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# Kota Kinabalu Port Expansion Project

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Synopsis :
The Kota Kinabalu Port expansion project in Kota Kinabalu, Sabah, Malaysia was started in May 1985 and successfully completed in November 1987 by Kawasaki Steel Corporation. The major feature of this projects is that KPP (Kawasaki plastic-coated pipe) piles, a total number of 1429, were used. This method was adopted as the most effective anti-corrosion protection system for the heavy corrosive marine environment which existed in this tropical area. Driving such a large number of piles offshore in close proximity to each other was an unusual undertaking. In addition, the execution plan was carefully considered in order not to damage the high corrosion resistance coating during installation of the piles. The problem of movements of existing piles due to new pile driving was investigated and an evaluation method was utilized for the movements of the adjacent ground. The results of this evaluation were then compared with actual data.

# Kota Kinabalu Port Expansion Project*


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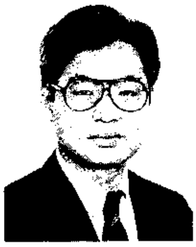


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## 1 Introduction

Kota Kinabalu Port, Sabah, Malaysia, is one of the ports managed by the Sabah Ports Authority (SPA). This port is a general trading port where wood—a source of revenue for Sabah—is shipped, as well as general consumer goods and industrial products. The port provides a definite economic base for Sabah (**Fig. 1**). Since service began in 1968, the size of this port has been increased through several expansion projects. The port has nevertheless been faced with problems such as insufficient harbor capacity due to an ever-increasing amount of cargo handling, obsolescence of facilities, and the need, for measures to cope with the worldwide trend toward containerization. To improve ferry transportation with Malay Peninsula and Sarawak, a movable jetty (roll-on roll-off) was constructed in the port. The construction of this jetty was reported in a preceeding paper.[1] The present expansion project was implemented in order to attempt the solution of these problems. Kawasaki Steel Corp. (KSC) organized a consortium with Kanematsu-Gosho Ltd. and a local contractor, and was awarded the marine construction contract in May 1985. The harbor expansion work was completed in November 1987.

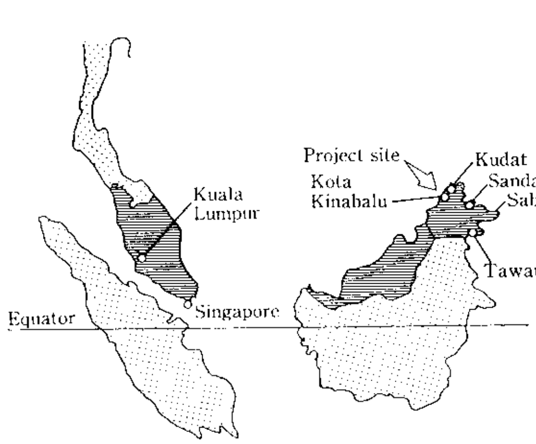



Fig. 1 Major ports in Sabah and project site location

* Originally published in *Kawasaki Steel Giho*, **20**(1988)4, pp. 291-298

This report presents an outline of the Kota Kinabalu Port expansion project, in particular the execution of the installation of KPP (Kawasaki polyethylene coated pipe) piles and concrete work.

## 2 Outline of Work

As shown in the plan in **Fig. 2**, this work was broadly divided into the construction of the new jetty (New South Jetty), expansion of the existing jetty (North Extension) and construction of power and water facilities. A typical section of the new jetty is shown in **Fig. 3**. Precast concrete units (PC units) were used as the deck slabs for the jetty. The quantities of major construction materials are presented in **Table 1**.

Since this work was carried out within an existing port facility, a phased-completion schedule (**Table 2**) was prepared to avoid disruption of the cargo handling func-

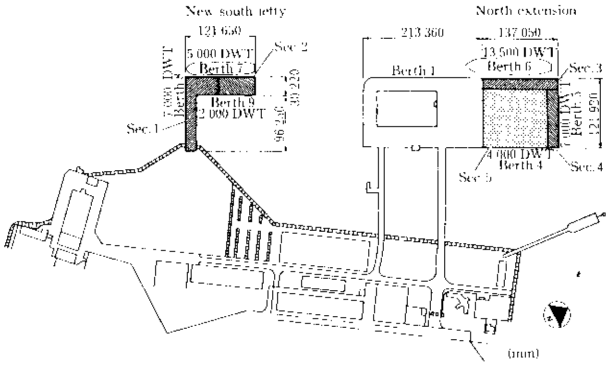



Fig. 2 General plan

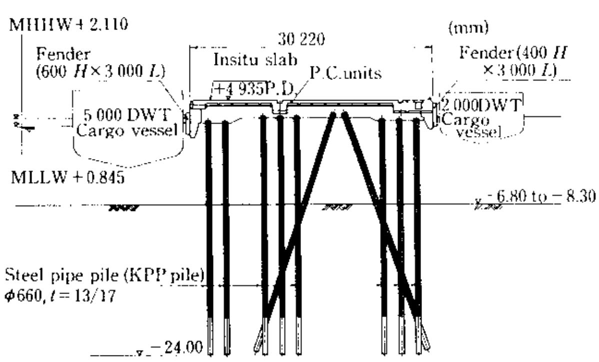



Fig. 3 Typical section

Table 1 Quantities of major items

| Item | | New south jetty | North extension | Total |
|---|---|---|---|---|
| KPP pile*1 | | | | |
| Number of piles | | 379 | 1 050 | 1 429 |
| Weight of pile | (t) | 2 899 | 8 030 | 10 929 |
| Concrete (Total) | (m³) | 7 018 | 17 628 | 24 646 |
| Precast unit | (m³) | 1 650 | 4 706 | 6 356 |
| Insitu concrete | (m³) | 5 368 | 12 922 | 18 290 |
| Ductile iron pipe | | | | |
| 150 mm O.D. for water supply | (m) | 400 | 400 | 800 |
| 200 mm O.D. for fire protection | (m) | 240 | 400 | 640 |

*1 660 mm O.D. x 17/13 mm thick

Table 2 Staged completion schedule

| Section No. | Project month | Completion date | Location | |
|---|---|---|---|---|
| Sec. 1 | 13 | 1986 Jun. 23 | Berth 8 | New south jetty |
| Sec. 2 | 15 | 1986 Aug. 31 | Berth 7, 9 | New south jetty |
| Sec. 3 | 20 | 1987 Jan. 31 | Berth 6 | North extension |
| Sec. 4 | 28 | 1987 Sep. 30 | Berth 5 | North extension |
| Sec. 5 | 35 | 1988 May 1 | Berth 4 | North extension |

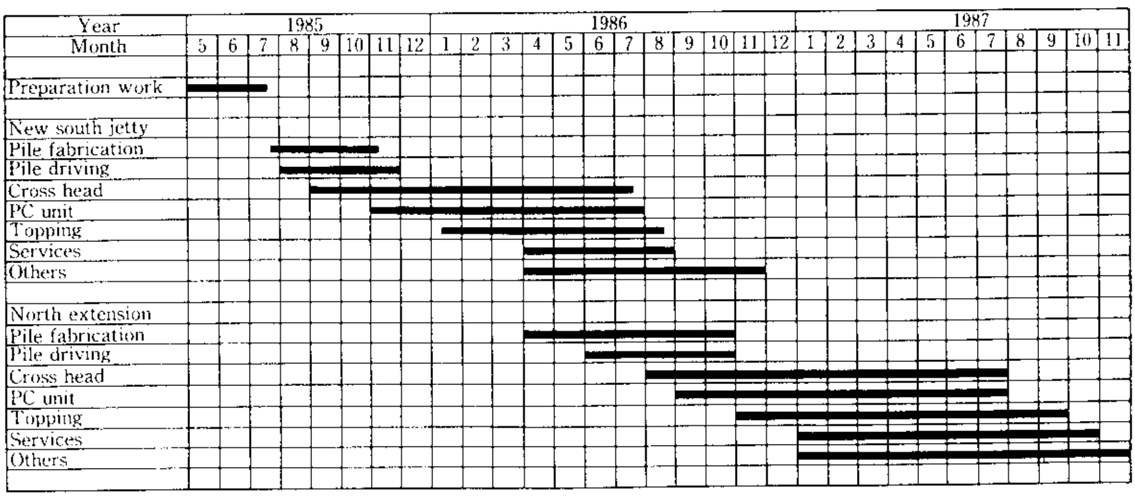



Fig. 4 Actual execution of construction

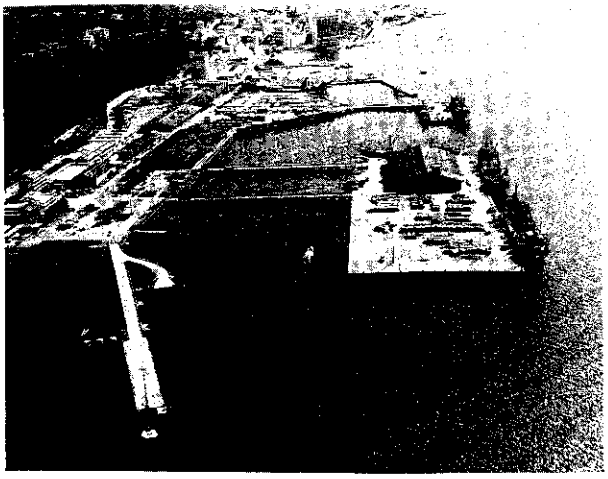


Photo 1 Overall view of Kota Kinabalu Port

tions. Accordingly, the "open water" area allocated for construction was limited and it was necessary to carefully control the execution of work in terms of time and space.

The project was completed in only 30 months, five months less than the originally agreed upon construction period. The actual construction schedule is shown in **Fig.** 4, and a general view of the completed jetty in **Photo 1**.

## 3 Execution of KPP Pile Work

### 3.1 KPP Piles

The corrosion protection of steel pipe piles is very important for offshore structures. Although corrosion protection with a coal tar epoxy coating was considered in the original design for this project, it was finally decided to adopt KPP piles (**Fig.** 5) which are heavy duty polyethylene-coating steel pipe piles.

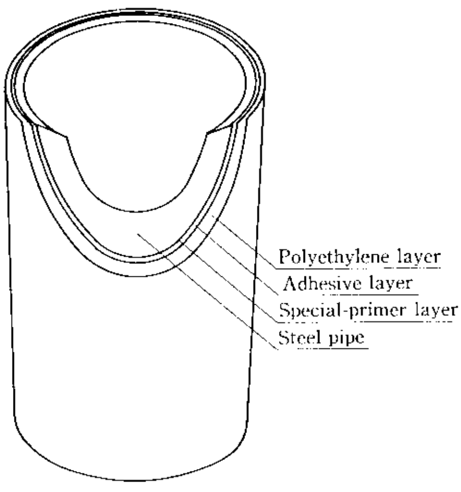



Fig. 5 KPP (Kawasaki Plastic-coated pipe) pile

Table 3 Comparison of steel pipe pile foundation system

| Item | Original design | Final design using KPP piles |
|---|---|---|
| Diameter (mm) | 660 | 660 |
| Thickness of steel pipe pile | | |
| Upper pile | 19.0 | 17.0 |
| Lower pile | 14.2 | 13.0 |
| Weight of steel pipe pile (t) | 12 035 | 10 929 |
| Corrosion protection treatment | | |
| Material | Coal tar-epoxy | Polyethylene |
| Thickness | 450 micron | 2.5 mm |

The KPP pile retains its corrosion prevention capability for a longer time without maintenance than the conventional corrosion protection method. The steel pipe is coated with a very durable polyethylene layer of 2.5-mm film thickness. The corrosion allowance can thereby be reduced and, as shown in **Table 3**, it was possible to save about 1 100 t of steel compared with the original design.

### 3.2 Pile Fabrication on Site

For the KPP piles used in this project, steel pipe sections 12 to 18.5 m in length were transported from Japan. The upper and lower sections of each pile were circumferentially butt welded, and pile toe plates were fabricated at the site. This field butt welding was conducted using submerged arc welding by rotating the pile with a turning roller. British Standards (BS) were strictly complied with for the welding procedure and inspection of welds, as shown below:

(1) Welding procedure trial, including the destructive test of specimens (BS 4870 and BS 709)
(2) Welder qualification test (BS 4871)
(3) 100% radiographic examination of circumferential welds (BS 4870)

For the examination of field circumferential welds, in particular (3) above, careful attention was paid to quality control.

### 3.3 Pile Driving Work

#### 3.3.1 Pile driving supervision

The pile driving supervision for this project was implemented using the following procedure:

(1) Test pile driving in positions specified by the design consultant
(2) Pile load tests conducted on test piles (at four locations)
(3) Evaluation and correction of a dynamic formula for bearing capacity of pile based on results of the pile load tests
(4) Pile driving supervision using the corrected dynamic

formula for bearing capacity of pile

Although it had been decided to apply Hiley's formula as the dynamic formula for the bearing capacity of the piles, a specific equation was not designated, and there was discussion between the general contractor and the consultant as to whether the corrected formula Eq. (1) below, generally adopted in Japan, or the original formula Eq. (2) below should be used. At the time of test pile driving, however, Eq. (2) was adopted as designated by the consultant based on a considerations of safety because the number of tests was small.

$$R_u = \frac{e_f \cdot 2WH}{S + C/2} \quad \cdots\cdots\cdots\cdots (1)$$

$$R_u = \frac{WH}{S + C/2} \times \frac{1 + e^2(P/W)}{1 + (P/W)} \quad \cdots\cdots\cdots (2)$$

where $R_u$: Ultimate bearing capacity of pile
$W$: Weight of ram
$H$: Ram height
$S$: Final set per blow
$C$: Temporary compression
$P$: Weight of pile and pile-cap
$e_f$: Efficiency of hammer
$e$: Coefficient of restitution

### 3.3.2 Driving sequence

It was necessary to plan the pile driving sequence for the North Extension based on a consideration of the following points:

(1) The driving sequence was limited in terms of the construction period because the North Extension work, which was divided into three sections, had to be completed and delivered in stages.
(2) Because of the large plan area, wire ropes for anchoring the pile driving ship interfered with the existing piles. It was therefore necessary to plan the driving sequence in a manner that minimized the movement of the wire ropes for anchoring.
(3) The adjoining existing jetty which was in service during the pile driving and the water area in which wire ropes for anchoring could be installed was limited.

The actual pile driving sequence is shown in **Fig. 6**. The formwork and reinforcing steel for stage 4 proceeded well in advance of the pile driving for stage 6. In driving the piles for stage 6, therefore, it was necessary to construct temporary dolphins using existing raker piles and to install wire ropes for anchoring with the aid of these dolphins because the installation of these wires across the stage 6 area was considered dangerous. The work efficiency was thus decreased. Furthermore, in order to prevent the contact of wire ropes for anchoring with existing piles below water and resultant damage to the pile coating, anchor locations were carefully selected and the coated areas were protected where contact was

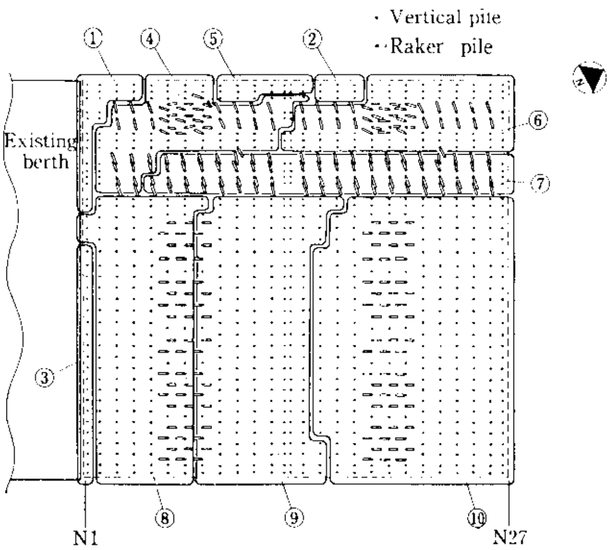



Fig. 6 Sequence of pile driving for north extension

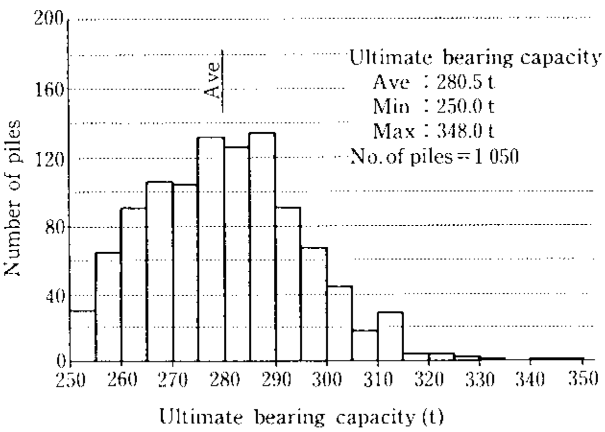



Fig. 7 Ultimate bearing capacity estimated by Hiley formula

unavoidable.

Ultimate bearing capacities of the North Extension calculated by Eq. (1) are shown in **Fig. 7**. Although the pile driving supervision was based on an ultimate bearing capacity of more than 240 t (more than twice the working load), the average ultimate bearing capacity was 280 t when proper supervision was implemented using Eq. (1).

The number of blows per pile averaged 618. This figure was very small compared with ordinary pile driving. The reason for this was that the upper subsoil layer was a weak stratum and, hence, blows occurred only during the pile penetration into the weathered shale bearing layer. A KB45 diesel hammer was used to drive the piles and the pile driving rate was generally 7 piles per working day.

### 3.3.3 Pile load test

As mentioned earlier, pile load tests were conducted at four locations as part of the bearing capacity quality control. A pull-out test was first conducted and then a compression test was carried out on the same test pile after redriving. A total of 8 tests were conducted. A typical soil profile is shown in **Fig. 8**.

These tests were carried out using the maintained load method specified in BS CP2004. The maximum load was 150% of the design load, with an 88-t result for the pull-out test and 180 t for the compression test. Results of the pile load tests are shown in **Table 4**, which also indicates the ultimate bearing capacity for a pile estimated using the dynamic formula for bearing capacity of pile specified in Eq. (1).

The test pile for the New South Jetty was pulled out in a second test at a load (75 t) corresponding to 85% of the maximum load. As a result, the adequacy of the bearing capacities of the raker piles was questioned. A lean mix concrete was therefore placed along the full pile length for raker piles only with a high working pull-out force to increase the dead weight of the piles.

In the compression test, a clear yield point load was not reached even under maximum load, and it was therefore considered that the relationship between load and settlement was almost linear and the bearing capacity was considered sufficient.

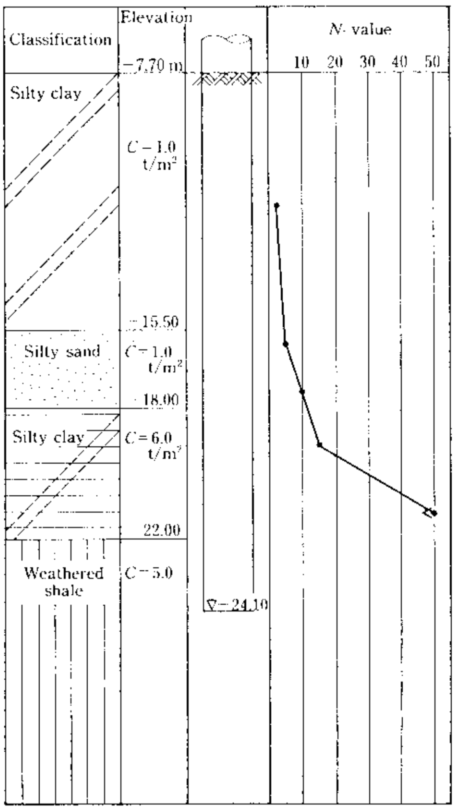


Fig. 8 Soil condition

### 3.3.4 Closed-end pile

Closed-end piles as shown in **Fig. 9** were installed for this project. In the pile driving operations, there was some concern that sufficient penetration might not be obtained due to the buoyancy effect acting on piles because of their closed ends. Eventually, however, no marked difference from the use of open-end piles was found.

Because the pile end was completely closed, however, the amount of soil displaced during the driving was large. In addition, piles were arranged in a dense configuration. As a result, piles already driven moved due to the lateral displacement of soil when succeeding piles were driven. The average observed values of pile top movement for each grid line are shown in **Fig. 10**. Piles were driven from grid N1 toward N27. The pile top displacement averaged 100 mm in the direction opposite to the travel direction of the pile driving ship.

It is well known, that existing piles shift when succeeding piles are driven with a large amount of soil

Table 4 Results of pile load tests

| Item | Tension test | | | | Compression test | | | |
|---|---|---|---|---|---|---|---|---|
| | New south jetty | | North extension | | New south jetty | | North extension | |
| | 1st | 2nd | T1 | T2 | 1st | 2nd | T1 | T2 |
| Pile data | | | | | | | | |
| Length (m) | 31.89 | 29.51 | 33.63 | 33.24 | 30.01 | 27.63 | 31.86 | 31.94 |
| Hammer energy (t·m) | 10.35 | 11.03 | 11.05 | 10.80 | 11.30 | 10.80 | 10.70 | 10.90 |
| Final set (mm/blow) | 0.83 | 0.51 | 0.56 | 1.32 | 0.46 | 0.24 | 0.60 | 1.00 |
| Rebound (mm) | 24 | 27 | 30 | 27 | 24 | 24 | 27 | 25 |
| $R_u$ (t) | — | — | — | — | 332 | 329 | 275 | 273 |
| Test result | | | | | | | | |
| Max. applied load (t) | 110 | 75 | 88 | 88 | 197 | 200 | 180 | 180 |
| Max. settlement (mm) | 11.05 | 48.00 | 5.05 | 5.94 | 11.63 | 12.11 | 13.35 | 10.07 |

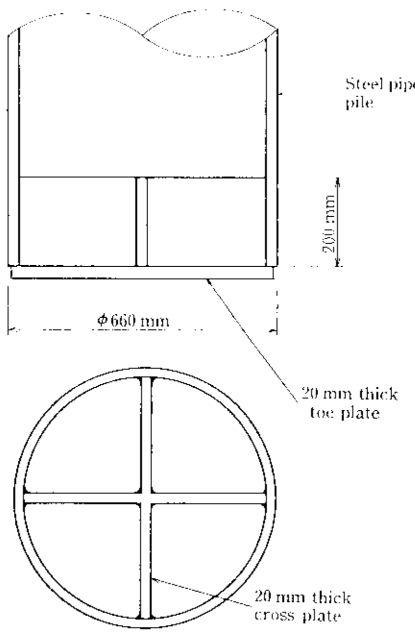



Fig. 9 Closed end pile

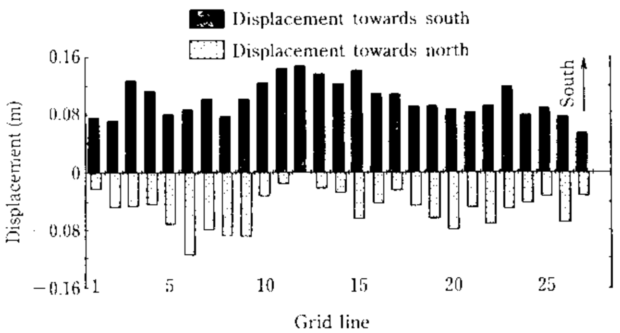



Fig. 10 Displacement of pile position caused by adjacent piling

being displaced. An analysis using finite element methods was attempted to clarify this phenomenon. In this analysis, the deformation behavior of the surrounding soil during pile driving was treated as a problem of axial symmetry and the constructive mode proposed by Sekiguchi and Ohta[2] was adopted as the soil model. The soil was assumed to be a clayey soil and attention was only paid to the undrained behavior due to pile driving. The creep and consolidation which occur after pile driving were not taken into consideration.

Results of an analysis of radial changes in the vertical displacement $V_r$ and horizontal displacement $U_r$ of the ground surface occurring when a pile penetrates the soil to a depth of 4 m are shown in **Fig. 11**. The ratio of the strength of clayey soil, $C_u$, to the overburden pressure $p$, i.e., $C_u/p$ changed at three different levels. It is apparent from this figure that the soil within about 2 m of a pile is subjected to large shearing forces associated with pile penetration and is in a state of yield; that the dispersion of analytical values resulting from the differ-

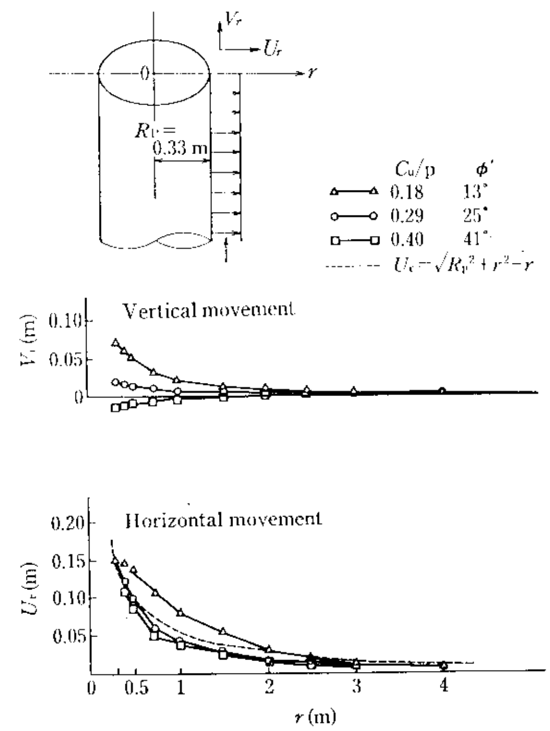



Fig. 11 Analysis of pile movement

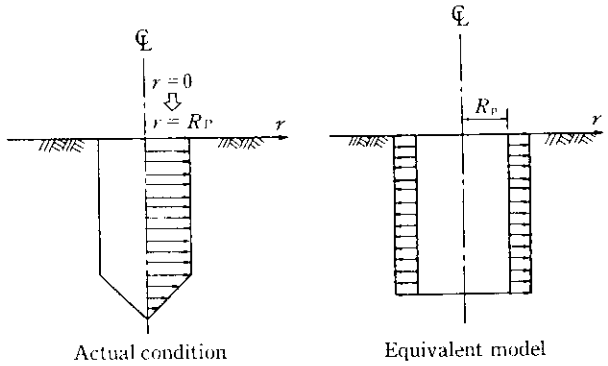



Fig. 12 Equivalent model for analysis

ence in $C_u/p$ is large; and that the decrease in the horizontal displacement $U_r$ also shows quite different tendencies. In the range of $r > 2$ m or so, however, the dispersion due to the difference in $C_u/p$ is small and almost the same tendency is observed.

If the soil displacement due to pile penetration is treated as a question of expansion of an equivalent cylinder as shown in **Fig. 12**, the horizontal displacement of an equivalent soil is given by Eq. (3):

$$U_r = \sqrt{R_p^2 + r^2} - r \quad (r > R_p) \qquad \cdots\cdots\cdots(3)$$

where $U_r$: Horizontal displacement of equivalent soil
$R_p$: Pile radius
$r$: Distance from pile center

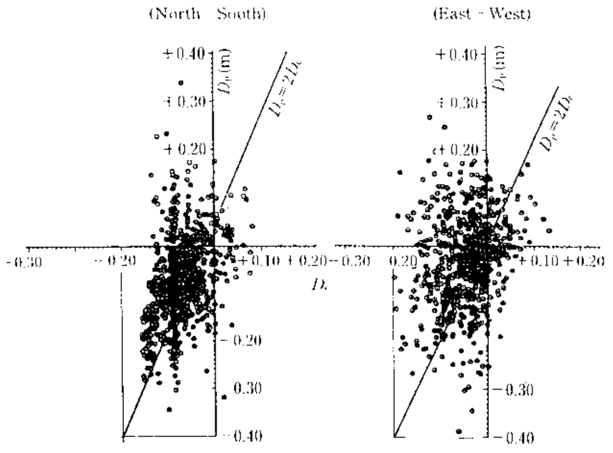



Fig. 13 Comparison between observed pile displacement $D_p$ and calculated ground displacement $D_c$

Values of horizontal displacement determined by Eq. (3) are also shown in Fig. 11. In the range of $r > 2$ m, these values are almost in agreement with those determined from the finite element method.

**Figure 13** shows a comparison between observed pile displacement and calculated ground displacement. The difference is quite large. If the observed value of pile head displacement is denoted by $D_p$, however, and the calculated value of ground displacement is expressed by $D_c$ ($D_c = \sum U_r$), the relationship $D_p \fallingdotseq 2D_c$ is observed both in the direction of the pile driving ship movement and in a direction at right angles to it.

The reason the pile head displacement is larger than the ground displacement is that in the actual ground, the ground surface is displaced more than the deeper layers, with the result that the pile inclines somewhat, resulting in a larger displacement at the pile head than at ground level.

## 4 Concrete Work

### 4.1 Staging

For this port expansion project, the pile driving work and the construction of concrete beams (called crossheads) to connect the piles were especially important in terms of the overall progress.

The entire procedure of construction of the cross heads was carried out over open water. For the North Extension, the plan area was particularly large and the area in which heavy equipment, such as the pile driving barge and other floating equipment could be used was limited. The method of installing the staging for concrete work was therefore very important.

In planning and designing the staging, the following conditions were taken into consideration:

(1) Tide level and the level of the bottom of the crossheads
(2) Wave action
(3) Deviation from the design center of the piles and stability during concrete placement
(4) Method and procedure for assembly and disassembly of staging
(5) Provision of steel materials for temporary support work

Wave action was considerably different in the dry season (May to October) and the wet season (November to April). Results of observations for the past year revealed that the wave height was 0 to 15 cm during the dry season and 0 to 50 cm during the wet season. In the three months of December to March, the monsoon is especially strong, making marine work in the open sea very difficult.

In selecting the type of temporary support, the above-mentioned conditions were taken into consideration. A comparison of the types evaluated is presented in **Table 5**.

The hanging type staging (1) in which the amount of steel embedded in concrete is small was originally adopted. However, it was later replaced with the hanging type staging (2) which provides larger tolerances on width for pile eccentricity resulting because piles shifted after driving as mentioned in 3.3.4. **Photo 2** shows the hangings finally installed.

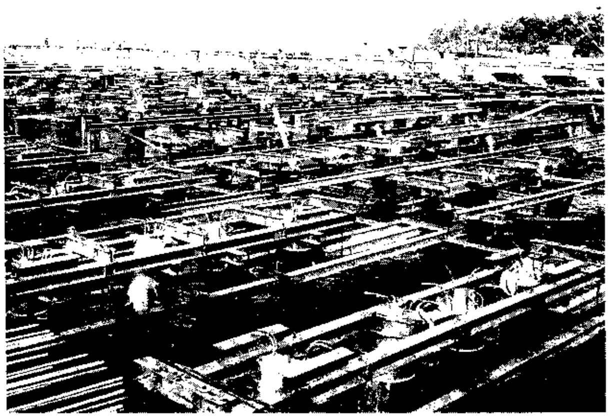


Photo 2 Support system for superstructure

### 4.2 Concrete Mix

Although it was possible to use ready-mixed concrete in Kota Kinabalu, it was decided to construct a batch plant on the site as a supply facility for all concrete because the amount of concrete placed was large and there was some concern about quality control and the capacity to supply the concrete. The concrete design mix used is presented in **Table 6**.

### 4.3 Hot Weather Concrete

Since Kota Kinabalu is situated in a tropical zone at latitude of about 6°N, temperature control of the con-

Table 5 Comparison of support system

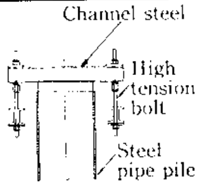
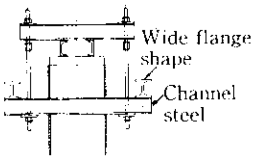
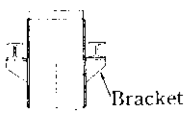
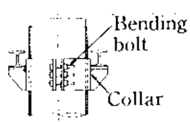
| Type of support | Sketch | Advantages | Shortcomings |
|---|---|---|---|
| Hanger type (1) |   | · Less temporary embedded steel than hanger type (2).<br>· Easy installation. | · Application to raker pile is difficult.<br>· Small allowance for pile deviation. |
| Hanger type (2) |   | · Large allowance for pile deviation.<br>· Location of hanger bolt can be selected at any point. | · Large quantity of temporary embedded steel.<br>· Takes time for installation. |
| Bracket type |   | · No embedded steel.<br>· Easy installation. | · Removal and repair of coating is required.<br>· Application to raker pile is difficult.<br>· Welding of bracket is affected by tide level. |
| Pre-fabricated bracket type |   | · No coating removal.<br>· No embedded steel. | · Fabrication of steel bracket is expensive.<br>· Bearing share capacity between bracket and pile has to be investigated. |

Table 6 Concrete design mix

| | | |
|---|---|---|
| 28 days strength | ($N/mm^2$) | 30 |
| Max. coarse aggregate size | (mm) | 20 |
| Slump | (mm) | 75-100 |
| Water/cement ratio | (%) | 45 |
| Fine/coarse aggregate ratio | (%) | 42 |
| Unit weight | ($kg/m^3$) | |
| Water | | 180 |
| Cement | | 400 |
| Fine aggregate | | 742 |
| Coarse aggregate | | 1 014 |

crete was very strict and the specifications for the project required that the concrete temperature at the point of discharge be 30°C or less. Although measures were already taken during the trial mix design stage to lower the concrete temperature during placement, the temperatures of aggregate, water (from public water supply) and cement were high and it was therefore necessary to take more effective action related to quality for concrete temperature.

It was decided to install a chilled water plant as shown in **Fig. 14** and to supply water at a temperature of 2 to 4°C. Furthermore, it was decided to place con-

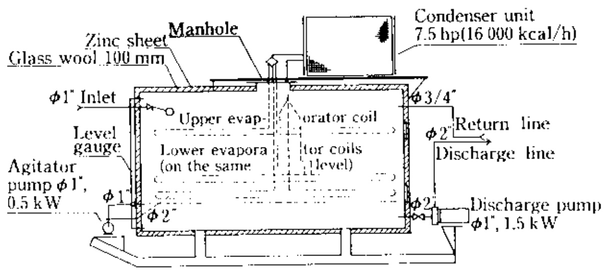



Fig. 14 Chilled water supply system (Tank capacity: 8 000 $l$)

crete early in the morning and at night as much as possible and avoide concrete placement during the daytime. When the large volume of concrete to be placed necessitated daytime pouring, however, ice was used for the primary chilling of the water.

The effect of using chilled water on the relationship between the temperature of the concrete during placement and 28-day compressive strength is shown in **Fig. 15**. Although the dispersion in strength is large, the compressive strength has a tendency to decrease inversely proportional to an increase in the temperature of concrete during placement. The amount of decrease in strength due to a temperature increase, however, is not large. If the curing after concrete placement was

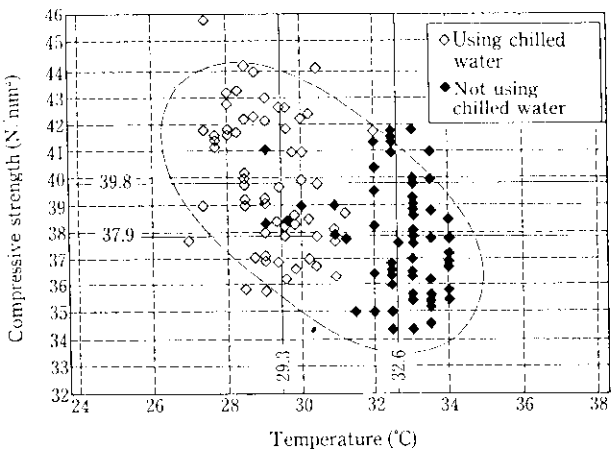


Fig. 15 Relation between concrete temperature and compressive strength

accomplished effectively, then the temperature differential would not pose a problem in the compressive strength.

## 5 Conclusions

This paper reports on the port expansion project executed at Kota Kinabalu in Sabah, Malaysia. The significant aspects of the work can be summarized as follows:

(1) For work in a severe corrosive marine environment in a tropical zone, the KPP pile (heavy duty polyethylene-coating steel pipe pile) presently considered most reliable was proposed as an alternative to the conventional coal tar epoxy coating plus corrosion allowance, which was the corrosion protection method in the original design. This method was adopted successfully for the complete project.

(2) There are only a few worldwide examples in which a large number of KPP piles have been driven into dense subsurface bearing soils in open water. For this project, careful attention was paid to the driving sequence, arrangement of the pile driving ship, and installation of wire ropes for anchoring in order to prevent damage to the coating. The damage was minimized using these measures.

(3) For installation of polyethylene-coated steel pipe piles, it is necessary to take measures to prevent damage not only during pile driving but also throughout the length of the project construction period. The progress of work was not impeded by these measures.

(4) It was possible to determine quantitatively the effect of the driving of closed-end piles on the displacement of existing piles, and a satisfactory concept of soil displacement prediction was determined.

(5) In spite of various limitations, hot-weather concrete was successfully placed at specified temperatures by using a simple chilled water plant.

This project was executed successfully under strict quality control by the design consultant from the United Kingdom, and the construction schedule was very tight. The work was completed in a short period of time, five months less than originally planned. This was primarily due to the cooperation of the parties concerned both in the host country and in Japan. The authors would like to express their sincere gratitude to all concerned.

### References

1) Y. Echigo, T. Sakaki, S. Nakamura, A. Nishiguchi, and M. Kobayashi: *Kawasaki Steel Technical Report*, No. 19 (1988), 113-119

2) H. Sekiguchi, H. Ohta: "Induced Anisotropy and Time Dependency in Clays", The 9th ICSMFE, Speciality Session 9, Tokyo (Japan), (1977), 229-238