CHAPTER 83

PROTECTION BY MEANS OF OFFSHORE BREAKWATERS

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INTRODUCTION

The coast of Tel-Aviv is characterised by a narrow sandy beach, and the seabed is the nearshore shallow waters is strewn with rocky ledges and outcrops. These rocks are of a marine conglomerate type and are covered with layers of fine to medium grained sand of various thickness. At depths greater than 5 m the seabed is predominantly sandy. During the summer season, when waves and swells seldom exceed 2 m in amplitude, the sandy strip of the beach maintains its minimum width of about 20 m. However, winter storms reduce the width of the beach in some places to zero, and there is a marked tendency to erosion and scour of the beach in front of the retaining walls of the alongshore promenade or in front of the coastal bluff.

In order to prevent this erosion and at the same time to enlarge the sandy beach area, we have proposed to erect in front of the beach a series of offshore breakwaters, either detached or groyne-connected. (Fig. 1)

The first of the series was erected off the Tel-Baruch beach just north of Tel-Aviv. There beach sand was practically non-existent, and in order to enable bathers to enter the water, a breach in the shallow rocky belt had to be blasted.

The Tel-Baruch breadwater is of a rubble-mound type, 200 m long and connected with the shore by a 100 m long groyne. It is founded on a rocky seabed at -3.0 m below M.S.L. and consists of a quarry-run core (0.5 to 250 kg units), protected on its seaward slope by a rock armour (2-6 tons units) laid on a 1:3 grade, and on its landward slope by a secondary armour (1-2 tons units) laid on a 1:1.5 grade. Its crown is topped by R.C. 0.25 thick slabs to a level of +1.0 m above M.S.L.

Shortly after its erection in 1965, a sandy tombolo has formed at both sides of the groyne, increasing in area until a permanent equilibrium has been achieved. Erosion of the beach on both sides of the breakwater was avoided, owing to a belt of beach rock which protects the foreshore to the south and to the north of the breakwater.

The second system was erected in front of the Tel-Aviv Sheraton and Hilton hotels. (Fig. 2)

It consists of twin breakwaters hore connected by groins. Each of the rubble-mound type breakwaters is about 250 m long and 200 m distant from the shore. They are based on a practically rocky seabed at a -4.0 m level and constructed of quarry-run (0.5 ~ 500 kg units) core and blanket, a seaward armour slope 1:2.5, consisting of 4-8 tons rock units and a landward slope 1:1.5 of 3-4 tons rocks. The crown of the breakwaters is capped by R.C. slabs, 0.25 m thick, and reach a level of +1.25 m. The gap between the breakwaters' heads is 135 m wide.

Soon after the completion of these breakwaters in 1968 the barren rocky foreshore behind them was transformed into a wide sandy beach protected from summer swells and safe to bathers. The sandy tombolos expanded steadily, reaching an equilibrium in about 3 years time, thus permitting the erection of beach amenities (wardrobes, services, showers, refreshment stands) on the vast sandy areas. Behind both these breakwaters the apices of the tombolos from a permanent contact with their inner edge, thus obstructing free passage of water, even during the winter season.

For this and other reasons it was decided to create protected beaches at the central coast of Tel-Aviv south of the newly built "Gordon" boating marina in such a way that all the year round ample gaps should be left between the breakwaters and the sandy tombolos behind them. Moreover, it was deemed desirable from the aesthetic and beach utilisation points of view to form the new shoreline behind the row of detached breakwaters in a sinusoidal shape. The crests of such sinusoids should lie behind the breakwaters at a distance of about 60 m from the actual shoreline, whilst their troughs, situated opposite the gaps between the breakwaters, should be about 20 m distant from the original shoreline. (Fig. 5)

This new concept put before the designers a dilemma how to proportionate the row of detached breakwaters (their length, distance from the shore and between each other) in order to fulfil the basic requirements of the new beach. In order to help the designers to find the right answers to these questions it was decided to resort to movable bed tri-dimensioned hydraulic model tests.

MAIN CHARACTERISTICS OF THE LOCAL CONDITIONS

In order to build a suitable movable bed model in a hydraulic laboratory wave basin and to submit it to a series of tests, it is necessary to collect as many data as possible concerning the physical conditions ruling in the coastal zone under investigation.

For this purpose a series of preliminary surveys and studies of the central Tel-Aviv coastal zone has been undertaken, consisting of:

a. Topographic and hydrographic mapping;

b. Wave observations and recordings;

c. Longshore currents measurements at various depths; littoral transport evaluation;

d. Sampling of sand from the beach and from the seabed at various depths;

e. Pricking into the seabed in order to ascertain the depth of hardpan underneath the sandy layers.

f. Assembly of meteorological and geomorphological data.

The results of the above-mentioned surveys and studies have made it possible to characterise and to analyse the coastal conditions as follows:

The tidal range seldom exceeds 50 cm; however, strong easterly winds may lower the water level to -50 cm below M.S.L., whilst persisting westerly storms may cause a temporary piling up of coastal waters, bringing their level to 70 cm above M.S.L.; but these are extreme events.

The shore of Tel-Aviv is basically sandy with an underlying rocky bottom, which is partly based in shallow water, and in some places near the coastline rocky ledges protrude above M.S.L. The sand cover above the rocky bottom increases in thickness from 1-2 m at the coastline to about 7 m at the -12 m contourline. The seaward slope ranges from an average of 1 to 40 nearshore, to 1 to 75 beyond the -7 m contour. Between the two zones there is a sand bar with a levigation ditch at about -4 m to -5 m depth. The orientation of the shoreline is about 19^{o} East of North. The grain-size distribution of sand samples taken at the foreshore and a certain depths indicates uniformity and sediment sorting, the foreshore samples being much coarser than the offshore samples. The mean diameter changes with the depth as follows:

Depth (m)	foreshore	-5	-7	-13
d (mm)	0.26	0.17	0.16	0.14

The maximum 90% diameter of foreshore samples reaches 0.40 mm.

The winds blow mainly from the NW to SW sector, i.e. from the open sea. Winds above 5 Beaufort (16 knots) strength blow only 5% of the time.

The distribution of significant waves amplitudes and periods shows that most of the storms and all waves higher than 5 m occur between November and April. Their direction is from the WSW and WNW sector, and they account

only for 1% of all waves. The average number of moderate storms (3-4 m high waves) during the summer period is 2 or 3 (1.5% of all waves).

The frequency of amplitudes and periods were found as follows:

Amplitudes (m)	6-8	5-6	4-5	3-4	2-3	1.5-2	1-1.5	0.5-1	0-0.5
%	0.37	0.45	1.1	1.58	4.1	9.9	28.0	11.9	42.6

Periods (sec)	10-12	9-10	8-9	7-8	6-7	5-6	4-5	3-4	Calm
%	2.0	2.8	7.9	16.5	19.6	23.3	14.4	3.1	10.4

The pattern of wave distribution around the year had to be simulated in the model in order to find the movements of the bottom observed in nature. The currents were measured, both at the surface and at various depths. The results of these measurements have shown that their pattern is irregular and their maximum velocities seldom exceed 25 cm/sec. These low velocities point to the fact that the currents themselves play only an auxiliary role in the movement of sediments put into suspension by shoaling waves.

The estimates concerning the littoral drift at the coast of Tel-Aviv were as follows:

The northward transport - about 400,000 cu.m. a year, the southward transport - much less, about 80,000 cu.m. a year. This leaves a net northward littoral transport of at least 320,000 cu.m. a year.

HYDRAULIC MODEL TESTS

The movable bed hydraulic model was built in the wave basin of the hydraulic Engineering Laboratory of the Haifa Technion (Israel Institute of Technology). The basin's dimensions were 40 m x 25 m x 0.9 m, and it was equipped with a 30 m long movable wave generator, capable of turning in various directions; an overflow level regulator and a movable measuring bridge spanning the area occupied by the model. Frequency and amplitude modulations were effected according to a prescribed program.

The model study began at the end of May 1971 and was completed 16 months later, i.e. October 1972.

The choice of the model scales was influenced by many factors, the most important being the relation between the length of the coastal strip to be tested and the actual dimensions of the wave basin. It was decided to reproduce the model beach with a distortion 1:2, using ground ebonite

as model sand. Thus the adopted horizontal scale was 1:150 and the vertical scale 1:75, while the model sand, the average specific gravity of which being about 1.8, was prepared with a grain size distribution found in the prototype, using a 1:1 scale. The scale of the sand is based in principle on the similitude of fall-velocities, assuming that the major part of the sand transport is usually carried in suspension, and the setting in the protected area is basically governed by the fall velocity - carrying capacity relation, too.

Calibration of the model was effected relative to the sedimentological processes observed in the vicinity of the existing structures (Tel-Aviv lighter basin, Sheraton and Hilton breakwaters). (Fig. 3) There were some indications in the calibration tests that the velocities reproduced on the Froudian scale with the adopted distortion were insufficient for the establishment of a reasonably small sedimentological time scale. Therefore it was decided to increase the model scale for waves, using a scale of 1:50 for wave hights, which were calculated on an energy basis. However, breaking of waves and overtopping of low breakwaters by them are governed by Froudian similitude. Therefore, the breakwaters had to be heightened in the model in such a way that the height of their crest above mean water level corresponds to a 1:50 scale, too. The most difficult problem was to establish a sedimentological time scale. It could not be made before the reproduction of sedimentological changes around the existing structures. Therefore, at the beginning of the calibration a tentative wave program was adopted, which corresponded to the statistical distribution of wave energy and wave direction, but the corresponding wave cycle did not necessarily represent a model year. Then theoretical calculations were made as to the possible sedimentological time scale, and the overall time of the wave cycle was modified accordingly.

Practically the time-scale was found by comparing the time-rate of the tombolo formation at the two existing breakwaters with their development rate in the model. Comparing model to prototype has led to the unavoidable conclusion that two different time scales have to be applied relative to the tombolo formation - one for the initial stage, when the changes are due to shifting of the local sand stock and to longshore transport, and the other when the development of the tombolos is due to the trapping of sand which arrives mainly from the unprotected offshore area. This is because an offshore transport was observed in the model, which was believed to be in excess of the corresponding offshore transport in the prototype and which could not easily be eliminated. However, the final equilibrium state of the tombolos was correctly reproduced, and this indicates that the similitude of the sedimentological process to be studied in the model was basically correct. The basic sedimentological time scale adopted in the beginning was 1:2190 = 1:2200, i.e. 4 model hours equivalent to one prototype year.

Inasmuch as the development of the tombolos in the first year was correctly reproduced on the adopted time scale, however, the additional growth caused by accretion of sand supplied by onshore transport was slowed down by about 1:3, as proved by the Hilton-Sheraton breakwaters test. At a later stage the sedimentological time scale was re-checked, and as average of 12 hrs. equivalent to one prototype year adopted. Altogether 14 different alternatives were tested in the model study.

TEST RESULTS

The original concept of the outlay of a complete system was a row of offshore breakwaters parallel to the shore with evenly distributed gaps between them, except two wider gaps: one 225 m and the other 310 m long. Also the existence of the first northernmost breakwater, which was erected at the same time as the "Gordon" marina breakwaters, had to be accounted for. The breakwater is situated about 250 m from the original shoreline. Therefore, alternatives 1 to 9 have shown breakwaters situated all at this distance from the shore.

Testing the first alternatives with two wide gaps between the breakwaters indicated serious erosion opposite these gaps. Therefore, other solutions were sought that would provide sufficient widening of the beach. It was made clear by the results of the tests that the proportion between breakwater lengths, gaps and distances from the original shoreline must have definite values, to be found by testing procedure, in order to meet the prescribed requirements concerning controlled tombolo formation. The fact that the hydrographic conditions are quite uniform along the shore section in question, and architectural considerations as well, lead to a symmetrical outlay of similar breakwater units equally spaced.

Alternative 9 was tested with six uniform breakwaters, 130 m and 120 m long gaps, all aligned with the existing northernmost breakwater (Fig. 4).

The result of the tests with this alternative seemed to be quite satisfactory, except that the tombolos showed an undesirable tendency to reach the northern units of the system, contrary to requirements.

Another alternative, No. 10, was then composed, similar to alternative No. 9, of six 130 m long breakwaters; however, the five intermediary breakwaters were displaced offshore by 45 m. Thus a wider lagoon was created, which made the connection of the tombolos with the breakwaters unlikely, except with the existing one, the position of which could not be changed anymore.

In addition, two oblique gaps were formed, which made the penetration of longshore currents, and thus circulation and exchange of water, easier. It was then decided to choose alternative No. 10 as the most suitable outlay of the system.

The choice of the best outlay was based on comparative tests using artificial fill; however, the final version had to be re-tested with a natural process, when the building schedule of the system is correctly reproduced in the model. Owing to the considerable differences in the sedimentological time-scales at the beginning and at the end of the construction period, a mixed average of 12 hours, equivalent to one prototype year for the building period of the whole breakwater system, was finally adopted.

The testing of the natural sedimentological process in the chosen outlay was repeated with three different versions, called alternatives Nos. 12, 13 and 14, differing between them in the time sequence of the construction and other details. Alternative 12 was tested for a construction period of one year for the whole system. In alternative 13 two other time schedules were tested, building of 3 breakwaters per year and building 3 breakwaters in 2 years. This would mean extension of the building period to two and a half and five years, respectively, considering additional five offshore breakwaters and the southern end unit.

The purpose of alternative No. 14 was to improve the distribution of sand accumulating along the protected beach by partition of the protected water area, leaving the groin of the central unit in place, or dismantling, if only partially. The nearshore part of it would very soon be covered, while the gap left between the groin and the breakwater would facilitate the exchange of water between the northern and the southern parts of the lagoon.

In the final version the central breakwater was moved back by 45 m to its position in alternative No. 9, i.e. in line with the end units. (Fig. 5) Thus two inter-connected symmetrical lagoons were formed with considerably improved sedimentological and hydraulic behaviour. The results of the test have shown that the accumulation of sand in the protected area is caused partly by the longshore transport. With an open ended system more sand is attracted from the longshore current, and consequently sedimentation is quicker than in a closed one. If the system is closed, this sand will probably by-pass the breakwaters, apart from the small part trapped behind them. This means that the influence of the construction of the system on the coastline is spread over a longer period, which is certainly beneficial to the coast. Consequently, the building schedule has a direct influence on the rate of sand accretion, though not on the final state of equilibrium. Construction of the whole scheme can be sped up by using artificial feeding. However, if the rate of construction is not exceeding one breakwater a year

(as it actually is), sufficient widening of the beach will be achieved by a natural process. After completion of the model study it was decided to extend the breakwater scheme to the south in order to connect it with the "Clore" park reclamation. It was suggested on the basis of the model study that this can be done by adding one more breakwater to the system, maintaining however the dimensions of the breakwater units and of the gaps, which were found to be optimal. Thus the finally recommended outlay is composed of seven offshore breakwaters and an end unit shore connected to the seawall of the "Clore" park reclamation. The head of the southern end breakwater No. 8 and the central breakwater No. 4 are aligned with the existing breakwater No. 1, nearest to the Gordon marina. The remaining five breakwaters are displaced by 50 m in the offshore direction. The length of each offshore breakwater is 130 m, and the gaps between their heads are 120 m long.

The central groin No. 4 will be dismantled only partially, leaving a distance between its head and the central offshore unit. The crests of the offshore breakwater are at +1.75 m above M.S.L.

With an outlay of the system described above, the shore development of the protected beach was expected to satisfy the basic requirements when its new equilibrium is established. (Fig. 6)

The model indicated a 100 to 200 m wide lagoon for swimming purposes, protected from waves mostly in the leeside (the "shadow") of the breakwaters where the beach widens, and to a lesser extend opposite the gaps where the beach narrows. At any rate, the average wave energy along the beach should actually be less than 50% of the energy in the open sea. As long as the waves do not overtop the breakwaters - and this will not happen if the wave heights are less than 2.0 m, as they actually are during the summer season - the water in the lagoon will be calm in general. If overtopping occurs during stormy periods, the reduction of wave heights by the breakwaters is less effective.

FIELD CONSTRUCTION WORKS

Breakwater No. 1 was completed in 1971, and a tombolo has formed that was very similar to the one predicted by the model for the same period of existence. In 1972 No. 2 breakwater was completed and its groin dismantled, using the rock to build the groin of No. 3 system. However, due to war and the policy of the authority concerned, No. 3 breakwater was completed only in 1975. It was decided to proceed with the construction, and this year No. 4 breakwater is in the process of being completed. (Fig. 6)

The method of constructing the offshore breakwaters is as follows:(Fig. 7)

First, rubble-mound type groins are constructed from the shore outwards by dumping on the seabed a core material to a +0.75 m level. This core material consists of a well graded quarry-run (0.5 to 500 kg units), protected on both sides by 3-5 ton rocks placed at a 1:1.5 stope. The width of the groin crest is about 5 m (a minimum necessary to enable the passage of crawler cranes, which place rock armour on the breakwaters). Then the construction of the breakwaters proceeds from their mid-section outwards by laying first on the seabed a 1.0 m thick quarry-run blanket, which is to protrude 3 m outside the outer toe and 2 m outside the inner toe of the breakwater mound. This blanket constitutes an anti-scour device, protecting the toes of the breakwaters and preventing excessive penetration of the armour rock into the sandy seabed.

The quarry-run core of the breakwaters is then dumped on the blanket and protected on its outer (offshore) slope by rock armour (5-8 ton units), placed on a 1:3 grade on its inner (inshore) slope by a somewhat lighter armour (3-5 ton units) laid on a 1:1.5 grade. The breakwater crests are capped to a +1.75 m level by the heavier armour rocks. Also the heads of the breakwaters are protected all around by 5-8 ton rock units placed on a 1:3 grade. The width of the breakwaters at their crest level is 5.25 m.

After completion of each breakwater, the connecting groin is dismantled and the rocks transferred to the next groin. These rubble-mound type structures have proved to be very stable, and even after heavy storms no significant damage have been observed until now.

Periodical hydrographic surveys executed in the area, as well as air photos, indicate that the sedimentation process is generally well in accordance with the prediction of the model study. Now one can see a substantial widening of the protected beach and the appearance of a convenient new bathing area. There has, however, been less sand accumulation at the two northernmost tombolos just behind breakwaters No. 1 and No. 2 than anticipated in the model study. This indicates that sand distribution along the protected shore is more balanced than predicted. No permanent erosion was observed on the adjacent shore due to the above development. This fact corroborates the preliminary assumption that the newly developed beach will not be widened on account of the neighbouring shore sectors, but on account of the differential alongshore sand transport, reducing only the quantity of sand that would otherwise be carried offshore due to the local hydrographical and sedimentological circumstances. (Fig. 8)

The implementation of the offshore breakwaters scheme, together with a substantial widening of the sandy beach behind them, will enable the establishment of a new ample alongshore promenade seawards of the existing one without jeopardising the newly created beach.

CONCLUSIONS

Many factors are involved in the reproduction of natural sedimentological processes in a wave basin with movable bed, and a misinterpretation of some of them may lead to erroneous conclusions. In the long run, the exact definition of the sedimentological time-scale is less important than the reproduction of seabed configuration, representation of sand grain particles and of wave spectra.

The exact full-scale sedimentation processes in the prototype are hardly known in detail. However, we can assume that the formation of sandy tombolos in the lee of offshore breakwaters results mainly from the interception of the littoral drift. Some of the sand enters directly downdrift the onshore zone behind a breakwater, whilst other sand particles by-pass the breakwater on the outside and are then directed by waves diffraction around its opposite head into the calm foreshore zone as a result of onshore movement caused by waves agitation. The accelerated sedimentological process in the model enables however to reproduce quite exactly the natural development of tombolos formation in the prototype, which, in order to reach a state of equilibrium requires a considerable period of time. Nowadays, even a well experienced designer of coastal structures requires a corroboration of his ideas by model studies, which by themselves are not substitutes for a careful and sound design, but serve as practical indicators of their behaviour in the prototype.

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4. I. Fried - "Foreshore and Beach Development of Tel-Aviv and Nathanya" - Symposium on Foreshore and Beach Development from the Coastal Engineering Aspect - Tel-Aviv, January 1975 (in Hebrew).

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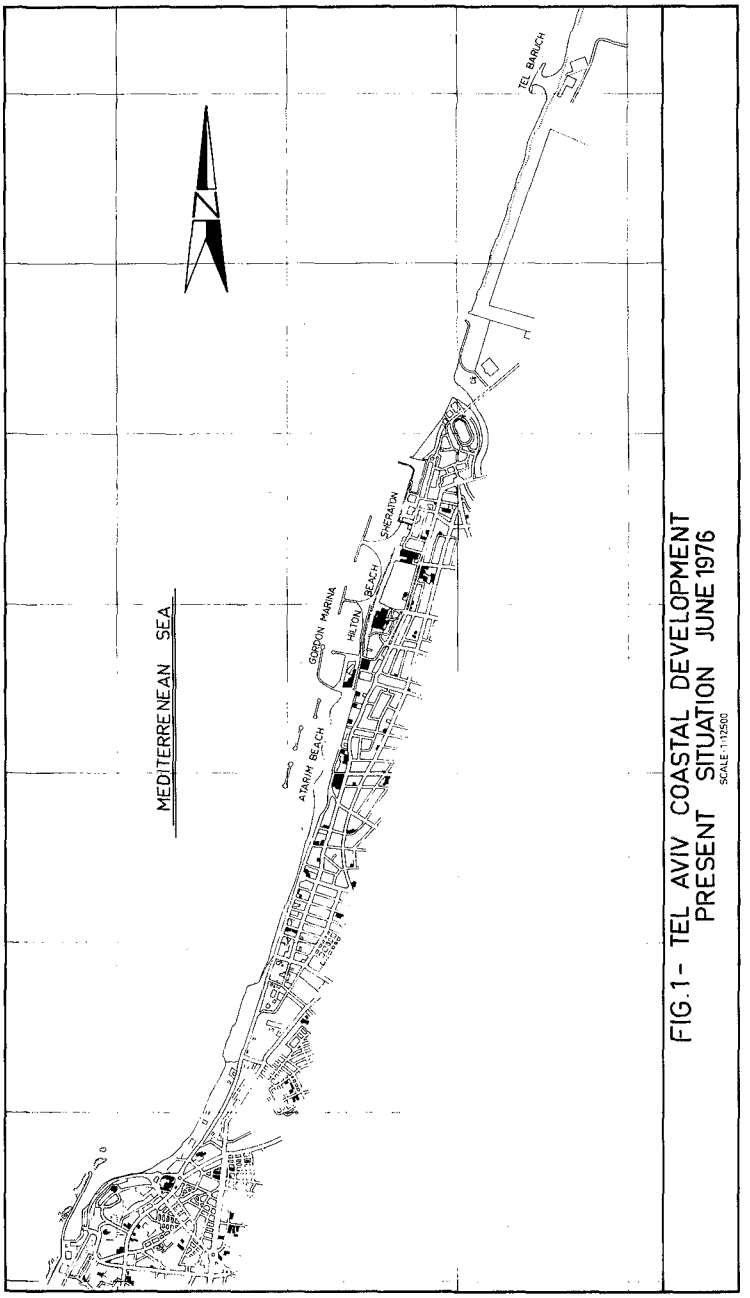


FIG.1- TEL AVIV COASTAL DEVELOPMENT
PRESENT SITUATION JUNE 1976
SCALE 1:12500

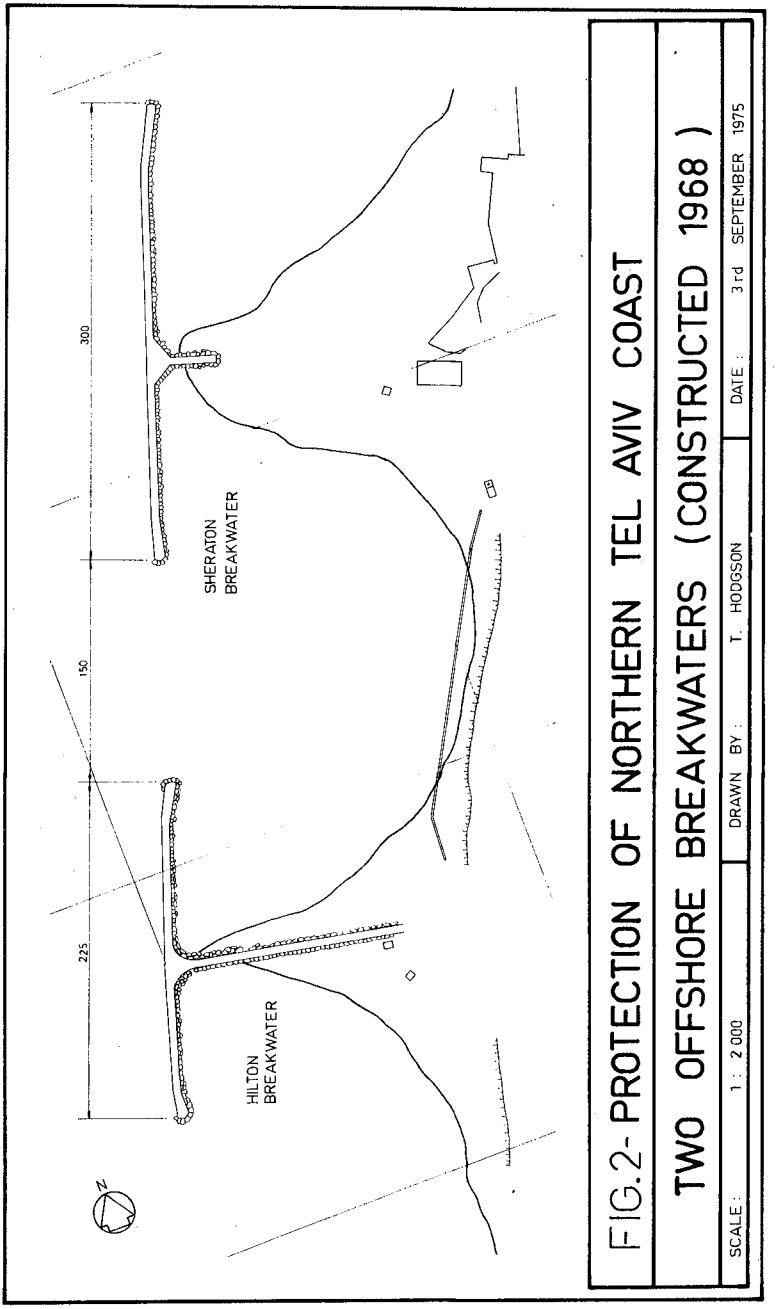


FIG. 2- PROTECTION OF NORTHERN TEL AVIV COAST
TWO OFFSHORE BREAKWATERS (CONSTRUCTED 1968)

SCALE : 1 : 2.000	DRAWN BY : T. HODGSON	DATE : 3rd SEPTEMBER 1975

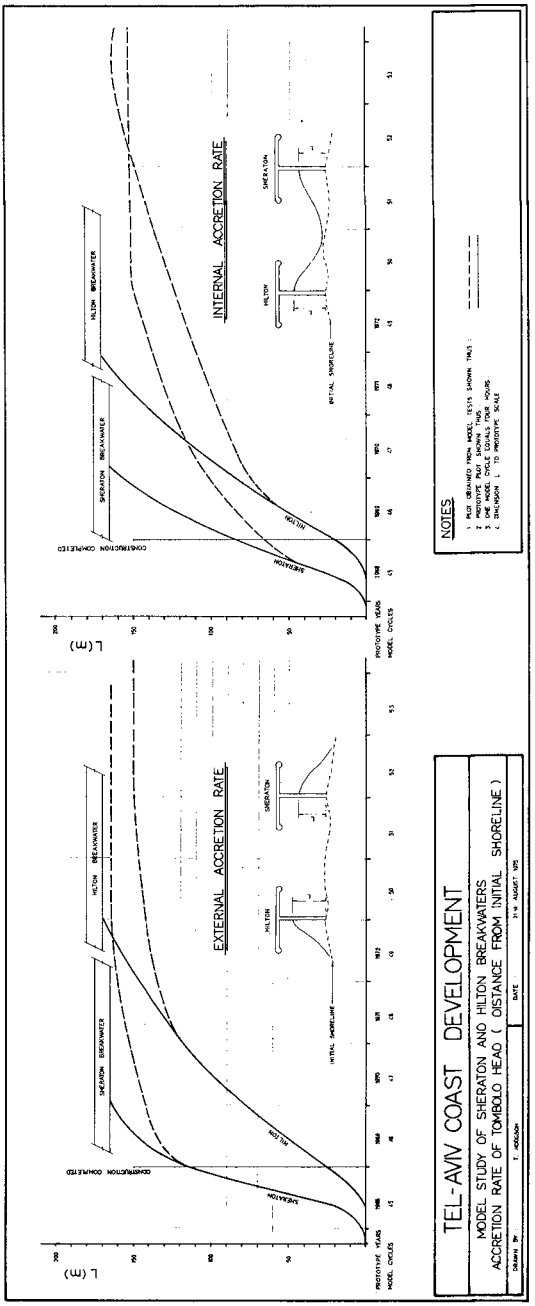


FIG.3 - SHERATON AND HILTON BREAKWATERS ACCRETION RATE OF TOMBOLOS

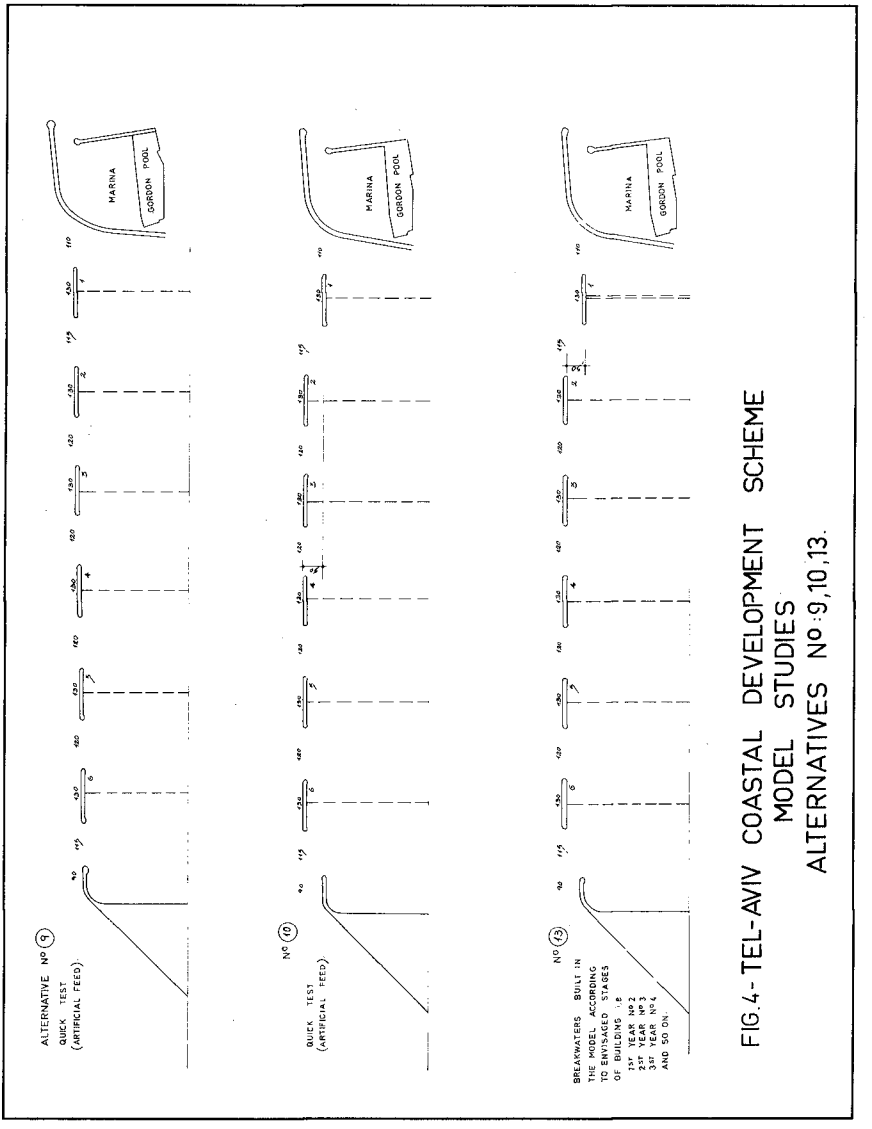


FIG. 4- TEL-AVIV COASTAL DEVELOPMENT SCHEME
MODEL STUDIES
ALTERNATIVES Nº: 9, 10, 13.

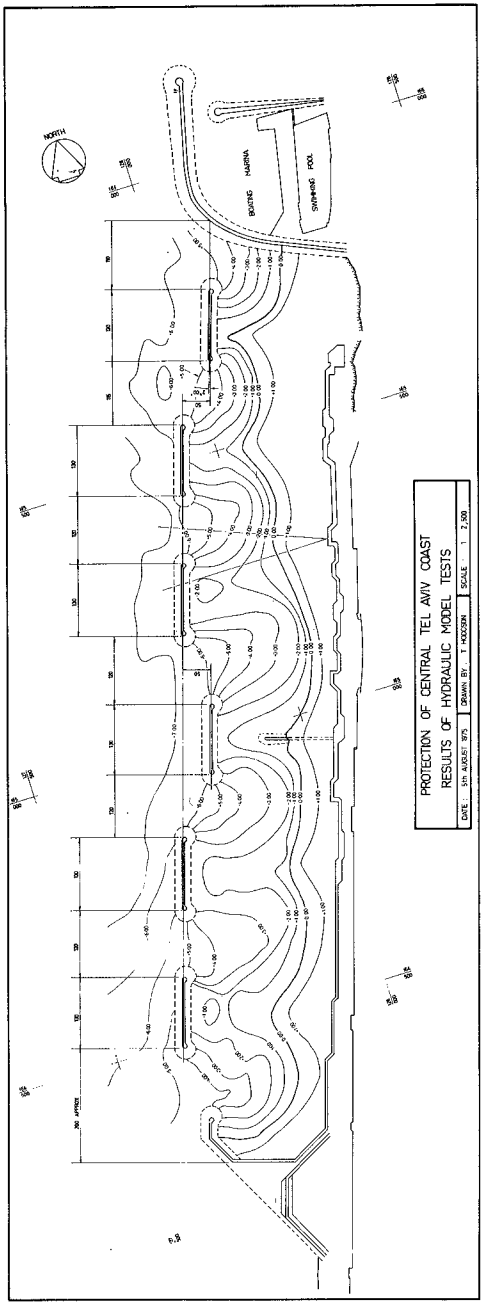
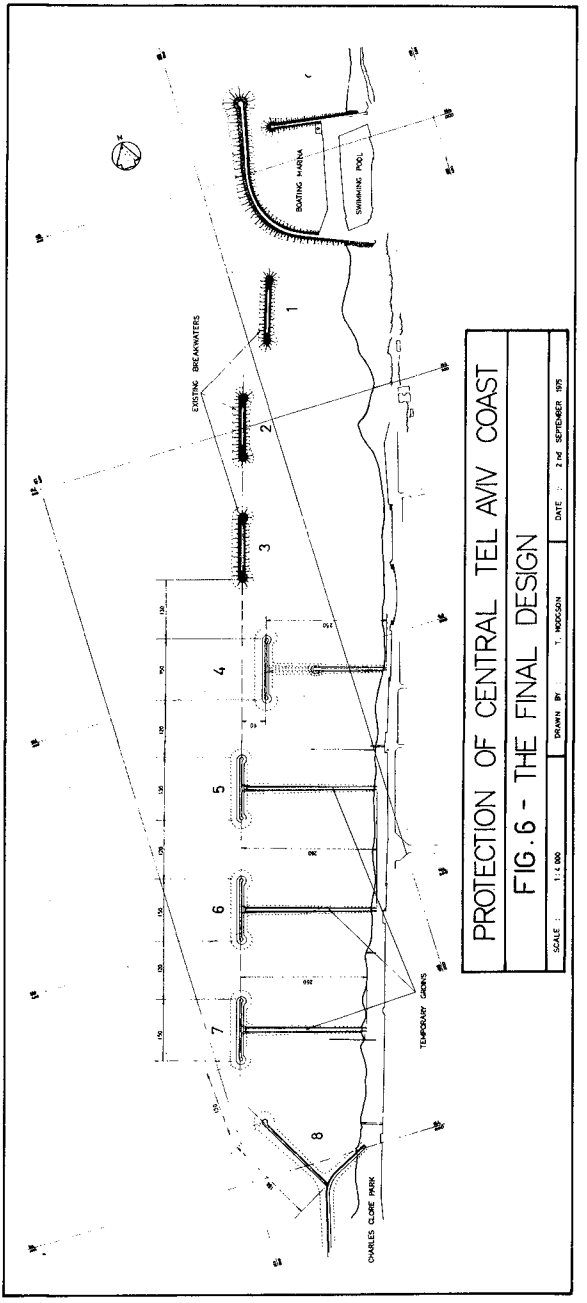


FIG.5 - FINAL RESULTS OF HYDRAULIC MODEL TESTS


PROTECTION OF CENTRAL TEL AVIV COAST
FIG. 6 - THE FINAL DESIGN
SCALE : 1:2 000
DRAWN BY T. MODGSON
DATE : 2 nd SEPTEMBER 1975
BOATING MARINA
SWIMMING POOL
EXISTING BREAKWATERS
TEMPORARY GROINS
CHARLES CLORE PARK
1
2
3
4
5
6
7
8

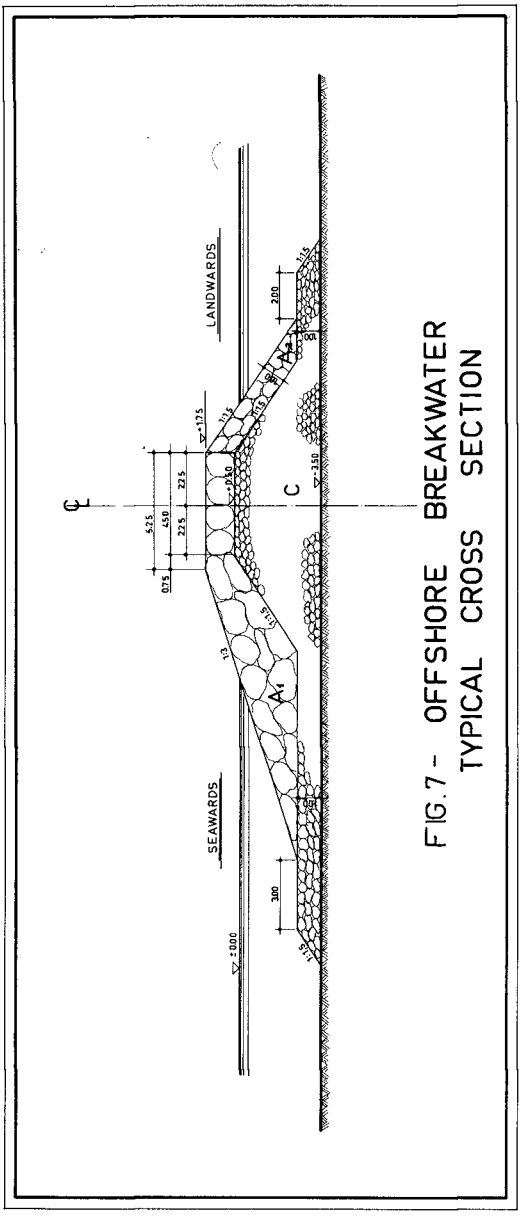


FIG.7 - OFFSHORE BREAKWATER
TYPICAL CROSS SECTION

FIG.8 – CENTRAL COAST OF TEL AVIV – JUNE 1976