

CHAPTER 66

CONSTRUCTION OF NAGOYA STORM-TIDE-PREVENTING BREAKWATER

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ABSTRACT

The Nagoya Storm-Tide-Preventing Breakwater is a large-scale Breakwater built as a link in the general Ise Bay Storm-Tide-Prevention Project. The entire length of this breakwater is about 8,250 meters, which extends from the mouth of the Nabeta River, transversing the northern part of Ise Bay, to the Chita town. (Fig. 4)

The greater half of this breakwater was built on an extremely unstable foundation, the nature of ground being of soft clay stratum with a thickness of 10 m - 30 m. The sand-drain method was adopted for improving the ground on which caissons were laid, forming a composite type section.

For this reason, during the process of construction a close work control was maintained constantly by applying boring, observation on the sinking, pore water pressure and other conditions to ascertain whether the sinking by consolidation took place according to the calculations laid in the design specification. Based on the date obtained in this manner the speed of the work was regulated and the work was conducted under strict control of soil mechanics.

The required volume of materials for the construction of the breakwater was so immense that a special care had to be given to ensure their smooth and efficient supply. And to speed up the work with a view to reducing the effects of wind and waves to a minimum, large machinery were employed. The present paper describes the engineering features of this great 8,250-meters breakwater project which was completed in a short period of 2 years and 8 months at a cost of 30 million dollars after overcoming numerous obstacles.

INTRODUCTION

The country of Japan, by reason of its geographical features and meteorological conditions, is vulnerable to disasters caused by extraordinary natural phenomena.

In winter heavy snow falls visit the areas on the Japan Sea sides in the north which is followed by floods due to the thawing of snows during the months of March and April. In June and July, due to the stationary front persisting along the Japanese archipelago a long-term rainy season called "Tsuyu" develops during which time very often concentrated torrential rains fall in limited regions of the country.

In August and September the Japanese archipelago is again subject to attacks by typhoons with heavy winds and rains. These typhoons with their devastating power cause disastrous damages both in man and property over the entire areas of the country. The damages thus sustained reached an annual average of 660 million dollars for the past 10 years.

The Ise Bay typhoon which attacked the central Japan at mid-night of September 26, 1959 was of the greatest in magnitude in the history of typhoons in Japan. The damages this typhoon caused were the greatest in its history with the loss of 5,000 human lives and 1,400 million dollars in property. In addition a wide area was flooded and the economic function of the disaster area was completely suspended for 40 days.

To forestall the recurrence of the disaster in the future, preventive measures emphasizing the storm-tide prevention were adopted, including the construction of foreshore embankments, sea walls, tide gates, storm-tide breakwater, etc. The Nagoya Storm-Tide Breakwater was a part of this project.

ISE BAY TYPHOON AND ITS DAMAGES

The typhoon No.15, later named as Ise Bank Typhoon, had its origin in a mild tropical atmospheric pressure (1008 mb) which appeared on September 20, 1959 west of the Eniwetok Island. This proceeded westward and later north-north-westward and developed into the typhoon No.15 on September 22. Then later, reaching the ocean about 600 km south-south-west of Iwojima at 15 hrs of September 23, this finally developed into a mammoth type of typhoon with 894 mb central pressure, maximum wind velocity of 75 m/sec and the average of 25 m/sec within 400 km from its center.

At 18 hrs, September 23, the typhoon with 55 km/hr. wind speed at the center landed on the spot 15 km west of the tip of Shio-no Misaki cape. Within the radius of 300 km of the storm area on land, a wind speed of 30 m/sec was observed with the lowest atmospheric pressure of 929.5 mb at Shio-no-Misaki.

This typhoon reached the north of Nagoya City at 22 hrs, September 26, with its central atmospheric pressure of 945 mb, while at

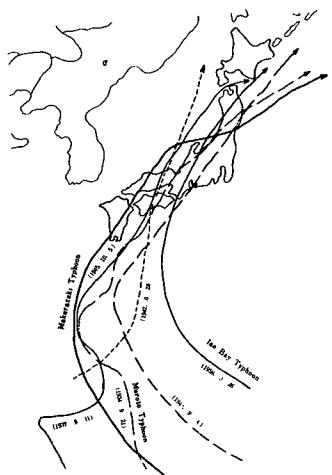


Fig. 1. Courses of the great typhoons.

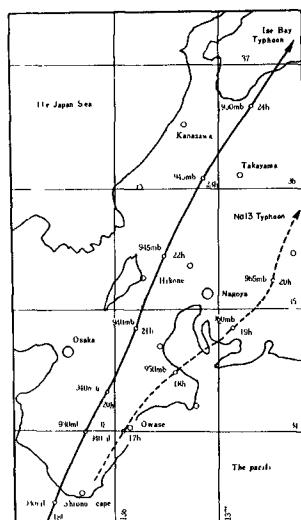


Fig. 2. Course of Ise Bay typhoon.

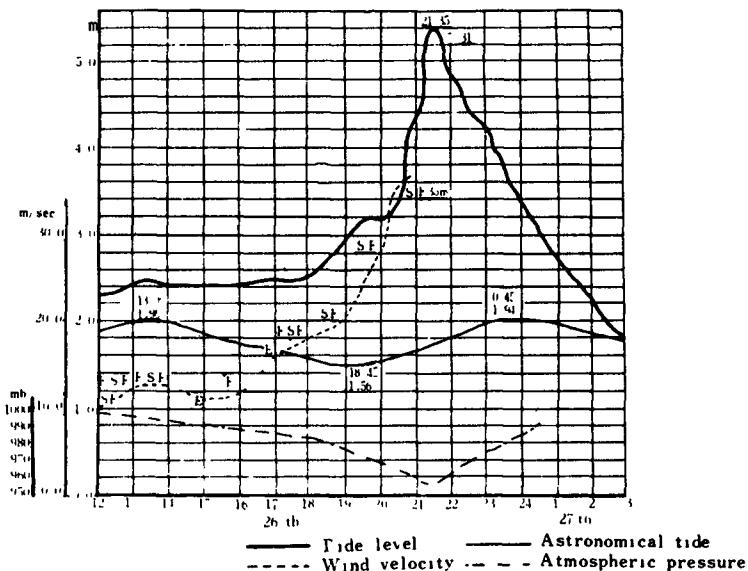


Fig. 3. Height of storm-tide of Ise Bay typhoon at Nagoya port.

Nagoya the lowest atmospheric pressure observed showed 958.5 mb and the maximum wind velocity of 37 m/sec (SSE). (See Fig. 1 and Fig. 2)

As may be noted in the Fig. 1 and Fig. 2, the Ise Bay Typhoon took the worst imaginal course where the high wind area coming from south had concentrated in the Ise Bay while the atmospheric pressure followed a downward grade.

Here in the Nagoya Bay at 21:30 hrs the greatest high tide ever witnessed rose 3.55 m over the estimated tide level which recorded its peak at N.P. +5.31 m.. (See Fig. 3) The highest waves then observed were about 3 m high.

The havoc wrought by the Ise Bay Typhoon had resulted in the largest of the kind in which about 5,000 people lost their lives and the damages caused in property reached a total of 1,400 million dollars, in addition causing the suspension of all industrial and economic functions in the Chu-kyo area for 40 days following the disaster.

The main cause for the damages of such magnitude is, no doubt, due chiefly to the gigantic scale of the typhoon itself but there are other factors which should not be overlooked, such as a large number of floating fallen trees, flooding of coastal banks caused by the extraordinary high-tide and embankment failures which took place at night. Besides, the densely conoentrated population area affected by the disaster happened to be the low land popularly known as the "Zero sea-level" belt.

BREAKWATER PLANNING

The result of the Ise Bay Typhoon that struck the central Japan was, as has been stated above the Chu-kyo area sustained the great damages. In order to forestall the recurrence of such disaster the Japanese government enacted the Storm-Tide Prevention Project.

An integrated general plan was instituted, including the field works as follows: port and harbor, sea coast, rivers, fishing ports, reclaimed land by drainage, reclaimed land, roads and others.

At first it was planned to effect a complete prevention of the sea waters from rushing in by the full restoration of the foreshore embankments, sea walls and tide gates. But it was discovered that these embankments and sea walls would impede the production and transportation activities in the factory area along the coast where the port and harbor facilities existed.

With a view to prevent the damages from storm-tide and waves and

to reduce the impediments as far as possible to the industrial advancement, it was planned to build not only the foreshore embankments and sea walls but an off-shore storm-tide breakwater. Thus doubling the bulwarks against storm-tide and waves, this storm-tide breakwater scheme was considered to be the most effective measure.

This plan consists of a great breakwater (See Fig. 4) which traverses the northern part of the Ise Bay extending for 8,250 meters starting from the Nabeta reclaimed land on the left bank of the Kiso River, and reaches the Chita peninsula on the opposite shore.

As the result of the construction of this storm-tide breakwater the following merits are found:

- (i) By shutting off an extraordinary storm-tide and violent waves caused by typhoons, it reduces the height of storm-tide in the area protected by the breakwater and also lessens the destructive power of waves against the foreshore embankment, thus ensuring a better safety and security.
- (ii) At the time of typhoon, it forms a bulwark for the port-harbor facilities and those of factories in the littoral districts where by functional reasons no foreshore embankments or sea-walls could be erected.
- (iii) A general merit resulting from the breakwater is that by maintaining an undisturbed water surface at all times it positively serves for the promotion of port and harbor functions in the Nagoya port.
- (iv) The land reclaiming work is made easier over a wide water area protected by the breakwater. It enables to lower the level of the reclaimed land and to make the construction of protecting walls simpler. A reduction in the cost is made possible for the preparation of the site for coastal factories. Further, the front of the reclaimed land can be utilized for a quiet anchorage. In this way, the progress was made in the formation of coastal industrial zones in the Chu-kyo area.

But this plan, on the other hand, presented problems as follows:

- a) As the land reclaiming work made progress it was feared that it might give an unfavorable effect on the disaster prevention by lessening the merit of reducing a height of storm-tide.
- b) Another fear was that at the entrance to the breakwater some heavy tides might interfere in the navigation of vessels.

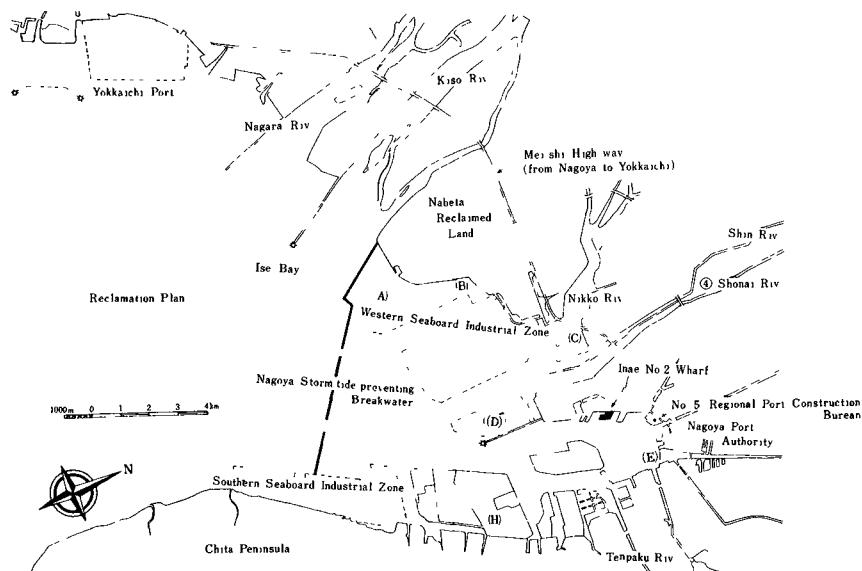


Fig. 4. Plan of Nagoya port.

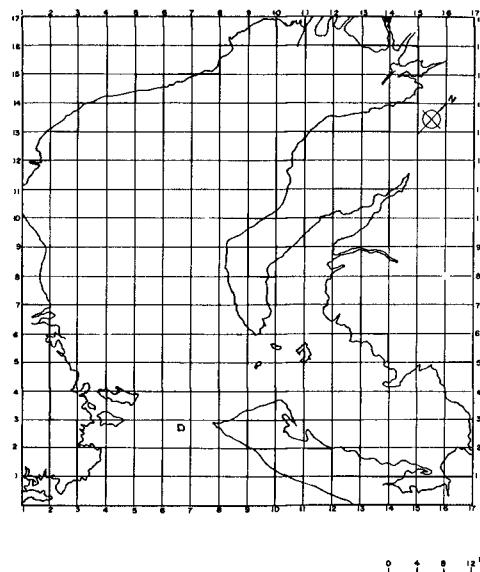


Fig. 5. The region of the tidal variation for the estimated typhoon by a numerical analysis using the computer.

These two problems, however, were satisfactorily settled as follows: The first problem was overcome by building a big-scale reclaimed land in front of the shore embankment to ensure safety from a disaster.

Since the second problem had no direct bearing on the disaster prevention, it was proposed to widen the port entrance to the breakwater so as to remove the impediments to the vessels' navigation. Having these two measures incorporated in the plan the decision was reached on the Storm-Tide Breakwater Construction Project.

INVESTIGATION OF BREAKWATER

ESTIMATION OF STORM TIDE AND WAVES BY USING MODELLED TYPHOON

An estimate of the storm tide has been made through two dimensional computation of the storm tide, using an electronic computer for the area inwards the mouth of the Ise Bay, viz., the area including the Ise Bay, the Atsumi Bay and the Chita Bay.

The Ise Bay Typhoon, as idealized, has been modelled for use in the above computations, with an assumption that it would take northward course similar to that of the Ise Bay Typhoon.

The grid clearance employed in this case was 2 km, while the time step was 55 sec. The distribution of atmospheric pressure was obtained from K. Takahashi's Formula:

$$P = P_o - \frac{P}{1 + \frac{r}{r_o}} \quad (\text{Wherein})$$

P_o : Atmospheric pressure at the outer-most circular isobaric lines of typhoon
 r_o : Radius of P_o
 P : P_o - Atmospheric pressure at the center of typhoon

Distribution of wind velocity was computed based on the above formula. The wind direction within the bay was assumed constant at the same time of the day.

As the result of the computation, it was made clear that the deviation of storm tide would be approximately 0.5 m lower in height to become 3.05 m when a breakwater would be built. (See Table 1)

The wave height and period have been estimated by use of the Sverdrup-Munk-Bretschneider Method, Sakamoto-Ijima Method etc. (These are the methods of estimating shallow-water waves in consider-

ration of the effect of the friction of water at the sea bottom. The shallow-sea wave height and period are derived from the Bretschneider method which serves for the estimation of the value of shallow-water wave height and period, while B. Wilson's computation of finding shallow-water wave for migratory minor wind area is extended for shallow-water waves.) The results showed that the effect of the breakwater on reduction of the wave-height would be 70 to 130 centimeters. (See Table 1)

TABLE 1. Effect of Nagoya Storm Tide Preventing Breakwater on Height of Tide and Waves

Name of Area	Designation	Without Breakwater		With Breakwater		Effect by Breakwater	
		Tidal Deviation (m)	Estimated Wave-Height (m)	Tidal Deviation (m)	Estimated Wave-Height (m)	Tidal Deviation (m)	Wave Height (m)
Nabeta	A	3.55	2.90	3.05	2.20	0.5	0.7
Umibe	B	3.55	2.90	3.05	2.10	0.5	0.8
Nanyo	C	3.55	2.90	3.05	2.10	0.5	0.8
Within Nagoya Harbor	D	3.55	2.90	3.05	1.60	0.5	1.3
	E	3.55	1.45	3.05	0.64	0.5	0.9

(Note: For designations A, B, C, D and E, see Fig. 4)

It was also found that the effect of the storm-tide preventing breakwater on reducing the height of storm-tide and waves would eventually reduce by more than 1 m the height of sea embankment and seawall inside the breakwater.

GEOLOGICAL INVESTIGATION

Exploratory boring have been conducted at 48 points in the total length of 1,741 meters along the line of the storm tide preventing breakwater. The soil profile along the line of the breakwater is shown in Fig. 6. As evidenced from Fig. 6, the area within 3 km west-side of Nabeta Bank is shallow waters with 5 to 8 meters thick sand layer overlying the bottom, while the area within several hundred meters eastside of Chita-Bank has thin layers of diluvial soil, both of which layers are favorable for construction of breakwater. The remaining portion, approximately 4.5 km including the Central Bank has considerable depth of 8 m to the sea bottom with soft layer extending to -20 m - -40 m covering the bottom, offering unfavorable conditions for construction works.

Thin-wall samples has been taken from this soft layer consisting of silty clay, at every 1 m along the line of the proposed breakwater.

The entire sample materials were subjected to measurement for unit weight and moisture content and to unconfined compression test. Approximately one fifth of the entire sample materials was also made the subject of consolidation test, measurement for Atterberg's limit, mechanical analysis and measurement of soil grain. A portion of the materials was further studied by triaxial compression test, single shear test, disturbance and recovery tests.

The shear strength of the silty clay, as the result of the foregoing tests, is shown on Fig. 7, where $C = 0.44 + 0.155Z$ at the Central Bank and Nabeta Drain, and $C = 0.5 + 0.18Z$ at Chita Drain (Z : Depth in meters below the sea bottom, C : represented in t/m^2) The consolidation characteristics was found to be as shown on Table 2.

TABLE 2. Consolidation Characteristics of Soil

Classification	Consolidation Coefficient C_v (cm^2/min)	Volumetric Shrinkage m_v (cm^2/kg)	
		N.P. - 1 kg/cm ²	N.P. - 10 kg/cm ²
Clay I	0.10	0.115	0.0130
Clay II	0.13	0.085	0.0110
Clay III	0.30	0.072	0.0093
Sandy Silt	1.30	0.032	0.0054

DESIGN OF BREAKWATER

DESIGN CONDITION

Based on the results of the investigations mentioned in the foregoing chapter, and observation data of the past, including those of the Ise Bay Typhoon, the following design conditions have been established.

Ocean-meteorological condition

- (i) Wave Direction: At right angle to the line of breakwater (S - SW)
 - Period: 7 sec.
 - Height: At Nabeta Bank 2.7 m
At Central Bank 3.0 m
At Chita Bank 2.5 m
- (ii) Tide L.W.L. : ± 0 m
H.W.L. : + 2.6 m
H.W.L. during typhoon season: +2.3 m
- (iii) Tidal Deviation: At Nabeta Bank (On western end) 3.55 m
At Central and Chita Banks 2.40 m

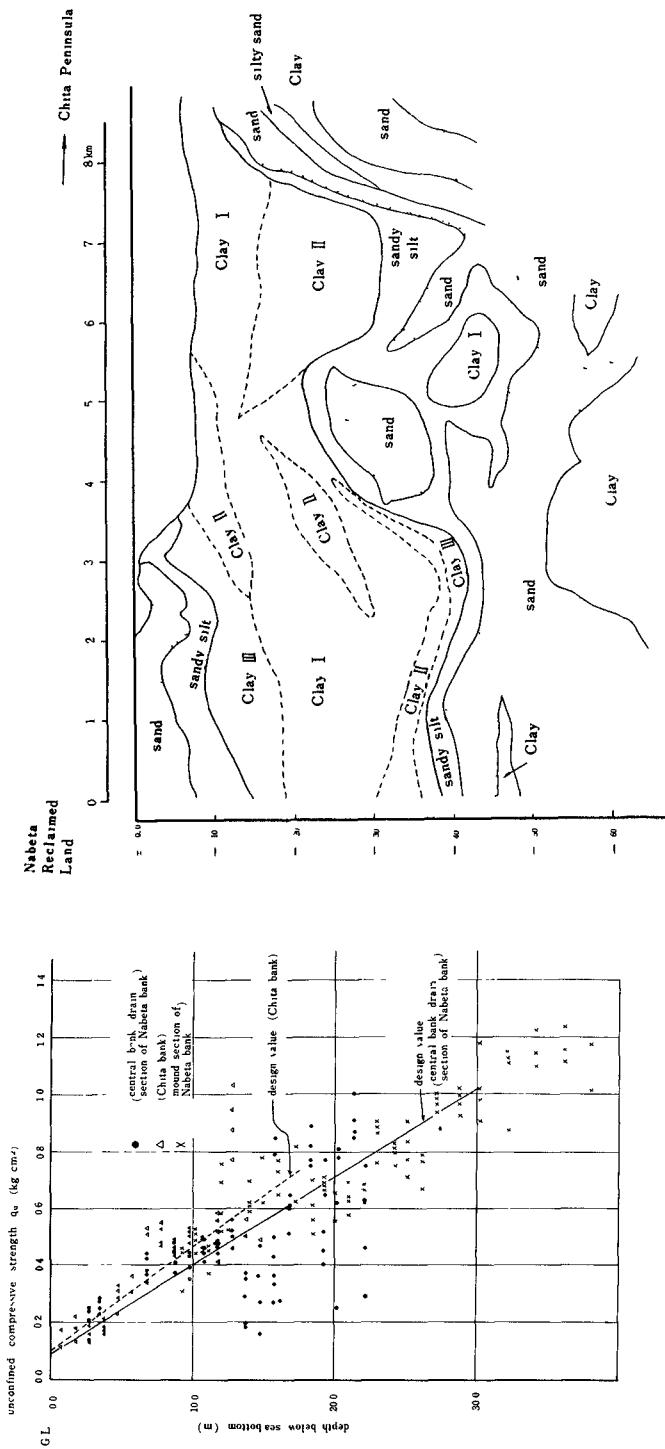


Fig. 7. Example of the initial strength of the clay foundation.

Fig. 6. Soil profile along the breakwater.

- (iv) Difference of water level between inside and outside the breakwater: $\pm 1.0 \text{ m}$
- (v) Crown level of breakwater ($= \text{H.W.L. during typhoon season} + \text{Storm Tide deviation} + \text{wave-height} \times 0.6 \text{ m}$)

At Nabeta Bank (on west end)	+7.5 m
At Central and Chita Bank	+6.5 m

Soil condition

- (i) Drain Section of Central and Nabeta Bank

Classification :	Silty clay
Cohesion :	$C = 0.44 + 0.155Z \text{ (t/m}^2\text{)}$ (Z : depth below the sea bottom)
Submerged unit weight :	$\gamma = 0.55 \text{ (t/m}^3\text{)}$
Strength increase ratio:	$C/P = 0.28$
Over-consolidation load:	$P_o = 1.6 \text{ (t/m}^2\text{)}$
- (ii) Drain Section of Chita Bank

Classification :	Silty clay
Cohesion :	$C = 0.5 + 0.18Z \text{ (t/m}^2\text{)}$
Submerged unit weight :	$\gamma' = 0.6 \text{ (t/m}^3\text{)}$
Strength increase ratio:	$C/P = 0.3$
Over-consolidation load:	$P_o = 1.67 \text{ (t/m}^2\text{)}$
- (iii) Mound Section (Nabeta Bank)

Classification :	Sand ($z_0 \text{ m} - -10 \text{ m}$)
	Silty clay ($-10 \text{ m} - -40 \text{ m}$)
Angle of internal friction :	$\phi = 30^\circ$
	$\gamma' = 1.0 \text{ t/m}^3$

Other conditions

Angle of internal friction of mound : $\phi = 35^\circ$
 Unit weight of mound (Sand and Stone) : 2.0 t/m^3
 Reinforced concrete : 2.45 t/m^3
 Concrete: 2.30 t/m^3
 Sea water: 1.03 t/m^3

Besides design conditions mentioned above, however, the followings were the prerequisites which had to be taken into consideration at the start of the design of the storm tide preventing breakwater.

- (i) In addition to calming down and shutting off the wave energy of ordinary type, the proposed breakwater is assigned the function of reducing the height of storm tide. Therefore, suction acting upon the foundation caused by the hydrostatic difference between inside and outside of the breakwater must be taken into consideration.
- (ii) Most of the supporting soil consists of soft clay, and the depth to the sea bottom is approximately 8 meters.
- (iii) Time available for construction is limited, and accordingly a considerable speed of execution is required. However,

this is handicapped by a considerable distance of several kilometers from the seashore to the construction site, where wind and waves are strong, again hampering application of delicate construction techniques.

- (iv) The total length of the breakwater being of large magnitude with short construction period, a large amount of construction materials has to be delivered to the site in a very limited time.

COMPARATIVE STUDY

As mentioned in the foregoing chapter, the soil condition along the line of the breakwater varies from place to place. Considering these varieties, the entire breakwater was divided into four sections (A, B, C and D) as shown in Fig. 8. The geometry of each section of the breakwater was determined to suit the peculiarity of the soil conditions.

700 m-long-Section (A) east of Chita Bank —— (Exchanged Section)

Unlike in the area of Central Bank, the 700 m-long Section east of Chita Bank has a comparatively thin layer of soft clay and favoured with a shallow water. (See Fig. 6)

This led to the possibility of applying two methods of improving the surface layer of soft clay - one to consolidate the clay layer by sand drain, the other to replace the clay layer with sand after excavating and removing the clay.

The former method is for building a composite type (caisson) breakwater on a sand-drain foundation, while the latter for constructing the same on a sand replaced foundation. Comparison of the two methods has proved that the latter, in which the depth of soil to be replaced reaches 17 meters below the sea bottom (composite type breakwater on the sand replaced foundation) is more economical and safe. Therefore, this type (See Fig. 9) was adopted for the 700 m-long-section (A) of Chita Bank with the total length of 1,555 meters.

3,100 m-long-section (D) of the western part of Nabeta Bank

As seen from Fig. 6, the western part of Nabeta Bank is favored with shallow water and good soil, and consequently, many plans have been recommended, including a stepped earth-mound, a rubble-mound, an L-type, and a steel sheet shell type breakwater. Upon careful consideration of such factors as economy, easiness of construction, construction time and others, a stepped earth-mound breakwater (See Fig. 10) has finally been adopted.

4,200 m-long-section (B) of Central Portion

The 4,200 m-long section of Central Portion, extending from Chita Bank to Central and Nabeta Banks has considerable depth of approximately 8 m to the sea bottom, which is coveredly a soft clay layer of 12 to 30 meters. To cope with the conditions, many sectional shapes, in addition to those mentioned in the foregoing paragraph, have been studied. Finally, the following four plans were taken up for further comparative study.

- (i) Composite (caisson) breakwater on a replaced foundation.
- (ii) Earth-mound breakwater on a sand-drain foundation.
- (iii) Light-weight steel structure (Fig. 11) on a sand-drain foundation.
- (iv) Composite breakwater (Fig. 12) on a sand-drain foundation.

With careful studies on their economy, construction time, feasibility, availability of material, the plan (iii) was found to be the cheapest but many difficulties were foreseen in structural point as well as in construction method. Meanwhile, plans (i) and (ii) were deemed costly and to require a great deal of construction materials whose procurement in a short span of time would be near impossibility. This led to the adoption of a composite (caisson) type breakwater on a sand-drain foundation. Even with this structure, 440 m^3 of sand and 240 m^3 of rubble are required for a unit length of 1 m.

Transient portion (C)

The transient portion between the composite breakwater and the stepped earth-mound breakwater on a sand-drain foundation situates itself in a shallow water of more or less 2 m with the bottom covered with a thin layer of sand.

Consequently, a concrete block breakwater system was adopted owing to the merit of its easiness of construction, as shown in Fig. 13.

The four sections mentioned above have thus been studied extensively by determining the various sectional shapes. The composite type breakwater on a sand-drain foundation for central portion, which posed numerous problems in the process of design and construction, is worthy of some description.

DESIGN OF COMPOSITE TYPE BREAKWATER ON SAND DRAIN FOUNDATION

Design of caisson and facing rubble

The caissons are of ordinary type and are not necessarily stressed in particular. However, the side walls of caissons are designed as a fixed slab on three sides. Arrangement of reinforcing steel are designed to meet the requirements of wave impacts, hydrostatic head

and towing the caissons to the site. The weight of rubbles was determined by the Iribarren-Hudson Formula.

Designing of sand-drain

The consolidation period must be shortened as much as possible in order to complete the breakwater within a short period of work. For this purpose it is desirable to narrow down the distance between sand-piles. However, in consideration of the work-efficiency in the open sea a square-type allocation with a pile separation of 2 meters was adopted.

The diameter of the sand-pile has relatively little bearing on the consolidation period. Therefore, 45 cm was adopted to facilitate the work. The work depth of the sand drain was fixed at -20 m taking into consideration the thickness of the clay layer, operation machines and local peculiarities.

Stabilization of the breakwater body

In the case of the breakwater for which the foundation is improved by sand-drain its stabilization was studied per each stage of the work including the time of completion in order to carry out the work in pieces.

For calculating the stabilization the circular sliding system was used in order to obtain a safety factor at the time of typhoon after the completion of the breakwater body as well as at each stage of the work. However, this safety factor is the most important problem. In the case of such a large-scale breakwater a destruction test through an experimental breakwater should have been carried out in order to confirm the degree of safety. However, this was impossible due to the short period of work. Therefore, we adopted a safety factor of more than 1.5 at each stage of work taking into consideration the past destruction examples, side resistance, permeation pressure, other possible errors in calculation as well as work conditions. However, at the first stage of work, an increase of strength is not expected in making calculation. On the other hand because it is considered that the strength will increase during the period of work which is comparatively long, the safety percentage was lowered to 1.4. As for local sliding which occurs along small circles there is a problem in the method of calculation itself. Since such circles pass through only a small portion of the clay foundation, the safety percentage was allowed up to 1.3.

As for the safety percentage at the time when the strength of 3 m of designed wave height is applied to the caisson, it was lowered to 1.2 in due consideration that the wave strength was not applied simultaneously to the whole length; that it was only instantaneous; and that the strength or reinforcement was considerably increased outside the area of sand-drain, etc. (See Table 3)

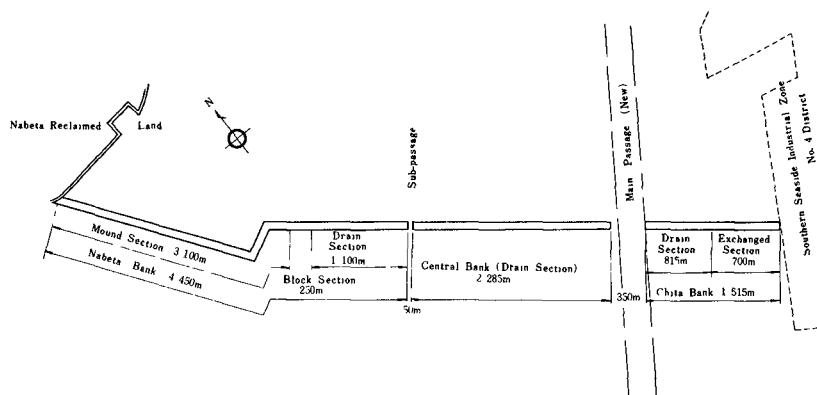
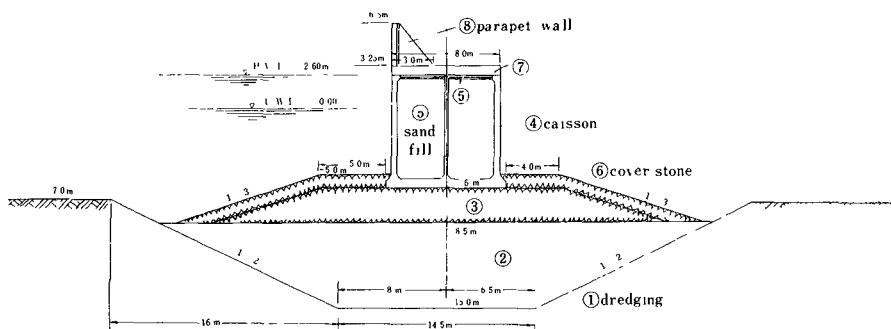


Fig. 8. Plan of the breakwater.



(1), (2) order of construction

Fig. 9. Standard cross section of exchanged section.

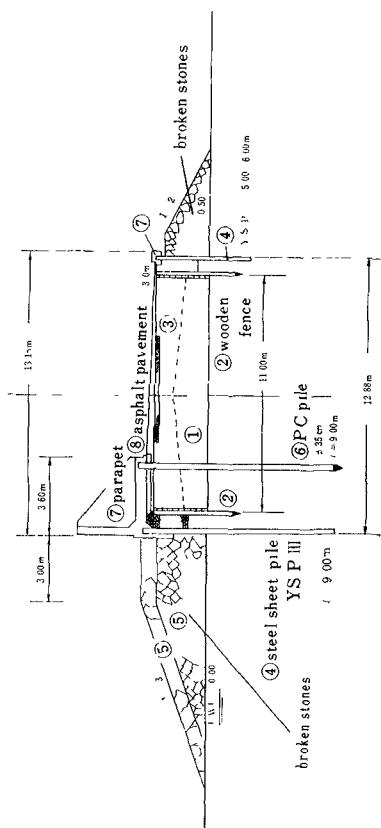


Fig. 10. Standard cross section of mound section.

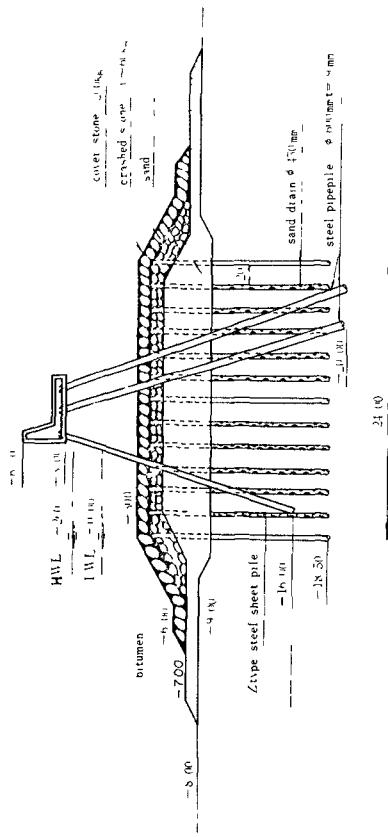


Fig. 11. Light steel structure built on the sand-drained foundation.

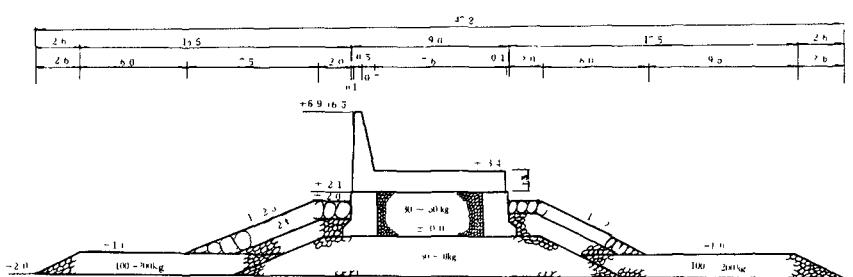


Fig. 13. Block structure section of Nabeta bank of the breakwater.

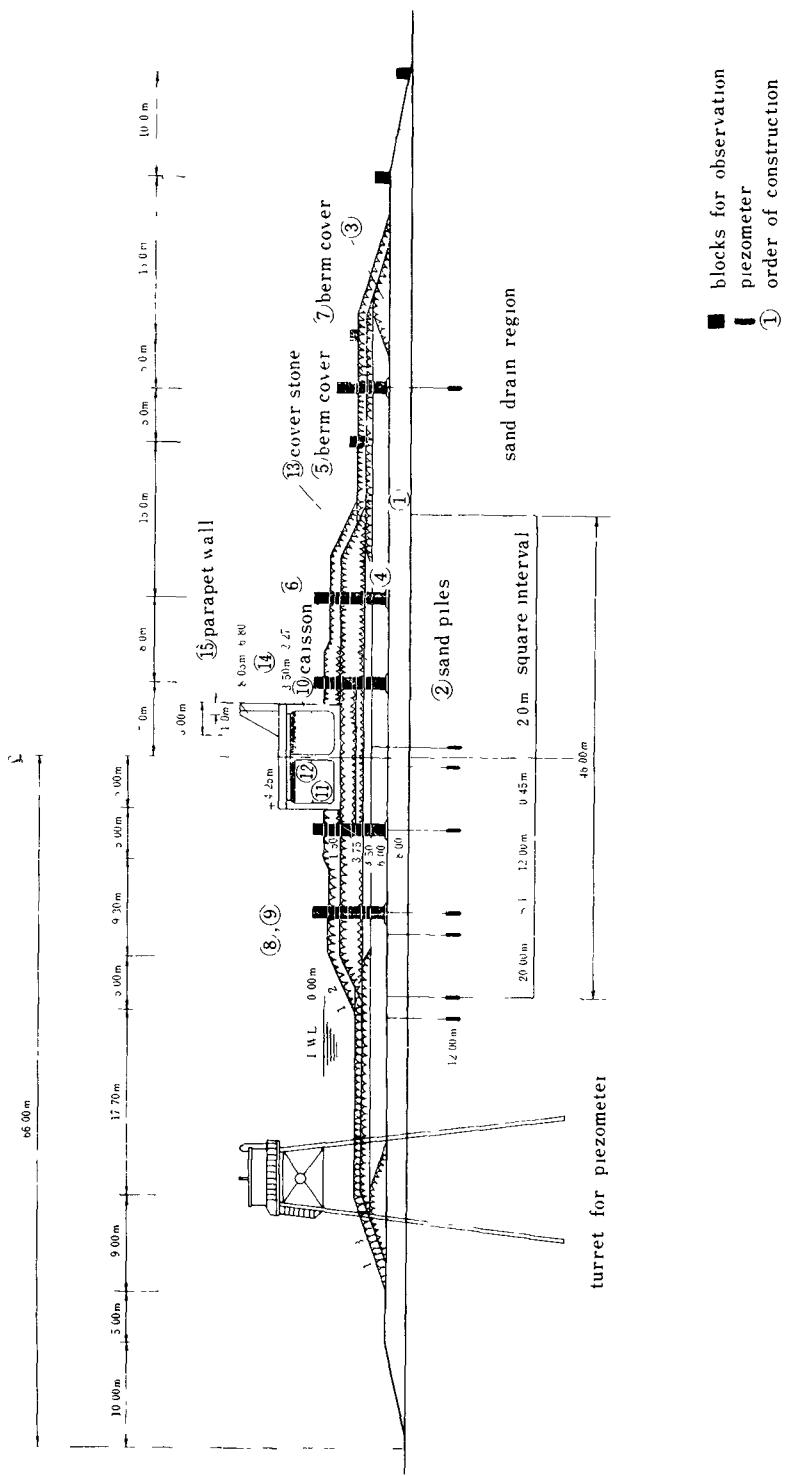


Fig. 12. Standard cross section of drain section. (Arrangement of investigation devices at 900m point)

The sinking was calculated based on the idea that it was devided into the area of sand-drain and the lower clay layer in the direction of the depth and that one sided drainage occurs in the area of sand-pile while two-side drainage occurs in the area of clay layer depending on the respective ratio of volume compressibility (m_v).

TABLE 3. Work Stages and Results of Safety Factor Calculation

Work Stage	Work itself	Period of work	Safety percentage		Note
			local sliding	total sliding	
I	Sand & gravel part Fig 1 - 9		-	1.43	
II	Setting of caisson, half of the filling 10 - 11	About 70 days after stage I (consolidation percentage about 80%)	1.63	1.71	
III	Filling of the inside, remainder of sand, rubbles, cover concrete, 12 - 13	About 30 days after stage II (about 50%)	(1.33)	1.51	[] in case there is no covering stone
IV	Upper part concrete parapet 14 - 15	140 days after stage III (about 100%)	1.41	1.51	
After completion		140 days after stage IV	1.46	1.24	Wave strength to caisson is considered.

CONSTRUCTION WORK OF THE BREAKWATER

ORDER OF CONSTRUCTION WORK

Construction work of the "exchange breakwater", "the shelf-type soil and sand breakwater" etc. was carried out in the order of 1 - 8 as shown in Fig. 9, Fig. 10., Each construction work was not so dif-

ficult but the sand drain breakwater must be constructed while paying attention to the stability of the breakwater body, the construction work was divided into 4 stages sufficiently applying the work management based on soil mechanics.

The construction work order of the sand-drain breakwater is shown in 1 - 15 in Fig. 12. First, sand with the thickness of 2 m is applied to the local foundation. Then a sand-pile is driven down to -20 m. After the driving of the sand-pile the breakwater is constructed in the order of sand-preventing embankment, the 1st layer of sand with the berm protection of thickness of 1.5 m, the second sand layer and broken stones in the central part -1.5 m from top end. In this way, this stage is completed. If the caisson is lowered immediately after this, there is a danger for a circle sliding to take place. Therefore, the work is suspended until after 70 days when the clay is consolidated 80% in the sand-drain area through the weight of the broken stone embankment. By this time the body itself sinks more than about 60 cm through consolidation of the clay layer.

As for the work stage II, if the caisson is lowered completely and the filling up work is completed, there is a danger of failure by circle sliding. Therefore, the work is stopped after the caisson is lowered and half of the filling is completed. After an elapse of about 30 days, the clay in the sand-drain area is consolidated up to 90% due to the weight of the embankment, and the consolidation due to the weight of the 2nd work stage reached 50%. At this time "subslub embankment" sinks down to about 80 cm and the caisson sinks down to about 20 cm. In this way the 3rd stage work can be started.

The 3rd stage work is finished when the remainder of the filling of the caisson and placing of concrete for the cover are completed. After the elapse of about 140 days after this, the clay in the sand-drain area is consolidated up to about 100% due to the weight respectively of I, II, III stage works. In this way the 4th stage work can be started. At that time the subslub embankment sinks down to about 1.2 m and the caisson sinks down to about 60 cm.

At the end of the 4th stage, the whole work is completed by placing the upper part concrete and the parapet. However, the sinking of the body after its completion will still continue, because while the consolidation of the clay layer in the sand-drain area is almost completed the consolidation of the lower clay layer is not yet finished. For this reason, the top end height of parapet is made higher by the degree of sinking upon estimating the volume of future sinking.

CONTROL OF THE WORK

In constructing the storm-tide-preventing breakwater there were two difficult problems characteristic of the soft foundation to which some reference was made before, in addition to problems characteristic of the ordinary work of breakwater. One is the problem of the stab-

lity of the breakwater body and the other is the sinking of the body. It is unthinkable that such consolidation phenomena as the additional reinforcement in the foundation which is taken into consideration in stability calculation, the sinking which affects the top end height of the breakwater etc. may not happen according to the program. Also the load conditions, such as the unit volume weight, sectional shapes etc. are not as accurate as those on the land. Therefore, a work control based on the so-called soil engineering such as the observation during the work and the adjustment of work calculation based on this observation was necessary. However, it was extremely difficult to estimate future volume of sinking accurately through work control and to decide the top end height of parapet of the breakwater.

Check-borings

In order to check whether the reinforcement of the foundation is advancing as estimated, check-boring was conducted at each work stage on more than 10 places to the length of the breakwater at points 8 m (part affected by the weight of caisson), 16 m and 35 m (outside sand-drain) distant from the normal line of the breakwater. (Refer to Fig. 8) Number of boring was 69, total digging length was 1,296 m and sampling was made at 869 places. The boring was made in such a way that concrete blocks with hole were placed in the part of the subslab embankment in advance, so that the subslab part may easily be penetrated, and the sampling was accomplished without touching the sand-pile. As for the soil experiment, unconfined compression (q_n), water containing ratio (w), unit volume weight (γ) etc. were adopted.

One example of reinforcement of the foundation is shown in Fig. 14.

Observation of the sinking

The sinking of the breakwater body can be divided into the sinking of the mound part and that of the caisson. The sinking of the mound part was measured by leveling from the survey tower a staff placed on the concrete block (in order to make it the same weight as the apparent weight of the subslab stones) with hole, using a diver in the same way as the boring. The measuring points are the same as the boring points. However, after it was found later that there was an extraordinary sinking outside the sand-drain area, observation was also made at points 30 m, 40 m, 55 m and 65 m distant from the normal line of the breakwater. The number of observations in the subslab part was 62 and caissons amounted to 305 cases. One example of the observation result is shown in Fig. 15.

Pore pressure measurements

The consolidation condition was investigated by burying 16 pore pressure meters to the depth of -12 at the point of 900 m of the central embankment. The distance from the normal line of the breakwater is the same with that of the boring. However, measuring was

made up to the points 23 m distant from the normal line. The pore pressure meters included 15 each of Manometer type and one electric resisting type. One example of the measurement result is shown in Fig. 16. However, it was impossible to apply the analysis of the measurement result to actual work management because there were many unsolved problems in time lag, consolidation analysis method, etc.

Other observations

The unit volume weight of sand and the weight of dead subslab stones were frequently measured because such rubble and sand in the subslab mound strongly affect the stability of the breakwater body.

In addition, the height of the tide, the height of the wave, the speed and direction of wind etc. at the construction points of the breakwater were observed for reference.

TABLE 4. Principal Materials for Storm Tide Preventing Breakwater

Work Parts	Foundation sand	Sand pile	Stone	Caisson	Concrete dispatched in site	Cement	Reinforcing bar	Steel sheet pile	Concrete pile	Asphalt	Labor
Central embankment	m ³		m ³	cases	m ³	t	t			t	persons
Central part	990,000	27,204	650,000	140	25,000						
Chita part	330,000	9,800	270,000	49	9,000						
Nabeta part	260,000	13,690	300,000	67	12,000						
Connecting part			30,000	Concrete lump 134	3,500						
Nabeta embankment	400,000		100,000		16,000			15,769	2,934	23,000	
Chita embankment	100,000		90,000	49	7,000						
Total	2,050,000	50,694 (about 230,000 m ³)	1,440,000	305	72,500	41,000	5,800	15,769	2,934	23,000	973,000

SUPPLY OF THE WORK MATERIALS

Principal materials used for this breakwater are shown in Table 4.

The total amount of sand used was about 2,550,000 m³. It became impossible to obtain good sand from the place as planned in the original schedule. We were forced to obtain most of the sand of somewhat inferior quality from inside the harbor except the sand to be used for sand-drain. The sand was obtained by pumping ships. Silt part con-

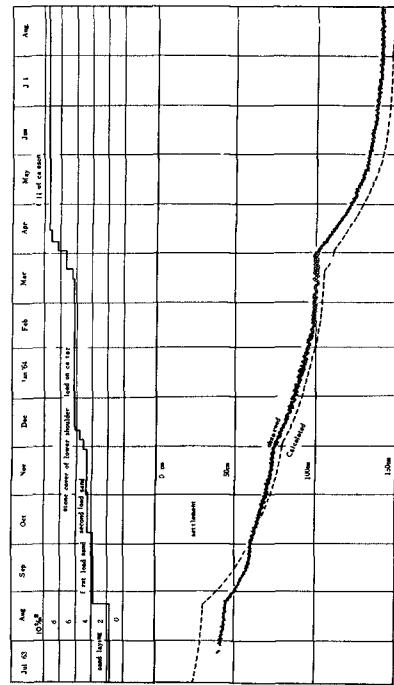
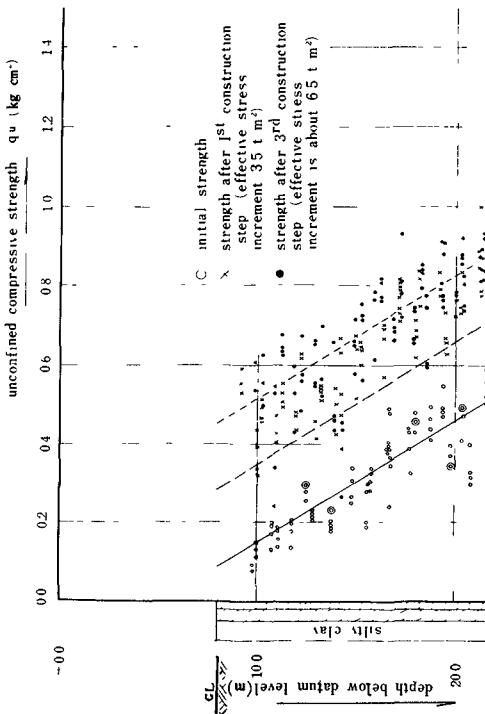


Fig. 15. Example of settlement analysis at 900m point (in sand drain region).

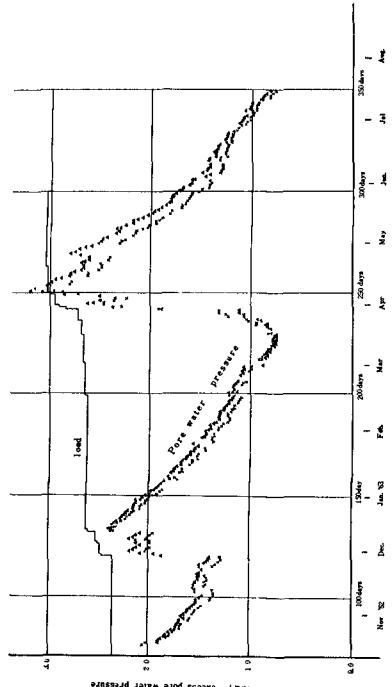


Fig. 16. Examples of pore pressure measurement.

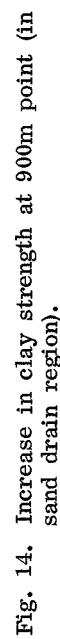


Fig. 14. Increase in clay strength at 900m point (in sand drain region).

tained in the same was eliminated by overflowing before loading on the receiving ships. As for the work of spreading the same for sand-drain, the sand was spread by barge-unloader-pump ships, namely, the method to spread same without damaging the soft foundation on the sea bottom. Foundation sand and exchange sand were placed directly by bottom-opening-soil-carrier ships because the base foundation is not damaged even if they are dumped directly.

The total amount of subslub stones is amounted to about 1,440,000 m³. This was more than 2 times the past supply capacity nearby. Because those stones were also greatly needed for other works, not only private carrier ships were hired but also the state constructed more than 4 loading centers in addition to newly building 34 side-opening carrier ships.

OPERATING MACHINERIES

It was necessary to complete the storm-tide-preventing breakwater within a short period of time. We adopted mostly the sand-drain method for the breakwater and for the improvement of the foundation. Therefore, it was necessary to wait for the consolidation of the clay foundation at each load-work stage. The work spots were located off the shore and the work was easily affected by wind or waves. For this reason larger operating machineries were needed in order to increase the work speed without being effected by wind or wave.

The total number of sand-pile was about 50,000. If we used the conventional sand-drain ships for driving sand-drain piles it would take longer time and would easily be affected by wind and wave. Further it was impossible to use too many ships because of the nature of the work places. For this reason, sand-drain ship "Soryu" of 1200 HP was built (Fig. 18). "Soryu" is equipped with 4 pile-driving towers capable of moving along grooves of 54 m in length and the piles are driven by vibro-hammer of V-3 type. This makes it possible to drive 4 sand-piles at one time. It is also possible to drive 28 sand-piles simply moving the towers, without moving the ship itself which takes time. The actual result shows that 350 piles were driven in one day at the maximum and it was 130 piles per day on the average. Because of the big size of the ship the work was carried out without difficulty even under conditions of wind velocity of 7 m/sec, wave height 0.5 m and tidal current 2 knots.

The amount of concrete needed for the upper part of the caissons and parapet was about 70,000 m³. Out of this amount about 50,000 m³ was needed on the sea. The period of its work was as short as about 10 months. Therefore, it was necessary to place such a great quantity of concrete as about 250 m³ per day on the average. On the other hand, because it was necessary to have a sufficient quality control and use good concrete even if it involved works on the sea, contractors built a big concrete mixer ship for the first time in Japan. The particulars of the ship are 40 m in length, 13.4 m in width and 1.4 m in

draught. Cement, gravel and sand are transported by a group of supply ships. It is capable to store 40 tons of cement, 220 m³ of aggregate and 50 m³ of water inside the ship. The concrete mixer is capable of producing concrete of about 40 m³/hr., carrying fresh concrete to the required places through belt conveyor. In this way the work progressed smoothly.

On the other hand, in order to produce 305 caissons, a slip-type caisson yard capable of producing 24 caissons simultaneously and with a floor space of 48,000 m² was constructed.

WORK PROCESS

The work was carried out making full use of the above-mentioned materials, work ships, machineries and facilities and under careful work process control. Actual work results per each work part are shown in Fig. 17.

In September, 1964 the storm-tide preventing breakwater of 8,250 m in length was completed without mishap within a short work period of 2 years and 8 months and using expenses amounting to about \$30,000,000 and 973,000 workers. At present, reclamation in the harbor and construction of the industrial area along the shore are steadily progressing.

COST

The total cost of the breakwater project amounted to approximately 30 million dollar and 36 hundred dollar per meter length of breakwater. The cost of each bank of breakwater is showed in Table 5.

Table 5. Cost of Each Bank of Breakwater

Name of Bank	Length (m)	Cost (Dollar)	
		Per Meter	Full Length
Nabeta Bank	3,350	1,460	4,900,000
Chita Bank	740	4,170	3,060,000
Central Bank	4,160	4,280	22,000,000

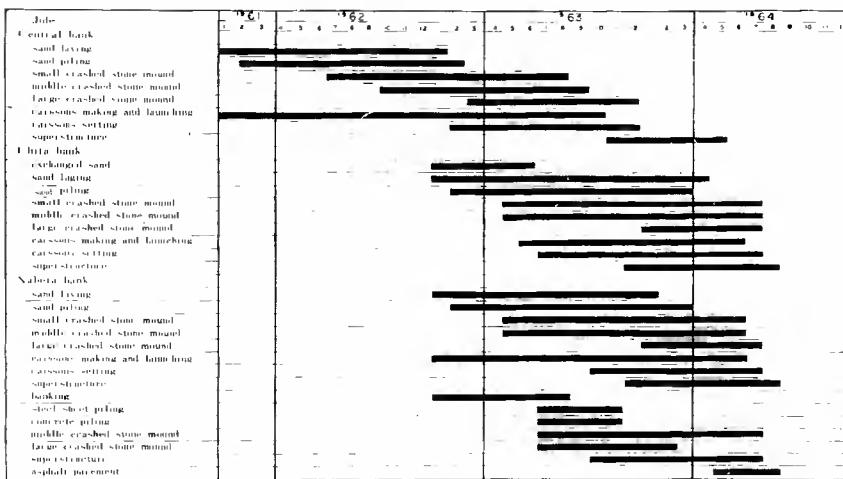


Fig. 17. Working schedule.



Fig. 18. "SORYU" craft for sand piling.

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