ZEEBRUGGE'S MAIN BREAKWATERS

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ABSTRACT

The design scheme of the Zeebrugge Outer Harbour, Belgium, consists mainly of two breakwaters protruding into the sea as far as 1,750 m beyond the existing môle or 3.000 m out from the coastline. The west outer breakwater is 4,450 m long, the east breakwater runs 4,300 m out from the seafront. The east outer harbour will accommodate terminals for liquid bulk products such as LNG. The west outer harbour will provide space to install two harbour bassins to suit general cargo, hazardous cargo, container and ferry traffic.

In the paper emphasis is put on the environmental design conditions (wave height, wave period, water depth), the development of preliminary designs and the final design. Some design features such as dimensions, wave breaking carpet, armour units, workable limits in respect to rock grade, etc ... are discussed.

By developing the design the rubble-mound breakwater has been judged to be the only viable alternative versus the caisson type breakwater, taking into account costs, technical risks, construction problems and flexibility under changing environmental conditions.

The main feature of the breakwaters is that a huge concrete parapet will have been avoided. A so called flat semi-cube armour unit has been developed. The main advantage lies in the substantial economic benefit whilst maintaining the same stability performance as a concrete cube armour unit.

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1. INTRODUCTION

The construction of the existing port of Zeebrugge has been a royal decision, pronounced by H.M. King Leopold II in 1881. A few years later the construction started and in 1907 the port has been inaugurated.

Almost a century later, in 1970, the Belgian Government decided to the extension of the port complex. Prior to this decision the port has been adapted to the prevailing traffic demands whilst remaining within the boundaries set by its initial development scheme of 1907. The annual throughput has been some 10 million tons in 1971 and 15 million tons in 1981.

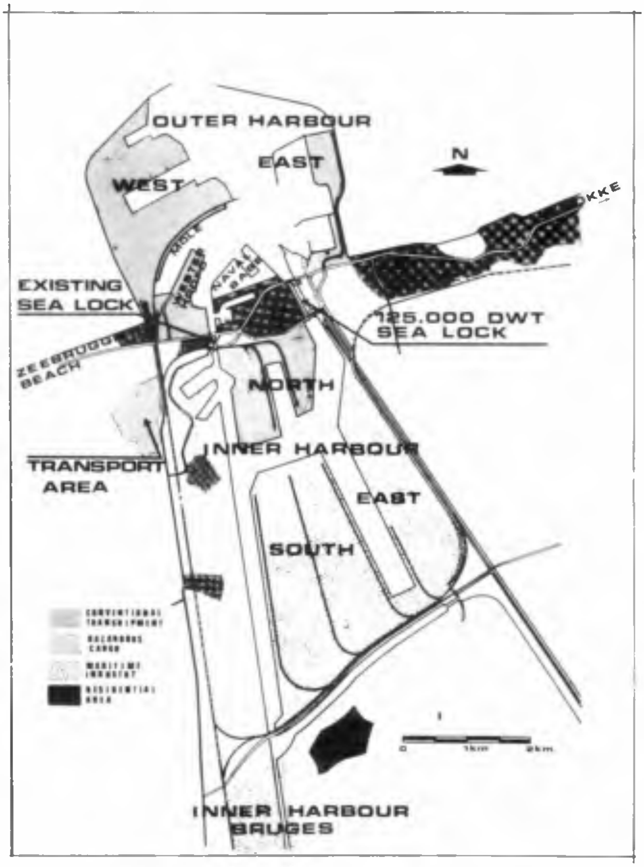

Fig. 1 : General layout of the Zeebrugge Port Extension Scheme

The master plan finally adopted provides 3,370 ha sub-divided (fig. 1) as follows :

- Outer harbour, including the existing outerport and naval base	1,165 ha
- Inner harbour	1,705 ha
- Port area of Bruges	450 ha
- Transport area	50 ha

The design scheme of the outer harbour consists mainly of two breakwaters protruding into the sea as far as 1,750 m beyond the existing môle. The west outer harbour breakwater is 4,450 m long. The east breakwater runs 4,300 m out from the seafront (fig. 2). The east outer harbour will accommodate terminals for liquid bulk product such as LNG. The west outer harbour will provide space to install two harbour basins to suit general cargo, hazardous cargo, container and ferry traffic.

2. PLANNING

The first phase of the east outer port sets up a newly built work harbour as the existing outer port is congested and no space is available to install the necessary harbour facilities and construction site yards.

Only the mound type has been retained for the service-port breakwaters. The sand-asphalt mound with open stone asphalt revetment has been selected against the rubble mound design by balancing construction costs, design and construction risks and practical construction aspects.

The southern east breakwater has been conceived as a sandfilled peninsula of ca. 36 ha. in order to install the land-based LNG-terminal facilities. The perimeter and sea defence of this peninsula are provided by rubble-mound breakwaters of ca. 1,360 m long to the west ca. 1,200 m long to the east and a north closure of ca. 500 m long.

The armour layers and inner slopes have been adapted to wave conditions and exposure time. The west slope (1:2) with 20 T cubic concrete blocks in single layer, the east slope (1:2) in quarry rock 3/6 ton grade and the north slope (1:1,5) in double layer 25 T cubic concrete blocks. The inner slopes vary from 2/300 kg quarry run to quarry rock 1/3 ton grade in double layers. Several design alternatives following different construction schemes and methods have been contemplated. Special attention has been paid to the filter construction between the rubble cores and the hydraulic sand-fill. (fig. 3)

The LNG-harbour proper will be completed by a semi-curved 975 m long, low crested rubble mound, the so called LNG-breakwater. The layout, planning and design of the LNG-terminal harbour works have been finally adopted to satisfy the stringent and draconic time schedule of the LNG-terminal project as a whole. (fig. 2)

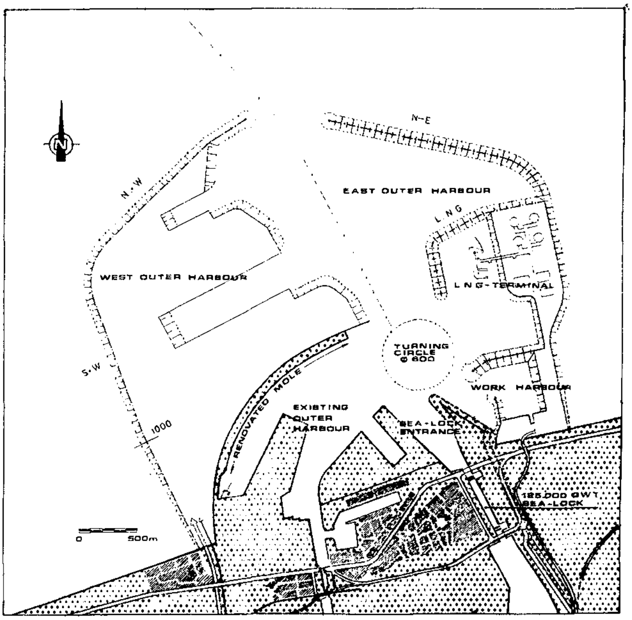

Fig. 2 : Layout Masterplan Zeebrugge Outer Harbour

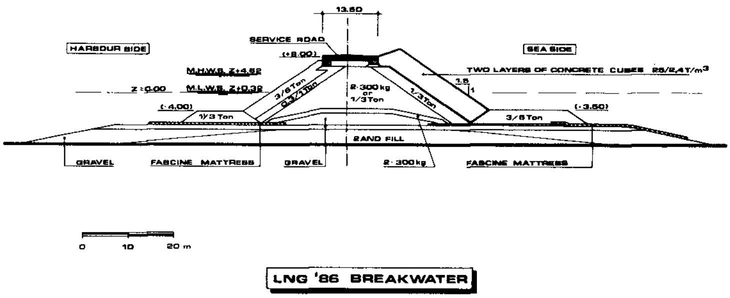

Fig. 3 : Typical cross section LNG '86 - breakwater

The northern section of the east breakwater is ca. 2,000 m in length. The west breakwater will be 4,400 m long. Up to 1,000 m out from the shore, the area between this breakwater root-section and the ancient môle has been hydraulically filled with seasand to create a secondary work and marshalling yard for the quarry rock to be worked up by truck dumper and crawler crane into the west breakwater. Because of the adjoining sandy beaches of the holiday resorts, this breakwater section has been conceived with due regard to the environmental visual impact.

In general the design up to the Z-4.00 bottom contour line (ca. 1,000 m out of the seafront) consists of a core built successively in sand, brick debris and quarry run with a concrete open pavement revetment on sublayers of crushed gravel and quarry rock of grade of 2/300 kg to 1/3 tons.

3. ENVIRONMENTAL DESIGN CONDITIONS

3.1. The **bottom** depth in the alignment of the outer harbour breakwaters ranges fairly between Z-5.00 and Z-7.00 (chart datum Z = mean low water spring + 0.08).

3.2. The **tide** is semi-diurnal with levels ranging from amplitudes of 4.40 m at mean spring tide to 2.80 m at mean neap tide. The **meteorological** set-up is up to 2.45 m in the defined design period and probability of exceedance.
The **tidal currents** at the final breakwater alignment will be (at surface) 1.2 m/s to 2.00 m/s at spring flood tide and 1.0 m/s to 1.6 m/s at spring ebb tide.

3.3. The **soil conditions** in the Zeebrugge area are rather difficult. A comprehensive geotechnical survey was undertaken with boreholes, static cone penetration tests and continuous seismic profiling.

The synthetically recorded geological profile is presented in fig. 4. The striking point is the rather pronounced accidental surface separating the tertiary sediments and the quaternary coverlayers. With as main feature the cuesta of the Bartonian clay formation with dips up to 30 % .

The less resistant bottom "sandwichlayers" consisting of alternate thin layers of soft clay and sand, mainly ca. 4 to 6 m in thickness, cover in the wester breakwater alignment, a considerable deposit of fairly dense to densely packed quaternary sand. In the eastern-alignment the deposit of quaternary sand is up to 15 m thick upon the tertiary Bartonian clay deposit. This tertiary layer rises towards the sandwich voverlayer at the head eastern breakwater.

4. DESIGN WAVE CONDITIONS

4.1. Design wave height

From 1958 to 1971 waves are systematically measured by shipborne wave recorders at the WESTHINDER lightship more than 40 km from Zeebrugge. Since 1977 wave riders record steadly at several locations in the vicinity of Zeebrugge. The measured data transmitted is analysed and synthesized by computer to statistical profiles of several relevant wave parameters.

The preliminary and final designs are based on the wave climate derived from the WESTHINDER data by refraction analyses. This analysis being carried out for different approaches, wave periods and waterlevels. The analysis showed that the existing seabed gives areas of wave concentration at the northern west and east breakwater with coefficients from 1.4 up to 1.9. The channel Pas van 't Zand / Ribzand dissipates wave energy, the coefficient at the planned harbourmouth can be 0.5.

The detailed designs are based on the more long-term wave data (1958-1971) compared and correlated with the more short-term measurements from the wave-rider buoys. In the graphs 5 and 6 the results of the comparison with the wave rider data of January '78 to December '80 is explained. Summarized :

- For the same probability of exceedance the originally established wave heights (H_s) are, compared with the wave rider values, over-estimated by :

  0.40 to 1.40 m at the N.E. - breakwater
  0.40 to 0.70 m at the W - breakwater
  0.10 to 0.50 m at the harbour entrance

- The design wave height of the outer harbour breakwater H_s = 6,10 m needs not to be changed as it is still defined by the limiting waterdepth.

- The design wave height of the LNG-breakwater and other breakwater sections defined by an average annual exceedance of 10 % (short-term risks of less than 5 years of exposure time) can be changed from H_s = 5.75 to 4.90 m.

- The workability defined on the H_s = 1.00 m and H_s = 2.00 m threshold values has been underestimated by roughly 7 to 8 % .

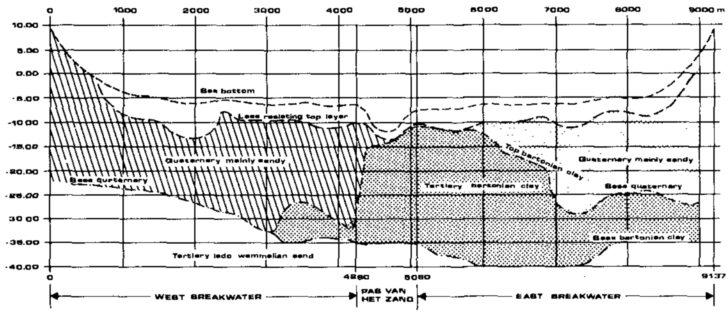

Fig. 4 : Geological profile outer harbour breakwater alignment

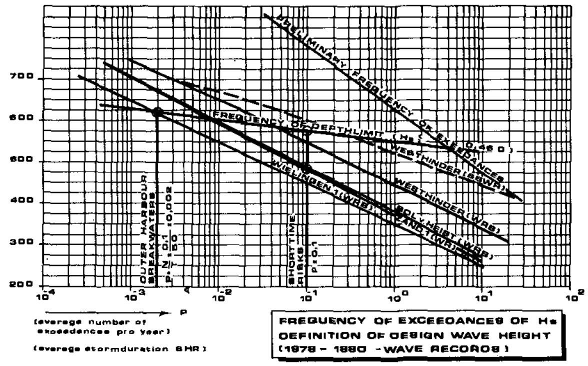

Fig. 5 : Wave height distribution - re-iteration and comparison procedure (wave rider measurements '78 - '80)

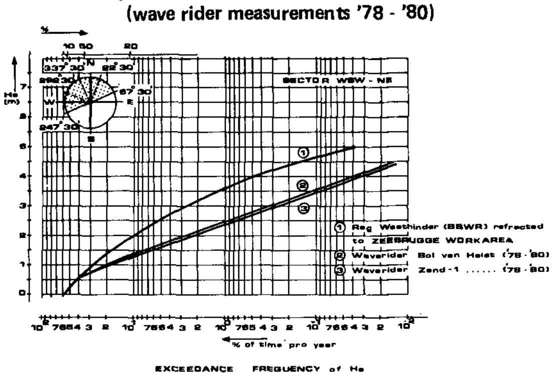

Fig. 6 : Wave climate at the Zeebrugge worksite

4.2. Design wave periods

From the Westhinder wave measurements and from the 1977 recorded wave-rider data the following conclusions can be made :

- the maximum mean period : 7 sec.
- the maximum mean peak period : 9 sec.
- the highest peak period measured : 10 to 11 sec.

As in the wave flume and wave tank studies performed by the LCHF at Maisons-Alfort, wave-trains are applied, the 9 sec. period is used finally for stability research and the 10 to 11 sec. period for research of phenomena during overtopping at maximum uprush.

4.3. Summary of design wave height conditions

The design wave height conditions, amended on the basis of the '78 - '80 wave records, are summarized in following table (fig. 7). By defining these conditions, the risk of exceedance of the design wave height has been fixed at 10 % in the design life or exposure time. For short wave risks (max. 3 to 5 years of exposure time) an average yearly frequency of 10 % has been taken. This results in the adopted design wave heights and risks levels reported in the table.

	Workharbour Northern breakwater	Southern East breakwater			LNG - breakwater	Outer harbour breakwater		
		East	North	West		SW	NW/NE	Mouth
Design wave height H_s (m)	4.00	3.00	5.75	5.75	4.90	5 à 6	6.10	6.10
Design sea level (m)	Z + 5.20	Z + 6.85	Z + 6.00	Z + 6.00	Z + 6.00	Z + 6.85	Z + 6.85	Z + 6.85
Accepted damage (%)	-	0.5	0.5	5-10	0.5	0.5	0.5	0.5
Probability of exceedance (%)	40	10	40	25	40	10	10	10
Lifetime or exposure time (year)	5	50	5	3	5	50	50	50

Chart datum Z = mean low low water spring + 0.08

Fig. 7 : Table : Summary of design wave height conditions

5. DEVELOPMENT OF DESIGN

5.1. For the LNG-breakwater and outer harbour breakwaters two main alternatives have been contemplated in the project definition and pre-project design stage :

- Rubble mound breakwaters
  designed according to different construction methods e.g.
  . mounds worked up over the constructed core by truck dumpers in combination with or without hydraulic dumped sand-fill. (fig. 8 and 9)
  . mounds with rubble cores and armour revetments worked up by purpose-built barges.

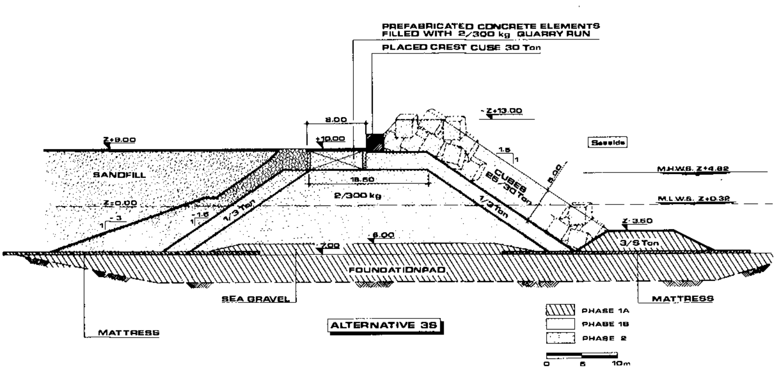

Fig. 8 : Development of design : rubble-mound breakwater alternative 3 b - core built by dumping over the head

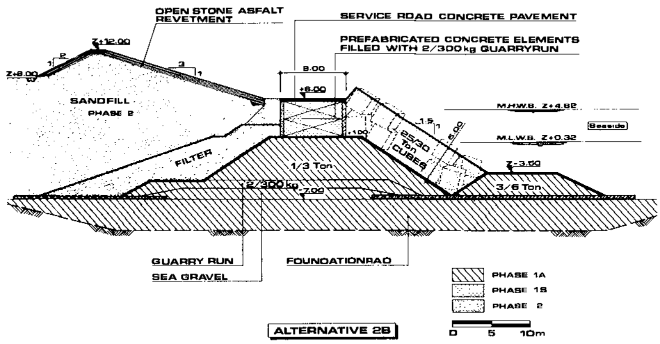

Fig. 9 : Development of design : rubble-mound breakwater alternative 2b - core built by dumping barges

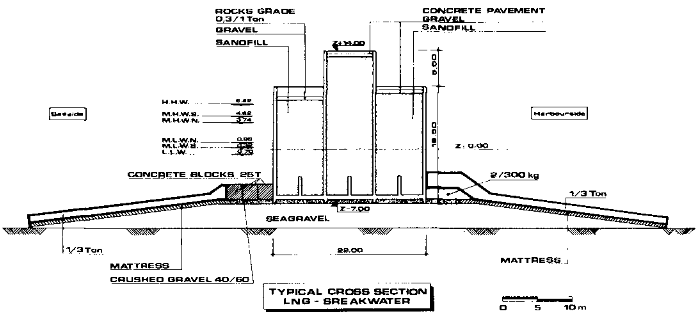

Fig. 10 : Development of design : caisson type breakwater (typical section at LNG-breakwater site)

Caisson breakwaters
prefabricated and floating caissons sunk on a prepaired foundation bed. Fig. 10 shows the conceptual design for the LNG-breakwaters.

After balancing the criteria of costs, risks, technical equivalence and construction aspects the hydraulic sandfill independent rubble breakwater type has been selected (3b).

The latter being amended for the southern west breakwater. As these sections are designed finally with an inner sand-beach, constructed simultaneously with the breakwater core.

5.2. The caisson type breakwater was judged to be unfeasible for the outer harbour breakwaters due to the unadequate soil conditions.

This initial conclusion should be amended. Indeed, the caisson type breakwater should be feasible as for almost all breakwater sections, now under construction, the detailed design specifies a foundation pad realized by a soil improvement by replacement of the soft layers with coarse dumped seasand.

As a matter of fact by the conceptual design of the breakwaters, in 1977/78, the seaworthy cutter-suction dredgers needed for such a foundation concept in unprotected waters, were not available. It was only later, during the construction of the northern service port breakwater and the LNG-terminal peninsula, that this soil improvement technique was successfully developped.

However the introduction and development of the soil improvement technique by replacement in open sea with dumped sand highlights once again the flexibility proper to the rubble mound design concept !

Indeed the soil amelioration technique has been assessed by evaluating the alternative of a mound laid directly on the existing seabed, where the overall stability is given by equilibrium embankments at both sides. Even in the instances of allmost equal costs, the soil improvement profile still offers a more controlled construction and a foundation bed with a higher quality level.

However, when choosing between rubble mound and caisson type breakwaters the other criteria are still valid.

- flexibility : the rubble mound offers more possibilities not only through adaption to local soil conditions, but also to the specific hydraulic conditions at the site (currents, waves, sedimentation, erosion, etc...).
- costs : compared with the rubble mound, the technical equivalent caisson type costs ca. 33 % more.
- construction risks : the construction problems on a site as Zeebrugge, exposed to waves, currents and siltation are of paramount importance by caisson type breakwaters.

6. FINAL DESIGN

The final design on the retained concept, the 3b-alternative, was finished in June '81. It consists mainly of (fig. 11) :

- *A foundation pad*

  A soil improvement by replacement with dumped sand is executed for all outer harbour breakwater sections. Seaworthy self propelled cutter suction dredgers with floating discharge hose remove the soft soil layers. The dumping of sea-sand is executed by trailing suction hopper dredgers. In general no extra densification has been revealed necessary after systematic controlling by static cone penetration tests of the performed works.

- *Bottom protections*

  Simultaneously with the sanddumping operations the sandlayers are covered by sea gravel layers of 0.60 m to 1.00 m thick placed by trailing suction hopper dredgers or splitbarges. Only a fortnight differ the sanddumping and sea-gravel carpet operations to avoid and excess of erosion of sand.

  Fascine mattresses, ballasted from 0.6 t/m^2 up to 1 t/m^2 by quarry run 2/80 kg and 80/300 kg grade by deck shovel barges, are covering the previous laid gravel layers.

  Where possible a 1.5 m thick core gravellayer is placed between the fascine mattresses.

  Gravel layers between fascine mattresses are covered by quarry rock 2/300 kg grade filterlayer.

  Erosion of sand and gravel between operations is detected by systematic surveys and echo-soundings. When necessary the eroded sand and gravel layers are refilled.

  To avoid an excess of materiallosses in the breakwater alignment these foundation works with bottom protection works have to be planned and executed well in advance of the core construction.

  A toe protection

  Provided by a large ca. 35.00 m wide wave breaking carpet on level Z-7.00 with fascine mattresses and quarry stone embankments of grade 3-6 ton up to a maximum level of Z-2,50 m with a gentle slope facing the seaside. The seaward 3/6 - ton and harbour side 1/3 - ton grade embankments are placed by dumpbarges or deck shovel barges.

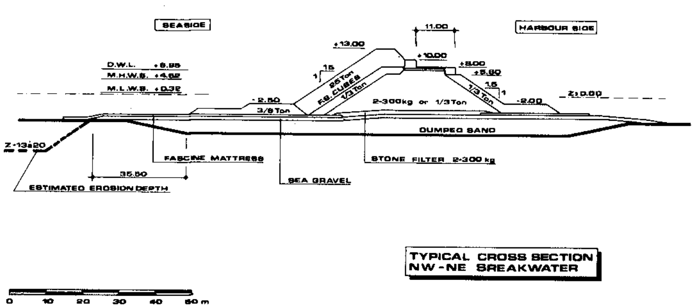

Fig. 11 : Typical cross-section NW-NE breakwater

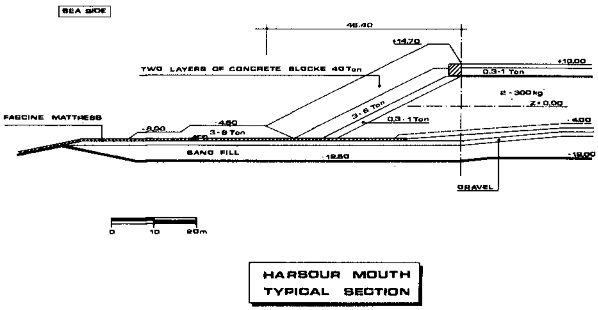

Fig. 12 : Typical cross-section (axis) at harbour mouth

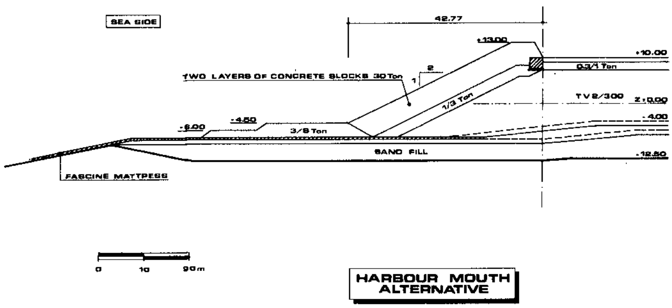

Fig. 13 : Typical cross-section (axis) - Design alternative harbour mouth (under construction)

- *A breakwater core*

  In quarry run 2/300 kg or 1/3 ton quarry rock with filterlayers 1/3 - ton grade. The armour revetment is realized by concrete cubes of 25 ton / 2,4 t/m^3 density on a slope of 1:1,5 in the running profile. The head of the breakwaters (figs. 12, 13) at the harbour mouth dips 1:2 with concrete cube units of 40 T and 2,4 t/m^3 on a 3/6 ton filter layer. The worklevel for the 40 ton grade truck dumpers, crawler crane, hydraulic cranes and bulldozers, working up the material, is Z + 6.80 m. This level guaranties in average a workability of minimum 90 % of the time.

- *A crest levelled to ca. Z + 13.00*

  As in the future the adjacent harbour area will be equipped with tidal basins and storage areas, no major overtopping will be allowed below the design wave conditions. The crest is finished by placing 30 t crest cubes outward and inward and a 1.00 m thick concrete pavement.

The southern west breakwater design differs slightly from the typical outer harbour design (fig. 14). The hydraulic sandfill of the protecting inner beach will progress simultaneously with the mound construction. The inner beach provides protection of the 1:1,5 innerslope in 1/3 - ton grade quarry rock against NW and N oblique wave attack as construction is running.

7. DESIGN FEATURES

7.1. Dimensions of breakwater

The width of the breakwater core is defined by following considerations (fig. 15):

- The structure is designed in such a way that the construction from core to finished crest goes continuous. The worklength e.g. the distance from the construction front to the finished crest is at maximum 200 to 300 m for the SW breakwater and 125 m up to 200 m for the NW/NE - sections.

- The serviceroad on level Z + 8.00 : min. 9.00 m wide, to assure maintenance works.

- The temporary workroads on levels Z + 6.80 to Z + 8.00 are 10.00 m wide : dubble track road for 40 - ton rockdumpers.

- The level of the construction front is defined on Z + 6.80 by 13.70 m wide (including the top of the 1/3 ton grade filterlayers. It assures a workability of min. 80 % and the manoevrability of crawler crane and one track for dumpers.

7.2. Crest of breakwater

To obtain a continuous progress in construction a parapet wall has been avoided (figs. 11, 14, 15). This has showed to be profitable both for the stability of the armour by better wave energy absorption and limited uprush. Force measurements in the wave flume concrete pavement with rather limited thickness, showed clairly that the overpressures in the core are limited and there is no risk for a resulting uplift force.

7.3. Wave breaking carpet

The sedimentological study conducted both on physical and mathematical models ascertain the possibility of huge erosion pits in front of the SW/NW - breakwater section (from Z - 13 up to Z - 20.00). The NW - breakwater (up to Z - 6.00) and the W harbour head section (Z - 14.00).

The aim of the designed carpet is to limit the wave action on the armour to the design wave height defined by the existing seabottom on Z - 7.00. Flume and wave tanks tests have demonstrated that a carpet of 35.00 m out of the 3/6 - ton grade toe protection on level Z - 7.00 will provide this limited wave action independent from the final erosion depth (Z - 13.00 and Z - 20.00 are tested). However the final erosion depth plays a significant part in the wave stability of the ballast of the fascine mattress. For the applied design conditions H_S = 6.10, T = 9 to 11 sec : an adequat ballast is found by :

Z - 13.00 : 1 T/m² 2/80 and 80/300 kg - grade
+ 0.75 T/m² 0.3 to 1 ton grade

Z - 20.00 : 1 T/m² 2/80 and 80/300 grade
+ 2,1 T/m² 1/3 ton grade

The performance of the 25 T/2.4 T/m³ - armour has been the same in both test series.

It should be noted that the judgment on damage of the mattresses ballast has been rather severe as the function of the carpet is essential to the breakwater safety and survey of damage detection and maintenance will be very difficult.

7.4. Armour units

For the main breakwater the groved concrete cubic block has been choosen as armour unit. However a rather interesting cost optimization was possible. The fig. 16 shows all different cubic and semi-cubic bloc types in use by the construction. All these cube types were cast in only three different forms (8,7; 10,42 and 12,5 m³) as for the harbour mouth the alternative 30,6 Ton will be choosen.

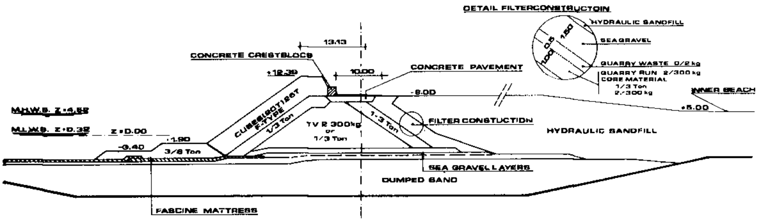

Fig. 14 : Typical cross-section - Design alternative at harbour mouth (Chainagepoints P1795 - P2150)

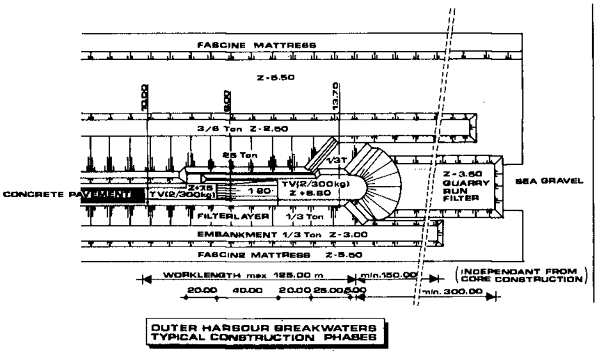

Fig. 15 : Breakwater construction in progress Worklength - typical construction phases

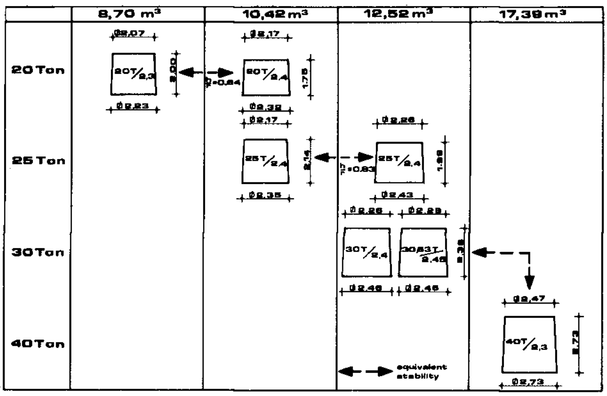

Fig. 16 : Summary of cubic or flat semi-cubic armour bloss applied or considered for the Zeebrugge breakwaters

Flume tests and wave tanks revealed that 25 T/2,3 t/m³ - cubes were necessary at the SW/NW bend section. However cost calculations show that armour layers of 25 T/2,3 t/m³, 25 T/2,4 t/m³ and 30 T/2,4 t/m³ differ only in marginal terms of max. 3% . In counterpart the safety margin for the 30 T layer yields up to 13 % of the significant design wave height.

By the same optimization process so called flat semi-cubes (F.S. - cube) were tested in the flume and wave tank. The maximum "flatness index" has been $h.r^{-1} \geqslant 0.B$ (h = height, r = bottom measure). The tests revealed no significant difference in stability between the cubic unit and the F.S. - cubic unit of equal weight. Costwise the difference is more substantial : 5 to 6 % of the armour cost in favour of the F.S. - cubes.

The choice of the groved cube for the armour is placed in evidence by comparing with a typical dolosse breakwater. The conceptual design is showed on fig. 17. By this concept two sizes of dolos-armour are compared with the cubes.

Type I	:	12 T / 5,22 m³	:	slight or no damage by design conditions but rocking could be considerable ($K_D \geqslant 15$).
Type II	:	19,55 T / 8,5 m³	:	beginning of movement (rocking) by design conditions $K_D = 10$. By 1,35 x H_{SO},slight damage can occur ($K_D > 23$).

Costwise the dolos armour revealed to be uneconomical compared with the cubes-armour-elements (tens of percent higher).

7.5. Core material

The core of the breakwater consists of rock material grade 2/300 kg or 1/3 ton. The stability of these quarry grade were tested with the construction phase of the breakwaters (fig. 15) both stationnary (static) and in progress (dynamic). The main results are (fig. 18) :

1) Construction in progress limits
   - upper threshold value for use of quarry rock grade 2/300 kg : $H_S = 1.00$ m
   - for 1.00 m > H_S > 2.00 to 2.50 m quarry rock grade 1/3 ton is to be used
   - by H_S > 2.50 m construction will be stopped and a provisional head with quarry rock grade 3-6 ton is to be constructed

2) Construction stopped at night and weekends

   by exceeding 0.80 m to 2.00 m significant wave height, a 1/3 ton provisional breakwater head is to be constructed. On the basis of this information the principal and contractor decided to stop every weekend from September to April with a provisional breakwater head in 1-3 ton grade quarry stone.

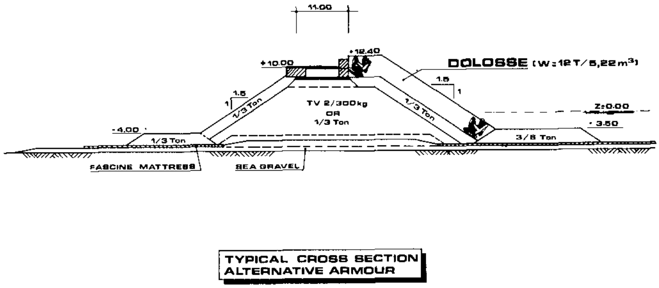

Fig. 17 : Typical cross-section NW/NE-breakwater with dolosse armour revetment

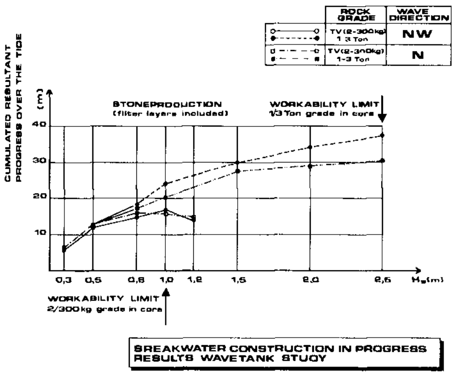

Fig. 18 : Breakwater construction in progress
Limits of 2/300 kg and 1/3 ton rubble grade

3) Detailed information has been becoming available concerning allowed protrusions of breakwater core versus armour revetment and damage to be accepted by special construction stages of breakwater e.g. by the inner slopes of the southern east peripherical breakwaters. This kind of study has been executed also for the west and east core of the LNG-terminal peninsula and the LNG'86 - breakwater. The results of all these studies differ only slightly despite the different locations and wave directions. A predominant factor is the average stone production into the core. In Zeebrugge the design production is 250 T/Hr.

7.6. Meteorological forecasting services

In order to maximise the exploitation of the information achieved by the flume tests, the principal of the works, the Ministry of Public Works - Coastal Service at Ostend - in cooperation with the Meteo Wing of the Belgian Air Force, has set up a special on site Meteorological Forecasting Service.

Twice a day a weather forecast is released with a correlated wave forecast. Directional windforce/wave-height correlation graphs have been established by the consultant on the basis of wave and wind data measured in prototype.

On this 24 hour-forecast the contractor's site managers and principal's site surveyors can judge work limitations in forthcoming shifts on breakwater constructions.

The meteo service is also equiped with an on-line wave analysis computer system, hard-ware programmed. This system is analysing on site wave data received from the wave rider buoy posted at the construction front. This is supplying sufficient information to forecasters and construction site managers about wave growth and height correlated to windforce and direction.

Besides the construction front wave-rider four to five other buoys are operational. The analysed wave data can become available if necessary within one week. In this way any damage can be correlated to the observed wave action.

These measures are only a part of a comprehensively established survey programme with analysis of data and evaluation of measures piloting the construction works.

7.7. Placement pattern of armour cubes

Once more this pattern is revealed of paramount importance. Two kinds of practical patterns are studied thoroughly. The first has been used on the LNG-terminal breakwaters, the second at the LNG '86 breakwater and outer harbour sections. The two differ only by the number of required positions of the crawler crane. By testing both patterns the first revealed a rather high risk to jam up by reaching the design conditions. The second pattern has better blocking properties.

8. STATE OF CONSTRUCTION BEGIN NOVEMBER '82

The present status of the Zeebrugge outer port extension scheme is summarized (fig. 2) :

- Project-planning and design : finalised July '81.
- Southern East-peninsula : commissioned to LNG - terminal operator at 1 September '81.
- Northern - East - breakwater : foundation works and gravellayers finished, fascine mattresses and quarry-run filter layers construction is still going but almost finished.
- Western - breakwater :
  - breakwater construction : chainagepoint P 1750 is reached by the core up to Z + 6.80.
  - the foundation works covered by gravellayers is finished to P 3900. The fascine mattresses with quarry-run gravellayer are finished up to ca. P 2200.

More than 50 % of the total deployed length of breakwater works is finished at present.

9. CONCLUSIONS

1. By developping the design, the rubble mound breakwater has been judged to be the only viable alternative taking into account costs, technical risks, construction problems and flexibility by changing environmental conditions.

2. The wave breaking carpet has shown to be an adequate solution in order to limit wave action on the armour.

3. A huge concrete parapet wall can be avoided and is not necessary for an economical design of breakwaters in the Zeebrugge conditions.

4. It is of paramount importance to check and analyse both the geotechnical stability of the rubble mound on the foundation bed and the internal stability of the mound under wave action. In this connection outer and even inner slopes of 1:1,5 show marginal stability with an inadequate safety facts in some specific cases.

5. Concrete cubes as armour units have been revealed for this case to be more economical and justified. A security margin of a 13 % can be obtained and approximately no costs when upgrading up to 30 T the in the flume and wave tank tests found cube of 25 T. Semi cubes flattened to index $h/r \geqslant 0.80$ reveals to be 5 to 6 % more economical than the cube of the same weight. By tests no significant stability difference was observed.

6. A proper planned placement pattern is demonstrated, once again, necessary.

7. A meteorological forecasting service equipped with wave rider buoys gives adequate information in order to construct in a controlled manner a rubble mound structure and to avoid excessive damage during construction hardly to repair.

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