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Modeling and control of pumping stations and equalization basins

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Rice University, 1992
RICE UNIVERSITY

MODELING AND CONTROL OF PUMPING STATIONS
AND EQUALIZATION BASINS

by

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A THESIS SUBMITTED
IN PARTIAL FULFILLMENT OF THE
REQUIREMENTS FOR THE DEGREE
MASTER OF SCIENCE

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MODELING AND CONTROL OF
PUMPING STATIONS AND EQUALIZATION BASINS
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ABSTRACT

Diurnal variations of flow rate and organic load cause difficulties in the operation of wastewater treatment plants. The man-made hydraulic shocks produced by the installation of fixed speed pumps upstream of the treatment processes further amplify the frequency and magnitude of these variations. The major objective of this study is to develop control strategies to minimize these variations.

A dynamic model has been developed for the operation of fixed speed pumping stations. The optimum control strategy based on this model can reduce the on-off pump cycles up to 86% for a typical day as compared with ordinary two point control. Optimum models and corresponding control algorithms have also been developed for the operation of in-line equalization basins. These models have shown potential for the reduction of the required equalization volume and can provide smoother outflow to the downstream processes. A reduction of tank volume by 63.5% has been obtained in the simulation using the optimum control strategy.
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Chapter 1
INTRODUCTION

Diurnal variations of influent flow rate and load (COD, BOD, SS, etc.) usually cause difficulties both in the design and operation of wastewater treatment plants. In addition, some man-made hydraulic shocks are produced by the operation of pumps (35), which further increases the frequency and magnitude of the diurnal variations. This is especially true for treatment plants with a fixed speed pump installation upstream of the treatment processes.

Peak flow rates and loads only occur for a few hours during a day. However, the treatment plant must have the capacity to handle these peaks, resulting in higher capital costs compared with the handling of the mean daily flow rate and load rate. Diurnal variations of flow rates and loads, as well as man-made hydraulic surges, can also have a deleterious effect on the subsequent treatment processes in the operation.

Since the reduction of diurnal variations and man-made hydraulic surges can significantly reduce the capital and operational costs, and prevent treatment process failure, much research has been devoted to smoothing the influent flow rate and organic load by pre-treatment processes.

Based on the problems mentioned above, this investigation concentrates on the reduction of diurnal
variations of influent flow and organic loading rates using equalization basins, and on the minimization of man-made hydraulic shocks by developing an optimum control strategy for the operation of fixed speed pumping stations.

1.1 Objectives

The objectives for this investigation are summarized as follows:

1) To explore the interactions between the design and control of fixed speed pumping stations and equalization basins.

2) To develop better control strategies for fixed speed pumping stations and for equalization basins;

3) To develop a software package using commercially available programs to be used by practitioners as an off-line simulation tool for the control of fixed speed pumping stations.

1.2 Thesis Contents

The thesis structure is shown in Fig.1.1. There are four parts in this thesis. Chapter 2 deals with the modeling of influent flow rate using time series (ARIMA) models and the selection of identified models. The final selected model for influent flow rate is applied in Chapter 3 and 4 to develop the control strategies. The dynamic simulation and control of fixed speed pumping stations using SIMNON, a
simulation language, are carried out in Chapter 3, where both conventional two point control and optimum control are simulated and compared. A dynamic model and the corresponding algorithm developed for optimum control of the equalization process are presented in Chapter 4. The interactions between the design and control of equalization basins are also analyzed there. A software package for the simulation of fixed speed pumps is briefly introduced in Chapter 5. The detailed computer programs and the menu system for this package are listed in Appendix A.
Fig. 1.1 Structure of This Thesis
Chapter 2  
INFLUENT FLOW RATE MODELING

Influent flow rate is one of the most important measurements in wastewater treatment plant operation and strongly influences the control strategies of operation. Significant variation in the influent flow and COD load depends on the sewer network connected to the plant. Each plant has its own influent flow and organic load pattern during a period of a week. Prediction of the influent flow rate pattern is beneficial to both the design and operation of wastewater treatment plants.

ARIMA models are powerful tools for modeling time series (especially for univariate time series) and for forecasting over short time periods. Well fitted models and accurate forecasts may be obtained by properly selected the models from the ARIMA model family.

The data was given by Vitasovic (30) for the Interbay Pumping Station, Seattle, during the dates of February 23 through March 8, 1990. Only univariate series models were considered.

2.1 Literature Review

The reasons for determining the influent flow and load pattern may be summarized as (a) to obtain predictions of flow rate and pollutant concentration by using both past
and present values of measurements, (b) to develop models for the system and (c) to design control strategies for the system.

There are two kinds of time series models that are frequently used to model the flow rate and concentration. One is based on the Fourier series and the other on the Box-Jenkins technique. Which model is adopted depends on the length of the available data, the fitting of models to the data, the purpose of the prediction model, and the characteristics of the data.

Stenstrom (1) used a finite Fourier series and field data to simulate an influent flow pattern. He superimposed a random noise component on this pattern in order to test plant control strategies. Computer simulation indicated that the use of predicted influent flow rate decreased the variability of the specific oxygen utilization rate (SCOUR) by 48% as compared to a control strategy without the flow prediction.

One of the first efforts to obtain predictions of influent flow rate was that of LaGrega (2). They used a seasonal ARIMA model to forecast average hourly flows to a conventional activated sludge process. A similar approach had been taken by Beck (3) in his study on the identification and adaptive prediction of urban sewer flows. There are examples of the application of ARIMA models to
establish models in the study by Berthouex and co-workers (4).

Vitasovic (5) used the Box-Jenkins method for forecasting influent flow rate as an input to the treatment system. Only univariate models were considered in his study. In order to obtain the best fit to his model he averaged data for each hour, with the most recent 240 hours being used to obtain average flows and the perturbations from the average flow pattern.

Dold (6) employed a simple technique for obtaining predictions of influent flow rates. He averaged flows for each time of day and this average flow was updated by new measurements. Based on the previous average value at time \( t \) and the measured flow at time \( t \) the new average flow rate at time \( t \) was obtained. Although the method is not as sophisticated as the Box-Jenkins technique, Dold has applied this approach at full scale with resulting improvements in plant operation.

2.2 Data Characteristics and Pre-Treatment

The raw data (two weeks) was plotted and is shown in Fig.2.1. By carefully looking at the plot it is obvious that there are several characteristics of the series as summarized below:

1> There are several storms which occurred during the measuring period, and which may significantly influence the
final form of the model. The purpose of this study is to model the dry weather flow rate to the treatment plant. Therefore, data pretreatment was necessary.

The heavy storms began on March 6, so the series is shortened to between February 23 and March 6 to eliminate the wet weather flows. There are still two storms included in the new series (ST1): March 1 and 5 (each lasting 2 to 3 hours). See Fig. 2.2.

On the basis of the new time series data (ST1) the two storms were picked out and the other extreme values at the valley of the flow rate in Fig.2.2 were also eliminated artificially. Therefore, series ST2 was formed with the storm flows substracted (Fig.2.3). During the model building
Fig. 2.2 INFLUENT FLOWRATE (ST1)
Without Heavy Storm Data

Fig. 2.3 INFLUENT FLOWRATE (ST2)
By Artificially Pick Out The Outliers
process ST2 was transformed into logarithm form in order to be able to fit ARIMA models to the data. This new series was designated ST3.

By carefully observing Fig.2.1 we can be sure that the two peak values on March 1 and 5 are storms, but we can not be sure that the valley values on Feb.28, March 1 and 5 are abnormal. They may be diurnal fluctuations although there is not enough data to prove this. The logarithm and square root transforms were applied to series ST1 to sharply reduce the influence from those outliers, forming series ST5 and ST6 respectively.

2> From Fig.2.1, there is a trend for the flow rate during the weekend to be less than those during the weekdays. This phenomenon has been observed by many researchers. To compensate, time series models may be built separately. In this study, because of the two week data available, an attempt was made to use one model to describe whole week periods.

3> It is obvious, according to Fig.2.1, that the flow rate has a periodical change within 24 hours, but that each one day cycle has a different mean value from the other days. It should also be noted that there is no constant variance and mean from each hour’s data. Therefore, this is a non-stationary seasonal time series.
2.3 Data Analysis

Pankratz (12) has suggested the importance of raw data analysis before any model building processes are carried out in order to get good quality models. The flow rate data ST1 was taken as an example to illustrate the modeling processes by the use of autocorrelation function (ACF) plots, partial autocorrelation function (PACF) plots, seasonal subseries plot and estimated cross-correlation plots of the original data with the residual after differencing. Statgraphics, a statistical software package, (46) is used for modeling the influent flow rate.

2.3.1 Autocorrelations

The ACF plot of the original series with a 24 hour time lag is shown in Fig.2.4. In this plot, the heights of the bars represent the estimated correlation coefficients $r_k$; the dashed lines are at zero plus and minus twice the large lag standard errors for each coefficient, which approximately represent the t critical value for each coefficient. From this plot three points can be gained: the coefficient bars go to zero after a 5 hour lag instead of cutting off to zero at one hour lag, which implies that the series may be a non-stationary series, and that 1 order of differencing in the ARIMA model may be needed (but after the first order of seasonal differencing, we will see that these non-seasonal characteristics of the series may not be very
strong). The plot also reveals that the flow rate does have a 24 hour periodicity, since the coefficient bars go from positive maximum to negative and then to the positive again at a 24 hour lag. Therefore the ARIMA seasonal models should be taken into account in modeling. The first three autocorrelation coefficients, at time lag 1, 2, and 3, have t-statistic values of 14.77, 7.13, and 4.23 respectively. These are much higher than the dashed line, which demonstrates that the coefficients are significantly different from zero with a 95% level of confidence, so at least 1 order of MA(1) or seasonal MA(1) is needed.
2.3.2 Partial Autocorrelations

The estimated PACF plot with 24 hour time lags is shown in Fig.2.5. There is a high bar at time lag 1 with a t-statistic value of 14.77 on the positive side, and another high bar at the lag 2 with a t-statistic value of 7.15 on the negative side of the plot. This shows that the flow rate at 1 and 2 hour lags (or a 24 hour lag for one day period) is highly correlated with the previous flow rate. The evidence also reveals that 1 order of autoregressive model is needed with a positive sign of φ₁, AR(1).

Fig.2.5 Estimated PACF for Flowrate
For Original Series ST1
2.3.3 Seasonal Subseries Plots

Strictly speaking, the Box-Jenkins ARIMA method applies only to stationary data series; that is, a series with a mean, variance, and autocorrelation function that are essentially constant through time. Based on the plots of the ACF and PACF analysis it was concluded that the flow rate data for the Interbay Pump Station is non-stationary both in hour to hour and 24 hour periods. Further evidence was acquired from the seasonal subseries plot in Fig.2.6 (for ST1). It also showed a periodical change in the hourly average flow rate over 24 hours, and no constant mean values were observed. There are several big residuals in the morning from 5 to 9 am, which were caused by the deep valleys on March 1, 2, and 5, and in the afternoon from 12 to 3 pm, which may have been caused by two storms on March 1 and 5. A first attempt was made to difference once for a non-seasonal process. The residuals for this differencing show that there is still no constant mean and variance, so 1 seasonal differencing was applied to the residual series. A subseries plot for this residual was made in Fig.2.7. It is obvious that the seasonality has been removed, and an almost constant mean was achieved. The t-test for the residual was carried out on the null hypothesis (H_0) of mean equal to zero. The computed t-statistic is 0.1244. The H_0 can not be rejected with a 95% level of confidence and 256 degrees of freedom.
As discussed above, 1 seasonal and 1 non-seasonal differencing were applied to time series ST1. This made the model complicated. Another attempt was made to only give 1 seasonal differencing to the series (ST1). Fig.2.8 is the residual subseries plot. Comparing Fig.2.8 with Fig.2.7, we find that although the former may not be as stationary as the latter, it is much closer to a stationary series, and that the model is much simpler with only 1 seasonal differencing. The t-test showed that the residual after 1 seasonal differencing has a mean=0 with 95% confidence level.

2.3.4 Cross-Correlations

Basically the Box-Jenkins ARIMA model contains two parts: a deterministic component and a random process (called noise). A good model should have a residual that does not correlate with the original series. The cross-correlation function was used to calculate the relationship between the original series and the residual after one seasonal and non-seasonal differencing. The results are plotted in Fig.2.9., in which the residual flow rate is strongly correlated with the original data both negatively and positively. This suggests that further Moving Average (MA) and Autoregressive (AR) models are needed to remove the deterministic component contained in the residual.
2.4 Models for Influent Flow Rate

The classical procedure for ARIMA modeling is model identification, parameter estimation, diagnostic checking and forecasting. In this case there are 5 series of data remaining after pre-treatment of the original data. Only series ST1 is taken as an example to explain the modeling process.

2.4.1 Modeling

As mentioned in 2.3.3 and 2.3.4, now the deterministic parts in the residual should be modeled. The ACF and PACF plots of the residual series after one seasonal differencing are shown in Fig.2.10 and Fig.2.11. By observing the plots

*Fig. 2.10 Estimated ACF for Residual with 1 order seasonal differencing*
we notice that there are bars at lag 1,2,3, especially at 24 hours, exceeding t-critical value; that is, the correlation coefficients are significantly different from zero. One order of coefficients should be given to moving average models. The PACF plot shows a big bar at time lag 2 hour, so AR(1) and SAR(1) (Seasonal Autoregressive) models may be also needed. Therefore, an attempt was made first to produce a first order ARMA model, and to give the same order to the seasonal ARMA (SARMA) model. There are several results that should be discussed for this ARIMA \((1,0,1)*(1,1,1)_s\) model.
According to the suggestions of Pankratz (12) and D'Agostino et al (10), the significance of the estimated parameters in the model was tested. The t-statistic for $\hat{\varphi}_1$ (the coefficient of the Seasonal Autoregressive model) of SAR(1) equals -1.4 and P-value equals 0.16 with 95% confidence level, so the hypothesis $\varphi_1 = 0$ could not be rejected. This means that the SAR(1) model may not be needed since $\varphi_1$ is not significantly different from zero.

Fig.2.12 and 2.13 show the residual ACF and PACF of the ARIMA model with order of $(1,0,1) \ast (0,1,1)_s$. The t-test for the parameters of $\varphi_1$, $\theta_1$, and $\theta^s_1$ is 10.7, -8.57 and 23.5 respectively. The ACF and PACF plots reveal that there are no significant correlations among the residuals. Fig.2.14 shows that the residuals of this model are random. All residual values fall in 75% Kolmogorov-Smirnov confidence intervals, which is consistent with the hypothesis that the residual is a random process. A Chi-square test was applied on the first 20 residual autocorrelations. The test statistic is 12.33, which is less than the critical value 28.8 with 95% confidence level, that is, the residuals are not correlated.
Fig. 2.12 Estimated ACF for Residual
with order of \((1,0,1)^*(0,1,1)\)

Fig. 2.13 Estimated PACF for Residual
with order of \((1,0,1)^*(0,1,1)\)
2.4.2 Results

In section 2.4.1 an example was presented to describe the modeling processes. In this part, the criteria for checking the goodness-of-fit of ARIMA models are summarized first. Then the estimated models of influent flow rate for the Interbay Pump Station are presented.

The criteria are:

A. Parsimonious (uses the smallest number of coefficients)

B. It is stationary

For AR(1)
\[ |\psi_1| < 1 \]
For AR(2)
\[ |\psi_2| < 1 \]
\[ \psi_2 + \psi_1 < 1 \]
\[ \psi_2 - \psi_1 < 1 \]

For SAR(1)
\[ |\theta_1| < 1 \]
For SAR(2)
\[ |\theta_2| < 1 \]
\[ \theta_2 + \theta_1 < 1 \]
\[ \theta_2 - \theta_1 < 1 \]
C. It is invertible.

For MA(1) \[ |\theta_1| < 1 \]
For MA(2) \[ |\theta_1| < 1 \]
\[ \theta_2 + \theta_1 < 1 \]
\[ \theta_2 - \theta_1 < 1 \]
For SMA(1) \[ |\theta_1| < 1 \]
For SMA(2) \[ |\theta_1| < 1 \]
\[ \theta_2 + \theta_1 < 1 \]
\[ \theta_2 - \theta_1 < 1 \]

D. It has estimated coefficients ( \( \varphi \) and \( \theta \)) of high quality.
   a. absolute t-test value above 2.
   b. \( \varphi \)'s and \( \theta \)'s not too highly correlated.

E. It has uncorrelated residual.
   a. check residual ACF.
   b. check residual PACF.
   c. check residual distribution.
   d. check residual periodogram and integrated periodogram.
   e. check correlation matrix of estimated COEFF.

F. It adequately fits the available data.
   a. root-mean-squared error (RMSE) is acceptable.
   b. mean absolute percentage error (MAPE) is acceptable.

G. It forecasts the future satisfactorily.

There are 5 time series after data pre-treatment, and a total of 6 ARIMA models are considered to be candidates for describing the behavior of the influent flow rate for the Interbay Pumping Station. Table 2.1 shows the model form, number of parameters and residual sum of squares (RSS). Table 2.2 shows the parameter quality, which can be evaluated by the values of the different test methods, their
Table 2.1  Estimated Model Forms

<table>
<thead>
<tr>
<th>model No.</th>
<th>time series</th>
<th>form</th>
<th>model forms</th>
<th>parameter value</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>ST1</td>
<td>x</td>
<td>$F_{S_t}=(1-B\theta_1)(1-B\theta_1^S)at/(1-B\phi_1)$</td>
<td>$\theta_1=0.5346$ $\theta_1^S=0.9121$ $\phi_1=0.6235$</td>
</tr>
<tr>
<td>S2</td>
<td>ST2</td>
<td>x</td>
<td>$F_{S_t}=(1-B\theta_1)at/(1-B\phi_1)$</td>
<td>$\phi_1=0.8479$ $\theta_1=-0.2069$</td>
</tr>
<tr>
<td>S3</td>
<td>ST3</td>
<td>log</td>
<td>$(F_{S_t})(F_t)=(1-B\theta_1^S)at$</td>
<td>$\theta_1^S=0.766$</td>
</tr>
<tr>
<td>S4</td>
<td>ST3</td>
<td>log</td>
<td>$F_{S_t}=(1-B\theta_1)(1-B\phi_1)$</td>
<td>$\phi_1=0.8737$ $\theta_1=-0.4563$</td>
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<tr>
<td>S5</td>
<td>ST4</td>
<td>sqrt</td>
<td>$F_{S_t}=(1-B\theta_1)at/(1-B\phi_1)$</td>
<td>$\phi_1=0.6426$ $\theta_1=-0.6$</td>
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<tr>
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<td>ST5</td>
<td>log</td>
<td>$F_{S_t}=(1-B\theta_1)(1-B\theta_1^S)at/(1-B\phi_1)$</td>
<td>$\phi_1=0.7347$ $\theta_1=-0.6831$ $\theta_1^S=0.8106$</td>
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</table>

<table>
<thead>
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<th>P-value</th>
<th>confid. (%)</th>
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Table 2.2  
Model Quality Features

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<th>Parameter No.</th>
<th>average T-value</th>
<th>Chi-test for corr.</th>
<th>resi.sum square</th>
<th>forecast sum-sqrt</th>
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<td>35.18</td>
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<td>26.18</td>
<td>1.927</td>
<td>79.53</td>
</tr>
</tbody>
</table>

* The mean and standard deviation of the random variables comprise two additional parameters which should be added to the number of parameters given.

corresponding probability, and by the RSS values and forecasting-sum-squared root values for each parameter.

\[ \phi_1 \text{ ---- AR process operator with order of } p \]
\[ \theta_1 \text{ ---- SAR process operator with order of } P \]
\[ \theta_1 \text{ ---- MA process operator with order of } q \]
\[ \theta_{S_1} \text{ ---- SMA process operator with order of } Q \]
\[ s \text{ ---- seasonal length } s=24 \text{ hour} \]
\[ P_{S_t} \text{ ---- } =F_t-F_{t-s} \]

2.5 Model Selection

Tables 2.1 and 2.2 show that there is no single model that is better than all the others in every criteria. A decision must be made to select the best of the six. Several multi-criteria decision making methods were considered for
this purpose, and finally the ELECTRE method (Elimination and (et) Choice Translating Algorithm) was chosen for the model selection. Basically, the ELECTRE method is applied to select alternatives with both quantitative and qualitative multi-criteria (14,15, 16), and has proven to be a powerful method in dealing with discrete systems.

There are two major advantages for the ELECTRE method, which are why this method was adopted in this research. The first is that any qualitative criteria can be added to the decision-making matrix to select the models, based on the purpose the models are to be used for; and the second is that the final ranked order (from the best to the worst) is robust and less influenced by (the weights which reflect) the preferences of the decision makers.

2.5.1 Description of ELECTRE

The ELECTRE method was developed by Benayoun and Roy et al (51). The method can be classified into two parts: ELECTRE I and ELECTRE II. Generally, ELECTRE I can be used to eliminate alternatives when the number of alternatives is large, by using threshold values $P,Q$ that are accepted by the decision maker. Then ELECTRE II is applied to rank the rest of the alternatives. Only ELECTRE II was used herein since there are only six alternatives. There are three important concepts in this method: concordance, discordance and threshold values.
The concordance between any two actions \( a_i \) and \( a_j \) is a weighted measure of the number of criteria for which \( a_i \) is preferred to \( a_j \) or for which \( a_i \) is equal to \( a_j \). The definition of concordance is given

\[
c(i,j) = \sum_{k \in I^+(i,j)} w_k + \frac{1}{2} \sum_{k \in I^-(i,j)} w_k \tag{2.1}
\]

where \( w_k \) is the weight on criteria \( k, k=1, \ldots, K \), i.e. The \( I^+(i,j) \) and \( I^-(i,j) \) are the sets of all criteria for which \( i \) is preferred to \( j \) or equal to \( j \), and are defined as

\[
I^+(i,j) = \{ k | y_{ki} > y_{kj}, k=1, \ldots, K \}
\]
\[
I^-(i,j) = \{ k | y_{ki} = y_{kj}, k=1, \ldots, K \} \tag{2.2}
\]

where \( y_{ki} \) and \( y_{kj} \) are the attributes of action \( i \) and action \( j \) on the criteria \( k \). Therefore, concordance can be thought of as the weighted percentage of criteria for which one action is preferred to another.

The discordance is defined as

\[
d(i,j) = \max_k [\Gamma_k(a_j) - \Gamma_k(a_i)] / \Theta(k, \Gamma_k(a)) \tag{2.3}
\]

where \( \Gamma_k(a) \) is a positive function on set \( A \) in \( K \) dimensions, when, in special cases, \( \Gamma_k(a_j) = y_{kj} \), it represents how much the attributes of action \( i \) are preferred over action \( j \) on the criteria \( k \). \( \Theta(k, \Gamma_k(a)) \) is a binomial function that represents an estimated value of an action.
There are three thresholds, $P^-, P^0, P^*$, for concordance and two thresholds, $Q^0$ and $Q^*$, for discordance. The thresholds satisfy the following conditions in ELECTRE II:

$$0 \leq P^- \leq P^0 \leq P^* \leq 1$$
$$0 \leq Q^0 \leq Q^* \leq 1$$

It should be mentioned that the thresholds for discordance ($Q^0$ and $Q^*$) may not always obey the above condition. These depend mostly on the method of use of equation (2.3) to obtain the disconcordance matrix. By using the thresholds for concordance and discordance matrices, a strong preference graph $(A, R_s)$ and weak preference graph $(A, R_w)$ are obtained, which can be used to rank the actions in order. The thresholds represent the preference structure and the toleration of the decision maker. For example, if the decision maker gives a high level of concordance and low level of discordance (higher $P'$s and lower $Q'$s), fewer preferred relations may appear the on strong and weak preference graphs. So the threshold values could influence ranking results.

Once the strong and weak graphs have been obtained the ordering processes can be carried out based on the graphs. There are three ranking procedures: forward ranking, reverse ranking and average ranking. Forward ranking obtains positive orders (from best to worst); then, all preferred relations in the graphs $(A, R_s)$ and $(A, R_w)$ are reversed to get reversed orders of the actions; finally, the two order
numbers for an action are averaged, and the final rank for this action is decided. The action ranked at first place should be the best choice.

2.5.2 Model Selection

Based on the identified models for wastewater flow rate (Table 2.1 and 2.2), the decision making matrix was formed and is shown in Table 2.3. There are 6 criteria chosen for selecting the models. The parameter number of criteria one represents the principle of parsimony that was strongly suggested by Box and Jenkins. It also reflects ease of use by the practitioner. The average T-value represents the parameter quality in each model. The Chi-test value is the standard for monitoring the correlation level among residuals.

Because only two weeks of data are available, the outliers in Fig.2.2 could not be verified as storms (on March 1 and 5) or abnormal values at the valley (on March 1, 2 and 5) distinct from the main trend of the flow rate. The original data without artificially picking out the outliers might better reflect the nature of influent flow rate. The need for data pretreatment was therefore added as criteria 6. The data series without pretreatment was considered as good and given level 3; the series without the outliers is fair and given level 1.
Another problem is the criteria of the residual sum of the squares (RSS). In Table 2.2 RSS for model S3, S4 and S5 is small compared with the other models because of the logarithm forms for S3 and S4, and the square root form for S5. In order to be able to compare the models based on this criteria, the RSS for model S3 and S4 is enlarged 1000 times, and the RSS for S5 is squared. The above treatments of RSS do not significantly influence the ranking results, because how much better model A is than model B on criteria k can only be reflected in the discordance matrix. The sensitivity analysis will prove the point later. To obtain the minimum values of some criteria in the decision matrix, minus signs were given to criteria 1, 3, 4 and 5.

<table>
<thead>
<tr>
<th>model No.</th>
<th>cr.No.</th>
<th>Parameter No.</th>
<th>average T-value for corr.</th>
<th>Chi-test square</th>
<th>resi.sum</th>
<th>forecast data pre sum-s-rt</th>
<th>treat.</th>
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</thead>
<tbody>
<tr>
<td>S1</td>
<td>-3</td>
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<td>14.26</td>
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<td>S3</td>
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<td>-2</td>
<td>4</td>
<td>17.62</td>
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Table 2.4  
Ranking Results

<table>
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<tr>
<th>Criteria</th>
<th>No.</th>
<th>Parameter average Chi-test</th>
<th>T-value for corr. square</th>
<th>resi.sum forecast data-pre</th>
<th>sum-s-rt treat.</th>
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<tr>
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<td>threshold</td>
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<td>$P^*$</td>
<td>$P^0$</td>
<td>$P^-$</td>
<td>$Q^*$</td>
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<tr>
<td></td>
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<td>4</td>
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</table>

The first group of weights and thresholds for concordance and discordance are given and the ranking results are shown in Table 2.4. Fig. 2.15 and 2.16 plot the strong and weak preference graphs for ranking. In the plots, the directions of the lines represent the preference of model (or action) $i$ over model $j$. For example, in Fig. 2.15, there is a line from model 1 to model 6, which indicates that model 1 is preferred to model 6. The results show that the model S3 with 1 order of both seasonal and non-seasonal differencing and 1 order of seasonal moving average is the best. It is also noted that the models with logarithm transform for the original data series are ranked first and second under multi-criteria comparison.
Fig. 2.15  Strong Preferred Graph P (7, 5, 3) Q (2, 1)

Fig. 2.16  Weak Preferred Graph P (7, 5, 3) Q (2, 1)
2.5.3 Sensitivity Analysis

Sensitivity analysis was performed to test the robustness of the ranking with respect to changes in the scale of concordance and discordance level, and changes in weight vector. The changes in weight vector and ranking results are shown in Table 2.5. The last two weight vectors in Table 2.5 were given by one expert who is familiar with the ARIMA model and another who is familiar with wastewater treatment processes. The results reveal that although the weight vector changes across a wide range, the first three

Table 2.5 Changes in weight vector

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<thead>
<tr>
<th>criteria No.</th>
<th>average T-value for corr. square</th>
<th>resi.sum square</th>
<th>forecast sum-s-rt treat.</th>
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<td>6</td>
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</tr>
</tbody>
</table>
ranked models are very stable in their places. Changes of order only happened in the last three models.

Sensitivity analysis for changes in scale was conducted only for the last two weighted vectors in Table 2.5. Tests were made by alternately changing the concordance and discordance level for each of the weight vectors. The test results are shown in Table 2.6. It indicates that the ranking orders of a good model are sensitive to change in discordance level, but not so sensitive to change in

<table>
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<tr>
<th>level</th>
<th>( P^* )</th>
<th>( P^0 )</th>
<th>( P^- )</th>
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<th>( Q^- )</th>
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<td>0.5</td>
<td>2.0</td>
<td>1.0</td>
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<td>0.5</td>
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<tr>
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<td>0.6</td>
<td>0.5</td>
<td>0.5</td>
<td>0.2</td>
</tr>
<tr>
<td>W4</td>
<td>0.7</td>
<td>0.6</td>
<td>0.5</td>
<td>2.0</td>
<td>1.0</td>
</tr>
<tr>
<td>W4</td>
<td>0.8</td>
<td>0.65</td>
<td>0.5</td>
<td>1.0</td>
<td>0.5</td>
</tr>
<tr>
<td>W4</td>
<td>0.8</td>
<td>0.65</td>
<td>0.5</td>
<td>0.5</td>
<td>0.2</td>
</tr>
</tbody>
</table>

<table>
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<th>S2</th>
<th>S3</th>
<th>S4</th>
<th>S5</th>
<th>S6</th>
</tr>
</thead>
<tbody>
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<td>rank</td>
<td>W3</td>
<td>3</td>
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<td>3</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>6</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>5</td>
</tr>
</tbody>
</table>
concordance scale. When the discordance scale is tightened (lower Q's values given), the models tend to be more uniformly ranked. The phenomena could be explained as when lower Q's values are given, the preferred relations between any two models on criteria \( k \), \( (k=1, \ldots, K) \), will become less apparent in graphs \((A,R_S)\) and \((A,R_W)\), because of the higher qualified requirement for preferred relations. The kernel points in the graphs increase. Fortunately, model S3 is still ranked in the first place no matter what the change in scales. Therefore, model S3 is taken as the time input pattern of wastewater flowrate in the simulation of the equalization basins and pump stations.

2.5.4 Forecasting Using Selected Model

One hour leading forecasting using this model, and another set of measured data (from March 19 to April 11, 1990) are shown in Fig.2.17. Examining the plots, the forecasted curve fits the measured curve well.
Fig. 2.17 1 hr Leading Predictions
Data from Mar. 19 to Apr. 11, 1990

Using Model S3
2.6 Conclusions and Recommendations

1. Municipal wastewater flow rates have both weekly and diurnal periodic characteristics. Generally, the flow rates during the weekend are more stable and have lower mean values than on weekdays.

2. These periodic characteristics should be taken into account when using ARIMA models to predict influent flow rates. The application of a log transform to the data series can reduce the effect of outliers.

3. A critical step in time series analysis is the examination of the autocorrelation function (ACF) and partial autocorrelation function (PACF). A seasonal subseries plot is an effective and easy-to-use method to detect if a time series is stationary, or if a stationary series can be obtained after differencing. The t-test and Chi-squared test are good criteria for testing the quality of the estimated parameters.

4. The form of the ARIMA model selected depends on the purposes for which the models are to be used. ELECTRE II, a multi-criteria decision-making algorithm, is a good technique for selecting the type of ARIMA model to be used.
Chapter 3

SIMULATION AND CONTROL OF PUMPING STATIONS

The design and operation of a pumping station, as well as the selection of the appropriate control strategy are difficult problems. The main reasons are large variations in flowrate and the comparatively small storage capacity provided by wet wells. The limitation of storage volume is not only because of the high capital cost, but also the maximum length of time that the wastewater can be stored without sedimentation of solids and generation of odor. For fixed speed pump installations, these limitations can result in too-frequent starting and stopping of the pumps, which will create man-made hydraulic shocks and consequently have a deleterious effect on the downstream processes (35); and will cause higher maintenance and repair costs and shorten equipment life. It has been estimated that electric motors have a "lifetime" of approximately 135,000 on-off times (21). The problem is also complicated by the requirement of high reliability for operation of pumping stations in order to avoid sewer overflow. This, in order to meet the design criteria of a pumping station with a fixed speed pump installation, results in oversized pump installation and operation, which will further increase the number of on-off
cycles of pumps, and cause larger hydraulic surges to the downstream treatment processes.

Pumping stations are the major consumers of energy in wastewater collection systems. Common characteristics of wastewater pumping stations are deep wet-well depths and comparatively low lift heads. Keeping water level in the wet well as high as possible and decreasing the height of the lift head can substantially reduce power required by the pumps.

Based on the problems mentioned above, the objectives of this part of the research are: (1) to minimize the number of on-off cycles of the pumps during operation, and therefore to minimize both the magnitude and frequency of man-made hydraulic shocks; (2) to reduce the power required by the pumps; and (3) to develop control strategies for the operation of fixed speed pumps to realize objectives (1) and (2). It should be noted, and will be proved later, that objectives (1) and (2) are in conflict with each other; that is, if the pump on-off times are reduced, the power required by the pumps will be increased and vice versa.

In general, there are four major components to a pumping station: influent flowrate, wet-well, pumps and the control system. The models for the four processes were established separately, put into the modules in SIMNON, and then chained together to simulate the different control strategies. The results showed that the dynamic simulation
of the performance of a pumping station can be beneficial to both the design and operation of pumping stations. By the simulation of their performance, the designs of the wet-well and pumps can be checked to see if the design objectives are reached. The simulation can also serve as an off-line tool to test the new control strategy before it is put into use.

Since there were no pump characteristic curves available, the pump curves used in the simulation were from *Metcalf and Eddy* (19).

### 3.1 Models

As mentioned above, a pumping station consists mainly of four components. The models for the first three components were established on the basis of statistics and physical laws in order to simulate the control processes. Control models will be developed and discussed in section 3.2.

#### 3.1.1 Influent Flowrate

Influent flowrate to a pumping station can be highly dynamic and plays a most important role in the control of the pumping station. The better the flowrates are known in advance, the better the performance of pumps can be controlled. In this research ARIMA models were employed to forecast input flowrate values for certain time intervals,
which is an important part of the optimum control of fixed speed pumps.

The identification, estimation and diagnosis checking of ARIMA models using data from the Seattle Interbay Pumping Station was explained in detail in Chapter 2. The final selected model takes the form

\[(F_{st})(F_t) = (1-B\theta^s) a_t\]  \hspace{1cm} (3.1)

where

- \(F_{st}\) --- seasonal differencing
- \(F_t\) --- non-seasonal differencing
- \(B\) --- back shift operator
- \(\theta\) --- seasonal moving average parameter
- \(s\) --- seasonal length. Here \(s=24\) hour
- \(a_t\) --- noise or random component, \(a_t = F_t - F_{t-1}\)

To further put this into forecasting form equation (3.1) becomes

\[F_t = F_{t-1} + F_{t-s} - F_{t-s-1} - \theta a_{t-s} + a_t\] \hspace{1cm} (3.2)

where

- \(F_t\) --- forecasted flowrate at time \(t\)
- \(F_{t-1}\) --- flowrate at time lag \(1\) time unite
- \(F_{t-s}\) --- flowrate at time \(t-s\)
- \(F_{t-s-1}\) --- flowrate at time lag \(t-s-1\)
- \(a_{t-s}\) --- random component, \(a_{t-s} = F_{t-s} - F_{t-s-1}\)
- \(\theta\) --- coefficient of SMA(1) model, \(\theta=0.766\)
These equations (3.1) and (3.2) were used in the natural logarithm form since the logarithm form can greatly reduce the magnitude of any outliers, therefore the predicted flow rate will take the anti-logarithm form in order to obtain real flowrate values.

There are two sets of hourly average flowrate data. One is from February 23 to March 8, 1990, with two week length. The other is from March 19 to April 11, 1990. The first set of data was used to identify the model (3.1), and the second was applied to check the forecasting accuracy of the estimated model. It should be noted that the accuracy of predictions using ARIMA models with a seasonal moving average term (SMA(1)) will decrease as the length of the prediction period increases, since the $a_t$ term in equation (3.2) is zero for predicting flowrate at time $t$. The predicted values will trend toward the mean flowrate for that day when the model is used for 24 hour forecasting.

3.1.2 Wet-well

Based on a material balance, the differential equation describing the relationship between influent flow rate, outflow rate and tank volume response is

$$\frac{dv}{dt} = F_{in} - F_{out}$$

To match the discrete signal of flowrate sampling the above equation can be rewritten in the form of a difference equation:
\[ V_t = V_{t-1} + (F_{in} - F_{out}) T_{deta} \]  \hspace{1cm} (3.3)

where

- \( V_t \) --- volume of tank at time \( t \)
- \( V_{t-1} \) --- volume of tank at time \( t-1 \)
- \( F_{in} \) --- mean influent flow rate from time \( t-1 \) to \( t \)
- \( F_{out} \) --- mean effluent flow rate from time \( t-1 \) to \( t \)
- \( T_{deta} \) --- length of time interval between two samples

In the traditional control of pumps, the water level in the wet-well is a major signal for control. Equation 3.3 can be further written in the relationship between influent, effluent and water level in the wet-well

\[ H_t = H_{t-1} + (F_{in} - F_{out}) T_{deta} / A \]  \hspace{1cm} (3.4)

where

- \( H_t \) --- water level at time \( t \), m
- \( H_{t-1} \) --- water level at time \( t-1 \), m
- \( A \) --- area of wet-well, m²

Here \( F_{out} \) is the outflow rate which is provided by the pumps. It is obvious that the wet-well level is a key measurement for relating influent and pump flow rates.

### 3.1.3 Pumps

There are two major variables for a pump: capacity and head. The capacity of a pump is defined as the volume of liquid pumped per unit of time. Head can be interpreted as the elevation of a free surface of water above or below a
reference datum. Fig. 3.1 illustrates the heads on a pump. The heads in a pumping system can be classified as the pump head and system head. The pump head is the head that lifts water from one elevation to another, which is dynamically changing due to the changing of flow rate of pumps and water level in the wet-well. The system head is the head caused by friction lost inside of the pump and piping, which varies only due to changes in the flow rate of the pump.

Basically there are three curves which represent the characteristics of a pumping system: pump head-capacity, pump efficiency and system head-capacity curves. The first two are obtained from the pump manufacturer. The third one can be obtained by calculating head losses at different flow rates for the piping system. Once the construction of the pumping station is accomplished, the shape of the system curve is fixed. Figs. 3.2 and 3.3 show the fixed speed pump capacity-head, system curve, and pump efficiency curve, which are used in this study. The intersection between the pump capacity-head curve and the system curve is the pump operating point. The operating point is called the best efficiency point (BEP) if it corresponds to the range of the highest pump efficiency.

There are several ways to represent the above curves by using mathematic models. There is a graphical method in which the pump curve can be divided into many segments. Any two adjacent points in the curve can be approximately
Fig. 3.1 Definition of Heads for a Pumping Station. After Metcalf and Eddy (19)
**Fig. 3.2 Pump Curve and System Curve**
The curves are simulated by SIMNON

**Fig. 3.3 Pump Efficiency Curve**
The Curve Simulated by SIMNON
represented by a constant value. Vitasovic (5) proposed a polynomial equation to describe the curves. In fact, for pump specific speeds less than 80, a parabolic equation is close enough to describe the characteristic curves of fixed speed pumps. For the efficiency curve, a cubic equation is needed. Figs. 3.2 and 3.3 were drawn using parabolic and cubic equations in SIMNON. The equations representing the pump models are:

\[
H_p = 28.929 + 5.866Q - 10.42Q^2 \\
H_S = 0.341 - 2.665Q + 6.642Q^2 \\
Eff = 0.75 - 0.142Q + 0.405Q^2 - 0.181Q^3
\]

(3.5) (3.6) (3.7)

where

\begin{align*}
H_p & \quad \text{--- pump lift head} \\
H_S & \quad \text{--- system head} \\
Eff & \quad \text{--- pump efficiency} \\
Q & \quad \text{--- pump flowrate at operating point}
\end{align*}

It should be noted that the operating point dynamically varies during the operation of a pumping system because of changes in the flowrate of pumps and water level in the well. To accommodate operating points for each pump at every sampling time numerical methods can be applied to obtain the solutions from the polynomial equations with orders more than 2. Newton's interpolation method is a effective method for numerically obtaining the solutions.
3.2 Simulation and Control of Fixed Speed Pumps

There are strong interactions between the design and operation of wastewater pumping stations. The size of wet-well, pump size and type, and number of pumps chosen in the design process, have a great influence on the control process. Pincince (21) has presented some useful equations to determine wet-well volume for fixed speed pumps under two point control. A constraint for determining the volume of the wet-well is that the maximum detention time is 30 minutes at minimum flowrate. The minimum detention time in the wet-well is decided by the pump cycle time, that is, pump starting time plus stopping time. The frequency of pump starts and stops is limited by the ability of the motor to cool itself, which is related to the motor's size and type.

The number of pumps is another important part in the design process and has a strong influence on the control mode. The advantages of using a large number of fixed speed pumps are to reduce the magnitude of man-made hydraulic surges to the plant, to diminish the pump cycling time because the size of the individual pumps is small, and therefore to reduce wet-well volume. Bouck and Webb (29) provided an example for the Orlando treatment plant in Florida. A total of eight pumps were used to handle the flowrate with a maximum discharge of 48,000 gpm and a minimum of 4500 gpm. The pumps can work in different combinations and provide 17 various flows and steps.
However, too many small pumps used in the operation may result in a lower efficiency compared with a few larger pumps. The optimum number of pumps is still a tough problem encountered by design engineers.

The interactions among the designated wet-well volume, pump numbers and size, wet-well water level, on-off set points, and control mode will be discussed in detail in the following sections. The design decisions and operational procedures are dynamically connected by SIMNON simulations. The control strategies for the pumping station will be tested by simulation.

3.2.1 Two Point Control

Two point controllers with a continuous measurement of the water level in the wet well are frequently used for the control of fixed speed pumps in wastewater collection systems (25). The major measurement is the water level in the wet-well. Feedback control was applied with two fixed speed pumps working in parallel. Fig.3.4 is a sketch of the control loop.

Typical dry weather flowrate data (April 5, 1990) was obtained from the Interbay Lift Pumping Station, Seattle (30), with an average flowrate of 48 MGD (the design average flowrate is 49 MGD). The flowrate profile is shown in Fig.3.5. The minimum flowrate is 34 MGD and maximum is 57 MGD. The two pumps used in the simulation are identical,
Fig. 3.4 Two Point Control Flow Chart
with a lift of about 15 meters (45 feet) and a capacity of 1.35 m³/sec. (21,400 gpm) for each pump. The pump head-capacity, efficiency and system head-capacity models are given in equations 3.5, 3.6, 3.7. To obtain the influence of the number of pumps on the control process, the number of pumps will be doubled in later simulations.

The wet-well volume used in the simulation was determined by the constraint that the maximum detention time be 30 minutes at the minimum flow. By observing the two week's flow rate data in Fig. 2.1 (Chapter 2), the minimum flowrate was found to be 15 MGD, so the volume of the wet-well is 1182 m³. The dimensions of the wet well are L*W*H = 19.7*10*6 meters. Since the wet-well depth is important for

Fig. 3.5 24 Hourly Prediction by Using Equation (3.2)
energy saving, it is chosen as 6 meters (the lift head is 15 meters). The width and length of the wet-well are arbitrarily selected. It should be noted that the volume of the wet-well is not considered fixed but instead will be adjusted in order to analyze its effects on the control variables and modes.

There are two control modes for the two point controller with two pumps. The control scheme is shown in Fig. 3.6. For mode A, when the water level in the wet-well rises and exceeds set point ML, pump 1 will be started, and when the level exceeds set point HL, pump 2 will be started. The two pumps will work together until the water level reaches set point LL, whereupon the pumps stop. For mode B, the procedures of starting the pumps are the same as mode A, but whenever the water level is less than set point ML pump 2 will stop, and pump 1 will stop when the level falls below set point LL.

Considerations were given to the analysis of the interactions of the number of pumps, wet-well volume, pump on-off times and power requirements in the comparison of the two control modes.

3.2.1.1 Comparisons of Control Modes

The sampling time was adjusted to 1 minute in the simulation. The set points LL, ML and HL are 0.5 m, 3.0 m and 5.5 m respectively. Fig. 3.7 through Fig. 3.10 show the
Fig. 3.6  CONTROL SCHEME FOR 2 PUMPS
working states of pump 1 and 2, the water level in the wet-
well and static head (elevation between water level in wet-
well and discharge point) for control mode A. Figs. 3.11 and
Fig. 3.12 are the working modes of the two pumps for control
mode B. Table 3.1 summarizes the results of simulation.

By comparing Fig. 3.7 and 3.8 with Fig. 3.11 and 3.12,
it is found that the pumps' working states in the two
control modes are quite different. For mode A the pump start
times during the day are quite uniformly distributed between
the two pumps. This is because the stopping set point for
pump 2 is the lower limit of the water level in the wet-
well. However, it also creates a larger magnitude of surges
than mode B. The drawback of control mode B is that the
starts and stops are more frequent for pump 2 than pump 1.

Table 3.1  Comparison of Control Modes A & B

<table>
<thead>
<tr>
<th></th>
<th>Mode A</th>
<th></th>
<th>Mode B</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>set point (m)</td>
<td>LL 0.5</td>
<td>ML 3.0</td>
<td>HL 5.5</td>
<td>LL 0.5</td>
</tr>
<tr>
<td>No. of pump</td>
<td>pump 1 17</td>
<td>pump 2 17</td>
<td>pump 1</td>
<td>pump 2 X</td>
</tr>
<tr>
<td>on-off cycles/d</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>min. cycle time</td>
<td>46</td>
<td>46</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>power required</td>
<td>0.0296</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: The sign X indicates that the pump is on all day
without minimum cycle time.
Fig. 3.7 Working State of Pump 2
for Mode A; 0.5 -- Off; 1 -- On

Fig. 3.8 On-Off State of Pump 1
for Mode A; 1.5 -- Off; 2 -- On
Fig. 3.9 Water Level in Wet-well for Mode A

Fig. 3.10 Static Head vs. Time for Mode A
Fig. 3.11 On-Off State of Pump 1
for Mode B; 0.5 -- Off; 1 -- On

Fig. 3.12 On-Off State of Pump 2
for Mode B; 1.5 -- Off; 2 -- On
(This problem can be overcome by daily alternation of the pump working order between 1 and 2).

The minimum pump cycle time for mode B is 26 minutes compared with 46 minutes for mode A. However, it should be mentioned that the power required by the pumps in mode B is reduced 4.5% compared to mode A. The reason is that the average water level in the wet well under the control of mode B is higher than that of mode A. Therefore the static head in mode B is less than that in mode A.

Based on the above discussions of the two control modes it is difficult to decide which control mode is better. However, the following considerations can be taken into account concerning the above analysis: if the set point ML for pump 2 can be dynamically changed in mode B, a reduction in both required power and on-off times for pump 2 may be achieved. Increasing the number of pumps may be another way to solve the problem. The first consideration will be discussed in the optimum control modeling section and the second later in this section.

3.2.1.2 The Effects of Wet-well Volume on the Performance of Pumps

In this section the influences of wet-well volume on pump behavior, control mode and energy utilization are analyzed based on the simulation results. The dynamic input of flow rate and pump responses are not taken into account
in the design stage. It is desirable to evaluate the required volume of the wet-well with dynamic input and pump control, so that a proper design volume can be obtained.

In order to investigate the effect of volume on the control modes, volume changes are made so that the original volume (1182 m$^3$) is taken as 1 and the changed volumes are given in ratio to the original one. All other conditions are the same as in section 3.2.1.1.

Fig. 3.13 shows volume changes vs. the number of on-off pump cycles during a day. It reveals that the decrease in volume of the wet-well will cause pump on-off cycles to increase. This is especially true for control mode B because the adjustments to accommodate the dynamics of the influent flows are handled entirely by pump 2, with pump 1 always remaining in the on state. When the reductions of wet well volume reach 80 to 60 percent of the initially used volume the pump on-off frequency increases sharply. The phenomena can also be observed in the minimum cycle time of pumps shown in Fig. 3.14. Therefore, if the volume is smaller, control mode A would be preferred in order to avoid frequent starting and stopping of the pumps, but would increase the magnitude of the surges. When the volume is increased the starting and stopping times are not significantly reduced according to Fig. 3.13, especially for control mode A. The pump on-off cycles seem to remain
Fig. 3.13 Effect of Wet-well Volume on Pump On-Off State

- Pump 2 of Mode A
- Pump 2 of Mode B

Fig. 3.14 Effect of Wet-Well Volume on Pump Minimum Cycle Time

- Mode A
- Mode B
relatively constant although more volume is provided for storing the water. So increasing the wet well volume may not be the best way to reduce pump on-off times although it can certainly do so. It also should be noted that the slope of both of the curves in Figs. 3.13 and 3.14 becomes less after 120 percent of the originally used volume. This suggests that the original volume used in the above simulation may be smaller than it should be.

Fig. 3.15 displays the power required by the pumps vs. the wet-well volume. For control mode A the minimum power required is at 120 percent of the original volume. The reduction is about 2% as compared with the maximum

![Graph of power required vs. ratio of original volume to volume changed. The graph shows two lines, one for Mode A and one for Mode B, with the power required decreasing as the ratio increases from 0.6 to 1.6.](image-url)
requirement at 60 percent of the original volume. For control mode B there seems to be no minimum value. However, the power required is 5% lower than that for mode A.

3.2.1.3 Effect of Pump Numbers and Size on Control Modes

In order to test the effect of pump number on the control mode the number of pumps is increased to four with a capacity of 0.668 M³/sec. (10588 gpm) for each. The four pumps work in parallel and a two point feedback control is applied to the pumping system. The pump models are

\[
\begin{align*}
H_p &= 28.929 + 5.866Q - 10.42Q^2 \\
H_s &= 0.0012 + 0.006Q + 18.164Q^2 \\
E_{\text{eff}} &= 0.75 - 0.285Q + 1.619Q^2 - 1.44Q^3
\end{align*}
\]

(3.8)  
(3.9)  
(3.10)

The \( H_p, H_s, E_{\text{eff}} \) and \( Q \) have the same meaning as in section 2.3

Fig. 3.16 shows the control scheme for the four pumping units. The working mode C is such that when the water level exceeds set point L2, pump 1 will start; if the water level continues and is higher than set point M pump 2 will start; the same procedures are for pump 3 and 4 to start at set point H2 and H1 respectively. When the water level is lower than set point M both pumps 3 and 4 will stop, and pumps 1 and 2 stop at set point L1. For control mode D, the starting process is the same as in mode C, but the stopping
Fig. 3.16  CONTROL SCHEME FOR 4 PUMPS
procedures are different. Pumps 4, 3, 2 and 1 stop working whenever the water level is lower than the set points H2, M, L2 and L1 respectively, that is, each pump is responsible for a certain volume of water in the wet-well.

Table 3.2 displays the simulation results of control modes C and D for 4 pumps. For control mode C, pumps 1 and 2 are in the on state all day. Comparing the control scheme for control mode C (Fig.3.16) and for mode A (Fig.3.6), they have a similar way for starting and stopping pumps. However, increasing of the number of pumps and reducing their sizes affects pump starting time, outflow profile and power required. The on-off cycles of two pumps in mode A are less than those in mode C. Also the minimum pump cycle time in mode A is longer than in control mode C. It should be noted

<table>
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<th>Mode C</th>
<th></th>
<th>Mode D</th>
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</tr>
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<tbody>
<tr>
<td>set point</td>
<td>L1</td>
<td>L2</td>
<td>M</td>
<td>H2</td>
</tr>
<tr>
<td>(m)</td>
<td>0.5</td>
<td>1.75</td>
<td>3.0</td>
<td>4.25</td>
</tr>
<tr>
<td>No. of pump</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>on-off cycles/d</td>
<td>X</td>
<td>X</td>
<td>28</td>
<td>16</td>
</tr>
<tr>
<td>min. cycle time</td>
<td>X</td>
<td>X</td>
<td>28</td>
<td>40</td>
</tr>
<tr>
<td>( minute )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>power required</td>
<td>0.02794</td>
<td></td>
<td>0.02769</td>
<td></td>
</tr>
<tr>
<td>( kwhr/m³ )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: The sign X indicates that a pump is on all day.
that the reduction of pump size can also decrease the required cooling time of the pump motors. (The reason for frequent starting and stopping of pump 3 in mode C is the flowrate valley from 5 to 9 am in the morning). On the other hand the outflow rate profile of mode C in Fig. 3.17 has a smaller magnitude of hydraulic surge than that of mode A. Furthermore, the power required in mode C is 6% less than that of control mode A.

An even greater improvement can be observed by comparing control mode D with mode B. In mode D, pump on-off times are reduced and minimum cycle time is increased, as compared to control mode B, in spite of the reduction in pump size, and consequently the required minimum cycle time of the pumps is reduced. The improvement can also be found in the outflow profiles (see Fig. 3.18), since both the magnitude and frequency of hydraulic surges are reduced. The power required in control mode D is 2% less than in mode B.

Using fixed speed pumps, amplification of both the frequency and magnitude of hydraulic surges is unavoidable, which can be observed from Fig. 3.17 and 3.18. (The dotted curves are the influent). Comparing control modes C and D based on the two figures, the former has a greater magnitude of surges than the latter, but lower frequency.

By comparing control modes A and B with C and D, it seems that increasing pump numbers and reducing pump sizes are a good measure to handle a large variation of influent
Fig. 3.18 Outflow Rate Profile
For Control Mode D

Fig. 3.17 Outflow Profile
for Control Mode C; 4 Pumps
flow rate, to reduce the magnitude and frequency of the hydraulic surges to the treatment system, and to increase energy utilization. If a combination of different size pumps is employed, a better outflow profile may be obtained. However, this may be offset by the advantages of using identical size pumps in a pumping system, namely a reduction in the number of spare parts needed, as well as simplification of maintenance and repair.

3.2.2 A Dynamic Model for Optimum Control

The purpose of control of a fixed speed pumping system is the reduction of pump on-off times (increasing the minimum cycle time of the pumps) so that hydraulic surges can be minimized, the life time of pumps can be increased, and their power utilization can be enhanced. The efficiency of power utilization consists of three parts: the pump efficiency, the lift head of the water, and the motor efficiency. The power used without considering the motor efficiency is called the power required by pumps. It takes the form

\[ P_i = S \times Q \times \frac{H_{td}}{E_{ff}} \quad (3.11) \]

where

- \( P_i \) --- power input or power required (kw/hr)
- \( S \) --- specific weight of water (kN/m\(^3\))
- \( Q \) --- capacity of pump (m\(^3\)/hr)
\( H_{td} \) --- total dynamic head (m)

\( \text{Eff} \) --- pump efficiency

For fixed speed pumps, the variable \( \text{Eff} \) in (3.11) does not change much because once the pump starts it almost always works close to its BEP. Attention must be focused on the total dynamic head \( H_{td} \), which is mainly composed of static head, friction head and velocity head in the piping system. For most wastewater pumping stations, the pumping system has a comparatively low lift head and friction head. Therefore the potential energy saving is in reducing the static head in \( H_{td} \) for equation (3.11).

Minimization of static head is expressed as: (No. 1)

\[
\min \quad Z_{1t} = (H_{up} - H_t)^2 \tag{3.12}
\]

where

\( H_{up} \) --- top limit of water level in wet-well

\( H_t \) --- water level in wet-well at time \( t \)

\( Z_{1t} \) --- squared minimum distance between top limit and dynamic water level at time \( t \)

Another objective (No.2) is to minimize the changes in outflow rate:

\[
\min \quad Z_{2t} = (F_{t-1} - F_t)^2 \tag{3.13}
\]

where

\( F_{t-1} \) --- outflow rate at time \( t-1 \) (at previously sampling time)
\[ F_t \quad \text{--- outflow rate at time } t \]
\[ Z_{2t} \quad \text{--- squared difference between outflow rate at time } t \text{ and } t-1 \]

For fixed speed pumps the outflow rate at time \( t \) can be written in the form of a combination of pump capacities

\[ F_t = Q_{1t}X_{1t} + Q_{2t}X_{2t} + \ldots + Q_{jt}X_{jt} \quad (3.14) \]

or

\[ F_t = \sum_{j=1}^{k} Q_{jt}X_{jt} \]

where

\[ j \quad \text{--- pump numbers } (j=1, 2, \ldots, k) \]
\[ Q_{jt} \quad \text{--- average outflow rate of } j-\text{th} \text{ pump at time from } t-1 \text{ to } t \]
\[ X_{jt} \quad \text{--- state variable of } j-\text{th} \text{ pump at time from } t-1 \text{ to } t \]

It should be noted that for fixed speed pumps the state variables of the pumps can only take two possible values: pump On defined as state 1 and pump Off defined as state 0.

Based on equation (3.4), the dynamic water level \( H_t \) in the wet-well in objective function (Eq.3.12) can be written

\[ H_t = H_{t-1} + (F_{it} - \sum_{j=1}^{k} Q_{jt}X_{jt})T_{deta/A} \quad (3.15) \]

where

\[ F_{it} \quad \text{--- mean influent flowrate from } t-1 \text{ to } t \]
\[ A \quad \text{--- area of wet-well} \]
In order to combine two different objective functions into a single function, an energy weighting factor $\alpha$ and a flow smoothing factor $B$ are introduced. By combining equations 3.15, 3.16 and 3.17 the state transform function for any sampling time can be written

$$\min Z_t = \alpha^* (H_{up} - (H_{t-1} - (F_{it} - \sum_{j=1}^{k} Q_{jt} \cdot X_{jt}) \cdot T_{deta}/A))^2 \nonumber$$

$$+ B^* (F_{t-1} - \sum_{j=1}^{k} Q_{jt} \cdot X_{jt})^2$$

(3.16)

For a single day the objective function (Eq.3.16) has the form

$$\min Z_d = \sum_{i=1}^{n} \left( \alpha^* (H_{up} - (H_{t-1} - (F_{it} - \sum_{j=1}^{k} Q_{jt} \cdot X_{jt}) \cdot T_{deta}/A))^2 + B^* (F_{t-1} - \sum_{j=1}^{k} Q_{jt} \cdot X_{jt})^2 \right)$$

(3.17)

where

$$i \quad \text{--- i-th sampling interval during a day}$$

$$\quad (i=1, 2, \ldots, n)$$

For the equations 3.16 and 3.17 at any sampling interval during a day, the following hard constraints apply:

$$H_{t-1} + \left( F_{it} - \sum_{j=1}^{k} Q_{jt} \cdot X_{jt} \right) \cdot T_{deta}/A \leq H_{up}$$

(3.18)
\[ H_{t-1} + (F_{it} - \sum_{j=1}^{k} Q_{jt}X_{jt}) \times T_{deta}/A \rightarrow H_{down} \quad (3.19) \]

\[ \sum I_{j0} \geq M_{S} \quad (3.20) \]

\[ X_{jt} = 0, 1 \quad (3.21) \]

where

- \( T_{deta} \) --- time interval between \( t \) and \( t-1 \)
- \( H_{down} \) --- lower limit of water level in the wet-well
- \( H_{up} \) --- upper limit of water level in the wet-well
- \( I_{j0} \) --- the time interval from \( t-1 \) to \( t \) of \( j \)-th pump at state 0 (off or stopping state)
- \( M_{S} \) --- minimum stopping time, which is decided by the capacity of pump to cool itself off

The constraint given in Eq.3.20 states that for each pump in operation, once it stops, the minimum time for cooling should be provided. If a combination of different size pumps is used, the cooling time changes with the pump sizes. For the traditional two point control it is very difficult to obey this condition, especially when the variations of influent are large and unknown. Therefore, equation 3.22 cannot be a hard constraint in practice.

As mentioned in section 3.2.1.1, it is desired that the set points in the wet-well can be dynamically changed, based on the influent flow rates at each sampling time or sampling interval, so that the best use of wet-well volume as well as both minimizing the static head and pump on-off times can be
realized. The dynamic models (Eq. 3.16 through 3.21) may be used to solve for an optimum control strategy of fixed speed pumps.

In practice, the water levels in the wet-well are usually measured instead of the flowrates, since the instruments for measuring water levels are less expensive and more reliable than flowrate meters. The control models developed above are based on the measurements of both the water levels and the flowrates. It is therefore desirable to give the forms of the above control models based only on the measurement of water level in the wet-well. The mean average influent flowrate $F_{it}$ from time $t-1$ to $t$ in Eqs 3.15 through 3.19 may be approximated by:

$$F_{it} = (H_{t-1} - H_{mt}) A/T_{deta} + F_{t-1} \quad (3.22)$$

where

$H_{mt}$ --- measured water level in the wet-well at time $t$

$F_{t-1}$ -- outflow rate at time $t-1$, which is provided by pumps at time $t-1$

Introducing equation 3.22 into equations 3.16 through 3.19 respectively, the following equations are obtained based on the measurement of water levels:
\[
\begin{align*}
\min Z_t &= a*(H_{up} - [2H_{t-1} - H_{mt} + (F_{t-1} - \sum_{j=1}^{k}(Q_{jt}*X_{jt}))^2 + b*(F_{t-1} - \sum_{j=1}^{k}Q_{jt}*X_{jt})^2]^{1/2} + \sum_{i=1}^{n}a*[H_{up} - (2H_{t-1} - H_{mt} + (F_{t-1} - \sum_{j=1}^{k}Q_{jt}*X_{jt}))^2 + b*(F_{t-1} - \sum_{j=1}^{k}Q_{jt}*X_{jt})^2]^{1/2} \quad (3.23)\\
\min Z_d &= \sum_{i=1}^{n}a*[H_{up} - (2H_{t-1} - H_{mt} + (F_{t-1} - \sum_{j=1}^{k}Q_{jt}*X_{jt}))^2 + b*(F_{t-1} - \sum_{j=1}^{k}Q_{jt}*X_{jt})^2]^{1/2} + b*(F_{t-1} - \sum_{j=1}^{k}Q_{jt}*X_{jt})^2) \quad (3.24)
\end{align*}
\]

The constraint equations are:

\[
2H_{t-1} - H_{mt} + [F_{t-1} - (\sum_{j=1}^{k}Q_{jt}*X_{jt})]*T_{deta/\Lambda} \leq H_{up} \quad (3.25)
\]

\[
2H_{t-1} - H_{mt} + [F_{t-1} - (\sum_{j=1}^{k}Q_{jt}*X_{jt})]*T_{deta/\Lambda} \Rightarrow H_{down} \quad (3.26)
\]

Equations 3.16 through 3.21 and 3.23 through 3.26 are a nonlinear, 0-1 integer programming problem. The combination of the solutions is 2^m, where m is the number of pumps. There are many methods available to solve these kinds of discrete equations (26,28). In this study the Implicit Enumeration Method (27) was applied. For feedback control based on measurement of the influent flow rate, the state equation 3.16 can be used and transformed from each sampling interval along with the constraint Eqs. 3.18 through 3.21. In the following section this kind of control will be discussed and the models will be checked. If the above models are used for open loop control using the predictions
of the ARIMA model, the model shown in Eq.3.17 can be used to pre-set the pump working mode. The control strategy for control of a fixed speed pumping station will be discussed in section 3.2.4.

3.2.3 Optimum Control of Fixed Speed Pumps

In this section the simulation conditions, such as the influent flow rate, pump curves, pump size and numbers, and sampling time, are the same as section 3.2.1, only the type of controller used is changed. A simple feed back control loop with a computer that can use a high level language (FORTRAN, BASIC, etc) is applied. The input signal is the influent flow rate $F_{it}$, equation 3.16 is solved along with the constraint equations, finally the 0-1 states ($X_{jt}$) of the pumps are decided so that the optimum outflow rate at each time $t$ is determined.

It should be mentioned that the energy weighting factor $\alpha$ and outflow smoothing factor $\beta$ were given the values of 0 and 1 respectively (see Table 3.3) in the following simulation; that is, in the simulation emphasis is placed on optimizing the outflow profile and reducing the on-off times of pumps. The relationship between the two weighting factors in the objective function 3.16 and 3.17 will be discussed in 3.2.4.

Table 3.3 shows the results of the simulation for 2 pumps in parallel with optimum control. Compared with the


<table>
<thead>
<tr>
<th>Optimum Control</th>
<th>2 Pumps</th>
<th>4 Pumps</th>
</tr>
</thead>
<tbody>
<tr>
<td>energy weighting factor $\alpha$</td>
<td>$\alpha = 0.0$</td>
<td>$\alpha = 0.0$</td>
</tr>
<tr>
<td>flow smoothing factor $B$</td>
<td>$B = 1.0$</td>
<td>$B = 1.0$</td>
</tr>
<tr>
<td>pump No.</td>
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<td>2</td>
</tr>
<tr>
<td>on-off cycles</td>
<td>15</td>
<td>0</td>
</tr>
<tr>
<td>min. cyc. time</td>
<td>55 (minute)</td>
<td></td>
</tr>
<tr>
<td>power required</td>
<td>0.03 (kwhr/cu.m.)</td>
<td></td>
</tr>
</tbody>
</table>

(2 pumps)

<table>
<thead>
<tr>
<th>Mode A</th>
<th>Mode B</th>
</tr>
</thead>
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<tr>
<td>set point</td>
<td>LL</td>
</tr>
<tr>
<td>(m)</td>
<td>0.5</td>
</tr>
<tr>
<td>pump No.</td>
<td>1</td>
</tr>
<tr>
<td>on-off cycles/d</td>
<td>17</td>
</tr>
<tr>
<td>min. cycle time</td>
<td>46 (minute)</td>
</tr>
<tr>
<td>power required</td>
<td>0.0296 (kwhr/m³)</td>
</tr>
</tbody>
</table>

(4 pumps)

<table>
<thead>
<tr>
<th>Mode C</th>
<th>Mode D</th>
</tr>
</thead>
<tbody>
<tr>
<td>set point</td>
<td>L1 L2 M H2 H1</td>
</tr>
<tr>
<td>(m)</td>
<td>0.5 1.75 3.0 4.25 5.5</td>
</tr>
<tr>
<td>pump No.</td>
<td>1</td>
</tr>
<tr>
<td>on-off cycles/d</td>
<td>X X 28 16</td>
</tr>
<tr>
<td>min. cycle time</td>
<td>X X 28 40</td>
</tr>
<tr>
<td>power required</td>
<td>0.02794 (kwhr/m³)</td>
</tr>
</tbody>
</table>
ordinary two point control modes A and B in 3.2.1.1, it can be seen that the pump on-off times or the outflow profile is improved. For control mode A, although the optimum control only reduces pump starting by two times, it starts and stops one pump and the other is in the on state all day, so the magnitude of hydraulic surges is substantially reduced. Compared with control mode B, the optimum control can reduce the pump on-off frequency by 57 % per day. Further improvement can be found by increasing the number of pumps to four. Table 3.3 and Fig. 3.19 show the on-off times of pumps 2,3,4, and the outflow profile for the optimum control. Here the comparisons are carried out only by comparing pump 3 and 4 on-off times with control mode C and D. In contrast with control mode C, the reduction of pump 3 and 4 on-off times during a day in the optimum control are 86 % and 69 % respectively. Comparing control mode D with the optimum control on pump 3 and 4, the reduction of pump on-off cycles is 67 % and 81.5 %. Also, by comparing Fig.3.19 with Figs. 3.17 and 3.18, both the magnitude and frequency of the hydraulic shocks in the effluent are greatly reduced.

As mentioned above, all the comparisons are only focused on the reduction of pump on-off times, which is a significant problem encountered in the day to day operation of pumping stations using fixed speed pumps. The significance of minimizing the on-off times of the pumps is
Fig. 3.19 Outflow Profile for 4 Pumps with Optimum Control

Outflow Rate (cu.m/sec.)

Time (min.)

- Outflow Rate
- Influent Flowrate
to extend the pump life and reduce maintenance, as well as to reduce the magnitude and frequency of hydraulic surges. However, it should be noted from Table 3.3 that the power required by the pumps under the optimum control is the greatest instead of the smallest compared with control modes A, B, C and D. The reason is, as mentioned at the beginning of this section, that the energy factor $\alpha$ is set to zero. Therefore, it is interesting to see how the two objectives can be weighted in order to consider both energy consumption and hydraulic surges.

3.2.4 Sensitivity Analysis

The objective of power reduction is in conflict with the objective of reduction of the number of pump on-off cycles, that is, if the power required by pumps can be minimized, the pump starting and stopping times will be increased in order to keep the water level in the wet-well as high as possible, and will consequently result in an increased number of hydraulic surges. The purpose of this analysis is to learn how sensitive the power requirement and pump on-off frequency are to variations in the energy weighting factor and outflow smoothing factor; and what the suitable values for combining the two weighting factors are.

The analysis was carried out by simultaneously changing the energy weighting factor $\alpha$ from 0 to 1 and outflow weighting factor $B$ from 1 to 0; the ratio of the two factors
\( \alpha/\beta \) is plotted versus power reduction and pump on-off times during a day. Fig. 3.20 shows the relationship between the power reduction and the ratio of weighting factors for the control of two pumps. At low ratios (0 to 0.25; \( \alpha=0.2, \beta=0.8 \)), the power cannot be reduced. A sharp increase of power reduction occurs in the range of 0.42 (\( \alpha=3, \beta=0.7 \)) to 1 (\( \alpha=0.5, \beta=0.5 \)). Referring to Fig. 3.21, the on and off times of pump 2 within this range are not very sensitive to the changes of the ratio, that is, the on-off times during a day do not increase significantly. Therefore the best combined values for \( \alpha \) and \( \beta \) are within this range. However, if the ratio value is higher than 0.66 (\( \alpha=0.4, \beta=0.6 \)) the soft constraint equation \( (3.20) \) cannot be satisfied; that is,

**Fig. 3.20 Power Reduction vs. Ratio of Weighting Factors (2 Pump in Parallel)**

![Graph showing power reduction vs. ratio of weighting factors](image)

---

\( \text{Ratio of Weighting Factors} \)

---

**Power Reduction**

---

**Upper Limit**
the minimum pump off time for cooling is not guaranteed. Ratio values above 0.66 are not feasible solutions for the optimum control, but may be possible in practice. It is interesting to observe the limits of power reduction and pump on-off times with the ratio of the two factors by relaxing the soft constraint (Eq.3.20). Even if the power reduction factor $\alpha$ is increased, ($\alpha$ value from 0.5 to 1 while $B$ value from 0.5 to 0,) the power reduction curve increases slowly and finally reaches its limit of 9.7 %, while the pump on-off time curve increases significantly and also reaches the limit. If the best ratio value is chosen as 0.66 ($\alpha=0.4, B=0.6$), the power reduction is 3.5 %, and the on-off cycles of pump 2 are 28. Compared with control mode B
with energy reduction of 4.8% and on-off cycles of 35 for pump 2, there is not much improvement for the optimum control of two pumps.

Fig. 3.22 and 3.23 show the relationships among the ratio of weighting factors, pump on off times and power reduction for the optimum control of 4 pumps. The plots indicate that the ratio of weighting factors is quite sensitive to power reduction in the range of 0.25 to 0.66 ($\alpha=0.1$, $\beta=0.9$ to $\alpha=0.4$, $\beta=0.6$), and is not so sensitive to the pump on-off times in this same range. Again if the ratio is greater than 0.66, there are no optimum solutions because the soft constraint equation 3.20 cannot be satisfied. If the best value of the ratio is taken as 0.42 ($\alpha=0.3$, $\beta=0.7$),

**Fig.3.22 Power Reduction vs.Ratio of Weighting Factors (4 Pumps)**

![Graph showing power reduction vs. ratio of weighting factors](image-url)
the corresponding power reduction is 5 %, and the number of operating cycles for pump 3 and 4 are 7 and 15 respectively. Compared with control mode D (Table 3.2; for control mode D, each pump stops at its lower set point), the power reduction is the same but the on-off times are greatly reduced.

Based on the above analysis, the conclusions are:
1) the more pumps in an optimum control system, the better the results which can be achieved; if there are only two pumps in a system the optimum control may be the same as control mode B (for control mode B, each pump stops at its lower set point); 2) although the reduction of pump size and increase of pump numbers in a pumping system can result in
the relaxation of the soft constraint of minimum cycle time and a reduction of on-off times, from the standpoint of energy saving, the reduction of power required will reach an upper limit; 3) The energy factor α becomes more significant when the ratio of depth of wet-well to the lift head and friction head is large; 4) the best α and β values lie in the range in which the power reduction is sensitive to the ratio of α to β, and pump on-off times are not sensitive in the same range.

3.2.5 Control Strategy

In a pumping system, if the number of pumps is large and the sampling interval is short, say 15 or 30 seconds, for a feedback control system the computer will take time to solve Eqs. 3.16 and 3.18 to 3.21 in order to decide the state of each pump. This could cause a time delay in executing the control actions by the regulator. To solve this problem, a control strategy was developed with a feedforward loop, plus feedback trim to adjust the inadequacy of the model on which feedforward is based. Using equation 3.2, the flowrates are predicted for the feedforward control 24 hours in advance. The computer then can solve Eqs.3.17 to 3.21 and decide the state of each pump at every sampling interval based on the estimated flow rates. However, since the ARIMA model 3.2 is not accurate for long term prediction, feedback trim based on the input
signal of water level will be needed to adjust the pre-set states of the pumps when the upper or lower limit levels in the wet-well are exceeded. The control strategy can be carried out by a local control loop with a computer.

Fig. 3.5 shows the predicted flowrate for 24 hours by using equation 3.2 on April 5, 1990, for Interbay Pumping Station in Seattle. The working states of the pumps (4 pumps in parallel) for the day are pre-decided based on the predictions. The corresponding outflow rate is predicted as in Fig. 3.24. Adjustments were made whenever the real measured water level exceeded the up or down limit in the wet-well (two set points) when the pre-decided pump modes were executing. The pumps start or stop and override the decided states when the above cases occur. Again a soft constraint was applied in that when the pump stops, it must remain in 0 state until it cools sufficiently. Fig. 3.25 shows the actual outflow rate profile after the adjustments were made. Once the adjustments are made the pumping system no longer keeps the optimum state until the period of change is over. However, in comparing the outflow profile with that of the two point control, improvement is observed.

Table 3.4 compares the two control strategies using the optimization developed in 3.2.2. It should be noted that the number of adjustments in the control strategy of feedforward plus feedback trim greatly depends on the accuracy of the
Fig. 3.24 Pre-decided Outflow Profile for Feedforward Plus Feedback

Fig. 3.25 Adjusted Outflow Profile for Feedforward Plus Feedback Trim
<table>
<thead>
<tr>
<th>Simple Feedback</th>
<th>Feedforward + Feedback Trim</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \alpha = 0.0 )</td>
<td>( \alpha = 0.0 )</td>
</tr>
<tr>
<td>( \beta = 1.0 )</td>
<td>( \beta = 1.0 )</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>pump No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>on-off cycles</td>
<td>5</td>
<td>4</td>
<td>1</td>
<td>0</td>
<td>7</td>
<td>4</td>
<td>1</td>
<td>1</td>
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<tr>
<td>adjustments</td>
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<td>0</td>
<td>0</td>
<td>0</td>
<td>9</td>
<td>2</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>min.cyc.time (minute)</td>
<td>97</td>
<td>121</td>
<td>22</td>
<td>27</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>power required (kwhr/cu.m)</td>
<td>0.0293</td>
<td>0.0296</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Prediction, since the storage capacity of the wet-well is quite small. Even small errors between the predicted and real flow rates will cause the water level to exceed the upper and lower limits in the wet-well. From this point of view, the feedforward plus feedback control with 24 hour predictions of influent flow rates may not be a feasible strategy for the control of fixed speed pumping stations. However, increasing the number of pumps is one way to solve this problem since small pumps require shorter periods of cooling time. The other method is to shorten the prediction time, since the model (Eq.3.2) is more accurate for short term prediction.
3.3 Conclusions and Recommendations

1. For traditional two point control, the wet well volume, pump size and number of pumps can strongly influence the pumping system. Increasing pump sizes and reducing pump numbers can result in large hydraulic surges. Decrease of wet well volume will result in frequent pump on-off cycles.

2. Since large hydraulic surges are undesirable in wastewater treatment plants, control modes in which each pump starts at its upper set point and sequentially stops at its lower set point are better than control modes in which all pumps stop at the lowest set point in the wet well. However, the former control modes increase the frequency of the surges.

3. Increasing the number of pumps is a good way to handle large fluctuations in influent flowrate and to provide a relatively smooth outflow. However, it will make the control system more complicated and may decrease overall efficiency since small pumps have lower efficiencies than large pumps.

4. Energy savings can be achieved by keeping the difference between the dynamic water level and the upper limit set point in the wet-well as small as possible during operation. The energy savings increase as the ratio of the lift head to the depth of the wet-well decreases.
5. Two point control does amplify the magnitude and frequency of hydraulic surges. However, control using the dynamic models developed can substantially reduce pump on-off cycles (up to 86 % for a typical day) as well as reduce the man-made hydraulic surges both in magnitude and frequency.

6. Feedforward plus feedback trim with a 24 hour prediction of influent flow rate may not be a feasible strategy for controlling fixed speed pumping systems since only limited storage capacity is provided by the wet-well. Inaccurate predictions may result in too many adjustments for the pre-set working states of the pumps. Since ARIMA models are best for short term predictions, it is suggested that a one hour or less leading prediction should be used in the feedforward loop.

7. The dynamic models developed in this study should be further tested before being applied to practice.
Chapter 4
SIMULATION AND CONTROL OF EQUALIZATION BASINS

Two of the design criteria for wastewater treatment plants are the hourly peak flowrate and organic loading rate. However, since the peak flows only occur for a few hours a day, the capacity of the treatment facilities may not be fully used for the remainder of the day. For conventional treatment plants, peak flows must be handled hydraulically, which consequently results in a higher capital cost compared with the handling of daily average flows.

Hydraulic and organic surges due to these hourly variations also cause difficulties in the operation of treatment plants. Temporary overloading can have a deleterious influence on treatment efficiency and hence on the receiving bodies of water. LaGrega and Keenan (2) have shown, in a full-scale experiment, that fluctuations of influent flow rate can have a significant impact on the performances of primary as well as secondary settlers, and on the quality of the final effluent. This same phenomenon has been observed by other researchers (31, 32, 33, 34). If the wastewater can be stored during peak flows and released during low flows, then both flow rate and organic loading will approach more constant values with respect to time. The
storage tank for accomplishing this is referred to as an equalization basin.

Dold (6) developed a model and control strategy for equalization basins and applied these to a full scale treatment plant. The diurnal fluctuations in influent flow rate were damped out, and improvements were observed in the performance of the primary and secondary settlers. Foess, et al (33) did a case study of the Walled Lake/Novi treatment plant, Michigan, which had a treatment capacity of 2.1 MGD. Their equalization system was effective in reducing influent flow variations but not in reducing concentration variations. They found that equalization did not significantly improve effluent quality for the secondary clarifier but did enhance the performance of the trickling filters. As concluded by Ongerth (34), after reviewing and evaluating several applications of equalization basins for domestic wastewater treatment plants in the U.S, flow equalization should be considered as an alternative to additional treatment capacity wherever the ratios of peak to average flow exceed levels of 1.3 to 1.5:1.

In summary, the benefits (34) from the application of flow equalization can be:

* equalization basins may be used in new treatment facilities to assist in the maintenance of the required effluent quality where advanced and sensitive treatment processes are included, to minimize peaking capacity of
treatment components, to permit process optimization, and to simplify operation.

* equalization may help overcome the design deficiencies and reduce operational problems for existing treatment facilities.

The objectives of this study are:

* to explore the interactions between design and operation of equalization basins using computer simulation;

* to simulate ordinary control modes, such as feedback PID control and feedforward plus feedback trim;

* to develop an optimum control strategy which will provide minimum hydraulic and organic variations to downstream processes.

Only typical dry weather flowrates are used as inputs to the equalization basin models and storm flow rates are not considered. The flowrate data is taken from the Seattle Interbay Pumping Station without storm flows. Since there was no concentration data available, the BOD$_5$ concentration data were taken from an EPA report on Evaluation of Flow Equalization in Municipal Wastewater Treatment (5).

4.1 Models

There are three major components in an equalization process: the influent flow rate, the equalization basin and
the control system. The models for the components will be introduced in this section.

4.1.1 Influent Flow Rate

The influent flowrate model (Eq.3.1 or 3.2, Chapter 3) is used in this chapter. The details of the identification of the flowrate model for the Seattle Interbay Pumping Station are given in Chapter 2.

The dry weather flowrate from March 30 to April 3, 1990 with an hourly average value is selected from a three week series of data. Using Eq.3.2, the flowrates for each hour and mean flowrates for each day were predicted one hour ahead and are shown in Fig.4.1 and Fig.4.2, respectively.

**Fig.4.1 Flowrate (3/30 to 4/3/1991)**
by 1 hour leading prediction
The predicted curve matches the measured flowrate curve with a relative error of less than 3%. Inaccurately predicted flowrates can be compensated for by the feedback control loop. This point will be addressed in later sections.

4.1.2 Equalization Basin

The hydraulic models for the equalization basin are the same as the wet-well models used in Chapter 3. Eqs.3.3 and 3.4 are employed herein.

For modeling the concentration of substances, the assumptions are made that the basin is completely mixed and that the substance is inert, i.e., does not participate in any reactions in the basin. For solids, it is also assumed
that no sedimentation takes place. Based on the flow and volume response equation 3.3, the corresponding effluent concentration can be derived from a material balance

\[ \text{Fin} \times \text{Cin} = \text{Fout} \times \text{Cout} + \frac{d(\text{Cout} \times V)}{dt} \]  

(4.1)

where

\begin{align*}
\text{Cin} & \quad \text{concentration in influent} \\
\text{Cout} & \quad \text{concentration in effluent}
\end{align*}

By substituting \( \frac{dV}{dt} = \text{Fin} - \text{Fout} \) into eq. 4.1, we obtain

\[ \text{Fin} \times \text{Cin} = \text{Fout} \times \text{Cout} + V \times \left( \frac{d\text{Cout}}{dt} \right) + \text{Cout} \times (\text{Fin} - \text{Fout}) \]

Therefore

\[ \frac{d\text{Cout}}{dt} = \frac{\text{Fin} \times (\text{Cin} - \text{Cout})}{V} \]  

(4.2)

Equation (4.2) can be further written in difference equation form

\[ \text{C}_t = \text{C}_{t-1} + \frac{\text{Fin} \times (\text{C}_{\text{int}} - \text{C}_t)}{V_t} \times T_{\text{delta}} \]  

(4.3)

where

\begin{align*}
\text{C}_t & \quad \text{tank concentration at time } t \\
\text{C}_{\text{int}} & \quad \text{effluent concentration at time } t \\
\text{C}_{t-1} & \quad \text{tank concentration at time } t-1
\end{align*}
4.2 Simulation and Control

The configurations of the equalization basins, the control modes and corresponding algorithms for each configuration are presented and discussed herein.

4.2.1 Equalization Basin Configurations

Basically, there are two types of arrangements of equalization basins: side-line and in-line, as shown in Fig.4.3. For the side-line basin, an overflow weir is used to split the flow between the downstream processes and the equalization tank. The height of the weir is fixed in the design, based on an estimate of the mean flowrate. Whenever the influent flowrate is greater than the design mean flowrate, or the water level is higher then the height of the weir, the excess flow will be diverted to the equalization basin, with the remaining flow continuing to the treatment processes. If the influent flow rate is less than the design mean flow rate, flow from the equalization basin will be pumped downstream to increase the flow rate to the treatment processes. Thus, a comparatively constant flowrate will be maintained and diurnal fluctuations will be damped. The equalization basin thus functions to store excess flow when the influent flowrate is higher than the design mean flowrate and to release flow when the influent flowrate is lower than the design mean flowrate.
Fig. 4.3  CONFIGURATIONS OF EQUALIZATION BASIN
The in-line type equalization basin has the same function as the side-line basin. The major difference is that the entire influent flow enters the equalization basin and the outflow rate is provided by pumps. However, the control strategies of the two systems can be significantly different. This will be addressed in more detail later.

It should be mentioned that variable speed pumps are often provided to withdraw the effluent from the equalization basin (34). However, if there is enough hydraulic head available, or there is a pumping station before the equalization basin, the effluent from the equalization basin can also be controlled by valves, or by motorized weirs. Olsson, (35) have recommended a model for a basin with weirs

\[ A\frac{dz}{dt} = F_{in} - N \alpha h^\beta \]  \hspace{1cm} (4.4)

where

- \( A \) --- the cross-sectional area of the tank
- \( N \) --- the number of weirs
- \( z \) --- the total water depth
- \( \alpha \) --- the weir coefficient and a function of the weir type and width
- \( \beta \) --- a weir coefficient depending on the weir type

(For example, it is 2.5 for a V-notch, 1.5 for a rectangular and 1.0 for a Sutro weir)
h --- the distance from the bottom of the weir to the water level.
The second term on the right side of Eq.4.4 represents the flow rate over the weirs.

One of the advantages of using valves or weirs for controlling the effluent of the equalization basin is to avoid the duplication of pumping units. Since sewers are usually quite deep where they enter the treatment plant grounds, it is usually not economical to construct an equalization basin with a large volume at such a depth from the ground surface. Instead, a pumping station will be built to lift the wastewater from the sewers to the equalization basin. If another pumping station is set after the equalization basin to regulate the effluent, costs will be increased because of the duplication of pumping stations. In the above case, tanks with movable weirs or with valves would be a better solution to the problem, since the pumps before the equalization tank can provide enough hydraulic head. In summary, the pumping units can be located either before or after the equalization tanks depending on the site conditions.

In the design process, the selection of side-line or in-line equalization basins also depends on the site and the purposes of equalization. Lessard (40) has shown that side-line basins are more suitable than in-line basins for equalizing stormwater flow rates. In-line basins are
frequently used for the damping of normal diurnal variations in flowrate. These conclusions have been reached by several researchers as well (2,32,34).

Air is usually provided to mix the tank and to prevent solids settling.

4.2.2 Determination of Equalization Volume

The volume of the equalization tank, or detention time in the tank, is a key factor in both the design and control of equalization tanks. A proper selected volume can be of great benefit to the control processes; and in contrast, optimum control can significantly reduce the required tank volume.

As mentioned before, wastewater flowrate is a highly dynamic and non-stationary process. Figs. 4.1 and 4.2 have shown that the influent does not have a uniform mean daily flowrate from day to day. Further, the magnitude of flow variations is larger on weekdays than on weekends, and the flowrate may change significantly from month to month as well as from season to season (33). This makes it difficult for the design engineer to select the daily mean flowrate to be used for determining the tank volume. Computer simulation can be a useful tool for determining the tank volume. Also, a lack of some of the knowledge needed for design can be compensated for by the control system.
To accommodate the difficulties in selecting the typical diurnal flow for volume determination, it is suggested by EPA (32,34), that the flow pattern selected yield a large enough basin to effectively equalize any reasonable dry weather diurnal flow. This often results in oversizing the equalization basin, which not only requires more capital investment, but can create odor problems as well. Moreover, the determination of equalization volume by design does not always consider the impact of the control system for the equalization tank. In fact, the control system for operation of the equalization process does have a significant effect on the required tank volume.

Basically, there are two methods for computing the equalization volume requirement as recommended by EPA (41). One procedure is based on the characteristics of the diurnal flow pattern. The second procedure is based upon the mass loading pattern of a particular constituent. Each of the methods can be further classified; for example, the method based on flowrate can be by graphical solution, tabular method, etc. The methods used depend upon their intended purposes: equalization of flow, or organic loads, or both.

For this research, the primary objective is to equalize flow and to examine the effects of different control modes on the performances of the equalization tank. Therefore the graphic technique is used herein. As mentioned before, because of the difficulty of selecting a typical dry weather
day, the graphs of cumulative volume for both a weekday (Friday, March 30) and a weekend (Saturday, March 31), as derived from Fig.4.1, are shown in Fig. 4.4 and 4.5. The dashed lines are the cumulative volumes in 24 hours, and the solid lines are the mean cumulative volume.

The required equalization volume can be found by summing the maximum differences between the solid lines and dashed lines on both sides of the mean cumulative curves. The required volumes and detention times for Friday are 3.1 MG and 1.7 hours with a mean flow rate of 48.6 MGD, and for Saturday are 2.6 MG and 1.4 hours with a mean flow rate of 45.2 MGD. There is a 19% decrease in the required volume on weekends from weekdays. However, these calculated detention

Fig. 4.4 Cumulative Volume for Flowrate
March 30, 1990 (Friday)
times (or volumes) seem much smaller than those used by other researchers (2,6,33). For example, LaGrge (2) has used 5.4 hours as the detention time for an equalization tank with a ratio of the maximum to the minimum flowrates equalling 2. According to Fig.4.1, the ratio of the maximum to the minimum flowrates for the Interbay Pumping Station is 2.54 (max. 56.6 and min. 22.3 MGD). The calculated volumes based on the two days' hydrographs in Fig.4.4 and 4.5 may not provide enough capacity to equalize the flows. Another attempt was made to use all 5 days of the flow data to determine the equalization volume, since these data include both the flow rates on weekdays and on weekends. The
required volume is calculated as 10.4 MG, and detention time is 5.5 hours with a mean daily flow rate of 45.3 MGD.

It is worthwhile to point out that if there are enough flow data and computing power available, the whole data series should be taken into consideration to determine an appropriate equalization volume and satisfy the requirements of EPA in the design phase.

4.2.3 Simulation and Control of Side-Line Equalization Basin

Side-line equalization basins are usually applied to equalize storm water, and are used in wastewater collection systems. However, in some cases side-line equalization basins are used in wastewater treatment plants to serve the purpose of damping diurnal variations of flowrate. The purposes of the study in this and the next section are to investigate the interactions between the design and control for side- and in-line equalization basins; to compare the two types of configurations, and to determine which one is the better option for equalizing diurnal flow variations. In this section, feedback control and feedforward plus feedback control for side-line equalization are simulated using SIMNON.
4.2.3.1 Feedback Control

As mentioned in section 4.2.1, the flowrate to the equalization tank at the diversion weir, or divider, is uncontrollable, since the height of the weirs is fixed based on the design daily mean flowrate. The only control variable is the effluent flowrate, which is provided by variable speed pumping units. Fig.4.6 shows feedback control loop and corresponding computer algorithm. There are two measurements: one is the influent flowrate ($F_{it}$) to the treatment plant, which is provided by the flow meter; the second is the the water level in the equalization basin. At each sampling time $t$, the influent flowrate is measured and compared with the design mean flowrate ($F_{mean}$). If $F_{it}$ is greater than $F_{mean}$, overflow occurs at the diversion weirs and the diverted water enters the tank ($F_{it}-F_{mean}$), while the effluent from the tank will be stopped in order to maintain $F_{mean}$. If $F_{it}$ is less than $F_{mean}$, there is no flow to the equalization tank, and the pumps will be started to provide the effluent to the mixing well to compensate for the deficiency between $F_{it}$ and $F_{mean}$. The signals for both the influent to ($F_{eit}$) and effluent from ($F_{eet}$) the equalization tank are fed to the controller to calculate the tank volume and concentration responses. These calculated values will be checked by the measurement of the water level in the tank. Whenever the water level falls outside of the
Fig. 4.6 Side-line Feedback Control Diagram
range of 5% to 95% of the depth of the tank, the effluent flowrate will be adjusted in order to prevent the tank from overflowing or emptying. The adjusted outflow rate from the tank will be fed back to the controller and the volume and concentration are calculated again. The water level limits (5% for lower and 95% for upper set points, respectively) are controlled using the following equations

\[ F_m = (V_t - 0.95*V)K_{up}/T_{deta} \]  \hspace{1cm} (4.5) \\
\[ F_p = (0.05*V - V_t)K_{down}/T_{deta} \]  \hspace{1cm} (4.6)

where

- \( V \) --- total volume of the equalization tank
- \( V_t \) --- dynamic volume at time \( t \)
- \( T_{deta} \) --- sampling time interval
- \( F_m \) --- outflow rate for control of overflowing equalization basin
- \( F_p \) --- outflow rate for control of emptying equalization basin
- \( K_{up} \) --- penalty factor for the upper limit
- \( K_{down} \) --- penalty factor for the lower limit

The two penalty factors control how fast the upper and lower limits of the tank volume can be recovered if overflowing or emptying of the tank occurs. The bigger the \( K_{up} \) and \( K_{down} \) values, the faster the recovery speeds; therefore, larger magnitudes of flow variations are produced. Eqs.4.5 and 4.6
are used throughout the control of the equalization basins. They function as hard constraints to maintain the volume limits.

The simulations are carried out using several tank volumes. As calculated in section 4.2.2, the equalization volumes (or detention time) are taken as 39,400 m³ (10.4 MG, d.t.=5.5 hr), 18,000 m³ (4.76 MG, d.t.= 2.5 hr), and 14,380 m³ (3.8 MG, d.t.=2.0 hr) respectively. The mean daily average flowrate for calculating these detention time is 172,000 m³/d (45.3 MGD).

For a detention time of 5.5 hr, Fig. 4.7 shows the original influent and the equalized flowrate. The outflow curve is quite smooth, but there is a overflow period from 23 to 30 hours (Saturday morning), that is, the flow from the diversion weirs bypassed the equalization tank during that period since the tank was already full and could not provide any damping capacity. In Fig.4.8, the dashed lines are the upper and lower limit set points for feedback control of the tank volume. The tank is almost empty for several hours, since from 60 to 80 hours (Sunday) the mean flowrate is much smaller than the daily average flowrate. From Fig.4.8, it is obvious that the tank volume is fully utilized during the five days, but is not used uniformly for each day. From the design point of view, the calculation of the required volume is correct for this simple feedback control mode. However, there is still potential for further
Fig. 4.7 Outflow after Equalization

d.t. = 5.5 hr; \ K_{up}=K_{down}=1.0

Fig. 4.8 Variation of Tank Volume

d.t. = 5 hr; \ K_{up}=K_{down}=1.0
reduction of the tank volume by applying advanced control strategies. This will be discussed in the optimal control section. In Fig. 4.9, the solid line shows the BOD load to the downstream processes after equalization, and the dashed line represents the BOD load without equalization. Comparing the two loading curves, there is no significant equalization of the BOD loading.

Since volume significantly affects equalization processes, the tank volume will be reduced to one half of the previous volume. (4.76 MG with a detention time of 2.5 hours). Fig.4.10 displays the influent to the treatment plant (dashed curve) and the outflow curve (solid curve) after equalization. The diurnal variations are greatly
Fig. 4.10 Outflow After Equalization
Detention Time 2.5 hours

Fig. 4.11 Variation of Tank Volume
d.t. = 2.5 hr; K_up = K_down = 1.0
damped. However, Fig.4.11 shows that the flow bypassed the tank for almost two days, and that it was empty for more than one day. It should be noted that all the bypassed flows are pumped to the mixing well. Energy is required for this configuration whenever an overflow occurs in the tank. The reason is that the height of the diversion weirs is fixed, and once the water level in the diversion channel is higher than the weir level, the flow will enter the equalization tank; that is, the flow into the tank is uncontrollable. From an energy conservation point of view, the pumping units are better set before the equalization basins so that the lifting of the overflow can be avoided when the basin is already full.

In all of the above simulations, the penalty factors $K_{up}$ and $K_{down}$ in Eqs. 4.5 and 4.6 are given the values of 1; that is, there is no penalty when overflow or emptying occurs. Fig.4.12 shows the effects of penalty factors on the outflow rate profile and the volume percentage profile by changing $K_{up}$ and $K_{down}$ from 1 to 1.2. Comparing this plot with Fig.4.11 in which $K_{up}$ and $K_{down}$ are given 1, the volume limits are well maintained by the higher values of the factors. However, the magnitude of the change in outflow rate is increased. Consequently, the penalty factors have significant impacts on the control process. Chosing the values for the factors will depend on the reliability and
degree of control of overflowing or emptying the basins allowed in the process operation.

Comparing the results of two given volumes for the equalization tank using feedback control, there is a trade off among the capital investment, efficiency of treatment processes, operation cost, odor problems, energy conservation and operation reliability. A larger tank volume may provide a smoother flowrate profile to the subsequent treatment processes. However, it also requires greater capital investment, and may create odor problems due to the longer detention time.
4.2.3.2 Feedforward Plus Feedback Control

In practice, the most common control measurement for an equalization tank is the water level in the basin since the measurement of water level is reliable and inexpensive. If a programmable logical controller (PLC or a small computer) is applied to a control loop, flowrate into the equalization basin can be calculated by the water level changes in the tank using an equation

$$Fe_{t} = (H_{t} - H_{t-1})A/T_{dela} + Fe$$

(4.7)

where

$Fe_{t}$ --- mean flowrate into the tank from time $t-1$ to $t$

$H_{t-1}$ --- the water level at $t-1$

$Fe$ --- flowrate out of the tank from $t-1$ to $t$, (it should be mentioned that $Fe$ is sometimes equal to zero; only if the flooding or emptying of the tank occurs it has numerical value)

$H_{t}$ --- water level at time $t$

$A$ --- area of the tank

$T_{dela}$--- time interval between $t-1$ and $t$.

The concept of feedforward plus feedback control in a loop is that the identified model 3.2 is used to estimate the mean flowrate ($P_{mean}$) for the next day and also to predict the flowrate ($F_{pt}$) at the next sampling time. Then the mean flowrate ($P_{mean}$) is taken as the set point for that
day in the controller. For each sampling time (6 minutes in the simulation), the outflow rate from the equalization tank is determined by the deviation between the predicted flowrate ($F_{pt}$) and the mean flowrate ($P_{mean}$) for that day. The influent ($F_{eit}$) from the diversion weirs to the tank can be calculated from the water level change from time $t-1$ to $t$. Inaccurate prediction can be corrected by feedback control, to maintain the water level within the upper and lower limits. Fig.4.13 shows the algorithm flow chart and configuration of the control loop for this control strategy.

The first attempt is to use the predicted mean flow rate ($P_{mean}$) as the set point in the controller. The results of the simulation are displayed in Fig.4.14 and 4.15. The volume of the equalization basin is 4.76 MG (detention time is 2.5 hr) in the simulation. In Fig.4.14, the solid line represents the outflow rate after equalization, and the dashed line is the influent without equalization. The equalized flow is comparatively smoother than the influent. However, small hydraulic surges are still present. The volume percentage changes presented in Fig.4.15 are more uniform than the figures displayed in section 4.2.3.1 for the feedback control. The volume penalty factors $K_{up}$ and $K_{down}$ are given the value of 1.05; that is, a 5 percent penalty is applied if the upper or lower limits of the tank volume are exceeded. The penalty factors are useful in
Fig. 4.13 Side-line Equalization Control Diagram
Fig. 4.14 Outflow Profiles
Setpoint as $P_{\text{mean}}$; $K_{\text{up}}=K_{\text{down}}=1.05$

![Outflow Profiles Graph](image)

Fig. 4.15 Variations of Tank Volume
d.t. = 2.5 hr; $K_{\text{up}}=K_{\text{down}}=1.05$

![Tank Volume Variations Graph](image)
restricting the limits, with the 95 and 5 percent volume limits being well maintained with only a 5% penalty.

Since the predicted mean flowrate ($P_{\text{mean}}$) of each day varies considerably (Fig 4.2), to determine the outflow rate using $P_{\text{mean}}$ as a reference level can also cause fluctuations, which are observed in Fig.4.15. Another attempt was made to determine the outflow rate from the equalization tank by differencing the predicted flowrate ($F_{\text{pt}}$) and the daily average flowrate ($F_{\text{mean}}$) which is fixed by the height of the diversion weirs in the design phase. Since $F_{\text{mean}}$ is taken as a set point throughout the equalization process, the fluctuations may only occur near this reference level, instead of $P_{\text{mean}}$. This method is successful as can be seen by comparing Fig.4.14 with Fig.4.16. In the latter plot, the outflow is smoother than in the former. The corresponding volume percentage change over time for this method is shown in Fig.4.17.

Neither of the methods for control of the equalization processes could improve the damping capacity of the organic loads. Damping of the organic loads is essentially the same as for feedback control.

In summary, the capacity of the side-line equalization for damping the diurnal variations of flowrate and load depends greatly on what needs damping (flow rate or organic load), tank volume, configuration and control mode. Feedforward plus feedback trim control with daily average
Fig.4.16 Outflow Profiles
Setpoint is F_{\text{mean}}; K_{\text{up}}=K_{\text{down}}=1.05

Fig.4.17 Variation of Tank Volume
Setpoint is F_{\text{mean}}; K_{\text{up}}=K_{\text{down}}=1.05
flow rate ($F_{\text{mean}}$) as a set point is better than feedback alone. From the viewpoints of energy conservation and operation reliability, the pumping units are better placed before the basin. However, this configuration requires a valve or a moveable weir to control the outflow from the basin.

4.2.4 Simulation and Control of In-Line Equalization Tank

In-line equalization basins are commonly used in industrial wastewater treatment, where the basins function as units for mixing and homogenizing different qualities of wastewater, and in domestic wastewater treatment to dampen the diurnal fluctuation of both flowrate and organic load. The major difference between in-line and side-line equalization is that all flows are routed to the basin with the in-line basin, and only a portion of the flow is routed to the basin with the side-line basin. This results in differences in the control mode and the corresponding algorithm. Fig.4.18 shows the control algorithm flow chart and control loop. The identified model (Eq.3.2) is employed to estimate the mean flowrate ($F_{\text{mean}}$) for each day and to predict the flowrate ($F_{\text{pt}}$) six (6) minutes ahead for each sampling time. There is no flow meter used in the control system. The influent flowrate is calculated by measuring the water levels in the tank from time $t-1$ to $t$ using Eq.4.7. There are two control modes simulated for in-line
Fig. 4.18 In-Line Equalization Control Diagram
equalization. First \( P_{\text{mean}} \) is taken as a reference level to control the flowrate out of the tank, and then the daily average flowrate (\( P_{\text{mean}} \)) is used as another set point to decide the outflow rate. Finally, the results of the two control modes are compared.

The tank volume used in the following simulations is 18,000 m\(^3\) (4.76 MG), which is smaller than the required volume for these types of control modes. However, the upper and lower volume limits are maintained by the penalty factors and a feedback control loop with a PI controller.

The equation for the PID controller is

\[
F_{\text{e}_t} = F_m + K_p \cdot \text{err} + K_i \cdot \Sigma \text{err} + K_d \cdot (\text{derr}/dt) \quad (4.8)
\]

where

- \( F_m \) --- bias of the controller
- \( K_p \) --- proportional band
- \( K_i \) --- integral gain of the controller
- \( K_d \) --- derivative gain of the controller
- \( \text{err} \) --- deviation between the set point and input signal
- \( \text{derr} \) --- derivative of the error (deviation)
- \( F_{\text{e}_t} \) --- controlled outflow rate from the tank

Equation 4.8 can be used as a PI controller by leaving out the derivative term.
Fig. 4.19 displays the influent and effluent profiles for the in-line tank. Obviously, the outflow rate is strongly affected by the set point $P_{\text{mean}}$. Although the effluent for each day is comparatively smooth, hydraulic surges occur when the predicted mean flowrates change their values from one day to another. These sudden changes in flowrate are undesirable in operation, even if the magnitude of the changes are not as large as for the influent. The idealized outflow profile should smooth out the surges between the days. For instance, at the end of the first day (around hour 24), the outflow rate suddenly decreases from 190,000 m$^3$/d to about 170,000 m$^3$/d. Meanwhile, from Fig. 4.20, the tank still has volume available to gradually accommodate this change (the difference of the above two figures). The problem is that the damping capacity of the tank is not effectively used due to the sudden change in the reference point $P_{\text{mean}}$. The question arises of how $P_{\text{mean}}$ can be changed smoothly. We will address this question in the section concerning the optimum control of in-line equalization. The changes of percentage volume for this control mode, in Fig. 4.20, are more uniform than in the previous simulations.

From Fig. 4.21, we find that a better damping of the organic load is achieved by the in-line basin than by the side-line basin. The reason is that all of the flow is routed through the tank and completely mixed.
Fig. 4.19 Outflow Profiles
Setpoint is Pmean; Kup=Kdown=1.05

Flowrate (1000 cu.m/d)

Time (hour)

Before Equalization
After Equalization

Fig. 4.20 Variations of Tank Volume
Setpoint is Pmean; Kup=Kdown=1.05

Volume Percentage (%)

Time (hour)

Volume %
Up & low Limits
Fig. 4.21 Load After Equalization
Set point is $P_{\text{mean}}$; $K_{\text{up}}=K_{\text{down}}=1.05$

- - Before Equalization  - - After Equalization
For the control strategy taking $F_{\text{mean}}$ as a set point, there is further evidence of this phenomena in Fig. 4.22, in which the organic load appears to have a smoother trend for the first two and half days. The corresponding water volume in the tank in Fig. 4.23 is at its maximum, therefore the concentration is well mixed due to the large volume of the water in the tank. When the tank is almost empty during the rest of the time, the damping effects are greatly reduced. Obviously, the magnitude of damping for organic loads depends on the given volume of the tank and the amount of water in the tank. The latter factor is determined largely by the control strategies that are applied to the equalization systems. For either case, taking the set point as $F_{\text{mean}}$ or $F_{\text{mean}}$, the in-line type configuration is more effective at reducing diurnal variations of loads than the side-line basin.

In order to avoid sudden changes in outflow rate from one day to another, the daily average flowrate ($F_{\text{mean}}$) is taken as the set point for this control mode. Fig. 4.24 displays the profiles for the influent and effluent. As in the side-line simulation, an improved outflow curve is obtained. The flow after the equalization basin is smoother and the diurnal fluctuations are greatly damped out. It appears that the control mode with daily average flowrate as a set point is a better option in the operation of equalization processes.
Fig. 4.22 Load After Equalization
Setpoint is $F_{\text{mean}}$; $K_{\text{up}}=K_{\text{down}}=1.05$

Fig. 4.23 Variations of Tank Volume
Setpoint is $F_{\text{mean}}$; $K_{\text{up}}=K_{\text{down}}=1.05$
Fig. 4.24 Outflow Profiles
Set point is $F_{\text{mean}}$; $K_{\text{up}}=K_{\text{down}}=1.05$
4.3 Optimum Control of Equalization Basins

The effluent flow rate profiles after equalization are not optimized by using the control algorithms used in section 4.2. It is desirable that the reference levels (or set points) should be dynamically changed through the control processes; the integration of the errors between the preferred reference flowrate and the controlled outflow rate should be uniformly distributed throughout the day without violating the upper and lower limits of the tank volume. Obviously, this desired outflow rate curve cannot be realistically achieved by ordinary control methods. In the following sections, efforts are made to solve these problems. Finally, an optimum control model and its corresponding control algorithm are developed for in-line equalization basins. The simulations are carried out and the optimized outflow rate profiles are compared with those in the previous sections.

4.3.1 Model Building

Since the equalization basin has the capacity to store water, there must be, at each sampling time, one optimum outflow rate among the many which can be determined in the control processes. It is desirable that the selected outflow at any time \( t \) should have a minimum deviation both from the reference level (set points) and the previous outflow rate
at time $t-1$. Hence, the objective function of the model at sampling time $t$ takes the form of

$$\min \quad Z_t = (F_{t-1} - F_t)^2 + (F_{mp} - F_t)^2$$

(4.9)

and the constraint equations for this function are

$$V_{t-1} + (F_{t-1} - F_t)^*T_{deta} <= 0.95*V$$
$$V_{t-1} + (F_{t-1} - F_t)^*T_{deta} >= 0.05*V$$
$$F_t >= 0.0$$

where

$F_{t-1}$ --- previous outflow rate at time $t-1$

$F_t$ --- current outflow rate at time $t$

$F_{mp}$ --- mean flowrate (reference level or set point)

$V$ --- total tank volume

$T_{deta}$ --- sampling time interval between $t-1$ to $t$

To simplify the constraint equations, assume $A=V_{t-1} + F_{t-1}^*T_{deta} - 0.95*V$ and $B=V_{t-1} + F_{t-1}^*T_{deta} - 0.05*V$; the constraint equations become

$$g_1(F_t) = A - F_t^*T_{deta} <= 0.0$$

(4.10)

$$g_2(F_t) = B - F_t^*T_{deta} >= 0.0$$

(4.11)

$$g_3(F_t) = F_t >= 0.0$$

(4.12)

In order to solve the objective function along with the constraint equations, the constraints of the objective
function are converted into the non-constraint equation by constructing a penalty function (SUMT Outside Approach Method (28))

\[ P(F_t, r_k) = Z_t(F_t) + r_k \sum_{i=1}^{m} [g_i(F_t)]^2 \cdot u_i(g_i) \] (4.13)

where 
- \( r_{ki} \) --- penalty factor for \( i \)-th iteration
- \( m \) --- number of constraint equations
- \( u_i(g_i) \) --- a discrete function, it has the value of

\[
\begin{align*}
\text{if } g_i(F_t) &\leq 0 \quad u_i(g_i) = 0 & (\text{constraint obeyed}) \\
\text{if } g_i(F_t) &> 0 \quad u_i(g_i) = 1 & (\text{constraint disobeyed})
\end{align*}
\]

Assuming \( r_k = r_{k1} = r_{k2} = r_{k3} \) (same weighted penalty for each iteration), substituting the objective function along with the constraint equations into Eq.4.13, and then setting \( \frac{dP}{dF_t} = 0 \), we obtain the optimum outflow rate at sampling time \( t \)

\[ F_t = \frac{[F_{t-1} + F_{mp} + T_{deta} \cdot r_k \cdot (A + B)]}{[2 + r_k \cdot (2 \cdot T_{deta}^2 + 1)]} \] (4.14)

Using Eq. 4.14, an outflow rate profile is obtained for the 5 days' flowrate data that was used in the previous simulations. The profile is shown in Fig.4.25. It appears that the optimized profile is not greatly improved in comparison with the in-line control types (see Fig.4.19 and Fig.4.24). There are also two hydraulic surges between time 80 to 100 hours (Monday and Tuesday). Furthermore, it seems
that the average flowrate of each day has a great influence on the formation of the curve. The problem with the curve is that the outflow rate is only optimized at each sampling time \( t \) using Eq.4.14, instead of being optimized for the whole day or the whole 5 days. One mean flowrate of a whole day taken as the reference level cannot reflect the trend of the flowrate from one day to another. It is desirable that the integrated errors between the \( F_t \) and \( F_{t-1} \) and \( F_{mp} \) through a day should be minimized and uniformly distributed throughout the day. Thus, the minimum errors or deviations in a whole day can be obtained by the following equation
\[
\min \ Z_d = \sum_{k=1}^{n} [F_{t-1} - F_t]^2 + [F_{mp_i} - F_t]^2
\] (4.15)

where

\( n \) --- sampling times during a day

\( F_{mp_i} \) -- mean outflow rate over a certain period during a day

\( i \) --- i-th segment during the day (\( i=1, \ldots, m \))

It should be mentioned that since \( F_t \) has been the optimum outflow rate for each sampling time, the integration of error \( Z_d \) has a minimum value only if the \( F_{mp_i} \) value (mean flowrate) can be dynamically changed over some period of time in a day for each iteration.

**4.3.2 Optimum Algorithm and Control Strategy**

As mentioned in the previous section, the minimum deviation \( Z_d \) can be realized by dynamically changing the value of the predicted mean outflow rate \( F_{mp_i} \). However, if the \( F_{mp_i} \) changes at every sampling time \( t \), the desired reference value is lost, resulting in a non-regulated fluctuation of flows. The \( F_{mp_i} \) should remain a constant value for a certain period of time, and should represent the trend of the flow during the day. The mean flowrate \( F_{mp} \) can be divided into several segments \( (F_{mp_i}, i=1, \ldots, m) \) during a day. For instance, if we divide one day into 4 segments, each segment has a length of 6 hours and \( m \) equals 4.
Fig. 4.26 shows the flow chart for the control algorithm. The detailed explanation for this chart is

1). using equation 3.2, predict the mean flowrate for the next day (Fm24) and the day after the next (Fm48).

2). decide the length of the segment; for example, if the length is 6 hours, then the segment number m is 4. m = 24/length.

3). determine the difference between any two Fmp_i (i=1,m) for the day by the equation: Delta = (Fm24 - Fm48)/m.

4). taking today's mean flow rate as Ftd; determine the trend and Fmp_i for the next day by comparing Ftd, Fm24 and Fm48. If Ftd>Fm24>Fm48, the Fmp_i should decrease from the high to low with the difference of Delta and be near the reference level of Fm24:

Fmp_i=Delta*(m/2+1-i)+Fm24 (1); if Ftd<Fm24<Fm48, the Fmp_i should increase from the low to high:

Fmp_i=Delta*(i-m/2+1)+Fm24 (2); if Ftd>Fm24<Fm48, Fmp_i should first decrease until Fm24 using (1), then increase using (2).

5). iteration for the optimum solutions:

<1>. define a small increment ad, and let ad = 0 first, then get the new value of segment mean flow rate determined in 4) by Fmp_i^{new} = Fmp_i^{*}(1+ad) (3); using equations 4.14 and 4.15, calculate the total error for that day Zd. 

<2>. give a small positive value to ad,(i.e. ad=0.001)
Fig. 4.26 Optimum Control Algorithm Flow Chart
and use (3) to calculate $F_{mp_i}^{new}$ again, and to obtain $Zd_2$.

<3>. determine the search direction: if $Zd_1 < Zd_2$, the search may be in the wrong direction, the ad should be given a negative value; otherwise, ad keeps the positive increment.

<4>. continue to increase ad until $Zd_{j-1} < Zd_j$ is found, then stop iteration; the adjusted $F_{mp_i}$ and corresponding outflow rates ($F_t$) at each sampling time for the next day are determined.

It should be mentioned that the length of the segments during a day can be changed in the algorithm (such as the length equals 2 hour, $m=12$; length=3 hour, $m=8$, etc.) The computer program can search the optimum solutions for any given length of the segment. If the length is too short, say 2 hours ($m=12$), the outflow rate appears unstable and many fluctuations are created since the reference level $F_{mp_i}$ is changed too often. However, if the length of the segment is too long, say 8 hours ($m=3$), the trend of the flow from one day to another cannot be well represented. The segment length of 6 hours ($m=4$) was found to be the best in this research.

Fig.4.27 shows the segment mean flowrate trend with a time length of 6 hours at each period. A comparatively smooth trend of flowrates from day to day has been obtained. Since this trend is taken as the reference level,
construction of a smoother outflow curve becomes possible. Fig. 4.28 shows the reconstructed outflow rate profile. The flow tends to decrease during the first two and half days and to increase for the last two days. The changes in flowrate are relatively small and smooth compared with the previous control simulations for both the side-line and in-line basins.

Optimum control of the equalization process based on the above models and control algorithm can also be accomplished by a feedforward plus feedback trim algorithm. The feedforward control is responsible for on-line prediction of the mean flowrate (Fm_{24}) 24 hours ahead of time. It also predicts the mean flowrate 48 hours (2 days) ahead of time (Fm_{48}), which will be used to determine the flow trend and the deviation of the mean flowrate between the two days due to the algorithm. The optimum outflow rate profile will be chosen one day ahead based on the algorithm. The pre-determined outflow rate at each hour or each sampling time will be used at the time t. The errors between the estimated flowrate and the actual flowrate may cause overflowing or emptying of the tank. This problem can be overcome by the use of a feedback loop with a PID controller. The corrections given by the feedback loop are issued by a PID controller (equation 4.8). A two point control mode with the penalty factors K_{up} and K_{down} to maintain the upper and lower limits of the tank volume is
Fig. 4.27 Mean Flowrate Trend for Fmpi
with each segment length of 6 hr

Fig. 4.28 Pre-set Outflow Rate
d.t=2.5 hr; segment length=6 hr
used and incorporated with the PID controller. The on-line water level signal at sampling time $t$ is fed back to the controller, and the volume response is calculated, finally the correction is made to the pumping units and the outflow rate is adjusted.

4.3.3 Dynamic Simulations

Based on the optimum control algorithm and strategy shown in Fig. 4.26, 5 days of operation are simulated in order to examine the strategy. The equalization volume (4.75 MG, detention time 2.5 hours) for the simulation is the same as for the conventional control modes for the in-line and the side-line configurations. Finally, the volume is reduced to 3.8 MG (14,380 $m^3$, the corresponding detention time is 2.0 hours and the mean flow rate is the same as in the previous simulations) to examine the potential of the optimum controller for reducing tank volume.

Fig. 4.29 shows the influent and the equalized outflow rate using the above optimum control strategy. Although there are still small fluctuations in effluent flow rate, the trend is reasonable and a smoother outflow curve is obtained. The volume response corresponding to the outflow is displayed in Fig.4.30. Comparing this plot with Fig.4.28, in which the volume changes are pre-determined based on the predicted influent flowrate, the volume percentage curve has changed greatly during the time between 40 to 80 hours. This
Fig. 4.29 Optimized Outflow Profile
\[ d.t. = 2.5 \text{ hr; PID; } K_u = K_{down} = 1.05 \]

Flow Rate (1000*cu.m/d)

Time (hour)

--- Before Equalization --- After Equalization

Fig. 4.30 Volume Change vs Outflow
\[ d.t. = 2.5 \text{ hr; PID; } K_u = K_{down} = 1.05 \]

Volume Percentage (%)

Outflow Rate (1000*cu.m/d)

--- Volume % --- Outflow Rate --- Up & Low Limi
is caused by the accumulations of deviations between the
predicted and real flowrate. They are corrected by the
feedback loop with a PID controller, as well as a two point
control with the penalty factors. The tank volume limits are
well maintained by the PID controller. There is no
substantial damping of the organic load. As previously
mentioned, the damping capacity for the organic load depends
upon the volume of water inside the tank. The more water in
the tank, the better the damping. Generally, the damping of
load variations requires more equalization volume than the
damping of flow variations.

Comparison of the optimum control with the more
conventional control modes for the in-line and side-line
basins is given in Figs. 4.31 to 4.33. Fig.4.31 shows the
three outflow rate profiles: the dot-dashed line represents
the result of optimum control for the in-line basin; the
dashed line represents the feedforward-plus-feedback control
for the in-line basin with $F_{\text{mean}}$ as the set point; and the
solid line represents the side-line control with $F_{\text{mean}}$ as
the set point. Ordinary methods of control cannot reflect
the trends of the flows from one day to another; and the
reference levels are fixed throughout the 5 day period,
which can result either in bypassing the tank during the
overflow periods, or in reduction in the outflows during the
emptying periods. The phenomena is further illustrated by
comparing the volume changes for the three types of controls
Fig. 4.31 Comparison of Three Controls
\( \text{d.t.}=2.5 \text{ hr}; \text{ Adjusted by PID} \)

- Optimum
- Side-line
- In-line
- Influent

Fig. 4.32 Comparison of Volume Usage
\( \text{d.t.}=2.5 \text{ hr}; \text{ Kup=Kdown=1.05} \)

- Optimum
- Side-line
- In-line
Fig. 4.33 Comparison of Load Damping
d.t. = 2.5 hr

Organic Load (1000*kg/d)

Time (hour)

Optimum  Side-line  In-line
(Fig.4.32). For optimum control, the water volume in the tank changes uniformly from one day to another. However, for the other two types of control, the tank is full during the first two and half days, and is emptying for the remainder of the time. This is undesirable since control of the overflow and emptying of the tank relies on the feedback control loop. Hence, higher reliability of the loop is required and the time delay in the loop must be considered and handled carefully. A comparison of the effects of the control methods on the damping of organic loads is shown in Fig.4.33. Side-line control has a lower damping capacity for organic loads when compared with the two in-line control system. For ordinary control of the in-line basin (dashed line), the loads are greatly damped during the first two days since the tank is full, and poorly damped for the last two days. By contrast, optimum control provides comparatively constant damping ability, although the magnitude of damping of the loads is not as large as with ordinary in-line control for the first two days.

In order to further examine optimum control, the volume of the equalization basin is reduced to 3.8 MG (14,380 m³, detention time is 2.0 hours); that is, the volume is decreased by 20%. Fig.4.34 and 4.35 display the simulated outflow rate and corresponding volume change profiles under the optimum control strategy. It should be noted that there is one short period of emptying in Fig.4.35 due to the
Fig. 4.34 In & Out Flowrate Profiles
\[ d.t. = 2.0 \text{ hr}; K_{up} = K_{down} = 1.05; \text{ PID} \]

Flowrate (1000*cu.m/d)

Time (hour)

Before Equalization

After Equalization

Fig. 4.35 Volume Change vs Outflow
\[ d.t. = 2.0 \text{ hr}; K_{up} = K_{down} = 1.05 \]

Volume Percentage (%)

Outflow Rate (1000*cu.m/d)

Time (hour)

Volume %

Outflow Rate

Up & Low Limit
accumulation of inaccurate predictions. Comparing Fig.4.34 with Fig.4.29, the reduction of 20% of equalization volume does not cause large hydraulic surges.

4.4 Conclusions and Recommendations

1. Determination of the required volume is a critical step in the design of equalization basins. The effects of control methods on the equalization process should be taken into account during design. Since the daily influent flow rate changes considerably, as much data as possible should be used to determine the basin volume.

2. The performance of a side-line basin for damping the diurnal variations of flowrate and organic load depends on the tank volume, configuration and control methods. A control strategy of feedforward plus feedback trim with the daily average flow rate ($F_{mean}$) as the set point is better than feedback only.

3. An in-line equalization basin is suitable for damping diurnal fluctuations of both flow rate and concentration.

4. Feedforward control with a PI or PID controller as feedback trim, as well as with two point control of the upper and lower depth limits of the tank, is suitable for control of both in-line and side-line equalization basins. The design mean flow rate ($F_{mean}$) can be used as the set point.
5. An optimum control system based on the predictions of dynamic models has shown significant potential for reducing tank volume and producing smoother outflow rates than the other types of control algorithms. The required volume can be reduced by 63.5% (from 10.4 MG to 3.8 MG) using the optimum control strategy. However, the control system should be further tested at pilot scale before implementation.

6. Since the sewer usually enters the grounds of the treatment plant at considerable depth, it is suggested that the pumping units should be placed before the basin to avoid duplication of pumping units and to reduce the risk of overflowing or emptying the basin. This configuration requires a moveable weir or a valve, instead of pumps, to regulate the outflow rate from the basin.
Chapter 5

A SOFTWARE PACKAGE FOR DYNAMIC SIMULATION
OF FIXED SPEED PUMPING STATIONS

Simulation of control processes in waste water treatment plants enables the practitioner to understand the dynamic behavior of waste water treatment processes and to test control strategies for conversion of unsatisfactory to satisfactory behavior. In recent years personal computers (PC) have become widely used in engineering, and inexpensive, easy-to-use software packages for PC's are commercially available. These packages can be combined to facilitate the application of dynamic modeling and control systems to both the design and operation of wastewater and transport systems.

The objectives of this chapter are: 1) to develop a software package that combines the characteristics of different software packages for use in dynamic modeling control system application (These package should be inexpensive, easy to use and readily available on the market); and 2) to use the package to simulate and control pumping stations (fixed pumps are taken as an example).

5.1 Characteristics of Individual Software Package

There are 4 software packages, in addition to DOS, that are considered as the basic elements of the system package.
An additional package is discussed for optional use. The characteristics and the basic functions are briefly described in this section.

5.1.1 Simnon (V 3.0)

Simnon (43) is used for simulating dynamic system and control processes. It is especially useful for simulating models based on differential equations and difference equations. Simnon also provides Macro facilities, which will assemble each individual sub-system into an overall system.

Since Simnon is primarily written for the simulation of controlled processes, it is convenient to use for ordinary controllers, such as P, PI, or PID controllers; and for control loops, such as feedforward, feedback, etc. If the user has more knowledge of automatic control, Simnon can be used for simulating advanced control processes, such as adaptive control, or for on-line parameter identification or optimization problems.

Simnon also provides the functions to exchange data with other software. In general, there are three levels of data in Simnon: time series, plots and models. Time series data can be exported and imported to and from ASCII files which can be directly read by many software packages. Simnon can use the programs inside MATLAB (48), which further enhances the ability of Simnon. The working plots produced by Simnon can be either directly transmitted to printer or
plotter, or further processed by presentation graphics software.

The latest version of Simnon (ver. 3.0) provides real time functions. These include data acquisition and monitoring, real time control, and real time process simulation.

5.2.2 Norton Commander (V 3.0)

The Norton Commander (44) provides many useful functions such as file management, manipulation and editing, etc. It is a good tool to facilitate applications of DOS to other software and is commonly thought of as a "DOS shell."

Norton Commander has an excellent file manipulation system. The files can be easily copied, renamed, moved, searched, jointed and edited. One of the most important characteristics of the Norton Commander is the file editing function. Any text files in ASCII, such as Simnon programs, Basic or Fortran programs or DOS batch programs can be easily created and edited.

Another useful function of Norton Commander is the Quick View command. It permits viewing the data or text files of other software packages, such as spreadsheets (Lotus, Symphony, Excel, MS Works, etc.), Databases (dBASE, MS Works, etc.), Word Processors (WordPerfect, MS Windows, MS Word, MS Works, etc.) and Graphics. Since the package developed in this section is the combination of several
other software packages, this function is of special importance.

5.1.3 MicroSoft Works (V 2.0)

Micro-Soft Works (45) combines functions of DBASE III, Lotus-1-2-3, Wordstar and Excel into one package, so it has a word processor, database, spreadsheet and presentation graphics. In addition to the above functions it has a communication module that allows a user to transfer files between personal computers.

The two most important features are the database and spreadsheet, which were used frequently in this study. The database provides many functions which are similar to dBASE III, but are easier to use. The database files can be exchanged with the spreadsheet. This is useful since Simnon can directly read the output files from spreadsheets when they are in ASCII format.

5.1.4 Software Carousel (V 2.01)

The most outstanding characteristic of Software Carousel (42) is the multi-partition function. It provides for as many as 12 partitions, each of which can contain a different application program. Although only one partition can be active at any given time, it is easy to switch among the partitions. When the switch is made, the application in
that partition is suspended. However, it will start again at
the point where it left off when it is reactivated.

Software Carousel can access DOS extended or expanded
memory. This makes enough room for each partition to run
comparatively large programs, such as Simnon, Microsoft
Works, Statgraphics, etc.

5.1.5 Statgraphics (V 2.6)

Statgraphics (46) is a powerful statistics package. It
contains many common and advanced statistic programs for
model building, data analysis, and time series applications.
It can be applied to modeling influent flow rate, pollutant
concentrations, etc., based on historical data. All of the
statistical models can be used for advanced control.
However, since this package is comparatively expensive, it
is suggested as an option.

5.2 Combination of Software Packages

There are two key problems in the combination of the
software packages: 1) how to organize them and to take
advantages of each package to serve the purposes of control
and simulation; 2) how to transport data and graphs among
the different packages. After examing several possible
solutions to the above, the DOS batch file technique was
finally chosen for combining of the software packages in
this study. Detailed listings of the DOS batch files are
given in the disk included with this thesis.

Table 5.1 shows some features of the individual
software packages. It should be noted that all the packages
have the ability to input and output ASCII files to and from
DOS, which is fundamental to the combination of the
packages.

<table>
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<th>Name</th>
<th>Price</th>
<th>RAM used</th>
<th>In &amp; output</th>
</tr>
</thead>
<tbody>
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<td>dollar</td>
<td>Kbytes</td>
<td>to DOS</td>
</tr>
<tr>
<td>Carousel</td>
<td>80</td>
<td>40 k</td>
<td>x</td>
</tr>
<tr>
<td>Norton Com.</td>
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<td>35 k</td>
<td>x</td>
</tr>
<tr>
<td>Simnon</td>
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<td>210 k</td>
<td>x</td>
</tr>
<tr>
<td>MS Works</td>
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<td>200 k</td>
<td>x</td>
</tr>
<tr>
<td>Statgraphics</td>
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<td>250 k</td>
<td>x</td>
</tr>
<tr>
<td>Total ($)</td>
<td>984</td>
<td></td>
<td>(excluding Statgraphics)</td>
</tr>
</tbody>
</table>

5.2.1 Configuration of This Package

Fig. 5.1 shows the structure of the package. The
configuration is based on the partition functions provided
by Software Carousel. Each partition is responsible for
certain tasks. Users can easily switch from one partition to
another. The connections between the partitions are
Fig. 5.1 Configuration of Software Package
accomplished by the Menu System (MS) which is written in DOS batch files.

5.2.2 Function Group in Each Partition

1) Central Control Menu

One of the objectives of this study was to develop linking programs for the individual software packages. A batch-file-based menu system is developed for this purpose. Copies of the menu screens are given in Appendix.

The Central Control Menu (CCM) is located in partition 1. It functions as a guide to the whole system. When the user becomes familiar with this screen, he may not need to go to it every time the system is started. The other features allow the operator to go to any of the other partitions (Fig.5.1) by simply pressing pre-defined function keys and suspending the current application. Here the advantages of the Carousel partitioning function are used.

There are also different sub-menus which are connected with CCM and among themselves. Therefore, a menu network is formed, which provides more convenience to the user. Each time the user wants to perform certain tasks, there is a menu that demonstrates the choices available. When the work has been done there is also a menu to provide the next choices.
2) Simnon Simulation

Simnon Simulation (SS) is located in partition 2. It is the central partition for simulation and control processes. The pumping station will be used as an example. The functions of SS are: (1) simulating different configurations of the pumping station, such as the number and size of pumps, the volume of the wet-well, etc.; (2) simulating pump and wet well performance during operation by using differential, difference or statistic models; (3) execution of control strategies; (4) producing plots of pump station behavior and ASCII tables or text files for use in other programs.

3) Model Base

Model Base (MB) is located in partition 3. In simulation and control some of the mathematical models are more conveniently solved using the Basic (52) language. The MB also contains the dynamic models developed in Chapter 3 for the optimum control of fixed speed pumps. The programs for the models in MB are arranged in module form.

Statgraphics is also installed in this partition. It can be used for manipulating time series data and identifying the statistical models. Statgraphics is not needed for the non-statistical models and control systems.
4) Data Base

The Data Base (DB) is located in partition 4. It stores, inputs, records and outputs data both graphically and as text. DB supports all the partitions, that is, it can be accessed from any partition. The Data Base makes use of the data base function provided by Microsoft Works. The data files in the DB can use the spreadsheet for an output format which can be accepted by Simnon, Basic, and Statgraphics. Simulated results can also be taken through the spreadsheet to store in the Data Base.

5) Users’ Interface

The Users’ Interface (UI) is located in partition 5. It serves as a junction to connect the partitions in the system and with the outside environment of the system. It consists of several layers of menus. The user can monitor the simulation process using either the function key to partition 2 or by using menus. He can go to the MB to run models or go to the DB to see historical plots and data files. In this partition the graphics and data files can be directly sent to the printer.

Microsoft Works is installed here. The user can directly start Works and prepare reports using the graphics and data in the Data Base. All the functions of Microsoft Works can be used.
The Norton Commander is installed under DOS. It is a convenient tool for file manipulation. It can be accessed by either going to DOS or by using the menus.

5.3 Dynamic Simulation of Fixed Speed Pumping Systems

Simulation of the control processes for fixed speed pumping systems is given as an example herein to illustrate the use of this combined software package.

5.3.1 Input Parameters for the Simulation

The input parameters include the geometry of the wet-well, the identified parameters of equations 3.5 to 3.7 for the pump, system and efficiency curves, and the number, size and lift head of the pumps. The menu will ask for the inputs of these parameters before the simulation starts.

The geometry of the wet well consists of its length, width and height. The user can change the dimensions for each simulation. The effects of the changes can be observed from the output plots of Simnon. The number and size of the pumps can also be changed in the same way for each simulation.

As discussed in Chapter 2, regression methods can be applied to estimate the parameters of equations 3.5 and 3.7 for the efficiency and head-capacity curves. The computer programs for polynomial regression are available in the Statgraphics (46) or CA-Cricket Graph For Microsoft Windows
(47). The system head-capacity curve can be obtained by calculating head losses at different flow rates for the piping system.

However, there is a simple way to input pump curves using the FUNC command of Simnon. The pump curves can be represented by manually picking out the coordinates from these curves. This coordinate data can then be stored in the Simnon data files. The FUNC function of Simnon can this data and in the simulations.

5.3.2 Input of Control Modes

There are two built-in control modes for fixed speed pump simulation. One is that each pump starts when its upper water level set point is reached, and all pumps stop when the water level in the wet-well falls below the lowest set point. For the other control mode, the procedure to start the pumps is the same as for the first, but the stop procedure is different. Each pump stops when the water level in the wet-well reaches its lower set point. The user can choose either of these control algorithms to simulate the performance of the pumping system.

5.3.3 Simulation

Once the parameter and control mode inputs are entered, the user can start the simulation. The simulation process is
directed by the menus. If there is any problem, the user can use the help option to obtain help from the system.

5.4 Recommendations for an Advanced Package

The software package developed in this study is based on the concepts of low cost and ease-of-use by practicing engineers. However, there are two areas where improvement is needed. One is dynamic data exchange (DDE), and another is the use of DOS batch files.

Dynamic data exchange (DDE) is a very important function in simulating real control processes. The data at any moment or sampling time should be continuously and automatically transmitted to each function group, so that the monitoring, dynamic plot display, data acquisition, and control action can be realized. The menus based on the DOS batch commands can only manually transmit data files to the function groups.

The answer to the above is the use of Microsoft Windows (49). It has a dynamic data exchange (DDE) function and can run as many as 12 applications at the same time. Also, the menu system based on DOS batch files is not needed since Window has its own menu system. This allows the user to start and manipulate each application (or each software package) by use of a mouse.

The possible configuration of software combination for future integrated packages could be:
1. Microsoft Windows
2. Simnon
3. Statgraphics
4. Microsoft Excel or Lotus 1-2-3
5. Microsoft Word

It should be mentioned that the newly released version of DOS (5.0) has new functions which may make it an attractive alternative to Norton Commander and Software Carousel for file management and task switching.
REFERENCES


50. Seaware Corp., The EBL PlusTM LANGUAGE User's Guide, Seaware Corp., P.O. Box 1656, Delray Beach, FL 33444.


52. IBM, Basic Handbook, IBM Corp., P.O. Box 1328-C, Boca, FL 33432.
### NOMENCLATURE

**Chapter 2:**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$B$</td>
<td>back shift operator</td>
</tr>
<tr>
<td>$\varphi_1$</td>
<td>coefficient of autoregressive (AR) model with order of $p = 1$</td>
</tr>
<tr>
<td>$\phi_1$</td>
<td>coefficient of seasonal autoregressive (SMA) model with order of $P = 1$</td>
</tr>
<tr>
<td>$\theta_1$</td>
<td>coefficient of moving average (MA) model with order of $q = 1$</td>
</tr>
<tr>
<td>$\theta^S_1$</td>
<td>coefficient of seasonal moving average (SMA) model with order of $Q = 1$</td>
</tr>
<tr>
<td>$s$</td>
<td>seasonal length, $s = 24$ hour</td>
</tr>
<tr>
<td>$F_t$</td>
<td>influent flow rate at time $t$, MGD</td>
</tr>
<tr>
<td>$F_{t-s}$</td>
<td>influent flow rate at time $t-s$, MGD</td>
</tr>
<tr>
<td>$F^S_t$</td>
<td>seasonal differencing of influent flow rate</td>
</tr>
</tbody>
</table>

**Chapter 3:**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_t$</td>
<td>dynamic volume of wet-well at time $t$, m$^3$</td>
</tr>
<tr>
<td>$V_{t-1}$</td>
<td>dynamic volume of wet-well at time $t-1$, m$^3$</td>
</tr>
<tr>
<td>$F_{in}$</td>
<td>mean influent flow rate from time $t-1$ to $t$, m$^3$/hr</td>
</tr>
<tr>
<td>$F_{out}$</td>
<td>mean effluent flow rate from time $t-1$ to $t$, m$^3$/hr</td>
</tr>
<tr>
<td>$T_{delta}$</td>
<td>length of time interval between two samples, minute</td>
</tr>
<tr>
<td>$H_t$</td>
<td>dynamic water level in the wet-well at time $t$, m</td>
</tr>
<tr>
<td>$H_{t-1}$</td>
<td>dynamic water level in the wet-well at time $t-1$, m</td>
</tr>
<tr>
<td>$A$</td>
<td>total area of wet-well</td>
</tr>
<tr>
<td>$H_p$</td>
<td>pump lift head, m</td>
</tr>
<tr>
<td>$H_S$</td>
<td>pumping system head, m</td>
</tr>
<tr>
<td>$Eff$</td>
<td>pump efficiency</td>
</tr>
<tr>
<td>$Q$</td>
<td>pump capacity at operation point, m$^3$/hr</td>
</tr>
<tr>
<td>$HL$</td>
<td>upper set point for control modes A and B, m</td>
</tr>
<tr>
<td>$M$</td>
<td>middle set point for control modes A and B, m</td>
</tr>
<tr>
<td>$LL$</td>
<td>lower set point for control modes A and B, m</td>
</tr>
<tr>
<td>$H_1$</td>
<td>highest set point for control modes C and D, m</td>
</tr>
<tr>
<td>$H_2$</td>
<td>second higher set point for control modes C and D, meter</td>
</tr>
<tr>
<td>$M$</td>
<td>middle set point for control modes C and D, m</td>
</tr>
<tr>
<td>$L_1$</td>
<td>lowest set point for control modes C and D, m</td>
</tr>
<tr>
<td>$L_2$</td>
<td>second lower set point for control modes C and D, meter</td>
</tr>
<tr>
<td>$H_{up}$</td>
<td>top limit of water level in wet-well, m</td>
</tr>
<tr>
<td>$H_t$</td>
<td>dynamic water level in wet-well at time $t$, m</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>energy weighting factor</td>
</tr>
<tr>
<td>$\beta$</td>
<td>outflow smoothing weighting factor</td>
</tr>
</tbody>
</table>
\[ Z_{1t} \] squared minimum distance between top limit and
dynamic water level at time \( t \)
\[ P_i \] power input or power required, \( \text{kW/hr} \)
\[ S \] specific weight of water, \( \text{kN/m}^3 \)
\[ H_{td} \] total dynamic head, \( \text{m} \)
\[ Z_{2t} \] squared difference between outflow rate at time \( t \)
and \( t-1 \)
\[ j \] pump numbers \( (j=1, 2, \ldots, k) \)
\[ Q_{j,t} \] average outflow rate of j-th pump at time from
\( t-1 \) to \( t \), \( \text{m}^3/\text{hr} \)
\[ X_{j,t} \] state variable of j-th pump at time from \( t-1 \) to \( t \)
\[ H_{down} \] lower limit of water level in the wet-well, \( \text{m} \)
\[ H_{up} \] upper limit of water level in the wet-well, \( \text{m} \)
\[ I_{j,0} \] the time interval from \( t-1 \) to \( t \) of j-th pump
at state 0 (off or stopping state), minute
\[ M_s \] minimum stopping time, which is decided by
the capacity of pump to cool itself off

Chapter 4:

\[ C_t \] tank concentration at time \( t \), \( \text{mg/l} \)
\[ C_{int} \] effluent concentration at time \( t \), \( \text{mg/l} \)
\[ C_{t-1} \] tank concentration at time \( t-1 \), \( \text{mg/l} \)
\[ N \] the number of weirs
\[ z \] the total water depth, \( \text{m} \)
\[ \alpha \] the weir coefficient and a function of the weir
   type and width
\[ \beta \] a weir coefficient depending on the weir type
   (For example, it is 2.5 for a V-notch, 1.5 for a
   rectangular and 1.0 for a Sutro weir
\[ h \] the distance from the bottom of the weir to the
   water level, \( \text{m} \)
\[ F_{\text{mean}} \] design daily mean flow rate, \( 1000*\text{m}^3/\text{day} \)
\[ P_{\text{mean}} \] predicted mean flow rate, \( 1000*\text{m}^3/\text{day} \)
\[ F_{it} \] influent flow rate at time \( t \), \( 1000*\text{m}^3/\text{day} \)
\[ F_{eit} \] flow rate to equalization basin at time \( t \),
   \( 1000*\text{m}^3/\text{day} \)
\[ F_{et} \] effluent flow rate from equalization basin at time
   \( t \), \( 1000*\text{m}^3/\text{day} \)
\[ V \] total volume of the equalization tank, \( \text{m}^3 \)
\[ V_t \] dynamic volume at time \( t \), \( \text{m}^3 \)
\[ T_{\Delta t} \] sampling time interval, minute
\[ F_m \] outflow rate for control of overflow of
   equalization tank, \( 1000*\text{m}^3/\text{day} \)
\[ F_p \] outflow rate for control of emptying of
   equalization tank, \( 1000*\text{m}^3/\text{day} \)
\[ K_{up} \] penalty factor for the upper limit
\[ K_{down} \] penalty factor for the lower limit
\( F_{Pt} \) predicted influent flow rate at time \( t \), 1000*\( m^3 \)/day

\( F_m \) bias of the controller, 1000*\( m^3 \)/day

\( K_p \) proportional band

\( K_i \) integral gain of the controller

\( K_d \) derivative gain of the controller

\( \text{err} \) deviation between the set point and input signal

\( \text{derr} \) derivative of the error (deviation)

\( F_{mp} \) mean flow rate (reference level or set point), 1000*\( m^3 \)/day

\( r_{ki} \) penalty factor for \( i \)-th iteration

\( m \) number of constraint equations

\( u_{i(g_i)} \) a discrete function

\( n \) sampling times during a day

\( F_{mp_i} \) mean outflow rate over a certain period during a day

\( i \) \( i \)-th segment during the day (\( i=1, \ldots, m \))
Appendix

COMPUTER PROGRAMS FOR THE SOFTWARE PACKAGE

In general, this software package consists of three parts: the combined software packages which are commercially available; the linking programs developed in this research; and the simulation programs for fixed speed pumping stations. A floppy disk containing detailed programs for the last two parts may be obtained from the Environmental Science and Engineering Department at Rice University.

1). The combined software packages are:
   A. Simnon
   B. Norton Commander
   C. Microsoft Works
   D. Software Carousel
   E. Statgraphics (optional)

2). The linking programs are written based on DOS batch commands. However, the window formats for the menus are written using the Extended Batch Language (EBL (50)), which is also commercially available ($48 for this package). Since EBL is 100% compatible with the DOS batch commands, all DOS batch commands can run under the EBL programs.

3). The programs for simulation and control of fixed speed pumping stations include two parts; one is the two point control method and another is optimum control of the fixed speed pumps.
4). The other programs on the attached floppy disk are the ordinary control of side- and in-line equalization basins; the optimum control of in-line equalization basins; and ELECTRE II for the multi-criteria selection of ARIMA models.

It should be noted that at least 2 MB expended memory is needed for Software Carousel to run this software package in multi-partitions.

5). The examples of menu screens are shown in the following pages.
### Control Menu (F1)

- **F1** Partition 1, Central Menu
- **F2** Partition 2, Simulation
- **F3** Partition 3, Model Base
- **F4** Partition 4, Data Base
- **F5** Partition 5, User Interface
- **#** Interrupt to DOS
- **+** On-Line Help

Press function key with hot key <alt>

### Simnon Simu Menu (F2)

- **1** Simnon Simulation
- **F1** Main Control Menu (Par2)
- **F3** Goto Model Base (Par3)
- **F4** Goto Data Base (Par4)
- **F5** User's Interface (Par5)
- **2** Send Data File
- **#** Exit System Simu
- **+** On-Line Help
MODEL BASE MENU (F3)

1. FIXED SPEED PUMP
2. NOT ASSIGNED
3. ELECTRE II
4. TWO POINT CONT.
5. TIME SERIES
6. OPTIMUM CONTROL
+ ON-LINE HELP
# EXIT FROM SYS.

Choose one option then press <enter>

Press function key with hot key <alt> together

F1 GOTO MAIN MENU
F2 GOTO SIMNON SI.
F4 GOTO DATABASE
F5 USER'S INTERFACE

DATA BASE MENU (F4)

1. FILE EDITING
2. IMPORT FILES
3. TRANSLATE FILES
4. SENDIG FILES
5. OTHER FUNCTION
6. GOTO WORKS
+ ON-LINE HELP
# EXIT FROM SYS.

Choose one option then press <enter>

Press function keys with hot <alt> together

F1 GOTO MAIN MENU
F2 GOTO SIMNON SI.
F5 USER'S INTERFACE
F3 GOTO MODEL BASE
USER'S INTERFACE MENU (F5)

1. GOTO WORKS
2. SEE SIMU.RESULT
3. SEE HIST. FILES
4. PRINT GRAPHICS
5. TIME SERIES MODEL

+ ON-LINE HELP
# EXIT FROM SYS.

Choose one option then press <enter>

F1. GOTO MAIN MENU
F2. GOTO SIMNON SI.
F4. GOTO DATABASE
F3. GOTO MODEL BASE

Press function key with hot key <alt> together