ABSTRACT

RUBBLE MOUND BREAKWATER DESIGN AND MODEL TESTING OF WATER INTAKE FOR SOHAR INDUSTRIAL AREA

by
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Key Words: Breakwater, rock, core-loc, oblique incidence,berm,stability,overtopping

The government of the Sultanate of Oman decided in 1999-2000 to develop the Sohar Industrial Area (SIA) in the Wilayat of Liwa. by a Royal Decree, a land area of 132 km^2 located about 20 km form Sohar town has been designated as the Sohar Industrial Port Area for the development of the Sohar port and energy based industries. The master plan for this development includes the Sohar Industrial Port and large scale industries such as refinery, power and desalination, fertiliser, methanol, polypropylene, polyethylene and aluminium smelter. The required water for these industries were planned to be obtained from sea water by an intake structure and the used water is planned to be discharged to the sea by an outfall structure. The Ministry of National Economy (MNE), owner of the works, appointed Fichtner Consulting Engineers of Germany (Fichtner) as the Project Management Consultant. Based on the basic design prepared by Fichtner, MNE awarded in 2002 the (EPC) contract to BCC-STFA Consortium, which is formed between Bahwan Contracting Company Oman (BCC) and Sezai Turkes & Feyzi Akkaya Construction Company, Turkey (STFA).

Originally, the project consists of a seawater intake designed for a maximum flow rate of 334,000 m^3/h and a return water outfall designed for a maximum flow rate of 325,000 m^3/h. Approximately 9,000 m^3/h is expected to be potable water. During the course of the design, it is recognised that the required intake water should be 800,000 m^3/h. The original intake structure layout is revised and a new lay-out with a dog-leg extension shown in Fig(1) is decided.

The intake breakwaters cross-sections and layout were initially designed by the BCC-STFA Consortium. The layout of the breakwaters were tested by numerical models by HRWallingford of UK(Ref.1). The basic parameters that were investigated during this investigation were the wave agitation in the Intake Channel, the currents in the Channel and the expected sedimentation in the Channel. During this investigation, the final layout of the Intake breakwaters were developed. This is the layout shown in Fig(1).

Hydraulic model tests were also made for investigating the wave agitation inside the intake channel and also for investigating the stability of the breakwaters at the DLH Reseach Laboratory in Ankara(Ref.2). HRWallingford of UK were the technical advisor for these hydraulic model tests.

The extreme wave conditions expected to occur is investigated by HRWallingford of UK and the extreme significant wave heights at different water depths along the intake breakwaters were given in Ref.(1). These wave heights were used during the preliminary design of the breakwaters.

The cross-sections of the Intake Breakwaters that is developed by BCC-STFA Consortium and hydraulically tested by model tests are shown in Figure (2).

At the main breakwater's most exposed part where the wave attack comes perpendicular to the breakwater $3m^3$ core-loc units were used. At the rest of the breakwater, quarry stone armour is used. The armour stone weight W is calculated using the Eqn(1) below (Ref.3) which includes the effect of oblique wave attack ($\beta > 30^0$) . The breakwater crest elevation is also designed using the wave run-up equation given in Eqn.(2) below (Ref.2) with an allowable wave overtopping criteria of 1 l/sec/m corresponding to R_p/50year return period wave.

$$W = \frac{\gamma_r H_s^{\ 3}}{K_D (S_r 1)^3 Cot\alpha} Cos^2(\beta - 30) \text{------------------------(1)}$$

$$\xi = \frac{\tan\alpha}{\sqrt{H/L_0}}$$

$$R_u = \frac{0{,}8x\xi x H_s x Cos(\beta - 15^0)}{1 + 0{,}5\xi} \text{------------------------------(2)}$$

where, R_u=Wave Run-Up corresponding to significant wave height
H_s=Significant Wave Height
$L_0=1{,}56T^2$
T=Significant Wave Period
α=Breakwater angle with the horizontal
β= The angle between the wave orthogonal and the perpendicular to the breakwater axis

The designed breakwaters are constructed at a 1/40 scale hydraulic model using irregular waves. The core-loc sections were constructed as dictated and witnessed by Sogreah experts and the tests were witnessed by HRWallingford experts. The following topics were investigated;

1- The wave penetration into the intake and optimization of the layout found at the numerical model
2- The stability of the breakwater armouring
3- The overtopping from the breakwaters
4- The transient water level change in the pump station and along the channel once the pumps started to pump
5- The current pattern inside the intake

All the preliminary design results were found satisfactory at the hydraulic modeling except the overtopping at the dog-leg were found excessive. The crest elevation of the dog-leg part of the breakwater is same with the deeper stone section of the breakwater which is exposed to oblique wave attack. Altough the crest elevation of the deeper region which has a breakwater slope of 2/3 were satisfactory for the overtopping criteria, the dog-leg part which has a breakwater slope of 1/3 and exposed to perpendicular wave attack did not satisfy the overtopping criteria. The reason is the effect of different angle of wave attack to these two parts of the breakwater. There is also a wave concentration effect at the turn from the deeper portion to the dog-leg portion due to reflected waves. This problem is solved by constructing a permeable berm in front of the dog-leg portion with the berm elevation at HAT level and with a berm width of 10m. This change solved the overtopping problem at the dog-leg portion.

The new research (Ref.4) demonstrated a stone layer thinner than the layer thickness recommended by the SPM(1984) (Ref.5). This was a question between the check engineer and the designer and it is decided to measure the layer thickness at Site. This has been done at a rock section constructed at Site and it is found that the thickness values came out to be close to SPM(1984) recommendations.

The sea bottom is silty sand. There were a discussion if this material may show a failure during construction and if there may be long term settlement and if there is a need for dredging of the sea bottom before construction. No dredging is made and the construction is conducted using land based equipment. Settlement monitoring plates are placed at every 50m at +4mCD elevation and the settlements were monitored. It is found that the settlements are less than 1cm after construction and no long term settlements were detected.

The paper describes all the above findings and gives the procedures conducted at the design and construction phases. The west breakwater construction is compleded and the core loc sections are under construction by the end of 2004. The construction will be finished by Sep 2005.

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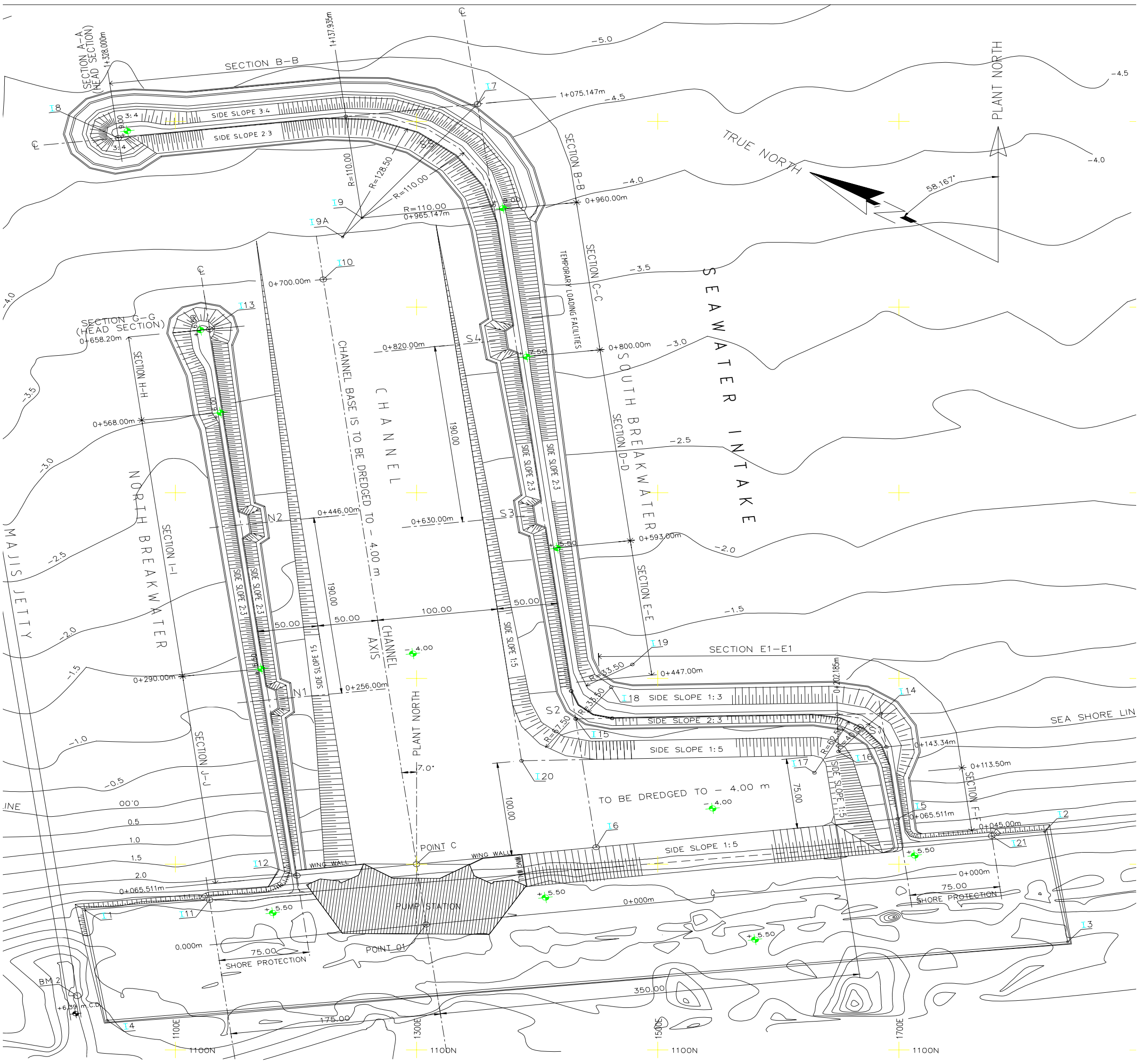
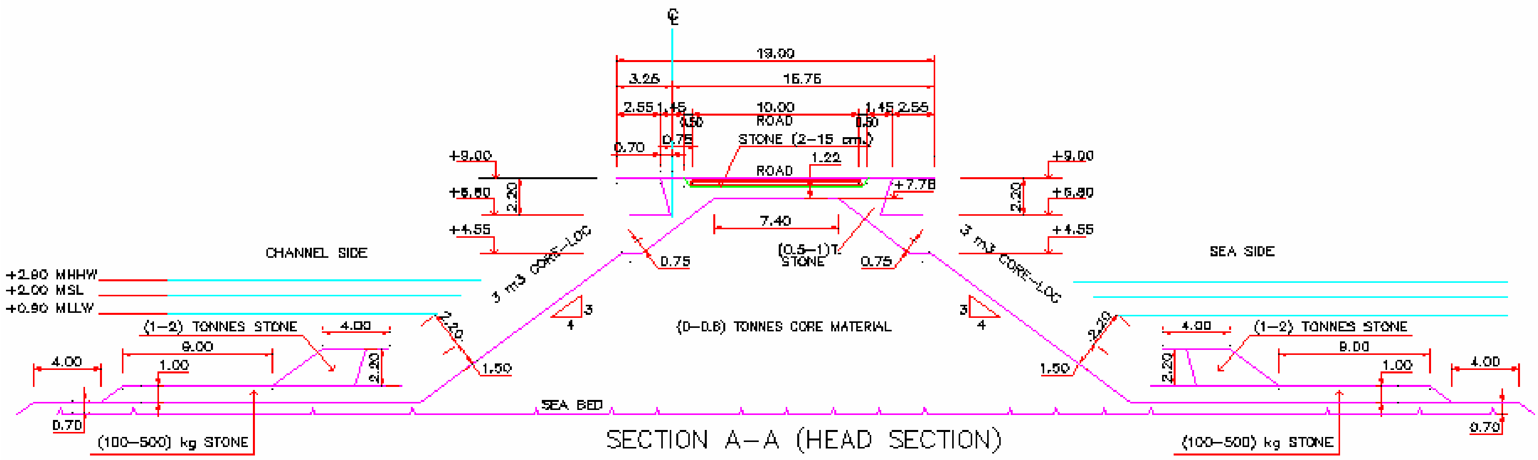
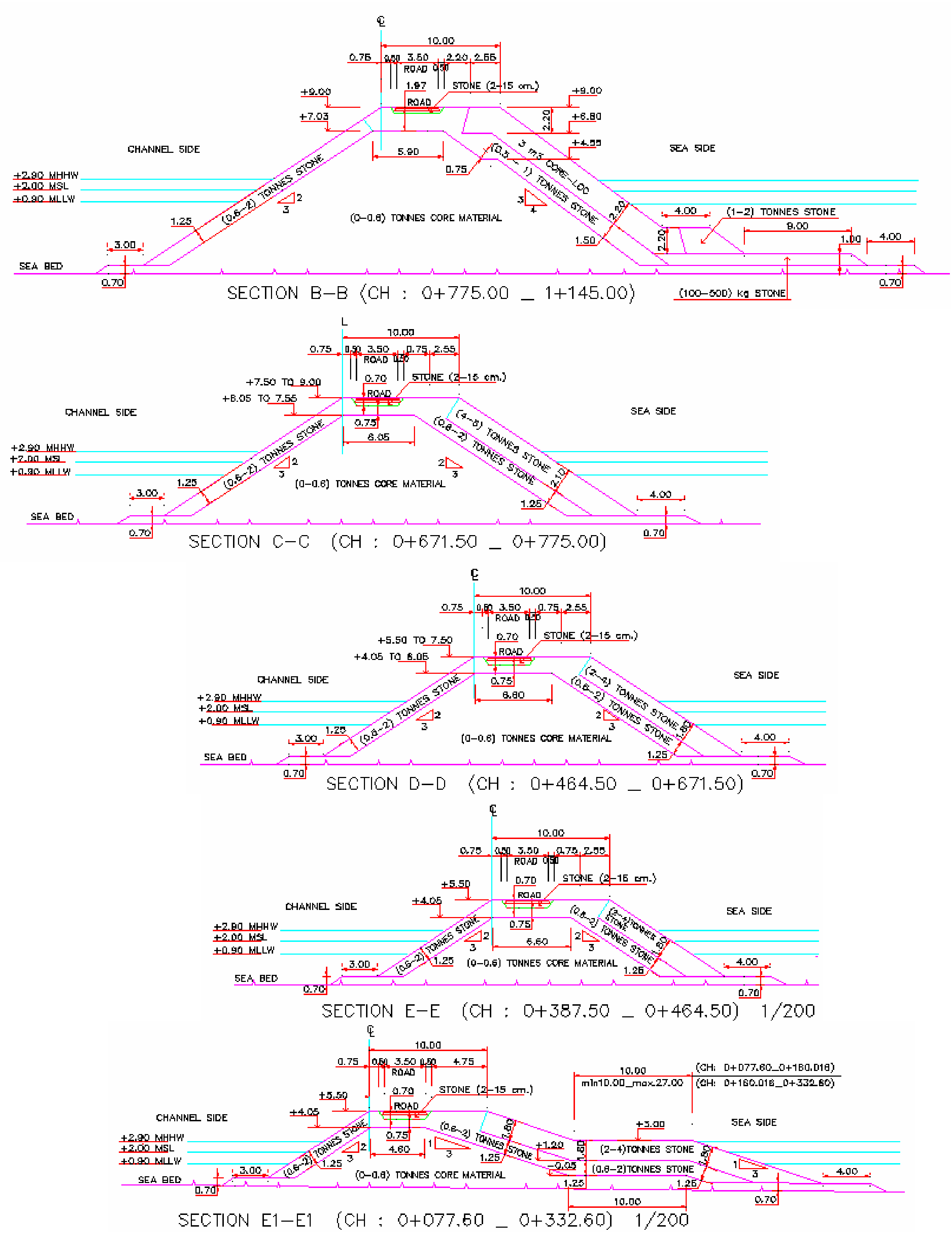


Fig(1)- Layout of the Intake breakwaters



Fig(2)- Main Breakwater Head Section



Fig(3)- Main Breakwater Trunk Sections