

DISSIPATION OF WAVE ENERGY IN A SEAWATER OUTFALL CHANNEL

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ABSTRACT

This paper describes the investigation of means of reducing wave action reaching the shoreward end of a power station cooling water outfall channel without resulting in significant head loss to the outflowing water. A variety of conceptual methods of reducing wave action in the outfall channel was examined. A physical model of the outfall was constructed. It was found that a rubble mound wave energy dissipator located in the outfall channel dramatically reduced wave action at the discharge seal pit.

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1. INTRODUCTION

South Africa's first nuclear power station is located on the south west coast and will use seawater as a coolant. Up to 86 cumecs of seawater will be abstracted via an intake basin comprising two rubble mound breakwaters. The intake basin has an area of about 4 ha in which suspended sediments drawn into the basin through its 900 m offshore, 9 m deep entrance will settle before removal by maintenance dredging.

The warm return water will be discharged through a shallow outfall channel into the surf zone south of the intake basin.

The cooling water system is designed to operate as a syphon to minimise pumping costs. At the outfall, four low level 3 m diameter pipes discharge into a seal pit, over a weir and into a tapered concrete channel through the beach. The weir crest is 40 m long with a level of 0,0 m G.M.S.L. (Geodetic Mean Sea Level). The 150 m long outfall channel is curved in plan and tapers to 20 m at the seaward end. The level of the concrete floor is - 2,0 m. The capping beams to the concrete sheet piled walls have a top level of + 2,2 m. Mean sea level is at + 0,15 m G.M.S.L., Mean High Water Springs at 0,86 m and Mean Low Water Springs at - 0,56 m.

The outfall structure was designed to resist full wave action, but after construction of the outfall, the Electricity Supply Commission (ESCOM) requested the designers to investigate means of reducing wave action in the outfall channel. This decision was taken for several reasons but generally it was ESCOM's intention to research any improvements that might add to operating efficiency and safety of the station and thereby maintain the very high standard set by ESCOM for this project.

This paper describes the investigation of means of reducing wave action reaching the shoreward end of the outfall channel without resulting in significant head loss to the outflowing cooling water.

2. PHYSICAL MODEL

A variety of conceptual methods of reducing wave action in the outfall channel was examined. It was decided to construct a physical model of the outfall and foreshore in the vicinity of the outfall to examine wave action and to develop and test methods by which wave action could be reduced.

A fixed bed, 1:30 scale Froude model was built in the hydraulics laboratory of the University of Stellenbosch.

A general view of the modelled outfall and surrounding shoreline is shown on Photograph 1.

The seabed over the prototype distance of 150 m from the end of the channel was constructed at a level equivalent to - 2,0 m GMSL. Further offshore the water depth was gradually increased to provide the depth required for the mechanical generator.

PHOTOGRAPHS



1: GENERAL VIEW OF MODEL



2: CLOSE-UP VIEW OF MODEL OUTFALL STRUCTURE

In the prototype, jet action of the water discharged from the outfall is likely to scour the sandy bed in front of the channel exit, but further seaward, an offshore bar is likely to form. The sea bed level at the offshore bar is unlikely to be lower than the chosen fixed bed level in the model ($\sim 2,0$ m) and hence the resulting depth limited waves entering the channel in the model were considered to be a reasonable simulation of prototype conditions.

Regular waves generated in the model were controlled to approach the breaking limit close to the channel entrance in order to maximise the wave energy entering the channel. Wave periods between 6 and 16 seconds were used in the model. It was found that 10 second waves produced effects as severe as any other periods in the model.

This period, which is typical of a large proportion of waves recorded at the site was therefore used in the majority of tests.

Photograph 2 shows the model of the outfall structure with the modelled pipe transitions between the outfall structure and straight lengths of 3 m diameter (prototype) pipes connecting the structure to a reservoir. Water was pumped from the model into the reservoir (via a V-notch weir) to simulate the cooling water flow through the outfall. By means of varying the recirculating flow and capping off the pipes leading to the model C.W. structure, variable flows through any combination of the four C.W. ducts could be simulated. Model tests included no-flow, 40 cumec and 80 cumec flow conditions. Forty cumec flow conditions simulated the closure of either Reactor 1 or Reactor 2 by directing the flow through the northern (left hand side on photograph 2) or the southern pair of ducts respectively. The model C.W. outfall structure had removable perspex slabs to represent the temporary stoplogs. Various still water levels ranging from $-1,5$ m to $+2,1$ m were used in the model. Most of the tests were carried out with the S.W.L. at $+1,6$ m as this was considered to be a realistic normal upper design condition and corresponded approximately to an event with a recurrence interval of 5 years. An extreme still water level of $+2,1$ m was also tested. It had been calculated that the recurrence interval of this event, based on an extrapolation of the available records of water level fluctuations due to tide, surge plus long period (50 s to 300 s) wave action, will be in excess of 50 years. (This is the minimum recurrence interval resulting from the most pessimistic of the data using a Weibull population distribution.)

In order to record wave induced pressure surges in the pipes, manometer tubes were fitted to each of the four perspex pipe transitions entering the rear of the seal pit. In addition, pressure transducers were fitted diametrically opposite the manometers on the northern and southern ducts and linked to an analogue recorder.

Wave heights at the closed stoplogs were also measured.

The wave direction used in the model was $247,5$ degrees (true bearing of wave orthogonal). This corresponded to the mode of the winter wave directional spectrum, ($1,25$ degrees north of the mode of the summer spectrum), was normal to the seabed contours between -5 m

and - 10 m and was 9 degrees north of the C.W. channel centreline at its seaward end.

3. MODEL TESTS OF THE UNMODIFIED C.W. OUTFALL

3.1 No-Flow Conditions

A full range of tests was carried out to study the behaviour of the C.W. outfall without modification when subjected to various discharge flow rates and wave conditions.

Wave induced pressure surges in the ducts under no-flow and various flow conditions were measured and in addition, observations were made of the height to which the waves rose against the closed stoplogs. General wave action in the channel and over-topping of the channel walls was noted.

Model tests were carried out over a range of conditions including various wave periods, water depths, stoplog closure and flow rate conditions.

Photograph 3 shows the general pattern of wave action in the channel under conditions of no-flow. The waves which enter the seaward end of the channel are generally compressed against the southern wall of the outfall channel due to the curvature of the channel, resulting in a variation of wave height across the width of the outfall structure, with larger waves at the southern end.

Waves reflected off the vertical face of the stoplogs and walls of the structure were observed travelling seawards, resulting in increased wave height where the waves interact.

For still water levels higher than about + 1,6 m, an increasing proportion of wave crest overtopped the channel walls and hence wave action due to standing waves caused by the interaction of incident and reflected waves was reduced.

Photograph 4 shows a test carried out with the waterlevel at + 1,6 m and wave period of 10 seconds. During this test the wave height at the stoplogs on the most southern side of the structure was such that a spout of water was projected through the gap between the overhead bridge (+ 5,0 m) and the stoplogs. These test observations appeared to correspond closely with the observations on site and corresponded to the most severe conditions observed in the model.

Pressure surges in the ducts for the no-flow conditions and stoplogs removed were also recorded.

3.2 Half and Full Flow Conditions

Flow of cooling water in the channel resulted in reduced wave action in the channel and hence reduced pressure surges in the



3: TYPICAL WAVE ACTION IN CHANNEL
(SWL +1,6 m GMSL)



4: WAVE ACTION ON STOPLOGS-SOUTH BAY
(SWL +1,6 m GMSL)

ducts, as compared with the no-flow condition. The degree of reduction depends on flow velocity in the channel and therefore is greatest for large flow rates and shallow depths, (i.e. at low tide levels).

As a result of the variation of wave height across the outfall structure, (increasing from north to south i.e. left to right in photograph 4), a corresponding variation of pressure surges was recorded in the ducts.

Surges in the most southern duct, for the case of an extreme S.W.L. of + 2,1 m and 40 cumecs flow in the northern ducts was found to be 1,5 m.

Tests carried out under 40 cumec flow showed that wave action in front of the no-flow half of the structure and pressure surges in the corresponding no-flow ducts with the stoplogs removed were slightly less severe than for the complete shut-down condition.

4. POSSIBLE MEANS OF REDUCING WAVE PENETRATION

To reduce wave action at the outfall structure, a number of structural alterations or additions were considered. These fall into two basic categories, those than sought to prevent waves reaching the seaward end of the outfall and those that modified wave action within the outfall channel.

From preliminary evaluations of the various schemes that were considered, the following conclusions were reached. Some qualitative model testing was used to support conclusions drawn.

Offshore Structures

In this category an offshore rubble mound breakwater constructed either linked to or isolated from the southern breakwater of the intake basin was considered. Its position, orientation and length would have needed careful study but in any case it would have been very expensive. However, it would also have had potential disadvantages in its deflections of the discharge plume and the changes it would have imposed on siltation which may have been encouraged in its shadow.

Such a breakwater for a somewhat lower cost could have been attached to the north side of the channel thus retaining the continuity of the discharge jet, although the jet would have been deflected to the south with a potential loss of cooling efficiency. Scour at the base of the structure would have been a problem.

To retain the jet discharge direction two rubble mound breakwater arms could have been built on either side of the outfall, with spending beaches each side of the present outfall. This solution would also have been expensive, but it was nevertheless tried in the model. The brief trial indicated that very little protection

was afforded and that the outflow currents reduced the spending effects.

Despite the advantage that offshore structures could have been constructed during operation of the outfall, it was concluded that offshore structures would be investigated further only if high cost were justified and all other options were relatively unsuccessful. They were not therefore considered further and it was decided that solutions inshore of the sea end of the outfall should be pursued as more likely to meet the requirements.

Outfall Channel Modifications

- Moveable Devices

Moveable devices were considered as a possibility for insertion or operation during extreme wave conditions or under particular operating or shut down conditions. Such devices included for example gated structures, floating breakwaters or caissons but in all cases would have been attended by maintenance problems. Disadvantages would have varied with the particular arrangement and operating conditions but would have included problems of moving, maintenance, disruption of flow and flow back-up. It was concluded also that periodic floating-in or launching a structure would have been impracticable in this coastal environment. These solutions were therefore rejected.

- Reduction in channel entrance width

Reduction of the outfall end width appeared attractive but of course higher discharge velocities at low levels would have caused flow back-up and consequent scour problems would have resulted. Nevertheless a trial was made on the model which indicated that significant wave action still penetrated the outfall channel.

- Increased roughness and/or canalising flow

The addition of increased roughness to the sides and bottom of the outfall channel was mathematically investigated and found to have little effect. Even the use of a number of splitter walls with artificial roughness was found to offer only a modest reduction in wave height. It would, in any case, have been very difficult to construct such splitter walls.

One new wall down the centre of the outfall channel was able to concentrate the 40 cumec flow over half the channel to equal the velocities of 80 cumec over the whole channel but did not reduce wave penetration in the no-flow condition. This solution would also have been somewhat difficult structurally and expensive to build.

- Change in Channel plan geometry

More fundamental changes to the plan geometry of the outfall channel

were then considered. Resonator basins could have been used to damp out wave action travelling along the walls, particularly on the south side. Each resonator needed to be designed for a narrow band of wave period and a number would have been needed to cover sufficient of the wave spectrum.

Fairly fine tuning of the group would have been needed to optimise the result and hence they would have been less efficient when non-designed conditions occurred. Construction would have involved cutting down substantial lengths of the outfall piles and driving a longer indented length at high cost.

A simpler constructional approach would have been to build a combination of deflectors and wave breakers in the channel. A typical solution on these lines was modelled and showed that benefits might be obtained from the absorption but that the deflectors were less effective. Structural problems would have been similar to the central walls and probably expensive to overcome. An advance on the wave absorption theme could have been to cut off some of the side piling and produce a spending beach just outside the channel. The cut off level would have needed to be low, only the outer ends of waves would have been affected, and outflow would have been distorted.

To avoid expensive removal and reconstruction in the channel, isolated wave absorbing devices could have been placed in the channel, for example, shaped perforated blocks. These were tried in the model but the wave period was too long for their effect to be noticeable. These would, of course, have caused a restriction to outflow.

Many of the above solutions were found to have inherent cost or effectiveness disadvantages, however two promising solutions were a long slab spanning the entire width of the outfall at a relatively low level, and a rubble mound wave energy dissipator (WED) constructed inside the channel.

The first of these was sufficiently interesting to seek a degree of optimisation by model tests, from which it was concluded that, located near the downstream end of the channel and spanning 20 metres across it, an unbroken slab extending over about 18 metres length of the channel was needed. The level of the slab soffit had to be set at about 0,0 GMSL to prevent the passage of wave energy whilst allowing outflow of 80 cumecs beneath it. Large wave forces were involved, however, and the slab needed to be heavy. Its weight required separate support outside the sheet piling, which was not designed to support such loads.

There was sufficient head loss in the cooling water outflow across the slab (approximately 600 mm under the worst conditions). Wave resonance between the outfall structure and slab occurred under certain conditions which required the introduction into the channel of simple anti-resonance devices. The slab cut out wave overtopping at the inshore end of the channel and would have been virtually maintenance free. However, there could have been problems of

stability during construction and a lengthy model study would have been needed to optimise the design. The cost was estimated to be 50% more than the WED mound and therefore this alternative was carried no further.

The wave energy dissipator (WED) mound comprises a periphery of heavy precast concrete blocks in the centre of the outfall commencing at a point 25 m seaward of the crest of the seal pit weir and extending about half way down the channel. Inside the blocks, a rubble mound is constructed up to a peak level of + 3.0 m GMSL in the centre of the channel. The layout tested is shown in photographs 5 and 6.

During the course of testing the possibility of splitting the mound into two parts, one along each side wall of the channel, (for easier construction) was examined, but the result was far less effective hydraulically. Only the centre channel mound was therefore evaluated during the remaining model tests.

It quickly became apparent that the WED was not very sensitive to detailed adjustments in design, it was effective over a wide range of wave periods, and it eliminated resonance.

It achieved a significant reduction in wave action at the outfall structure under no-flow conditions and substantially reduced surge in the pipes when flow was present. It produced an insignificant loss in head in cooling water outflow.

The effectiveness of the WED in reducing wave penetration is due to three basic hydraulic phenomena:

- (i) As a wave travels up the outfall it moves between the channel walls and the sloping rock bank. The latter causes wave refraction, so bending the wave front at that end and allowing its energy to be partly spent in the top of the rubble mound.
- (ii) As the shoreward end of wave diffracts around the round head of the WED and the energy which remains is spread over a greater width of channel before reaching the outfall structure leading to a reduction in wave height.
- (iii) During the half flow condition, the WED concentrates almost all the flow down one side of the channel, which effectively results in the same wave energy exclusion by current in that side of the channel as in the full flow condition over the full channel. The wave energy reaching the operating side of the outfall is thus further reduced.

The WED had one potential problem, namely how to construct it taking account of wave action and whilst still permitting outflow up to 40 cumecs in the channel. Further consideration, however, led to the conclusion that suitable temporary works could be devised to make the scheme entirely practicable.



5: MODELLED WAVE ENERGY DISSIPATOR (SWL -0,5 m GMSL, flow 80 cumecs)



6: MODELLED WAVE ENERGY DISSIPATOR (SWL +1,0 m GMSL, flow 80 cumecs)

5. MODEL TESTING OF THE RUBBLE MOUND WAVE ENERGY DISSIPATOR

Various preliminary tests were carried out to finalise the general arrangements of the WED concept and these included tests on a series of tall blocks projecting above the water surface around the perimeter of the mound to 'deflect' the wave towards the mound, additional V-walls along the length of the mound and a pier connecting the outfall structure with the shoreward end of the mound. These were found to have nominal benefit with respect to reducing the wave activity at the structure in relation to the cost of providing such refinements. A definite reduction in wave energy dissipation was noted for a reduced mound length.

Model testing of the WED has shown that it is capable of dissipating a large proportion of the incoming wave energy. There was also a significant reduction in wave activity and pressure surges on the no flow side of the structure under half flow conditions.

For the + 1,6 m design water level the maximum wave height recorded at the stoplogs was found to be 1,2 m for the no flow condition with all stoplogs in place - photograph 7.

It was found necessary to have 'toe' blocks to the rubble mound in order to ensure the stability of the rock in the mound under extreme low tide conditions when the velocity in the channel resulting from an average discharge of 86 cumec was greatest. An extreme low water level test equivalent to a water level of - 1,5 m (recurrence interval in excess of 1000 years) showed flow conditions to be acceptable and that the toe blocks would be stable.

Tests at the extreme high still water level of + 2,1 m indicated that the surge in the southern duct would be 0,6 m for the 40 cumec flow condition. The WED rock mound was also noted to be stable for the + 2,1 m water level and no flow condition.

6. APPLICABILITY OF MODEL RESULTS TO PROTOTYPE

Comparison of the roughness in the model channel with that in the prototype indicated that the model was marginally rougher than the prototype. For the 80 cumec flow and still water level at + 1,6 m the additional friction in the model could be expressed as an additional 10 mm of prototype head backup in the outfall chamber. The additional friction would result in very slightly lower channel velocities and correspondingly less wave reduction in the model, leading to conservative observations of surge.

With the WED in place the relative roughness of prototype and model are comparable.

The use of regular waves of equal height in the model represents the fairly severe condition of the wave train of maximum wave heights compared with a normal wave spectrum which would include a spread of wave height and wave periods.



7: WAVE ACTION ON STOPLOGS WITH WAVE ENERGY DISSIPATOR IN PLACE
(c.f. photograph No. 4)
(SWL +1,6 m GMSL)



8: TEMPORARY WEIR IN OUTFALL CHANNEL



9: COMPLETED WAVE ENERGY DISSIPATOR IN SERVICE

7. CONSTRUCTION

The WED was designed in such a manner that it could be constructed with minimum interference with the operation of the power station contractor. This was particularly important since cooling water pumping tests (40 cumecs) were scheduled to commence prior to completion of construction. Furthermore, these tests were likely to be intermittent and of unspecified duration.

A temporary weir (Photograph 8) had been built in the outfall channel ahead of construction of the WED to minimise C.W. flow velocities in the construction area and as far as possible to reduce the return of sand removed from the construction area of the channel. The weir would also afford a degree of wave exclusion.

The weir consisted of tubes made from anchovy fish net filled with 58 mm aggregate. Each tube had a mass of about 2 tonnes. This design successfully accommodated large settlements into the sand infill on which part of it was constructed and facilitated easy removal.

The rock mound consists of a 1 metre deep underlayer of 0 to 1 tonne quarry run rock placed within the precast concrete toe blocks and the remainder of rock is 1 to 3 tonne mass.

The completed structure is shown in Photograph 9.

8. CONCLUSION

It was found that a rubble mound wave energy dissipator located in the outfall channel dramatically reduced wave action at the discharge seal pit. Under maximum discharge the additional head loss in the channel due to the WED was found to be negligible. The authors believe that the rubble mound wave energy dissipator provides an economical and highly effective means of suppressing wave action in an outfall channel without creating impediment or head loss to the cooling water discharge.