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# Construction of New Products-Berth for 80 000-DWT Vessels

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# Construction of New Products-Berth for 80 000-DWT Vessels*




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## 1 Introduction

With an aim of reducing costs for in-plant distribution of steel products such as coils, sheets, plates, and pipes, Chiba Works was making notable efforts in labor-saving, mechanization, use of bulk-sized equipment, and automation in the wide scope of physical distribution field. In order to meet ever diversifying and sophisticating users' needs for steel products, measures most urgently required were to secure exclusive-use storage yards and improve in cargo-handling facilities. In addition, considering the growing trend of vessels built larger so as to reduce marine transportation costs, it was necessary to construct a large products-berth capable of accommodating a cargo ship of up to 80 000 DWT.

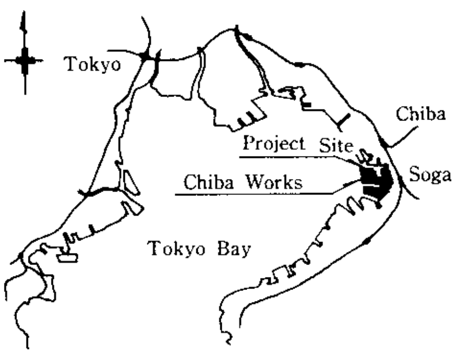


Fig. 1 Location map

This large products-berth was completed in the West Plant of the Chiba Works in October 1985. As shown in **Fig. 1**, this products-berth is located in Chiba Harbor in the east part of Tokyo Bay. Designed for a dredge depth of 15.5 m, the berth is a quay type wharf structure 300 m long and 41 m wide with a concrete deck supported on steel pipe piles.

This report presents an outline of the engineering design used for the construction of this berth for 80 000-DWT vessels, with emphasis on the design, site measurements, new materials used (heavy-duty anticorrosion steel pipe piles), and a new cantilevered pile-driving system (KST system) employed.


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## 2 Design Conditions

### 2.1 Soil Conditions

The geologic profile at the site is shown in **Fig. 2**. At depths of 40 m or more below the Arakawa River construction site datum level AP (Arakawa Pale), there is a sandy diluvium with *N*-values of 50 or more, which is composed of sandy group layers belonging to the Narita Stratum. This diluvium was, generally, utilized as the bearing layer. At depths of 30 to 40 m below AP, clay diluvium exists mainly composed of tuff clay, which alternates with sandy layers and whose unconfined compressive strength is 3 kgf/cm$^2$. These strata are generally over-consolidated. At depths of 30 m or less below AP, an upper sandy diluvium exists with a thickness of about 20 m near the east end of the berth and 7 to 10 m at the west end. A sandy layer about 8 m thick with *N*-values of 50 or more extends over a distance of 60 m along the eastern section of the berth. This layer was used as the bearing layer for this 60-m section after verifying bearing conditions during pile driving. The upper diluvium contains a soil layer with *N*-values of 5 to 20 mainly composed of sandy alluvium up to a depth of 5 m below AP—the level of the existing seabed. Based on the *N*-values, grain size distribution and results of triaxial compression tests with cyclic loading determined that the possibility of liquefaction of the sandy alluvium was low.

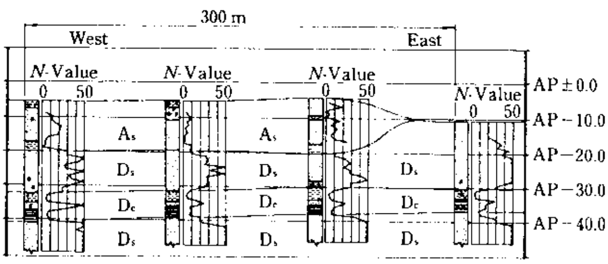



Fig. 2 Subsurface soil profile

### 2.2 Natural Conditions

Wave conditions at the site are not severe because this berth is located in the eastern portion of Tokyo Bay, north of the main channel to Chiba Harbor, which is generally a protected area. The higher high water level (HHWL) in Chiba Harbor is 2.7 m above AP; the wave height for an abnormal situation is 1.9 m, and the waves cycle 4 sec. The velocity of the tidal current in this harbor area is 0.1 knot.

Considering the local conditions and the relative importance of the structure, the design seismic coefficient during an earthquake was assumed to be 0.2.

### 2.3 Loading Conditions

In view of steel products to be handled at this berth, the loads on the concrete deck plate of the berth are high. Three 150 tf tractor-trailers can travel at the same time below the berth crane and the deck portion behind the crane rails was designed to serve as temporary storage with a superimposed load of 4 tf/m$^2$. Two berth cranes with a lifting capacity of 50 t and weighing 610 t each were considered, as well as the possibility of their being close to each other at certain times in the loading cycle for a typical vessel.

The berthing capacity was determined based on the assumption that a 80 000-DWT vessel comes alongside the berth at a 1/4 contact points at a berthing speed of 15 cm/s. The berth was also so designed as to permit the berthing of barges of 1 000 DWT or less at both ends of the berth.

## 3 Outline of Works

### 3.1 Berth Construction

As shown in **Fig. 3**, this berth is a reinforced concrete deck structure consisting of H-section beams (900 mm × 300 mm) as main girders, supported by vertical steel pipe piles 800 mm in diameter and batter steel pipe piles 900 to 914 mm in diameter. The sandy diluvium with *N*-values of 50 or more existing at depths of 40 m or more below AP was used as the bearing layer. The deck elevation of the berth was set at 5 m above AP so that waves higher than the design wave height would not come over the berth in case of abnormal high tide levels. A water depth of 15.5 m below AP was planned for the front of the berth and the slope gradient of the seabed into the harbor was set at 1 : 2.5.

### 3.2 Construction Schedule and Quantities of Civil Engineering Works

The construction schedule and quantities of civil engineering works are shown in **Table 1**. After a permit to use the required water area was obtained in March 1984, dredging was begun in June, and pile-driving in October the same year. In January 1985, concrete placement began and appurtenant works such as fender installation were completed in October the same year. The construction period was 17 months.

The number of piles driven was 463 with a total weight of 5 350 t. The volume of concrete placed was 13 500 m$^3$, and the volume of dredging amounted to 209 000 m$^3$.

The total number of cranes with a lifting capacity from 15 t to 150 t was 1 495. Pile-driving was conducted with floating equipment for 354 days. The total manpower employed on this project was as large as 37 414 man-days.

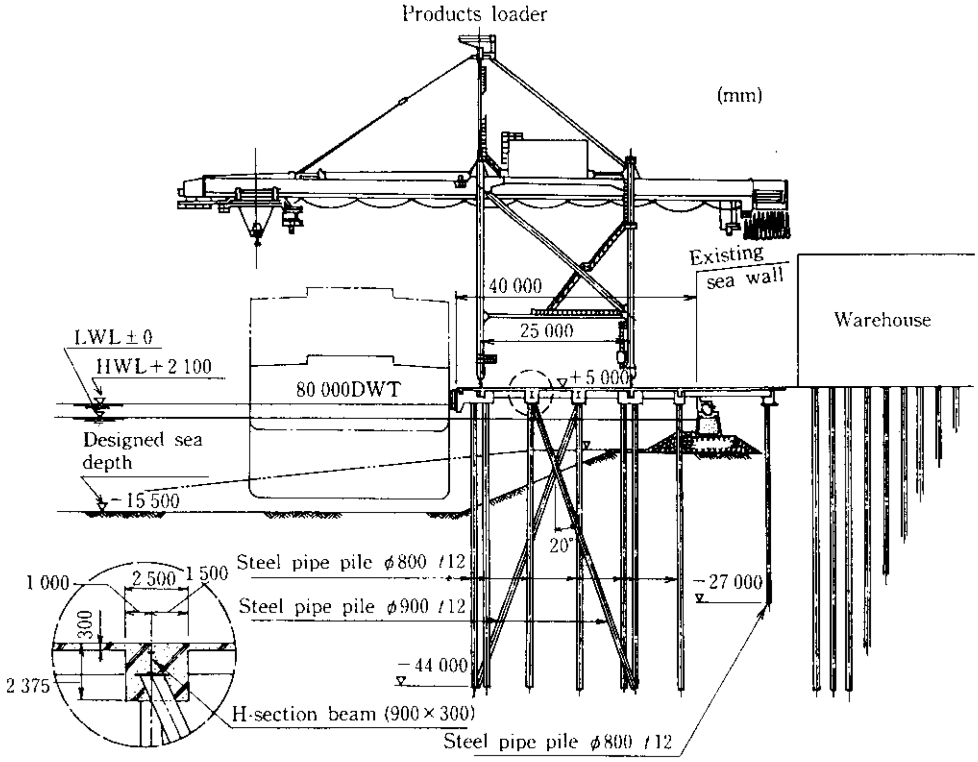
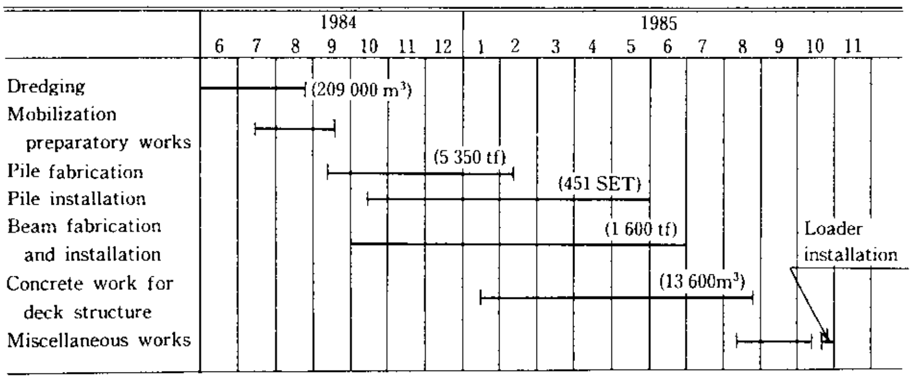



Fig. 3 A typical cross-sectional view of the product berth

Table 1 Project schedule



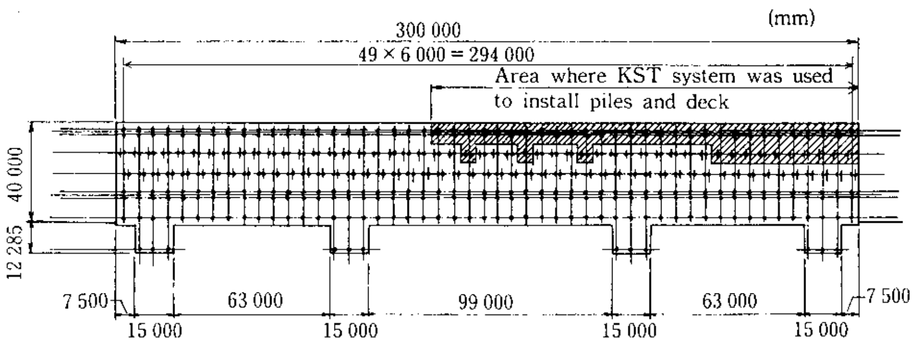




Fig. 4 Pile arrangement

For this project, 451 steel pipe piles over open water were driven from pile-driving barges and using the KST system. The pile arrangement is shown in **Fig. 4**. The pile-driving barge were equipped with KB70 or MH-72B diesel hammers. KB45 diesel hammers were used with the KST system.

## 4 New Material and New Pile-Driving System

### 4.1 Heavy-Duty Anticorrosion Steel Pipe Piles (KPP Piles)

Heavy-duty anticorrosion steel pipe piles (KPP piles) developed at Kawasaki Steel were used for the foundation support pipe piles for this berth. A KPP pile is manufactured by coating the shot-blasted steel pipe surface with a melted polyethylene sheet. The thickness of the anticorrosive polyethylene coating film is 2.5 mm or more in the spiral bead area. It is anticipated that this coating will provide 40 years of service life.

The range of installation of the polyethylene coating was from the bottom of the concrete superstructure to depths of 3 to 4 m below the seabed and corrosion is not expected to occur within this coated length. Conventional steel pipe piles were covered with concrete down to the mean water level (MWL) for corrosion protection. The use of KPP piles, however, made it possible to raise the level of the bottom of the concrete superstructure above the high water level (HWL), making it possible to reduce the weight of the superstructure. This weight-saving in the superstructure permitted a decrease in the horizontal force due to dead load during earthquake conditions, thus allowing the number of piles and the cross section of each pile to be reduced. Since work in the tidal zone was not necessary, control of the schedule was easier and quality and safety improved.

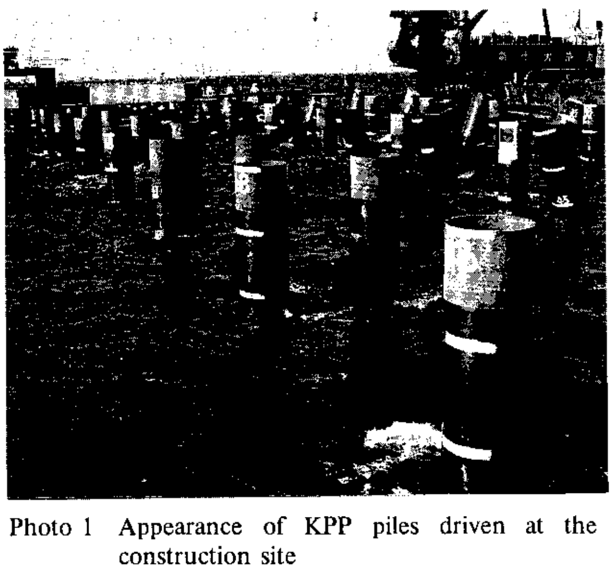


Photo 1 Appearance of KPP piles driven at the construction site

Rubber sheets and nylon slings were used to prevent damage to the polyethylene coating of pipe during transportation and temporary storage of the KPP piles. In order to prevent damage from the steel anchor wires of the pile-driving barges during pile driving, already installed piles were covered with protective steel pipe or the anchor wires were lifted by an auxiliary floating crane. Slight damage to the polyethylene coating was repaired by melting a polyethylene stick. Since portions of the steel pipe to be welded were not coated with polyethylene to avoid the influence of welding heat, the steel surface of such portions was cleaned after welding and was covered with heat-shrinkable polyethylene tubing. **Photo 1** shows the KPP piles after installation.

### 4.2 Cantilevered Pile Driving System for Marine Structures (KST System)[1]

#### 4.2.1 Outline of KST system

The KST system is an offshore construction system developed jointly by Kawasaki Steel Corp., Shimizu Construction Co., and Toa Harbor Works Co. to ensure minimal effect of wave or open water conditions on pile-driving operations, permit batter pile installation in any direction, and provide a high level of pile-driving accuracy. In this system, the equipment travels on piles driven previously by the equipment itself and permits continuous driving of the piles. The equipment used in this system is illustrated in **Fig. 5**. A batter pile being

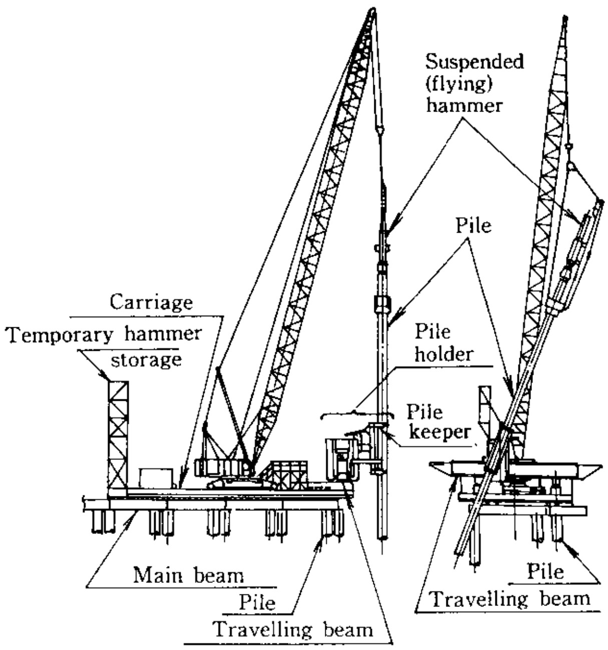



Fig. 5 KST pile driving equipment

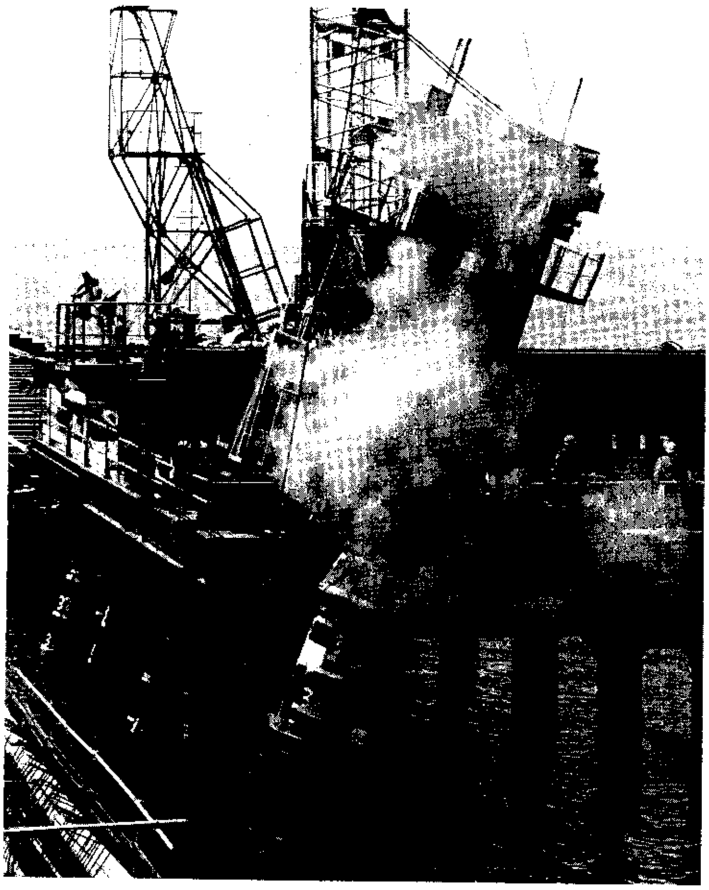


Photo 2 Batter pile driving by KST system

driven by the KST system is shown in **Photo 2** along with other piles already installed.

The pile holder is a device for holding a pile in a specified place and it can be titled to a maximum batter angle of 20 degrees. The parts of the pile holder that come into contact with the surface of the piles are equipped with rubber rollers to prevent damage to the piles. The flying hammer, which is a KB45 diesel hammer, is suspended from a steel wire cable. The carriage, on which various pieces of equipment are loaded, moves on travelling beams.

#### 4.2.2 Results of execution of works

The area where the KST system was used for driving piles is shown in Fig. 4. The specifications of the piles are given in **Table 2**. The number of days required for installing the 84 piles in this section was 34 and the performance rate was 2.47 piles a day. The pile-driving operation was possible even under rough sea conditions when a wave height of 0.8 to 1.2 m occurred and the operation rate was 74%. The pile-driving accuracy included 75% of the piles being installed within 5 cm from the specified locations, and all of the piles within 10 cm. It was demonstrated that the KST system is affected very little by wave conditions, and that the pile-driving accuracy is high. It was also determined that batter piles could be installed with angles up to 20 degrees in all directions.

Table 2 Pile specifications by KST pile driving system

| | Diameter (mm) | Thickness (mm) | Length (m) | Number of piles (piles) | Total number (piles) |
|---|---|---|---|---|---|
| Vertical pile | 800.0<br>800.0 | 12.0<br>12.0 | 48.0<br>31.0 | 41<br>30 | 84 |
| Batter pile | 914.4<br>914.4 | 15.88<br>15.88 | 49.0<br>34.0 | 3<br>10 | |

Measurements were made at the site of the acceleration of the KST pile-driving equipment and the stresses in its members during pile driving and travel of the carriage. As a result, it was determined that no resonance of the complete carriage occurs during pile driving and that dynamic stresses generated in each member during the travel of the carriage are very small. Stresses in each member during the travel of the carriage and those generated in the pile holder shaft during pile tilting were almost the same as design values.

### 4.3 Control of Bearing Capacity of Piles

To estimate the bearing capacity of piles accurately based on information obtained during driving is an

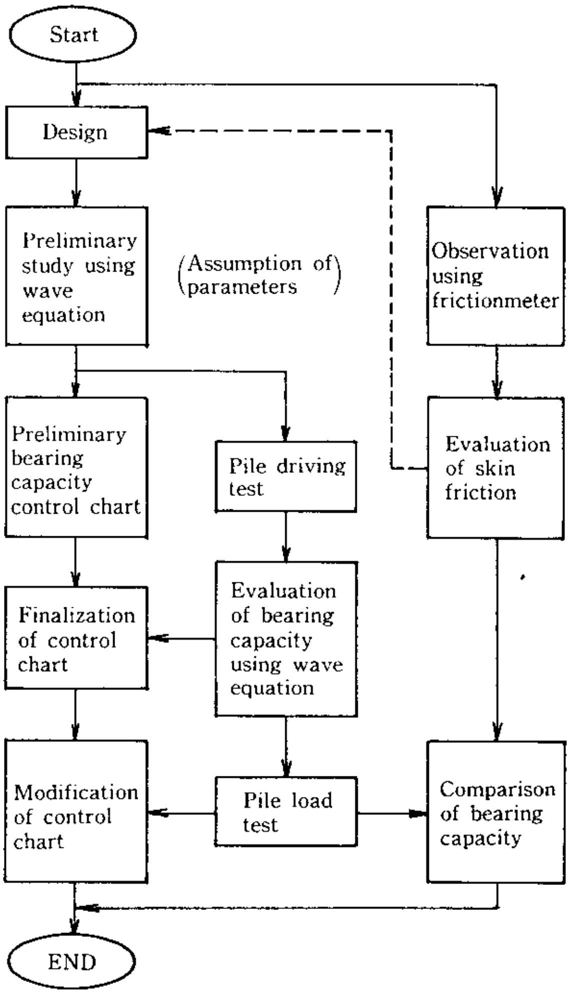



Fig. 6 Flowchart for bearing capacity control

important factor for the quality control of a pile installation. In this project, the pile installation was controlled in accordance with the sequence of operations and involving the estimation[2,3] of the bearing capacity using the wave equation, as shown in **Fig. 6**.

Parameters, such as skin friction and pile tip bearing ratio, were first established based on the results of a soil survey and equations using *N*-values, then the stress distribution during pile driving was estimated, and the type of hammer to be used and the wall thickness of piles was determined. At the same time, a tentative control chart for bearing capacity was prepared indicating the relationship between the final penetration and bearing capacity of the pile. Next, the stress distribution during pile driving was measured, and each parameter was established so that the stress distribution determined from the wave equation was in agreement with measured values, and finally the dynamic ultimate bearing capacity was determined. A final bearing capacity control chart was then prepared. A correction factor in which the degree of restoration of the subsoils was taken into consideration was determined from the relationship between the static ultimate bearing capacity and dynamic ultimate bearing capacity obtained from static vertical load tests, and a modified bearing control chart was prepared. The bearing capacity control of the piles was based on this modified chart.

## 5 Conclusions

An outline of the construction of the large products-berth at Kawasaki Steel's Chiba Works located north of the main channel to Chiba Harbor is presented in this paper. The construction of this berth has the following features:

(1) By using heavy-duty anticorrosion steel pipe piles (KPP piles), weight-saving in the superstructure and reduction of the burden of maintenance and control were achieved. Work in the tidal zone was unnecessary, and improvements were made in work progress, quality, and safety.

(2) The adoption of the cantilevered pile driving system, KST system, for marine structures to construction sites was determined to be quite suitable because it was found that pile-driving operations were possible even under rough sea conditions. This method was found to be little affected by wave conditions, and the pile-driving accuracy was very high.

(3) The execution of the project was controlled by taking into consideration field data–i.e. control of the bearing capacity of piles using the wave equation and measurement of the behavior of the existing seawall (caisson revetment) resulting from dredging and pile driveup operations adjacent to it.

The authors would like to express their sincere appreciation to Professor Minoru Matsuo of Nagoya University and Professor Takahide Horiuchi of Meijo University for their guidance related to the control of the bearing capacity of piles as well as the concerned parties of Shimizu Construction Co., Ltd., Ohbayashi Corp., Taisei Corp., and Toa Habor Works Co., Ltd. for their cooperation in the execution of this project.

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