CHAPTER 190

THE INFLUENCE OF WAVES ON THE HYDRAULICS OF SEA OUTFALLS

by

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1. INTRODUCTION

Most of the larger cities in Denmark are located near the sea. About 60% of the sewage is therefore discharged through sea outfalls. Most of these sea outfalls were built in the seventies. In most cases a plastic pipeline were used, and the pipeline was ballasted with armoured concrete blocks and burried in a trench 1 - 2 m underneath the seabottom. To give the necessary initial dilution a diffusor with one or several contracted, horizontal outlets ends the pipeline.

The Danish coasts are in general shallow and exposed to waves. The littoral sanddrift is considerably and the sea outfalls need to cross one or more bars. The diffusors are often placed outside the bars, but nevertheless sediment transport can occur around the diffusor. Figure 1 gives some typical data for Danish sea outfalls.

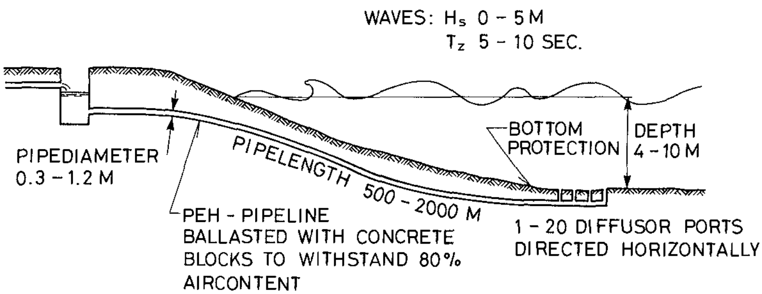

Figure 1. Range of data for Danish sea outfalls.

As a large number of outfalls were designed and established in a short span of time, the practical problems of placing such a rather sophisticated structure in one of the most difficult areas were often underestimated. Many failures have been seen and a considerably number of sea outfalls have an insufficient function. A typical problem is the sediment intrusion into the diffusor, which can block up the ports, and another example is the lifting of the pipeline due to airpockets and insufficient sandcover of the pipe.

This paper discusses some hydraulic aspects of the influence of waves on the pipeline and the diffusor. These effects have not been reported earlier and they might in some cases be connected to the mentioned failures, but it is not pretended that they are general key points in the design of sea outfalls.

2. NUMERICAL MODEL

It is obvious that the influence of waves on the hydraulics of sea outfalls needs an unsteady description and, furthermore, it is also obvious that due to the important non-linear friction analytical solutions are out of question. The straight-forward procedure is then to apply a numerical computer-model.

The numerical model is basically an extension of a water hammer model. The water hammer theory for the unsteady flow in an elastic pipeline is based on the simultaneous solution of the equations of motion for each discrete pipesection all along the pipeline. The equations of motion are the conservation of momentum (1) and theconservation of mass (continuity) (2) including the elasticity of the water and the pipe (Wylie and Streeter, 1983):

$$g \frac{\partial H}{\partial x} + V \frac{\partial V}{\partial x} + \frac{\partial V}{\partial t} + F \frac{V^2}{2d} = 0 \tag{1}$$

$$\frac{a^2}{g} \frac{\partial V}{\partial x} + \frac{\partial H}{\partial x} + \frac{\partial H}{\partial t} = 0 \tag{2}$$

where

- g acceleration of gravity
- H hydraulic head
- x distance along pipeline
- V velocity
- t time
- f Darcy-Weisbach friction factor
- d diameter of pipeline
- a wave celerety for unsteady flow

The wave celerety, a, for pressure and velocity changes is app. 300-400 m/sec in a plastic pipeline and 1,100-1,300 m/sec in a steel pipeline, which are the extremes of practical pipelines in this respect.

2.1 The method of characteristics

To solve the equations (1) and (2) the method of characteristics is convenient. The idea of this method is to choose a computational grid, in which the equations are simplified to a pair of ordinary differential equations (see again Wylie and Streeter, 1983).

The unkowns are the hydraulic head H_p and the velocity in the new point P, figure 2. These are calculated from known values in the two points $i-1$ and $i+1$ one time-step earlicr.

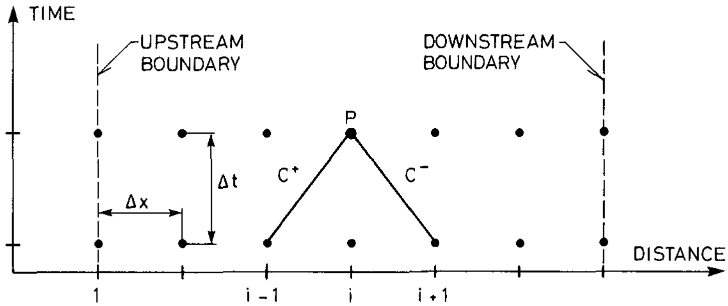

Figure 2. Computational grid for method of characteristics.

2.2 *Upstream boundary condition*

The upstream boundary condition will often be a pump or a reservoir. Most critical is the reservoir, because of the reflection of the fluctuations. A running pump on the other hand absorbs most of the fluctuations in pressure and flow. But a stopped pump with a closed check valve should be considered if necessary.

In this study the upstream boundary was described by assuming a fixed head in the upstream reservoir and a hydraulic entrance head loss between the reservoir and the pipeline.

2.3 *The diffusor boundary*

The downstream boundary is the multiport diffusor with the unsteady pressure field from the waves moving across the diffusor ports. From physical arguments it can be stated that the effect of elasticity can be neglected in the diffusor. Consequently the conservation of mass and momentum can be solved in an ordinary finite difference scheme where the non-linear terms required a solution by iteration.

The moving hydraulic head outside the ports was computed based on linear wave theory. Both the effect of monocromatic waves and complex wind waves have been studied. The wind waves were simulated accordingly to the JONSWAP - spectrum (Hasselmann, 1973). This rather narrow spectrum is known to fit well to the fetch limited wave climate around Denmark. It was assumed that the wave energy travelled in the same direction. This assumption showed out to be acceptable as seen later on.

The linear wave theory gives the following hydraulic head outside each diffusor port

$$H_D = \sum_{i=1}^{n} \left[a_i \sin(\omega_i t + k_i x + \varphi_i) \right] / (\cosh(k_i h)$$

where

H_D	head outside the diffusor
a_i	surface amplitude of wave component
$\omega_i = 2\pi f_i$	cyclic frequency of wave component
$k_i = 2\pi/L_i$	wave number
L_i	wave length
φ_i	random phase
h	depth of water

The surface amplitude a_i was found from the JONSWAP - spectrum

$$S(f) = \frac{\alpha g^2}{(2\pi)^4} f^{-5} \exp\left[-\frac{5}{4} (\frac{f}{f_m})^{-4} \right] \gamma^{\exp\left[-\frac{1}{2\sigma^2} (\frac{f}{f_m} - 1)^2 \right]}$$

where

α	$= 0.076 x^{-0.22}$	factor
x	$= g F U_{10}^{-2}$	factor
f_m	$= \dfrac{3.5 g x^{-0.33}}{U_{10}}$	peak frequency
γ	$= 3.3$	factor
σ	$= \begin{cases} 0.07 & \text{if } f \leqslant f_m \\ 0.09 & \text{if } f > f_m \end{cases}$	
F		fetch
U_{10}		wind velocity

3. RESULTS

As the result of this first step of research is the computer model described above, it feels naturally not to give general conclusions but only some typical examples. Figure 3 shows data for the sea outfall used in the following examples.

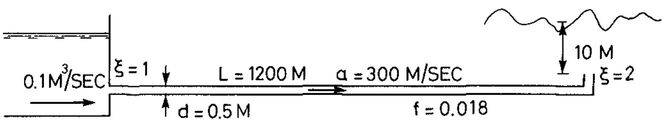

Figure 3. Data for examples.

3.1 Total hydraulic capacity

The total head loss in the pipeline is not significant increased due to the waves in the case of a single port diffusor. For the multiport diffusor a slight increase (0 - 5%) of the head loss can occur.

3.2 Resonance

It is well-known from physics that the wavelength of the basic mode standing wave in a pipeline open in each end is the double of the length of the pipe. The resonance period for a pipeline with L = 1200 m and a = 300 m/sec is then 8 sec. Figure 4 shows a timeplot of how this resonance built up from steady state.

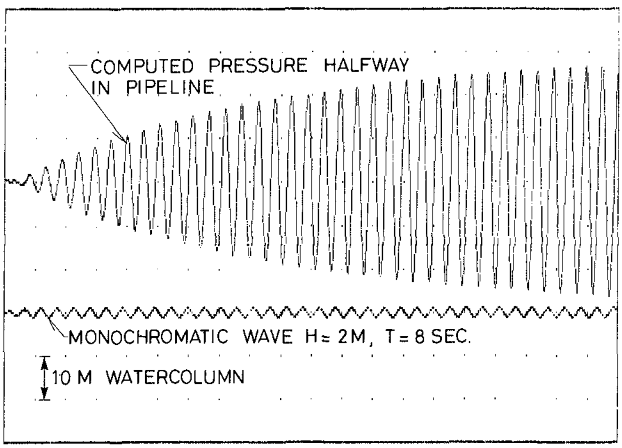

Figure 4. Resonance due to monochromatic waves.

Figure 5 shows the computed results if the waves are irregular according to the JON-SWAP - spectrum.

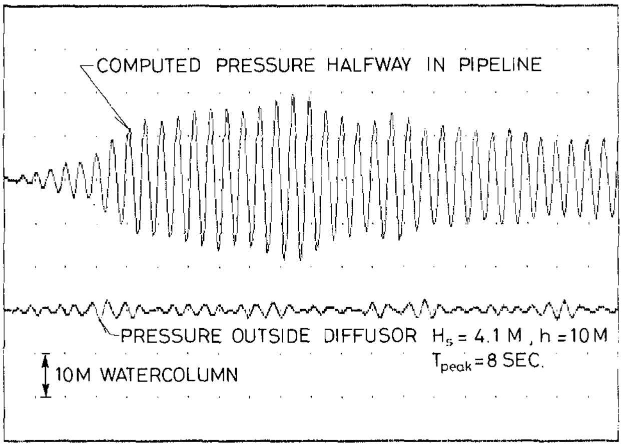

Figure 5. Resonance due to wind waves.

The figures show that the dampening of the standing waves apparently is very small and the pressure fluctuations are surprisingly large. For the moment this has not been experimental verified and the results can only be interpreted as an impulse to further studies.

The above mentioned examples are based on the basic mode of resonance in the pipeline. Resonance should of course be expected on other modes depending on the waves and the pipeline. The resonance periods for a pipeline open in each end are 2 L/a, L/a, 2/3 L/a, 1/2 L/a, etc. If the pipeline is closed in the upstream end (stopped pump and closed check valve) the first resonance periods are 4 L/a, 4/3 L/a, 4/5 L/a, etc. In this connection the long periods are expected to give the strongest resonance because of the lowest dampening.

3.3 Back flow in diffusor

In this example the outfall from figure 3 was connected to a diffusor with 5 ports with a diamater of 0.15 m and a head loss factor $\xi = 2.0$. Over this diffusor wind waves propagated as in figure 5 with $H_s = 4.1$m and $T_{peak} = 8$ sec. Furthermore, the length of the pipeline was L = 1500 m to avoid resonance. Figure 6 shows an instantaneous picture of the flow distribution, where the waves created a back flow in one of the ports.

From an analysis of the results it can be concluded that the unsteady flow in the diffusor to some degree follows the flow near the bottom due to the waves. As the head losses through the ports are smaller than the head fluctuations, it is understood that the local effect of the waves between two adjacent ports is most important. This knowledge can

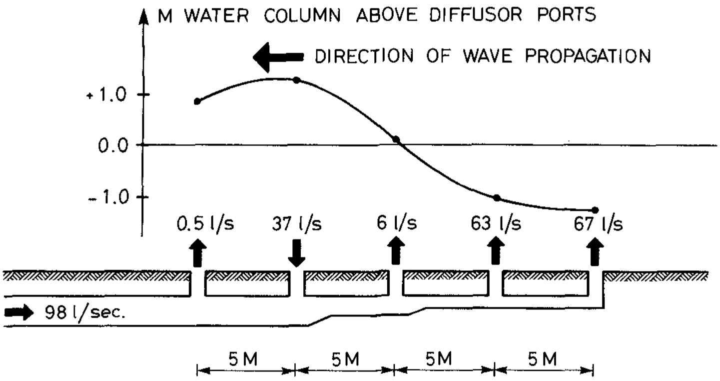

Figure 6. Instantaneous flow in diffusor under waves.

then support the assumption of only applying a uni-directional wave spectrum as the boundary condition to the computation; or with other words, the waves are not short-crested in relation to the distance between two diffusor ports, which only is a fraction of the water depth.

4. CONCLUSION

The results summarized above indicate that waves can cause interesting hydraulic phenomena in sea outfalls on shallow coasts. The results also explain why this has not been realized directly earlier; simply because the resonance in the main pipeline and the back flow in the diffusor cannot be observed in the upstream end of the pipeline.

The author will appreciate very much to be informed about observations on these effects.

5. REFERENCES

Hasselmann, K. et al. (1973): Measurements of Wind - Wave Growth and Swell Decay during the Joint North Sea Wave Project (JONSWAP). Deutsche Hydrographischen Zeitschrifft., Reihe A (8°), Nr. 12.

Wylie, E.B. and Streeter, L. (1983): Fluid Transients. FEB Press, Ann Arbour, Michigan.