Modeling of Induced Circulation

LALE BALAS[1] and ASU İNAN[2]
[1]Department of Civil Engineering, Engineering Faculty
[2] Department of Construction Education, Technolgy Faculty
Gazi University
06550Ankara TURKEY
[1] lalebal@gazi.edu.tr,
http://www.mmf.gazi.edu.tr/insaat/english/academicstaff/cv/lalebalasi.htm
[2] asuinan@gazi.edu.tr, http://www.fbe.gazi.edu.tr/kazalar/English/asuinani.htm

Abstract: - Marinas located along the coastline of an enclosed sea are subjected to water quality problems due to insufficient water exchange resulting from the weakness of the tidal motion. For such marinas forced flushing measures may need to be designed. In this paper, two forced flushing schemes for enhancing flushing rates of marinas in enclosed seas are discussed. First forced flushing scheme is the removal of the surface layer of water in the marina by mechanical pumping and the second one is the wave pump. The induced circulation patterns by the use of the Morning Glory type intake structure and wave pump experiments were performed in the laboratory and forced circulations were simulated by the developed three dimensional hydrodynamic and transport model, HYDROTAM-3. Turbulence has been simulated by a two equation k-ω turbulence model. With the application of the morning glory type intake and the wave pump, the flushing ability of the marina increased about 10 times and 12 times respectively, compared to the tidal flushing alone. Model predictions provided encouraging results. HYDROTAM-3 reproduces the velocity field that is in good agreement in the intensity and spatial scale with the current measurement.

Key-Words: - Marina, forced flushing, modeling, turbulence, wave pump, mechanical pumping, finite element, hydrodynamic, transport.

1 Introduction

Water quality in estuaries, coastal lagoons, marinas and river basin is one of the important subject in environmental water problems.

Surface water as important components of the natural environment, needs to be protected from all pollutant sources because man's own survival depends on their efficient use [1]. Hydrodynamics and pollutant loads dispersion characteristics are determinant factors in an integrated river basin management [2] and in an integrated coastal management. Mathematical models are useful tools for water management practices [3]. The ecology of shallow water estuaries are influenced by freshwater inflow and the adjacent open sea due to tide or wind generated water exchange [4].

The construction of a marina disturbs the natural balance of the coastal system and normally deteriorates the water quality in and around the project site. The breakwater, purpose of which is to provide a physical barrier to waves, can become a barrier to other natural processes as well. Water enclosed in a marina basin has a restricted contact with the outside sea and water exchange is possible only through the entrance. The cross sectional area of the entrance is usually small and the exchange is low especially in areas where the tidal range is small [5]. The presence of piers and crafts complicates the situation. Water which enters into the basin, can not circulate freely. Limited water circulation may result in poor water quality levels in the marina. The most decisive factor for water quality in marinas is the flushing ability, that is, the level of water exchange with outside. It has generally been assumed that the most adverse biological effects within a marina may be prevented, if flushing is sufficient and upland drainage and other pollutants are diverted away from the marina.

The tidal motion is the main agent causing the flushing of marinas located on coastlines where the tidal range is sufficiently large [6]. There are studies performed on the optimum aspect ratio of a marina, i.e. the ratio of basin length to basin width (L/B) [7]. When there is a strong tidal forcing, one general result is that if the aspect ratio is in between 1/3 and 3, the flushing quality is the best. Dipole formation would also effect the tidal flushing mechanism [8]. The tidal ranges along the shores of Turkey which is surrounded by the enclosed seas are typically in the order of 0.2 to 0.3 meters. Such a weak tidal motion can not alone induce flushing action to maintain the water inside the marina at a reasonable clean state. In such situations, it is often necessary to apply some special design features to enhance flushing of marinas [9]. One such water quality improvement scheme for relatively small marinas is to flush out water inside the marina by a wave pump [10]. An alternative scheme is the removal of the polluted marina water from time to time by a forced outflow from an intake [11]. If the tidal forcing is not strong or there exists a broad continental shelf, meteorological forcing could be dominant. When the density gradients are significant, density currents could contribute to the flushing as well [12]. Flushing ability of marinas located along shores of lakes, enclosed seas or river inlet marinas usually is not sufficient due to very small tidal ranges. In this study, the use of a Morning-Glory spillway like structure to pump out polluted surface water from the marina and the use of wave pump to force a jet of clean water inside the marina to improve flushing performance has been investigated. Both physical and numerical models are used in the determination of induced circulation patterns and the results obtained are compared.

2 Numerical Model HYROTAM-3

The developed implicit baroclinic three dimensional hydrodynamic transport model (HYROTAM-3), is capable of computing the water levels and water particle velocity distributions in three principal directions by solving the Navier-Stokes equations [13]. The numerical model can simulate the flows induced by the density currents. The density of sea water is a function of its salt content (or salinity) and its temperature. The temperature and salinity variations are calculated by solving the three dimensional convection-diffusion equation. As the turbulence model, modified k-ω turbulence model is used. Model includes two equations for the turbulent kinetic energy k and for the specific turbulent dissipation rate or the turbulent frequency ω [14].

The turbulent kinetic energy equation is given by [15];

$$\frac{dk}{dt}=\frac{\partial}{\partial z}\left[(\sigma^{*}\nu\ )\frac{\partial k}{\partial z}\right]+P_k+F_k-\beta^{*}\varpi k \quad (1)$$

and the specific dissipation rate equation is given by;

$$\frac{d\varpi}{dt}=\frac{\partial}{\partial z}\left[(\sigma^{*}\nu\ )\frac{\partial \varpi}{\partial z}\right]+\alpha\frac{\varpi}{k}P_{\varpi}+F_{\varpi}-\beta\varpi^{2} \quad (2)$$

where horizontal diffusion terms in which q is k or ω, are;

$$F_q=\frac{\partial}{\partial x}\left[\sigma^{*}\nu\quad\frac{\partial q}{\partial x}\right]+\frac{\partial}{\partial y}\left[\sigma^{*}\nu\frac{\partial q}{\partial y}\right] \quad (3)$$

The stress production of the turbulence is defined by;

$$P=\nu\left[2\left(\frac{\partial u}{\partial x}\right)^2+2\left(\frac{\partial v}{\partial y}\right)^2+\left(\frac{\partial u}{\partial y}+\frac{\partial v}{\partial x}\right)^2\right]+\nu\left[\left(\frac{\partial u}{\partial z}\right)^2+\left(\frac{\partial v}{\partial z}\right)^2\right] \quad (4)$$

Eddy viscosity is calculated as;

$$\nu=\frac{k}{\varpi} \quad (5)$$

At high Reynolds Numbers, the constants are used as; $\alpha=5/9$, $\beta=3/40$, $\beta^*=9/100$, $\sigma=1/2$ and $\sigma^*=1/2$. Whereas at lower Reynolds numbers they are calculated as;

$$\alpha^*=\frac{1/40+R_T/6}{1+R_T/6};$$

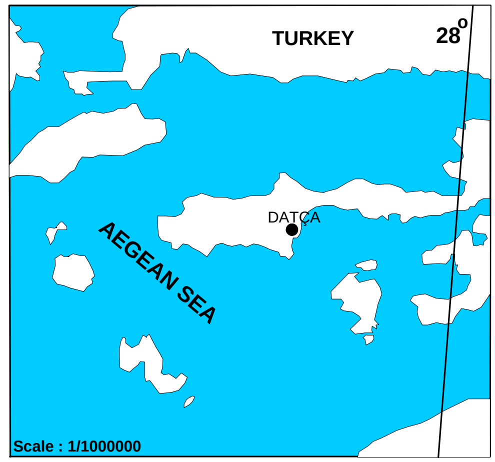
$$\alpha=\frac{5}{9}\frac{1/10+R_T/2.7}{1+R_T/2.7}(\alpha^*)^{-1};$$

$$R_T=\frac{k}{\varpi\nu};\beta^*=\frac{9}{100}\frac{5/18+(R_T/8)^4}{1+(R_T/8)^4} \quad (6)$$

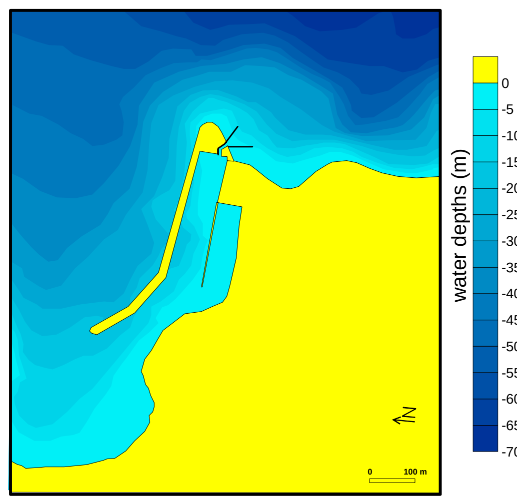
where, R_T is the Reynolds number of the turbulence. The solution method is to use a composite finite difference-finite element method [13]. Equations are solved numerically by approximating the horizontal gradient terms using a staggered finite difference scheme.

3 Induced Circulation by the Use of a Wave Pump in Datça Marina

Model has been applied to Datça Marina to predict the forced circulation patterns by the use of a wave pump. Model predictions are compared with the measurements performed in the Laboratory. Datça Marina is located 2 km south of the town of Datça and is protected by a breakwater approximately 600 meters long (Fig.1a-b). Laboratory experiments were carried on the forced flushing of the marina with the wave pump. The length scale of the marina was 1:100. The prototype length of the basin is 460 m and the width is 220 m. The water depth inside the marina changes from 1.6 m to 15.3 m in the prototype. Governing forces concerned were gravity and inertia forces whereas the effects of surface tension and viscosity were negligible. Therefore, Froude Law was the best to describe the phenomenon. According to Froude Law, Froude numbers for both model and prototype must be equal, therefore the velocity scale was calculated as 1:10.



(a)



(b)

Fig. 1. (a) Location of Datça Marina location, (b) Water depths(m) and layout of Datça Marina

The wave pump (Fig.2) is located on the main breakwater and aligned with the dominant wave direction without forming too much perturbation inside the marina. This can be achieved by a proper arrangement of the pump. Inflow caused by the pump should be surface current with a high momentum rather than waves entering the basin. The wave pump consists of two vertical walls starting from a certain depth and getting closer to each other as they reach the breakwater. The aim is to compact the wave energy at a narrower section and thus to get higher wave heights.

In the vicinity of the breakwater a ramp with a slope of 3:8 is placed at the bottom so the depth is reduced artificially. With the increasing wave height and reducing water depth, waves are forced to break on the ramp and so only the flushing effect of waves is let in the marina with a high velocity.

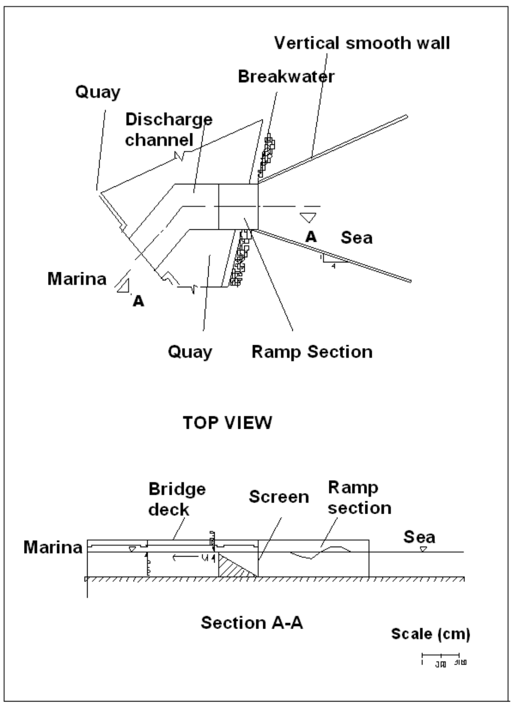


Fig.2. Details of a wave pump

Datça Marina is subjected mainly to waves from NNE, NE, ENE, E, ESE, SE, SSE and S directions. The related wave rose based on the hourly wind data of the period 1982-2006 is given in Fig.3 [16]. Waves propagating from S become ineffective due to refraction and shoaling as they reach the marina site. The topography of the marina prevents the northern winds to be effective due to very limited fetch distances although they might be considered as the dominant wind directions. Among the rest of the directions, SE is defined as the most significant wave direction. Nevertheless, due to the limited fetch distances, significant wave heights for all directions are small being less than 1.5 m. Therefore SE was decided as the direction of the wave pump. Therefore SE was decided as the direction of the wave pump. The test case for the significant wave period of 5 sec and the significant wave height of 1.5 m is presented in this paper.

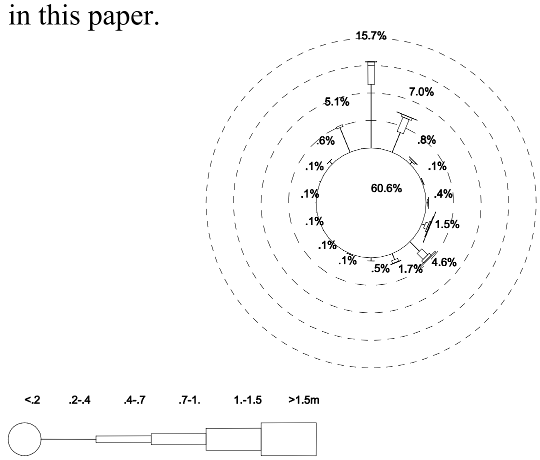


Fig.3. Datça wave rose (directional frequencies and wave heights) for the period of 1982-2006.

The paths followed by the floats in the physical model (Fig.4a) were compared with the results obtained from the numerical model (Fig.4b). The average velocities along the paths followed by the floats in both physical and numerical models are compared in Table (1).

Table 1. Comparison of the average velocities along the paths followed by the floats in physical and numerical models for the wave pump (Velocity scale = 1:10).

Wave Pump			
Path no	Velocity (cm/s)		
	Physical	Numerical	Error (%)
1	1.92	1.86	3.1
2	2.13	2.40	12.7
3	3.72	3.85	3.5
4	3.47	3.55	2.3
5	5.85	5.58	4.6
6	6.21	6.32	1.8
7	0.52	4.22	-

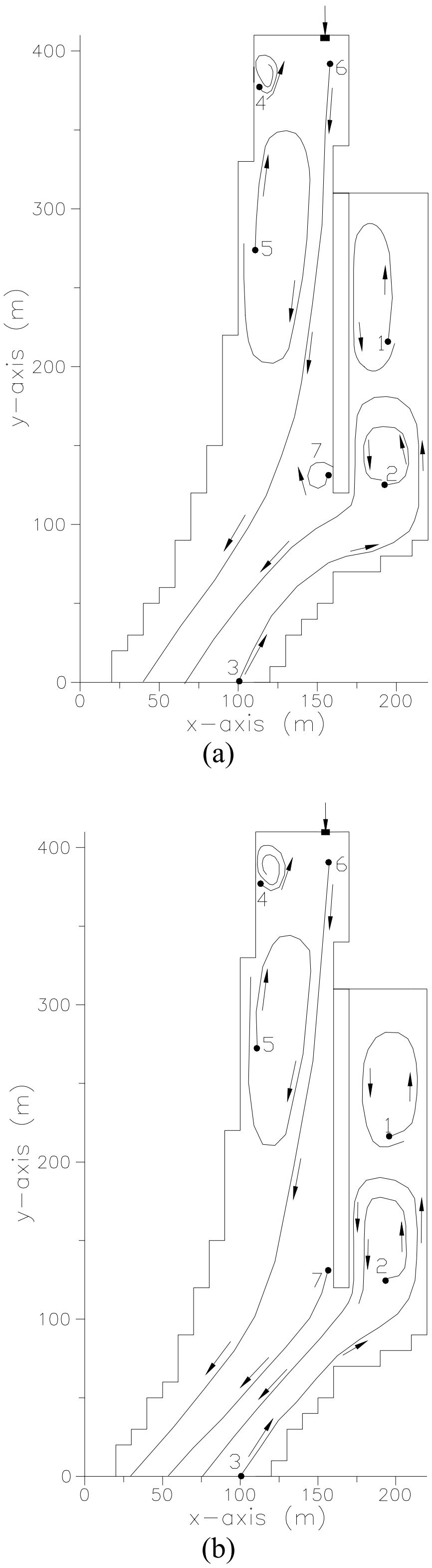


Fig.4. □: Wave pump location, a) paths followed by the floats in the physical model, b) results obtained from numerical model.

Physical and numerical model results show nearly the same circulation patterns with almost the same average velocity components. Except from the path no 7, model again provides more or less the same results. Some floats released at the mouth of the pump and catch the right flow path reach considerably high velocities. They seem to travel on almost straight paths and thus leave the basin quickly. This causes average current velocities in the range of 5 – 6.5 cm/s in prototype scale. For the float released at the end of the pier (path no 7), in the physical model there occurs a reverse gyre with a quite small velocity, almost a dead region. However such a gyre could not be simulated by the numerical model. Reason for this could be the disability of the turbulence model not to account for the probable reflection that might occur at the end section of the pier. Applied turbulence model reproduces the velocity field that is in good agreement in the intensity and spatial scale with the current measurement data.

It is obvious that water entering to the basin with a high momentum will influence the water particles on the way highly. Shear effect of flow propagating rather with a high current speed again forms shear force nearby. Shear effect causes water particles to move on closed orbits especially on the upper left corner of the basin. Whereas, a small and slow circulation may be observed at the right section. In addition to this, basin may be said to be flushed better under these circumstances. Despite the closed and round circulation pattern at upper left corner, the whole basin is almost totally flushed.

Wave pump alternative has an attraction with its being free of cost of operation. Once the structure is constructed properly, flushing mechanism is a result of natural phenomenon related with the waves. There is not any operation cost other than maintenance, which will be taken into consideration in some years period. On the other hand, the system causes extra perturbation due to the intrusion of jet flow especially in the vicinity of the pump. This will bring more agitation inside the marina and needs more attention to maneuvering and mooring inside the marina. In addition to these, since the energy utilized by the wave pump is the energy of the wind waves present at the marina site, the degree of its contribution to the flushing totally depends on the existence of the waves. The pollution of the marina is expected to be the severest during summer months. So, small number of occurrence of waves during summer months may cause wave pump not to be very effective for flushing.

4 Induced Circulation by the Use of a Morning Glory Type Intake Structure in Datça Marina

HYDROTAM-3 was applied to Datça Marina to predict the induced circulation patterns by the use of a Morning Glory type intake structure and by the use of a wave pump. Model predictions were compared with the measurements performed in the Laboratory explained above. Laboratory experiments were carried on the forced flushing of the marina with a Morning Glory type intake structure. The surface water was withdrawn by installing a structure similar to a morning-glory spillway. In its application to a dam, the morning-glory requires energy to discharge the excess water. This energy is obtained from the potential energy released by elevation loss. Such a source is not usually present in a marina application. Therefore a pump is required to acquire the driving energy needed. The structure consists of a vertically placed conical shaft connected to a pump by a horizontal discharge pipe placed on the sea bed (Fig.5). Inside the intake structure, two sheet plates are placed crossing each other in order to prevent vorticity. The water taken from the basin was pumped into the open sea. The water pump was ‘Standard SLG2 centrifugal type with 4HP power and 65 % efficiency. Discharge capacity of the pump was 2.5 m^3 / hr. Surface water at the marina was pumped to the open sea. This initiates the movement of nearby water particles through the marina. The selection of discharge point is of vital importance. It should be selected such that the perturbation caused by pumped water would not affect the circulation in the basin.

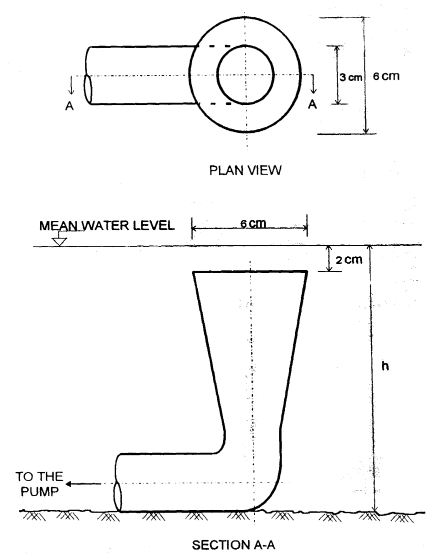
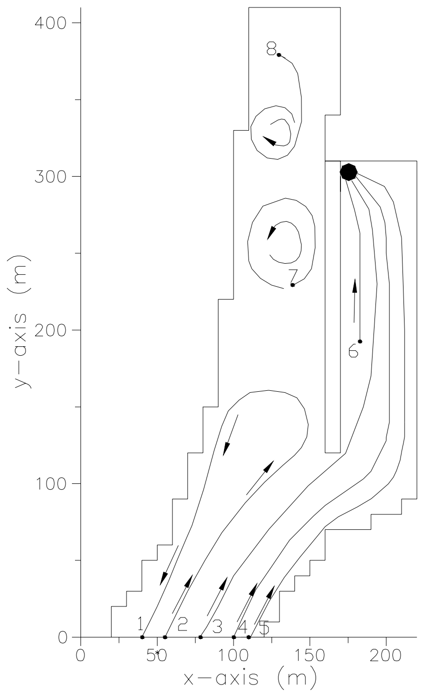


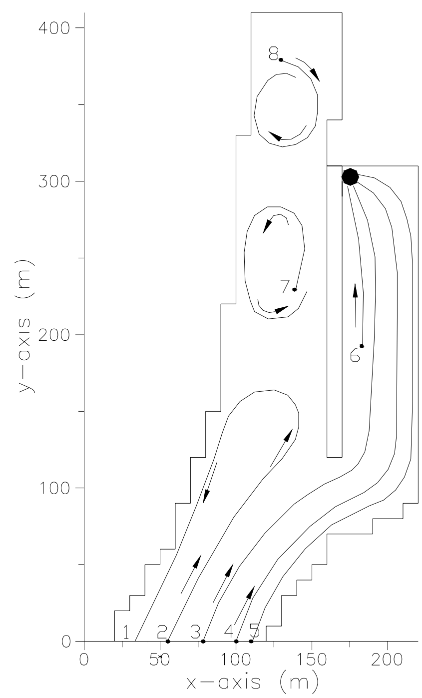
Fig.5. Morning-glory spillway like intake structure.

Velocity measurements were taken with a 'Minilab SD-12' microscale 3-axis ultrasonic current meter. Surface velocity patterns, were observed by using mini floats. The eddy viscosities were calculated by the k-ω model. The water depth was divided into 6 layers of equal thickness. The density of water was assumed to be constant and equal to 1025 kg/m^3. At t=0, the pump was started, so that the water began to flow in the intake, whereas the remaining part of the water body was assumed to be at rest and the water surface was assumed to be horizontal. The land boundaries were taken as fixed boundaries. The grid system used had a square mesh size of 10x10 m. Velocities measured in the physical model at the grid points neighboring the intake were used as the boundary conditions of the intake location in the mathematical model. The morning intake was placed at the center of two predetermined grid points. Selection of the location of the intake structure depends on providing almost no dead regions inside the marina when the system operates. Another criteria for the selection may be the effective use of the marina (i.e. to use the possible minimum area for the utilization of the intake structure to free more space for the users). Considering these, intake structure was placed at two grid points. One of the locations was at the uppermost left corner on the right of the basin that was separated by the pier. The other one was also located on the right but it was a little bit downwards and can be said to be in the middle. It is important to locate the Morning-Glory intake structure at the corners not to cause any difficulty for manoeuvring inside the marina. Current velocities at surface layer were measured using floats as drogues. Floats were round (less than 1 cm in diameter), white foams in order to minimize the friction forces and surface tension. The paths followed by the floats in the physical model (Figure 6a) were compared with the results obtained from the numerical model (Figures 6b). The average velocities along the paths followed by the floats in both physical and numerical models were compared in Table (2). Morning-Glory Alternative I (MG1), being at the upper part of the right basin, seems to produce a circulation mainly at the right section of the entrance, which finds its way through the sink whereas, flow entering the basin from left part of the marina entrance is prevented by the pier. Left section is mainly affected by the shear. The general pattern in the left section is two regimes turning around oppositely on their own. This means no flushing of the marina for this section is possible caused by the effect of Morning-Glory on Location I.

Morning-Glory Alternative II (MG2), is selected as the middle of the right basin. The paths followed by the floats in the physical model (Figure 7a) were compared with the results obtained from the numerical model (Figure 7b). The average velocities along the paths followed by the floats in both physical and numerical models were compared in Table (2).

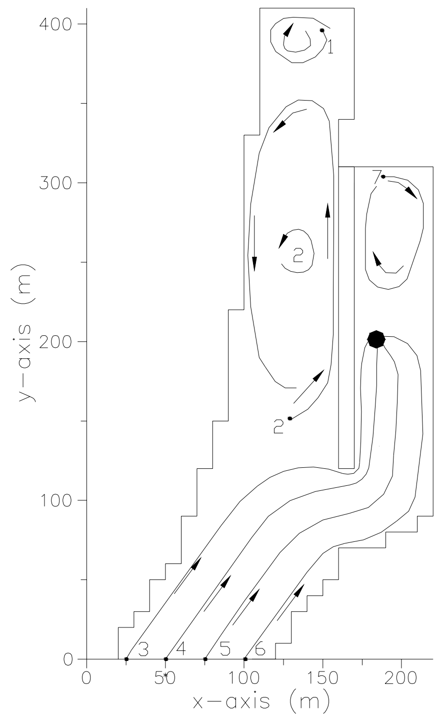


(a)

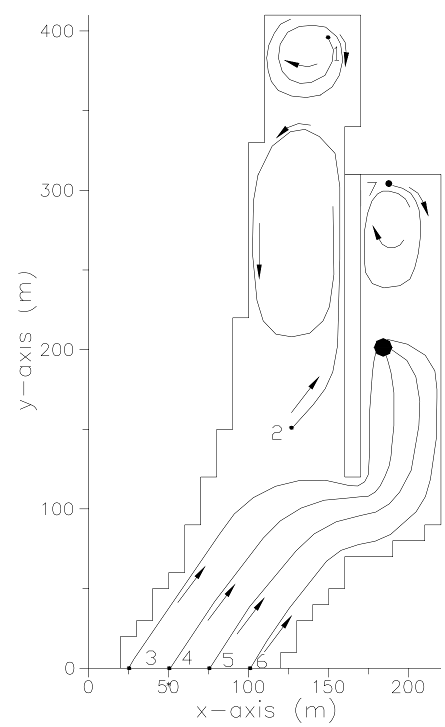


(b)

Figure 6. Morning Glory location I, a) Paths followed by the floats in the physical model, b) Results obtained from numerical model



(a)



(b)

Fig.7. Morning glory location II, a) Paths followed by the floats in the physical model, b) Results obtained from numerical model.

Location II for Morning-Glory more or less resembles the flow patterns of the first location. This time, water around the entrance section and portions of the marina on the way to sink is totally forced to reach the sink. Again due to the shear effects, at the right basin above the sink, a round, closed circulation is formed. It is hard to say that Alternative II has a proper effect on the flushing of left section of the marina.

Table 2. Comparison of the average velocities along the paths followed by the floats in physical and numerical models for the morning glory (Velocity scale = 1:10, P:Physical, N:Numerical, E:Error).

Morning Glory Location I				Morning Glory LocationII			
Path no	Velocity (cm/s)			Path no	Velocity (cm/s)		
	P	N	E(%)		P	N	E (%)
1	0.57	0.66	15.8	1	0.24	0.26	8.3
2	0.62	0.71	14.5	2	0.42	0.49	16.7
3	1.75	1.79	2.3	3	0.91	1.05	15.4
4	1.42	1.40	1.4	4	1.87	1.92	2.7
5	0.93	0.87	6.5	5	1.81	1.79	1.1
6	1.82	1.93	6.1	6	1.0	0.94	6.0
7	0.33	0.35	6.1	7	0.31	0.26	16.1
8	0.21	0.19	9.5				

Morning-Glory alternatives are mainly affected by the pier. Wider intervals were observed at the paths in the vicinity of the intake and the pier entrance. The amount of discharge equal to the pumped out, flows back through the entrance. As the discharge is forced through the narrow entrance of the section lying just at the end of the pier, velocities are observed to be greater. Passing through the entrance area increases and the discharge is distributed along the whole basin cross-section, hence the velocities decrease. Especially in the sections where the intake has no or limited effect, velocities are too small.

The morning–glory alternative will provide rather a calmer marina area. The current velocities are calmer excluding the nearby sections of the intake structure and portions of the marina where the section gets narrower. The system has both construction and operational costs. It is only possible to pump water out by the mechanical help of a water pump. Moreover, the periodical cleaning of solid wastes on the surface will be much easier inside the marina where these pollutants are forced to move towards the sink. The collection of such particles will then be handled in a more limited area. The morning–glory intake alternative is a more controllable system compared with the wave pump.

The physical and numerical model results show nearly the same circulation patterns with almost the same average velocity components. The applied turbulence model reproduces the velocity field that is in good agreement in the intensity and spatial scale with the current measurement data. Especially for Morning–Glory alternatives, model gives more or less the same results. For both of the MG1 and MG2 alternatives, current velocities obtained from the physical and numerical models vary in the range of 1-17 %. For the wave pump, except from the path no 7, model again provides more or less the same results. Some floats released at the mouth of the pump and catch the right flow path reach considerably high velocities. They seem to travel on almost straight paths and thus leave the basin quickly. This causes average current velocities in the range of 5 – 6.5 cm/s in prototype scale. For the float released at the end of the pier (path no 7), in the physical model there occurs a reverse gyre with a quite small velocity, almost a dead region. However such a gyre could not be simulated by the numerical model. Reason for this could be the disability of the turbulence model not to account for the probable reflection that might occur at the end section of the pier.

5 Tidal Flushing

The dominant tidal constituent for the area is M2 tide. Along the Turkish coastline, the tidal ranges are small, typically in the order of 0.2 to 0.3 meters. The tidal flushing ability of the marina is examined by a one dimensional flushing model.

A no conservative substance with a first order decay reaction is considered. Its concentration is C_o at the start of the computations. The intrusion of the pollutant into the coastal water body continues at a constant rate P. As the model is one dimensional, the pollutant concentration is assumed not to vary spatially. This requires complete mixing inside the water body at all times. The model equation, stating the conservation of the pollutant mass in the enclosed water body is written as [5]:

$$\frac{dC}{dt} = -(k+\frac{Q}{V})C + \frac{P}{V} \quad (7)$$

in which, C: the instantaneous pollutant concentration; k: decay coefficient; Q: entering discharge; V: water volume inside the marina.

When the flushing discharge is due to the tidal action, the bulk conservation equation becomes:

$$\frac{dC}{dt} = -kC + \frac{P}{V} \text{ for } nT \le t \le (n+\frac{1}{2})T \ldots (\text{ebb tide}) \quad (8)$$

$$\frac{dC}{dt} = -(k+\frac{Q}{V})C + \frac{P}{V} \text{ for } (n+\frac{1}{2})T \le t \le (n+1)T \ldots (\text{flood tide}) \quad (9)$$

in which, T: tidal period; n: a positive integer ("zero" included) and t=0,T,2T,…, are the times of mean high tide level (i.e. the onset of the ebb tide). The timely variations of the tidal discharge Q and the water volume V are used as:

$$Q = -\frac{1}{2}A_S R w \sin(wt) \quad (10)$$

$$V = V_S + \frac{1}{2}A_S R \cos(wt) \quad (11)$$

where, A_s: Surface area at mean sea level; V_s: Mean tide level volume; R: Tidal range (from mean low level to mean high level) and w:$2\pi/T$.

The solution giving the pollutant transport concentration at the time of high tide after the n th tidal cycle is:

$$C_n = a^n C_o + \frac{b(1-a^n)}{1-a} C_a \tag{12}$$

in which,

$$a=\frac{M-1}{M+1}e^{-kT}$$

(13)

$$b=\frac{2}{kT}\left[\frac{M}{M+1}(1-e^{-k\frac{T}{2}})-a(1-\frac{k^2}{M(k^2+w^2)})+ae^{k\frac{T}{2}}(1+\frac{k^2}{M(k^2+w^2)})\right] \tag{14}$$

$$M=\frac{2V_s}{A_s R}=2\frac{\bar{h}}{R} \quad ; \quad C_a=\frac{PT}{2V_s} \tag{15}$$

where, h: mean water depth, M: flushing parameter. If the substance is conservative then k=0 and equation for C_n still holds if the parameters a and b are redefined as:

$$a=\frac{M-1}{M+1} \quad ; \quad b=\frac{M(1+\frac{M-1}{M+1})}{M+1} \tag{16}$$

The one dimensional model presented above is applied to Datça Marina. The model provides a quick assessment of the degree of tidal flushing to be expected. The mean depth of the Marina is 5 m. and the surface area is about 101000 m^2. The tidal motion at the site is semi-diurnal type. The mean tidal range is about 0.2 m. The flushing parameter has a value of 50. The C_n/C_o ratios are computed for a conservative pollutant and results are plotted in Fig. 8.

Fig.8 depicts the situation that if no pollutant is introduced into the marina (i.e. for Ca/Co=0), it takes 116 days that require 219 tidal cycles for the pollutant amount in the marina to be flushed out by the tidal currents alone up to level of 99 %, i.e. to have the value of Cn/Co=1%. If the pollutant continues entering the marina waters, the flushing period of the marina gets much longer. Furthermore, it is computed that Cn=Co for all times if the pollutant addition rate is such that Ca/Co=0.01.

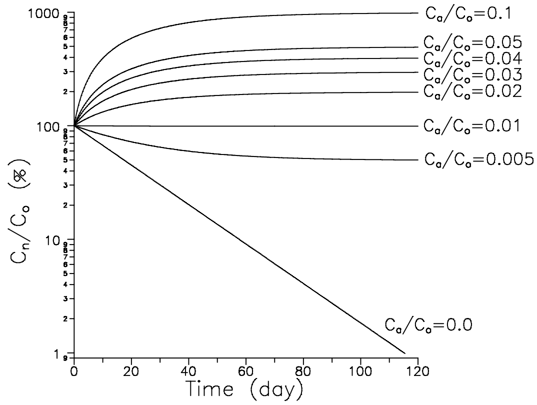


Fig. 8. Changes of pollutant concentration with time for various rates of pollutant addition.

As it is also observed from Fig.8 the cleansing of the water body can not be realized at all if Ca/Co>0.01. These indicate that flushing of the marina by the tidal motion alone is far from being sufficient for self cleansing. With the application of the morning glory type intake and the wave pump, the flushing ability of the marina increases about 10 times and 12 times respectively, as a result of the increase in the exchanged volume of the water body.

6 Conclusions

Forced flushing of a marina is investigated by both physical and numerical model studies. The periodical removal of surface water from a marina by pumping through a morning-glory type intake structure may be a feasible solution to enhance flushing of the basin as a remedy against water pollution. With the application of the morning glory type intake, the flushing ability of the marina increases about 10 times compared to the tidal flushing alone. The pumping operation should be carried out during the flood tide only to derive the greatest efficiency from such a system. The best location of the intake for the removal of water and the optimum discharge rate are two crucial issues for designing such a scheme. The first question becomes especially important in the case of complex basins where the problem of "dead regions" is normally more significant. Such features have traditionally been investigated in physical models. As a result, flushing of marinas using special structures like "Morning-Glory" and "Wave Pump" can be said to be effective ways for forced flushing of Datça Marina. It is for sure that utilization of any of the alternatives in Datça Marina will result in a better water quality.

The three dimensional hydrodynamic transport model (HYDROTAM-3) presented in this paper is shown to be capable of predicting forced circulation patterns and the level of flushing enhancement with very reasonable accuracy. The model which can serve as a powerful design tool, may also be used to predict the natural flushing rates caused by tidal motion, wind effect, a water inflow due to longshore currents driven by breaking waves, and fresh water inflow in the case of a river marina.

References:

[1] Duarte, A.A.L.S., Vieria, J.M.P., 2009, Estuarine hydrodynamics as a key-parameter to control eutrophication process, *WSEAS Transactions on Fluid Mechanics*, Issue 4, Vol. 4, pp. 137-147

[2] Celikyay, S., Cengiz, B., 2006, Ecological Planning Strategies on Bartin River Region, *WSEAS Transactions on Environment and Development*, Issue 2, Vol. 8, pp. 1091-1096

[3] Duarte, A.A.L.S., Boaventura, R.A.R., 2008, Pollutant dispersion modelling for Portuegese river water uses protection linked to tracer dye experimental data, *WSEAS Transactions on Environment and Development,* Issue 4, Vol. 12, pp. 1047-1056

[4] Duarte, A.A.L.S., Vieria, J.M.P., 2009, Effect of tidal regime on estuarine residence time spatial variation, Proceedings of the 5th International Conference on Energy, Environment, Ecosystems, and Sustainable Development, WSEAS Press, Athens, Greece

[5] Özhan, E. and Balas, L., 2003, Simulation of Water Exchange in Enclosed Water Bodies, *Lecture Notes in Computer Science*, Vol 2658,2003, pp:195-204.

[6] Sannino, G, Bargagli A, Artale V., 2004, Numerical modeling of the semidiurnal tidal exchange through the Strait of Gibraltar, *Journal of Geophysical Research.*,Vol.109,2004, doi:10.1029/2003JC002057.

[7] Falconer, R. A. and Nece, R. E., Modelling Tidal Flows and Flushing in Marinas, *Marina Developments* (Ed. W. R. Blain), Computational Mechanics Publications, Southampton, Chapter 5, 1993, pp.103-142.

[8] Wells, M.G. and van Heijst, G.-J.F. A model of tidal flushing of an estuary by dipole formation, *Dynamics of Atmospheres and Oceans*, Vol.37, 2003, pp.223-244.

[9] Özhan, E., Water Quality Improvement Measures for Marinas Subjected to Weak Tidal Motion, *Proc.Second Int.Conf.on Coastal and Port Engineering in Developing Countries, COPEDEC 3*, Mombasa, Kenya, Vol.2, 1991, pp. 1337-1350.

[10] Bruun, P and G Viggoson, The Wave Pump, *Journal of Waterway, Harbours and Coastal Eng Div, ASCE*, Vol 103, No WW4, 1977, pp 449-469.

[11] Özhan, E., Balas-Hapoğlu, L. and Başaran, S.A., Forced Flushing of Marinas in Enclosed Seas, *BORDOMER'97, International Conference on Coastal Environment and Conservation*, Vol. 2, 1997, pp. 65-70.

[12] Debler, W. and Imberger, J. Flushing criteria in estuarine and laboratory experiments, *ASCE Journal of Hydraulic Engineering*, 122:12, 1996, pp 728-734.

[13] Balas, L. and Özhan E., An Implicit Three Dimensional Numerical Model to Simulate Transport Processes in Coastal Water Bodies, *International Journal for Numerical Methods in Fluids*, Vol.34, 2000, pp: 307-339.

[14] Neary, V.S., Sotiropoulos, F., Odgaard, A.J., Three Dimensional Numerical

Model of Lateral Intake Inflows. *Journal of Hyraulic Engineering,* Vol.125, 1999,pp: 126-140.

[15] Balas, L. and İnan, A. Three Dimensional Modelling of Turbulence, *Lecture Series on Computer and Computational Sciences*, Vol 4, 2005, pp:48-55.

[16] Öztürk, C., Hydraulic and Numerical Modelling of Water Circulation in Datça Marina caused by a Pumped Discharge and a Wave Pump, *MSc Thesis*, Institute of Science and Technology, Middle East Technical University, 1999, 84 pg.