JAPAN INTERNATIONAL COOPERATION AGENCY (JICA)

NATIONAL INSTITUTE OF DEVELOPMENT (INADE)
THE REPUBLIC OF PERU

THE STUDY
ON
THE INTEGRATED WATER POLLUTION CONTROL
FOR
PUNO INTERIOR BAY OF LAKE TITICACA
IN
THE REPUBLIC OF PERU

DATA BOOK

JANUARY 2000

PACIFIC CONSULTANTS INTERNATIONAL, TOKYO
in association with
ENVIRONMENTAL TECHNOLOGIC CONSULTANT CO., LTD., TOKYO

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DRAFT FINAL REPORT

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THE STUDY
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(DATA BOOK)

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1. Introduction

The dynamic modeling of sewer network was carried out for analyzing the existing and future sewer systems in the Puno city.

Foundation of the dynamic modeling is computer simulation, based on the software package MOUSE (abbreviation for Modeling Of Urban Sewers) developed at the Danish Hydraulic Institute.

The aim of the dynamic modeling was to study the function and capability of the sewerage system. A detailed description of the sewerage system can be found under the respective sections of the Main and Supporting Reports.

The dynamic modeling activity comprises the following tasks:

1. Development of simulation model for the existing sewer system.
2. Evaluating and analyzing the existing sewer system to identify the needs for rehabilitation and replacement of pipe network.

This section describes in short terms the above mentioned tasks.

2. Numerical models

The models applied for simulation of the sewerage system is the MOUSE Pipe Flow Model. The MOUSE Pipe Flow Model is a computational tool for simulations of unsteady flows in pipe networks with alternating free surface and pressurized flow regimes. The computation is based on 1-D, free surface[1] flow equations (Saint Venant Equations).

Contrary to the rational method, the fully dynamic description gives a better theoretical foundation for a flow model because the full equation of momentum makes it possible to describe all forces affecting the flow conditions. This description allows flow features such as backwater effects and surcharges to be precisely simulated.

[1] Pressurized flow computations are facilitated through implementation of a narrow slow, as a vertical extension of a closed pipe cross-section.

Inputs to the model are descriptions of the elements in the pipe network and boundary conditions, such as wastewater inflow. MOUSE enables a description of a variety of networks elements, such as standard and irregular shaped pipe cross-sections, manholes, basins, weirs, pumps, gates, flow regulation etc.

The features implemented enable a realistic and reliable simulation of the performance of both existing network systems and, those under design. However the reliability of such a simulation model depend on the available data background and data quality.

3. The Sewerage model

MOUSE Pipe flow models for the existing and future sewer networks were created. The model of the existing network was compared with flow monitoring data in wet and dry season to assess hydraulic performance of the system.

3.1 Data Background

3.1.1 Network data

Network data was based on the field survey results from the PRONAP detailed design study. Additional inspection and field survey were performed by the JICA study team to supplement the above data.

The sewered area was divided into sub-catchments (sub-networks) to simplify the model.

The so-called Hazen-Williams formula was applied to determine the friction loss in the force main. Additional hydraulic losses such as entrance and valve loss were also accounted for. The entrance loss was set to the total velocity head in the force main and the valve loss was set static to $\Delta H_V = 1.5$ m.

$$\Delta H_f = 6.82 \frac{L_R}{D^{1.17}} \left(\frac{v}{C}\right)^{1.85}$$

where

L_R = Length of force main (m).

C = Hazen formula flow rate (≈110).

D = Force main diameter (m).

ΔH_f = Friction loss (m)

v = velocity in force main (m/s)

The entrance loss ΔH_E is set to:

$$\Delta H_E = 1.0\frac{v^2}{2g}$$

where

g = gravitational acceleration (ca 9.8m/sec^2)

The static valve loss of 1.5 m is directly subtracted from the pump (Q,ΔH) relations before implementing the relations into MOUSE (ΔH is here the total energy loss + lifting between the pump wet-well and the receiving node after the force main in MOUSE). The entrance loss can be specified directly as manhole energy loss in MOUSE. The friction loss is more complicated and has to be converted into to the equivalent loss using Manning's formula. The following expression for the Manning's number can be derived when setting the Hazen-Williams friction loss equal to the friction loss extracted from the Manning's formula.

$$M = \sqrt{C^{1.85} D^{-0.1633} \frac{L_M}{L_R} 0.9310\, v^{0.15}}$$

where

M = The Manning's number (m$^{1/3}$/sec).

L_M = The force main length defined in MOUSE (m).

L_R = The real force main length (m).

D = Diameter of the force main (m).

v = Flow velocity in the force main (for peak flow) (m).

The length of force main is typically reduced in MOUSE in order to reduce the volume occupied in the Pricemann slot[2]. Furthermore the Pricemann slot has been further been reduced in size.

The volume occupied by the Pricemann slot plays an important role during start up of the pump station. Since the fluid is incompressible, the friction loss is immediately established in the force main when the pump starts. Since some volume is "hidden" in the Pricemann slot in the MOUSE model, the same fast respond as observed in reality cannot be expected in the MOUSE model. Thus, decreasing the volume occupied by the

[2] More detailed information on the Pricemann slot can be found in the MOUSE manual.

Pricemann slot significantly reduces this drawback. Accordingly the force mains longer than 400 m has been set to 400 m and the width of the Pricemann slot has been set to 0.1 % of the force main diameter.

However the correct energy line is established approximately 60 seconds after the pump starts.

3.1.2 Boundary conditions

The boundary conditions consist of wastewater flow and infiltration. Infiltration rate was set as 0.1 l/s/km based on the results of the flow survey. Infiltration and inflow during rain events can reach more than 100 percent of the daily average flow rates. Illegal drainage connections to the sewerage system from houses must accordingly be avoided, since this can cause surcharge to the ground during rain events.

The average per-capita wastewater production was measured at 107 l/capita/day without infiltration in 1999. This value is derived from the observed domestic per-capita wastewater flow of 92 l/d divided by the ratio to the total wastewater flow of 0.86, which includes commercial and industrial wastewater. The same value will increase to 110 l/capita/day in 2008, at the end of phase I development.

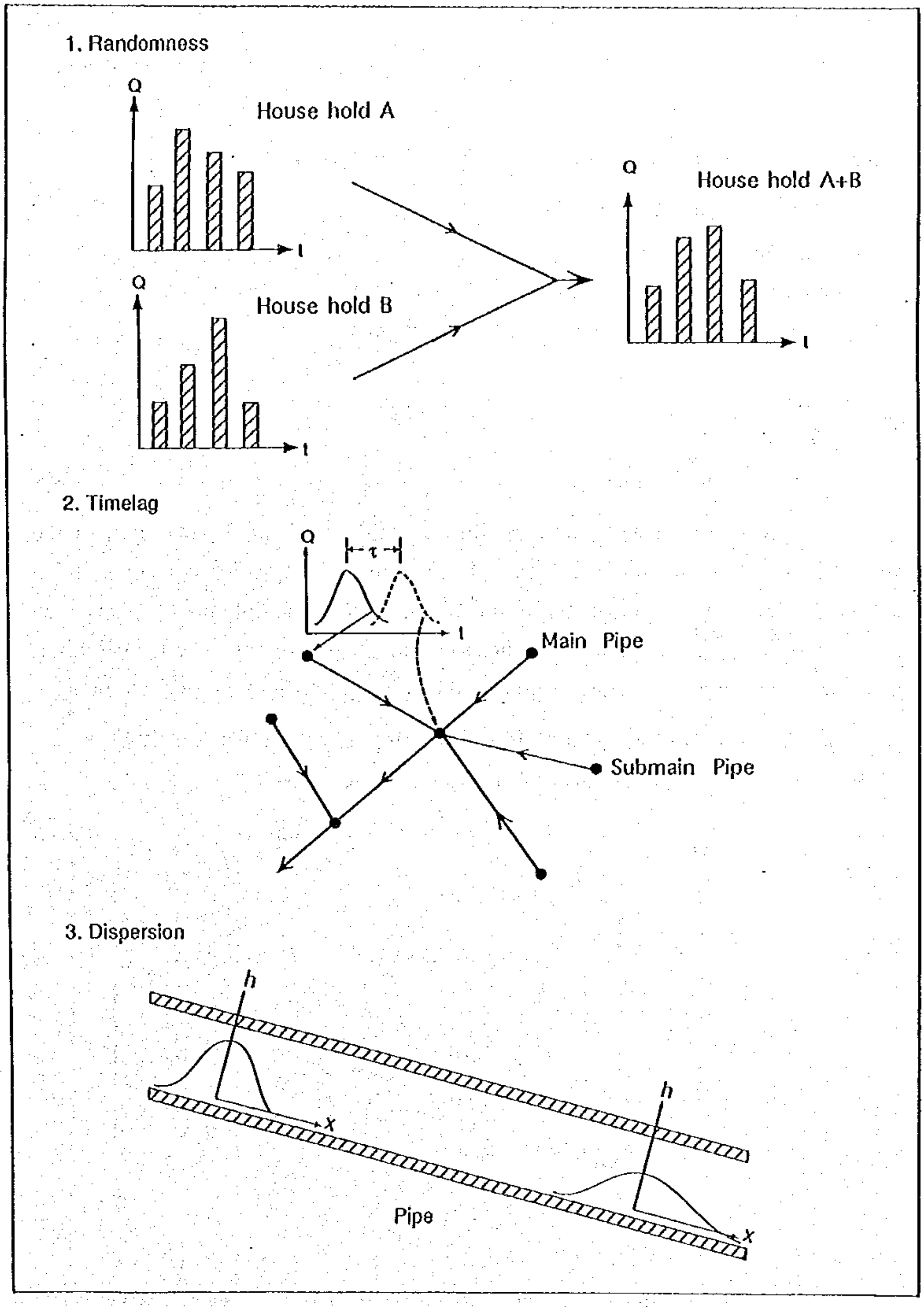
The daily wastewater flow pattern is based on the flow survey conducted in the dry season (July 1999). The data from this survey have been analyzed in order to extract reliable wastewater time series. Determination of the hourly peak flow rate plays an important role in designing the sewerage system.

The daily peak flow factor reduces as larger populations are taken into consideration. The peak factor reduces due to 3 effects, these are:

1. Reduction through the diversity in the wastewater production between different households.
2. Time lag due to transportation in the system, reducing the overlap between different households' peak production of wastewater.
3. Dispersion effects in the pipes, manholes retention basin's and pump wet-wells.

These effects are illustrated in Figure 3.1. Effect (1) is most significant for population sizes between 2-500 people after which the effect fades out. Effects (2) and (3) become more important for larger population sizes (1000 people and up).

Figure 3.1 Reducing Effects to Daily Peak Flow Factors



3.2 Simulation results

The following simulations were carried out.

- Present model as July 1999 with an average wastewater flow of 107 l/day/PE (population equivalent).
- Phase I (year 2008) model with average flow of 110 l/day/PE.

The results of interest are:

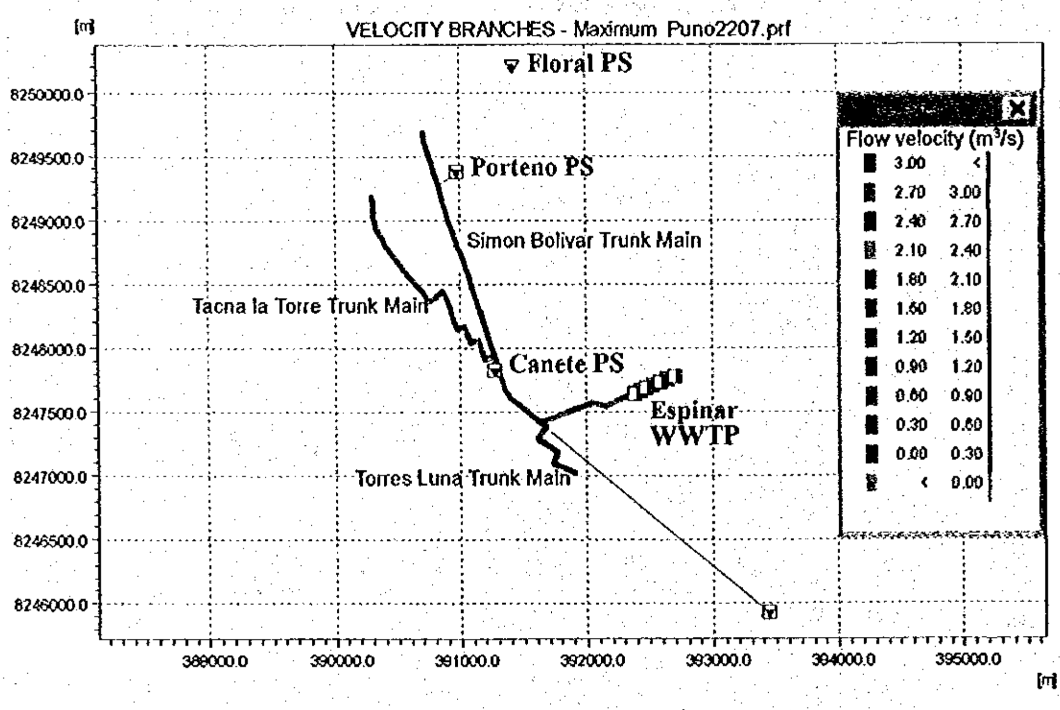
1. velocities in the pipes.
2. how full the pipes are.
3. inflow to the Espinar wastewater treatment plant.

3.2.1 Present Condition

(1) Flow velocities in the pipes

Figure 3.2 shows the expected minimum flow velocity in the plan plot. Part of the sewer network has the maximum velocities lower than 0.6 m/s, which is recommended minimum velocity for self-cleaning of the pipes. Sedimentation in the pipes may occur especially in the dry season as observed in the Puno city. Periodical high-pressure water cleaning is required as practiced by EMSAPUNO to remove sediments.

Figure 3.2 Maximum Flow Velocity in the Existing Sewer Network in 1999



(2) Water levels in the trunk mains

Figures 3.3 – 3.6 show the expected maximum water levels in the longitudinal profile of the trunk mains, Simon Bolivar, Tacna la Torre, Torres Luna and the principal interceptor (ϕ900) to the Espinal wastewater treatment plant.

The maximum water levels in the pipes are generally low, less than one third of the pipe diameters. In a few locations, maximum water levels exceed the pipe diameters. At the time of the flow survey in the dry season (22 July 1999), pump on/off levels in the Canete pump station were set as 3808.5 m and 3807.8 m A.S.L. respectively, which caused the high water levels in the Simon Bolivar trunk main close to the pump station as shown in Figure 3.3. The sewer pipes flowing completely filled provide conditions particularly favorable for sulfide generation. In such conditions, very little oxygen may be transferred to the wastewater from the atmosphere. Resulted anaerobic condition causes H_2S generation, which may become a significant problem such as:

- severe corrosive conditions for unprotected sewer pipes and manhole walls produced from cementitious materials and metals occur when sulfuric acid (H_2SO_4) is derived through the oxidation of hydrogen sulfide by bacterial action. The corrosive effects of sulfuric acid make the surface cementitious material converted to a pasty mass which may fall away and expose new surfaces to corrosive attack.
- H_2S gas is extremely toxic. Deaths have resulted from an H_2S concentration of as low as 0.03 % (300 ppm) in the air. Before sewer inspections through manholes, ventilation must be provided to lower H_2S concentration to the safe level.

Figure 3.3 Longitudinal Profile of the Simon Bolivar Trunk Main in 1999

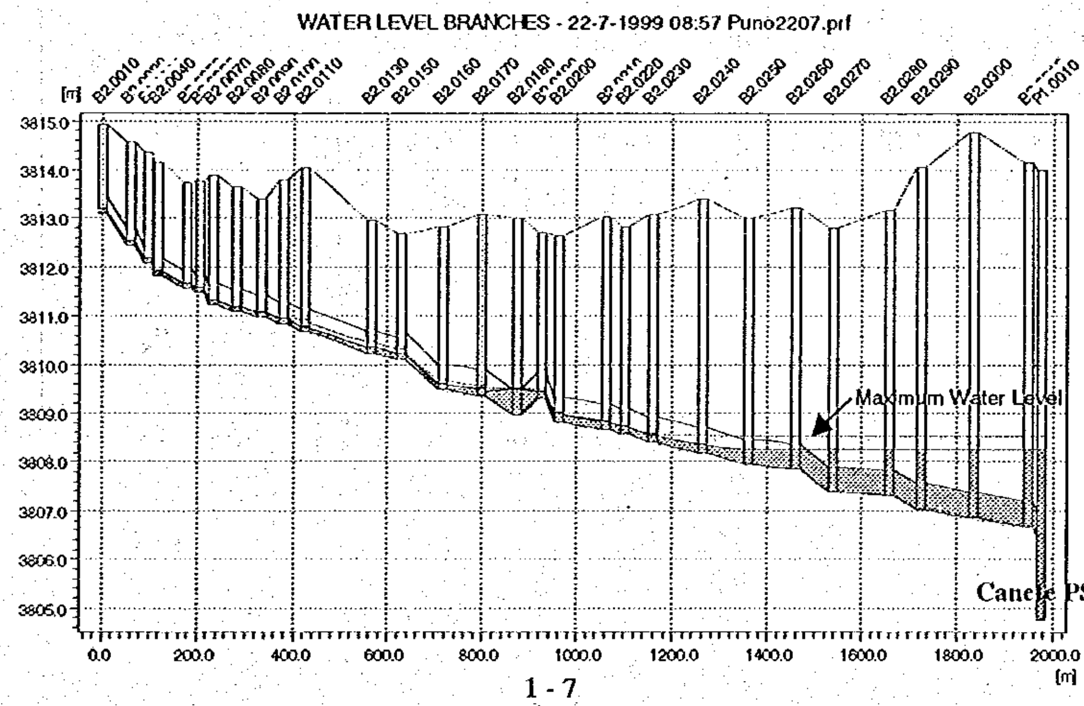


Figure 3.4 Longitudinal Profile of the Tacna la Torre Trunk Main in 1999

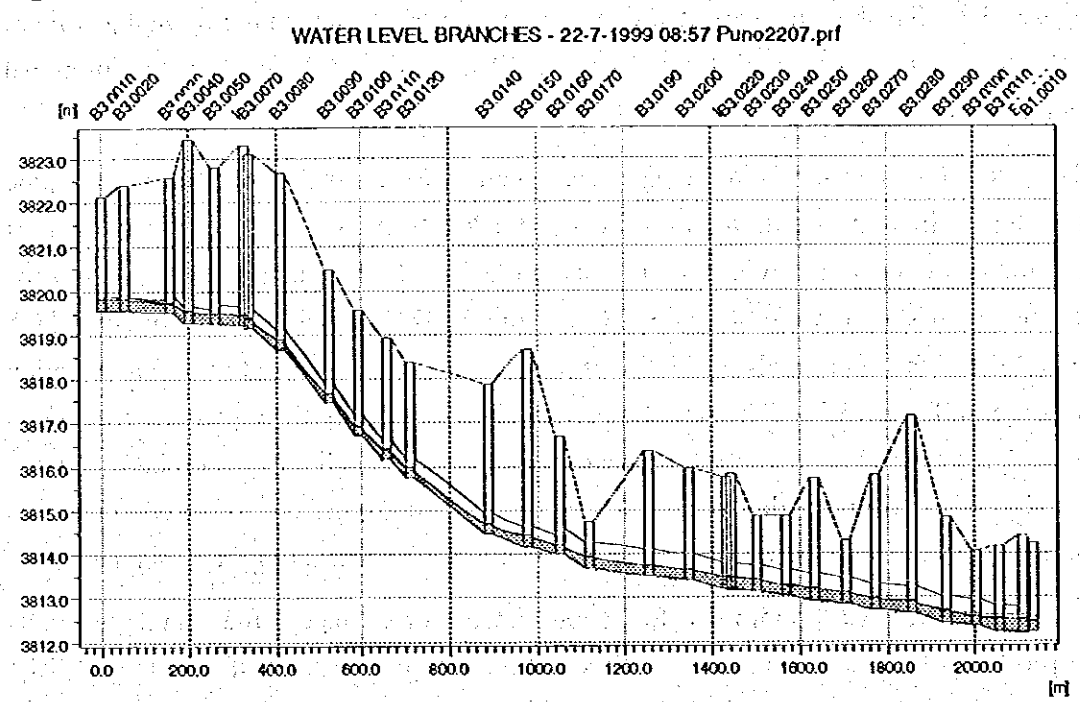


Figure 3.5 Longitudinal Profile of the Torres Luna Trunk Main in 1999

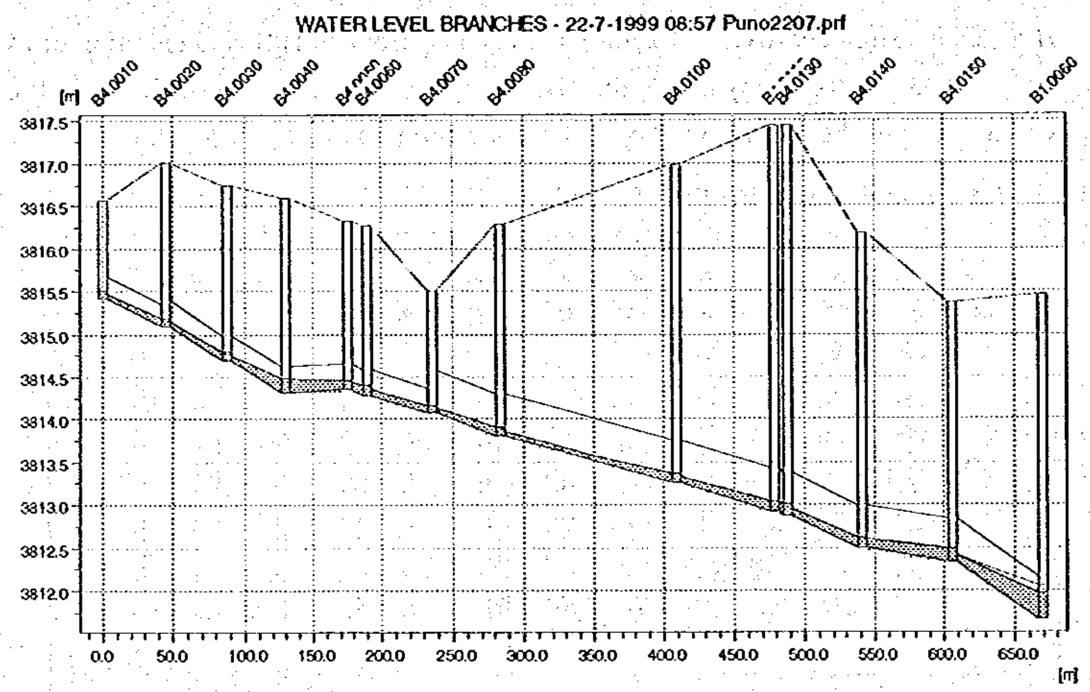
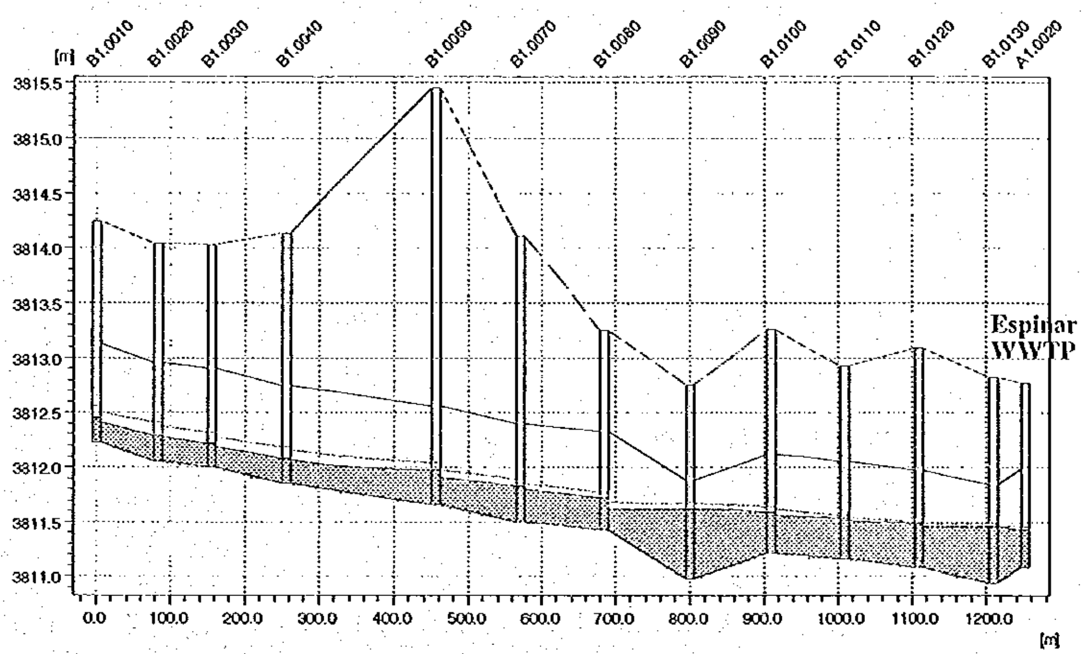


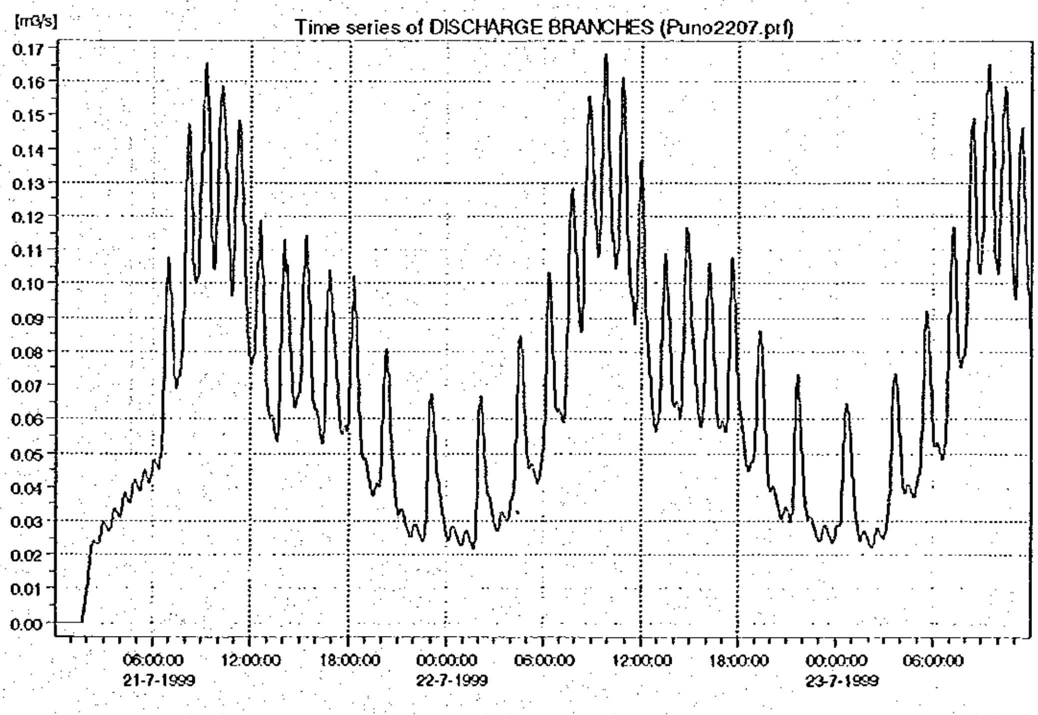
Figure 3.6 Longitudinal Profile of the Principle Interceptor in 1999



(3) Inflow to the Espinar wastewater treatment plant

The inflow to the treatment plant is shown in Figure 3.7. The peak flow occurs at around 9 am as observed in the flow survey in the dry season.

Figure 3.7 Expected inflow to the Espinar wastewater treatment plant in 1999



3.2.2 Phase I (Year 2008) model

Major improvement of the sewer network through phase I development is as follows:

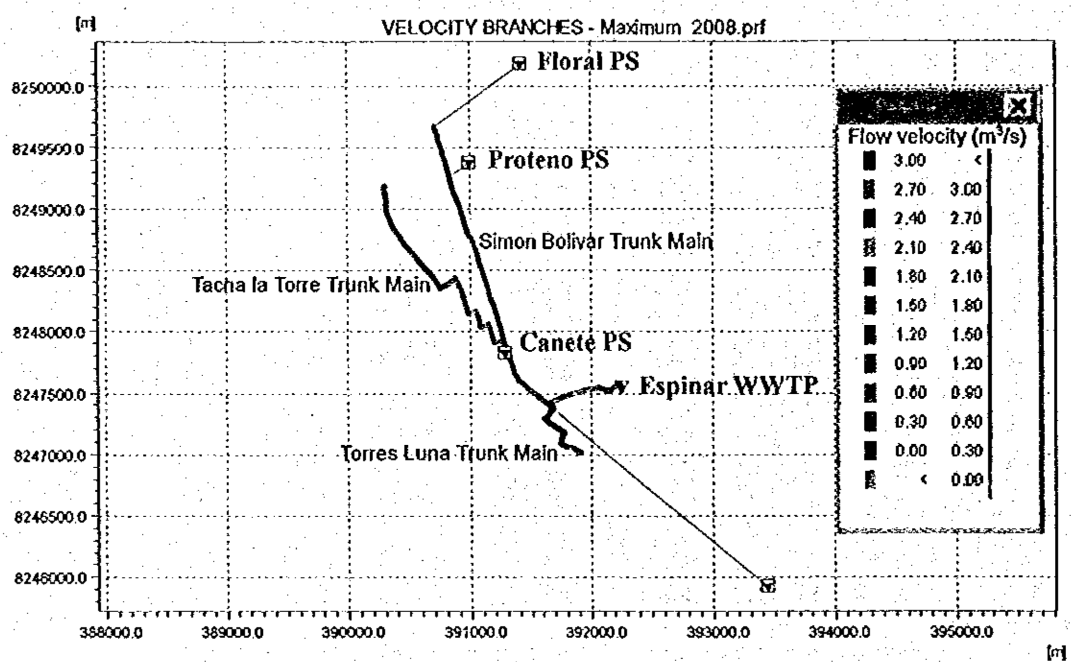
1. Extension of collection pipe network by 25 km.
2. Replacement of the part of the trunk mains (Simon Bolivar, Tacna la Torre, Torres Luna).
3. Introduction of El Puerto pump station.
4. Diversion of wastewater from Chejona area to the Aziruni pump station.

The above improvement is reflected in the developed model.

(1) Flow velocities in the pipes

The velocities in the pipes during the peak flow will increase from the present condition (Year 1999) and sedimentation problems will become less pronounced in Year 2008 as shown in Figure 3.7. Maximum velocities less than 0.3 m/s are still observed mainly in the Torres Luna trunk main.

Figure 3.8 Maximum Flow Velocity in the Phase I Sewer Network in 2008

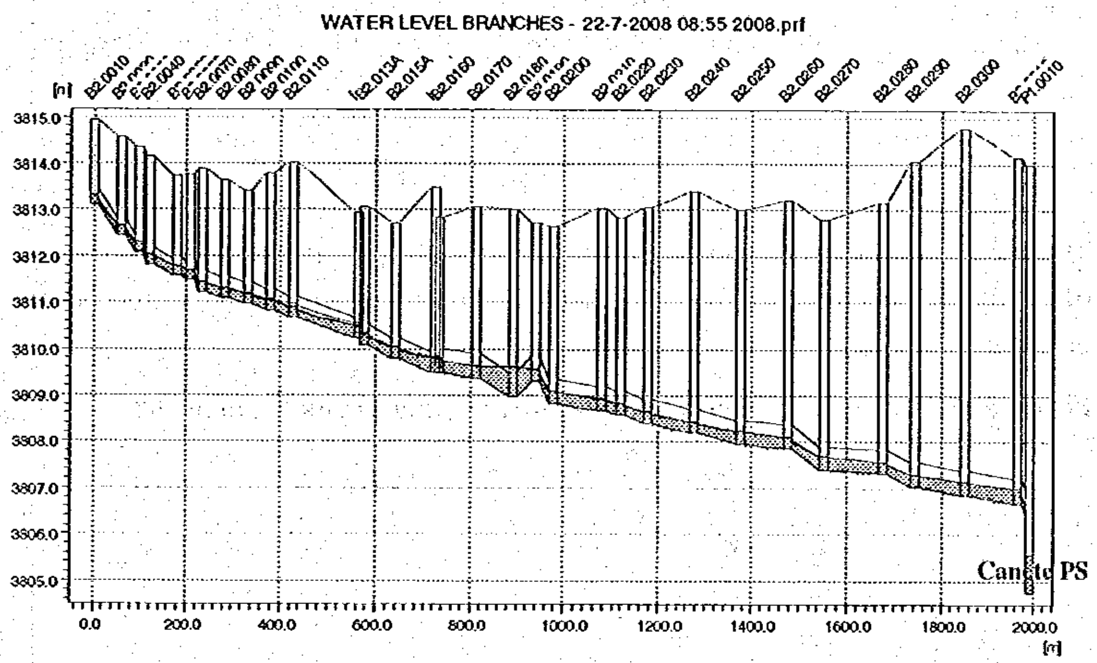


(2) Water levels in the trunk mains

Figures 3.9 – 3.12 show the expected maximum water levels in the longitudinal profile of the trunk mains in 2008.

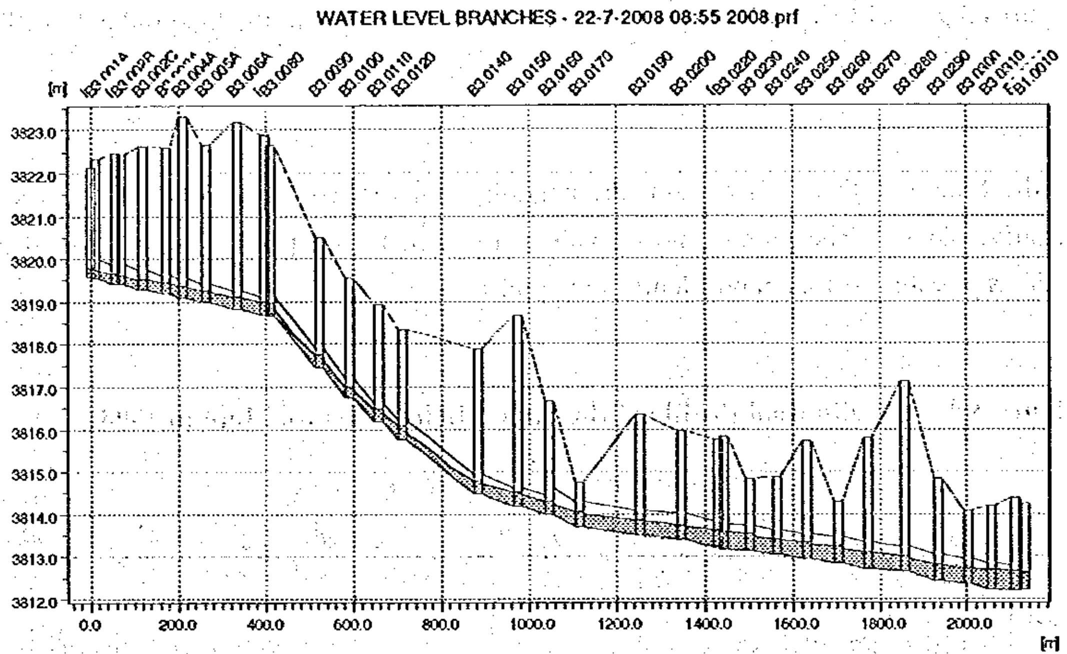
In the Simon Bolivar trunk main, the conduit from <B2.0180> to <B2.0190> has a negative slope. The pump on/off levels at the Cancte pump station are lowered to address possible corrosion problems in the pipes.

Figure 3.9 Longitudinal Profile of the Simon Bolivar Trunk Main in 2008



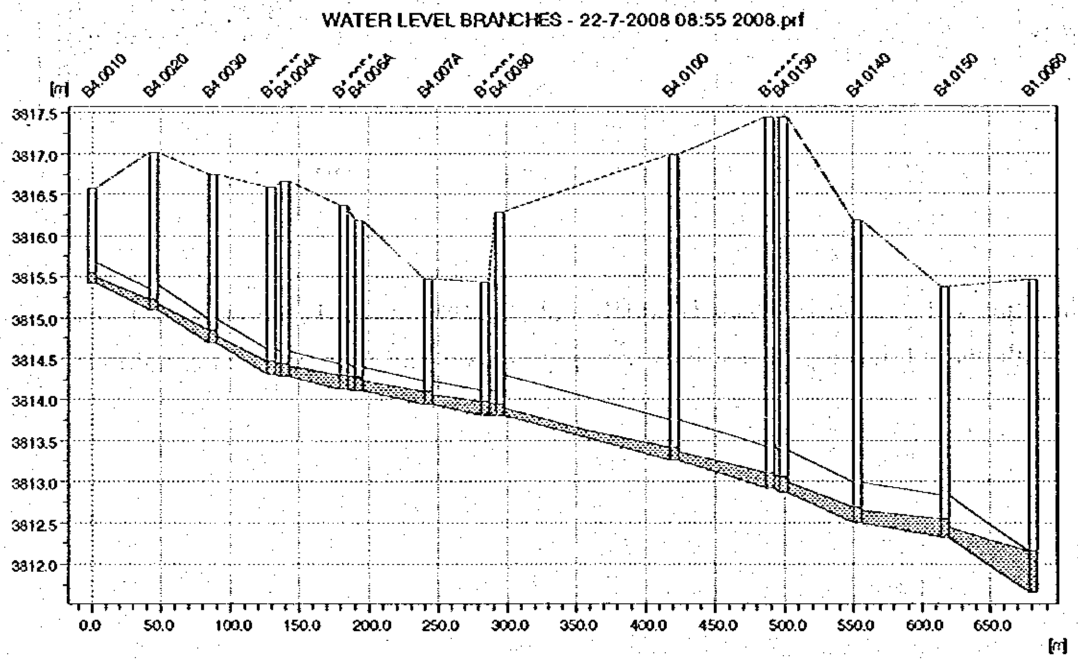
The pipe replacement between B3.0010 and B3.0080 in the Tacna la Torre trunk main through phase I development solves flow problems found in the present condition.

Figure 3.10 Longitudinal Profile of the Tacna la Torre Trunk Main in 2008



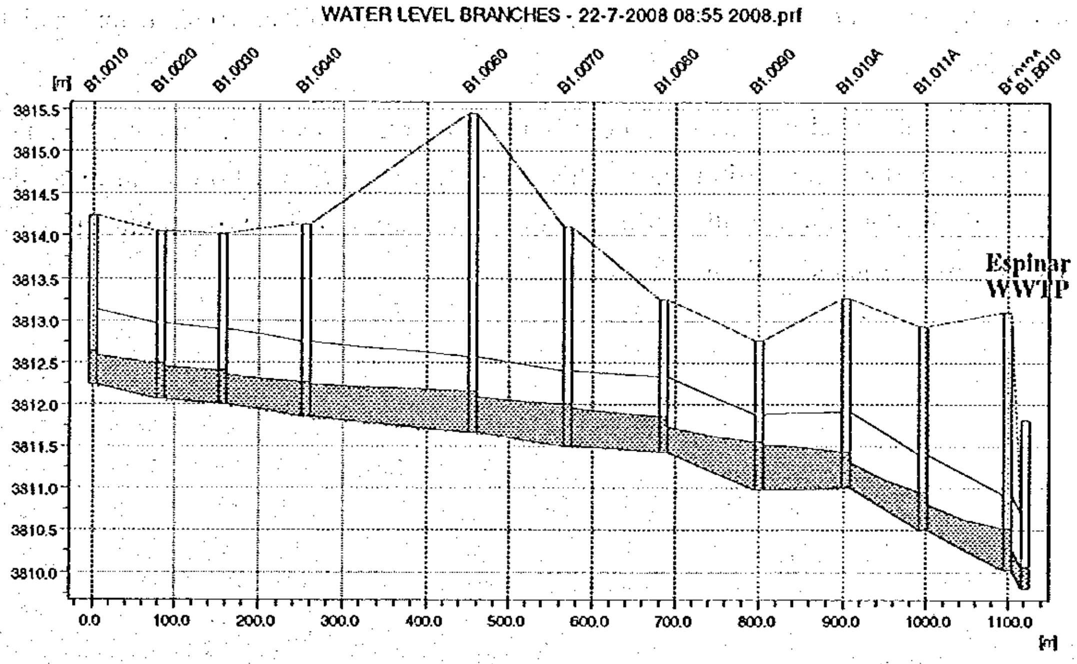
The maximum water levels do not reach the half of the pipe diameters in the most part of the Torres Luna trunk main. This results in the low velocity of the wastewater flow in this main.

Figure 3.11 Longitudinal Profile of the Torres Luna Trunk Main in 2008



The principle interceptor is less than 50 % filled during peak hours in the most parts.

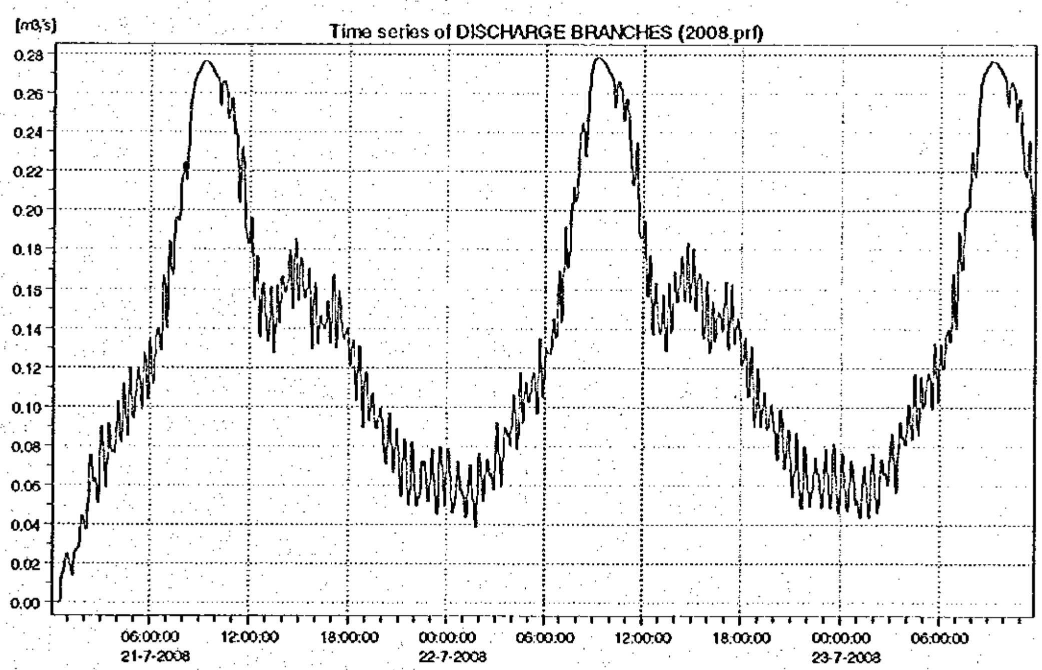
Figure 3.12 Longitudinal Profile of the Principle Interceptor in 2008



(3) Inflow to the Espinar wastewater treatment plant

The inflow variation to the treatment plant is shown in Figure 3.13. The peak flow occurs at around 9 am and reaches 0.28 m³/s.

Figure 3.13 Expected inflow to the Espinar wastewater treatment plant in 2008



(4) Overall evaluation of phase I development

With proposed development of phase I, the sewer network will be in sound condition under dry weather. The negative slope in the Simon Bolivar trunk main shall be investigated.

At present, observed wet weather flow is two to three times of the dry weather flow because of inflow and infiltration. This causes wastewater overflow from manholes, especially in the Simon Bolivar trunk main. Infiltration and inflow control program shall be established to address this problem.

2. DESIGN PROCEDURE FOR EXTENDED DETENTION POND

2. DESIGN PROCEDURE FOR EXTENDED DETENTION PONDS FOR CONTROL OF URBAN RUNOFF QUALITY

(Excerpt from "Controlling Urban Runoff: A Practical Manual For Planning And Designing Urban BMPs" by Thomas R. Schueler, July 1987)

1. DESIGN OF EXTENDED DETENTION PONDS (SUMMARY)

• QUANTITY DETAINED:

At a minimum, the volume of runoff detained should be equivalent to the runoff volume produced by a one inch storm. This volume is sufficient to achieve both high levels of particulate removal and downstream channel erosion protection for most of the storms that occur during a year. Higher levels of control can be achieved when the runoff volume from the one or two year storm is detained.

• DURATION:

24 hours of extra detention are needed for optimal pollutant removal for the design detention volume. The control device should be adjusted so that smaller runoff events (0.1 to 0.2 inches), which normally pass through the pond quickly, are detained for at least a minimum of six hours. In larger watersheds, up to 40 hours of extended detention may be needed for streambank erosion control. As a final check, the runoff velocity of the downstream channel at the extended detention release rate should be computed to make sure that it is not erosive.

• TWO STAGE DESIGN:

A two-stage pond design is recommended when extended detention is applied to dry ponds. The upper stage of the pond is sized and graded (2% minimum) to remain dry except during infrequent large storms, while the bottom stage is expected to be regularly inundated. The volume of the bottom stage should be set to store the runoff produced by the mean storm (approximately 0.45 inches). The bottom stage will frequently be too wet to mow, and is best managed as a wetland or as a shallow pool. Both techniques act to prevent resuspension of previously deposited materials. Extra storage, over and above stormwater and extended detention requirements, should be provided within the bottom stage, or at the inlet to account for 20 years of sediment deposition.

• **WETLAND CREATION:**

Wherever possible, a wetland marsh should be created in the bottom stage of an extended detention pond to help remove soluble pollutants that cannot be removed by conventional settling. Wetlands also provide wildlife habitat and hide unsightly debris and sediment deposits that frequently accumulate near the riser. The area of the wetland should be adjusted so that the average annual watershed loading (as computed by the Simple Method) does not exceed 45 pounds of phosphorus or 225 pounds of nitrogen per surface acre of wetland. Water depths of 6-12 inches are needed for optimal wetland growth. The wetland should be planted with native species which are suited to that environment.

• **EXTENDED DETENTION CONTROL DEVICE:**

In dry ponds, a vertical, internally controlled extension of the low flow orifice is the most trouble-free design, since it can withstand partial clogging and gradual sediment accumulation, and also can be used to set water levels. If the control device is below the ground surface, it should be protected with filter cloth and/or wire mesh, and encased in a trench of stone or gravel with a diameter greater than the orifice. The device should also have an above-ground extension, with a tight-fitting replaceable cap to facilitate clean-out. In wet ponds, a negatively-sloped pipe protected by a wire mesh that extends from riser and withdraws water from at least a foot below the surface should be used.

• **PILOT CHANNELS**

A riprap, concrete or paved low flow channel is required to route water through the upper stage of the extended detention pond. The pilot channel should end at the lip of the lower stage, where riprap or gabion baffles are placed to reduce velocities and spread out the flow path of the runoff reaching the lower stage, thus preventing scour and resuspension.

• **SIDE-SLOPES:**

Side slopes should be no steeper than 3:1 (h:v), and no flatter than 20:1 (h:v).

• **POND BUFFER:**

A minimum 25 foot wide buffer strip away from pond to the nearest lot should be reserved, and landscaped using low-maintenance grasses, shrubs and trees. A landscaping plan should be prepared for the pond and buffer, that improves the appearance for adjacent residents, meets specific design functions, and provides local wildlife habitat.

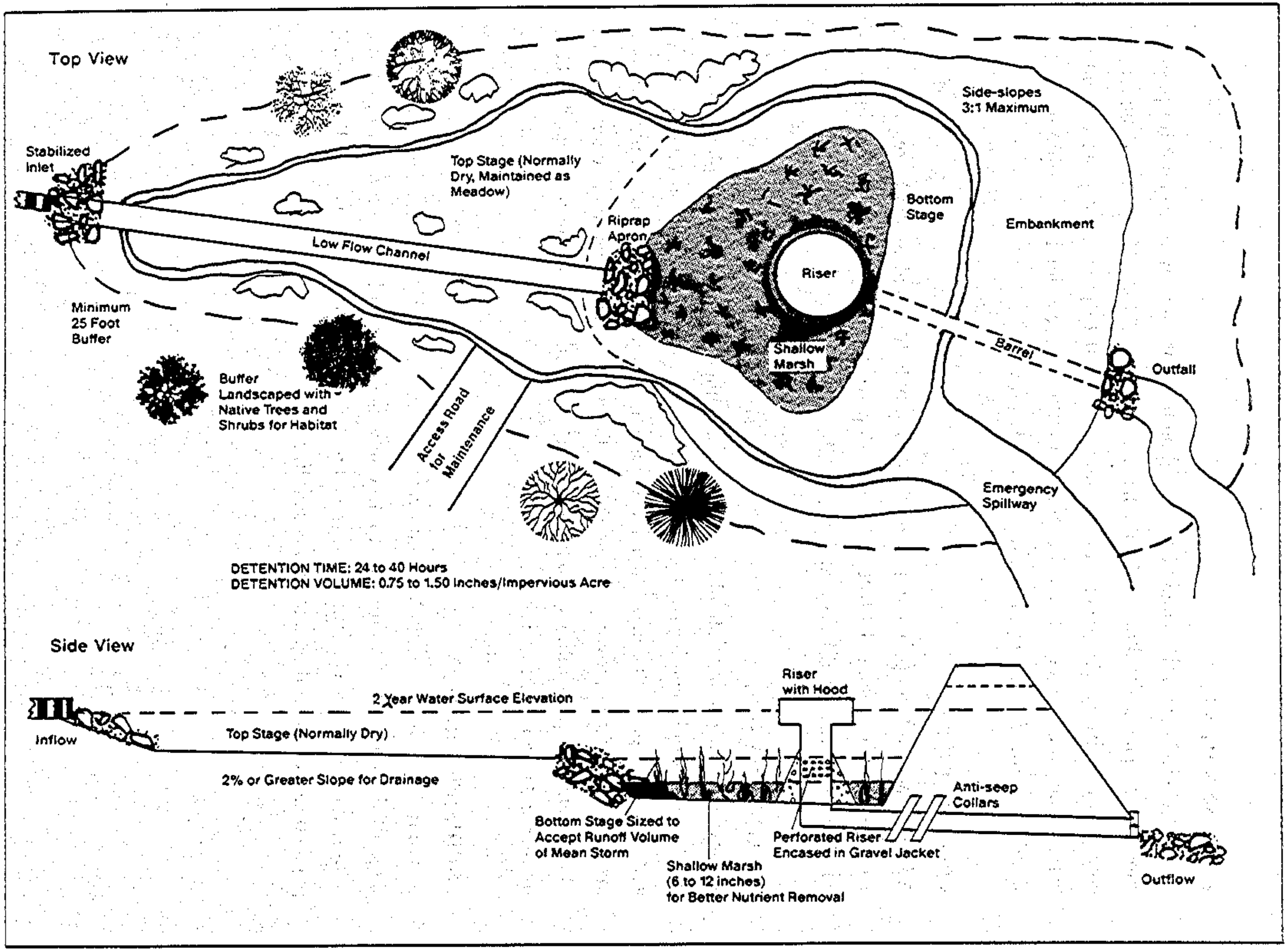
• EMBANKMENT:
At least 10-15% extra fill should be allowed on the embankment to account for possible subsidence. The embankment should have at least one foot of freeboard above the emergency spillway. Anti-seep collars should be used to prevent seepage around the barrel. The embankment should be graded to allow access for heavy equipment, and should be mowed twice a year to prevent woody growth.

• SITE ACCESS:
Adequate access from public or private right of way to the pond should be reserved. The access should be at least 10 feet wide, on a slope of 5:1 (h:v) or less, and stabilized to withstand the passage of heavy equipment.

• MAINTENANCE:
Wet-weather inspections should be conducted annually, with as-built plans in hand. Inspections should emphasize the condition of the extended detention control device and low flow pilot channel. Extended detention facilities should be maintained as a meadow to reduce mowing frequency (2 times per year) and maintenance costs. Maintenance responsibilities should be clearly vested with funds reserved for both routine and non-routine activities.

• SEDIMENT REMOVAL:
A five to ten year sediment clean-out cycle is recommended. Extra storage in the lower stage of the pond can be provided to accommodate sediment deposition. Also, on-site sediment disposal areas should be reserved to reduce removal costs. Do not begin final pond construction until upland area is stabilized.

Figure 1.1 Schematic of Extended Detention Pond Design Feature



2. POLLUTANT REMOVAL

Settling is the primary pollutant removal mechanism associated with extended detention. As such, the degree of removal is dependent on whether a given pollutant is in particulate or soluble form. Removal is likely to be quite high if a pollutant is particulate, whereas very limited removal can be expected for soluble pollutants. Unfortunately, some of the urban pollutants of greatest concern occur primarily in soluble forms (e.g, nitrate and ortho-phosphorus). Removal of these soluble pollutants may be obtained if the lower stage of the extended detention pond is managed as a shallow wetland to utilize natural biological removal processes.

2.1 Settling Behavior of Urban Pollutants

The settling behavior of urban pollutants has been evaluated in a series of laboratory and field studies. Grizzard et al. (1986), Driscoll (1986), and Whipple and Hunter (1981) have utilized experimental settling column data to assess pollutant-settling behavior over time. In each study, urban runoff was introduced into four to six foot deep plexiglass chambers and the change in pollutant concentration over time was measured at sampling ports located at different depths on the column. In addition, the long term pollutant removal performances of two extended detention ponds have been evaluated in local field monitoring efforts. During the Washington NURP study (MWCOG, 1983b) a dry pond (Stedwick) in Montgomery County, Maryland was modified to achieve 6-12 hours of extended detention, and monitored over a 18 month period. Interim results are also available for an extended detention pond (London Commons) monitored in suburban Northern Virginia (OWML, 1986a). Together, these studies provide a basis for estimating the detention time needed to obtain maximum possible removal for specific pollutants of interest listed below.

SEDIMENT

The settling column experiments indicated that 60-70% of urban sediments settle out within the first six hours. The remaining sediment may take as much as 2 days to settle out (Figure 2.1). Maximum removal rates after 48 hours of detention ranged from 80-90%. The rather slow sediment settling rates are primarily due to the very fine-grained particle distribution of sediment in urban runoff (OWML, 1983). Washington NURP field monitoring at the Stedwick extended detention pond generally supports the lab measurements. The pond was estimated to remove 65% of incoming sediment over the long term (MWCOG, 1983a), which is similar to the 6-12 hour removal rate reported in the settling column study (Figure 2.2). An average storm removal of approximately

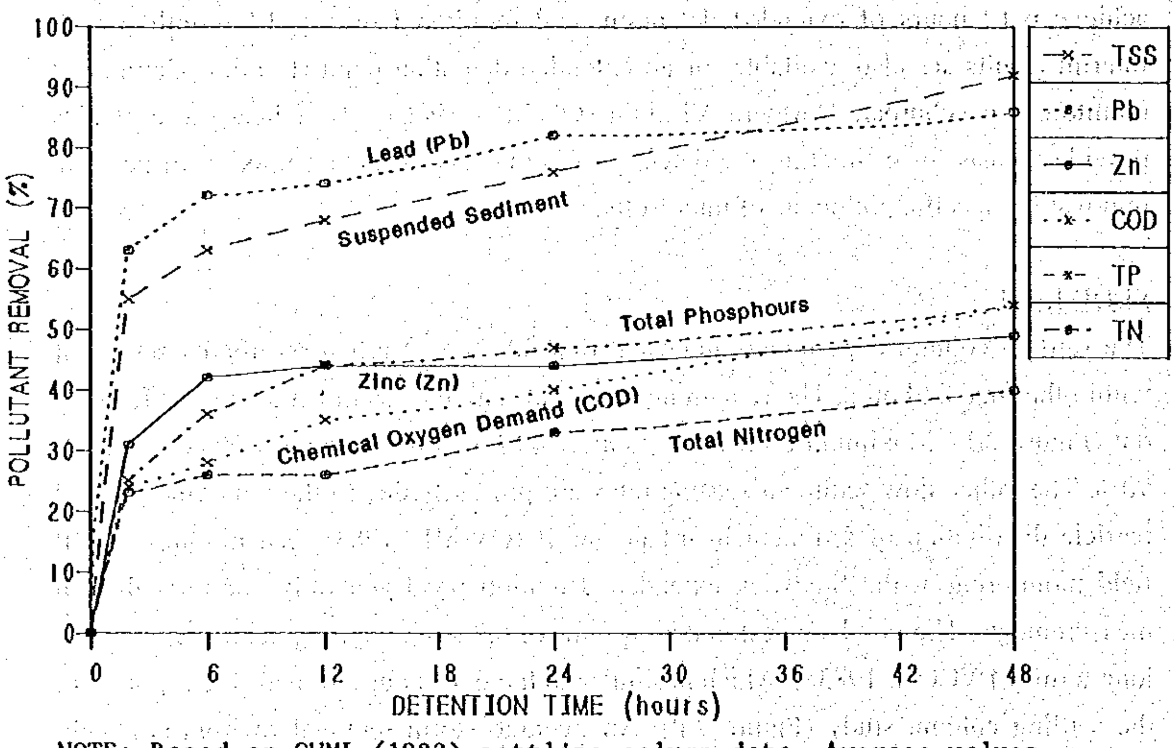
65% was also reported for the London Commons pond (OWML, 1986a), which also experienced relatively brief detention times (estimated at 6-12 hours).

PHOSPHORUS

Both settling column studies indicated a maximum upper limit of about 40-50% removal for total phosphorus after 48 hours, with most of the removal occurring within the first 6 to 12 hours. The upper limit for phosphorus removal by settling is due to the fact that soluble forms comprise over half of all phosphorus found in urban runoff. Nearly all the particulate phosphorus settled out in the OWML experiments, accounting for the majority of observed removal. In addition, a small fraction of soluble phosphorus adsorbed to sediment and eventually settled out during the experiments.

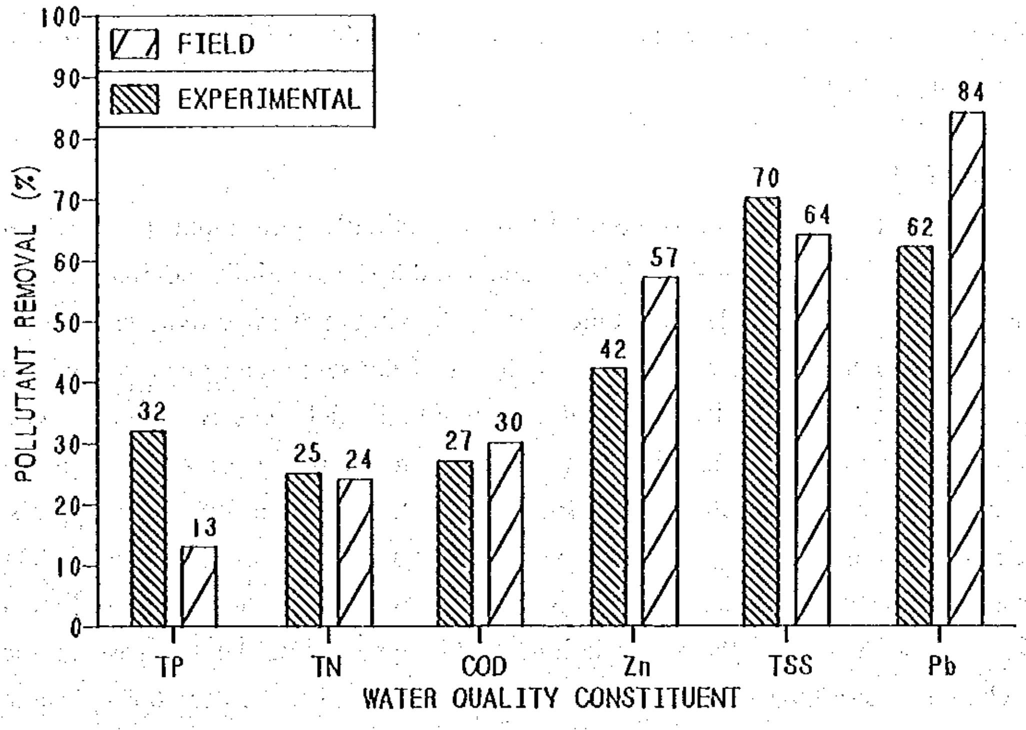
The field studies showed variable performance in removing phosphorus. Less than 15% of total phosphorus was removed at the Stedwick site over the long-term; whereas, initial results at the London Commons site indicated much higher average (70%) total phosphorus removal (OWML, 1986a). However, it is very likely that the long-term total phosphorus removal at the site is much lower, since very low (or even negative) removal rates were reported for larger storms. Resuspension of total phosphorus was cited as the likely cause.

Figure 2.1 Removal Rate vs. Detention Time For Selected Pollutants



NOTE: Based on OWML (1983) settling column data. Average values for seven tests. Removal equivalent to 4 feet of settling.

Figure 2.2 Urban Pollutant Removal after 6 to 12 Hours Detention Time Comparison of Lab Studies and Field Measurements



NITROGEN

In the OWML (1983) settling column experiments, the upper limit on nitrogen removal achieved after 48 hours of detention was about 40%. Again, this is due to the predominance of soluble forms of nitrogen that comprise about 70-80% of the total nitrogen found in the Washington, D.C. area urban runoff (NVPDC, 1983). Field studies at the Stedwick extended detention pond suggested a long-term total nitrogen removal rate of about 25%, which compares well with the lab studies (Figure 2.2). Almost all of the particulate nitrogen settled out from the pond, but only limited settling of soluble nitrogen forms was reported. A higher average storm removal of total nitrogen was reported (52%) at the London Commons site (OWML, 1986a), although the long-term removal rates may not be as high.

ORGANIC MATTER

Organic matter, as measured by BOD in Whipple and Hunter (1981) and COD in OWML (1983), exhibited similar settling behavior in the column tests. Average maximum removal after 32 and 48 hours, respectively, was about 40-50%. Organic matter exhibited rapid settling rates over the first 6-8 hours, followed by gradual but steady removal thereafter. Long-term COD removal rates at the Stedwick site were on the order of 30%, which compare favorably to the six-hour detention removals observed in the lab (Figure 2.2).

TRACE METALS

Settling of most trace metals in the column tests was initially quite rapid. Lead, which has a close affinity with suspended sediment, exhibited essentially similar settling behavior (Figure 2.1) (Whipple and Hunter, 1981). Maximum average removal after 48 hours was greater than 90%, with about two-thirds of the settlement occurring within the first six hours. Long-term lead removal measured in the field was even greater, with 84% removal recorded after the first 6 hours. Maximum removal of zinc was much lower, averaging about 50% in the OWML experiments and about 30% in Whipple's. Unlike lead, most of the zinc (<70%) in urban runoff is in soluble form (NVPDC, 1983). However, a significant portion of the soluble zinc appears to adsorb to sediment particles and settle out of the water column. This appeared to be the case at the Stedwick site, where long-term removal rates were estimated to be near 60%, despite the fact that less than 20% of the incoming zinc was in particulate form at the site.

OTHER POLLUTANTS

Whipple and Hunter (1981) noted an order of magnitude reduction in bacterial counts after 32 hours of detention. Also, about 60-70% removal of hydrocarbons was reported over the same interval.

2.2 Additional Removal by Biological Means

Biological removal of soluble pollutants can be achieved by creating artificial wetlands in the lower stage of a dry extended detention pond. Marsh plants, algae and bacteria that grow on the shallow, organic rich sediments can take up soluble forms of nutrients needed for their growth. Also, the marsh sediments are an excellent substrate for pollutant sorption. The degree of pollutant removal attained in shallow wetlands is uncertain, but appears to be dependent on the size of the wetland in relation to pollutant load delivered to it (Nichols, 1983). Removal varies seasonally, with the most removal during the growing season, and the, least removal occurring in the late fall and winter after the plants have died back.

Wetlands can sometimes become a net source of nutrients in the fall and winter months, as nutrients stored in above-ground plant tissue are "pumped out" to the water column during senescence. Indeed, the only permanent sinks for pollutants in an artificial marsh are gradual burial in the sediments, harvesting, and occasional episodes of denitrification. However, even though much of the incoming nutrient load may only be temporarily stored in wetlands, the nutrients are released at a time of the year when they will have the least direct impact on receiving waters.

3. DESIGN TIPS FOR ENHANCING POLLUTANT REMOVAL

3.1 Detention Time

For water quality purposes, detention times of at least 24 hours are probably necessary to achieve maximum removal of most pollutants. While most of the settling occurs within the first 12 hours in the settling column experiments, it is advisable to provide further detention since several hours may be needed before ideal settling conditions develop in a pond. Slightly longer detention times may be needed for downstream channel erosion.

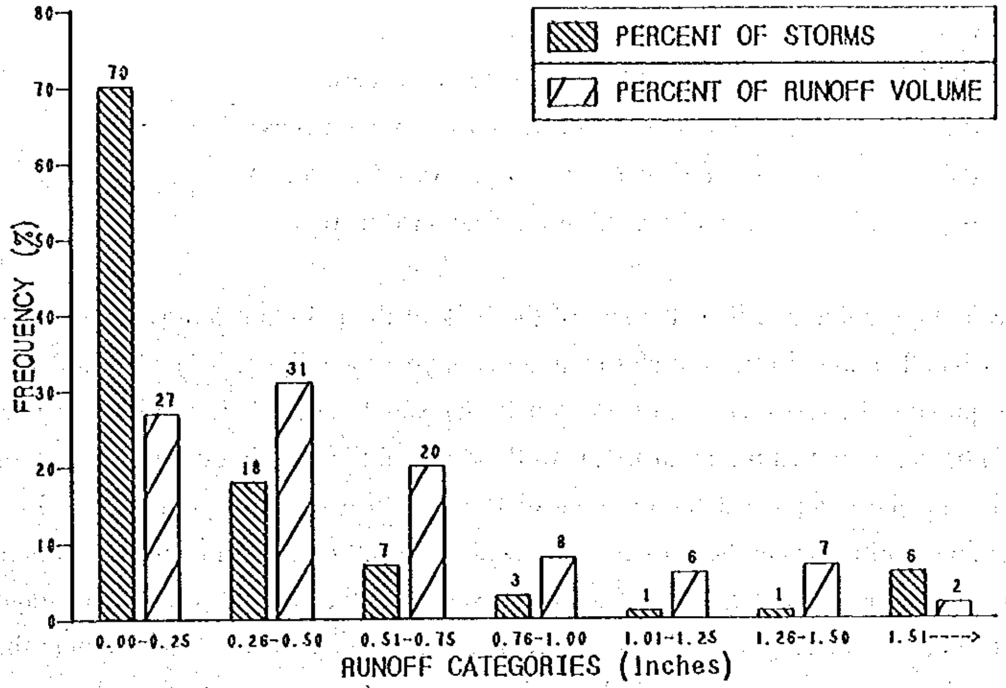
3.2 Achieving Adequate Detention For All Storms

One of the most difficult problems in extended detention design involves sizing the control device so that it provides adequate detention time for the entire spectrum of storms. For example, if an extended detention pond is designed to store and release the one-year storm over a 24 hour period, storms smaller than the one year storm event will pass through the orifice much more rapidly, and in some cases, may only have an average detention time of a few hours. Unfortunately, small storms deliver a majority of the annual runoff volume to the pond (Figure 3.1). As a result, the annual pollutant removal of the extended detention pond may be reduced if the small storms are not adequately detained.

Therefore, it is recommended that the pond designer perform several storage routing calculations (TR-20 method or equivalent) to determine the approximate detention time for the smaller, more frequent runoff events. Grizzard et al. (1986) suggest that as a target the average detention time in the pond should be 24 hours for the entire spectrum of storms each year. This can be done if the maximum detention time for the maximum detention volume is about 40 hours. Figure 3.8, which shows the approximate size distribution of storm runoff events in the Washington, D.C. area for moderately developed small watersheds, can be used to estimate inflow hydrographs for small storms for the routing calculations (i.e., the runoff volumes can be converted to SCS Triangular Unit Hydrographs).

As a general rule, it is recommended that the average detention time for small runoff events (0.1-0.2 inches) should be no less than six hours.

Figure 3.1 Frequency Distribution of Runoff Events in Moderately Developed Watersheds



NOTE: Based on Washington D.C. metropolitan area data; 300 storms, seven sites, I=10-30%.

3.3 Quantity Detained

The amount of runoff detained heavily influences the pollutant removal performance of an extended detention pond. Incoming runoff is only partially treated if a storm exceeds the detention storage volume provided in the pond. At a minimum, extended detention ponds should be sized to accommodate the runoff produced by the mean storm, and preferably should be capable of storing the runoff volume of a one-inch storm. However, in many cases, the stricter storage requirements recommended above for streambank erosion control (1.0-1.5 inches * Rv) will govern how much extra detention storage is needed.

3.4 Pond Shape: Two Stage Design

A two stage extended detention pond is recommended to improve pollutant removal and reduce maintenance requirements. Basically, the upper stage of the pond is intended to be dry except during large infrequent storms, whereas the lower stage is sized to accept regular inundation. As a general rule, the lower stage should have a minimum volume equivalent to:

(EQ 3.1) Vol b = [(Rm)(Rv)/12] (A)

where

Vol b	=	volume of bottom stage (acre-feet).
Rm	=	volume of mean storm (0.4 to 0.5 inches).
Rv	=	rainfall/runoff coefficient (see Chapter 1).
A	=	area of contributing watershed (acres).

The lower stage volume will be the site of the bulk of the pollutant removal, as it will normally handle about 50-90% of storms in a given year (see Figure 3.1). Care must be taken to prevent the resuspension of previously deposited materials in the lower stage. This can be done by creating an artificial wetland to stabilize the bottom sediments, or by modifying the extended detention control to create a permanent pool. The risk of resuspending pollutants can be further minimized by installing a riprap apron or gabion baffle between the the pilot channel of the upper stage and the bottom of the lower stage. The two stage design (Figure 1.1) helps to reduce the velocity of runoff as it enters the lower stage, prevents concentrated flows from scouring or resuspending deposited sediments, and improves the overall settling characteristics of the lower stage.

3.5 Marsh Establishment

Wetland vegetation in the lower stage of an extended detention pond enhances removal of soluble nutrients and has several other benefits as well. Emergent marsh plants such as three-square, sedges, spatterdock, switchgrass and bulrush provide an attractive habitat for both wildlife and waterfowl, enhance sediment trapping, prevent sediment resuspension, and conceal trash and debris that normally accumulate near the riser.

Studies of the capacity of wetlands to assimilate wastewater indicate that they perform best when exposed to relatively dilute nutrient loads. Nichols (1983) presents summary data from many sites around the nation that suggests that maximum levels of nutrient removal can be achieved if loadings do not exceed 45 pounds of phosphorus or 225 pounds of nitrogen per surface wetland acre per year. Until more accurate criteria are developed from ongoing research on actual extended detention wetlands, these guidelines may be used to size artificial wetlands (see example 3-1).

The inlet-controlled slotted standpipe (Figure 3.2a). is probably the best control device for creating shallow wetlands in extended detention ponds because it can regulate water levels within the lower stage, and also maintain target detention times even when partially clogged.

3.6 Pilot Channels

Erosion will often occur within the low flow channel through the upper stage of an extended detention pond, unless it is stabilized by riprap. The lack of channel protection within the pond can actually make a pond a net sediment source (Schaefer, 1986; MWCOG, 1983b). However, pollutant removal is impaired if the pilot channel extends all the way through the lower stage to the riser, as sediment and other pollutants are often deposited on the pilot channel and can be subsequently resuspended. Optimally, in a two stage pond design, the stabilized low flow channel should extend to the lip of the lower stage of the pond.

3.7 Pond Slopes

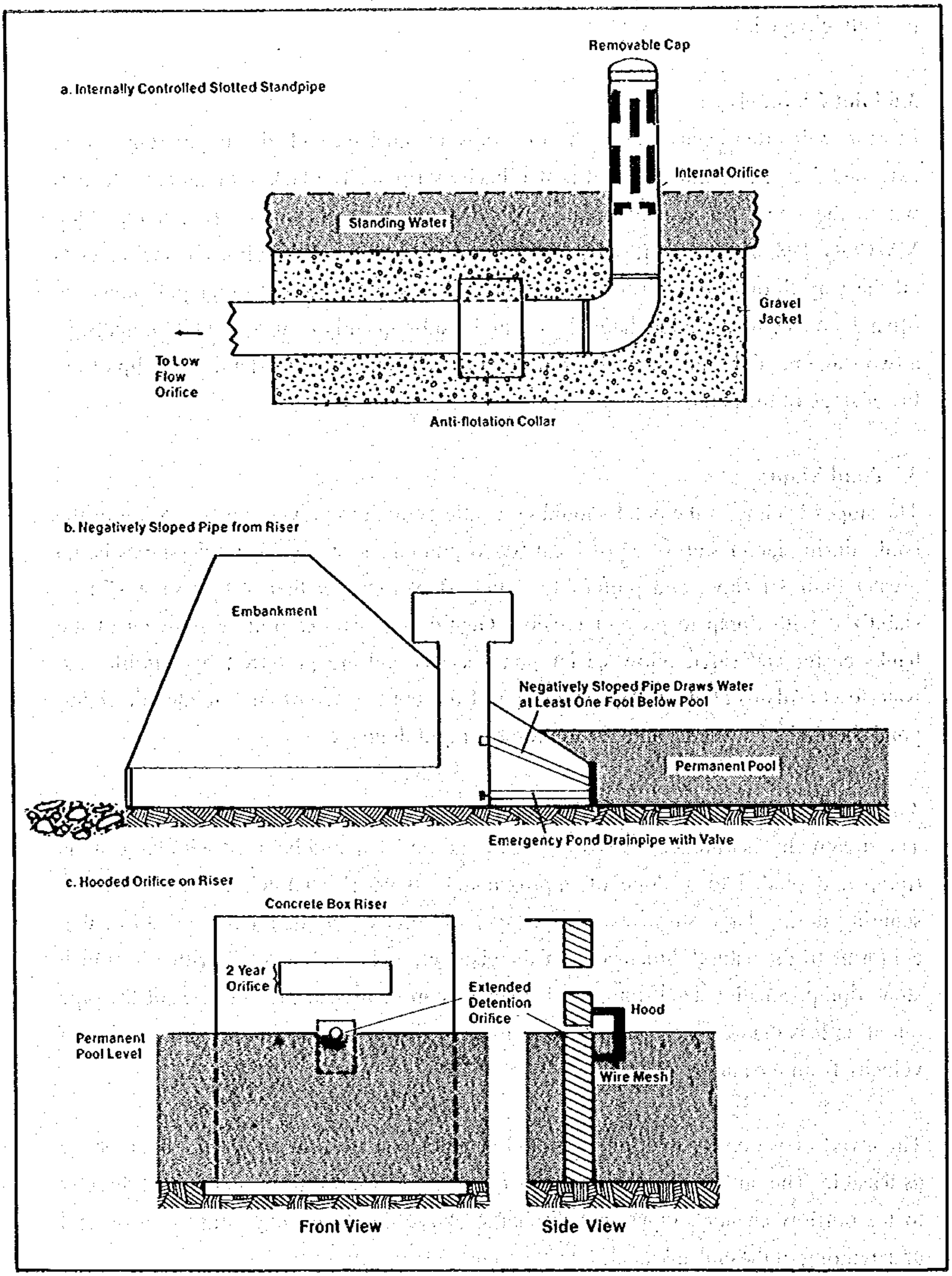
The slopes leading to the pond should be gentle enough to prevent gully erosion of the banks during larger storms. Most local SWM guidelines suggest that side-slopes be no greater than 3:1 (h:v), and preferably flatter. Banks steeper than 2:1 (h:v) should be stabilized with riprap to prevent erosion. Gentle slopes make routine mowing of the banks easier and safer, allow easier pond access and are preferred by wildlife and waterfowl (Adams et al., 1983). The slope of the upper stage of an extended detention pond should be between 2 and 5% to promote rapid drainage.

3.8 Inlet and Outlet Protection

The stream channel immediately below the pond outlet should be lined with large stone riprap and graded to a slope of approximately 0.5% (MNCPPC, 1984) to prevent scouring during large storm events. A layer of filter cloth should be laid down that conforms to the natural dimensions of the channel, and then anchored with 18-30 inch stone riprap. Smaller sized riprap (9-12 inches) can be used if the diameter of the pipe outfall is less than 24 inches. Stilling basins can also be helpful in reducing the runoff velocity from the pond.

The invert elevation for inlet pipes should be as close to the surface of the upper stage as feasible. The outfall pipe should discharge at the bottom of the embankment directly to the outflow channel. Pipes that discharge above this level may cause erosion and undercutting of the embankment.

Figure 3.2 Methods for Extending Detention Time in Wet Ponds



EXAMPLE 3-1: DETERMINING THE FEASIBILITY OF USING A WETLAND TO AUGMENT EXTENDED DETENTION POLLUTANT REMOVAL

Given a 30 acre, 35% impervious townhouse development in a watershed that drains to an extended detention dry pond, calculate the volume of the lower stage of the pond and assess the feasibility of using the lower stage as an artificial wetland to augment pollutant removal:

STEP 1. The volume of the lower stage is equal to:

$[(Rm)(Rv)/12](A)$

$[(0.45)(0.36)/12](30) = 0.4$ acre-feet.

STEP 2. The annual nutrient load to the lower stage is given by:

$L = [(P)(Pj)(Rv)/12](C)(A)(2.72)$

where

P = rainfall depth (inches) over desired time interval

Pj = factor that corrects P for storms that produce no runoff

Rv = runoff coefficient, which expresses the fraction of rainfall which is converted into runoff.

C = flow-weighted mean concentration of the pollutant in urban runoff (mg/l).

A = area (acres).

for N = [(40)(0.9)(.36)/121(2.00)(2.72)(30) = 176 lbs/yr

for P = [(40)(0.9)(.36)/12](0.26)(2.72)(30) = 23 lbs/yr

STEP 3. Assume that the lower stage will be six inches deep to promote optimum wetland conditions. The area of the bottom stage is then: (0.4)/(0.5) = 0.8 acres

The average annual loading per wetland acre is:

(176)/(0.8)= 220 lbs/acre/yr of nitrogen

(23)/(0.8) = 29 lbs/acre/yr of phosphorus

STEP 4. Since the average annual wetland loading is below the recommended limits of 225 lbs/acre and 45 lbs/acre of N and P, respectively, the 0.8 acre artificial wetland should be large enough to provide significant pollutant removal.