1 INTRODUCTION

Urban development is widely considered to have a significant impact on the hydrological response of the catchment in which it occurs (Miller, 1982; Green, 1992). There is, however, some uncertainty as to the magnitude of the affect that catchment urbanization has on peak flood discharges. Most literature reviewed reflects the general belief that discharges are significantly increased for all recurrence intervals but there is a body of thought that contends that the effects are localized and small (Fourie, 1990, Alexander 1993). Chapter 2 of this report contains a qualitative examination of the impacts of urbanization that support the belief that there is a significant increase in flood peaks and discusses the counter argument that the impact is small and localized.

Stream channel morphology is greatly impacted by urbanization and Chapter 2 also contains a review of these effects.

The purpose of this, rather detailed, discussion on the effect that urban development has on catchment hydrology is to provide a background for the discussion on the use of artificially constructed detention storage to help restore the balance.

The need to manage the impact of urbanization on stormwater runoff is well recognized and increasingly stringent controls, for example the U.S. Environmental Protection Agency's stormwater permit program, are being imposed in many countries. This level of control has not yet been reached in South Africa but many local authorities are working in that direction, for example, the Eastern Metropolitan Local Council's Technical Services Director now requires an engineer of record for stormwater as well as structural aspects of developments within its jurisdiction, and the Greater Johannesburg Transitional Local Council offers a rebate on bulk service contributions for stormwater to developments where on-site stormwater management is implemented.

DECLARATION

I declare that this project report is my own, unaided work. It is being submitted for the Degree of Master of Science in Engineering in the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination in any other university.

Christopher John Brooker

on this 25th day of July 1997

ABSTRACT

Detention storage is a well tested, and generally accepted, method of attenuating flood hydrographs, but relatively little data is available from the monitoring of full scale installations. An onstream pond was constructed at Sunninghill Park and details of 15 inflow and outflow hydrographs recorded over a single summer. The analysis of the data shows that the ponds is effective in attenuating the the peaks of all events up to a maximum recorded peak inflow of 1.25 m^3/s. The attenuation ratio, and the discharge coefficients of the pipe outlet both appeared to increase as the peak inflow increased.

Attenuation by detention storage relies on reservoir routing that has to be analysed for design, an alternative algorithm that makes use of the stage / surface area relationship of the pond, and avoids the need to calculate the stage / storage curve, is presented.

It is generally agreed that urban development increases the frequency of runoff, and the peaks of spate flows and small floods from small catchmenrs. The magnitude of its impact on rarer floods from larger catchments is disputed, and more data is required before this question can be resolved. The adverse effect that urban development has on stream channel morphology is also not disputed, and detention storage may help to reduce this problem.

The cost and benefits of detention schemes are highly site specific and cannot be generalized. Peripheral issues made dominate the decision making process, and some of these are discussed with reference to case studies of 4 implemented schemes.

List of Tables

List of Figures

Table of Contents

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DETENTION STORAGE FOR THE CONTROL OF URBAN STORMWATER RUNOFF, WITH SPECIFIC REFERENCE TO THE SUNNINGHILL MONITORED CATCHMENT

Christopher John BROOKER PrEng

Degree of Master of Science in Engineering by advanced coursework and research

A project report submitted to the Faculty of Engineering, University of the Witwatersrand, Johannesburg, in partial fulfilment of the requirements for the degree of Master of Science in Engineering.

Johannesburg, 1997

2.2.6. Counter Theory

Most of this work is, however, based on the analysis of short data sets from relatively small catchments, or on theoretical analyses using computer models, and reflects the impact of urbanization on relatively frequent floods, for example the mean annual flood analysed by Leopold. The peak discharge of rare floods is probably also increased by urban development but this may not always be true.

In an analysis of 114 flood hydrographs on streams in the vicinity of Tulsa, Texas, Beard and Chang (1979) concluded that the effects of urbanization on stormwater runoff were not as great as expected, and that the proportion of impervious surface in a catchment did not significantly affect the rapidity of runoff. They conclude that 40% of the impervious area created by urban development is effective in increasing runoff from small storms.

Fourie (1990), in an analysis of recorded floods in the Apies River basin in Pretoria for the period 1905/06 to 1950/51, found that there was no evidence of a trend showing increase in the peak discharges of rare floods. He concluded that urban development probably only increases the peak discharges for relatively frequent floods in small catchments, and has no material effect in rare floods from large catchments.

Only time will tell whether this conclusion is correct. Long data sets, with a real time correlation between rainfall and stream flow, are required. A system of telemetered rain and stream gauges is being established in the Greater Johannesburg Metropolitan area (Brooker, 1996). Data from this network may go some way towards answering the question in the future.

2.3. Impact of Urbanization on Channel Morphology

There may be some dispute over the effect that urbanization has on flood flows, but there is clear and unequivocal evidence that urban development within a catchment has a significant impact on the subcritical to supercritical will greatly reduce the volume of water stored in the stream, by reducing the flow depth in the main channel and increasing the discharge required before the onset of overflow onto the flood plain.

Improvements in channel efficiency, generally effected to free flood plain land for urban development, therefore tends to increase peak flood discharges from the catchment in two ways. Firstly the average velocity of flow is increased, so reducing the time of concentration of the system, and secondly the volume of storage available in the system is decreased, so reducing the potential for attenuation of the hydrograph by channel routing.

2.2.5. Conclusion

The foregoing is a discussion of some of the mechanisms by which urbanization can increase stormwater runoff, and it has been generally accepted that urban development within a catchment can cause an increase in the peak discharge and volume of spate flow that will result from a particular storm. The discharge or volume for any given frequency or recurrence interval therefore increases, or conversely, the frequency of occurrence of a spate with given characteristics increases. This contention is supported by many authors. Based on flood data collected in the USA Leopold (1968) showed that the mean annual flood from a 2.6 km^2 (1 square mile) catchment may increase up to six fold, Neller (1988) found significant increases in runoff and other hydrological parameters, and Miller (1982) quotes several other sources and concludes "It has been sufficiently established that the magnitude and frequency of flood flows are increased by urbanization".

These increased discharges resulting from urbanization of stream catchments are the direct cause of increased costs to the community. Direct flood damage costs arise because the affected stream breaks its banks more frequently and the escaping flood water causes physical damage to property. There are also indirect costs as discussed later.

agreement between computed and measured water surface levels to be obtained using values of Manning's n in the range 0.1 to 0.175 for the steep channel in contrast to the values in the range 0.035 to 0.05 estimated using standard texts. The discharge was determined by slope area calculations on a 6.4km long reach with a gradient of 0.3% immediately downstream. The measured water depths corresponded to Froude numbers of about 0.4, except immediately downstream of the breach where the value was found to be 0.9. Froude numbers corresponding to the depths computed for the smoother channel were in the range of 1.7 to 2.2.

Trieste concludes that this effect is due to the influence of other forms of energy loss besides boundary friction. These mechanisms would include extreme turbulence, scour, debris transport, expansion and contraction, eddies, hydraulic jumps, and momentum transfer between the high velocity flow in the main channel and the slower moving water over the flood plain. All of these can be observed to be active in flood flows in natural streams, but seldom in smooth, relatively straight and uniform artificial channels.

In smooth artificial channels, where boundary shear provides the dominant mechanism for energy dissipation and Manning's equation is valid in its true form, the regime will be supercritical even for very shallow depths of flow. Design standards for stormwater drainage channels seldom permit slopes flatter than 1% on the grounds that the channels have to be self scouring to reduce maintenance costs. At this slope flow in a concrete channel with a Manning's n of 0.015 can be shown to be super critical for all depths greater than about 1cm (0.011m)

The impact of urbanization in replacing natural channels with artificial smooth structures is therefore not simply confined to the hydraulic effects of increasing the velocity and concentrating the dissipation of energy at downstream end of the channel. It may also have a significant hydrological consequence. Changing the regime from generally greatly reduced by the increase in density of the drainage network. This increases the efficiency of collection, so reducing the lag time (defined as the delay between the time of the centre of mass of the storm hyetograph and the centre of mass of the runoff hydrograph). Graf (1977), in an analysis of the impact of sub-urbanization, as opposed to high density urban development, on the drainage density of the South Branch of Ralston Creek in Iowa, showed that the number of links in the drainage system and the drainage density, (defined as the length of link per unit area of the catchment) increased significantly, and that these parameters greatly influenced the shape and lag time of the storm hydrograph. He concluded that problems associated the hydrologic impacts of suburban developments on the drainage network are equally as serious as those caused by changes in impervious surfaces.

Channel Efficiency

Channel efficiency is often increased. Canalization of streams has frequently been used as a means of reclaiming land from flood plains to allow development of these areas. New channel cross sections are proportioned for the most efficient use of materials, and so closely approximate the most efficient hydraulic section, with the hydraulic roughness kept to a minimum. Steps may be constructed in the channel invert in an attempt to reduce the bed slope and provide sites of local energy dissipation, but the structure of a natural stream with pools, and riffles or chutes is seldom recreated. Average flow velocities are greatly increased.

Trieste (1992), in general observations, found that there are few situations in flood flow in natural channels where supercritical flow occurs over a reach length greater than 8m (25 ft). These observations were confirmed by him in an analysis of the measured water surface profile in a natural channel 1.6 km long with an average gradient of 3.2%, following the failure of the Quail Creek Dike in Utah, USA. Using the DAMBRK model of the National Weather Service he found the best

This initial abstraction is greatly affected by changes in the imperviousness characteristics of an urbanizing catchment. Paved areas are generally constructed to be smooth and commencement of runoff is almost instantaneous with only a shallow depth of precipitation required to wet the surface and fill small depressions. Clearly this depth will depend on the characteristics of the surface and general topography, but typical values given in the literature are about 1mm.

The ground surface in pervious areas is naturally irregular and, where vegetation is present, the surface area that must be wetted before runoff can take place is very much greater, resulting in increased interception. Typical values given in the literature for initial abstraction from pervious and undeveloped areas, for example by Green (1984) in recommendations for parameters to be used with Witwat, fall in the range 2 to 5 mm. Other values from elsewhere in the literature are summarized by Torno (1992).

- Impervious surfaces, pavements and roofs: 1.0 - 2.5 mm
- Lawns 2.5 - 5.0 mm
- Pasture 5.0 mm
- Forest litter 8.0 mm

In addition Horton (1919) and others have shown that the volume of interception storage by wetting of vegetation can be significant, ranging from 0.25 mm to 13 mm. Torno (1992) gives 1 mm as a typical value for grass turf.

The net effect is an increase in runoff depth of about 4 mm per storm event, which translates to 40 m^3 volume per hectare, or 4 000 m^3 per square kilometre of paved area.

2.2.4. Drainage System Efficiency

Drainage Density

The impervious areas created by urban development are frequently connected directly to drainage ways, and the overland flow lengths are

A similar equation is given in the reference for rainfall intensity in the coastal region of South Africa and numerous other equations relating rainfall intensity to storm duration can be found in the literature. Table 2.2, which gives the results of the simultaneous solution of equations 2.1 and 2.2 for a 100m long plain at a slope of 3%, is presented to show the affect of increasing hydraulic roughness on time of concentration and average overland wave celerity.

Table 2.2 : Comparison of times of concentration and effective rainfall intensity for different surface roughness on a 100m long plain at 3% slope

Manning's Number n	Concentration Time [min]	Effective Rainfall Intensity [mm/h]	Average Wave Celerity [m/s]
0.01	2.23	241	0.75
0.013	2.63	236	0.63
0.016	3.01	231	0.55
0.2	16.86	137	0.1
0.3	22.9	117	0.07
0.4	28.63	103	0.06

Although this is a greatly simplified analysis it does clearly illustrate the point that decreasing the hydraulic roughness of the flow path will decrease the time of concentration of the catchment, thereby decreasing the duration of the critical storm, and increasing the peak discharge resulting from the increased intensity of precipitation for a given recurrence interval. With all other factors constant reducing the surface roughness will therefore increase the peak flood discharge for any storm frequency

2.2.3. Initial Abstraction

Runoff does not commence simultaneously with rainfall, initial demands, predominantly surface wetting and depression storage, have to be satisfied before water will start to flow from the surface.

n = Manning's number

L = Overland flow length (m)

i_e = Excess precipitation rate (mm/h)

S = Surface slope (m/m)

C = 6.99 (a constant depending on the units of the other variables)

Table 2.1 gives a summary of the affect of decreasing Manning's number, associated with different ground surfaces, for an overland flow length of 100m at a constant slope of 3% (0.03 m/m), and an excess precipitation rate of 100 mm/h.

Table 2.1 : Time of concentration for a 100 m long plain at a 3% slope, subject to 100 mm/h excess rainfall, for different Manning's Numbers.

Surface Type	Roughness Range [n]	Time [min]
Concrete / Asphalt	0.01 - 0.013	3.2 - 3.7
Bare sand	0.01 - 0.016	3.2 - 4.2
Lawns	0.20 - 0.30	19.1 - 24.4
Dense shrubbery and forest litter	0.4	29

This analysis can be taken further by solving the Equation 2.1 simultaneously with Equation 2.2, derived by Op ten Noort and Stephenson (1982), for the general relationship between storm duration and rainfall intensity.

Inland region

$$I = (7.5 + 0.034 \times MAP) \times R^{[illegible]} / (0.24 + t_d)^{[illegible]} \qquad \text{Equation 2.2}$$

where t_d = storm duration (hr)

MAP = mean annual precipitation (mm)

R = recurrence interval (yr.)

2.2. Flood Peak Discharges

2.2.1. Infiltration

The proportion of impervious to pervious surface within the catchment is increased as urban development proceeds. If it is assumed that there is total runoff from impervious surfaces, and that all impervious areas are directly connected to the drainage network, this change would increase the proportion of runoff to precipitation closely to the proportion of impervious area in the catchment. A large proportion of the impervious surface in developed catchments, particularly in suburban areas is, however, not directly connected to the drainage network so stormwater is given another chance to infiltrate into the soil when it flows from impervious to pervious areas. As the degree of saturation of the soil in the catchment increases the relative effect of the increased proportion of impervious area will be reduced. Once all pervious areas are saturated the impact will be reduced as the only abstraction will be related mainly to the saturated conduction capacity of the soil. Impervious areas may, however, still have a significant impact, particularly if the rainfall intensity is low and the saturated conductivity of the soil is high.

2.2.2. Surface Roughness

The impervious areas are generally hydraulically much smoother than pervious surfaces. Typical values for effective roughness in the form of Manning's n for overland flow are given in the literature, for example by Green (1984) or by the American Society of Civil Engineers (Torno, 1992). The effect of the hydraulic roughness on the time of concentration of a small catchment can be demonstrated by calculating the time of concentration for one dimensional overland flow using the formula derived from Kinematic-wave theory. The relevant equation is:

$$t_c = C \, x \, (n^{0.6} \, x \, L^{0.6}) / (i_e^{0.4} \, x \, S^{0.3}) \qquad \text{Equation 2.1}$$

where t_c = time of concentration (minutes)

2 URBANIZATION

2.1. Introduction

Urban development changes the hydrological characteristics of a catchment in many ways and these effects appear to be reflected in changes to the peak discharges of flood runoff from the catchment. Much of the literature supports the theory that peak flood discharge resulting from a given storm over an urban catchment would be significantly higher than the discharge caused by the same storm over the catchment in its undeveloped state. Miller (1982) stated "It has been sufficiently established that the magnitude and frequency of floods have been increased by urbanization" But perhaps the strongest indication that this concept is generally accepted by practising engineers is the prevalent use of the calculation sheets published by the South African Department of Water Affairs and The National Transport Commission and other published runoff coefficients for the calculation of stormwater runoff using the Rational Method. (Alexander, 1990; Rooseboom, 1986; Stephenson, 1981). These calculation sheets advocate the use of the Rational Method:

$$Q_T \qquad C_T\, i\, A$$

for the computation of stormwater runoff, and all give increasing values of the runoff coefficient "C" for increasing degrees of urban development.

This chapter explores, in a qualitative way, some of the changes that support this belief.

Channel morphology is affected by catchment urbanization and some evidence of the magnitude of the effects is discussed.

of the recorded hydrographs are given in Appendix A. The construction of the detention pond in Sunninghill is described in Appendix B.

Some conclusions to be drawn from the investigation and some recommendations are discussed in Chapter 6.

required in the routing calculation. An algorithm using this procedure is described in Chapter 4.

It is not really possible to generalize on the costs and benefits of stormwater management facilities because each installation is highly site specific. Certain aspects, for example earthworks and land costs, are common to most projects and these are discussed in Chapter 4. In most cases land is the most costly element.

Detention storage is a management technique that is frequently advocated, but less often implemented, for the attenuation of flood hydrographs to reduce peak discharges. One of the reasons that detention ponds are not widely accepted is the land area that they occupy. To counter this resistance ponds can be designed to have multiple uses, dry ponds may be used as playing areas for formal or informal recreation and wet ponds are used for improvements in water quality. The last part of Chapter 4 in this report discusses the design and construction of some stormwater detention ponds that have been constructed in the Gauteng area.

It is seldom that the opportunity arises to monitor the operation of any implemented project in civil engineering, too frequently works are designed and constructed then handed over to the owner or Local Authority for operation or maintenance. Monitoring of stormwater management facilities will possibly become mandatory in the future if Local Authorities are to ensure that their investments are functioning as designed. The construction and monitoring of the Sunninghill facility has therefore provided valuable data for hydraulic and hydrological design of future facilities as well as valuable experience in the setting up of monitoring programmes. The author is therefore very grateful to the Department of Civil Engineering of the University of the Witwatersrand for the opportunity afforded him to gain experience in this field. Chapter 5 describes the operation of the detention pond at Sunninghill and gives the results of the monitoring carried out. Details

Engineering literature and practice reflects this trend Procedures and manuals of the past that emphasized drainage systems are being replaced by new documents that place a much stronger emphasis on management of the stormwater, for example the new ASCE document "Design and Construction of Urban Stormwater Management Systems" (Torno, 1992) largely replaces the previous "Design and Construction of Sanitary and Storm Sewers".

The introduction of the approach into South Africa was largely due to the efforts of Miles with the publication of the National Building Research Institute document R/Bou 619 "Stormwater Drainage In Urban Areas" (Miles ,1979). There does, however, appear to be little actual application of stormwater management options in South Africa. Efficient piped drainage systems, designed to remove stormwater from the site as rapidly as possible, being implemented in most urban developments. Chapter 3 discusses some of the stormwater management options that could be implemented and looks at some of the reasons these have met with resistance from Local Authorities and developers alike. Detention storage is a viable technique for stormwater management and its use is emphasized in that chapter.

Reservoir routing is the numerical technique applied to calculate the outflow hydrograph from a detention pond with a specific stage/storage and stage/discharge relationships. All algorithms known to the author required the prior calculation of the stage/storage curve for the reservoir but this requirement appears to be an artefact of the manual calculation procedures where graphs were plotted to facilitate the calculation procedure. The need to calculate the stage/storage curve can be avoided by making use of the stage/surface area curve measured directly from the contours or digital elevation model of the reservoir basin. Since the numerical calculation process relates more closely to the change of storage with stage, than the actual volume of storage, the use of the stage/surface area curve of the reservoir has the added advantage of slightly reducing the computational effort increased stormwater runoff, as described in Chapter 2. Replacing this storage can attenuate flood hydrographs and reduce peak discharges by increasing the critical storm duration, and by routing the resulting hydrograph. The reservoirs may also reduce the total volume of runoff by providing opportunities for infiltration and evaporation. Numerous schemes are described in the Proceedings of the Conference on Stormwater Detention Facilities (De Groot ed. 1982). Some South African detention storage schemes with which the author has personal experience, or that have been described in the literature are discussed briefly at the end of Chapter 4.

Even small volume detention basins can have a worthwhile benefit. Storage of 40 m^3 per hectare is sufficient to eliminate the effects of the loss of depression and interception storage.

Although a distinction is generally drawn between retention and detention facilities there is, in reality, a continuum.

Detention Storage

Detention ponds work by temporarily detaining some of the floodwater in the pond and releasing it slowly after the hydrograph peak has passed, so reducing the peak by attenuating the hydrograph as well as by increasing the catchment time of concentration. They may also help to improve the quality of the runoff, as the stored water has time to lose some of its sediment load, and can be screened to remove much of the floating debris that it carries. The deliberate use of detention ponds for improvement of water quality requires careful design and is beyond the scope of this report. Design parameters will depend on many factors, such as the nature of the pollutants to be removed from the stormwater, the possibility substances previously bound to the sediments re-entering solution or suspension if the sediment becomes anaerobic, etc.

These ponds are, however, tried and tested stormwater management tools with many advantages.

despite the initial capital outlay. Each application is, however, unique and must be assessed on its own merits as an element in an overall catchment wide plan.

3.3.1. Infiltration Pitches and Swales

With the imminent promulgation of legislation in the USA and Canada requiring the control of stormwater on site much research is being done on the implementation of swales and infiltration pitches for example in Adelaide, South Australia, by Bekele et al (1993) and by Hopkins and Argue (1993) in New Brompton, South Australia.

Swales are shallow trenches that may be partially filled with permeable material such as stone ballast. Their purpose is to encourage infiltration of stormwater into the soil and to provide storage and hydraulically inefficient drainage paths to encourage hydrograph attenuation by routing.

Infiltration pitches are open areas of land on which stormwater is allowed to spread out, again to encourage infiltration and provide temporary storage. Infiltration pitches may be provided with trenches or ditches filled with permeable material to encourage infiltration.

3.3.2. Channel Rehabilitation

Rehabilitation of urban waterways is taking place in many parts of the world, both for stormwater management reasons, and for the benefits that accrue to the community in the form of public open space. These rehabilitation efforts may not always be successful or acceptable to all concerned (Luger & Davies 1993). Brookes (1988) and Purseglove (1988) give guidance on the characteristics of natural streams that should be aimed for in the rehabilitation process.

3.3.3. Detention Storage

The provision of storage reservoirs within the drainage system can replace some of the storage lost as urban development causes

- Increasing of channel and pipe storage by increasing the depth of flow using weirs, orifice plates, vortex type throttles etc.
- The use of porous pavements on roads and parking areas.
- The use of permeable pavements, e.g. grass blocks and hard lawn pavers, where traffic volumes are low and high structural strength roads are not required.
- Reduction in impervious areas by encouraging grassed or gravelled areas and discouraging paving where possible, for example under utilized parking areas, or reduction in road widths where traffic volumes are low.
- Encouragement of disconnected impervious area as opposed to directly connecting impervious areas to the drainage system.

A comprehensive list is given by Stephenson (1981 p4).

These stormwater management practices may be difficult to implement, and do have some real and some perceived disadvantages.

- In general they require space where development is most intense and land values highest.
- They are seen by most Local Authority engineers as potential maintenance problem areas.
- Capital cost of construction is often higher than for a traditional drainage system.
- Infiltration of stormwater into the soil is in conflict with road design philosophy that attempts to keep the structural layers as dry as possible to preserve their strength.

Careful planning, design, and education can, however, overcome most of these problems. For example the dual use of land for stormwater control and recreation may help to spread the high property cost between funding agencies, or the calculation of the total lifetime cost of the management element in association with the downstream drainage elements may prove that the managed system has a lower overall cost the older conduits, currently being repaired, appear to have been decked over at least 70 years ago.

Underground drainage systems have the major advantage that they free land, that would have been occupied by surface structures, for development. They do, however, have inherent problems:

- Their capacity is restricted to the capacity of the inlets.
- They are susceptible to blocking, especially at their inlets.
- Their capacity to handle excess flows is very limited.

Riley et al (1986) blame much of the damage caused by flooding from a very severe storm in Sydney in 1984 on the failure, either by surcharging or blockage, of the underground elements of the drainage system.

3.3. Attenuation

An alternative to increasing the efficiency of the drainage system to cope with the increased runoff from the urbanizing catchment is to institute measures to ameliorate the impact of the urbanization on the catchment. These measures can also be used to reduce the risk to property developed on sites vulnerable to natural flooding. Attenuation of the high flood flows allows a reduction in the capacity of downstream hydraulic structures designed to accommodate a flood peak of given recurrence interval and hence can directly reduce the capital cost of the drainage system.

Amongst the options for flood attenuation are:

- Detention storage at scales varying from small, on site, schemes such as roof top storage, through large tanks under roads, to major reservoirs on the scale of large dams.
- The operation of existing water supply and irrigation dams to reduce downstream flood peaks.
- Construction of swales and infiltration pitches or french drains.

The provision of urban drainage systems to convey stormwater away from the area as rapidly as possible has been the prevalent design philosophy until relatively recently. In 1979 the National Building Research Institute conducted a survey of the stormwater drainage practices of municipalities, local and central government departments, and consulting engineers. In their analysis of the results of this survey Watson and Miles (1982) showed that 62% of respondents designed for

> *"The most rapid removal of the excess runoff from the individual areas drained"*

and only 1 % stated that they made any attempt to ameliorate the effects of urban development on runoff to reduce flood peaks.

This approach has, at least in theory, been superseded by a philosophy of "stormwater management" where an attempt is made to control runoff and reduce peak flows. In parallel with this there is often a conscious division of the system into 2 components, a minor system for convenience only, and a major system to discharge runoff from severe storms safely, and with minimal damage. The provision of an efficient drainage system does, however, remain a valid management option in certain instances. It is often convenient, particularly where open land is scarce and highly priced, to drain stormwater away as efficiently as possible, and then to attempt to reverse the impact of this efficient drainage system by providing storage or some other management element further downstream.

3.2.1. Underground Systems

Underground drainage systems, comprising pipes and culverts, have been the traditional way of dealing with storm water in urban areas almost since the beginning of widespread urban development. Many of the main drains traversing the City of London and its boroughs have been decked over for several hundred years. In Johannesburg some of

Although the document does not specifically include the limitation or attenuation of flood peaks, or make a specific objective of the implementation of stormwater detention facilities, it does include the statement that:

> *"Post development rates of discharge should, if possible, be limited to pre-development values for a range of recurrence intervals"*

It also makes provision for the preparation of master drainage plans that, inter alia, identify and reserve sites for the provision of flood and water quality control structures including wetlands.

Control of stormwater in urban areas can be achieved by a range of techniques, from providing efficient systems to drain the water away as quickly as possible, to total on-site management where development does not increase the peak discharge or volume of runoff above the pre-development levels. Control of the peak discharges and volumes is most often achieved by providing some form of attenuation system.

These publications, the later discussion on Sunninghill in Chapter 5, and case studies in Chapter 4, show that the concept of stormwater management has both popular and institutional approval here, and schemes can be successfully implemented in South Africa with public approval if correctly approached.

3.2. Efficient Drainage Systems

Construction of formal drains began early in the life of the City of Johannesburg. The reports of the Town Engineer show that by 1905 an extensive network of underground drains had been constructed in the South West District and parts of Newtown. By 1910, 47.8 km of drains had been constructed in a total township area of 8 325 ha, and by 1920 the length of drain had increased to 95.2 km in a total township area of 8 500 ha. (Whitlow and Brooker, 1990a) control measures in Japan, and, closer to home, South African Committee of Urban Transport Authorities (CUTA) published UTG4 Guidelines for Urban Stormwater Management in draft in 1988. (Hay et al 1988). The proceedings of the 1993 Conference on Storm Drainage, held in Niagara, USA, include papers from Canada, France, Japan, Nigeria, South Korea, Taiwan, and the United Kingdom.

Johannesburg Urban Stormwater Management Policy

In January 1993 the Management Committee of the Johannesburg City Council approved a proposal setting out the structure of an Urban Stormwater Management Policy for the city. The objectives this document sets out as policy are:

- The provision of an appropriate stormwater drainage system for the management of "normal" stormwater runoffs for the protection of property from damage.
- The prevention of injury or loss of life, and the reduction of damage to property by the run-off from abnormal storms.
- The prevention of land and watercourse erosion,
- The protection of water resources from pollution and the implementation of an integrated approach to water quality management.
- The preservation of natural water courses and their ecosystems.
- The prevention of ill health related to water borne diseases.
- The implementation of a plan for the total management of dams within the urban environment.
- The implementation of measures to guard against the negative aspects of rapid urbanization, where water courses are affected.

In order to do this it states that the Council must be proactive in water course management and plan and budget to achieve these objectives, and should strive for a regional water quality and stormwater management approach.

3 STORMWATER MANAGEMENT OPTIONS

3.1. Introduction

Since the late 1970's strong emphasis has been placed on the practical implementation of stormwater management measures, and articles referring to stormwater management have appeared in the popular scientific and general professional literature. For example Zukovs et al (1982) describe some aspects of township layout to accommodate streams, Fowler and Wolfe (1982) discuss some practical technologies for urban runoff control, and Miles (1984) looks at aspects of stormwater drainage for lower income group developments. Taming the Flood (Purseglove, 1989) is truly a popular book that looks at the practice and malpractice in the history of stormwater, flood, and wetland management in Britain since medieval times. Brooker (1991) identifies stormwater management facilities as an opportunity for co-operation between Civil Engineers and Landscape Architects, and Rabinovitch and Leitman (1996) emphasize that stormwater and flood management practices played a significant role in the successful redevelopment of the city of Curitiba in Brazil.

In the United States of America practical implementation is common, and frequently required by the legislation controlling development of land. Recent practice manuals (Torno, 1992) cover design and construction aspects in detail, and as early as 1982, sufficient practical experience had been obtained to justify a successful, dedicated conference on the subject of Stormwater Detention Facilities (DeGroot ed, 1982)

The literature emanating from many different countries shown that the concept is well accepted throughout the world, for example Geiger (1990) describes work done in the then Federal Republic of Germany, Loong (1990) discusses the use of ponds for stormwater retention in Kuala Lumpur, Malaysia and criticizes the implementation of improved drain capacity measures in that city. Seguchi (1990) discusses flood erosion in urban rivers are described by Douglas (1985), and in the author's experience erosion of the banks of the incised channel of the Braamfontein Spruit in Craighall has exposed artefacts showing that sediments 2m below the surface are only about 50 years old.

The gradual increase in the channel dimensions, as the cross section adjusts to accommodate the increased frequency and peaks of discharges, and the increasing inadequacy of the hydraulic structures are less direct costs, that take a longer time to be felt, and are often ignored by the authorities until disaster forces action.

2.4. Impact of Urbanization on Water Quality

A detailed discussion of the impact of urban development is beyond the scope of this report. Quantity and quality of runoff are, however, closely tied and detention storage is frequently used in the dual role of reducing peak discharge and reducing the concentrations of pollutants. In the overview and summary opening the conference on the Design of Urban Runoff Quality Controls, Roesner et al (1988), commenting on the devices available for the control of urban runoff quality state:

> *"Among all of these devices the most promising and best understood are detention and extended detention basins and ponds"*

Most efficient use of resources will therefore be achieved by considering both the quality and flood peak control possibilities of detention ponds.

proposed a qualitative relationship to define channel morphology of alluvial streams, of the form:

$$Q_s \, x \, d \;\alpha\; Q_w \, x \, S \qquad \text{Equation 2.3}$$

where Q_s = Sediment discharge

d = Particle size

Q_w = Clear water discharge

S = Bed slope

α indicates proportionality

all units consistent

Early in the urbanization process the capacity of the catchment to contribute sediment is high, because construction strips large areas of vegetation, but this capacity is reduced as more impervious surfaces are created and runoff becomes sediment deficient. The increased clear water discharge resulting from catchment urbanization will tend to cause the stream to decrease its bed slope by degradation or to increase the sediment discharge, resulting in a channel of increasing depth and cross sectional area.

The impact on channel morphology of the increased runoff of stormwater as a result of urban development begins very early in the life of a city, Whitlow and Brooker (1996b), in a review of the annual reports of the Johannesburg City Engineer, found that canalization had begun in the town of Johannesburg within 10 years of the establishment of the City. The 1911 report referred to the construction of a 6.1m wide by 3.05m deep canal to curtail the growth of the donga of the spruit in Bezuidenhout Valley that was described as being up to 6.7m deep and 30.5m wide. The amount voted for this construction was R450 000 in 1994 terms.

Stratigraphy in sediments in urban streams can show channels incised through very recent deposits. Patterns of recent sedimentation and channel morphology of rivers draining that catchment. This effect has been reported by several authors. Robinson (1976) and Whipple et al (1960) found that cross sectional areas of stream channels with urban catchments were typically twice that of streams with similar sized but rural catchments. Whitlow and Gregory (1989) found an average increase in the width of a stream in Harare to have been 74% in a 12 year period.

Neller (1988) compared the erosion patterns of 2 streams draining similar sized small catchments in New South Wales, 5 years after the completion of housing construction in one of the catchments. Despite the relatively low density of development, 13.6% impervious area in the urban catchment, the bank erosion rate of the channel remained 3.6 times higher and knickpoint retreat 2.4 times greater. He concluded that the urban channel was not inherently unstable, but that the increased erosion was the result of changes in catchment hydrology. Significant increases in total runoff (7.8 times), number of runoff events (3.7 times), and peak storm runoff (3.5 times), and a significant decrease in the average time to produce runoff (63%) were recorded.

In Johannesburg the author has observed that the Braamfontein Spruit is increasing the cross sectional area of it channel by increasing the width, and has reduced the slope as far as possible in the short term by eroding to the point that the channel bed level is now largely controlled by a series of rock outcrops. These effects have not b[illegible]ntified, but the increasing need for bank stabilization to protect services and in response to complaints by owners of riparian property is clear evidence that this is the case (Whitlow and Brooker, 1996a). The General Plan of the Township of Parkhurst, compiled in 1904, clearly shows the boundaries of a vlei where there is now a deeply incised channel (Brooker, 1991).

The mechanism of the process is clear. Stream channel geometry is formed by all discharges but the basic shape, width, depth, and meander pattern result from the bank full discharge. In 1947 Lane channel morphology of rivers draining that catchment. This effect has been reported by several authors. Robinson (1976) and Whipple et al (1960) found that cross sectional areas of stream channels with urban catchments were typically twice that of streams with similar sized but rural catchments. Whitlow and Gregory (1989) found an average increase in the width of a stream in Harare to have been 74% in a 12 year period.

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The mechanism of the process is clear. Stream channel geometry is formed by all discharges but the basic shape, width, depth, and meander pattern result from the bank full discharge. In 1947 Lane stream's sediment load caused by settling within the pond will, to some extent, counter this. As is the case with most dams, some increased erosion immediately downstream of the outlet can be expected until the sediment carrying capacity of the flow is satisfied.

Water Quality

In catchments undergoing urban development, particularly where gradients are steep, the amount of sediment picked up by the overland flow is high. Detention of the spate flows will reduce the silt load carried downstream.

The provision of on-stream storage therefore reduces the maintenance cost of the drainage system but, unless the storage provided is large, has little effect on the capital cost of the hydraulic structures.

Disadvantages

Storage ponds clearly have many disadvantages and constraints that may mitigate against their implementation. Some of these disadvantages and constraints are described in detail later in this report and some will become apparent only when a specific project is contemplated. Storage ponds are not a panacea for all stormwater management problems and, as stated previously, each application should be approached on its own merits.

silting and the accumulation of debris, particularly at the pond outlet, and have to be cleaned out regularly, and especially immediately after they have been in operation. Careful design attention needs to be paid to accessibility, particularly as the bottom of the pond may be wet and muddy when cleaning is most important.

Wet ponds need careful control to maintain water quality to prevent the development of smells. Mosquitoes and other pests need to be controlled.

Maintenance activities will depend on the nature of the storage provided, some issues that require consideration are:

Wet Ponds

- maintenance of water level
- control of water quality
- control of mosquitoes
- control of sediment

Infiltration ponds

- blocking of surface by fine sediment
- pollution of groundwater

Porous media reservoirs

- clogging of voids
- blocking of outlets

Dry ponds

- accumulation of sediment and debris
- clogging of outlets
- maintenance of other facilities where dual use is envisaged
- control of damp areas

Roof top and other parking areas

- clogging of outlets
- waterproofing
- deterioration of structural layers

There is also a probability of structural failure of the pond and the consequences of this occurrence should also be considered.

4.5. Design Considerations

There are several other aspects that need to be considered in the design of a stormwater detention system. It is these peripheral issues that appear to have contributed most to the resistance to detention systems for the control of stormwater found within many local authorities in South Africa.

4.5.1. Structural Integrity

By their nature detention ponds are often constructed in flood plains where founding conditions are difficult, and embankments may be subject to overtopping more frequently than is the case with ordinary dams. Detention ponds are most often in, or upstream of urban areas, so the consequences of the wave following the collapse of an embankment superimposed on the already high flood stage in the stream can be catastrophic. Careful attention therefore needs to be given to the design of the different elements to ensure their structural integrity. Steep earth embankments need particular care as they may be subject to rapid draw down conditions as the detention pond empties.

4.5.2. Environmental Considerations

Although detention systems are environmentally beneficial on balance, they will have some adverse environmental effects. Wetlands often provide suitable, and economically attractive, sites for the construction of ponds, and all aspects of the environmental impact of the projects need to be considered.

4.5.3. Management and Maintenance

As with all engineering structures, and stormwater structures in particular, detention ponds require maintenance. They are subject to or

$$Tc \quad -b / 2a$$

This value for the critical storm duration can be tested in the model, and used to calculate new coefficients for the parabola if necessary. The method has been found to converge rapidly.

The approach is equally valid for complex, catchment wide stormwater management planning where there may be multiple ponds in series or parallel, and combinations of other measures. It must, however, be stressed that the curve for a complex system is not necessarily a parabola with a single maximum, local maxima may exist and the model should be tested at remote values to check for this condition.

It is also important to determine the critical storm duration for all ponds in the system. Typically the critical duration will increase down the catchment.

A sensitivity analysis must be made to determine the affect of the assumed model parameters, such as imperviousness ratio, surface roughness, overland flow length and slope or entry time, and depression storage on the critical storm duration.

4.4. Probability of Failure

The designer of a stormwater detention system must be aware that there is always a probability that the design flood will be exceeded, and the consequences of this must be taken into account. The existence of a detention pond will distort the discharge / frequency curve for the water course, flood peaks up to the design recurrence interval will be reduced, but the peak discharges of larger floods may be unchanged. The presence of the pond may therefore create a false sense of security in the floodplain downstream. The major drainage route downstream of the detention structure must be identified and the implications of failure of the pond considered.

hydrograph with a reasonably smooth curve, and δh was arbitrarily chosen as 0.01m. Analysis results were checked for volume changes by integration of the discharge / time curves of the inflow and outflow hydrographs, and for mathematical correctness by ensuring that the peak of the outflow hydrograph fell on the falling limb of the inflow hydrograph. All errors were found to be within acceptable engineering limits given the other assumptions such as the linearity of the stage / surface area curve, and the contour spacing at which this curve was measured.

4.3. Effect on the Hydrograph

Reservoir routing has two effects on the hydrograph, the peak is attenuated and the peak is lagged, there is no effect on the hydrograph volume. This makes the design of detention storage ponds a process of trial and error, because the critical storm duration for the combination of available storage and outlet characteristics cannot be calculated, and is frequently very much longer than the critical storm duration for the catchment without storage.

This calculation process can be shortened by predicting the critical duration after three trials have been made. The procedure works best if the critical duration is guessed and the inflow and outflow hydrographs are computed for three storm durations, one close to the guessed critical duration, and the others some time shorter and longer than the guessed duration. The coefficients a, b, and c of a parabola of the form :

$$Qp = aT^2 + bT + c \qquad \text{Equation 4.10}$$

T = storm duration

can be calculated and the derivative set equal to zero to give a better approximation to the critical storm duration, Tc.

$$0 = 2aT + b$$

4 DETENTION STORAGE

4.1. Detention Routing

Detention storage simply provides storage volume in the continuity equation

$$-V_i + V_o + \Delta S = 0 \qquad \text{Equation 4.1}$$

V_i	=	Volume of inflow over time step
V_o	=	Volume of outflow over time step
ΔS	=	Change in storage in the reservoir

In most flood routing cases the inflow is known, and there is some known relationship between the volume of storage and the volume of outflow for the time step. The equation can be solved for the outflow and storage hydrographs using a reservoir routing algorithm such as the modified Puls method described by Wilson (1975) pp 156 - 160, or that developed by the author and described below.

4.2. Reservoir Routing Algorithm

4.2.1. Description

The author's program differed from all others tested in one major respect. All of the commercial programs tested required the stage storage curve for the pond to be input as data. Calculation of this relationship is, however, unnecessary and appears to be an artefact of the manual calculation techniques such as the Modified Puls Method used in the past. The author's algorithm used the stage / surface area curve that can be measured directly off the mapping of the reservoir basin either manually or by almost any CAD / DEM package. Volume calculations using DEM modelling packages have, until recently, proved to be notoriously problematic, and remain time consuming.

The program uses a Newton-Raphson numerical scheme to compute a very good solution to the continuity equation, with the only taken up rapidly as the flood rises, and on-stream ponds make less efficient use of their available capacity in attenuating peak discharges of larger floods, but they do have a significant effect on the peaks of smaller events.

Ponds may also be constructed as compound on and off stream structures, with inlets designed to allow the bypass of both the low flows and the rare flood flows. Hydraulic design of inlet and outlet structures can be difficult and require physical modelling to ensure efficient and safe operation at the full range of design discharges and pond stages.

3.3.4. Applications

The selection of a flood attenuation technique would therefore depend on the design objective. Small scale ponds and infiltration pitches are most often used in on-site applications to control run-off from relatively small areas, and would have to be implemented as part of a catchment wide policy to have any significant effect on stream flow. On-stream ponds are beneficial where spate and small flood flows need to be controlled, for example to alleviate the effects of urbanization on stream bank stability. Off-stream ponds are particularly successful in truncating the flood hydrograph peak, to prevent the downstream discharge from exceeding the capacity of a structure or channel, for example the Bawden Park detention pond discussed in Chapter 4, or the Mount Pleasant dam on the Klip River upstream of Ladysmith in Kwazulu Natal.

Equilibrium Channel Size

It has been shown earlier that the literature supports the contention that equilibrium channel size is determined by all flood discharges, and that a stream typically creates an incised channel to accommodate the 2 year recurrence interval flood. Attenuation of spate and small flood flows by on-stream storage can, therefore, have a significant benefit in reducing the size of the channel downstream. The reduction of the

Acquisition of the land occupied by detention ponds is often the highest single cost component and, for economic efficiency, the pond should be designed to have multiple uses wherever possible. The ponds can be designed to be dry most of the time providing park land for active or passive recreational use, or they can remain wet, with pools and reeds, to encourage birds and other wildlife. Although these wet areas may facilitate the infiltration of stormwater into the soil, they perform a different function from retention ponds, as their purpose is primarily aesthetic, and their benefit mainly environmental. for example the ponds in the Sunward Park detention ponds discussed in Chapter 4.

Retention Storage

Retention ponds control runoff by storing stormwater until it percolates into the soil or evaporates. They are often combined with detention ponds in a "blue green" system.

On and Off Stream

Storage can be provide on-stream, where it intercepts all flow and begins to fill from the start of the flood or off-stream where it is bypassed by flows up to a certain threshold. The description is based on the functioning of the detention structure, and not necessarily on its geometry.

An off-stream pond can be created by constructing a barrier across a stream, and providing an opening to pass base and spate flows unhindered, so that the storage only becomes effective when the stream stage rises. Alternatively side overflow weirs or siphons can be provided to divert some, or all, of the flow above a certain discharge into the storage pond. Off-stream storage has no effect on peaks below a certain threshold, and therefore is of no benefit for spate or smaller flood flows.

On-stream storage lies in the path of all flow in the stream, and is most frequently combined with a wet pond, as it will begin to fill whenever there is flow in the stream. The available storage capacity may be

Qo_i = outflow rate at start of time step

Qo_f = outflow rate at end of time step

h_i = water level in pond at start of time step

h_f = water level in pond at end of time step

$Qo(h_i)$ is known from the previous time step or the initial conditions

$Qo(h_f)$ in unknown

Storage ΔS is a function of the surface area of the pond

$\Delta S = \{ A(h_i) + A(h_f) \} \times (h_f - h_i) / 2$ Equation 4.6

$A(h_i)$ = pond surface area for water level at start of time step

$A(h_f)$ = pond surface area for water level at end of time step

4.2.3. Newton-Raphson Iterative Scheme

A Newton-Raphson iterative scheme can be set up to compute h_f, the water level at the end of the time step, because all variables are known, either as initial conditions at the start of the time step, or as explicit functions of h_f.

$- Vi + Vo + \Delta S = f(h_f)$ Equation 4.2

$h_{fi} = h_{fo} - f(h_{fo}) / f'(h_{fo})$ Equation 4.7

A forward difference approximation is used to estimate the derivative $f'(h_f)$, with a sufficiently small value for δh

$f'(h_f) = \{f(h_f + \delta h) - f(h_f)\} / \delta h$ Equation 4.8

$h_{fi} = h_{fo} - \delta h \times f(h_{fo}) / \{f(h_{fo} + \delta h) - f(h_f)\}$ Equation 4.9

aesthetic pleasure from a well designed pond are intangible and cannot be adequately accounted in the cost benefit analysis.

Parker et al (1987) define a topology of flood losses, equally applicable to flood protection benefits, as direct or indirect, and tangible or intangible, as set out in table 4.1, and give detailed techniques for assessing benefits under different conditions.

Table 4.1: A topology of flood losses (Parker et al 1987)

		Measurement	
		Tangible	Intangible
Form of Loss	Direct	Damage to buildings and contents	Loss of an archaeological site
	Indirect	Loss of industrial production	Inconvenience of post-flood recovery

Assessment of potential flood losses, necessary to determine the benefits associated with a flood alleviation scheme, is difficult and uncertain, particularly where indirect costs are concerned. Cost data from previous floods needs to be treated with care to ensure that collected data is reliable and not inconsistent, e.g. based on replacement or depreciated costs.

Potential damage costs should not be determined for a single event, but must be assessed for the design life of the project and for a full range of probable floods. A financial analysis, using a technique such as that proposed by Annandale (1988), to determine the present value of all potential flood damage during the life of the project is required. The present value of probable damage costs can then be compared with the present value of the alleviation scheme to ascertain the cost effectiveness of the alleviation measures.

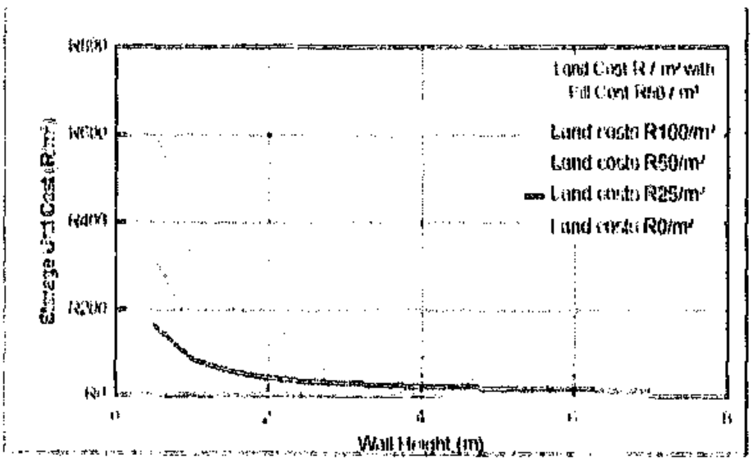
4.6.3. Environmental Impact Assessment

Each project therefore needs to be individually analysed objectively and subjectively. A thorough environmental impact analysis, that emphasizes the sociological and environmental issues as strongly as it were varied from zero to R100 /m², roughly corresponding to the range between the cost of land in a public park to the cost of land in an upper socio-economic level residential township. Land costs in commercial areas could be considerably higher.

Fill costs were taken in the range from R20 /m³ to R60 /m³, roughly corresponding to the 1997 rates for cut to fill in the immediate vicinity of the wall and fill from a commercial source.

Figure 4.4 shows the sensitivity of the cost of storage to the cost of the land and Figure 4.5 shows the sensitivity to the cost of the earthworks. These figures show clearly that, for an impoundment behind an earth embankment, the total cost is very much more sensitive to the land cost of the site than to the construction cost of the embankment.

Figure 4.4 : Relationship between storage cost per unit volume and wall height for varying land cost in a valley with a longitudinal slope of 2% and side slopes of 6%



chnology for the model village. In Bawden Park there was almost no difference in the estimated costs of the schemes, but the peripheral considerations predominated.

Groundwater recharge was a primary consideration in Westfleur and to a lesser degree in Graceland. No infiltration was considered in either Sunward Park or Bawden Park.

Water quality improvement is expected in Westfleur, Sunward Park, and Graceland. The Bawden Park scheme is off channel and bypassed by floods up to about a 10 year recurrence interval, and by the first flush of larger events. This scheme therefore does not offer any improvement to the quality of stormwater runoff, although an accumulation of silt and floating debris does occur in the pond, so there is a small and infrequent benefit.

Storm rainfall characteristics are different, Westfleur is in the Western Cape where long duration, relatively low intensity, high volume, storms are anticipated. The other three are in Gauteng where flood causing storms are typically short, intense, convection events of relatively low volume.

4.8.2. Sunward Park

In late 1982 and early 1983, when the civil engineering design was done for the development of Sunward Park and extensions in Boksburg, it was found that the township area was bisected by a surface drainage line running from north to south. This drainage line was typical of the East Rand topography, appearing on the available contour mapping as a shallow depression, and the town planning, completed before 1973, had not made adequate provision for the accommodation of the floodwater that would concentrate there. The total catchment area draining to this depression was measured to be 61.8 ha at the northern boundary of the township and 102.8 ha where it exited through the southern boundary of the township. The township

4.8. Case Studies

4.8.1. Introduction

Four case studies of detention storage schemes implemented recently in South Africa are presented here. Each illustrates different criteria that were applied in the decision on whether to implement the schemes. Three, namely Sunward Park, Bawden Park, and Graceland were designed by the writer. The fourth, the Westfleur and Atlantis Industria stormwater management system, received the SAICE Western Cape Regional Award in 1987 (Civ. Eng. SA 1988).

The marked differences and similarities between the schemes are summarized briefly here before the schemes are described in some detail to illustrate these points.

The Graceland and Westfleur schemes were planned and implemented as part of new developments so the requirements of the schemes could be considered as the land use was determined. The Sunward Park scheme, although implemented as part of the initial construction of township services, was imposed on the development by short sighted town planning that failed to provide an adequate surface drainage route through the proclaimed stands. The Bawden Park scheme was implemented in a long established urban area to resolve a pressing flood problem that had arisen as a consequence of road development.

The catchment areas affected by the schemes vary greatly. The Westfleur scheme covers some 20 km², the Bawden Park catchment is 6 km² in extent, the catchment area to the lowest of the Sunward Park dams is 1 km² and the Graceland catchment is only a few hectares.

The primary motivation for the Sunward Park and Westfleur schemes was cost saving. In both cases the managed stormwater scheme cost about 50% of the alternative directly drained solution. In Graceland some cost saving was achieved, but the primary reason for accepting the detention scheme was appropriate, environmentally sensitive.

analyses the financial costs and benefits, should be considered. At very least a scoping exercise, as set out in the Department of the Environment Guidelines (Dept. Envt. Affairs, 1992) should be completed.

4.7. Increased Flooding

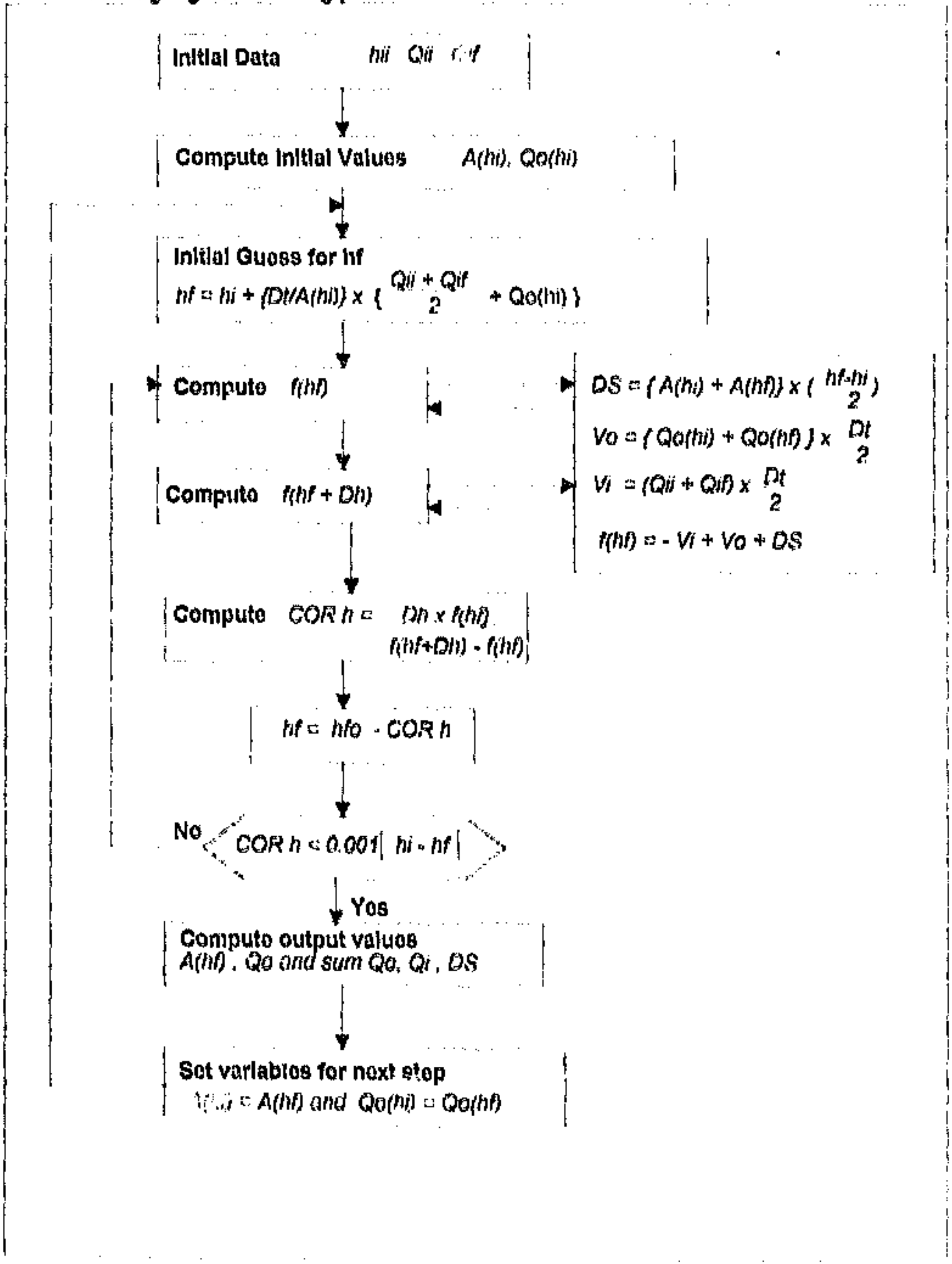
A major objection that is often raised against the provision to detention ponds, as a catchment wide flood management option, is the possibility of increased flooding, if the detention storage alters the timing of the hydrograph peaks so that they coincide. Boyd (1993) reviewed the modelling of eighteen regional and on site detention storage projects, and concluded that it was unlikely that detention storage would have an adverse effect on downstream flood peaks, but that the engineer had an obligation to ensure that no adverse effects did occur, and that the total catchment should be analysed.

A procedure, as discussed earlier in this report, should be followed to determine the critical storm durations, and ensure that critical combinations of conditions are identified and tested.

The validity of the assumption underlying most flood discharge calculations, that the worst condition results from a storm covering the entire catchment, with a duration equal to the time of concentration, should be critically reviewed for each case. The probability of storms covering smaller parts of the catchment, and occurring in some critical sequence, must be assessed.

The possibility of upstream flooding, particularly by backwater effects in gently sloping valleys must be considered. Properties adjacent to the pond are also susceptible to flooding and the effect of a flood exceeding the design event must be considered.

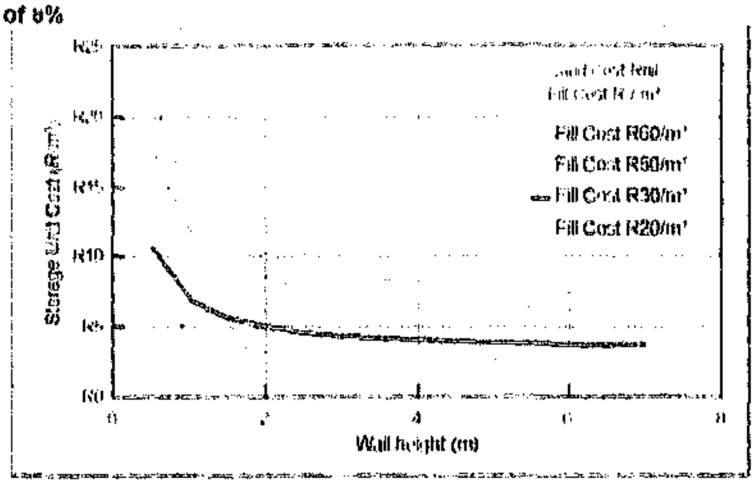
Figure 4.1 : Flow chart for the computation of the Newton-Raphson reservoir routing algorithm using pond surface area



The scheme was not specifically tested for sensitivity to either δh or Δt

A practical value was found for Δt in the range 2 to 5 min to give an

Figure 4.5 : Relationship between storage cost per unit volume and wall height for various fill costs in a valley with a longitudinal slope of 2% and side slopes of 5%



All of these figures show that the unit cost of storage diminishes rapidly with increasing wall height, until the wall reaches a height of about two metres. These analyses do not take into account the establishment costs which would be proportionally higher for smaller earthworks projects.

Where the storage volume is created by excavation the cost will depend on the nature of the insitu material, and be directly proportional to the volume of storage required. Excavation in hard material costs more than excavation in soft soil but not necessarily more than excavation in very soft material where specialist techniques may be required.

Other Costs

Amongst the other items that will affect the final implementation cost are:

- Embankment and structural founding conditions
- Inlet structures
- Outlet structures including emergency overflows
- Channel bank protection within the pond

When the HP86 computer became obsolete the program was not rewritten to run on IBM compatible machines, the availability of suitable commercial programs, namely Hydrosim and Stormwater, for these machines made the in-house development of software unnecessary. The reservoir routing routines used by the commercial programs were tested against that used by the author's program and found to yield the same results.

4.2.2. Numerical Scheme for Reservoir Routing Algorithm

If it is assumed that all variables change linearly over the time step, and a sufficiently short time step is chosen to ensure that the error inherent in this assumptions is small, then the routing equation can be solved as follows:

Continuity

$-Vi + Vo + \Delta S = 0$ Equation 4.3

Vi = volume of inflow over time step

Vo = volume of outflow over time step

ΔS = Change in storage

Inflow

Vi from inflow hydrograph

$Vi = (Qi_i + Qi_f) \times \Delta t / 2$ Equation 4.4

Qi_i = inflow rate at start of time step

Qi_f = inflow rate at end of time step

Δt = time step

Outflow

$Vo = (Qo_i + Qo_f) \times \Delta t / 2$

Vo is a function of the outlet characteristics of the system

$Vo = \{Qo(h_i) + Qo(h_f)\} \times \Delta t / 2$ Equation 4.5

approximations being the stage / discharge relationship for unsubmerged pipes, where the inlet shape is taken as rectangular with a given aspect ratio, and the assumption that compound stepped weirs will act as individual rectangular weirs with no significant interaction.

The numerical scheme, as set out in Figure 4.1, is a procedure to find the root of the equation:

$$-Vi + Vo + \Delta S = f(h_i) \qquad \text{Equation 4.2}$$

$f(h_i)$ is a function of water level in reservoir

using a forward difference numerical approximation for the derivative $f'(h_i)$.

This algorithm was used as the basis of the flood routing computer program written by the author to run on an HP86 computer, as presented in the code given in Appendix C.

The program was capable of computing routing through reservoirs with multiple pipe or box culvert outlets and compound stepped weirs with up to three levels plus spill over the wall crest. Broad, sharp or ogee weir crests could be considered by changing the discharge coefficient, and each crest level in compound weirs could be given different coefficients. Trapezoidal and Vee notch weirs could not be analysed using the program as presented, but could be incorporated by simply adding appropriated overflow functions to the code. The program was restricted in the way in which multiple pipe or culvert outlets were handled in that all conduits had to have the same invert level, height, cross sectional area, and discharge coefficients. To approximate the unsubmerged discharge characteristics of circular, or other irregularly shaped, conduits the program calculated the mean breadth of each structure as the ratio of area to height, an approach that avoided the computational effort required to determine critical depth.

- Downstream channel protection measures and energy dissipation structures
- Embankment protection
- Peripheral structures e.g. fencing, signage etc.
- Subsoil drainage
- Monitoring equipment
- Aesthetic improvements, landscaping, grassing etc.

Increased maintenance costs are often cited by Local Authority engineers as their most pressing objection to the construction of detention ponds. Whilst it is true that ponds require maintenance their presence localizes many of the activities that would have to be carried out anyway, for example:

- the debris and silt trapped in the pond after a storm would have to be cleaned out of the system downstream.
- reduced downstream peak discharges can reduce the long term maintenance requirements of the downstream channel.

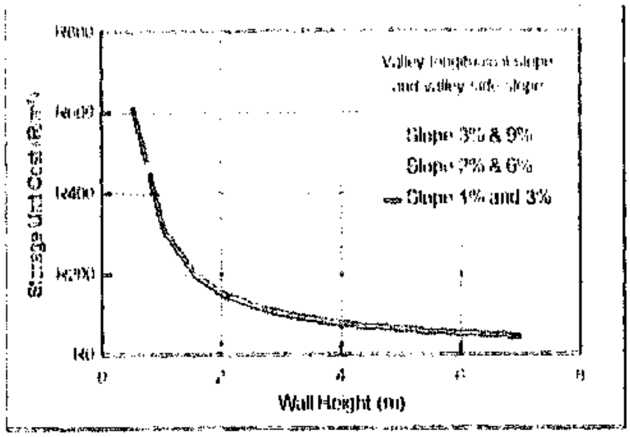
The overall balance of maintenance costs and benefits therefore needs to be considered, and not only the immediately apparent additional burden placed by the pond itself.

4.6.2. Benefits

Benefits too are highly site specific. Ponds may be constructed to protect the high value contents of a particular building, for example the Wierda Valley pond in Sandton, or to obviate the need to construct a costly culvert under a busy road, as was the case with both the Bawden Park detention pond in Booysens, which greatly distorts the financial benefits.

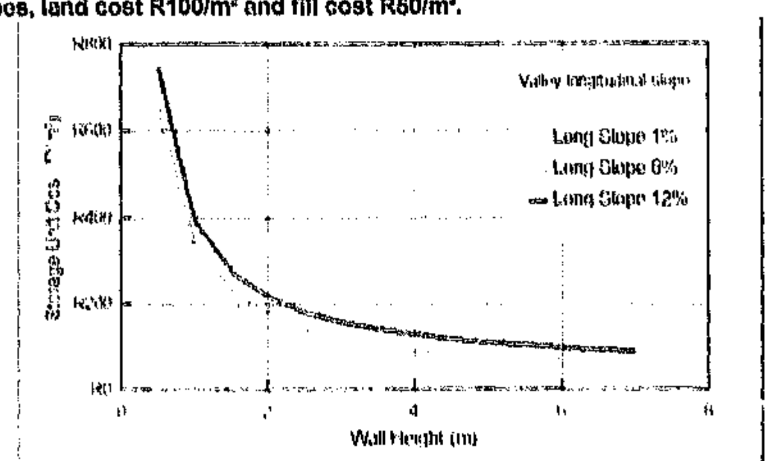
Other benefits such as the recreational opportunities offered by multiple use ponds, nature conservation areas, improved water quality, energy dissipation reducing the erosive power of the flowing water, and the

Figure 4.2 : Unit cost of storage for different valley topography, land cost R100/m² and fill cost R50/m³.



Further analysis showed that the unit cost of storage is totally independent on valley side slope, and Figure 4.3 shows that cost variation with longitudinal slope is small, even when the range of slopes is exaggerated.

Figure 4.3 : Unit cost of storage for exaggeratedly different valley longitudinal slopes, land cost R100/m² and fill cost R50/m³.



Land and Fill Costs

The same analysis showed that the cost per unit volume of storage was sensitive to both the cost of the land that would be inundated by the pond and the cost of the fill used to build the embankment. Land costs

For the purpose of this report the land occupied by the wall and the verge required for freeboard around the pond can be ignored. The relationship between storage volume and surface area for a triangular valley is then:

$A = 3 x V_s / h$ Equation 4.13

A	=	Pond surface area

The total cost is then:

$CT = R_L x A + R_F x V_w$ Equation 4.14

CT	=	Total Cost
R_L	=	Unit cost of land
R_F	=	Unit cost of fill

The cost per unit volume of storage is then:

$C_V = CT / V_s$ Equation 4.15

Valley Topography

The effect of valley topography is shown in Figures 4.2 and 4.3. Valleys in the Johannesburg area typically have longitudinal slopes of between 1% and about 4%, with side slopes ranging between 3% and 10%, Figure 4.2 shows that, within these ranges the unit cost of storage is nearl; ˙ :dependent of valley longitudinal and side slopes.

Storage Per Unit of Fill

Where the storage volume is created by impoundment behind an embankment the fill and land costs are likely to dominate, and cost per unit volume of storage can be taken as the ratio of storage volume to wall plus land cost. If extraneous costs such as founding conditions, inlet and outlet works, and downstream protection are ignored, then a simple expression for cost per unit of storage can be developed ,and used to identify those factors that influence this unit cost most heavily.

For the purposes of this report the analysis was carried out for a valley with uniform longitudinal and side slopes, because a simple algebraic formula could be derived for the wall volume and the storage volume. A similar analysis could be done analytically for a valley with any geometrically definable shape, or numerically using at digital terrain modelling package for a randomly shaped surface.

For a uniformly sloping valley the following relationships apply:

Wall volume

$$V_w = w \times h^2/S_b + 2 \times h^3 / (3 \times S_b \times S_w) \qquad \text{Equation 4 1}$$

Storage volume

$$V_s = h^3 / (3 \times S_b \times S_l) - h^3 / (3 \times S_b \times S_w) \qquad \text{Equation 4.12}$$

V_w	=	Volume of wall fill above valley floor (m^3)
V_s	=	Volume of storage in pond to top of wall (m^3)
w	=	width of wall crest (m)
h	=	wall height above valley floor (m)
S_w	=	wall slope equal for both faces (m/m V:H)
S_l	=	valley longitudinal slope (m/m V:H)
S_b	=	valley side slope equal both sides (m/m V:H)

4.5.5. Aesthetics

Engineers often do not have a keenly developed aesthetic sense and specialist advice, from a Landscape Architect or other suitably qualified professional, should be taken to ensure that the pond fits into its context and does not become an eyesore (Brooker, 1992). The inlet and outlet structures, that are often the most offensive, can either be disguised, or changed in form and emphasized to make an artistic engineering statement. It is as easy, and no more costly, to build a visually attractive and interesting detention pond as it is to build an ugly utilitarian one, if sufficient thought goes into the design.

Investigation into public perceptions of water quality have shown that, the general public often places more emphasis on sensual indicators, sight, smell and even taste, than on scientific test results (Quick and Johansson, 1992). The designer of the pond needs to be aware of the public will judge the success or failure of the project by what it sees and feels, not on theoretical engineering considerations such as changes to flood probabilities downstream.

4.6. Cost / Benefit Analysis

4.6.1. Costs

It is not possible to produce a generic cost / benefit analysis for detention ponds, each case needs to be treated individually.

Typically costs per unit volume of storage will depend heavily on the major cost items:

- Volume of fill or excavation
- Excavation cost
- Land cost
- Outlet works cost

and each of these aspects needs to be considered separately for each individual site.

4.5.4. Safety and Public Health

Even if the pond does not have a dual use as an active recreational area rapidly rising water levels in a detention pond represent a severe threat, and the pond outlet is potentially particularly dangerous. The rate at which the water level in the pond is expected to rise will, to a large extent, determine the required safety measures.

Careful attention must be paid to the outlet design to minimize the chances of people being trapped, and the pond must have multiple escape routes. The sides of the pond should ideally all be easily climbed by a panicked person fleeing from rising floodwater, and not only a rational person who can search for a strategically placed ladder. The likely presence of young children needs to be considered, particularly where the pond is to be used as a playing field.

Sufficient, easily understandable and prominently placed signage, warning of the potential dangers, must be erected.

Urban stormwater may carry a high pollution load depending on the land use in the catchment. The first flush of stormwater after a long dry spell may have a quality not much better than raw sewage and, if this is allowed to deposit its debris and silt load in an area used by people, public health problems can certainly arise.

The design of the pond should, ideally, separate the active recreational areas from places where polluted material is likely to be deposited. Where this is not possible provision should be made to keep people out until the necessary cleaning up is complete.

On balance, however, stormwater ponds are probably safer than the smooth, fast flowing drains with unclimable sides, where flow velocities may exceed that speed at which a man can run.

- The balance of the catchment, about 50 ha in extent that feeds into the water course, either overland or via a piped outfall more than 400 m upstream of the culvert.

The stream gauge has been in operation for several years and a reasonable record of runoff events has been recorded. The location of the weir is shown on drawing Figure 5.1.

5.3.2. Detention Pond

During the winter of 1992 a detention pond was constructed upstream of the culvert under Edison Crescent, at the location marked "A" on drawing Figure 5.1, and the stream diverted so that all but very high floods are routed through the storage. The inflow to the pond is measured using a Crump weir, and the outflow measured by using the stage/discharge function of the 600 mm diameter outlet pipe from the pond. Water depths in the headwater pond of the inlet weir and in the detention pond itself were measured using pressure sensors and digitally recorded on data loggers. The design and construction of this pond is discussed in Appendix B.

5.3.3. Raingauges

Four autographic rain gauges are located within or very close to the catchment. The portions of the Thiessen polygons representative of these gauges within the catchment vary between 7,9 ha and 37,2 ha. Raingauge 2 situated in the centre of the catchment covers the largest area, so the degree of coverage is regarded as good.

5.3.4. Groundwater

Four boreholes are used to monitor ground water levels. With suitable instrumentation these boreholes could be used to give an indication of subsurface flows. It is probable that the foundation of Edison Crescent dams the flow of the perched water table and this could be checked by constructing a well close to the stream gauge.

of paving, and single unit residential properties of about 1 200 m^2 in area make up the balance at the catchment.

The 1990 Sandton 1:2 000 orthophoto shows that most of the residential 1 property in the township is developed but that some of the commercial or residential 2 property is still to be covered.

5.2.4. Stormwater Drainage System

The stormwater drainage system in the catchment is efficient. Roads run generally downslope and are well provided with kerb inlets at about 80 m intervals, draining into pipes that discharge directly into the water course that is not lined. From the layout it appears that no attempt had been made during the planning and design of the township to increase flow path lengths or reduce stormwater runoff in any other way. The system may be described as one for stormwater drainage rather than stormwater management.

5.3. Catchment Monitoring

The catchment is well instrumented to allow the computation of a water balance. Inflows are rainfall and municipal piped water while outflows are surface runoff, evapo-transpiration, percolation into the deep groundwater, sub-surface runoff in the perched water table and sewage flow. Numerous projects have been conducted by the Water Systems Research Group using this instrumented catchment and a detailed description of the instumentation is beyond the scope of this report.

5.3.1. Outfall Gauge

A stream gauge in the form of a Crump weir is located in the water course immediately downstream of the culvert under Edison Crescent. The catchment at this point is 2 hydrologically distinct elements:

- The formal piped system of Edison Crescent and Faraday Road, about 14 ha in extent that discharges into the water course only at the culvert.

5.2.2. Soils

The soil units were mapped as part of a regional investigation by HKS Consulting Engineers (1979) and found to be typical of the underlying basement granite geology. Three units occur within the catchment, hillslope, gulleywash and gulleyhead.

Hill slope.

The expected stratigraphy would be sandy hillwash overlying residual granite clayey sand with density increasing with depth. This soil stratigraphy frequently exhibits a perched water table in the hillwash or reworked residual soil of lower density.

Gulleyhead.

Less pervious soils occur in the gulleyhead soil unit close to the watershed at the east end of the catchment. The soil profile typically comprises a 0,2 m to 0,3 m thick layer of hillwash overlying a ferruginous pebble layer or ferricrete horizon between 0,3 m and 1,0 m thick. The hillwash is relatively pervious but the ferruginised layer limits infiltration to depth, generally resulting in a perched water table.

Gulleywash.

The soils of the main westward flowing drainage line are typical of the gulleywash profile. The profile is generally a silty sand topsoil about 0,3 m thick overlying a slickensided clay or clayey sand alluvium underlain by clayey sand residual granite. The gulleywash unit is poorly drained with the clayey soils allowing little infiltration.

5.2.3. Development

Development within the catchment is typical of an upper socio-economic township in Gauteng. Land use varies between high density office and shopping centres with extensive paved parking areas directly connected to the drainage system through to a significant proportion of park. Townhouse developments with differing proportions

5 SUNNINGHILL MONITORED CATCHMENT

5.1. Introduction

Monitoring of stormwater detention ponds is regarded as important for several reasons, Watt and Paine (1993) in their description of a case study in Ontario give three:

- the biological, chemical and physical processes are poorly understood,
- the knowledge base on field instrumentation is sparse,
- guidelines or regulations for performance monitoring are imminent in some jurisdictions.

The biological and chemical processes are not relevant to this study as it does not deal with water quality. The physical processes are important, hydraulic characteristics of pond outlets are not well defined, particularly where high debris loads are likely to occur. Whilst a legal requirement for performance monitoring of stormwater management installations does not appear to be not imminent in South Africa at this time, several local authorities are developing stormwater management policies that may offer a financial incentive to developments where stormwater runoff is restricted. It is likely that some form of performance monitoring will be introduced in the future.

5.2. Catchment Description

5.2.1. Location and Topography

The Sunninghill monitored catchment is located in Sandton, Gauteng, to the north of the N3 and to the west of Megawatt Park. It is 65,7 hectares in extent and drains westward from an elevation of 1 520 m to 1 460 m with a maximum drainage route length of about 1 200 m. The topography is gently rolling with typical slopes of 5% to 7%.

4.8.5. Atlantis Industria Stormwater Management System

This stormwater management scheme, implemented between 1978 and 1986 in the Western Cape, is described in some detail in The Civil Engineer in SA, February 1988 and will not be described again. It is presented here as an example of a regional scheme, covering some 20 km^2 with 11 detention ponds, in contrast to the 3 local schemes described above. One of its unique features is the deliberate recharge of a ground water aquifer to return water to the boreholes feeding the town water supply.

excavating a trench about 0.8 m deep and 1.2 m wide 300 m long around the soccer field, lining it with geofabric, and backfilling with about 0.6 m of 19 mm stone. Geofabric was then placed over the stone and covered with about 50 mm of soil and the top of the trench trimmed to trapezoidal shape. The field, and the soil over the swale were planted with kikuyu grass. Stormwater is directed onto the surface of the swale at 2 points on the long side, and escapes via a single 450 mm diameter pipe in the middle of the opposite side of the field. The swale itself provides about 180 m^3 of storage, equivalent to about 20 mm of excess rain on the contributing catchment, and a low soil berm was constructed along the edge of the field to provide an additional 2 000 m^3 of emergency storage.

Benefits anticipated include:

- A significant attenuation of the runoff hydrograph
- Improvement of the runoff water quality by filtration through the grass and stones.
- Trapping of floating debris.
- A reduction on the irrigation water volume required for the soccer field because infiltration from the swale will augment the soil moisture under the playing area.

Possible problems anticipated included:

- Unacceptably long lived wet patches on the field reducing its use to the community.
- Gradual blocking up of the voids in the stone drain reducing its effectiveness.
- Unacceptable accumulation of debris on the playing field.

To the best of the author's knowledge none of these problems has materialized in the 5 years that the system has been in operation.

The hydraulic operation of the culvert and inlet and outlet structures was checked in a 1:30 scale model constructed by the Department of Civil Engineering of the University of the Witwatersrand.

The detention pond was completed in 1995 and operated during a storm in October 1996, when the water depth in the basin reached about 1 m. Some damage to paving and dry stack retaining walls was done by fast flowing, turbulent water immediately below the inlet, and a significant quantity of sediment, mainly mine sand was deposited in the basin. The necessary maintenance work is still to be done (July 1997)

4.8.4. Graceland

The model village of Graceland was developed, on behalf of a consortium of industries in the Wadeville area of Germiston, to house middle income workers and supervisory staff. The project was open to any business in the area, but an initial criterion was that the purchaser of a home had to work within a reasonable bicycle ride of the township. Any housing subsidy was a matter between the employer and the purchaser of the home, but the employer was required to contribute a sum towards the community infrastructure in the township. A school, clinic, and recreational facilities could therefore be developed rapidly, and without financial support from the local authority.

The township is roughly rectangular, about 600 m by 400 m, and bounded by the Elsburg Spruit along its south western border. A recreational complex, including a soccer field has been established roughly in the middle of the development. Stormwater is managed within the township on sound hydrological principles, there are a minimum of pipes and roof runoff is directed first onto grassed areas before draining onto roads.

On the side of the township stormwater from an area of about 8 ha is directed into a stone filled swale that forms a shallow depression around the perimeter of the soccer field. The swale was constructed by

- An additional detention pond upstream in the catchment could be required for the system to provide an adequate level of protection to the downstream properties.
- The outlet from the pond would be via a 2 m diameter pipe jacked through the variable fill forming the Booysens Road embankment, and the construction of this outlet would require the demolition and reconstruction of the left abutment of the existing culvert.

All of these factors were taken into account in the decision to implement the stormwater management option, and construct the detention pond.

Hydrological modelling was done using Hydrosim, with the catchment discretized into about 60 sub-catchments, and the effect of the off channel detention ponds analysed using the reservoir routing module in that program. Numerous combinations of inlet and outlet capacities, as well as sensitivity to catchment hydrological characteristics were computed and the final 50 year recurrence interval design discharges are given in table 4.4

Table 4.4 : Booysens flood management, Bawden Park detention pond 50 yr.. recurrence interval design peak discharges.

Storm Duration (min)	Upstream Discharge (m^3/s)	Bypass Discharge (m^3/s)	Inflow (m^3/s)	Outflow (m^3/s)
30	87.7	62.9	24.8	12.4
45	86.9	62.9	24	13.3
60	82.9	62.9	20	13
75	79.1	62.9	16.3	11.9

The table shows that peak discharges change relatively little with storm duration, but that the peak inflow and outflow discharges occur for different storm durations.

- The construction details of the old culvert were unknown, concrete volumes and quantities of reinforcement could not be determined, so demolition times could not easily be anticipated.
- Geotechnical investigation showed the fill of the old Booysens Road embankment to be very variable, so the total extent of the 5 m deep excavation could not be determined in advance.
- The basement walls of the building on the left bank immediately downstream of the culvert were very close to the abutment of the culvert outlet.
- The main Johannesburg to Cape Town telecommunication cables crossed the culvert in the Booysens Road sidewalk.
- A dry detention pond could accommodate a soccer field, so making active use of the derelict park.
- Existing downstream flooding problems would be aggravated by increasing the drainage capacity of the system.

In favour of simply increasing the culvert capacity were:

- About half of the land area required by the pond had been leased to the SPCA for 30 years.
- Of the total excavation volume of 20 000 m^3 required to create the detention pond about 13 000 m^3 was hard rock quartzite that required blasting in the close proximity of a brick lined outfall sewer and operating businesses.
- Haul distances for spoil were long, and significant traffic disruption could be expected.
- The hydraulics of the culvert, and particularly the complex siphon outlet into the detention basin, could not be calculated with confidence.
- The hydrological characteristics of the catchment were changing and future changes in runoff volumes could not be predicted with confidence
- Immediate maintenance requirements would be increased.

original design calculations showed the capacity of the old culvert to be equal to that of the new reach, subsequent calculation showed that the old culvert was a significant constriction in the system, with a capacity of about 55 m^3/s compared with about 90 m^3/s provided in the new culvert. Experience on site during the construction of the road, and later discussions with long time residents of the area, proved that the old culvert was unable to pass fairly frequent floods, but that this did not present a problem because flood water simply accumulated in the low lying park land immediately upstream of the culvert. The construction of the Booysens Road culvert in about 1910 had created an informal detention pond upstream of the road embankment.

Construction of Klip River Drive with its new culvert, and the placing of 19 000 m^3 of fill in the park, inadvertently removed this detention facility, indirectly causing two floods, in December 1986 and again in January 1990, that caused damage totalling about R30 million immediately upstream of the new culvert inlet.

Alleviation Measures

Two possible alleviation schemes were considered; firstly the replacement of the old Booysens Road culvert with a new structure matching the capacity of the new Klip River Drive culvert, or secondly a stormwater detention pond to replace the storage removed from the system when the road was built. Estimated costs of the two options were similar and intangible factors weighed heavily in the decision on which option to implement.

In favour of the detention pond were:

- Booysens Road carried about 11 000 vehicle movements per day each way across the culvert, traffic deviation would be difficult and the costs associated with increased trip times could not be estimated accurately.

Table 4.2 : Sunward Park detention ponds, design discharges and dam volumes.

Location	Volume m³	Surface Area m²	Peak Discharge 5 yr. RI m³/s	Peak Discharge 50 yr. RI m³/s
Nicholson Rd Culvert			6.7	13.9
Dam 1	4,000	6,300	5.5	12.8
Dam 2	6,850	10,400	4.6	10.3
Dam 3	11,618	15,400	2.8	5.8
Kingfisher Rd Culvert			2.9	5.9

Cost savings were also substantial. No analysis of the constructed cost was done, but the cost estimates, as set out in the design report for the drainage system from South Rand Road to the southern boundary of the township, are given in table 4.3. The equivalent 1997 cost is based on the variation in the consumer price index, and both estimates exclude establishment costs and taxes such as GST or VAT.

Table 4.3 : Comparison between estimated costs of Sunward Park stormwater management system and alternative free drainage system.

Design Alternative	Estimated Cost 1983 Rands	Estimated Cost 1997 Rands
Stormwater drainage system with no attenuation	R333 600	R2 015 000
Implemented system with three detention ponds	R150 000	R906 000

4.8.3. Booysens Flood Management

Background

In the early 1980's the City Council of Johannesburg constructed Klip River Drive down the West Turffontein Valley, from Rifle Range Road in the south to Booysens Road in the north. The road started very close to the watershed in the south and closely followed the route of the West Turffontein canal that had been constructed in about 1935. The canal was decked over in several places, including a reach about 200 m long immediately above the old culvert under Booysens Road, where about 19 000 m³ of fill was placed in the park land adjacent to the old canal to accommodate the required road grades . Although the therefore did not require the plotting of a 50 year floodline in terms of Section 169A of the Water Act.

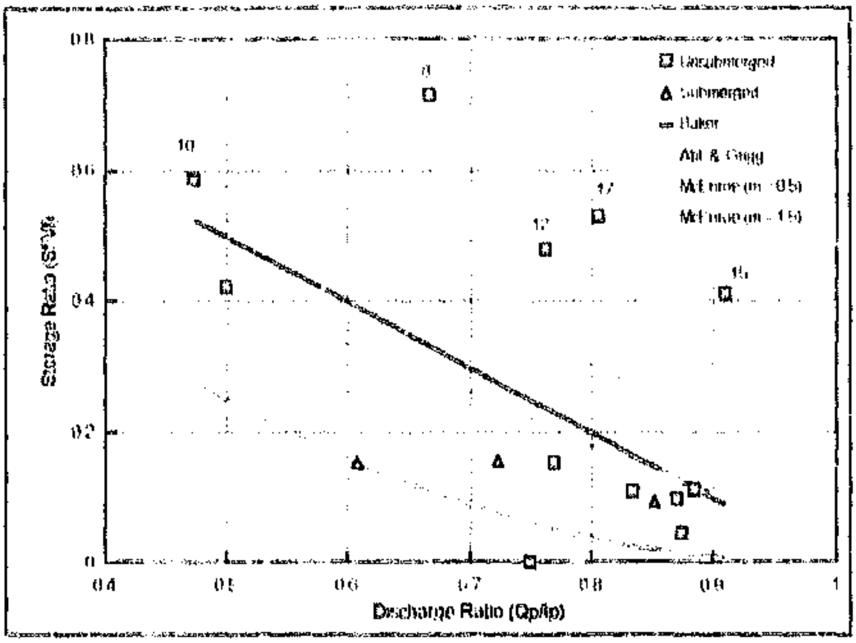
The township layout included a park stand along this water course from culvert under South Rand Road on the northern boundary, south to Kingfisher Drive, from where it was expected that the stormwater would be contained in a pipe.

The stormwater management system was designed by the author. Hydrographs were generated using a variation of the Rational Method described by James (1981) and flood routing calculations were made using an earlier version of the program given in Appendix C. Several alternative designs were considered, and the implemented solution required the construction of three shallow earth dams with gabion weir spillway outlets. These dams operate in series as on channel, blue-green, ponds with small, permanently wet, lakes of 400 m^2 to 700 m^2 maintained for aesthetic reasons. Because of the permeability of the 0.5 m high gabion weirs the dams drain freely down to the operating water level of the wet ponds, but greatly retard flow from small events with insufficient volume to raise the water level to weir crest height.

Wall side slopes are flat, 1:6 V:H, to allow easy maintenance by tractor mounted mowers, and to ensure stability of the embankment slopes even under overtopping flow. The slopes and the dry area within the basins is protected by mixed veld grass native to the area, maintained by periodic mowing. When inspected by the author in 1997 the dams were found to be in good condition with little apparent silting of the wet ponds.

Significant attenuation and retardation of the flood peaks is attained. the design discharges and detention pond volumes are given in table 4.

Figure 5.6 : Storage Ratio vs. Discharge Ratio for the Sunninghill detention pond for the period 14 November 1992 to 31 January 1993 Compared to Published Design Guidelines.



Sf/Vf and Qp/Ip

where Sf is the required flood storage

Vf is the flood volume

Qp is the outflow peak and,

Ip is the inflow peak

these indicators for data recorded at Sunninghill are set out in Table 5.3 and plotted against the design guideline curves in Figure 5.6

Table 5.3 : Storage to Volume and Peak Outflow to Peak Inflow Ratios for the Sunninghill detention pond for the period 14 November 1992 to 31 January 1993.

Event Number	Stored Volume Sf m^3	Flood Volume Vf m^3	Peak Outflow Qp m^3/s	Peak Inflow Ip m^3/s	Storage Ratio Unsubmerged	Storage Ratio Submerged	Discharge Ratio Qp/Ip
10	54	92	0.02	0.04	0.59		0.47
16	3	7.1	0.02	0.03	0.4[illegible]		0.60
4	320	2140	0.76	1.25		0.15	0.61
8	1	1.4	0.00	0.00	0.71		0.67
9	156	1013	0.56	0.76		0.16	0.72
13	0	1.4	0.00	0.00	0.00		0.75
12	64	178	0.03	0.04	0.46		0.76
5	41	266	0.10	0.13	0.15		0.77
17	456	890	0.08	0.10	0.53		0.81
1	14	130	0.04	0.05	0.11		0.83
6	187	1956	0.52	0.61		0.00	0.85
11	36	373	0.20	0.23	0.10		0.87
7	16	307	0.07	0.08	0.05		0.88
14	16	90	0.04	0.04	0.11		0.88
15	316	766	0.10	0.11	0.41		0.91

The graph in Figure 5.6 shows that the measured results obtained from Sunninghill agree fairly well with the design guidelines. The points labelled 8, 10, 12, 15, 17 represent unreliable data as can be clearly seen from the hydrographs plotted in Appendix A.

- Inaccuracies in the outlet stage/discharge curve because of variations in the assumed geometry of the pipe mouth, particularly the abrupt changes in diameter around the socket.
- The effect of ignoring velocity head would be proportionately higher for shallower water depths.
- Variations in inlet and outlet stage/discharge curves caused by the build-up of debris.
- Inaccurate calculation of the stage/storage curve for the pond as a result of sparsely surveyed points, and the assumption of a smoothly varying curve between calculated values at 0.5m contour intervals.
- The assumption of a level pond in the reservoir would be increasingly invalid for smaller flows.
- Errors inherent in the instrumentation would be increasingly apparent for shallow water depths.

5.5.4. Routing Calculation

The theoretical reservoir routing program was tested against the measured results for Event No 4, and the resulting curve is plotted on the hydrographs for that event in Appendix A. The measured and computed curves agree fairly well with the times of peak discharge corresponding closely. The reversal of the peak discharges of the outflow hydrographs, for the first peak the measured value is higher than the computed value but for the second peak the computed value is higher, supports the assumption that the discharge coefficients decrease with decreasing discharge.

5.6. Design Guidelines

Several agencies have published guidelines for the preliminary sizing of detention ponds (McEnroe, 1992). The curves contained in these guidelines are mostly presented as relationships between the dimensionless ratios:

These inlet coefficients agree reasonably well with values given in the literature, e.g. Stephenson (1981, p246) gives the following values for guidance:

unsubmerged	C_b	0.77	
		square	bevel
submerged	C_c	0.57	0.65
	C_b	0.79	0.83

The outlet from the pond is a spigot and socket pipe with its socket forming the inlet through the headwall, giving an approximately bevelled edge. Pond depths were measured using a piezo-resistive sensor in a housing close to the outlet. Whilst this housing would have reduced the velocity somewhat it would not have been as effective as a formal stilling well and a residual velocity head would have existed. The effect of velocity was ignored in the measured head levels and in the discharge calculations, and this probably accounts for the apparently high unsubmerged discharge coefficient.

The residual velocity head would also have had some effect on the calculated volumes as it would have caused an under estimate of the average water level in the pond.

5.5.3. Reasons for Scatter

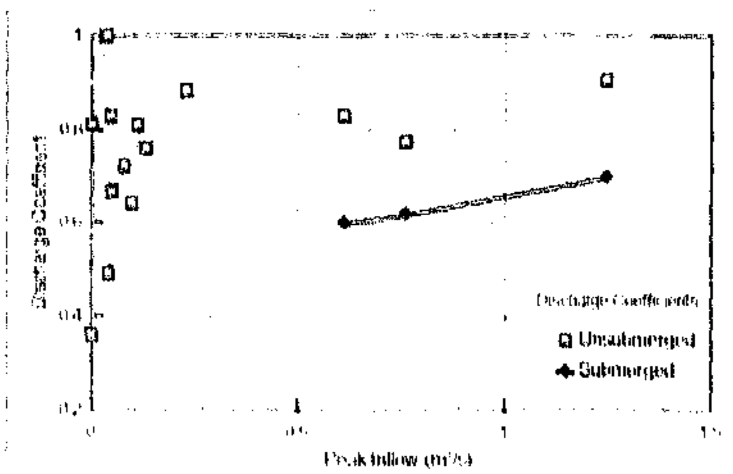
These graphs show a considerable scatter of data for the smaller events, but a relatively good relationship between the data points for larger flows. Some general causes of inaccuracies whose effect would be greater for smaller discharges are:

- Inaccuracies in the inlet stage/discharge curve as a result of construction inaccuracies in the crump weir crest.
- Turbulence in the headwater pond of the inlet weir would cause erroneous readings of the upstream headwater levels.

5.5.2. Discharge Coefficients

Figure 5.5 shows the relationship between the discharge coefficients found to give the best visual fit for the hydrographs plotted in Appendix A and the peak measured inflow. There is considerable scatter of the low flow data but for higher discharges that measured data appears to be consistent. A trend of increasing values of discharge coefficients with increasing discharge is apparent for both the submerged and unsubmerged conditions, but more data would be necessary to confirm this from plots of the data alone. The shapes of the plotted hydrographs for Event Nos. 4 and 5 in Appendix A also indicate decreasing discharge coefficients with decreasing discharge. In Event No. 4 it is the comparison with the computed hydrograph discussed below that gives this indication. In Event No. 5 the best fit, obtained for the first peak over estimates the peak outflow associated with the second inflow peak indicating that a reduced discharge coefficient would give a better fit for this second, lower discharge.

Figure 5.5 : Discharge Coefficients for the Sunninghill detention pond for the period 14 November 1992 to 31 January 1993



Further monitoring would be necessary to determine the ability of this pond to attenuate hydrographs of greater volume.

Table 5.2 : Ranked events and attenuation ratios for the Sunninghill detention pond for the period 14 November 1992 to 31 January 1993.

Event Number	Peak Inflow m^3/s	Peak Outflow m^3/s	Ratio Qp/Ip
8	0.00	0.00	0.67
14	0.00	0.00	0.75
16	0.03	0.02	0.50
10	0.04	0.02	0.47
12	0.04	0.03	0.76
18	0.04	0.04	0.88
1	0.05	0.04	0.83
7	0.08	0.07	0.89
17	0.10	0.08	0.81
15	0.11	0.1	0.91
5	[illegible]	[illegible]	0.77
11	0.23	0.20	0.87
6	0.61	0.52	0.85
9	0.79	0.55	0.72
4	1.25	0.76	0.61

Figure 5.4 : Graph of attenuation ratio and Outflow vs. Inflow for the Sunninghill detention pond for the period 14 November 1992 to 31 January 1993

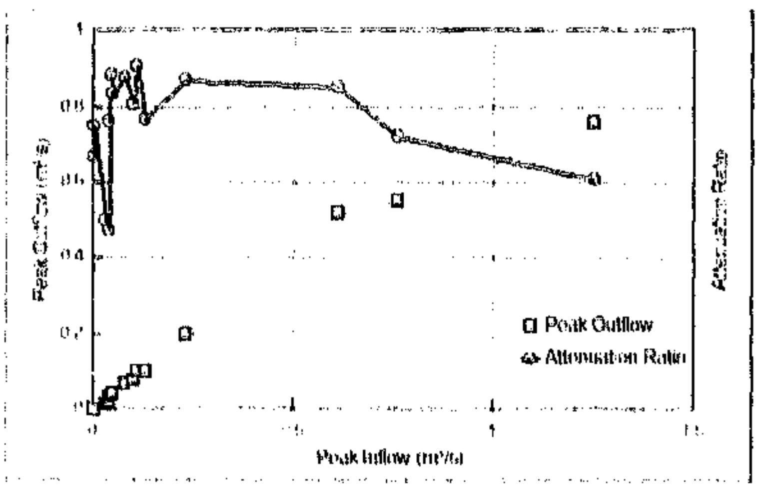


Table 5.1 summary of computed peak inflow and outflow discharges for the Sunninghill detention pond for the period 14 November 1992 to 31 January 1993.

Event Number	Date	Start Time	End Time	Peak Inflow	Peak Outflow	Reduction %
1	14/11/92	21:48	23:06	.048 m^3/s	.040 m^3/s	16.7%
4	4/01/93	15:52	17:30	1.25 m^3/s	0.76 m^3/s	39.2%
5	8/01/93	17:02	19:12	0.13 m^3/s	0.10 m^3/s	23.1%
6	10/01/93	15:50	18:00	0.61 m^3/s	0.52 m^3/s	14.8%
7	22/01/93	14:16	16:32	0.08 m^3/s	0.07 m^3/s	12.5%
8	25/01/93	06:00	08:36	0.6 l/s	0.4 l/s	33.3%
9	25/01/93	18:00	20:36	0.76 m^3/s	0.55 m^3/s	27.6%
10	28/01/93	05:46	09:22	.038 m^3/s	.018 m^3/s	52.6%
11	28/01/93	18:56	21:38	0.23 m^3/s	0.20 m^3/s	13.0%
12	29/01/93	02:40	09:20	0.042 m^3/s	0.032 m^3/s	23.8%
13	29/01/93	16:58	18:46	0.8 l/s	0.6 l/s	25.0%
14	29/01/93	19:32	22:14	0.043 m^3/s	0.038 m^3/s	11.6%
15	30/01/93	02:04	09:32	0.11 m^3/s	0.10 m^3/s	9.1%
16	30/01/93	15:04	17:32	3.2 l/s	1.6 l/s	50.0%
17	31/01/93	02:22	09:50	0.098 m^3/s	0.079 m^3/s	19.4%

5.5.1. Attenuation Ratios

Table 5.2 ranks the events into order of peak inflow and gives the attenuation ratio, peak outflow : peak inflow ($Qp : Ip$) for each event. Figure 5.4 is a graph showing the attenuation ratio and the relationship between the peak inflow and outflow. This graph shows that the Sunninghill detention pond makes efficient use of the available storage for the range of hydrographs measured. The ratio $Qp : Ip$ decreases as the peak inflow increases, indicating that the degree of attenuation increases with increasing inflow peak.

The data collected is, however, insufficient to allow more than a superficial assessment of the attenuation efficiency of this pond. The hydrographs that yielded useful results with relatively high inflow peaks, namely events 4, 6 and 9, were all peaky with relatively small volumes.

5.4.3. Calibration

Some degree of calibration of the system could be achieved without resort to model testing of the hydraulic structures:

- The volumes predicted by the measured inflow and outflow and by the stage/storage relationship of the pond itself can be compared.
- The mathematical relationships governing the shapes of the inflow and outflow hydrographs define the relative positions of the maxima and minima. The peak of the outflow hydrograph must lie on the falling limb of the inflow hydrograph, and for compound events, the local minimum of the outflow hydrograph must lie on the rising limb of the inflow curve.

In each case the discharge coefficients, Ch, Cv and Ch, and the submergence ratio y/D, were adjusted to obtain the best visual fit in both the hydrographs and the volume curves. The coefficients for the crump weir at the inlet were not adjusted for each event.

5.5. Results

The data record for the period from mid November 1992 to early January 1993 has been examined and 15 useable events identified that are summarized in Table 5.1. The plotted hydrographs are given in as Figures A1 to A15 in Appendix A.

debris and other temporary blockages. Further monitoring, with a visual record of the state of the outlet, would be required to determine this.

The correction for the effect of slope on discharge given by Henderson (1966 : p261) was not applied explicitly but as part of the discharge coefficients.

Figure 5.3 shows a typical stage/discharge curve for the pond outlet with the unsubmerged discharge coefficient $C_b = 0.9$, the submerged discharge coefficients $C_c = C_b = 0.7$, and submergence assumed to occur at $y/D = 1.2$. These coefficients give a smooth transition between the unsubmerged and submerged discharge curves but this did not always occur in the selection of values to achieve the curve matches used in the analysis of the data.

Figure 5.3 : Stage discharge curve for 600 mm diameter pipe outlet from detention pond

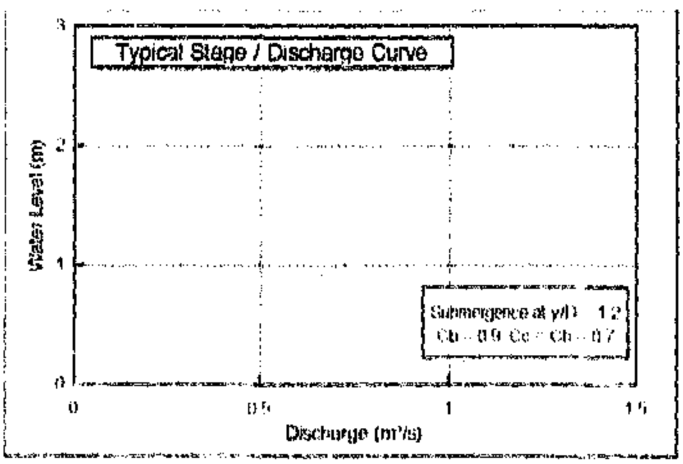
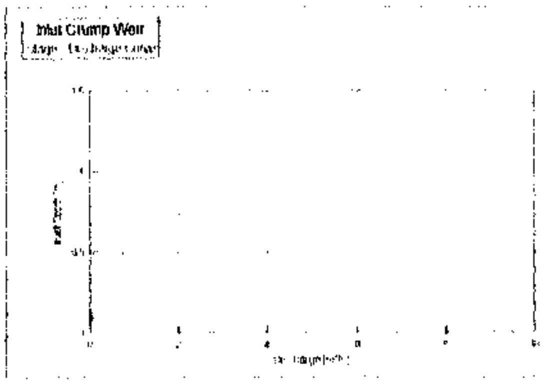


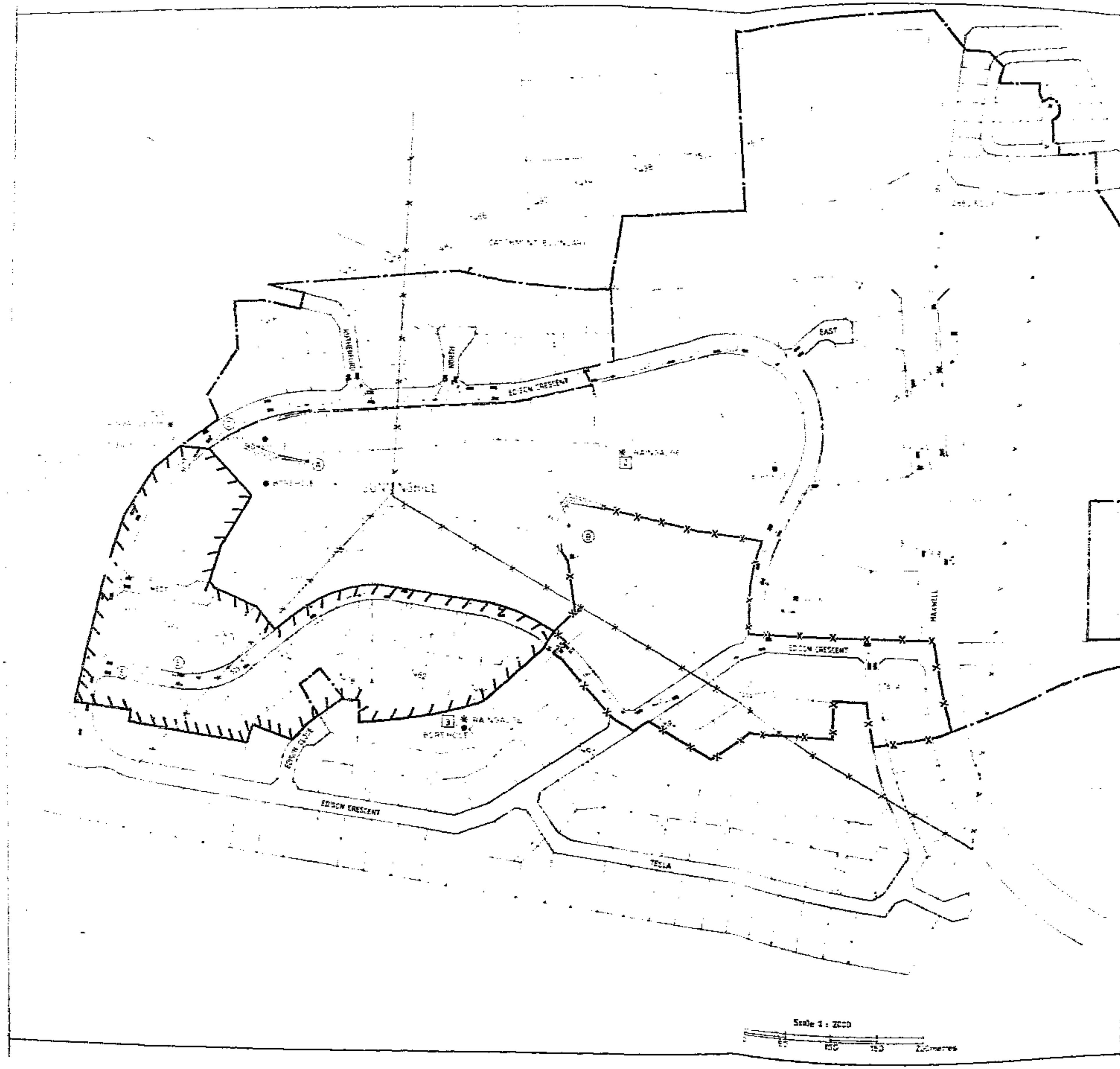
Figure 5.2 : Stage discharge curve for the Crump weir controlling inflow to the detention pond.



5.4.2. Outflow

The outlet from the pond is a 600 mm diameter stormwater pipe leading from a vertical headwall. Outflow is calculated by measuring the water level in the pond, and applying this to the stage/discharge relationship for the pipe inlet. The slope of the pipe is steep so inlet control could be assumed for all discharges.

The relationship between headwater depth and discharge for unsubmerged, inlet controlled circular conduits is not explicit and the solution to the equation would be time consuming if done numerically for each water level. A table was therefore set up which related the critical depth in the pipe to the critical energy level and hence to the headwater energy level as described by Stephenson (1981: pp 244 - 246). The table was then used in reverse, with coefficients applied to the theoretical discharge at a critical energy equal to the headwater depth to allow for entrance losses. The coefficients Ch, Cv and Ch are described by Stephenson (p246). Analysis of the data showed that these coefficients were not constant, but increased with the pond water level as shown on Figure 5.5 This variation may be constant as a function of the hydraulics of the inlet, or it may depend on the effects of



TOTAL AREAS	
CATCHMENT TO (A)	[illegible] ha [illegible] ha
(B)	[illegible] ha
(C)	8,26 ha
(D)	5,76 ha
(E)	[illegible] ha
RS 1	12,2 ha
RS 2	37,2 ha
RS 3	7,9 ha
RS 4	12,4 ha
	69,7 ha

Figure 5.1

SUNNINGHILL MONITORED CATCHMENT

2770 | 01

5.3.5. Sewage

A sewage gauge also exists below the catchment which is monitored to give a comparison between wet and dry weather flows as well as to identify any response to storm rainfall.

5.4. Detention Pond Monitoring and Results

5.4.1. Inflow

The inflows to the pond are determined by measuring the water level in the diversion structure upstream of the Crump weir and applying these levels to the stage discharge relationship of the weir. Analysis of the data indicates that the discharge coefficient of the weir may not be constant but probably increases as some function of headwater level. There was, however, insufficient data available to test this assumption, and the variation in the discharge coefficient was not taken into account in the volume as it was felt that inherent errors elsewhere, such as the very rough conditions in the headwater pond of the weir overshadowed this consideration. Figure 5.2 shows the stage/discharge curve for the inlet weir, calculated using equations B1 and B2.

Figure A1 Event 1 Hydrographs

Volumes to Date

Stage - Discharge

A 1 APPENDIX A : GRAPHS

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- Build up a register of stormwater management facilities operating in this country. As much design information as possible, for example construction costs, multiple land use, catchment characteristics, storage volumes, inlet and outlet characteristics, should be recorded.
- Establish a monitoring programme to record the functioning of these facilities. The frequency with which they called upon to operate, the frequency with which they fail, both hydraulically and structurally and the maintenance effort required to keep them operational should all be recorded. Quantitative, and qualitative anecdotal information would be useful.
- Poll Local Authority Engineers to establish reasons for their resistance to the implementation of stormwater management schemes. Some of these reasons, that may be real or perceived, include:
  - pervious bad experiences
  - high construction costs
  - difficult or costly maintenance
  - lack of understanding and knowledge
  - operational dangers including possibly trapping people or increased downstream flood risk
  - probability of failure, both structural and hydrological
  - excessive land requirements
- Poll the opinions of Councillors and Local Authority Politicians to establish, inter alia:
  - their level of knowledge
  - attitudes to the environmental degradation resulting from the impact of urban development on stormwater runoff
  - attitudes to stormwater management vs. drainage systems
  - prejudices, including probability of failure, construction costs, maintenance costs etc.
- Poll public opinion along the same lines as Councillors and Politicians

The unit cost of storage behind an earth fill embankment in a triangular valley appears to be independent of valley side slopes and relatively insensitive to valley longitudinal slope.

All of the 15 events measured in the Sunninghill detention pond showed a significant degree of attenuation with a percentage reduction ranging from 9% to 50%, and the larger events generally showing a proportionately greater reduction.

Evaluation of the data showed that discharge coefficients are not constant but appear to increase as the discharge does. There is, however, significant scatter in the data, particularly for the smaller events. Further monitoring will therefore be necessary to confirm this and ascertain the reason.

All of the reliable data points recorded in the Sunninghill Park detention pond yielded a relationship between storage ratio and discharge ratio that fell within the published design guidelines.

6.1. Further Research

Stormwater management facilities in general, and detention facilities in particular, have not been implemented in South Africa as often as could reasonably be expected. Traditional stormwater drainage systems still appear to be favoured, even where projects appear to be ideally suited. In the author's experience little has changed since the research into attitudes and implementation of stormwater management systems completed by Watson and Miles in 1979. (Watson and Miles, 1982). The reasons for this resistance are not clear, but may be related to lack of knowledge and perceived problems on the part of design engineers, local authority engineers and councillors, and the public generally. It is, however, clear that little progress will be made until the reasons are understood and future research should focus in this direction.

Some suggested topics are therefore:

many factors to be explicitly solvable, and is largely by trial and error. The process can be accelerated by using a predictor / corrector method described in the text, but should be undertaken with care because local maxima may be exist, and it is essential to determine the critical storm duration for each pond individually.

The construction of flood detention ponds may, under certain circumstances, increase the level of flooding downstream, if they cause the peaks of hydrographs to superimpose where they would not have done before. This situation is unlikely, but it is incumbent on the design engineer to check unusual combinations of events to ensure that such situations do not arise.

Secondary issues such as structural stability, environmental considerations, ease of maintenance, public health and safety, and aesthetics should be accorded a design status almost equal to that of the hydraulic and hydrological operation of the ponds. Stormwater management facilities are generally financed with public money, and will be judged by the public using different criteria from those applied by the design engineer.

Detention ponds are of benefit in attenuating flood peaks and generally improving downstream water quality. Improvements to water quality may not be significant unless deliberately addressed in the design. Water quality can deterio[illegible] example if pollutants bound to the sediments are released into solution.

Detention storage is generally cost effective. The cost / benefit calculations are, however, highly site specific and cannot be generalized. Peripheral considerations frequently play an important role in the decision making process.

Costs are often dominated by the value of the land occupied, and, wherever possible, facilities should provide multiple use opportunities on the land that they occupy.

6 CONCLUSIONS

It is generally agreed that urbanization increases small flood and spate discharges from small catchments, but there is some dispute as to whether the peaks of rarer events and floods in large catchments are affected. More data is required before this question can be answered with certainty.

It is well established that urbanization does have an adverse effect on stream channel morphology. Erosion rates are increased even when the level of development is relatively low. Stream channels do not become inherently unstable, erosion rates accelerate in response to the increased frequency of run off and increased spate flow peaks.

Current stormwater management techniques may not be adequate to prevent the adverse impact of urbanization, and measures such as over control of the peaks may need to be implemented in sensitive catchments.

Detention storage is an effective stormwater management technique that has been successfully implemented in numerous schemes. Measured data on the operation of stormwater management facilities in general, and detention ponds in particular, appears to be scarce and more monitoring of installations is required, to permit the effective validation of computer based models, and give guidance to designers.

Monitoring of stormwater management facilities should not be restricted to hydraulic and hydrological information, operational and maintenance information is equally important. Local authorities often resist the construction of detention ponds on the grounds that their maintenance load will be increased, data comparing the relative levels of maintenance effort required in managed and unmanaged urban catchments is urgently required.

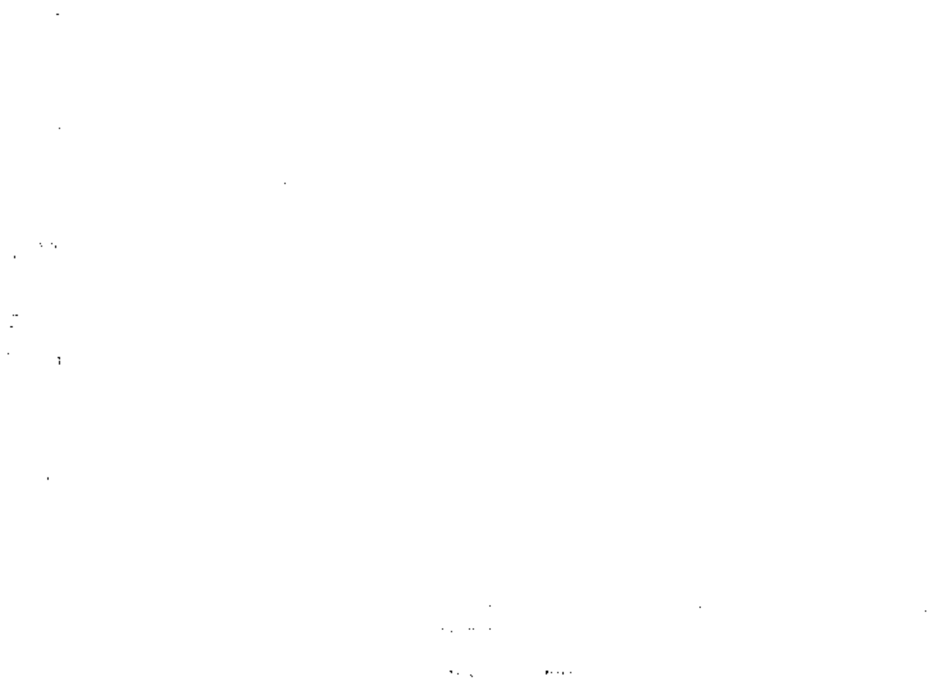
The determination of the critical storm duration for a single detention pond, or a system of ponds within a catchment is dependent of too

Figure A7 - Event 9 Hydrographs

Volumes in Dam

Stage - Discharge

Figure A5 : Event 7 Hydrographs



Volumes in Dam

Stage - Discharge

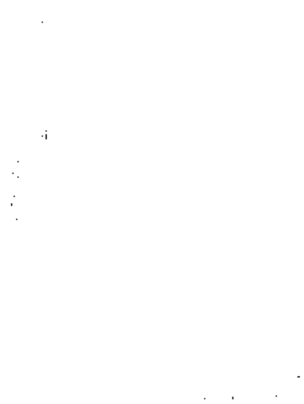


Figure A13 - Event 15 Hydrographs

Figure A12 : Event 14 Hydrographs

Figure A11 - Event 13 Hydrographs

Volumes at Dam

Stage - Discharge

Figure A10 : Event 12 Hydrographs

Figure A9 : Event 11 Hydrographs

Volumes at Dam

Stage - Discharge

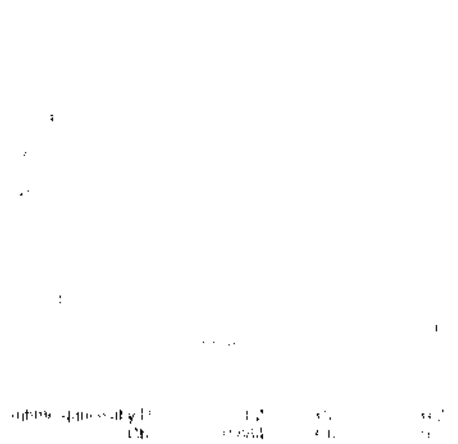
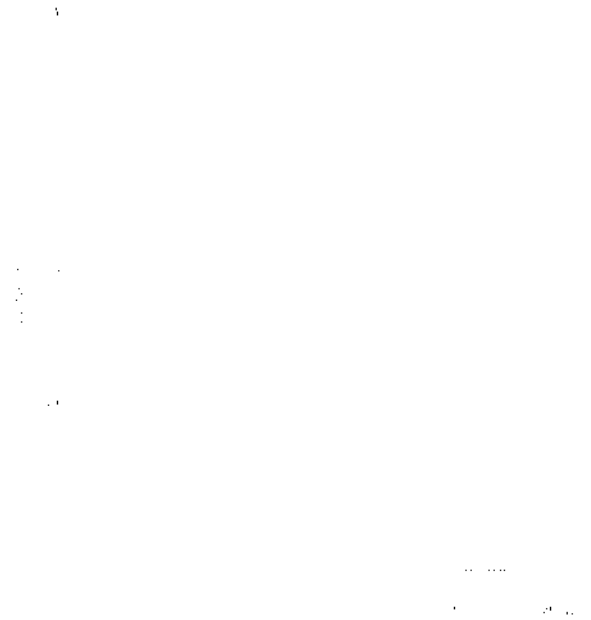
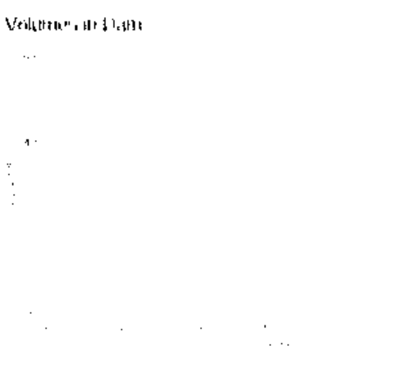


Figure A8 - Event 10 Hydrographs





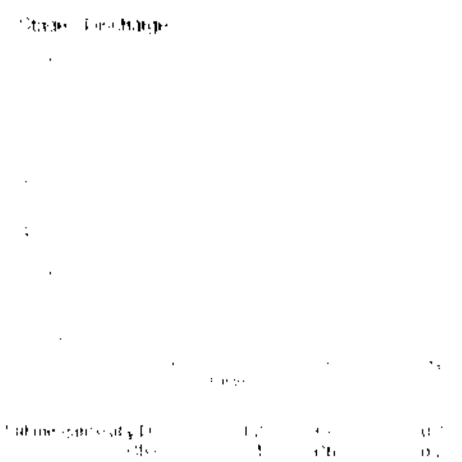


Figure A6 : Event 8 Hydrographs

Volumes in [illegible]

Stage - Discharge

Figure A4 : Event 6 Hydrographs

Volumes in Dam

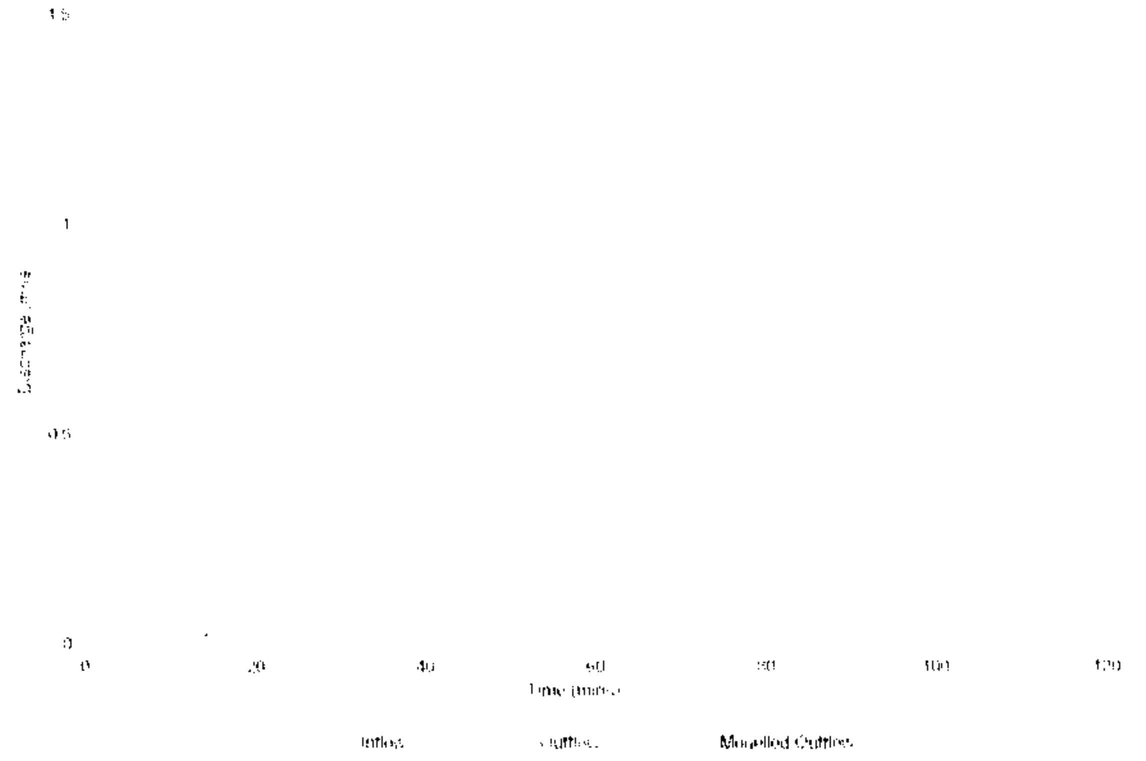
Stage - Discharge

Figure A3 : Event 5 Hydrographs

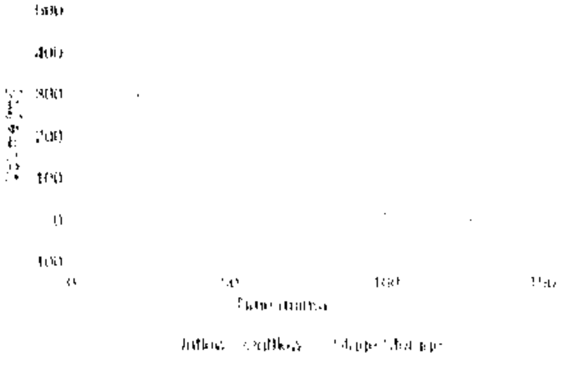
Volumes

Stage Discharge

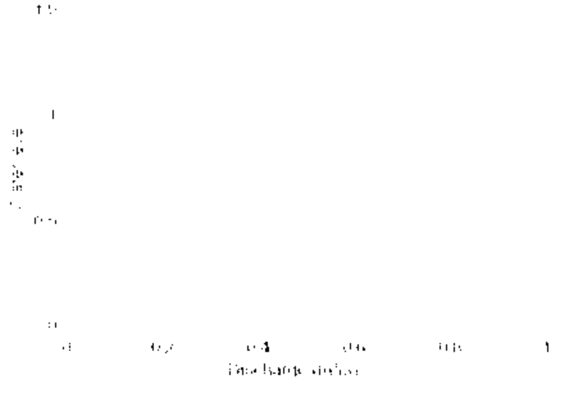
Figure A2 : Event 4 Hydrographs



Volumes in Data

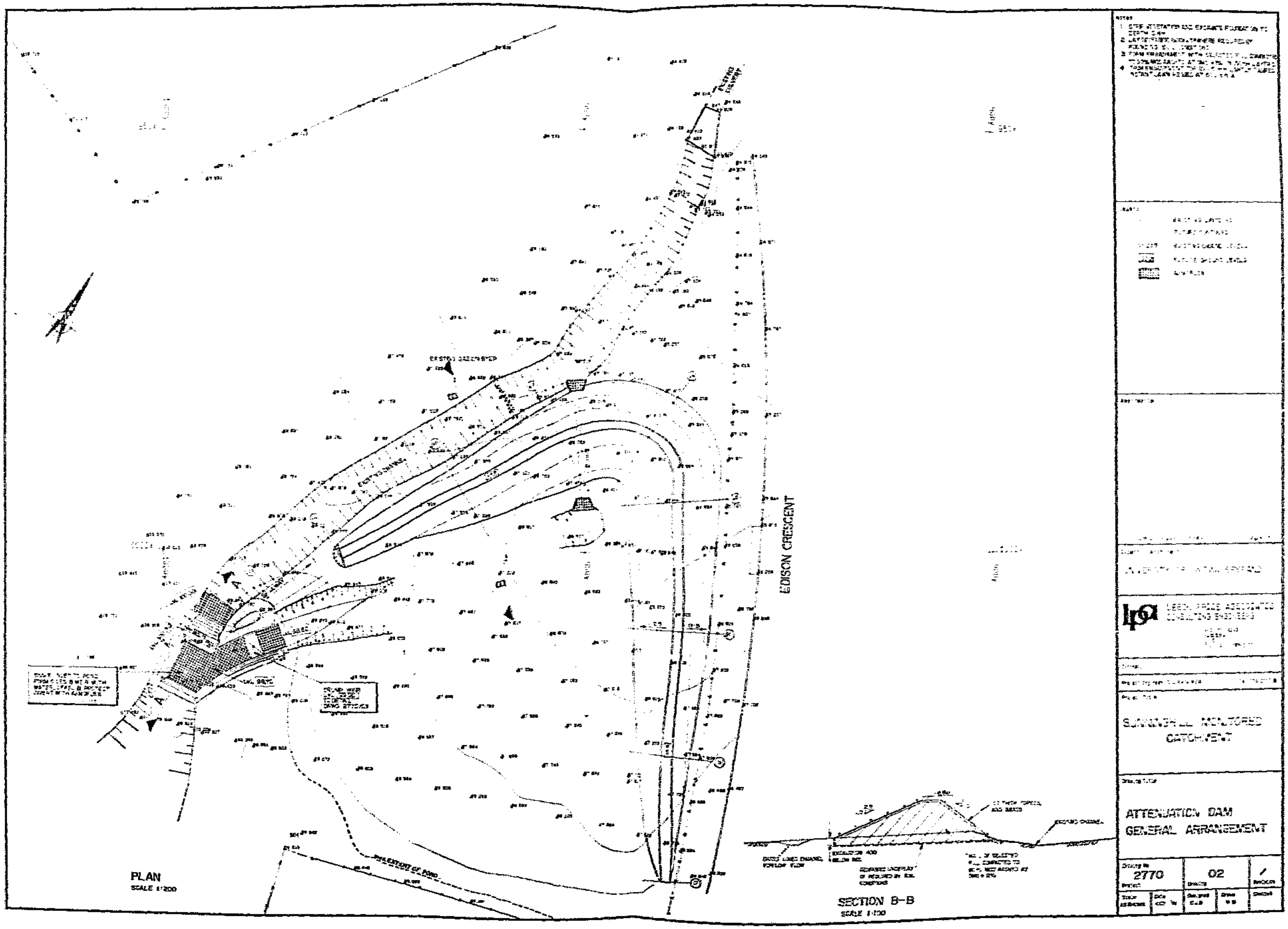


Stage - Discharge



Submergance at y/D = 1.2 Cc = 0.7

Cb = 0.907 Cb = 0.7



PLAN
SCALE 1:200
EDISON CRESCENT
SECTION B–B
SCALE 1:100
ATTENUATION DAM
GENERAL ARRANGEMENT
2770
02

B 1 IMPLEMENTATION

B 1.1. Possible Locations

Several sites exist in the catchment where storage could be provided for stormwater attenuation and of these 2 lent themselves well to monitoring:

The first was immediately upstream of the Edison Cr culvert, at the low point in the catchment, where there was space to construct a pond with capacity of about 1 400 m^3. The catchment area here is about 50 ha and the analysis showed that a reasonable degree of flood attenuation could be attained, the initial design hydrographs are shown on Figures B.1, B.2 and B.3.

The second possible location was at the outlet of the pipe draining Faraday Road into the water course about 400 m upstream of the Edison Cr culvert where the pipe could be discharged into a small onstream pond shown as point "B" on drawing 2770/01. The volume of storage required to achieve a significant reduction in flow would be about 300 m^3 and the system need not be complicated. Monitoring of this pond could be used to compare the direct runoff from efficient drainage system in Edison Crescent to a system with a degree of management. The catchment at B is 6.91 ha while that at C is 8.[illegible] ha and D is 5.79 ha. The land use is similar in all 3 of these sub-catchments so the results of this comparison would be reliable and provide a basis for determining the true effectiveness of minor attenuation facilities.

The first site was considered appropriate for this investigation and the possibility of constructing a small pond at the outlet from the Faraday Road pipe was left for a future project.

Figure A 15 : Event 17 Hydrographs

C 1 APPENDIX C : COMPUTER CODE

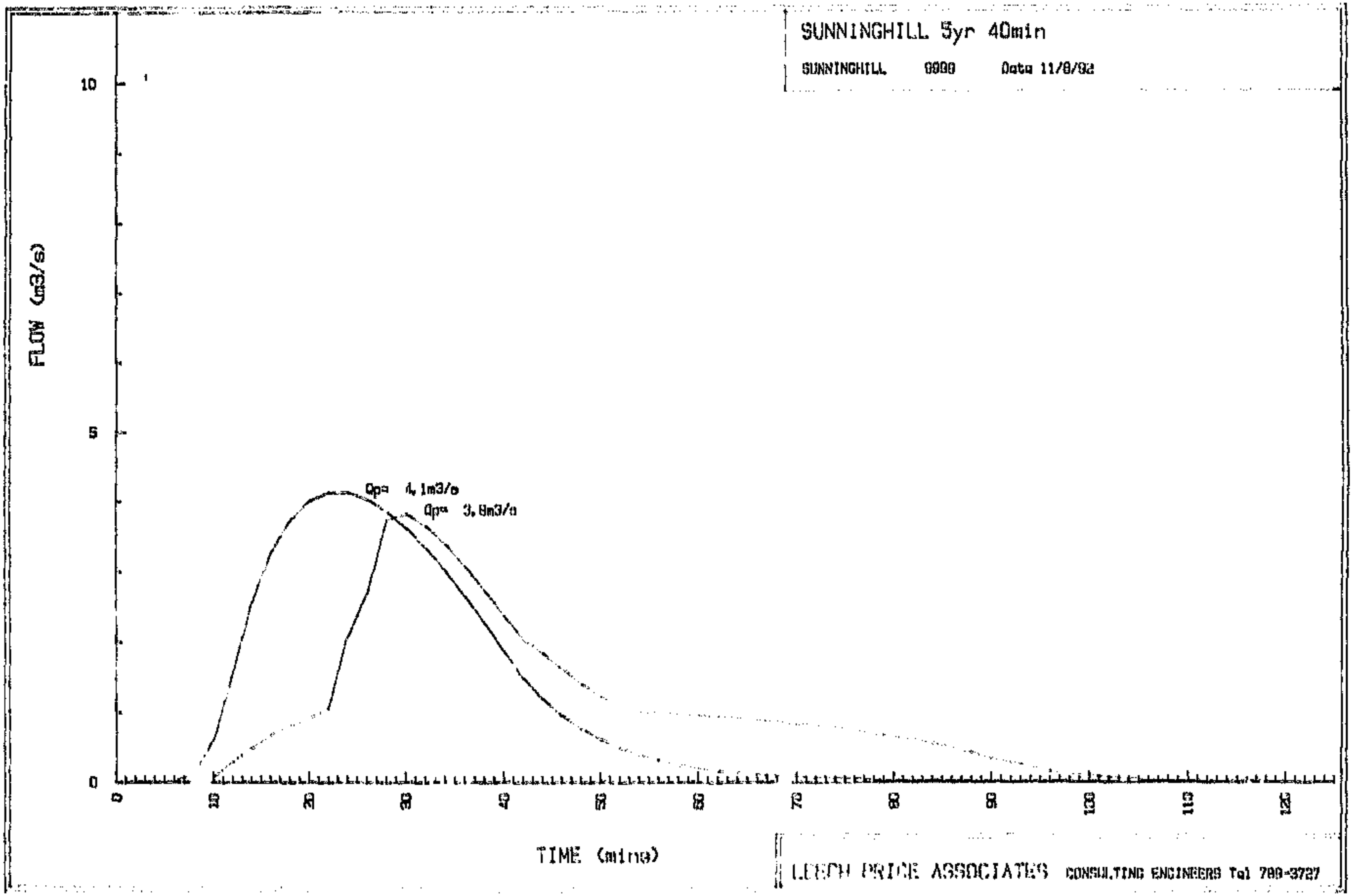
B 1.3. Construction

Tenders received for the construction of the dam and inlet structure ranged between R25 000 and R45 000 for all construction work but not the supply of fill, Armorflex or Envirowall, or for the planting of grass on the embankment surface. Bryan Westcott Construction commenced in construction March 1992 at an estimated total cost of R41 000 Additional costs were incurred. the supply of suitable free fill ran out and it became necessary to buy material and, additional armouring at the inlet was found to be needed.

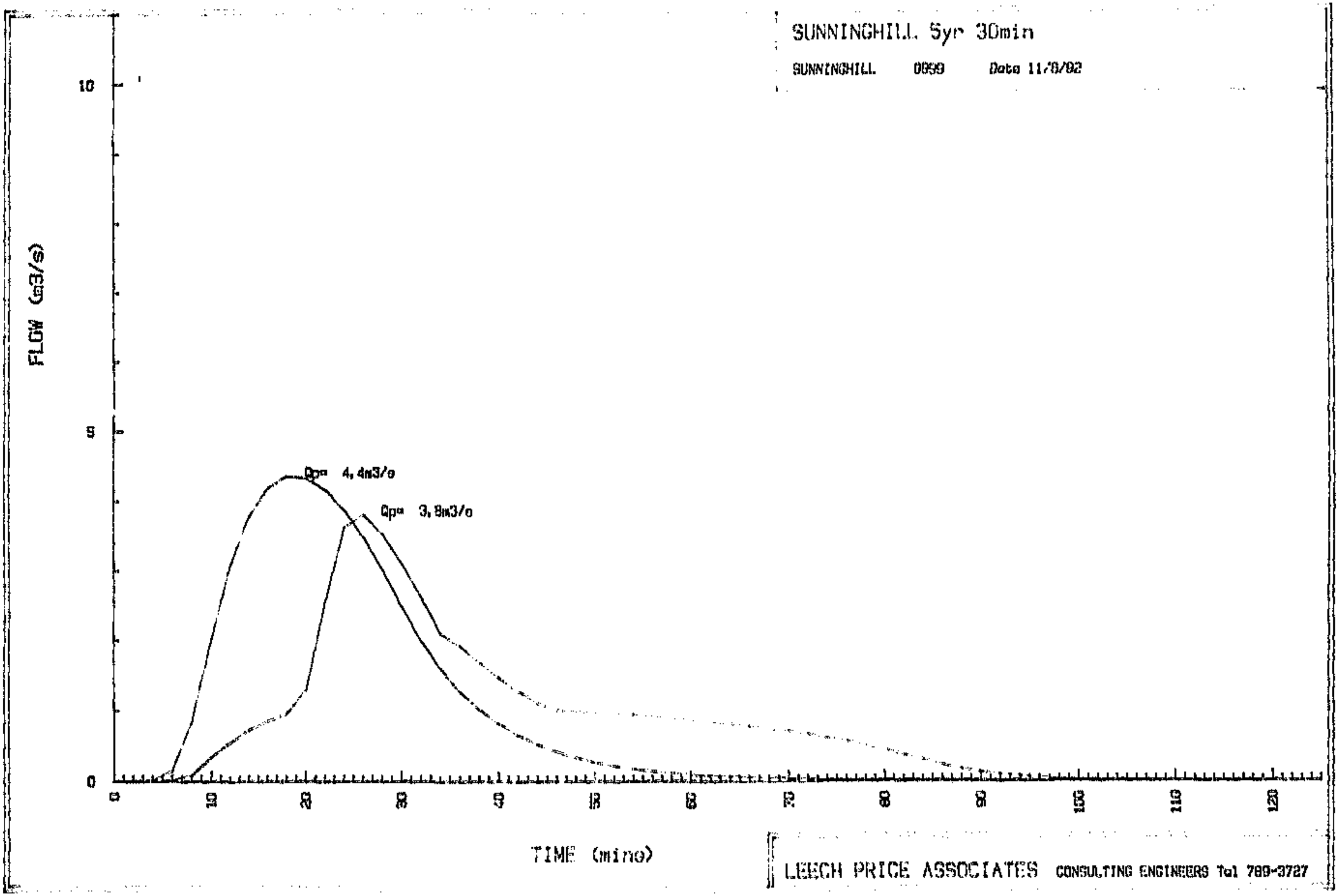
The final cost of construction of the system was

Construction by Bryan Westcott	R25 920
Additional fill and handling	R3 500
Envirowall and Armorflex at inlet	R3 750
Pipes for outlet (donated by Rocla)	R nil
Armouring at inlet by Concor	R9 300
Planting by Sandton Parks Dept	R nil
Sub-total	R42 470
Add VAT at 10%	R4 247
TOTAL COST	R46 720

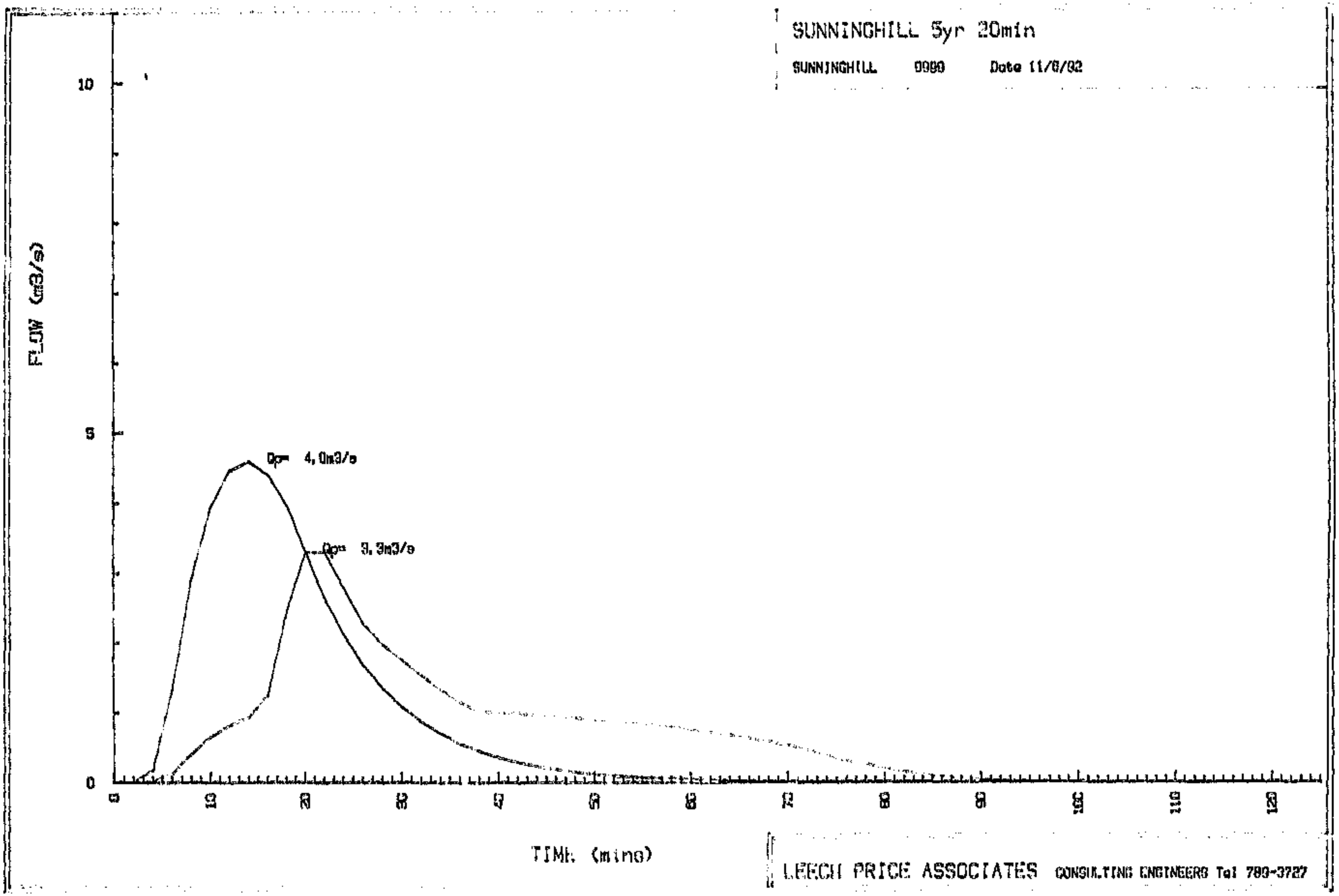
During the construction of the pond certain reservations were expressed by the residents of Sunninghill. In order to address these reservations meetings were held with the Sunninghill Ratepayers Association committee and an explanation of the project published in their news letter.



SUNNINGHILL 5yr 40min
SUNNINGHILL 0000 Date 11/8/92
FLOW (m3/s)
10
5
0
Qp= 4.1m3/s
Qp= 3.8m3/s
0
10
20
30
40
50
60
70
80
90
100
110
120
TIME (mins)
LEECH PRICE ASSOCIATES CONSULTING ENGINEERS Tel 788-3727



SUNNINGHILL 5yr 30min
SUNNINGHILL 0059 Date 11/8/92
FLOW (m3/s)
10
5
0
Qp= 4,4m3/s
Qp= 3,8m3/s
0
10
20
30
40
50
60
70
80
90
100
110
120
TIME (mins)
LEECH PRICE ASSOCIATES CONSULTING ENGINEERS Tel 789-3727


SUNNINGHILL 5yr 20min
SUNNINGHILL
10
5
0
FLOW (m3/s)
Qp= 4.0m3/s
Qp= 3.3m3/s
0
10
20
30
40
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60
70
80
90
100
110
120
TIME (mins)
LEECH PRICE ASSOCIATES CONSULTING ENGINEERS

triangular storm distribution and abstracting losses in the proportion 1-C, where C is the Rational Coefficient. The runoff hydrograph is then computed by Muskin, routing. Hydrographs can be generated by this method quickly and have a realistic shape. Other models could have been used for the design but the facility to generate hydrographs by this method exists in the software suite developed by the author that was used for the routing of the hydrographs through the detention pond. The code of the program used is reproduced in Appendix C.

Reservoir Routing

Routing of the design hydrographs through the storage was done using a program written by the author to run on an HP86 computer. The algorithm used by this program is discussed in detail in Chapter 4 of this report and the code of the program is reproduced in Appendix C

Outlet

The design procedure showed that a single 600 mm diameter pipe outlet allowed efficient use of the storage and that significant reductions in the peak discharge could be achieved. The design hydrographs for the 5 yr. recurrence interval event are shown on Figure B.1, B.2 and B.3. The correct calculation of discharge through an unsubmerged circular orifice is complex, and in the detailed analysis of the measured data use is made of a look-up table. The program used for design approximates the circular pipe to an equivalent rectangular opening, the resulting error is small, and in most cases, is confined to the bottom of the rising and falling limbs of the hydrographs

Inlet weir

The inlet structure was designed to divert all flow up to a peak discharge of 3.0 m^3/s into the pond. For discharges above 3.0 m^3/s the water level in the diversion structure would rise above the bypass crest allowing an increasing proportion of the flow to continue down the old stream bed. The drain under the bypass crest also allows very low flows also continue down the old stream bed.

The diversion weir is a Vee notch Crump type designed in accordance with DWA TR126 (van Heerden et al. 1986). The discharge equation for the weir is:

$$Q = Cd * m * H^{2.5} \qquad \text{for } H \le h'$$

$$Q = Cd * m * \{ H^{2.5} - (H - h')^{2.5} \} \qquad \text{for } H > h'$$

where:

Q	=	discharge	[m^3/s]
Cd	=	coefficient of discharge	
m	=	crest Vee notch slope { B / (2 * h')}	[m/m]
H	=	upstream water depth	[m]
h'	=	Vee notch height	[m]
B	=	total width of crest	

as-built

B	=	2.8 m
h'	=	0.4 m

The discharge coefficient given in TR126 is Cd = 1.585 but the data collected to date shows a better fit with a slightly different value for Cd. This is discussed below.

Inflow Hydrograph

The design inflow hydrographs were calculated using the method of James (1981) which is essentially a derivation of the Rational Method where the excess rain hyetograph is generated by assuming a

B 1.2. Design

In order to obtain a useable range of results reasonably rapidly the storage was constructed online in the form of a blue-green pond with provision for the high flows to by-pass the pond. This type of system does not make the most efficient use of the available volume to attenuate high flood peaks as the pond fills on the rising limb of the hydrograph and may reach its full capacity before the arrival of the peak of the inflow hydrograph. It does, however, provide very effective attenuation of the frequent spate and small flood flows. The relative advantages and disadvantages of online and offline storage are discussed elsewhere in this paper. Figure B.1 is a reduced copy of the General Arrangement drawing showing the layout of the earth emnbankment and the inlet and outlet control structures. A typical cross section through the wall, showing the foundation preparation required to reduce the likelihood of the wall squatting on the poor subgrade, is also shown.

Dam Wall

The size of the dam wall constructed was dictated by a combination of financial and aesthetic considerations as well as the physical limitations of the site. The site selected is not ideal for the provision of storage because it tapers to an acute angle at its lowest point and is relatively steeply sloping, requiring a wall with a maximum height of 4 m to achieve a storage volume of 1500 m^3. There is a residential stand immediately to the west of the site where subsoil drains had been installed to protect the foundations of the house. The level of these drains determined the maximum pond water level as it could not be permitted to submerge the outlets of the drains.

Inlet and outlet characteristics were determined to make efficient use of the available storage. In this respect the need to accumulate data for research purposes conflicted with the best use of the storage to limit downstream flood peak discharges.

Appendix B 2

Figure A14 : Event 16 Hydrographs

Volumes in Dam

Stage - Discharge

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

```
[illegible]
[illegible]
[illegible]
[illegible]
20990 REM distance ... water level above stone and wall
[illegible]
21100 REM distance ... water level between stone and wall
[illegible]
21150 REM    distance ... water level above step 1
[illegible]
21190 REM    distance ... water level above step 2
[illegible]
21350 REM    distance ... water level above step 3
[illegible]
21490 REM    distance ... water level above top of wall
[illegible]
22100 REM distance to horizontal ... water level below stone
[illegible]
22150 REM          distance horizontal ... water level within stone
[illegible]
22190 REM distance to horizontal ... water level above stone and wall
[illegible]
22220 REM distance to horizontal ... water level between stone and wall
[illegible]
22290 REM      distance horizontal ... water level stone 1 to step 2
[illegible]
22340 REM      distance horizontal ... water level step 2 to step 3
[illegible]
22350 REM      distance horizontal ... water level step 3 to top of wall
[illegible]
22390 REM      distance horizontal ... water level above top of wall
[illegible]
22470 REM change ... horizontal ...
22500 REM                              water level below stone and wall
[illegible]
22600 REM                              water level within stone for ...
[illegible]
[illegible]
22650 REM                              water level between stone and wall
[illegible]
22690 REM                              water level between stone and step 1
[illegible]
[illegible]
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```
1650 PRINT [illegible]
1660 PRINT "[illegible]"
1670 FOR I=1 TO [illegible]
1680 PRINT "[illegible]"
1690 [illegible]
1695 PRINT
1700 PRINT "[illegible]"
1705 FOR I=1 TO [illegible]
1710 PRINT "[illegible]"
1720 NEXT I
1730 PRINT [illegible]
1740 PRINT [illegible]
1741 IMAGE [illegible]
1750 PRINT [illegible]
1760 REM [illegible]
1770 IMAGE [illegible]
1780 IF [illegible]
1781 IF [illegible]
1790 IF [illegible]
1800 IF [illegible]
1810 IF [illegible]
1820 PRINT [illegible]
1830 [illegible]
1840 [illegible]
[illegible]
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[illegible]
10 DIM [illegible]
[illegible]
20 [illegible]
30 [illegible]
40 [illegible]
50 [illegible]
60 [illegible]
70 DISP "[illegible]"
80 DISP "[illegible]"
90 DISP "[illegible]"
100 DISP "[illegible]"
110 DISP "[illegible]"
120 DISP "[illegible]"
130 DISP "[illegible]"
140 DISP "[illegible]"
150 DISP "[illegible]"
160 INPUT [illegible]
170 DISP "[illegible]";
180 INPUT [illegible]
190 DISP "[illegible]";
200 INPUT [illegible]
210 DISP "[illegible]";
220 INPUT ANS$
230 IF ANS$[1,1]="Y" THEN [illegible] GOTO [illegible]
240 IF ANS$[1,1]="N" THEN [illegible] GOTO [illegible]
250 [illegible] GOTO [illegible]
260 CLEAR @ DISP "[illegible]" @ [illegible]
270 DISP "[illegible]";
280 INPUT ANS$
290 IF ANS$[1,1]="N" THEN [illegible]
300 IF ANS$[1,1]="Y" THEN PRINT [illegible] @ GOTO [illegible] ELSE BEEP @ DISP "WHAT ?" @ GOTO [illegible]
310 PRINT "      [illegible]"
320 PRINT USING [illegible]
330 IMAGE [illegible]
340 PRINT USING [illegible]
350 IMAGE [illegible]
360 PRINT "      [illegible]"
[illegible]
```

[illegible]

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[illegible]
10410 '
10420 '
10430 [illegible]
10440 [illegible]
10450 [illegible]
10460 RETURN
10470 '
10480 '
10490 '
10500 [illegible]
10510 '
10520 '
10530 '
10540 PRINT " This routine [illegible]"
10550 PRINT " through a catchment [illegible] method and"
10560 PRINT " [illegible]"
10570 PRINT " [illegible]";
10580 INPUT [illegible]
10590 IF [illegible]
[illegible]
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[illegible]

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[illegible]
2540 IF MAN$="900" THEN [illegible]
2550 PRINT USING 2530 ; [illegible]
2560 '
2570 NEXT I
2580 DISP " [illegible] ";
2590 INPUT ANS$
2600 IF ANS$="[illegible]" THEN [illegible]
2610 IF ANS$[1,1]="N" THEN [illegible]
2620 IF ANS$[1,1]="Y" THEN [illegible] ELSE DISP "[illegible]" @ GOTO 25[illegible]
2630 RETURN
2640 '
2650 '
2660 '                    [illegible]
2670 [illegible]
2680 '
2690 '
2700 '
2710 IF [illegible] THEN [illegible]
[illegible]
```

```
6720 RETURN
6730 !
6740 !
6750 !
6760 SUB [illegible]
6770 !
6780 !
6790 !
6800 DISP "[illegible]"
6810 INPUT [illegible]
6820 DISP "[illegible]"
6830 GOSUB [illegible]
6840 IF [illegible]
6850 FOR I=1 TO [illegible]
6860 [illegible]
6870 NEXT I
6880 GOTO [illegible]
6890 FOR I=1 TO [illegible]
6900 ON ERROR GOTO [illegible]
6910 [illegible]
6920 NEXT I
6930 OFF ERROR
6940 DISP USING 6950 ; [illegible]
6950 IMAGE [illegible]
6960 IF PRINTER "NO" THEN [illegible]
6970 PRINT USING 6950 ; [illegible]
6980 FOR I=1 TO [illegible]
6990 DISP USING 7000 ; [illegible]
7000 IMAGE [illegible]
7010 IF PRINTER "NO" THEN [illegible]
7020 PRINT USING 7000 ; [illegible]
7030 [illegible]
7040 NEXT I
7050 RETURN
7060 !
7070 !
7080 !
7090 SUB [illegible]
7100 !
7110 !
7120 !
7130 DISP "[illegible]"
7140 GOSUB [illegible]
7150 DISP "[illegible]"
7160 INPUT [illegible]
7170 IF [illegible]
7180 IF [illegible]
7190 IF [illegible]
7200 FOR [illegible]
7210 [illegible]
7220 [illegible]
7230 NEXT I
7240 [illegible]
7250 DISP "[illegible]"
7260 GOSUB [illegible]
7270 IF [illegible]
7280 DISP "[illegible]"
7290 [illegible]
7300 [illegible]
7310 [illegible]
```