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Optimum Sewage Discharge Strategy for Coastal Waters

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To improve the water quality, particularly for sea bathers, the behaviour of wastewater from sewage outfalls in water adjacent to Swansea, UK, was studied using a mathematical model. The water quality in the sewage receiving basin was determined using factors like the outfall diffuser location(distance from land boundary), sewage treatment scheme, discharge time, and bacteria decay rate, *etc.* With respect to these factors, an optimal strategy for sewage discharge was then investigated to minimize bacteria levels along the bathing beaches. As water quality criteria, predicted faecal coliform levels were monitored along the coast adjacent to the outfall locations. The resultant values were compared with EC Mandatory(<2000, 95 % of 20 samples) and Guideline Standards(<100, 80 % of 20 samples). For the advective-diffusion equation, the non linear advective terms were represented using the ULTIMATE algorithm and the third-order accurate QUICKEST scheme to avoid numerical diffusion. Details of the simulation results are then presented as an optimal policy for sewage discharge in the region.

Key words : bathing water quality, sea outfall, faecal coliform, ULTIMATE QUICKEST scheme, sewage treatment scheme

1. Introduction

Urban wastewater sewage is pumped into coastal basins everyday. The amounts of sewage vary greatly according to the level of the treatment scheme. However, much of the sewage is either raw or receives little treatment. Sewage consists of a mixture of domestic wastewater, industrial effluent, and storm water run-off. Typically, this includes human waste, detergents, fats, solvents, petroleum products, and heavy metal contaminants. Human waste conveyed in sewage carries pathogens and a high content of decomposing organic matter, which then cause serious ecological problems when they are released into coastal waters. In particular, some of the bacteria and viruses are hazardous to human health. While the actual risk of contracting a fatal illness as a result of bathing in polluted coastal water or eating contaminated shellfish is low, many studies have show that the chances of catching other less serious illnesses, e.g. ear, nose and throat infections, or diarrhoea and vomiting, are much higher[1,2]. Coastal sewage pollution can thus be a serious threat to bathers in coastal waters. Furthermore, since most coastal areas are vulnerable to sewage pollution, how urban sewage is treated and where it is disposed are both extremely important.

There have been various studies about establishing an outfall system and operation in rivers, estuaries, and coastal seas. The main purpose is to achieve a swift mixing and dilution of the effluent as well as a rapid transport of the water mass away from the outfall diffuser. As the tide oscillates back and forth in estuarine environments, the same water can repeatedly pass by the discharge site and receive multiple dosages of effluent[3]. Such concentration peaks have been studied to prevent sewage effluents from going back to the source[4~7]. As such, an intermittent discharge is recommended in such cases instead of a continuous discharge so as to avoid the potential of a peak pollution concentration[8]. Meanwhile, the location of sea outfalls should be based on the behaviour and fate of the sewage after discharge. The adverse effects of sewage discharges can be minimized by dilution with large volumes of seawater and by natural purification processes. The following are important factors in determining an outfall location: initial dilution, speed and direction of travel after initial dilution, rate of dispersion during travel, *etc.* When sewage discharges from diffusers on the sea bed, a turbulent buoyant jet is merged into a plume. Ambient water is then entrained into the plume due to dispersion processes caused by the plume rising. As a result, the plume is rapidly diluted, keeps rising to the point of neutral buoyancy, and then starts spreading laterally. At this stage, the turbulent mixing decreases rapidly and the initial mixing ends, so a far-field model can be applied. Near-field models deal with the initial dilution processes occurring in the immediate vicinity of the sewage outfall, and have been successfully applied to the initial mixing processes[9~11]. However, near-field models are unable to accurately predict the effluent dispersion advected away from the outfall. In contrast, far-field circulation models deal with processes such as water column stratification, currents varying in time and space, and the long distance dispersion of pollutants, although they can result in erroneous predictions resulting from inaccuracies in the initial mixing processes[12].

In the current study, a mathematical model was used to study the effluent dispersion from sewage outfalls to improve the water quality, particularly for sea bathers, along the waters adjacent to Swansea, UK. A sewage outfall already existed 0.8 km from the land boundary and the effluent was treated according to the secondary treatment scheme in the region. The simulations were focused to identify an optimal strategy and avoid adverse effects when operating sewage outfalls based on minimizing the concentration levels dispersed along the bathing beaches. Various factors influencing the effluent dispersion and dilution were included in the tests; the sewage treatment scheme (free-, primary-, secondary-, and tertiary-), outfall diffuser length (distance from land boundary; 0.8, 1.5, 2 and 3 km), discharge-time scheme (9 cases including continuous discharge), and constant bacteria die-off rate. For the water quality criteria, the predicted faecal coliform levels were monitored along the coast adjacent to the outfall location. The resultant values were compared with EC mandatory and guideline standards.

A 3-D layer-integrated model was used for the numerical experiments. The current study focused on the dispersion processes on a large scale under barotrophic conditions. In solute transport models, higher-order difference schemes generally reduce the numerical diffusion errors. Nonetheless, non-physical oscillatory solutions can still arise if sharp changes occur in the local gradients. For example, negative concentrations are frequently encountered in the vicinity of outfalls as a result of such oscillations. Therefore, this study used a highly accurate scheme to treat the advective terms based on applying the ULTIMATE algorithm and third-order accurate QUICKEST scheme[13].

2. Materials and Methods

2.1. Governing Equations

The governing equations used in the hydrodynamic model are based on the 3-D Reynolds equations for incompressible and unsteady turbulent flows. As in most current 3-D tidal circulation models, the current study adopted the assumption of a vertical hydrostatic pressure distribution. As such, the governing equations of mass and momentum can be written in the following form :

$$\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}+\frac{\partial w}{\partial z}=0 \tag{1}$$

$$\frac{\partial u}{\partial t}+\frac{\partial uu}{\partial x}+\frac{\partial vu}{\partial y}+\frac{\partial wu}{\partial z}=fv-\frac{1}{\rho}\frac{\partial p}{\partial x}+\frac{1}{\rho}\left[\frac{\partial \sigma_{xx}}{\partial x}+\frac{\partial \tau_{yx}}{\partial y}+\frac{\partial \tau_{zx}}{\partial z}\right] \tag{2}$$

$$\frac{\partial v}{\partial t}+\frac{\partial uv}{\partial x}+\frac{\partial vv}{\partial y}+\frac{\partial wv}{\partial z}=-fu-\frac{1}{\rho}\frac{\partial p}{\partial y}+\frac{1}{\rho}\left[\frac{\partial \tau_{xy}}{\partial x}+\frac{\partial \sigma_{yy}}{\partial y}+\frac{\partial \tau_{zy}}{\partial z}\right] \tag{3}$$

$$\frac{\partial p}{\partial z}+\rho g=0 \tag{4}$$

where t=time, x,y,z=Cartesian co-ordinates in the horizontal and vertical planes, respectively, u,v,w=components of velocity in the x,y,z directions, respectively, p=pressure, g= gravitational acceleration, f=Coriolis parameter, ρ =density of fluid, and $\sigma_{xx}, \tau_{yx}, \tau_{zx}, \tau_{xy}, \sigma_{yy}, \tau_{zy}$

=components of the stress tensor in the x-z and y-z planes, respectively. The density ρ is determined from the equation of state:

$$\rho = \rho_o (1 + \alpha S) \tag{5}$$

where S=salinity concentration, α =constant, and ρ_o =density of fresh water.

The mass balance equation for the general solute concentration C is written as:

$$\frac{\partial C}{\partial t} + \frac{\partial uC}{\partial x} + \frac{\partial vC}{\partial y} + \frac{\partial wC}{\partial z} = \frac{\partial}{\partial x}\left(\upsilon_x \frac{\partial C}{\partial x}\right) + \frac{\partial}{\partial y}\left(\upsilon_y \frac{\partial C}{\partial y}\right) + \frac{\partial}{\partial z}\left(\upsilon_z \frac{\partial C}{\partial z}\right) + C_s + C_b + C_k \tag{6}$$

where υ_x, υ_y, υ_z=turbulent diffusion coefficients in the x,y,z directions, respectively, C_s=source terms including direct and diffuse loading rate, C_b=boundary loading rate, and C_k=total kinetic transformation rate[14].

For the faecal coliform modelled in this study, the kinetic transformation rate C_k of the equation (6) is written as :

$$C_k = -kC \tag{7}$$

where k is the decay rate.

2.2. Numerical Methods

The hydrodynamic equations from (1) to (4) were solved using the finite difference technique on a regular square mesh in the horizontal plane and a layer integrated finite difference scheme on an irregular mesh in the vertical plane. Depth integrated equations were then used to determine the velocities using this water elevation field, with two iterations performed during every half time step. The Alternating Direction Implicit method was modified to solve the depth integrated equations. When solving the layer integrated equations the vertical diffusion terms were treated implicitly, with the rest of the terms being solved explicitly. A robust scheme for dealing with the problems of flooding and drying was also included. More details of the hydrodynamic model are given by Lin and Falconer[15]. When solving the solute transport equation (6), an operator splitting algorithm was used. The advantage of this method is that the original equations can be rearranged into small sub-equations and each sub-equation can be solved using the most suitable scheme. In an estuarine and coastal model, the horizontal dimension is much larger than the vertical one, thus equation (6) was rearranged into the following two equations:

$$\frac{\partial C}{\partial t} + \frac{\partial wC}{\partial z} = \frac{\partial}{\partial z}\left(\upsilon_z \frac{\partial C}{\partial z}\right) + C_v \tag{8}$$

for i= 1,2..., $Imax$, j=1,2..., $Jmax$ and $t \in (t_n, t_{n+1})$ and

$$\frac{\partial C}{\partial t} + \frac{\partial uC}{\partial x} + \frac{\partial vC}{\partial y} = \frac{\partial}{\partial x}\left(\upsilon_x \frac{\partial C}{\partial x}\right) + \frac{\partial}{\partial y}\left(\upsilon_y \frac{\partial C}{\partial y}\right) + C_h \tag{9}$$

for k=1,2..., $Kmax$ and $t \in (t_n, t_{n+1})$ where C_v and C_h=the kinetic transformation terms in the vertical and horizontal directions respectively.

Equation (8) was discretised in an unequal spaced grid format, using the same grid as that used in the hydrodynamic model. Since the diffusion term is more important in this equation, the equation was solved using the implicit finite volume method with the total mass flux approximated using a power-law scheme[16]:

$$-a_S C_{k-1}^{n+1} + a_P C_k^{n+1} - a_N C_{k+1}^{n+1} = b \tag{10}$$

where

$$a_N = D_n A(|Pe_n|) + [-F_n, 0]$$

$$a_S = D_S A(|Pe_S|) + [F_S, 0]$$

$$b = \frac{\Delta z}{\Delta t} C_k^n$$

$$a_P = a_N + a_S + \frac{\Delta z}{\Delta t}$$

where

$$A(|P_e|) = [0, (1 - 0.1|P_e|)^5 \tag{11}$$

the symbol $[a, b]$ is used to denote the greater of a and b, subscripts n, s denote the control volume face, P_e=grid Peclet number, D=diffusion conductance, and F=mass flow rate. Their definitions can be found in Patankar[16]. Equation (9) was solved using the ULTIMATE QUICKEST scheme, originally developed by Leonard[17], and modified for estuarine and coastal flow circumstances by Lin and Falconer[15].

2.3. Numerical Experiments

Swansea Bay receives domestic sewage from various sources including the Mumbles outfall, River Neath, Afan, and Tawe(Fig. 1). The Mumbles outfall discharges a 70 % contaminant level for the total amount of treated sewage. The Mumbles outfall is located adjacent to several bathing beaches, while the other pollutant sources are located quite a distance from any beaches and would not seem likely to produce any significant burden of pollution. As such, only the Mumble *outfall was considered in the current study.* The Mumble outfall diffuser is 3.5 km long with an internal diameter of 1.3 m, discharging into Swansea Bay at National Grid Ref. SS 689929 (Fig. 1). The discharge rate was calculated based on the size of the crude sewage tank and discharge time. Treated wastewater is discharged via the outfall for 4 hours starting 1 hour before high water in Neap. However, in Spring there is a break from 0.5 hours before high water until 0.5 hours after high water, although the starting and finishing times of discharge are the same as those in Neap. This operating strategy is still in use.

There are various factors influencing the dispersion of sewage effluents, including the outfall diffuser length, sewage treatment scheme, discharge time, and discharge rate, *etc.* Each constraint was investigated to identify the optimal strategy for sewage discharge based on minimizing the bacteria levels along the bathing beaches adjacent to the outfall diffuser location. For the purposes of comparison, two conditions were considered for the simulations. First, faecal coliform was used for the dispersion effects, since faecal coliform bacteria are found in the intestinal tract of both humans and animals. As such, they are commonly used as indicator organisms in water quality studies, and their presence indicates that faecal material has contaminated the water.

Furthermore, there are two European standards for bathing water quality as set by the EC Bathing Water Directive(76/160/EEC)[18] : the Mandatory Standards(also known as Imperative or Minimum Standards) and Guideline Standards, which are twenty times stricter and focus on the levels of sewage-derived bacteria. The maximum levels of bacteria permitted per 100 *ml* sample of seawater are given as 95 % of 20 samples <2,000 for the Mandatory and 80 % of 20 samples <100 for the Guideline Standards. The second condition was that the bacteria were monitored along the bathing beaches adjacent to the outfall diffuser location, since bathers generally only swim within a limited shallow coastal region. Hence, the predicted maximum levels of faecal coliform were recorded at first wet grids juxtaposed to dry cells representing land in the computational domain.

The discharge constraints mentioned above were tested following the procedure specified in Fig. 2, with selected options to exceed the EC standards being given as the basic discharge conditions for the next test. Each simulation was carried out using a constant decay rate.

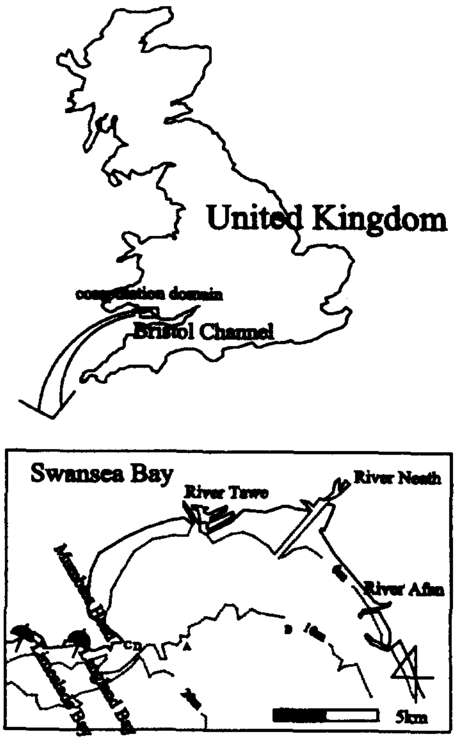

Fig. 1. Map showing Swansea Bay with various discharge sources and bathing beaches(A and B : velocity measurement point, C : water elevation measurement point and D : outfall location).

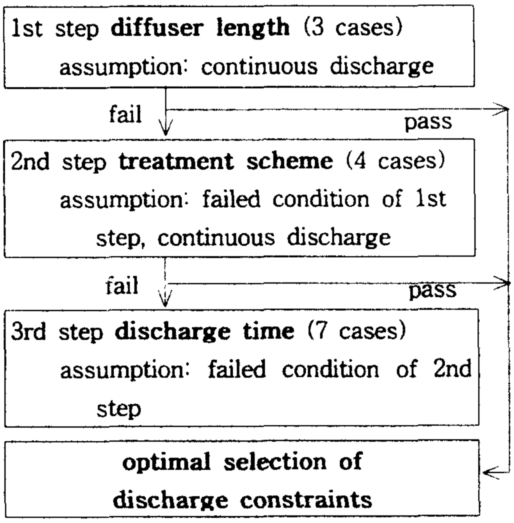

Fig. 2. Simulation procedure with constraints and assumptions.

3. Results and Discussion

3.1 Hydrodynamic Calibration

Swansea Bay is located along the northwest coast of the Bristol Channel, Wales(Fig. 1). The computational domain was represented using a horizontal mesh of 94 by 60 grid cells, equally spaced at 250 m intervals. A representative depth of the basin bed elevation below datum was required at the centre of the sides of each grid. An Admiralty Chart was used to give the bathymetric data. Six layers were used vertically with a 5m thickness. The western open seaward boundary was assumed to be a water elevation boundary. The tidal elevation was calculated using tidal harmonic constant components(M_2, S_2, O_1, and K_1) and then specified at the northern point along the boundary for both Spring and Neap. The flow along the southern open boundary was assumed to be parallel to the boundary, as indicated by the general flow patterns given in the tidal atlas. The simulation started at high water.

The hydrodynamic model was calibrated by comparing the model results with velocity measurements at points *A* and *B* specified on the Admiralty Chart and the water elevation at Mumbles Head pier(Fig. 1). As seen in Fig. 3 the measured and predicted velocities were in close agreement at all sites for both Neap and Spring. The predicted water elevation agreed closely with the calculated values(Fig. 4). The predicted velocity fields were illustrated over the computation domain for the maximum flood and ebb phases in Spring (Fig. 5).

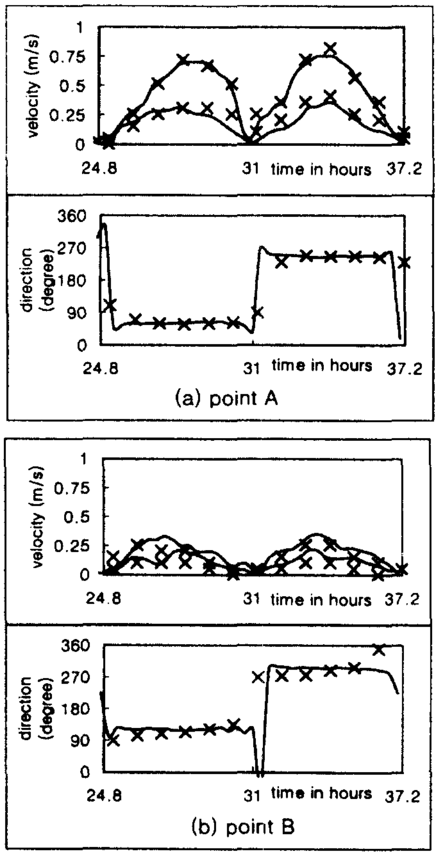

Fig. 3. Comparison of measured(×) and predicted(-) velocities at points A and B for Neap and Spring.

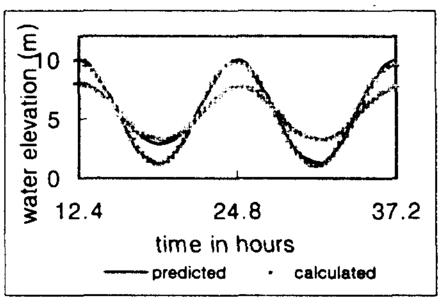

Fig. 4. Comparison of measured and predicted water elevations at point C for Neap and Spring.

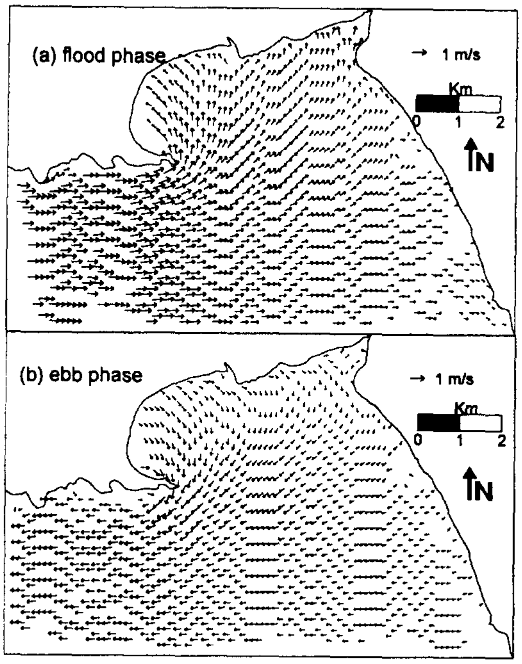

Fig. 5. Predicted velocity fields for maximum flood and ebb phases in Spring.

3.2. Discharge Simulations

3.2.1. Outfall Diffuser Length

Water quality and cost are the two most important factors when considering the design of a sea outfall. In general, the outfall diffuser length determines the impact of its effluent discharge on the receiving basin; The longer, the better. However, the cost will be higher depending upon the diffuser length. As such, the potential area of sewage dispersion should be investigated by simulating the transport and mixing processes of the effluent plume in the relevant coastal waters. It is normally recommended to build an outfall at a location where the water depth is sufficient to enforce an initial mixing of the buoyant sewage, and the plume flow speed is strong enough to carry the sewage effluents seaward. In the study region, the outfall was established 0.8 km away from the land boundary and currently in use. However, this study also investigated various outfall locations at 1.5, 2 and 3 km south of the established outfall location. The simulations were run to compare the effects of the diffuser length, while the maximum levels of faecal coliform were monitored along the coast adjacent to the outfall(Fig. 1). The following assumptions were made for the simulations : - free-treatment scheme(crude sewage discharge), constant discharge rate(no holding tank and/or pumping facilities) for both Spring and Neap, and constant mortality rate of T_{90} at night and during the day(50 hours).

The numerical tests including the conditions mentioned above, see Fig. 6, indicated that diffuser lengths of 2 and 3 km resulted in very low levels of faecal coliform during most of the simulation time, meaning that these locations met with the guideline standards(76/160/EEC)[18]. In contrast, the diffuser lengths of 0.8 and 1.5 km showed peak levels of 1575 and 360 cfu/100 ml for Spring and 2277 and 426 cfu/100 ml for Neap, respectively. Although the 0.8 km condition exceeded the mandatory standard, *i.e.* >2000, since the duration was only half an hour, it was still acceptable. All other conditions met with the standard(Table 1). As the tidal currents are weaker in Neap than in Spring, the peaks and environmental impact can be larger in Neap due to a weaker flow and corresponding lower dispersion. Accordingly, the conditions of Neap with 0.8 and 1.5 km diffuser lengths were applied to all subsequent simulations.

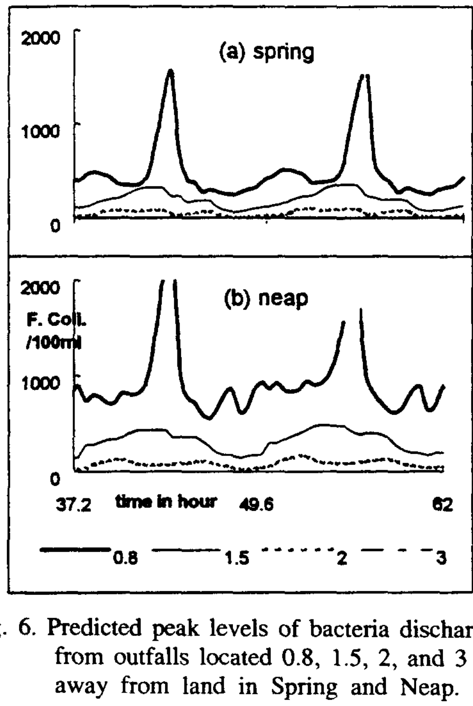

Fig. 6. Predicted peak levels of bacteria discharged from outfalls located 0.8, 1.5, 2, and 3 km away from land in Spring and Neap.

Table 1. Tests of various diffuser lengths for Neap and Spring tides(0 : pass and × : fail)

diffuser	mandatory		guideline	
length	spring	neap	spring	neap
0.8	0	0	×	×
1.5	0	0	×	×
2	0	0	0	0
3	0	0	0	0

3.2.2. Treatment Scheme

Urban wastewater collected from a sewage treatment plant was discharged into a receiving basin through a pipe after being processed by primary-, secondary-, and tertiary- treatment schemes, *etc.* Primary treatment involves settling for several hours to remove 50 % of the bacteria and viruses and 50～60 % of the suspended solids. Secondary treatment stimulates the biological activity of the sewage and removes 75～99 % of the bacteria and 90～95 % of the suspended solids. Tertiary schemes or disinfection systems include chemical, physical and ultraviolet(UV) processes. The last option appears to present no threat to the marine environment, since this treatment is non-additive and removes all viral agents as well as bacteria from the sewage effluent. In the current study, each type of system was assumed to have crude sewage reduction rate of 50 %, 90 %, and 99 %, respectively. The purpose of all these treatment schemes is to use advanced technology to improve water quality, however, this also involves higher establishment and operational costs. At the time of the current study, secondary treatment schemes were in operation in the study region. Secondary treatment schemes are considered as the standard minimum level of treatment throughout the European Union for all coastal sewage discharge serving populations of more than 10,000 people and for estuarine discharges which serve more than 2,000 people(91/271/EEC)[19]. However, for the sake of comparison, all treatment schemes were tested in the present study. Based on the results in the previous section, the diffuser lengths were given as 0.8 and 1.5 km during Neap. The discharge rate and T_{90} were assumed to be constant, as seen in the previous section. Fig. 7 shows that every condition met with the mandatory standards. Yet, only the secondary and disinfection treatment schemes predicted low(<100) peak levels and met with the guideline standards. As such, the free- and primary- treatment schemes did not comply with the guideline standards at diffuser lengths of 0.8 and 1.5 km(Table 2).

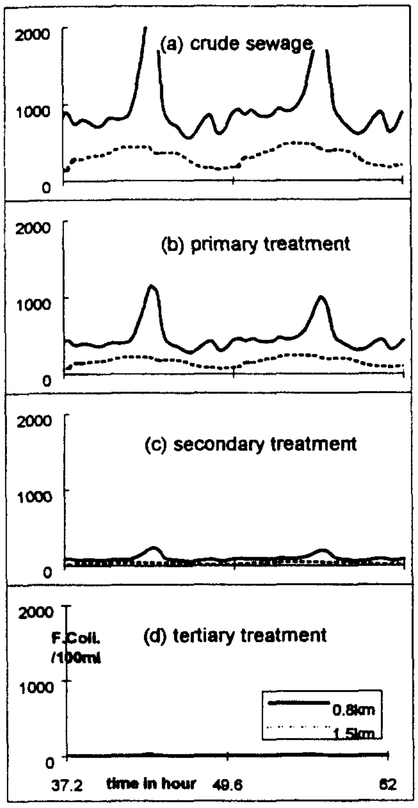

Fig. 7. Predicted peak levels of bacteria discharged from outfalls located 0.8 and 1.5 km from land with various treatment schemes during Neap.

Table 2. Tests of various treatment schemes at two diffuser lengths, 0.8 and 1.5 km, with Neap tide(0 : pass and × : fail)

treatment	mandatory		guideline	
scheme	0.8km	1.5km	0.8km	1.5km
free	O	O	×	×
Primary	O	O	×	×
Secondary	O	O	O	O
tertiary	O	O	O	O

3.2.3. Discharge Time Scheme

One of the constraints that should also be considered is the discharge time associated with the tidal phase. In coastal environments, since the tide oscillates back and forth, water can repeatedly pass by outfall locations and thus receive multiple dosages of effluent. As such, a continuous discharge is not generally used to avoid a potential peak pollution concentration. Instead intermittent discharge is highly recommended[8]. The conditions due to a tidal phase are specified in Fig. 8, with the tidal period given as 12.4 hours representing a lunar semi-diurnal tide : -

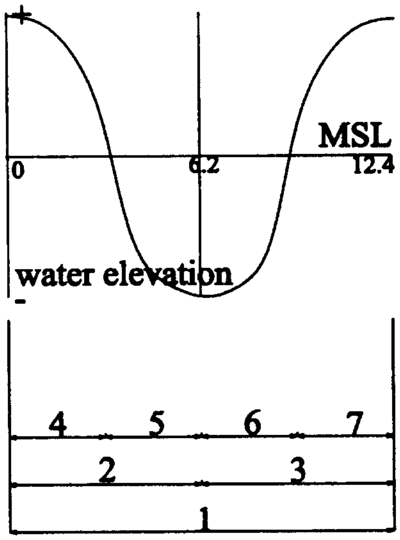

Fig. 8. Various cases of discharge time schemes.

note :
case 1 - discharge continuously
case 2 - discharge during ebb
case 3 - discharge during flood
case 4 - discharge during high water to mid-ebb
case 5 - discharge during mid-ebb to low water
case 6 - discharge during low water to mid-flood
case 7 - discharge during mid-flood to high water

Three conditions were given: during Neap, diffuser length, and decay rate, as shown in the previous tests. Assuming a constant amount of discharge per single tidal cycle, the sewage was discharged at various rates depending on the discharge-time schemes specified in Fig. 8. At a diffuser length of 0.8 km, free- and primary-treatment schemes were simulated using 7 discharge-time schemes. Under free-treatment conditions, cases 2, 5, and 7 exceeded the mandatory standards. Cases 3, 4s and 6 did not exhibit reduced coliform levels compared with case 1. To avoid confusion, Fig. 9 only shows 3 of the 7 cases, with cases 2 and 3 representing two groups respectively. With the primary-treatment scheme, case 5 exceeded the mandatory standards (max. 5300 cfu/100 ml), with higher levels arising in cases 2 and 7, as seen from the results of the free-treatment discharge. The other cases resulted in similar distributions to case 1.

In contrast, none of the free- and primary-treatment schemes passed the guideline standards, irrespective of the discharge-time conditions at a diffuser length of 1.5 km (Fig. 10). Case 2 exhibited lower coliform levels than case 1. However, the difference was not significant and none of the cases complied with the guideline standards. The maximum levels were predicted in descending order for cases 6, 7, 4, 3, 5, 1, and 2.

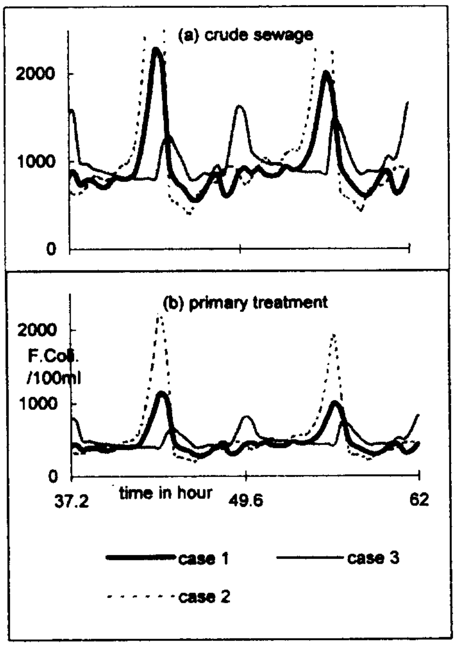

Fig. 9. Predicted levels of bacteria discharged from outfall located 0.8 km away from land with free(crude sewage)- and primary- treatment schemes during Neap.

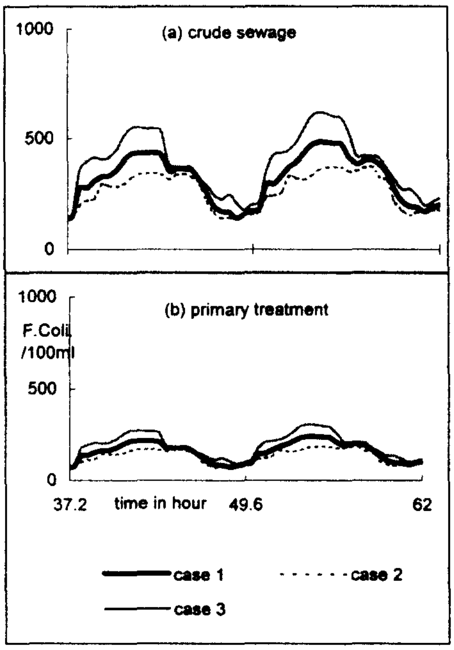

Fig. 10. Predicted levels of bacteria discharged from outfall located 1.5 km away from land with free(crude sewage)- and primary- treatment schemes during Neap.

Accordingly, the constraint of the discharge-time did not effectively decrease the peak levels for this study region. On the contrary, some cases actually increased their maximum level, *e.g.* cases 2, 5, and 7 with a diffuser length of 0.8 km and cases 6 and 7 with a diffuser length of 1.5 km. It is worth noting that some cases apparently deteriorated the water quality and the cases were not identical at all outfall locations. This fact strongly suggests that detrimental conditions can be avoided through numerical simulations prior to determining outfall operation strategies.

4. Conclusion

The behaviour of wastewater from sewage outfalls in Swansea, UK was studied to improve the water quality, particularly for sea bathers, using a mathematical model where the advection terms were treated using the third-order accurate ULTIMATE QUICKEST scheme. The predicted maximum levels of faecal coliform were monitored along the coast adjacent to the outfall location and compared with EC mandatory and guideline standards. Various factors influencing the effluent dispersion were simulated to identify an optimal strategy for operating sewage; including the diffuser length, sewage treatment scheme, and discharge time scheme. The main findings from the corresponding coliform level predictions can be summarized as follows :

i) Assuming a continuous discharge of crude sewage, diffuser lengths of 2 and 3 km resulted in compliance with both the mandatory and guideline standards. However, diffuser lengths of 0.8 and 1.5 km resulted in peak levels of 1575 and 360 cfu/100ml for Spring and 2277 and 426 cfu/100ml for Neap, respectively, which only passes the mandatory standards.

ii) At diffuser lengths of 0.8 and 1.5 km during Neap, various treatment schemes were simulated, including free-, primary-, secondary-, and tertiary-treatment schemes, which demonstrated that only secondary- and tertiary-treatment schemes met with both the mandatory and guideline standards. Whereas the other two options exceeded the guideline standards, although satisfying the mandatory standards.

iii) Seven discharge time schemes were tested using free- and primary- treatment schemes with outfall diffusers of 0.8 and 1.5 km during Neap. None of the experiments were found to effectively decrease the coliform levels. Instead, some conditions even increased the maximum levels.

iv) From the experimental results, it was confirmed that the current strategy of sewage operation successfully passed the EC standards; 0.8 km distance, secondary treatment scheme, and discharge during 1 hour before highwater until 4 hours after highwater. Yet, when considering that the current microbiological standards are not stringent enough to minimize the risk of contracting a serious illness, beaches passing only the mandatory standards still present a health hazard to bathers. Hence, a secondary treatment scheme is clearly essential for passing the guideline standards for most discharge time conditions in the study region.

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