REPORT
ON
SECOND HARBOUR ENTRANCE PROJECT
OF KAOHSIUNG HARBOUR

August 1968

Overseas Technical Cooperation Agency

Tokyo, Japan.

Preface

The Kaohsiung Harbour Bureau, Taiwan Provincial Government, has taken up the Second Entrance Project to open a new entrance at Kaohsiung Harbour.

The Government of Republic of China, wishing to have technical assistance on selection of break water type and construction plan and layout of the project, requested the Government of Japan to provide the services of the experts in this field under the technical cooperation scheme.

On this background, the Overseas Technical Cooperation Agency despatched the experts to the Kaohsiung Harbour Bureau to assign them to study economical and technical feasibility of the project.

This is the report especially prepared by Mr. Z. Wada Bureau for Ports and Harbours, Ministry of Transport, who engaged in consultation work on the technical aspect of the project for use of the Kaohsiung Harbour Bureau.

It is hoped that this report will help the Bureau succeed the Project.

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REPORT ON SECOND HARBOUR ENTRANCE PROJECT OF KAOHSIUNG HARBOUR

Contents

I Summary of WADA Report (May 1966)

1. Layout of Breakwaters

1-1. The layout of breakwaters for the Second Harbour Entrance should be determined after due consideration that the breakwater would serve as a shelter against waves and that a 75,000 DWT large-vessel scheduled to come into this port could be maneuvered smoothly. (Fig. - 1)

The outline of this layout is as follows.

South Breakwater runs 1,500 m towards W-direction from shore-line and then turns 30° to N - direction with 700 m. Total length is 2,200 m and the depth of the Breakwater head is - 12.5m.

North Breakwater consists of 1,150 m normal to shore-line. The depth of the Breakwater head is - 10.5 m.

1-2. The layout of South Breakwater (main breakwater) is determined after due consideration against waves as follows;

Waves under direct influence of typhoon attacking from S - W direction, and waves under indirect influence of typhoon from S - SW direction.

In case of the Original Plan, the layout was considered not to be effective against SW waves. Moreover waves generated at ordinary time have, as a matter of course, SW-direction (normal to shore-line). So the layout is determined by rectifying the pattern of this Original Plan.

Waves due to winter monsoon are estimated to be frequent in direction WNW $\sim$ WSW with the center W direction and 2 m in its maximum height.

North-Breakwater is laid out to be against these waves, but it might be seemed not to be so much effective because the Entrance is opened to W-direction.

However this Entrance is considered out of danger against these waves by reason of that the existing First Entrance, as a proto-type model, has stood without any dammage for long time.

It will be recommended to consider further about this problem by conducting model test.

1-3. It is very dangerous that a large-vessel sets astern and stops with its own control force inside break waters. So usually tug-boats are used to pull such a large-vessel from the enterance.

When a 75,000 DWT vessel comes in by 5 knot, generally it is necessary to keep the "stopping distance" about 2,300 m $\sim$2,500 m even if she might be assisted by tug-boats. In case of the Original Plan the stopping distance is presumed only about 2,200 m and a little length lacked and the inner port channel is presumed to run per pendicularly to the berth line. So the Original Plan is rectified to secur the navigation of such a vessel by means of incling the direction of the channel to E - W direction and extending it further more.

2. Design of Breakwaters

2-1. Design Conditions

(1) Wave height and wave direction

(a) Waves under direct influence of typhoon:

design wave height	$H_{1/3}$	= 6 m
period	$T_{1/3}$	= 11 ~ 12 sec.
wave direction		W - S

(b) Waves under indirect influence of typhoon:

design wave height	$H_{1/3}$	= 4 m
period	$T_{1/3}$	= 10 sec.
wave direction		SW - S

(c) Waves by winter monsoon:

design wave height	$H_{1/3}$	= 2.0 m
period	$T_{1/3}$	= 6 - 8 sec.
wave direction		W

(2) Estimation of wave force

Wave force is estimated as a complete breaker after due consideration of estimated breaking depth of wave and the relation between the breaking depth of significant wave height ($H_{1/3}$) and real irregular wave. (In practice the breaking depth should be considered about 1.5 time deeper than estimated one by significant wave height.)

(3) Sea level

M.H.W.L. ; + 1.2 m

(4) Necessary crown height of the breakwater

The necessary crown height of breakwater should be 4.5 m above the Cardinal Datum Level considering that it generally needs 0.6 H above H.W.L. for designing.

For the rubble type breakwater on shore, it is decided a little higher, i.e. + 5.0 m.

(5) Structure of breakwaters

After the report was issued by Japan Survey Mission, Advisery Committee in the Republic of China were held to discuss this problem in July 1965 and September 1965.

Then, at the meeting in January 1966, it was adopted that a rectangular concrete caisson type would be better for the breakwater.

In accordance with this determination, the breakwater were designed as a rectangular concrete caisson type one in main part. The composition of breakwater was considered as follows. (cf, Appendix Fig - 1, 2)

(a) The initial plan

- Rubble type one runs just beyond the sand bar located about 400 m from shore line.

- Concrete caisson type one follows the former in deeper part.

(b) The alternated plan

Examining the initial plan in connection with the base of construction works, the transportation system of rubbles and working speed, the initial plan has been changed as follows.

- Rubble type one for shallow part (from shore line to - 5 m part in depth); i.e. 260 m of South-Breakwater and 200 m of North-Breakwater.

2-2. Design of concrete caisson type breakwater (c.f. Appendix Fig. - 1 and - 2)

(1) The height of caissons was determined to get the minimum thickness of rubble mound 1.0 m and the average 1.5 m. The crown height of caisson from Cardinal Datum Level was + 2.0 m after consideration of the relative difficulty of rubble filling works and in order to prevent flowing out of filled materials by wave force during the works and so on.

The top of upper concrete is + 3.5 m high and the parapet constructed on it is + 4.5 m high.

(2) 20 - ton concrete blocks are employed for foot protection, ; i.e. three pieces on the outside and two pieces on the inside in the cross section of the breakwater. While the slope of rubble mound is covered with 2 - ton stones.

(3) The dimension of caissons are shown in the Table - 1. In this case the maximum weight of caisson counts 1,320 ton, then it should be rectified to 1,500 ton since it is too heavy for the slip-way.

2-3. Design of apart in shallow water (less than - 5.0 m in depth) (cf. Appendix Fig. - 1 and -2)

(1) Since it is difficult to execute sea works on shallow water part, the rubble type is adopted and the riprap works should be executed from shore line step by step. The dimension of the rubble type breakwater is as follows.

South Breakwater	height of the top	+ 2.0 m
	width	17.0 m

North Breakwater	height of the top	+ 2.0 m
	width	15.0 m

Big stones should be placed on the slope part by crawler crane or derrick situated on the breakwater.

Upper concrete works should be at first begun by paving with 50 cm thickness for the traffic and the rest of it should be executed straight out after completion of the riprap works.

(2) Necessary weight of armouring stone for the slope is estimated by Hudson's Formula as follows.

water depth	necessary weight
0 ~ - 1.0 m	2 ton
near -3.0 m	5 ton

3. Construction of breakwaters

3-1. Quantity of construction works

Total quantity of construction works are shown in Table - 2. The calculation was based on the followings.

(1) Increase ration of rubble amount.

1'. in shallow water site (rubble type breakwater)

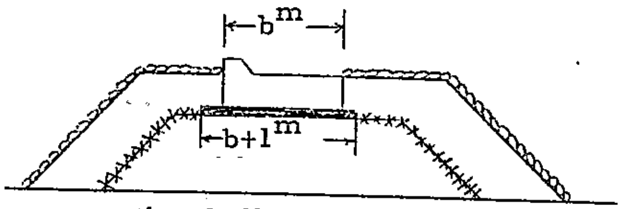
- 20% for rubble-stuffing (30 ~ 150 kg) by land works
- 10% for armouring-stone placed (1 ~ 5 ton) by crane ashore

2' . in deep water site (caisson type breakwater)

- 25% for foundation rubbles (30~ 150 kg) by sea works
- 10% for foot protecting stones (2 ton) by sea works
- 10% for sand-stuffings into caisson by sea works
- 10% for other stuffings into caissons by sea works

(2) Leveling works

On the deep water site and the shallow one, the necessary leveling works was considered as shown in the next sketch.

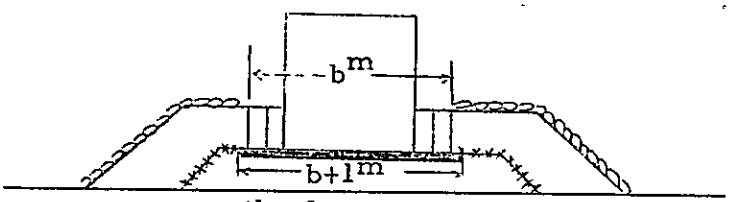


on the shallow water site

the accurate leveling works ----------------

the rough leveling works ------------------

the leveling works of armouring stones ------



on the deep water site

3-2. Estimation of Working days

The days suitable to work on the sea are estimated 8 monthes (240 days) as results of 10 years'eye-observation (1952 - 1961), field-observation (Oct. 1965 - Jan. 1966) and 13 years' data of typhoon (1945 - 1957).

Days impossible to work might be about 1 month (wave height is more than 1.0 m)

Days difficult to work might be about 3 monthes (wave height is 0.5 - 1.0 m)

Days suitable to work might be about 8 monthes (wave height, is less than 0. 5 m)

Working days on the land are estimated 300 days per year.

3-3. Foundamental views of execution program (cf. Appendix Fig. - 3)

(1) The period of works is assumed for 7 years. This program involves that depth of the channel should be - 14 m for a 75, 000 DWT vessel and be capable to accept a 10, 000 vessel, which needs - 9 m ~ - 10 m depth, in 5 years.

(2) Main breakwater (South-breakwater) should be complete prior to other works as fast as possible and North breakwater would be covered with the main one.

(3) The site of construction base is a important element for determination of construction methods, and it seems that the most suitable site is the land reclamation site (scheduled for a ship-building dock in future) in Hsiao-Kong area after comparison with the following 4 sites.

The land reclamation site in Hsiao-Kong area

The neighbouring site of scheduled thermal-power station (Tar-Ling-Pu) (Tar-Ling-Pu)

The neighbouring inner-harbour side of breakwater construction site (Hung-Mao-Kong)

The neighbouring outer-harbour side of breakwater construction site (Hung-Mao-Kong)

(4) If Hsiao-Kong would be selected as the site of construction base, ripraps used for the rubble type breakwater (in shallow water site) could not be conveyed without the existing roads

(capacity; 100 m^3/day). So the caissons should be used as many as possible while the capacity of roads would be used to the full extent for the rubble type breakwater.

(5) In order to construct the caisson type breakwater (in deep water site), the necessity of navigations between the base site and outer-port is occured. So it cannot help hastening the short-cut of sand bar for harbour entrance. (The excavation works should be executed on the later half of the 2nd year after the construction works of breakwaters would be started.)

(6) To the site of excavation from siltation, it is scheduled to construct the tetrapod groyne at the north side of the shortcut.

(7) It is occurred to dredge a channel for execution from the base-site to the outer-port. (The existing water depth of the inner-port is -0.2 m.)

3-4. Preparatory Works

(1) Quarrying Works

Honpi-tou stone is a kind of lime-stone. It is not always suitable one, but not useless. (cf. the extract of page 24 - 25)

As a quarrying method, the bench cut blasting method with a downward drilling is effective.

Transport distance is rather short as 8.5 km from quarrying field to Hsiao-Kong base.

(2) Road repairing works

The road needs repairs for the stone transport from Honpi-Ton to Hsiao-Kong base. It costs about 1,330,000 NT$ and takes first one year.

(3) The construction base

It needs about 22,000 m^2 area for a caisson yard, a rubble storage yard, a repairing site for working boots and their installations and precinct roads. Further adding about 10,000 m^2 area as a basin for working boats, it needs up to 32,000 ~ 35,000 m^2 in total.

Now from the requirement of working boats, the basin should have - 3.0 m water depth and a berth with a length of 350 m at least.

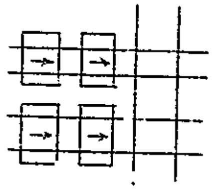
(4) Molding forms of caisson

In consequence of comparison between sliding system of molding forms and the metal molding forms, the cost of sliding forms system is less than half one of metal molding forms. But collectively taking the relative difficulty of works, the number of working-men and the all-night works (in case of sliding form) into consideration, metal form is seemed rather better though the cost of materials might be expensive a little.

(5) Equipments for preparation of caissons

In case that the working period might be 5 years, it should be necessary to prepare 29 - 33 caissons per year and to equip the plant which could manufacture 4 caissons at a time. Then the drydock system was roughly designed by the size of 90 m × 20 m.

In the case of slip-way, several patterns can be proposed, while the rough design has been done as for the one outlined below.



The cost hardly differs in both types, if anything, the dry-dock is expensive a little. Though there are merits and demerits in both types, it is desirable that the determination should be done after due consideration of the utility of facilities when they might survive after the works completed.

3-5. Execution Programme of breakwaters (cf. Appendix Fig. - 4)

Capacity and quantity of working boats, trucks and other facilities necessary for the transportation of stones and the execution of rubble type breakwater (shallow water site) and concrete caisson type breakwater (deep water site) are estimated in consideration of execution methods and quantity of works.

3-6. Rough estimation of the construction cost of the breakwaters

Fig. - 3 shows a rough estimation of the unit cost per 1 m of the breakwater and of the total cost. In addition, it might need to add the repairing and the maintaining cost of working-boats, machinery used for execution.

4. Dredging Programme (cf. Appendix Fig. - 3)

4-1 Volume of spoil

Volume of spoil from the channel area,

the boundary of which is the East end of shore revetment of inner port entrance, amounts to nearly 9,000,000 m^3. Volume of spoil from the channel area for working boats (from Hsias-Kong base) amounts to about 360,000 m^3, assuming the water depth of - 3.0 m, which consists of 260,000 m^3 from the part of inner-port with 50 m width and 100,000 m^3 from the part of sand bar with 50 m width.

Volume of spoil from the basin for temporary storage of caisson amounts to nearly 600,000 m^3 assuming the depth of - 6.0 m and this basin needs the area of 20,000 m^2 (100 m × 200 m) that enables the easy navigation of tugboats.

4-2. Dredging programme

(1) As the riprap works of caisson type breakwater and other sea works are scheduled to begin on the second half of the 2nd year, in 2 years and a half from starting it should be necessary to complete the channel for works in innerport. For these works, the Hotso-kon might be utilized and the volume of spoil to be dredged would be 200,000 m^3 in the 1st year and 100,000 m^3 in the first half of the 2nd year.

(2) The short-cut works of sand bar would be executed in the second half of 2nd year by the newly built dredger.

(3) After short-cut of sand bar, the new dredger would be used for the dredging works of the channel for caisson transportation and the temporary basin for caissons.

(4) From the 3rd year on, the outer channel would be deepened step by step according to the extension of the breakwater. And from the 5th year on, about 1,500,000 m^3 should be dredged

by Tokai-hon, since it might exceed the capacity of new dredger.

(5) Deepening programme of outer channel, which needs the excavation with the total volume of 9,000,000 m^3, is outlined as follows.

Year	by New Dredger	by Tokai-Hon	Dredging Depth	Dredging Width
3rd year	1,500,000 m^3	m^3	- 6.0 m	full extent*)
4th	1,500,000		- 8.0	full extent*)
5th	1,500,000	500,000	- 10.0	full extent
6th	1,500,000	500,000	- 12.0	full extent
7th	1,500,000	500,000	- 14.0	full extent
Total	7,500,000	1,500,000		

*) The width of the Entrance being narrowed.

4-2. Comparison of new type dredger

After due comparison of the barge-line system consisted of the loading pump dredger and barges, and the direct pipe line system by means of the pump dredger, the net cost of works is estimated about 6 NT$/$m^3$ for both systems.

There is a plan to use the spoil from the 2nd Entrance for the offshore reclamation. So it should be desirable if the spoil is useful for reclamation.

In this case a 2,000 HP pump dredger would be suitable for this works.

5. Additional Works (cf. Appendix Fig. - 3)

5-1. Shore revetment of inner-port

At the site of short-cut, steel sheet-piles would be used for the shore protection of inner-port. (the extent is 300 m for North-side shore and 350 m for South-side one.) And the south-side shore between the Entrance and the South breakwater is just facing the opening of the breakwater, so it should be protected by ripraps against waves and the back to be filled.

5-2. Shore revetment outside of the breakwater

Rubble type revetment is adopted for protecting the offshore reclamation. The revetment is scheduled to be constructed at the site of 400 m offshore from shore line and with the length of 600 m per year from the 3rd year to the 7th year. This shall be reclaimed with spoil after one year settlement by step.

5-3. Temporary north-breakwater of inner-entrance.

On the second half of the 2nd year after the begining of works, sand bar should be excavated to permit the working vessel's traffic.

For the protection against the siltation of this channel, the temporary breakwater of tetrapods should be constructed. This breakwater is composed of two layers of 4-ton tetrapods with the extension of 100 m, the upper of which consists of 3 pieces and the lower of 4 pieces.

And the head part of this breakwater, which extends 15 m, is composed of one layer of 4 tetrapods with the weight of 4 tons.

The part on shore is composed by the rubble mound with the extension of 30 m. Construction period of this work is one year of the 1st year and construction cost is estimated 710,000 NT$.

6. Necessary Quantity of Execution Facilities and Execution Equipments 4,900,000 US$ of the YEN-Credit might be required for the preparation of them.

(as shown in Table - 4)

7. Whole Working Programme and Fund Raising Plan
(Refer to Table - 5 and 6.)
Total cost of works (NT$ portion) is shown below.

breakwater works	280,840,000 NT$	(including working boats, installations, working base and road)
dredging works	105,500,000 NT$	(including new dredger, channel for working boats)
additional works	33,310,000 NT$	(including shore revetment of inner entrance temporary breakwater of tetrapods, shore protection & reclamation works outside of breakwaters.)
other expenses	90,350,000 NT$	(including custom and compensation etc.)
Total	510,000,000 NT$	

8. Conclusion

8-1. The whole works could be carried out within the budget proposed by Kaohsiung Harbour Bureau.

8-2. By the construction of offshore breakwaters, it is inevitable to be dammaged to some extent during the work. So it is desirable to make the project take these matters in consideration from the beginning.

8-3. Since this execution project is provided as the basis of the Second Harbour Entrance construction works, it is necessary that the basic plan should be rapidly decided after setting sufficient examination and the detailed design should be fixed as soon as possible.

May 1966

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II New Type Breakwater proposed by Second Harbour Engineering Office

1. On the shallow water section (0 ~- 3 m), tetrapods (2 ~ 4 tons) are used as the armouring after riprap works executed from shore line.

2. South Breakwater (- 3 ~ - 5 m section)

North Breakwater (- 3 ~ - 11 m section)

. They are designed as a cylindrical caisson type as shown in Fig. - A. Each caisson is 17 m in diameter and has cutting edge of 1 m under its bottom.

The execution method is as follows.

(1) to dredge the exsisting sea-bottom by 1 m depth

(2) to level the sand sea bed

(3) to settle the caisson with 1 m sinking by means of its dead-weight and the cutting-edge.

3. South Breakwater (- 5. 0 m ~ - 12. 5 m Section)

The South Breakwater is designed as a composition of cylindrical caissons of which diameter is 17 m as shown Fig. - B. It is seemed to manufacture three caissons joined with each other at the dry dock.

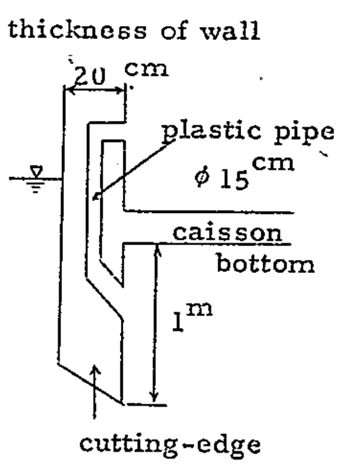
These caissons with 1 m cutting-edge under their bottom are placed by the following method. The procedure;

(1) Dredging works is executed first of all,

(2) Divers level the sea-bottom,

(3) Caissons are settled with 1 m sinking by means of their dead-weight and cutting-edge.

A caisson is poured by way of siphon system, and as they are sinking, the pressed water between the bottom plate and the sand surface is discharged through the plastic pipes, which are equipped on the side wall of caisson by 12 per one caisson. The water discharged through these plastic pipes is useful for the caisson settlement.



And the both foot sides of these caissons 1-ton ~5 - ton stone are laid with the thickness of 2.5 ~ 3.0 m and the width of 7 m ~ 10 m for the protection against scouring.

Design Conditions are as follows.

design wave height	$H_{1/3}$ = 6.0 m
wave force	P = 1.25 wH (combined value of Sainflou Formula & Hiroi Formula)
bottom surface	fine sand (thickness 6 m, N = about 10)
second layer	silty-sand (thickness 2 m, N = about 3)
third layer	sand (thickness 2 m, N = 10 and more)

The allowable bearing capacity of sea-bottom is 24 t/m^2. At the site of three caissons joined with each other in depth

- 12.5 m, reaction force is estimated to distribute linearly 24.1 ton to 6.8 ton when the design wave force might act on the caissons.

4. Superiority of the New-Type to Conventional Type breakwaters.

Superiority of the New-Type is assumed by the Second Harbour Engineering Office as follows.

(1) The new one is superior to the other in every view point of stability.

(2) For the construction works, a conventional type needs 40 parties of divers, but the New-Type only 10 parties. In addition to this condition, the New-Type would further enable to reduce the volume of ripraps very much.

(3) As for a conventional type, the reflection of waves arises considerably much by reason of the vertical and flat wall.

While as for a new type, the reflection of waves is less and the running wave along the breakwater will be reduced effectively.

(4) The construction cost of the new type is about 80,000,000 NT$ cheaper than the one of a conventional type. It accounts for 18 % cost-reduction of the total construction costs of 416,000,000. NT$.

III Comment on Plan of New-Type proposed by Second Harbour Engineering Office

1. It is doubtful that the leveling works of sandy bottom might be easy to execute after dredging works of 1 m. Probably, pump-dredgers may be adopted for the dredging works. However, the leveling works for the preperation of three caissons joined with each other might be nearly unpractical.

Then the leveling works of rubble mound may be executed easily from a view point of the unit labour of divers. In Japan, we have no experiences of the leveling works on the sandy-·'tom by means of divers, but, we have studied the leveling works after the dredging works by a drugsuction pump dredger.

So, the examples in Port of Moji and Port of Nagoya are appended at the end of this report for reference.

2. Three caissons joined with each other may not float with the horizontal level. Further more it would be very difficult to place these caissons with horizontal level only by means of the control of water pouring.

Setting apart from rubble mound foundation, it is too dangerous to place these caissons on the sand bottom without rectifying the inclined situation of them.

Namely, if these caissons on the sand would incline a little at the time of placing works, the even contact of caisson bottom on the sand might be doubtful, and the bearing capacity of the sand would be partially overed.

3. Allowable bearing capacity of the rubble mound is said to

be 40 t/m^2. (based on the Design Standards for Port & Harbour Structures in Japan) But to avoid important trouble it is usual in Japan that approximately 30 t/m^2 is taken as the upper limit of the allowable bearing capacity on the rubble mound.

In the case that the cylindrical caissons (diameter; 17 m) proposed by the Second Harbour Engineering Office were placed on the sand, ($\phi = 30°$, N = 10) the allowable bearing capacity would be estimated as follows.

Safety Factor = 2 ; 23 t/m^2

Safety Factor = 2.5; 19 t/m^2

(Above estimation is based on the Japan Design Standards for Port & Harbour Structures, which notes the safety factor to be over 2.5 as a standard.)

It should be necessary to survey whether the allowable bearing capacity of 24 t/m^2 estimated by the 2nd Harbour Engineering Office might be expected or not.

4. There might be some fear for the sinking method of caissons. That is to say; the pressed water between the caisson bottom and the sand surface might act as jet water, which should scour out the sand towards the outside of caissons, and in consequence of this phenomena there might appear the cave under the bottom of caissons.

For preventing this phenomenon 12 pieces of plastic pipe (diameter = 15 cm) per one caisson are certainly designed to discharge the pressed water.

But it is questionable whether these pipes might play their roles or not by reason of the blockade of pipes depending on a certain sinking speed of caissons.

5. Even if the sand were leveled horizontaly and the pressed water under caissons could be discharged towards outside, it could not be expected of the even contact of the caisson bottom to the sea-bottom by reason of the lack of penetration depth of edges and the slight inclination of caissons.

Furthermore in this case, there might arise the problems on the distribution of foundation reaction.

6. It is certain that the protection works against scouring is executed by ripraps along the outline of cylindrical caissons. If these ripraps might be scoured away, however, it should be apparent for the sand under the caissons to be flow out easily. In this case, the caissons would be inclined or broken down.

In general the foot protection against scouring is very much difficult for these structures. (cf. 1) Page 39 ~ Page 41 ; Report of Model Tests of Port & Harbour Research Institute, Ministry of Transport, Japan Gov.(cf. 2) Page 42 ~ Page 44 ; Report of the Settlement in Port of Naoetsu)

7. The proposeal of the 2nd Harbour Engineering Office says that the cylindrical caissons hardly reflect the wave towards outside.

On the other hand it is possible that more scouring at the foot of hollow parts towards outside of caissons will occur.

8. The Office says that the cylindrical

type is cheaper than the combination type of rubble and caisson by 80,000,000 NT$, and that the total construction cost may be reduced by 18% of 416,000,000 NT$.

If there might not any other technical problems in the caissons placed on such a sandy bottom, however, it should be necessary to compare the cost of three cylindrical caissons (with the cutting-edge) with that of a rectangular one (with the cutting-edge).

IV Stones and Divers

1) The shortage of quantity and poor quality of stone

2) The lack of divers

are the reasons why the 2nd Harbour Engineering Office gives up the caisson type breakwater on rubble mound.

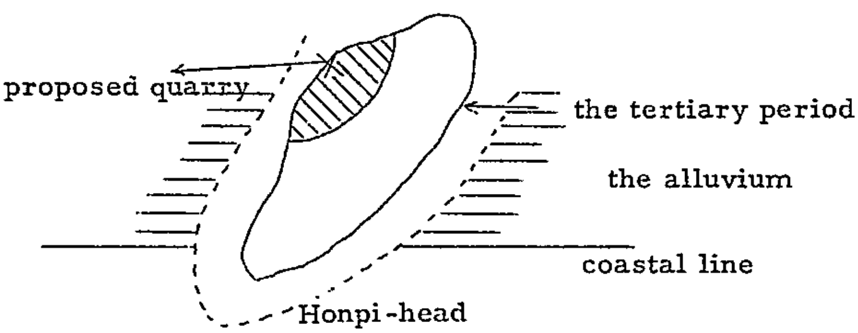
1. Quality of Stone

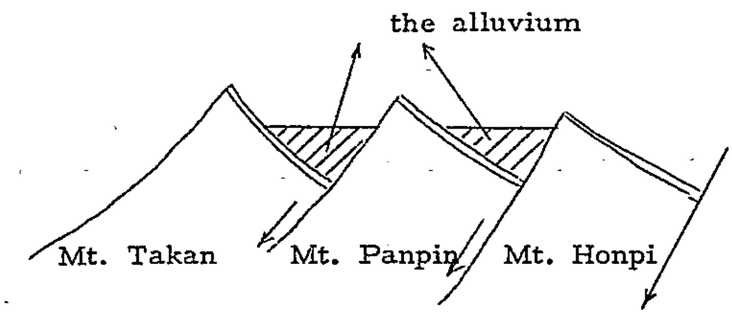
Stones of Mt. Honpi-ton, which are lime stones, are not good but can be available to use.

(the excerpt from Page 32 of Wada's report in May, 1966)

The coast in this site was formed by rising after sinking, therefore fossil shells were discovered in stones of Mt. Honpi ton and the drowned valley were discovered in Shia Tansui river, Mt. Honpi-ton. Stones of Mt. Honpi-ton are uplift coral reef limestones made of aqueous rocks piled up in the sea about a million years ago. Therefore the chemical composition consists mainly of CaO and SiO2 a little.

As many fissures exist, it may not be expected to get enough big stone so much. The laterite in the fissures is not weathered but seems to be the surface soils and the inner part of it doesn't seem to be weathered such as the surface.





The results of the examination, with which was entrusted the Material Laboratory of Civil Engineering in Chen Kong University, are as follows.

Bulk specific gravity	2.39
Bulk specific gravity (saturated state with dry surface)	2.51
Apparent specific gravity	2.71
Absorption of water	4.98 %
Compressive resistance	365 kg/cm^2

2. Necessary volume of stone

The necessary ripraps is 260,000 m^3 for the cylindrical caisson type breakwater estimated by the Second Harbour Engineering Office and 360,000 m^3 for the caisson type breakwater on rubble mound estimated by Wada in 1966, both of which are not too much different.

Considering that the cylindrical caisson type breakwater needs the addition of outside rubble to protect against the scouring, the difference of necessary volume of rubbles between the two is to become smaller.

3. The Transportation of Stone

The transport distance from Mt. Honpi to the base at Hsiao-Kong is about 8.5 km and its short distance gives very good conditions. Examples in Japanese ports are shown as follows;

The Transport Distance of Stone in Ports along the Coast of the Japan-Sea

Port of	Akita	by land	27 km
"	Sakata	"	25 "
"	Niigata	"	30 "
		by sea	65 "
"	Toyama	by land 14 km + by sea	30 "
"	Nanao	by land	12 "
"	Kanazawa	"	32 "
"	Tsuruga	"	14 "

The Transport Distance of Stone in Ports along the Coast of the Paciffic Ocean

Port of	Yokohama	by sea	110 km
"	Kashima	by land 30 km = by sea 120 km (ship with 200 - 250 m^3)	
		by sea 530 km (ship with 600 - 1,000 m^3)	
"	Nagoya	by sea	74 km
		"	83 "
"	Kinuura	"	45 "
		"	25 "
"	Shimizu	"	43 "

Port of Shimizu by sea 250 km

Table of Transport Distance of Rubbles in each Port

Port (Destination)	Route and Distance	Origin of Rubbles
Akita	← by land 27 km	Mikurabana
Sakata	← by land 25 km	Kosagawa
East Niigata	← by land 30 km	Shibata (Mt. Taihei)
	← by sea 65 km	Sado Island
Toyama	← by sea 30 km Port of Uozu ← by land 14 km	Okuhirazawa
Nanao	← by land 12 km	Futamiya
Kanazawa	← by land 32 km	Wakabara
Tsuruga	← by land 14 km	Oiwake
Keihin	← by sea 110 km	Manazuru
Kashima	← by land 30 km	Onahama
"	← by sea 120 km	"
"	← by sea 530 km	Owase
Nagoya	← by sea 74 km	Toba
"	← by sea 83 km	Hatazu
Kinuura	← by sea 45 km	Toba
"	← by sea 25 km	Hatazu
Shimizu	← by sea 43 km	Izu Peninsula
"	← by sea 250 km	Nigiri

4. Unit Labour and number of Diving parties

(1) In reference to Wada report in 1966, the necessary leveling works and unit labour of diving party in case of caisson type breakwater on the rubble mound are as follows.
(cf. Page 62 of Wada Report 1966).

This unit labour of diving paty is estimated from the past data till 1965 in Japan.

works	necessity	works/party /day	period
accurate leveling works (head of foundation)	73,617 m^2	5 m^2	4 years & 8 monthes
Rough leveling works (slope parts of foundation)	29,502	12	4 years & 8 monthes
Leveling works of armouring stones	47,132	16	4 years & 10 monthes

This is estimated on the assumption that 7 years are for the complete constrution works, 2 years for the preparation and nearly 5 years for breakwater works (caisson type) at deep sea. The available days of sea works per one year is estimated 8 monthes (240 days) according to Wada Report (Page - 25) in 1966;

Days impossible to work:	about 1 month (wave height more than 1 m)
Days difficult to work:	about 3 monthes (wave height 0.5 m ~ 1.0 m)
Days possible to work:	about 8 monthes (wave height less than 0.5 m)

According to the above consideration, the number of required diving parties are reported as follows. (cf. Page - 62 of Wada Report in 1966)

The accurate leveling works	13.2 = 14 parties
The rough leveling works	2.2 = 3 parties
The leveling works of armouring stones	2.6 = 3 parties
Total	20 parties

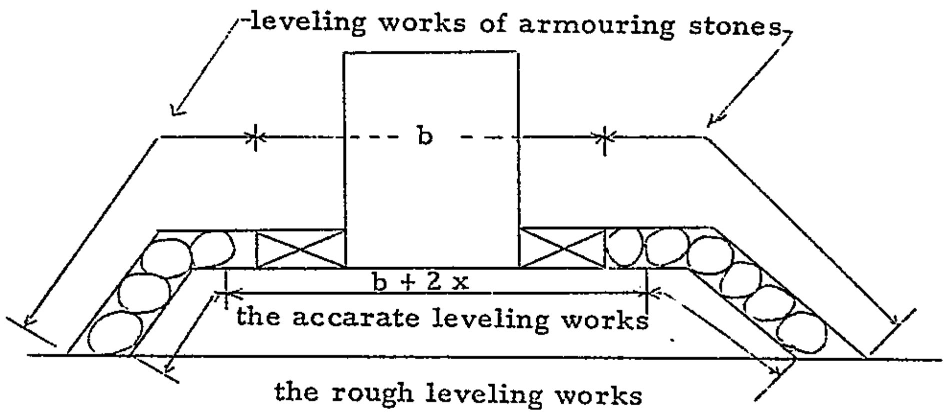
(2) Standard unit labour of divers in Japan is as follows. (excerpt of "Estimation standards for Contracted Works" edited by Bureau for Ports & Harbours Ministry of Transport in Feb. 1967 and adopted from Apr, 1967)

The accurate leveling works	7.0 m^2/day/party
The leveling works of armouring stones	14.0 "
The rough leveling works	17.5 "

(assuming seven hour work per one day)

(3) The actual results of unit labour and necessary number of diving parties in principal ports of Japan are as follows.

Actual results of unit labour of divers in principal ports of Japan



$\begin{cases} x = 1 \text{ m : in case that no foot protecting blocks exist.} \\ x = 0.5 \text{ m : in case that foot protecting blocks exist.} \end{cases}$

(on basis of "Estimation Standards for Contracted Works")

Actual Results of the Leveling Works (m^2/day/party)

Port	accurate leveling works	leveling works of armouring	rough leveling works
Akita	-	21.6	22.8 (average of results in 1966, 1967)
Sakata	-	20.4	18.0
New Port of Niigata	18.0	16.8	33.6
New Port of Toyama	9.6	12.0	14.4
New Port of Kanazawa	10.2	13.8	15.0
Tsuruga	17.4	15.0	23.4
Average	13.8	16.6	21.2
Kashima	5.0	22.0	22.0 (in 1966)
	5.0	9.0	19.0 (in 1967)

In Port of Kashima, weather conditions are bad all the year round, because of the swell coming from the Paciffic Ocean and there are few days of the wave height below 1 m. Therefore the diver can work only 4 - 5 hours per one day on the average.

Extension in New Port of Niigata

	1966	1967
East Breakwater		
mound works	271m	229m
caisson works	390	230
West Breakwater		
mound works	445	362
caisson works	313	338

Total		
mound works	716	591
caisson works	703	568

In Neew Port of Niigata, the construction of mound precedes the caisson works by one year, so it may be considered that the mound works consist of the rough leveling and the leveling of armouring and the caisson works consist of the accurate leveling works.

The Extension in Port of Kashima

	1966	1967
North Breakwater	199 m	135 m
South Breakwater	386	422
Total	585	557

The Number of diving parties per one year

New Port of Niigata	12 - 15 parties
Port of Kashima	10 parties (in 1966)
	20 parties (in 1967)

Note: In Port of Kashima, the number of parties increases because of taking unavoidably concentric works by reason of bad conditions of weather in 1967.

(4) Assuming that 8 years would be needed as the construction period of the Second Harbour Entrance of Kaohsiurg Harbour, 5 years would be enough for that of caisson type breakwater at least. (Considering the period of preparation and construction works in the shallow sea, six years might be enough.)

The extension of caisson type breakwater is estimated

3, 000 m in length by the 2nd Harbour Engineering Office. So if the construction period may be 5 years, the breakwater must be constructed by 600 m per year.

(5) On the assumption that the period needs 5 years, the working days are 240 days per year and the unit labour is conservatively estimated on the basis of Japanese data, the number of diving parties for the construction of caisson type breakwater on rubble mound is as follows.

Works	Necessary Area	Works/ day/party	Period	Necessary number of parties
The accurate leveling works	73, 617 m^2	7 m^2	$240^{day} \times 5^{year}$	8 ~ 9
The leveling works of armouring stones	47, 132 m^2	14 m^2	"	3
The rough leveling works	29, 502 m^2	17.5 m^2	"	1 ~ 2
Total				12 ~ 14

(6) The number of diving parties for construction of caisson type breakwater on rubble mound is 12 - 14 and if considering the decrease of unit labours, 20 parties in maximum seem to be sufficient for working. (In Wada Report in 1966, 20 parties are necessary for the construction period of 4.8 years.)

(7) Anyhow, it is desirable to carry out the possible training of diving works as soon as possible.

V. Examples of Damaged Structures on The Sand Foundation in Japan

1. No. 5 - Breakwater in Port of Kobe

(1) Structure

This breakwater was constructed to protect the frontage of Maya-Wharf. Having taking the clay strata into consideration, we adopted the two-storied cylindrical cellular caisson as Fig. - 2 shows.

This cellular caisson was made of prestressed and precasted concrete of which diametre is 15.85 m and thickness of wall is 0.15 m.

By means of vacuum sinking system, the lower part that has 11 m height penetrated the clay strata, which has the thickness of 8 m, i.e. laid from - 11 m in sea depth to - 19 m, and was settled on the gravel layer.

The upper part of cellular caisson which was placed on the lower one has 11.5 m height.

The height of the top of cellular caisson after setting works is + 2.5 m and that of the completed body becomes + 5.0 m after the upper concrete works on the cellulars.

The joints between the upper cellulars and the lower ones were set by rubber packing set and the gaps between every cylindrical caisson were filled up with steel pipes of large diametre, which were 2,000 mm in diametre, 9-14 mm in thickness and 28,500 mm in vertical length.

These steel-pipe piles were drived into - 23.0 and the upper part of them was fixed by H-steel bars which connect the upper part of piles with the upper works of cellulars.

(2) Damage

This breakwater was completely broken down by the Typhoon No. 20 in September 1964. The wave that attacked the breakwater was supposed to be 3.8 m in wave height ($H_{\frac{1}{3}}$) and 10.2 sec in period ($T_{\frac{1}{3}}$). (Design Wave Height $H_{\frac{1}{3}} = 3.0^{m}$)

This wave broke down the upper cellular caissons and the upper reinforced concrete rings completely or partially, which extended almost over the total extension of No. 5 Breakwater. Many of them were destroyed down to the land side, but 25 % of them towards the offshore side. The tilting direction of the lower cellular caissons coincided with that of the upper.

The steel pipes of large diametre which were driven for filling up the gaps stood mostly as they had been.

(3) Causes of damage

The safety factors of the upper cellular caissons against design wave ($H_{\frac{1}{3}}$ = 3.0 m) and estimated attacking wave ($H_{\frac{1}{3}}$ = 3.8 m) are respectively shown as follows.

Safety Factor of the Upper Cellular Caisson

type of failure	against design wave	against estimated attacking wave	notes
slide	2.08	1.44	Design Condition H 1/3 = 3.0 m sea level = +1.7 m
fall (friction between wall and fills)	1.45	0.93	

type of failure	against design wave	against estimated attaching wave	notes	
shear failure of inner materi-als	1.34	0.85	at time of damage	H 1/3 = 3.8 m sea level = +3.1 m

This table shows us the cause why the upper part sustained the damage as follows,

1) The wall of cellular caisson could not resist the internal shearing force and was sheared down.

2) The friction between cellular wall and filled sand was too small for the caisson to behave as a body aginst a external force. This carried the separation of the wall from filled sand and the flow out of stuffed sand from the joints.

3) The tilt of the lower cellular caisson caused upper ones to slide.

4) Vibration by wave action caused the stuffed sand to liquefy and shearing resistance of sand to decrease.

5) Survival of steel pipes of large diameter means that H-type steel beam destroyed the upper works of cellar caisson which was falling down, and it caused stuffed sand to flow out.

6) Next, let's go a step forth to examine tilt of lower cellulars caissons which caused the upper ones to slide. Why did the lower cellular settled on the gravel stratum tilt ?

In desgning these cellular caisson, it was assumed that cellular caisson had its own bottom and the distribution of reaction formed triangular.

Now, the values of toe pressure estimated on the basis of this assumption are showen as follows.

Toe Pressure Value of the Lower Cellular Caisson

with regards to soil pressure	For design wave	For estimated attacking wave
in case of neglecting active & passive pressure	65 t/m^2	101 t/m^2
in case of considering both pressures	57 "	84 "

On the contrary, the bearing capacity of foundation was estimated by means of the following two methods and the ultimate bearing capacity was supposed to be 200 t/m^2. That is to say; ① The ultimate bearing capacity estimated by Terzaghi's Formula for circular foundation are shown as follows.

N-value	angle of internal friction	ultimate bearing capacity (t/m^2)
10	33°	88
20	33°	208
30	33°	450

Considering that the lower cellular caisson reached the gravel stratum of 20 in N-value, the ultimate bearing capacity of 208 t/m^2 might be expected.

② By observation data obtained when the lower cellular caisson were forced in the sea-bed, it is estimated that the ultimate bearing capacity is 205 t/m^2.

The foundation reaction is estimated at 65 t/m^2 for the design wave or at 101 t/m^2 for estimated attacking wave (both of them are the values in case of neglecting soil pressure on the outside wall of caisson), so the ratio of ultimate bearing capacity to the reaction, i.e. safety factor, is respectively 3 or 2.

It is supposed that these values should not cause the sinking tilt of lower cellular caisson. However, it is the assumption of triangular distribution of reaction that has problems in itself.

As a precondition to assume the triangular distribution of reaction, the bottom of structure needs to be uniformly rigid.

As for the No. 5 Breakwater, the bottom of cellular caisson on gravel stratum is only the concrete ring with 0.15 m width except soft subsoil which cellulars penetrated into.

Therefore, it should be supposed that the distribution of reaction might be concentrated remarkably to circular wall by reason that soft subsoil should be deformed but that cellular caisson would not deformed virtually in propotion as the reaction increased by the action of horizontal force. That is, the value of reaction on toe of cellular caisson might become larger than not only the estimation above but also the ultimate bearing capacity of gravel, which should be supposed to have produced the sinking tilt to the lower caisson.

(4) Conclusion

The No. 5 - Breakwater in Port of Kobe has offered us many instructive experiences mentioned above, then it is the problems bottom reaction that I want to insist on. That is to say; the bottom reaction had not triangular distribution as in the initial design because the cylindrical cellular caisson had no uniformly rigid bottom, which caused the tilt of lower part, I suppose.

Whether structures without bottom should be constructed on sand or on gravel, it needs to take care of and to consider this point.

2. West - Breakwater in Port of Tagonoura

(1) Structure

The Port of Tagonoura is one of the artificial port excavated on shore, facing the Pacific Ocean via the Suruga Bay and has sustained damage by scouring for several times. (cf. Fig - 3) The original breakwater was composed of 5 - tons tetrapods and 40 m in extension length, but destroyed and flowed out completely enough whole extent by the typhoon on 11th July in 1958.

Then, the method of pheumatic caissons forced into the sand was adopted as the restoration work as showen in Fig. 4.

(2) Damage

Damage by the No. 24 Typhoon in Sep. 1965 was as follows.

1) The westside bottom of No. 7 caisson was scoured to sink by 1.5 m and tilted to offshore-side. Comparing the ground level with that before typhoon, it has been proved that the scouring depth of this part was 6.5 m.

2) The offshore-side (South - West - Side) front toe of No. 10 caisson was scoured to sink by 1.5 m and tilted to offshore-side by 13 degree.

This means that the scouring depth might be 8 ~ 9 m.

3) The joints between No. 9 caisson and No. 10 and between

No. 10 and No. 11 were broken down. For restoration works, we gave it up to recover them horizontaly as they had been and were going to start the works according to the plan shown in Fig. - 5.

We set foot protecting caissons and blocks in front of breakwater to protect it from scouring and to prevent more progressing of damage, the foot of the No. 10 caisson and its neighbouring ones was covered by 1 ton stones and asphaltmastic. However, the No. 26 Typhoon attacked this breakwater in Sept. 1966 and deepened the damage sustained by No. 24 Typhoon in 1965 as follows.

1) The tilt of No. 7 caisson and No. 10 become larger.

2) The joints between No. 9 and No. 10 and also between No. 10 and No. 11 were destroyed.

3) No. 3 caisson for foot protecting that was under execution in front of No. 10 caisson tilted over by 30° degree.

4) Comparing the chart of water depth with that before this typhoon, the scouring was about 5 m between No. 7 - No. 9 caissons, about 7 m in front of No. 10 and about 2 m in front of the No. 11.

(3) The results of model test for scouring and restoration works (by Port & Harbour Research Institute of Ministry of Transport)

In order to examine the method for restoration works, the Port & Harbour Research Institute carried out model tests, the results of which is shown in outline as follows.

1) The effects of foot protecting blocks and caissons.

Fig. - 6 shows the result of tests when foot protecting caissons and blocks are set in front of West-Breakwater. According to this result, it might be supposed that the scouring between breakwater itself and foot protecting caissons was about 6 m in depth but that there were little influence on breakwater itself.

Foot protecting blocks should have the effect to prevent the scouring in front of the caisson, but they sustained the scouring of nearly 3.5 m and it seemed not to be so much effective for their purpose.

Then, Fig. - 7 shows the case that there was gap of about 2 m between foot protecting caissons each other. Sea water which went and returned through this gap scoured about 3 m in front of No. 10 caisson and 7.5m in its gap.

The state of scouring in case that foot protecting blocks was not set in front of caissons was nearly same as the case with foot protection in case of Fig. - 6 and Fig. - 7.

2) The effect of tetrapod

1" In case that tetrapods were thrown directly on the sea bed. ; Fig. - 8 shows the result of testing when tetrapod was thrown in front of breakwater.

Even if we might do so, the breakwater should surely suffer from scouring extending to the bottom of front toe.

2" In case that tetrapods were placed on asphalt mats. Fig. - 9 and Fig. - 10 shows the result of tests in case of using asphalt mats. Short mats were used in case of Fig. - 9 and the scouring was as much as one in case of no mats.

I think we could not expect the protecting effect against scouring of the mats of such length.

While, longer mats were adopted in case of Fig. - 10, which hardly produced the sinking of tetrapods. It seems, however, this case might be expected little effect on protecting against the scouring in near parts of the breakwater.

3) The effect of backside caissons. Fig. - 11 shows the result of testings in case that new caisson breakwater might be constructed in backside of West-Breakwater, which should play a role of fore-foot protecting.

In this case, the scouring extended to the bottom of No. 10 caisson in only five minutes after the beginning of wave action.

So, No. 10 caisson should sink by nearly 6 m finally and we could not expect to the West-Breakwater the function of foot protecting works.

4) The effect of ripraps

Fig - 12 shows the result of testing when rubbles were filled between No. 10 caisson and the back ones.

Sand was sucked out of the bottom of rubbles, which sunk by nearly 15 m. And the sinking of No. 10 caisson and the scouring of front toe were hardly different from that in case of no rubbles. It means that such width as this filled rubble is negligibly too small comparing wave length to control the wave action of scouring.

(4) Conclusion

Considering the above-mentioned investigations, i.e. the damage of No. 10 caisson, tilting of foot protecting caissons by No. 26 typhoon and the result of a series of model testing by Port and Harbour Reserch Institute, it was decided to construct new breakwater of larger caisson in innerside of the existing one giving up the existing west breakwater.

The scouring of sand is one of the most significant experience which we obtained in Port of tagonoura, then more strict caution against secoring is needed to structures which have no foundation mound and are directly constructed on sand.

3 West-Breakwater in Port of Naoetsu

(1) Structure

Port of Naoetsu is one of the principal port situated near the border between Niigata Pref. and Toyama Pref.

As shown in Fig 13, West-Breakwater was constructed in 1963 and 127.75 m in its extension with -7.0~8.0 m sea depth.

This structure, shown in Fig-14 and Fig-15, was composed of mass of prepacked concrete directly placed on sand because of no caisson yard near the construction site.

That is to say, steel molding forms, which had hexagonal cross section with 8.0 m in upper width, 14.2 m in lower one, 10.0 m in height and 15.0 m in longitudinal length and also had 1.0 m deep cutting edge in both side of bottom, was placed on sand. And by the weight of prepacked concrete filled in, the cutting edge sunk to be fixed by 1 m below the existing sea bed.

This cutting edge was contrived for prevention from scouring and sliding. And the whole prepacked concrete breakwater was excuted at 18.5 m interval of center, so the gap of 3.5 m between neighbouring parts was connected by prepacked concrete in situs using the moulding forms.

On the inner and outside foot of breakwater was laid down the brushwood mats, on which foot protecting blocks was placed.

Then, riprap blocks were thrown at random in the front of No. 14 caisson placed at the front end of breakwater, and tetrapods further covered the riprаps to protect against scouring.

(2) Damage

The coast along the Japan-Sea is usually attacked by waves due to seasonal wind in winter again and again.

In November, 1963, these wind waves ($H_{\frac{1}{3}}$ = 5.0 m) caused severe damage of scouring and sinking to the end of breakwater extending a length of 36 m, and in Dec, 1963, the same part was sustained more sinking and a part of upper concrete broken by the wave with 5.0 m height.

Then after, furthur sinking was caused by the wind wave ($H_{\frac{1}{3}}$ = 5.0 m) in Feb, 1964. Fig - 15 shows the result of survey on the sinking in Apr, 1964, which says as follows.

	Outer side	Inner side
At the foot of No. 14 caisson	2.4 m	1.9 m
At the foot of No. 13 caisson	1.4 m	1.0 m

The sinking of foot protecting blocks placed outer and inner side appeared approximately 2.0 m in depth.

(3) Conclusion

The sinking of the prepacked concrete type breakwater directly placed on sand and the foot protecting blocks are coincident with the result of the model test for Port of Tagonoura conducted at Port and Harbor Reseach Institute. (written in page 39~41)

So it would be agreeable that the sinking of these caisson and blocks are due to the scouring of sand under then caused by waves.

It is very instructive that such type breakwater, which is placed on sand without rubble mound, should be taken most care of the scouring.

4. Groin of Niigata New Port

(1) Structure

The Niigata New Port is one of the artificial ports excavated on shore facing the Japan sea. Heavy waves generated by the seasonal wind beat upon this port in winter, and the waves with more than 4.0 m height are ordinarily generated about forty times for one winter season. The direction of the waves is confined between NW and NNW in general, and the period of those predominantly 10 ~ 12 seconds. (The wave height in design of the main breakwater is taken as $H_{\frac{1}{3}} = 6.6$ m.)

In this port, the displacement of sand is very remarkable, and the amount of sand drift in one year is estimated about 1,000,000 m^3, and the direction of its displacement is from west to east. The development of bars and troughs is remarkable in shore.

As shown in Fig. - 16, this groin was so constructed as to prevent the movement of sand in the basin surrounded by the two breakwaters (the West-and the East-breakwater).

The structure of this grain is shown in Fig. - 16 and - 17, and we can see that steel pipe sheet-piles with 812.8 mm in diametre and 9.5 mm in thickness are derived in a row.

The construction of them was completed in September 1967.

In the design of it, the scouring depth at the No. 2 sites which sustained the damage was estimated - 2.0 m.

And if it should be considered that those steel pipe sheet-piles could be equivalent to the piles resisting to the lateral force, it was estimated that the scouring depth of - 5 m would be the limiting condition which distinguishes the long pile behavior and the short pile one.

(2) The damage of Groin and their causes

The groin was completely constructed in September 30. After the completion, heavy waves beated upon it by several times, and at last the steel pipe sheet-piles with 68.8 m in length placed on the middle part of the groin was broken down toward western side by the waves with H max = 6.76 m and $H_{\frac{1}{3}}$ = 5.28 m in the early morning, November 13 (cf. Fig. -18).

The sounding immediately made after damage showed that the water depth of the broken parts was so deep as - 9 m.

By the diving investigation, it could be found that the damaged steel pipe sheet-piles were not bended but fallen down.

According to these investigations, it might be decided that the cause of this damage would be the lack of penetration of the steel pipe sheet-piles against the scouring.

But it has not yet been explained why the scouring occured in this part concentratedly, and then in order to clarify this behavior the model tests have been carried out using the movable bed. Fig. - 19 shows the scouring after sustaining the damage.

(3) Conclusion

Fig. - 18 was obtained from the survey of the both sides of the groin composed of steel pipe sheet-piles immediately after sustaining damage, and Fig. - 19 is for the survey of the surrounding parts of the groin after a few days.

Because that some days has passed, we can see in Fig. - 19 that there exists a few siltation after the time of sustaining damage. But as we can see in the sounding map shown in Fig. - 19, though it is for some days after sustaining damage, the scouring occured in a very narrow part surrounding the steel pipe sheet-piles (5 m width in each side of the steel pipe), and moreover a very deep scouring could be observed.

In the outside of this region, however, a remarkable change in the water depth could not be found. The fact mentioned above explaines the following evidently; the wave along the steel pile groin scoured the sandy bed, and this intensive scouring which is beyond our prediction (the scouring with 8 m in maximum depth) broke down the groin.

In the case of the rubble mound type breakwater, the scouring starts from its toe and progresses into its inner part gradually in general, and therefore in order to protect the scouring, the width of the mound needs to be large enough.

But as we see in the case of this groin which is composed of a rigid structure on the sandy bed, the scouring near the structure is very intensive and harmful. This example was a instructive experience for us with regard to the scouring.

VI Preparation plan of crafts and other construction facilities.

1. When Z. Wada made a field investigation in May 1966, it had been known to some extent that the amount of Yen-credit for the construction of the 2nd Harbour Entrance was 4,700,000 U.S.$.

Then the preparation plan of craft and other construction facilities was proposed on baisis of that estimated cost, but if it should be possible that the amount of Yen-credit were enlarged, it would be more desirable to purchase some craft and other construction facilities in order to proceed the work more smoothly.

2. With regard to the purchase of the new dredger, the drug suction pump dredger (hopper capacity of 2,000 m^3 and dredging depth of 18 m - 1.6 m of tidalrange) which is needed by the 2nd Harbour Engineering Office would be fitted for.

VII Conclusion and Recommendation

1. Our opinion about the new type of breakwater proposed by the 2nd Harbour Engineering Office has been already described in Chapter III, and are summarized as follows:

(1) It would be hardly possible to level the sandy sea bed.

(2) It would be hardly possible for the water confined under the bottom of caisson to go out with the sand in it when the caisson sinks owing to its dead weight and its cutting edge.

(3) It would be very difficult to set the three joined cylindrical caissons on the sandy sea bed evenly.

Even if it should be possible to level the sandy sea bed and to discharge the water confined under the bottom of caisson, the uniform contact of the caisson with the seabed could not be expected when the penetration of the edge were slightly lacking and cylinders were inclined. And therefore there would be some problems as to the distribution of foundation reaction.

(4) The allowable bearing capacity for the cylindrical caisson with 17 m in diametre which is shown in the plan of the 2nd Harbour Engineering Office is estimated, under the condition of = 30° (N-value = 10) for the sandy sea bed, to be 19 t/m^2 where the safety factor is taken as 2.5 .

There might be some problems in taking the safety factor as 2.5 (in Japan the safety factor of 2.5 is usually taken as standard for the important structures). Furthermore it would be necessary to re-examine whether the value of 24 t/m^2 for the allowable bearing capacity of the sea bed is reasonable as shown by the 2nd Harbour Engineering Office.

(5) As to the structures placed directly on the sandy bed, the scouring to its foundation must be considered especially.

2. Examples of the damage of structures constructed on the sandy bed in Japan.

Examples of the damage of structures constructed on the sandy bed have been described on the four ports in Chapter V, and they will be summarized as follows.

(1) The 5th Breakwater in Port of Kobe

It is necessary to take into special consideration of the distribution of the subsoil reaction force caused by bottomless caisson and the bearing capacity of foundation in its relation.

(2) The Breakwater in Port of Tagonoura

The scouring of the sandy bed is very remarkable and carful attention should be paid to the scouring of the foundation of structures placed directly on the sand without rubble mound.

(3) The West-Breakwater in Port of Naoetsu

It is not a cylindrical but a rectangular structure with the cutting edge of 1 m placed on the sandy bed, but it is necessary to consider the settlement of the structure itself and of the foot protecting blocks and also the sucking out of sand under the structure owing to the scouring action of the wave.

(4) The groin in Niigata New Port

It was composed of the steel pipe-piles and was broken down because of the deep scouring of the sandy bed where there existed no protection for its foundation. In the case of the rubble mound, the scouring occurs at the toe of the mound and then proceed to its inner part gradually in general, therefore the width of the mound must be enlarged to protect against the scouring. But in the case of this groin, the rigid structure was constructed directly on the sand and the scouring near the structure was very remarkable and severe.

3. As mentioned above, it would be very desirable to reexamine some questionable points of the cylindrical type proposed by the 2nd Harbour Engineering Office.

4. In the case of the breakwater consisted of rubble mound and conventional caisson, the amount of stone necessary for their construction might appear as an unfavorable point by the opinion of the 2nd Harbour Engineering Office, but it does not seem to be so unreasonable in comparison to that of the cylindrical type proposed by the 2nd Harbour Engineering Office's Plan. It seems the stone available at present has not the best quality for rubble mound, however, it is not so bad as to be unsuitable for use.

Since the good stone can be obtained from the quarry comparatively close to the construction site, there might be hardly problems as to stone.

And now, the divers will be needed about 12 parties (maximum 20 parties) for the breakwater of conventional type, and then it had better start training them immediately.

5. As described in the previous report of WADA in 1966, the damage cannot be avoidable to some extent during construction of breakwater in the open sea. This should be borne in mind in planning the construction works.

6. It would be desirable to increase the Yen Credit in order to buy the new dredger and others.

7. Today, the training course for the engineers has been considerably progressed on OTCA base. In Japan the construction of ports has been proceeded day by day and the damage during the construction works of breakwater has been also occurred. Then it would be desirable to send more engineers to Japan in order to see port construction works fully.

It would be desirable that persons who direct and superintend the construction of the 2nd Harbour Entrance visit and observe the port construction in Japan, to say nothing of the engineers who participate in its construction directly.

Today when the construction of the 2nd Harbour Entrance is started practically, the reporter heartily hope that these proposals will be realized as soon as possible.

Appendix - 1. The Leveling Method used for The Trail of Dredger in The Kanmon-channel.

"Moji" Port Construction Office is now adopting the leveling method used for the trail of the drug-suction dredger 'Kaihomaru' that carries the dredging work in the East-South branch of the Kanmon-channel.

There might be little references in that works because that the subsoil is not sandy but silty, but I would like to introduce the outline of the work.

1. The leveling work site and the property of soil

The leveling site has the width of 150 m and the length of about 1.0 km.

The soil of sea bed is very soft silt including 30 percent clay and having considerably high water content of 90 ~ 110 percent.

2. The dredged site

Fig. - 20 and 21 show the trail of the drug-suction dredger 'Kaihomaru' and the sea depth after leveling works in 9 cross-sections with intervals of 120 m. The cross-section of dredged site in the normal direction to the fairway looks like the teeth of a comb with the wave length of 2 ~ 3 m and the ditch depth of 0.6 - 1.5 m (3 m in maximum).

While the longitudinal Section in parallel to the fairway looks the gentle slope with 5 ~ 10 m -wave length and 0.2 ~ 0.5 m difference of altitude.

3. The leveling method used for the dredged site

There have been many various methods for leveling the dredged site.

In the past time, we can find some leveling methods such as by a nearly 98 ton I-beam hanged by a floating crane in America or by a steel pipe in Japan.

Making reference to these examples, Moji Port Construction Office decided to use tentatively the method by blading bar which can be prepared considerably without trouble and operated speedily.

The spad (14 tons) in the front part of the dipper dredger 'Kaio', which was in the programme of a scrapped vessel, was used as the beam for the leveling works. No. 25 floating crane (lift capacity; 100 ton) and two tug-boat (350 PS) was used as working boats.

4. The effect of the leveling works

The difference of level on 30 surveying point with intervals of 5 m on 4-cross-section, which were selected from 9 cross section of the two Fig. - 20, - 21, was investigated in order to study the effect of the leveling works.

The Fig. - 22 shows the conclusion of this investigation and appendix table in Fig. - 22 shows the average thickness of the part scraped and reclaimed as the leveling works.

According to this investigation, the difference which was 44.7 ~ 65.6 cm in average before the leveling works changed into 16.7 ~27 cm and the average thickness of the scraped part was 35.0 - 73.5 cm and of the reclaimed one was 17.5~ 55.0 cm.

5. Problems in this leveling work method

(1) The level of blading bar was adjusted to the sea-bed by the echosounding, but the sea-bed was left uneven because of the insufficient control of sounding, which also caused the lack of pulling power since the bar sank under the sea-bed.

(2) The trail of tug-boat zigzagged by the un-equal subgrade reaction to the blading bar.

(3) The leveling-distance in the running direction of tug-boat was most effective in the distance of 500 - 1,000 m. The vessel-speed should be decided according to the speed of lifting operation (up or down) on the floating-crane in connection with the sounding work.

6. The leveling works on the sandy bed.

These was a example of the leveling works after dredging the sandy bed by the grab dredger at the Shinpin area in Port of Moji. In this case the blading bar had 5 m width, 2.5 ton weight and the blading hook in the lower this blading bar, however, was not effective because of the occurence of the swelling parts by the excessive thrust of the hook. So practically the leveling works was done in only one day and had not satisfactory actual results to reference.

In this case the working site was -10 ~ -11 m in sea-depth.

Appendix - 2 The Leveling Boat in Port of Nagoya.

In the Port of Nagoya the drug suction dredger "Kairyumaru" is taking part in the dredging works of navigations, and the leveling method of the uneven dredged sites has been one of the long - pending problems. So after various consideration, the construction of the leveling vessel was decided and was set up on Jan. 1967, and was completed on Feb. 1968. This vessel is now under test operation and the effect of this vessel remains unexplained. The following are the dimensions for reference.

1. Manoeuvrability

This ship must have high manoeuvrability in order to approach exactly the dredged site by drug suction dredger, and so it has the colt nozzle rudder with the changeable two-shaft.

2. Methods of positioning and sounding

In order to get the exact position of ship during the works, the positioning method by radio has been adopted. While in order to make exact sounding, the echo sounder with many elements is used.

3. The leveling work method

Four systems such as the blading bar system, the spattering system, the pumping system and the jetting system were considered as the leveling work method. The blading bar system was finally adopted, for it seemed to be better rather than others and had the actual results of leveling works in Japan and U.S.A., although it was very primitive.

Besides it has been considered that this boat may be equiped with pumps in future for the spot dredging works on shallow part although it is planned for deep area in principle.

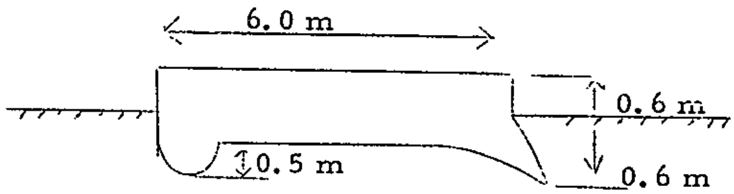
The pipes of water jets, which assists the blading bar in cutting works, are attached to the blading-bar.

4. The subsoil readtion in the leveling works

The form of the blading-bar and the subsoil reaction in the leveling works are basic in the design, and these was decided as follows by the indoor examination in Port and Harbor Research Institute and the fieled examination in the 5th Port Construction Bureau.

The form of the blading-bar is 10 m in width, 6 m in length, and 0.6~1.2 m in thickness.

The form of the blading-bar



The subsoil reaction to the blading-bar was decided as follows by the two examinations mentioned above.

The subsoil reaction to the blading-bar in works

Case	Reaction		
	Soundy soil (ton)	Cohesive soil (ton)	Adopted value (ton)
Upward reaction	8	12	12
Horizontal reaction	18~21	15 ~ 22	22

5 Condition in the leveling works

In addition to the form of the blading-bar and the subsoil reaction, pre-conditions to decide the design condition of the leveling boat are as follows.

Sailing speed	above 10 knot
The speed of leveling works	1.0 knot to the earth (in case that the boat runs against the current flow with 1.5 knot.)

Conditions of weather and sea at the time of works

Wind velocity	10 m/sec
Maximum wave height	1.0 m

The deepest leveling depth	- 18.0 m

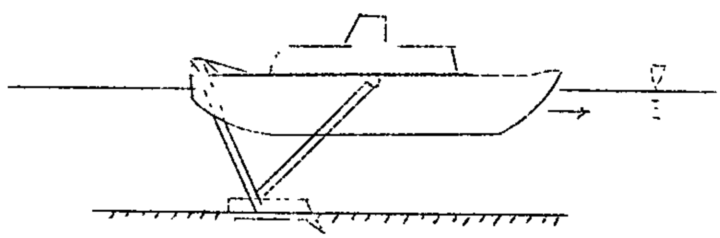
6. The dimensions of the leveling boat

(a) Main engine

This ship is equipped with two engines of 1100 ps in order to get the necessary sailing speed and working velocity.

(b) Hull

In order to obtain the expected requirements, this leveling boat was decided to be 590 ton in gross tonnage, 38.0 m in length, 10.0 m in width, 4.5 m in depth and 3.4 m in draft.



(c) Blading - bar

The weight of the blading - bar and its arm was decided in such way that their weight in the sea was balanced with their upward reaction during the leveling works.

The section of the arm was designed to accept one - thirds of 22 ton of the horizontal reaction in the upward direction during the leveling works.

The leveling - bar is suspended by three points, i. e. two points in the bow and one in the stern, (by two electric motors) and is operated according to indicators of the sea depth, the inclination and so on which are equipped at the bridge.

At the joint section between the arm and blading - bar the shear - pin, which is broken in case of unexpected overload, is installed, and in case of the breakdown of the pin, the measure to pull up the blading - bar has been considered.

(d) The hold for spoil and the jet pump

In case of the spot dredging in the future, the hold with the capacity of 120 m^3 and the jet pump for agitating the spoil in discharging them out, are also installed.

In order to use this jet pump as a assistant in excavation jet pipes are equipped at 5 m intervals in the front of the blading - bar through the arm.

7. This ship was constructed at the cost of 250, 000, 000 yen to perform the leveling works of the sandy bed in the deep sea, but it is under test driving and its effect is not evident.

Anyhow the leveling work in deep sea should be very difficult.