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WATER QUALITY MODELING OF MUNICIPAL DISCHARGES FROM SEA OUTFALLS, MUMBAI

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Abstract. The island city of Mumbai with a population of about 10 million generates about 2000 million liters per day (mld) of sewage from the seven service areas of the city sewerage network and discharges it into the adjoining west coast and the two creeks in the Arabian Sea. This has resulted in degradation of coastal water quality, contamination of the adjoining beaches and seafronts. The Municipal Corporation of Greater Mumbai has therefore, undertaken the task of delineating appropriate sewage disposal system to achieve cleaner marine ecosystem through marine outfalls at specific locations. This paper presents the results of the mathematical simulations on the impacts of discharge is-a-vis the length of the outfall and level of land treatment apriori. The results of the simulation indicate the level of bacterial pollution to be higher near the diffuser locations as compared to nearshore regions. 48 h simulation result analysis shows that FC counts near the diffuser location will be in the range of 2000–8000 counts per 100 ml.

Keywords: mathematical simulations, Mumbai, sea outfalls

1. Introduction

The sewerage system in Mumbai has been divided into seven service areas and serves a population of about 10 million (Figure 1). The sewage from these service areas is discharged into the open west coast and the adjoining creeks with only preliminary treatment thereby affecting the water quality of receiving water body. Of the seven service areas, five discharge the sewage from their respective areas into the west coast and this constitutes about 70 percent of the total sewage discharged. The present wastewater discharges on the west coast have created many critical areas where water quality is severely impaired in terms of physicochemical and bacterial indicators (NEERI, 1994). The rapid economic growth and population increase in Mumbai projected for the year 2015 AD and the sewage flow is anticipated to almost double.

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This paper presents the results of mathematical simulation on the impacts of the sewage discharges on water quality for the various management options.

2. Current Water Quality Status of Mumbai West Coast

The assimilative capacity of the sea is high enough to have considerable dilution effects on parameters such as Biochemical Oxygen Demand (BOD), Dissolved Oxygen (DO), nutrients and other physicochemical parameters. The microbial parameter particularly Fecal Coliforms (FC) concentration remains the best pollution indicator for coastal waters.

The DO and bacterial (FC) profiles in the 24 km long west coast are presented in Figures 2 and 3 respectively. The DO levels show that about 5 km length of sea on the west coast is highly polluted, mainly the regions such as Malad and Mahim, which receive about 50% of the wastewater discharge from the city. The other 8 km stretch of the coast shows moderate depletion of the DO. Other parameters such as BOD and nutrients are low in most of the regions, whereas bacterial contamination in the west coast of Mumbai is high with FC levels ranging from 10^3–10^4 counts per 100 ml. Only about 25% of the west coast have fecal coliform counts below 500 per 100 ml.

If this sewage is continued to be discharged without treatment, the coastal water quality will be adversely affected, as there will be severe stress on the assimilative capacity of the receiving water body. The development of wastewater management scenario for Mumbai warrants consideration of the natural sink in the ocean. The feasibility of using this requires optimal design of wastewater treatment and disposal so that the natural assimilative capacity of the ocean is not depleted and there are no significant impacts on the flora and fauna of west coast.

The Municipal Corporation of Mumbai has undertaken the task of management of these discharges. Under the plan, 3 km long marine outfalls are planned to be constructed at Worli, Bandra, Malad and Versova for discharge of post treated sewage. Various options involving the level of land treatment and length of outfalls in increments of 1 km upto 8 kms were explored. The options of land treatment were mainly primary and secondary.

3. Mathematical Models

To assist the hydro environmental impact assessment, it is often necessary to simulate flow patterns and monitor pollutant transport processes. This is often carried out by use of computational hydrodynamic and solute transport models. There are certain limitations in using these models which include the accuracy of predicting turbulence and sediment transport interactions, chemical and biological processes relating to water quality parameters and the numerical difficulty in treating high

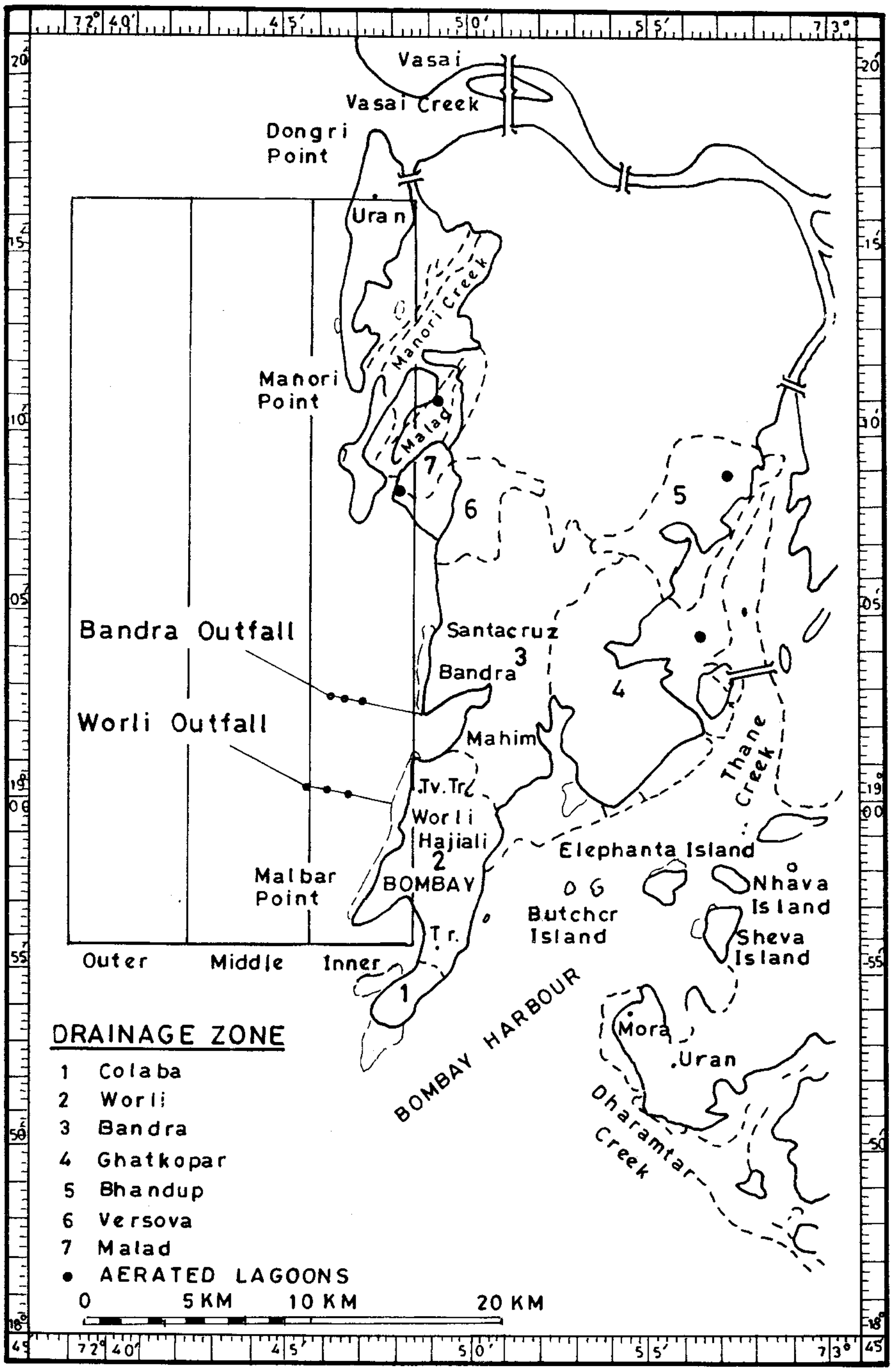

Figure 1. Modeling Domain and Seven Service Areas of Mumbai.

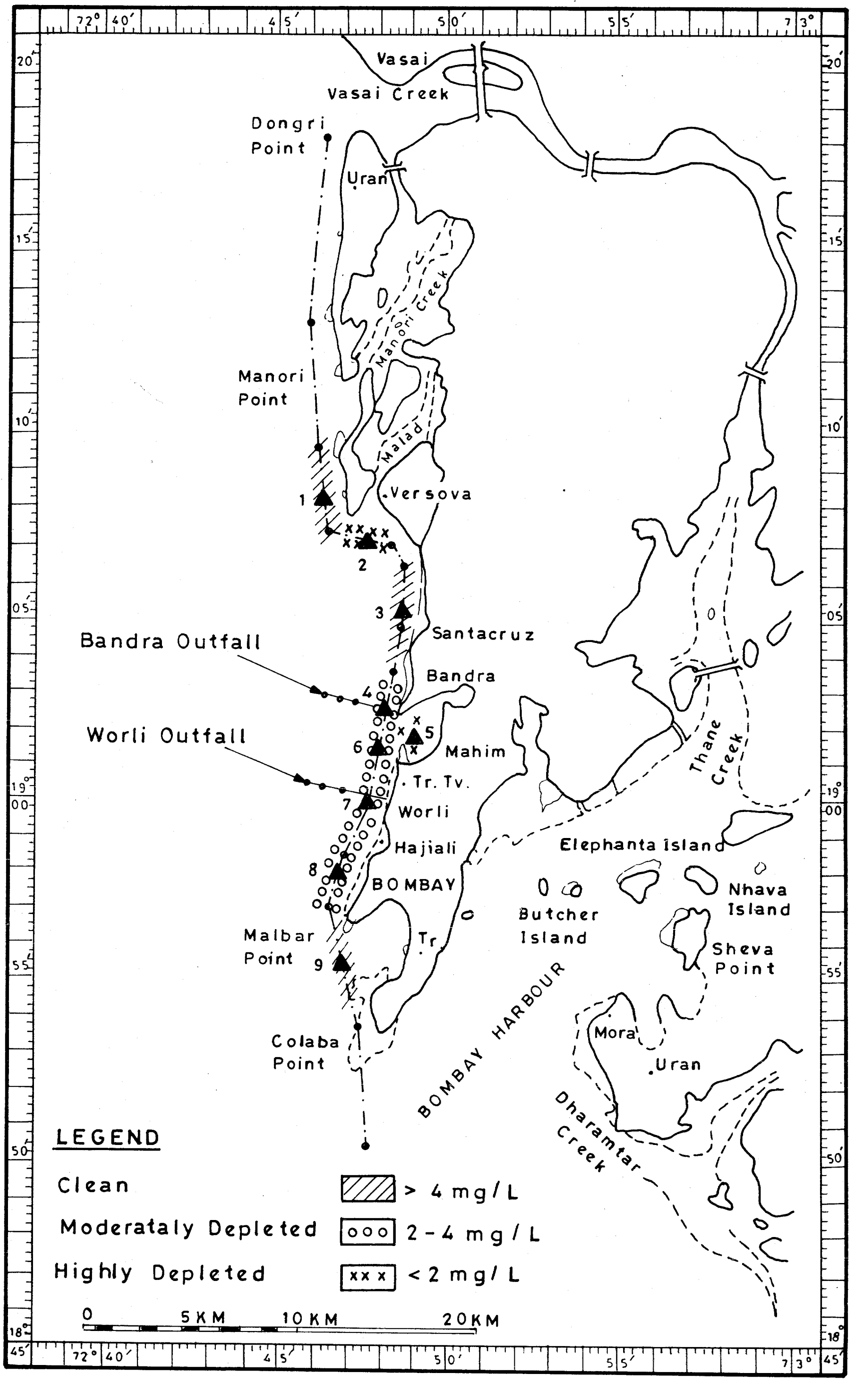

Figure 2. Present DO levels at Mumbai West Coast.

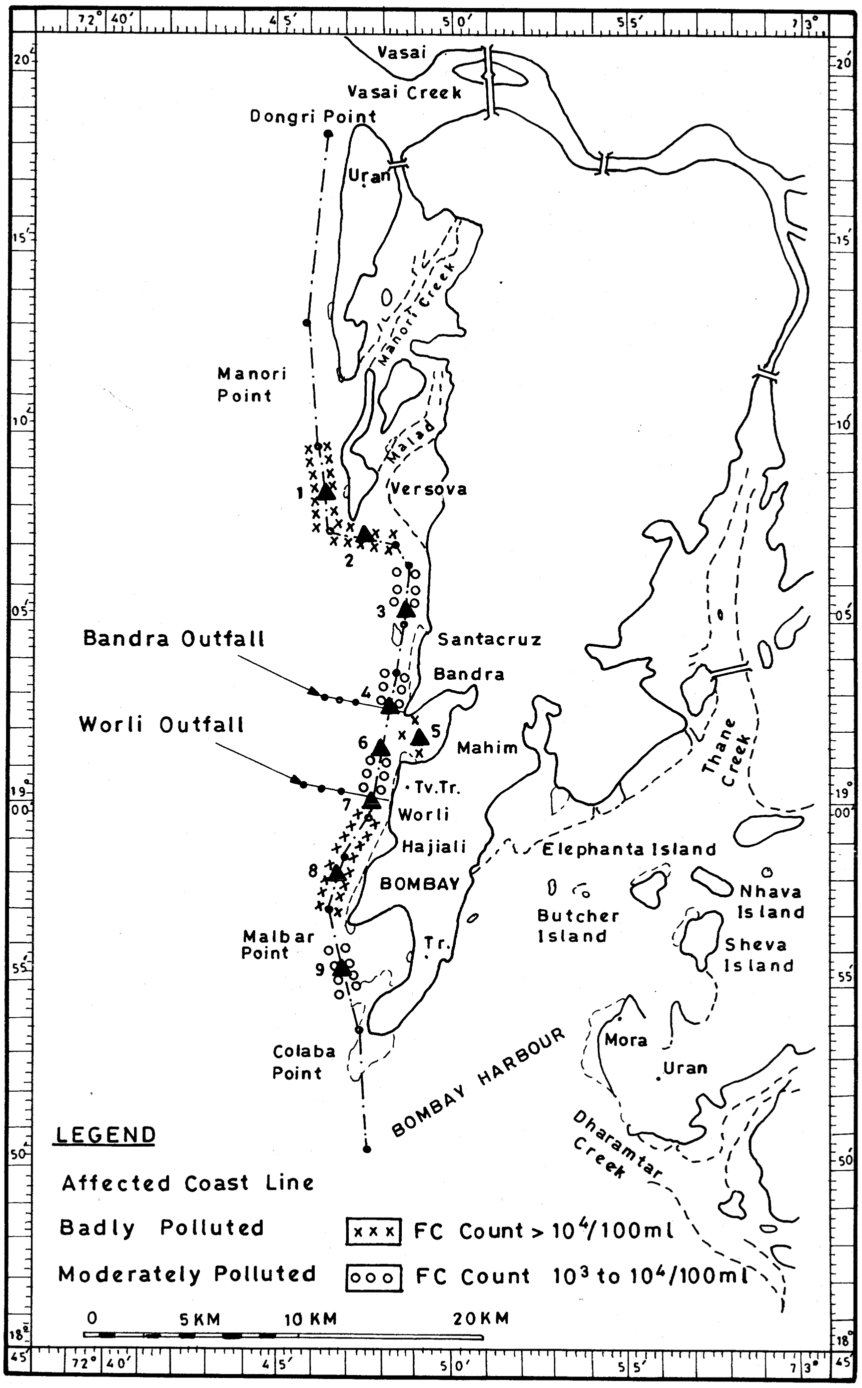

Figure 3. Present Microbial Quality at Mumbai West Coast.

solute concentration gradients or discontinuities based upon discrete solutions. In spite of these disadvantages, numerical models have been increasingly used for hydro-environmental assessment studies in the light of increasing computer speed and capacity and development of more sophisticated computational techniques.

Numerical models can simulate flow and pollutants transport processes at the prototype scale without distortion and are generally more economical, flexible, transportable and adaptable than physical models (Chen and Falconer, 1992).

3.1. GOVERNING EQUATIONS FOR HYDRODYNAMIC PROCESSES

Numerical modeling of fluid flow is based on the principles of continuity of mass and conservation of momentum within the body of the fluid to be modeled. In many cases, the flow is defined by the Reynolds equation (Reynolds, 1874), which describe the three dimensional turbulent motion of the fluid. For flows, which show little variation in the vertical mixing it is acceptable to integrate these equations over the depth of water, resulting in simplified or two-dimensional equations of motion. Such models are valid when the flow is predominantly horizontal with good vertical mixing or the vertical variations in flow are insignificant. These conditions are often met for flows within nonstratified estuaries and coastal waters, where the water depth is small in comparison with the horizontal model domain.

When integrated over depth, the equations governing fluid motion are:

Conservation of mass:

$$\partial\eta/\partial t + \partial p/\partial x + \partial q/\partial y = q_m \tag{1}$$

Conservation of momentum:

$$\partial p/\partial t + \partial\beta pU/\partial x + \partial\beta pV/\partial y = f q - gH\cdot\,\partial\eta/\partial x + \rho_a/\rho\cdot C_w\, W_x$$

$$\sqrt{W_x^2 + W_y^2} - gp\,\sqrt{p^2 + q^2}\,/\,H^2C^2 + \varepsilon[2\cdot\partial^2 p/\partial x^2 + \partial^2 p/\partial y^2 + \partial^2 q/\partial x\partial y] \tag{2}$$

$$\partial q/\partial t + \partial\beta qU/\partial x + \partial\beta qV/\partial y = -f p - gH\cdot\partial\eta/\partial y + \rho_a/\rho\cdot C_w W_y$$

$$\sqrt{W_x^2 + W_y^2} - gq\,\sqrt{p^2 + q^2}/H^2C^2 + \varepsilon[2\cdot\partial^2 q/\partial x^2 + \partial^2 q/\partial y^2 + \partial^2 p/\partial x\partial y] \tag{3}$$

where:

p (=UH), q (=VH)	Discharge per unit width in x and y directions respectively (m^3/s/m);
q_m	Source discharge per unit horizontal area ($m^3/s/m^2$);
U,V	Depth average velocity components in x and y directions (m/s);
β	Momentum correction factor for a non-uniform vertical velocity profile;
f	Coriolis parameters due to the Earth's rotation (=2ω Sin$\oslash$, with ω =angular rotation speed of the Earth and $\oslash$ = geographical angle of latitude; ω =2 π /(24 × 3600) =7.27 × 10^5 radians/s);
g	gravitational acceleration (=9.806 m/s^2);
H	Total water depth = η + h;
η	Water surface elevation above datum;
h	Water depth below datum;
ρ_a	density of air ($\cong$ 1.292kg/m^3);
ρ	density of fluid (kg/ m^3);
C	Chezy roughness coefficient ($m^{1/2}$/s);
C_w	Air /fluid resistance coefficient assumed to be 2.6x 10^{-3}; Ekman (1905)
ϵ	Depth averaged turbulent eddy viscosity (m^2/s);
x, y	Coordinates (m)
W_x, W_y	Wind velocity in x and y directions

3.2. GOVERNING EQUATIONS FOR TRANSPORT PROCESSES

When a cloud of dissolved or suspended material is released into a receiving water, the cloud will propagate, dilute and spread as it moves with the flow due to the effects of advective, diffusive and dispersive transport processes. The advection refers to the transport of the material by an imposed current system, such as that due to a tide in estuarine and coastal waters. The diffusion includes the scattering of particles by molecular and turbulent motion. The dispersion, as distinct from diffusion, is the dilution process associated with the stretching out and distortion of a cloud of solute in a non-uniform flow by the effect of velocity shear and consequential averaging of the flow distribution over the depth for two-dimensional models (Smith, 1992).

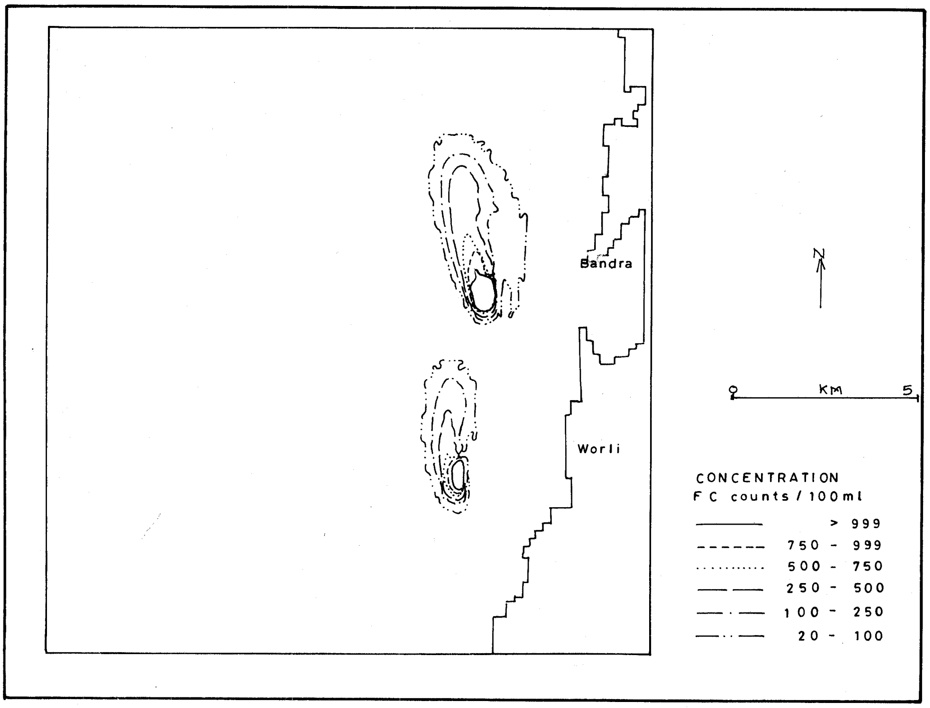

Figure 4. Fecal Coliform Counts at 3 km Worli and Bandra Outfalls (Spring tide, Primary Treatment, Minimum Initial Dilution) at 31 h.

For a horizontal or quasi horizontal flow, the three dimensional solute mass balance equation can be integrated over the water depth to give the two dimensional depth advective diffusion equation as follows (Adams, 1987).

$$\partial HS/\partial t + \partial HUS/\partial x + \partial HVS/\partial y = \partial/\partial x\,[D_{xx}\,H\cdot \partial S/\partial x + D_{xy}H\cdot \partial S/\partial y]$$

$$+ \partial/\partial y\,[D_{yx}\,H\cdot \partial S/\partial x + D_{yy}\,H\cdot \partial S/\partial y] + \Phi_s \qquad (4)$$

where:
S = depth averaged solute concentration (unit/volume) or temperature (°C)
D_{xx}, D_{xy}, D_{yx}, D_{yy} = depth averaged dispersion-diffusion coefficients in the x and y directions respectively (m^2/s), which were shown to be of the form (Preston, 1985; Holly and Usseglio-Polatera, 1984);

$$D_{xx} = (k_l p^2 + k_t q^2)\,\sqrt{g}\,/\,C\,\sqrt{p^2+q^2} \qquad (5)$$

$$D_{xy} = D_{yx} = (k_l - k_t)\,pq\,\sqrt{g}\,/\,C\,\sqrt{p^2+q^2}$$

$$D_{yy} = (k_L \cdot q^2 + k_t p^2)\,\sqrt{g}\,/\,C\,\sqrt{p^2+q^2} \qquad (6)$$

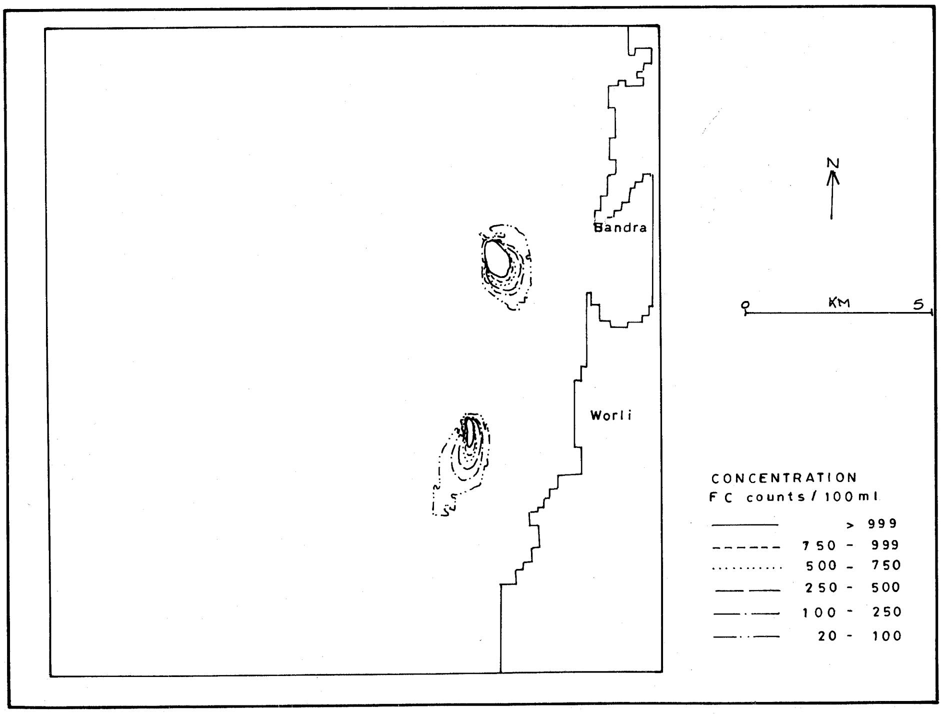

Figure 5. Fecal Coliform Counts at 3 km Worli and Bandra Outfalls (Spring tide, Primary Treatment, Maximum Initial Dilution) at 31 h.

in which k_l and k_t are the depth averaged longitudinal dispersion and lateral turbulent diffusion coefficients respectively, and have values of 5.93 for k_l and 0.23 for k_t. Φ_s = a function which may be used to represent sources, sink and decay terms, including contributions from outfalls, cooling water intakes or outlets, etc.

4. Modeling Domain

The area modeled extends over a region covering upto 15 km into the sea from Mumbai coast and 35 km along the west coast, 10 km north of Bandra and south of Worli, the site of the headwork's of the outfalls. As shown in Figure 1 the study region is bound by latitude 18°56′ on south, 19°16′30′ on north and longitude $72^0 39'$ on west and by Mumbai coast on the east. The study area was divided into three major zones, Outer, Middle and Inner each covering an area of 35 × 5 km with a grid resolution of 100 × 100 m.

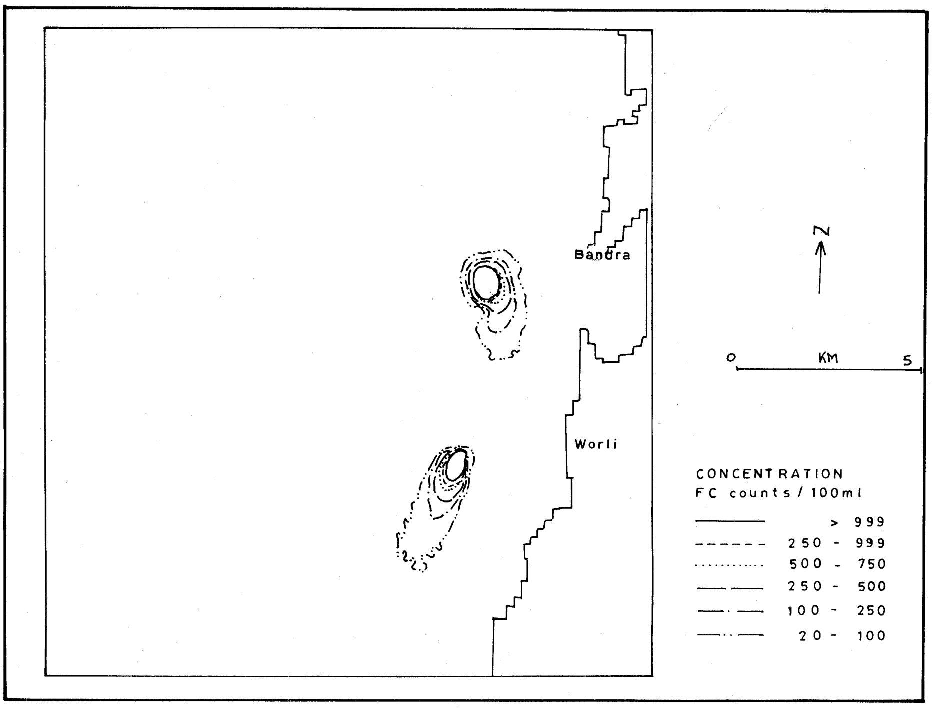

Figure 6. Fecal Coliform Counts at 3 km Worli and Bandra Outfalls (Neap tide, Primary Treatment, Minimum Initial Dilution) at 31 h.

5. Model Inputs

The finite difference method has been used to solve the governing differential equations previously given for the hydrodynamic and solute transport processes. A regular computational mesh needs to be set up, which consists of a series of grid cells and covers the modeling areas.

The modeling warranted in-depth data collection of bathymetry, ocean surface currents, depth current profile, tidal elevations and meteorology. Measurements of tidal elevations were made by automated tide gauges at the north and south boundaries of the domain. Instruments were located at strategic locations for current measurements. A die-off rate of T_{90} = 4 hs (Time taken for 90 percent die-off) was taken. The rate of discharge of sewage through the outfall after the land treatment, length of outfall considered for simulation and the level of land treatment provided are presented in Table I. The sewage sample analysis at the pumping stations of all the service areas on an average show a FC concentration of 9×10^7 per 100 ml. For primary level treatment this initial population was observed to be almost same excepting the difference in absolute number and for secondary treatment this initial population was lower by a magnitude of two. Accounting was also made for initial dilution effects. The maximum and minimum initial dilutions that were considered

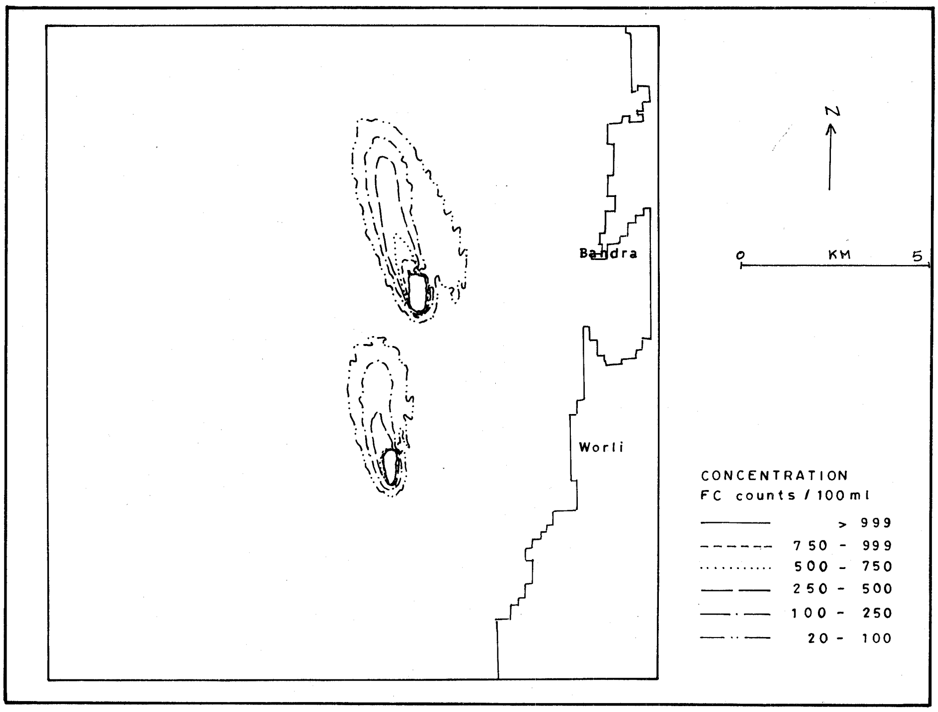

Figure 7. Fecal Coliform Counts at 5 km Worli and Bandra Outfalls (Spring tide, Primary Treatment, Minimum Initial Dilution) at 31 h.

for Worli outfall was 380 and 200 and that for Bandra outfall was 280 and 150. These dilutions were based on the wastewater discharge and the prevalent ambient current.

6. Simulation results

The lengths of the outfalls have been varied from 3 km to 8 km ith combination of primary and secondary treatment. In this paper Worli and Bandra outfalls and respective land treatment options has been discussed. The simulations were carried out for 48 h. The simulation was done for the different tidal conditions of spring and neap. Figures 4 and 5 present the model results for primary treatment with minimum and maximum dilutions. These figures indicate that for Worli a waste field patch of 1.4 kms × 0.8 km of FC concentration above 200/100 ml is at a distance of 1.2 km from shore and the same patch increases to 1.6 kms × 1.7 km at a distance of 1.2 km from shore when the minimum initial dilution value has been taken. The dilution effect does not change the waste field size of higher concentration to a significant level. As is evident from the above figures, during maximum

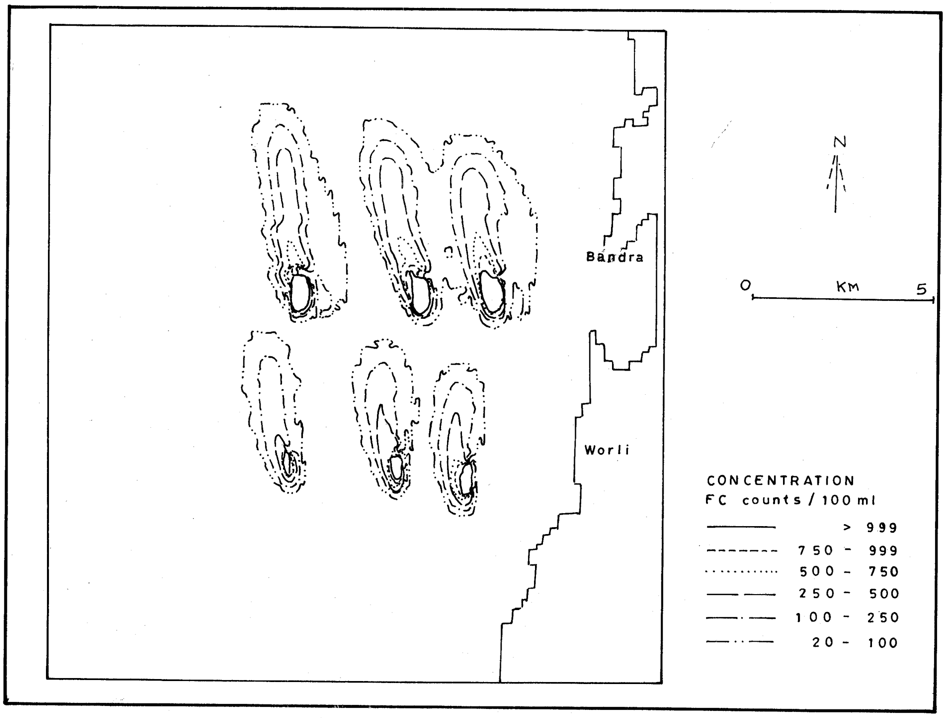

Figure 8. Fecal Coliform Counts at 3, 5 and 8 km Worli and Bandra Outfalls (Spring tide, Primary Treatment, Minimum Initial Dilution) at 31 h.

TABLE I
Scenarios considered for simulation

Stations	Outfall Length (kms.)	Discharge (mld)	Discharge (m3/s)	Treatment
Worli				
	3,4,5,6,7,8	727.5	8.42	Primary
	3,4,5,6,7,8	727.5	8.42	Secondary
	3,4,5,6,7,8	727.5	8.42	Primary
	3,4,5,6,7,8	727.5	8.42	Secondary
Bandra				
	3,4,5,6,7,8	1246	14.42	Primary
	3,4,5,6,7,8	1246	14.42	Secondary
	3,4,5,6,7,8	1246	14.42	Primary
	3,4,5,6,7,8	1246	14.42	Secondary

dilution overall patch size of FC waste field reduces. However, the middle portion of the patch with highest FC counts does not reduce significantly.

The results of these two outfalls indicate that the bacterial waste-field movement and the size and its distance from the shore vary with spring and neap tidal conditions for primary treatment. The difference between the spring and neap tide is mainly witnessed in terms of size of the waste field. The neap tide waste field is generally smaller in size as seen in Figure 6 compared to that of the spring tide options. As the length of the outfall is increased to 5 km with primary treatment, the waste field does not change in size to a significant level as is evident from Figures 7. The major change in the size of the waste field and concentration is witnessed only when the secondary treatment is provided.

The model results presented for Bandra with primary treatment indicate that a patch of 1.8 km × 1.0 km of FC concentration above 200/100 ml is found at a distance of 1.2 km from shore for maximum dilution which increases to 2.0 km × 1.2 kms in size at a distance of 1.6 km from shore when the minimum initial dilution values is taken for simulation.

A considerable decrease in the patch size takes place when the outfall length changes from 5 to 8 km as is evident from the Figure 8. This change could be attributed to the larger water column available due to major shift in the depth profile after 5 km leading to higher dilution and dispersion. The figure also indicates that the decrease in FC patch size is significant for the secondary treatment options. A secondary treatment scheme implementation at Bandra with 3 km outfall and minimum dilution would uniformly give FC concentration of less than 50/100 ml similar to Worli.

The change in the outfall length with 1 km increment leads to waste-field location shifting further away from the shore as presented in Figure 8 for primary treatment and 3, 5 and 8 km long outfalls.

7. Conclusion

The neap and spring tide results show a difference of the spread of the wastefield. The wastefield size for the neap is smaller than spring tide. With increase in the length of the outfalls, the waste field patch shifts further away from the shore by difference about the increment by which the length of the outfall increases. The patch size reduces with increase in outfall length. Secondary treatment results in major reduction in the bacterial concentration by about two order of magnitude. Even with the combination of secondary treatment and 3 km length outfall the above is achieved.

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