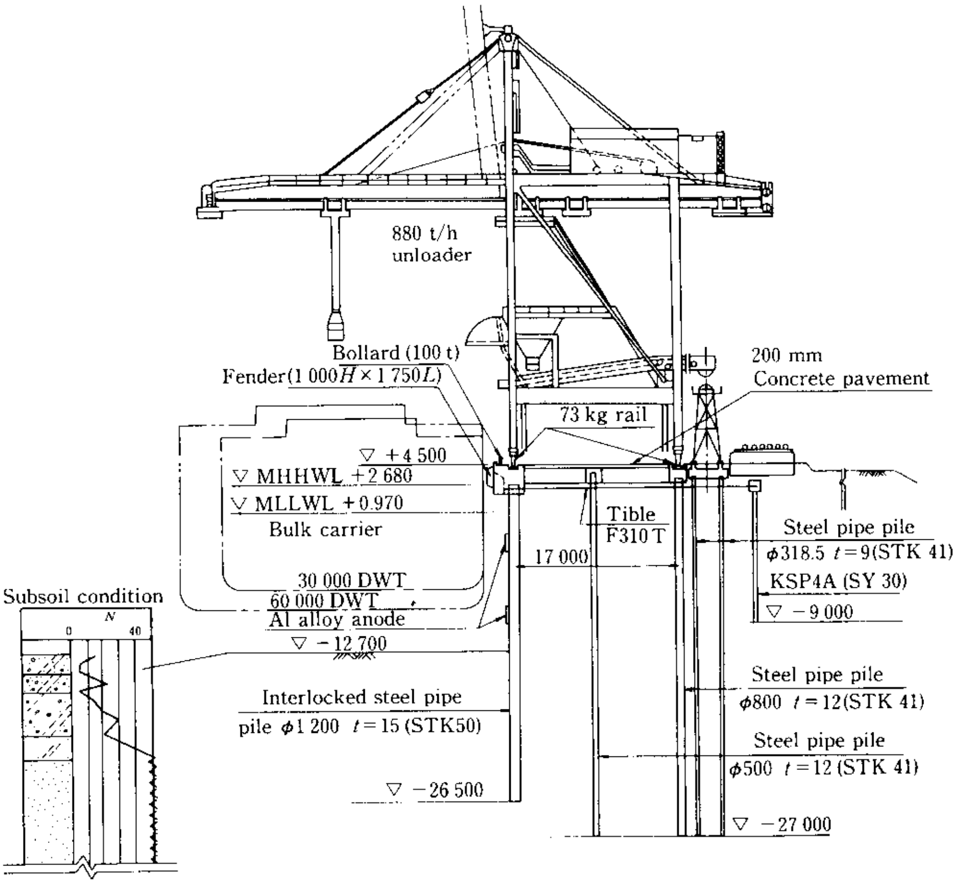


Fig. 3 Typical cross section of wharf structure supported on interlocked steel pipe piles

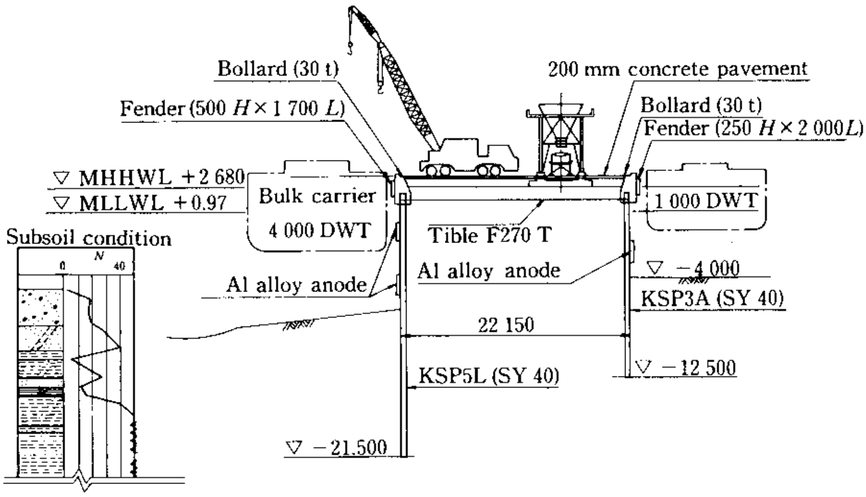


Fig. 4 Typical cross section of sheet pile double wall structure

Figure 3 shows a typical cross-section of the wharf (Berth No. 1) using interlocked steel pipe piles, as proposed by KSC, where interlocked steel pipe piles are used not only as retaining structure but also act as foundation piles to support the unloader. Of the total 665 m of wharf length, 95 m (Berth Nos. 4 and 5) were constructed using steel sheet pile double wall structure (**Fig. 4**). As indicated, the steel sheet pile double structure was designed wide enough with wall breadth-to-wall height ratio as large as $B/H = 22.15\ \text{m}/11.5\ \text{m} \doteqdot 2$, in order to insure stability similar to that of a steel sheet pile cellular cofferdam structure.

High quality sandy soil required for backfill behind the wharf was not available at the construction site except by dredging seabottom soils in front of the port complex. The hydrographic survey revealed that the bottom topography was relatively complex, and in order to obtain an adequate amount of dredged sand, the wharf site was moved 20 m toward the land from the location originally planned by NDC.

This modified plan proposed by KSC was considered to be both technically and economically sound, and was therefore adopted. It is significant to note that information on the actual soil conditions permitted the wharf structures to be changed from reinforced concrete caisson type to the interlocked steel pipe pile and steel sheet pile system, and the merits of the latter structure were justified economically, structurally and considering the most suitable construction method.

3 Natural Conditions

3.1 Weather and Marine Meteorology

The air temperature at the construction site varies from a minimum of 22°C to a maximum of 35°C, the annual mean being 28°C. The annual mean relative humidity is as high as 85%, and, from July to December the relative humidity increases to 90% or more. **Figure 5** shows the mean monthly rainfall observed at the copper smelter site. The annual rainfall in this area amounts to about 2 300 mm, and more than half of it is recorded in the rainy season from July to December. Leyte Island located at 11°N is affected somewhat by typhoons from January to March[2], however the construction site is located on the west coast of the island which is protected from the direct influence of a typhoon by the central mountain range running from north to south.

At the construction site, winds from north-north-west, south-south-west and north-north-east prevail. The south-south-west wind which occurs frequently from March to August causes wave action at the construction site location along the east coast of Cebu Island (See Fig. 1). The construction site is therefore often affected by waves during this time period.

The observation of water level in this area reveals that the mean lower low water level is EL + 0.970 m and the mean higher high water level is EL + 2.680 m. A tidal current of 0.3–0.6 knots is not considered strong enough to affect the construction work and operation of vessels.

3.2 Topography

As shown in **Fig. 6**, the construction site is located on the west side of the Santa Cruz Peninsula between Dupon Bay and Matlang Bay. The Peninsula is about 1.6 km wide, and a coastal terrace runs in a south-to-north direction. The shore line at the construction site is formed by coral reefs 50–100 m wide, and beyond that the bottom slopes steeply on about 1/8 gradient. At the south and north ends of the construction area, pockets of soft soil were encountered perpendicular to the shore line where sedimentary clay exists.

3.3 Soil Conditions

Since the topography of the construction site was relatively complex and the sea bottom was quite irregular, it was anticipated that the soil conditions would be rather complex. Since assessing the soil conditions at the con-

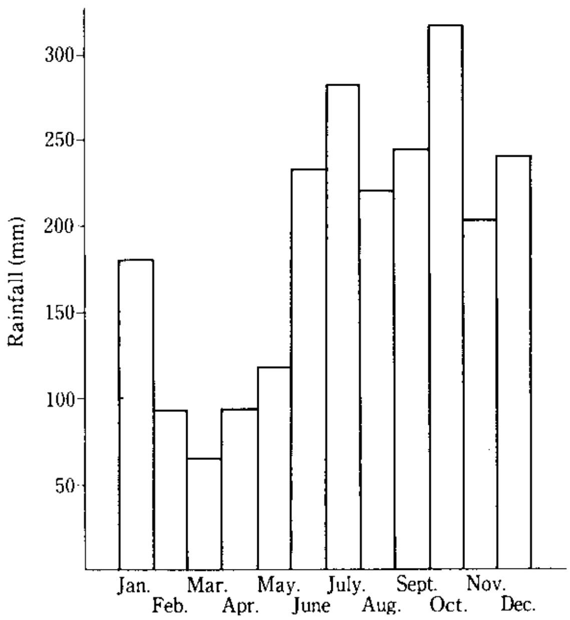


Fig. 5 Mean monthly rainfall (from 1981 to 1982)

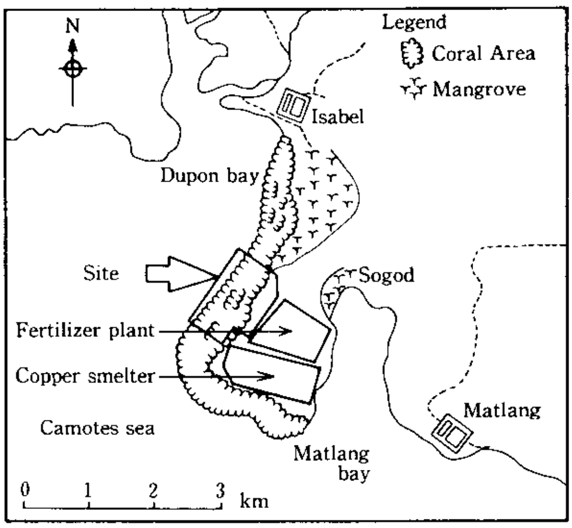


Fig. 6 Location of construction site

struction site as accurately as possible during the early stages of planning and design is one of the most important factors for executing this project as planned, offshore borings were made at 29 locations (total length 625 m) and land borings were made at 5 locations (total length 75 m). **Figure** 7 shows a typical soil profile based on the results of the soil borings. At the site of the wharf construction, a Tertiary layer deposit was encountered at a level below EL − 20 m. The soil was found to be sandy silt with a higher *N* value, considered adequate for a bearing layer for the pile foundation. The Diluvial layer consisting of sandy silt included extensive sand, and was deposited above the Tertiary layer in 3-5 m thickness. An 8-12 m thick Alluvial layer containing a large amount of coral was encountered above the Tertiary layer.

4 Project Schedule

Figure 8 shows the work schedule for the project. The survey and soil investigations at the site were begun in December 1981 and completed in March 1982. Design was completed in the time period between January and July 1982. Procurement and manufacture of steel materials to be imported from Japan commenced in February 1982. The construction of temporary facilities including field offices and living quarters was begun in March 1982, together with the dredging. Since there was no wharf available to accommodate a cargo ship near the construction site, the construction materials imported from Japan were transshipped using flat barges from Mindanao Island before being transported to the construction site. The first load of construction materials arrived at the site in May 1982, and pile driving was begun immediately. In September 1982, a portion of the wharf (Berth Nos. 4 and 5) became available, and subsequently it was possible to unload the materials shipped from Japan directly at the site, and delivery of the interlocked steel pipe piles was begun. The overall project work took 27 months, and the civil works for the present construction were completed in February 1984. **Photos 1** and **2** show the port facilities under construction and after completion, respectively.

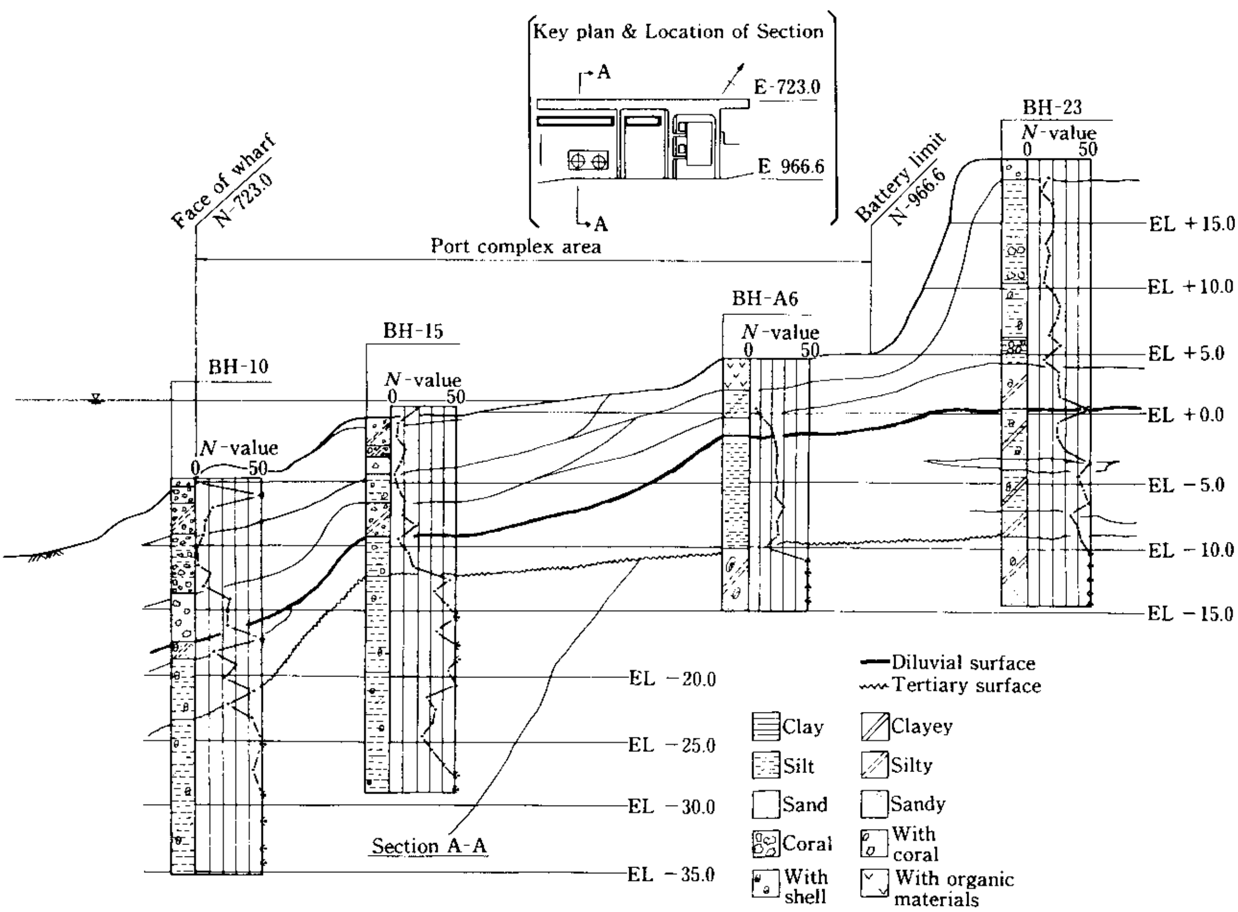


Fig. 7 Typical soil profile

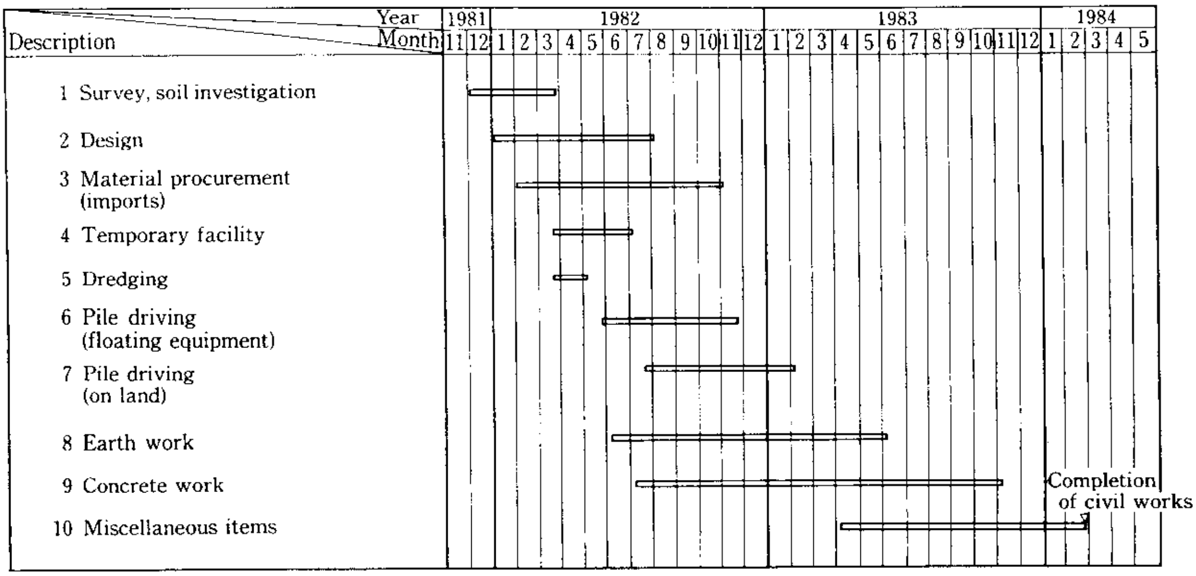


Fig. 8 Project schedule

5 Site Work

5.1 Outline of Site Work

The site work included a number of items, such as dredging, reclamation, wharf and shore protection, access roads and drainage system, foundations for material-handling equipment, cathodic protection system, navigation aids and other ancillary equipment. The quantities required for the major items of wharf construction, i.e. dredging, reclamation and pile driving, are given in **Table 2**.

5.2 Dredging and Reclamation

Dredging was executed using a 4 000 hp cutter suction dredger. The purpose of dredging was to excavate the required draft at the sea side of the wharf and basin, and to obtain the selected sand required for backfill behind the wharf. The quantity of soil to be dredged was 290 000 m^3, of which 180 000 m^3 of select sand was stockpiled on shore for the land reclamation and the remaining silty soil was disposed of off-shore.

The density of the coral reef existing in the dredged area was N-value = 30 maximum, and it could be dredged satisfactorily using a 4 000 hp cutter suction dredger. The cutter suction dredger operated normally 16 h/day, and could dredge an average volume of 250 m^3/h.

Figure 9 shows the grain size distribution of the fill materials. Mixed soil in the figure refers to the mixture of dredged sand with excavated material, with 50% or higher sand content of grain size greater than 74 μm. Three types of fill materials were selected for use depending upon the design features and type of structures to be constructed. The total amount of fill materials was 330 000 m^3, consisting of 150 000 m^3 dredged

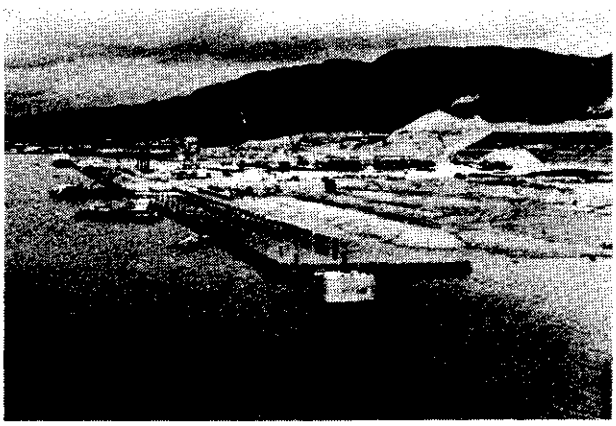


Photo 1 Wharf structure under construction

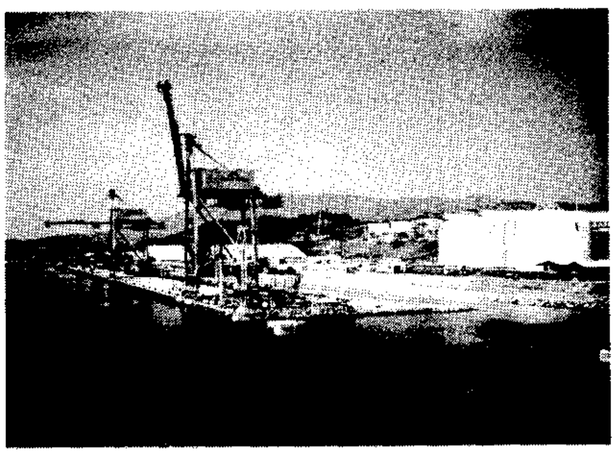


Photo 2 Completed port facility

Table 2 Quantities of major items of wharf construction

Location	Description	Quantity	Remarks
Berth No. 1	Interlocked steel pipe pile Steel pipe pile Steel sheet pile Concrete	210 pcs 116 pcs 893 pcs 6 470 m^3	ϕ1 200, t14, L=23.0-33.0 m ϕ318.5, ϕ500, ϕ800; L=23.5-30.0 m KSP II, KSP VA, KSP VL; L=5.0-19.0 m
Berth No. 2	Interlocked steel pipe pile Steel pipe pile Steel sheet pile Concrete	151 pcs 106 pcs 415 pcs 3 510 m^3	ϕ800, ϕ1 200; L=21.0-23.0 m ϕ318.5, ϕ500, ϕ600; L=22.0-23.0 m KSP IIIA, KSP IVA; L=11.0-11.5 m
Berth No. 3	Interlocked steel pipe pile Steel pipe pile Steel sheet pile Concrete	19 pcs 75 pcs 459 pcs 2 630 m^3	ϕ800, L=21.0 m ϕ318.5, ϕ500, ϕ600; L=22.0-25.0 m KSP VL, KSP IIIA; L=11.5-23.5 m
Berth No. 4 & No. 5	Steel pipe pile Steel sheet pile Concrete	2 pcs 919 pcs 2 000 m^3	ϕ318.5, L=25.0 m KSP IIA, KSP IIIA, KSP VL; L=10.5-23.5 m
Breasting dolphin	Steel pipe pile Concrete	8 pcs 100 m^3	ϕ800, L=39.0-40.5 m
	Dredging Earth work	290 000 m^3 330 000m^3	

Note; Quantities for steel pipe piles and steel sheet piles include both marine installation and those driven on land.

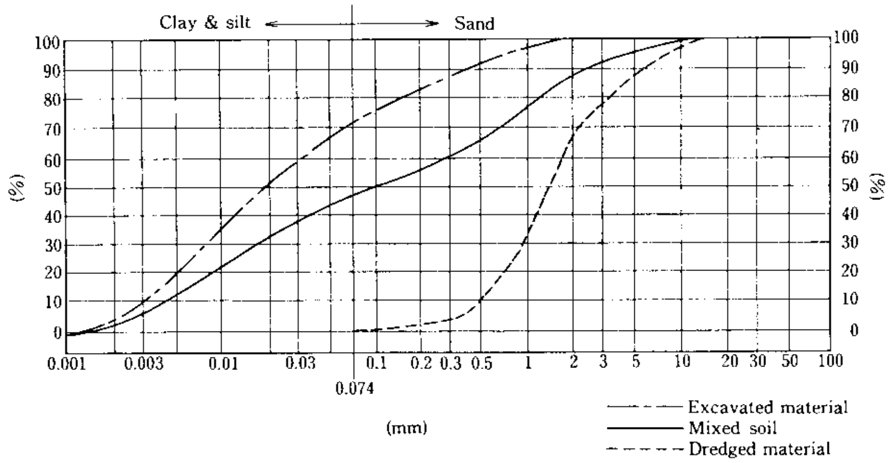


Fig. 9 Grain size distribution of fill materials

sand, 120 000 m^3 mixed soil and 60 000 m^3 excavated material.

5.3 Pile Driving Work (Marine Installation)

One of the major tasks of the construction program was to execute the pile driving (marine installation) for the interlocked steel pipe piles, steel sheet piles and steel pipe piles, which constituted the principal wharf structures, accurately and as scheduled. **Photo 3** shows the pile driving operation (marine installation), using both MB-40 and MB-70 diesel hammers depending upon the subsoil conditions and the pile size. **Table 3** shows the operation records of floating pile driving equipment.

The accuracy of pile driving (plan position) was required to be within ±10 cm. Before driving piles, a survey stage was built on an extended line normal to the wharf, and the pile position was determined using intersecting lines from the survey stage and a land station, to guide the pile driving barge to the proper location.

For the interlocked steel pipe piles, driving was under-

Table 3 Operation records of floating pile driving equipment

	Interlocked steel pipe piles	Steel pipe piles	Steel sheet piles
1. Dimensions of piles	1 200 $\phi \times 15t$ 800 $\phi \times 13t$	500 $\phi \times 12t$ 600 $\phi \times 12t$ 800 $\phi \times 12t$	KSPVL
2. Length of piles	L=21.0-33.0 m	L=22.0-40.5 m	L=21.5-23.5 m
3. Quantity of piles	380 pcs	145 pcs	897 pcs
4. Calender days	71 d	21 d	63 d
5. Operational days	45 d	14 d	33 d
6. Non-operational days (Due to bad weather) (Due to other causes)	26 d (21 d) (5 d)	7 d (4 d) (3 d)	30 d (11 d) (19 d)
7. No. of piles driven per calender day	5.4 pcs/d	6.9 pcs/d	14.2 pcs/d
8. No. of piles driven per operational day	8.4 pcs/d	10.4 pcs/d	27.2 pcs/d

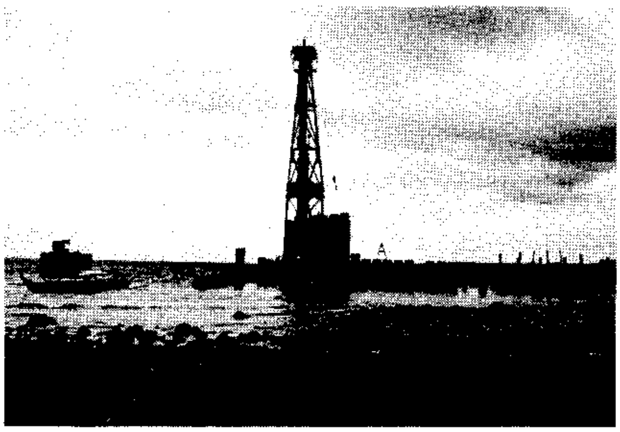


Photo 3 Driving interlocked steel pipe pile

taken while monitoring the elongation or shortening in the direction of the alignment, and when the driving error in the direction of the alignment exceeded the specified tolerance, the hydraulic positioning guide on the pile driving barge and/or the guide for obtaining the interlocked steel pile spacing was adjusted or modified in order to maintain the accuracy of the pile driving. The bearing capacity of the pile was determined using the dynamic pile driving record data.

5.4 Concrete Works

The total amount of concrete used for the civil works was 18 300 m^3, of which 14 700 m^3 was used in the wharf construction.

All the materials used were procured in the Philippines, and a batching plant (capacity 30 m^3/h) was installed at the site to supply concrete. Ordinary Portland cement procured from cement plants in Mindanao and Cebu Islands was transported directly to the construction site using barges. The chemical composition of the cement was approximately the same as that made in Japan, however, the set time was slower, with final setting being 0.7 to as much as 3.5 hours longer than that made in Japan. The compressive strength was also somewhat lower, requiring 1.3 to 1.4 times as much cement as the Japanese cement for obtaining identical strength.

The aggregate was prepared at a crushing plant from river gravels taken from a river about 6 km from the construction site. Aggregate procured in Leyte Island generally tended to have a greater washing test loss, which was another reason for requiring a higher cement content. Reinforcing bars were manufactured in the Philippines in accordance with ASTM standards. Samples were sent regularly to a university in Cebu City for inspection and testing to check strength and other mechanical properties. For the civil engineering construction, about 800 t of reinforcing bars were used, of which 550 t was for the wharf construction. The preparation and installation of the bars took 26 man · h/t, on an average.

6 Structural Characteristics of Wharf

In order to check the safety of the structures and to collect the required technical data, several observations of the behavior of the structures were made during and after the construction. The behavior of the steel sheet pile double wall structures at Berth Nos. 4 and 5 during the construction and the vibration characteristics of the wharf after completion of construction are discussed in the following section.

6.1 Steel Sheet Pile Double Wall Structure

Berth Nos. 4 and 5 were constructed using a steel sheet pile double structure, as shown in Fig. 4. While this is a self-standing wharf structure having a number of advantages as an earth-steel composite structure, it is necessary to provide adequately for stability during construction, because it is flexible structure[3)] compared with others. The balance of soil pressure acting upon the two steel sheet pile walls, tensioning of Tibles (a trade name for a polyethylene resin coated steel strand), and wave action affect the stability of the structure during construction. In order to evaluate the structural conditions during the course of the construction and to take the appropriate countermeasures quickly whenever they are required, backfilling of the space between the sheet pile walls was implemented while the behavior was monitored as described below.

(1) Displacement of the steel sheet pile wall and tension of Tible were measured.
(2) Displacement was determined by integrating readings of sliding inclinometers with the tip assumed as being fixed. The guide pipes for the inclinometers were installed down to EL − 20.0 m on Berth No. 4 and EL − 18.5 m on Berth No. 5.
(3) Tible tension was measured with two center hole type load cells mounted on both sides of the cross section.
(4) In order to supplement these measurements and to monitor the behavior of parts other than the cross section being measured, an alignment measurement was also conducted with a transit from the survey stage, to determine the displacement of the steel sheet pile head.

The steel sheet pile double wall structure executed here has the following characteristics compared with ordinary structures.

(1) The sea bed has a gradient of about 1/8, and hence the sectional rigidity and length of sheet piles are different between the front and rear sides of the structure.
(2) Wall breadth is much greater than wall height, the former being nearly twice the latter.
(3) Heads of sheet pile walls on both sides are connected with highly flexible Tibles.

Figure 10 shows the changes in Tible tension which occurred with time as the backfilling progressed. **Figure 11** shows the vertical distribution of sheet pile wall displacement. The Tibles were maintained in tension through the initial and second stages of tensioning, whereas the tension increased as the soil pressure increased while the backfilling proceeded after the second tensioning. When, after completion of backfilling, steel pipe piles were placed temporarily on the wharf to apply a surcharge load of about 5 t/m^2, the actual Tible tension was measured to be 10 t and 12 t, which was much lower than the design value (65 t) calculated by using the design method for a steel sheet pile quay wall structure. The head of the sheet pile was bent inward extensively from Tible tensioning, however, the displacement diminished after the second tensioning even when backfilling proceeded. This phenomenon suggests that the displacement of sheet pile head depends upon forced tensioning of Tible, and is affected

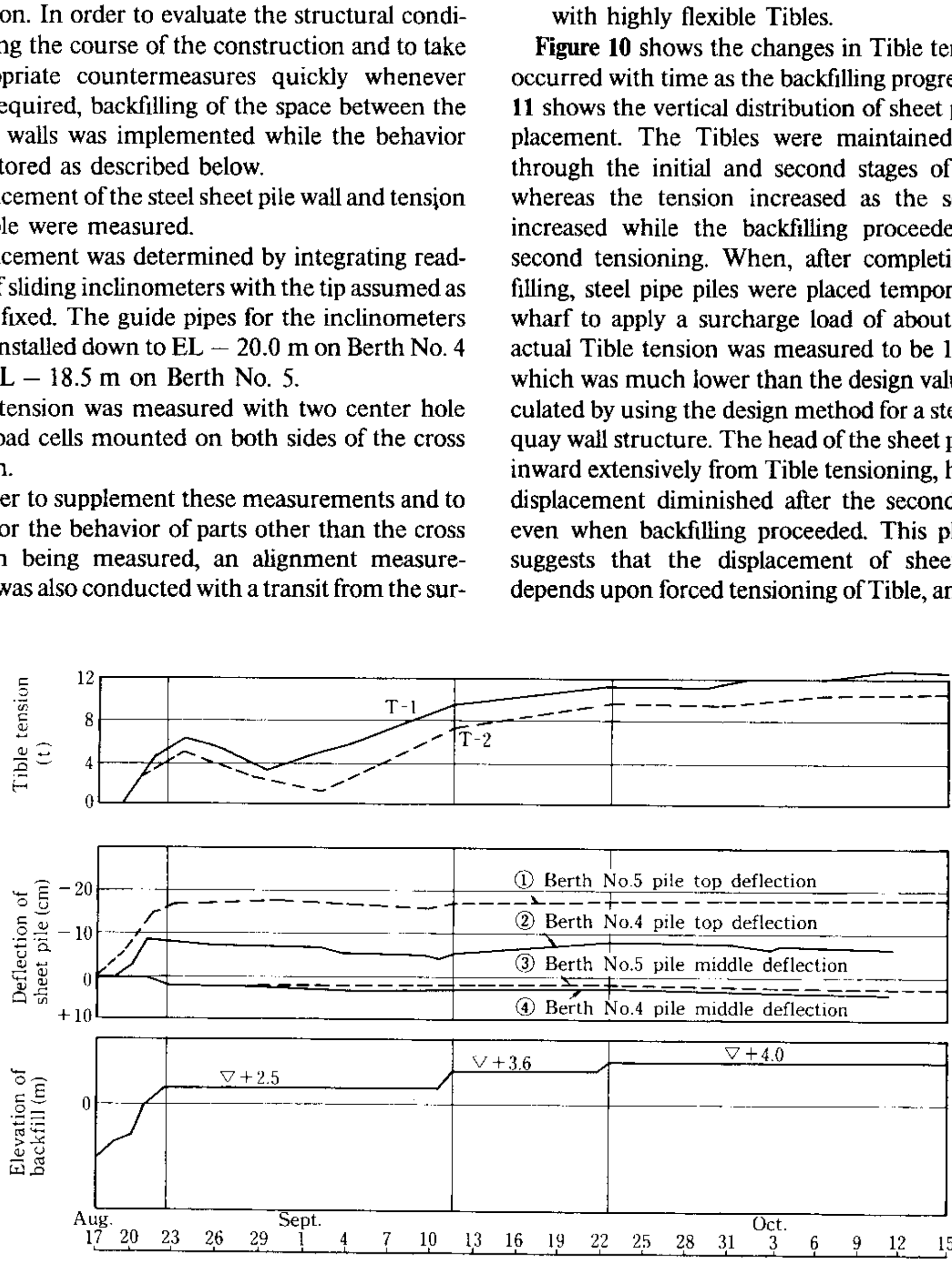
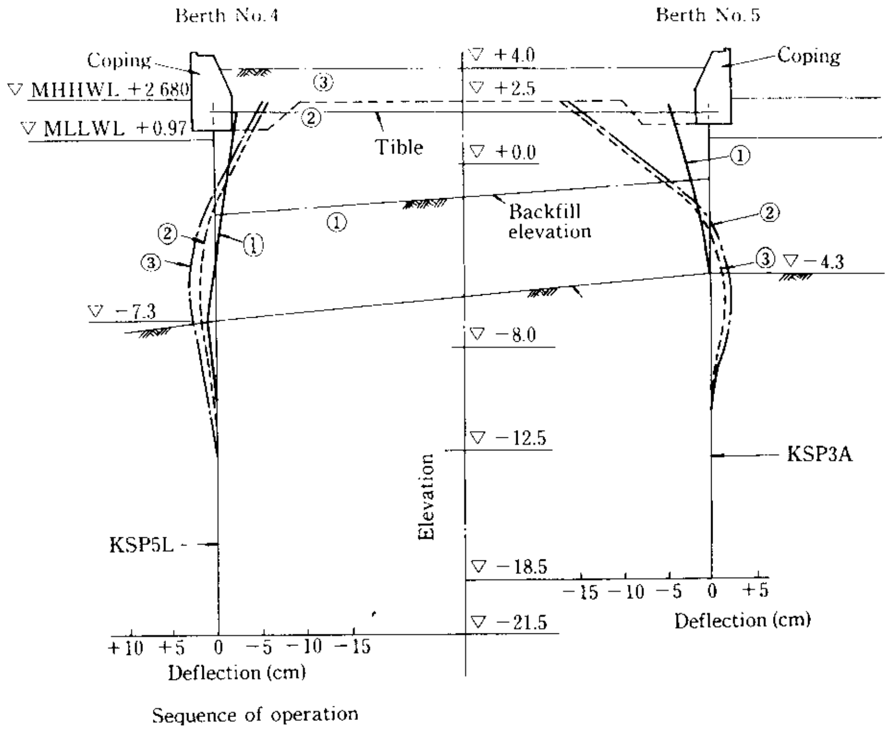


Fig. 10 Deflection of sheet pile and Tible tension of sheet pile double wall structure



——Aug. 19, 1982 After initial backfilling & Tible tensioning

----Aug. 22, 1982 After 2nd level of backfilling & Tible tensioning

—·—Sept. 30, 1982 After completion of backfilling

Fig. 11 Deflection of sheet piles of sheet pile double wall structure

very little by the elastic deformation of the Tible itself.

6.2 Vibration Characteristics of Wharf Structure

Comparing measured data on the vibration characteristics of a structure with the structural design is indispensable for checking design reliability. In order to evaluate the natural vibration characteristics of the completed wharf structure and determine its antiseismic properties, microtremor measurements were made.

Microtremor sensors using measurable ranges (frequency, 0.3–50 Hz; acceleration, 2.0×10^{-2}–1.0×10^{3} Gal; displacement, 2.0×10^{3}–1.0×10^{8} μm) were installed at the center top of each of Berth Nos. 1, 2, 3, 4 and 5, to measure the acceleration and displacement at the same point. The vibration wave forms detected by the sonic probe were amplified and recorded on a cassette data recorder. The time to acquire data was usually fixed at 3 min, and an attempt was made to minimize the effects of vibrations caused by waves, running vehicles and other factors in this period.

The record was analyzed with a Fourier analyzer, and the results are shown in **Table 4**, which gives the maximum displacement, maximum acceleration and natural frequency (natural period) of microtremors for each component. "Component" in Table 4 refers to the direc-

Table 4 Results of measurements of displacement, acceleration and frequency by microtremor sensor

Berth No.	Component	Displacement (mm)	Acceleration (Gal)	Natural frequency Hz (s)
4 and 5	H_{N-S}	1.39×10^{-3}	1.8	0.63 (1.60)
	H_{E-W}	0.005	1.5	0.44 (2.29)
	V	2.50×10^{-3}	1.8	0.69 (1.46)
3	H_{N-S}	1.19×10^{-3}	1.9	0.69 (1.46)
	H_{E-W}	0.005	1.8	0.78 (1.29)
	V	2.50×10^{-3}	1.8	0.69 (1.46)
2	H_{N-S}	1.20×10^{-3}	1.9	0.63 (1.60)
	H_{E-W}	0.007	1.6	0.75 (1.33)
	V	1.74×10^{-3}	1.6	0.59 (1.78)
1	H_{N-S}	1.35×10^{-3}	2.1	0.63 (1.60)
	H_{E-W}	0.007	1.2	1.13 (0.89)
	V	0.005	1.8	0.56 (1.78)

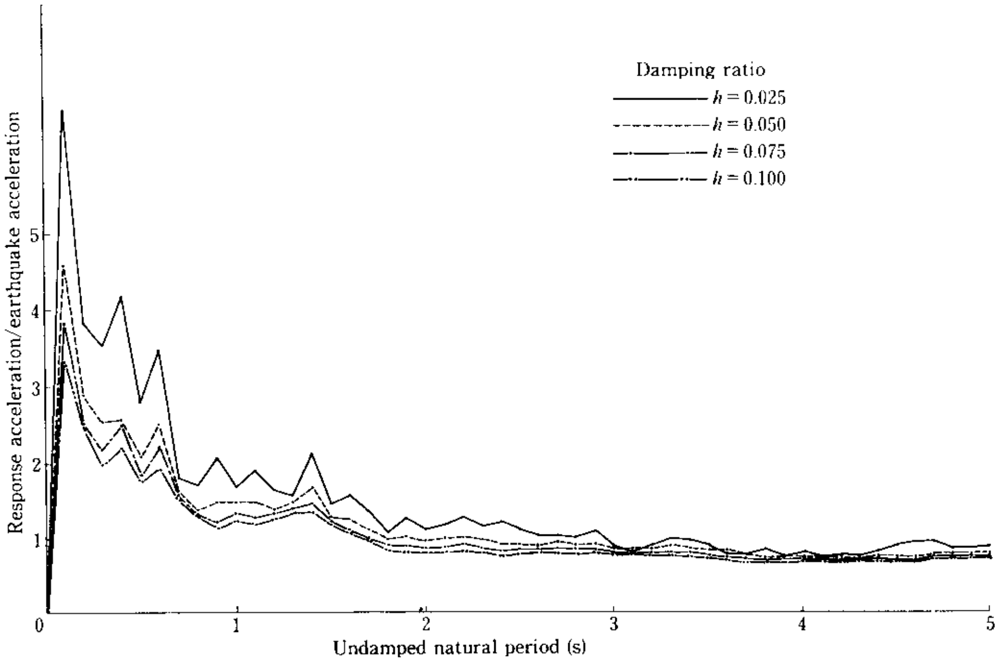


Fig. 12 Acceleration response spectra

tion in which the vibration was measured: H_{N-S}, axial direction of berth; H_{E-W}, direction normal to berth; and V, vertical direction.

The following conclusions were derived from the data presented on Table 4.

(1) The maximum displacement of the microtremor was nearly identical for different berths. The displacement in the direction normal to the berth was 4-5 times as great as that in the axial direction.
(2) The maximum acceleration of microtremor was 1.7 Gal or so, irrespective of berths and directions.
(3) The natural frequency, which is the most important parameter for antiseismic design, was about 0.65 Hz in the axial direction, irrespective of the berths, but varying extensively in the direction normal to the berths, depending on the structural patterns of each berth. That is, as mentioned above, while Berth Nos. 4 and 5 were a steel sheet pile double wall structure, the other berths were backfilled wharf structures with steel pipe piles driven as foundations for ship loader and unloader, and hence, are considerably more rigid than the former. This tendency was observed particularly for Berth No. 1.

The response of a structure is to be examined considering both acceleration and velocity response spectra for a degree-of-freedom system using acceleration wave forms of actual earthquakes. Since the acceleration wave forms of actual earthquakes were not available, the acceleration of time course obtained by multiplying the microtremor acceleration records acquired on the original ground near the construction site with strength characteristics given by the following formula were adopted as simulated seismic wave forms.

$$F_0 = t \cdot e^{-\beta t} \quad \cdots\cdots (1)$$

Where

t: time
β: 1.0

The acceleration and velocity response spectra obtained in this way are shown in **Figs. 12** and **13**, respectively. Since components in the direction normal to berth axis were of interest for design, the natural frequency in H_{E-W} direction for each berth, shown in Table 4, was applied to the acceleration response spectra given in Fig. 12, to obtain 1.05 for Berth Nos. 4 and 5, 1.45 for Berth No. 3, 1.55 for Berth No. 2 and 1.45 for Berth No. 1. It may be concluded from these results, in view of the seismic response for the structure, that the response rate of Berth No. 2 is relatively high, while that of Berth Nos. 4 and 5 is lower. That is, among berths constructed for this project, Berth Nos. 4 and 5 constructed with the steel sheet pile double wall structure exhibited much higher antiseismic properties.

7 Conclusion

Kawasaki Steel Corporation has been involved in the planning and construction of several port facilities in the Philippines since its construction of the port facility for Philippine Sinter Corporation (PSC) between 1974 and 1977.

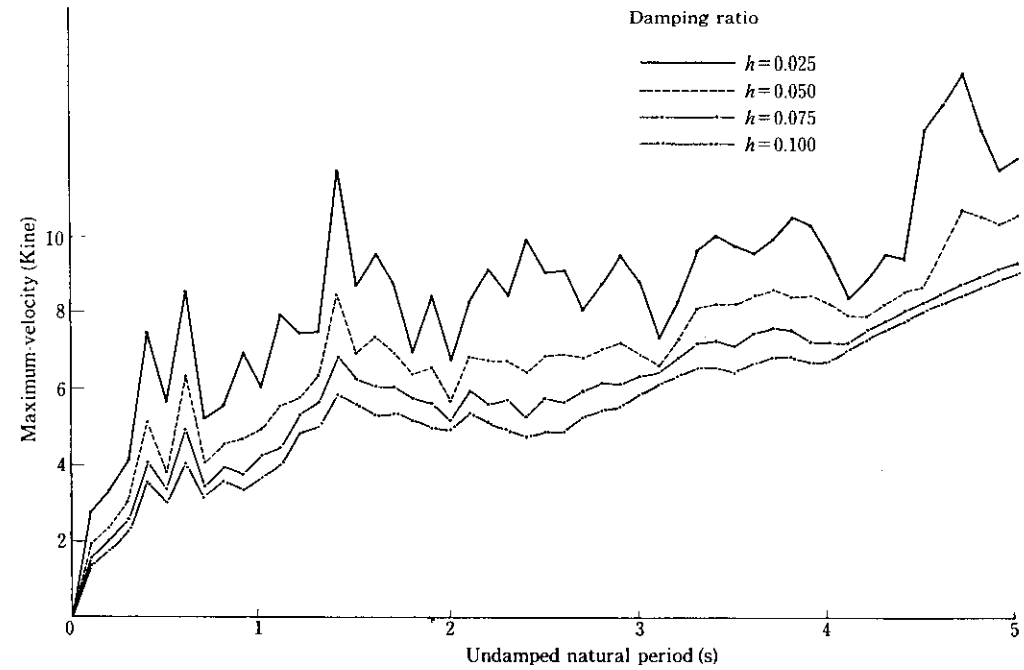


Fig. 13 Velocity response spectra

This type of civil engineering structure is very much dependent on the local conditions, and therefore requires flexible engineering techniques. A number of KSC's innovative techniques based on past experience in the Philippines contributed to the success of this project.

The authors would like to express their sincere gratitude to the Philippine and Japanese engineers who participated in the construction project in various ways. The success of this project was very much dependent on their mutual understanding and cooperation.

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