CHAPTER 4

CHAPTER 4

DESIGN STANDARDS

4.1 INTRODUCTION

The characteristics of stormwater runoff (discharge, volume, duration, pollutant concentration, temporal pattern, etc) are highly variable and structural stormwater management measures are required to operate over a wide range of discharges. As stormwater runoff generation is dependent on a number of probabilistic factors, particularly catchment antecedent conditions and meteorological factors, a probabilistic approach to selecting design standards will need to be adopted. The design event is a probabilistic or statistical estimate based on a statistical analysis of the likely recurrence of a rainfall or flow event. If a design rainfall is used in the estimation of the design flood event, it is not always necessary that the amount of rainfall occurring at a given time will result in the estimated flow magnitude. Occurrence of the rainfall when the catchment was wet might result in a higher flow than the design flow and the converse if the catchment was dry. The very nature of a probabilistic approach in selecting design events suggests that above-design conditions will occur and good design practices will need to include evaluation of the performance of these structures under these conditions.

The appropriate selection of design standards for these structures will depend on a number of factors and required operating criteria. An economic-risk approach is one means of achieving a balance between the economic consequence of above-design operation and the cost associated with provision of a higher design standard. Other methods could include the adoption of a more subjective means of prioritisation of performance criteria. Typically, considerations which need to be taken into account in selecting the appropriate design standard for hydraulic structures include the operation of the structure during above-design events, the consequence (often in terms of public safety, social well-being, disruption to ordinary living and services, flood damages and environmental impacts) of above-design events, and the frequency of above-design occurrences.

Implicit in the adoption of a probabilistic approach to selecting design standard is the concept of designing a structure for a given risk of "failure", or perhaps more appropriately, risk of above-design operating conditions. The terms "recurrence interval" and "return period" are commonly used in water engineering practices but often in a manner which can potentially be misleading, particularly in regards to the appreciation of the inherent random characteristics of rainfall and runoff events. This is particularly a problem when disseminating information to the public and decision-makers. A common mis-conception is the implication that these probabilistic events are exceeded at regular intervals as defined by the "return period" or "recurrence interval".

Two more acceptable probability terms are "average recurrence interval" (ARI) and "annual exceedence probability" (AEP). The term average has been added in the former to reflect the notion that the recurrence interval of a particular sized rainfall or runoff event is, in the long-term, equal to the average recurrence interval so specified. The term AEP is perhaps the more technically correct term in that it represents the probability of rainfall or runoff events exceeding the design value at least once each year and that this probability remains unchanged throughout the life of the structure.

4.2 RISK-BASED APPROACHES TO SETTING DESIGN STANDARDS

4.2.1 General

There are a number of risk-based approaches currently used in practice to select the appropriate design standard of hydraulic structures. When a large decision making authority has many structures (such as stormwater management measures including underground pipes, gross pollutant traps, oil and grease traps, wetlands, retarding basins etc), the consequences of the "failure "of any one structure will often be small when compared to the authority's overall operations. When these structures are spread over diverse geographic regions the probability of simultaneous "failures" will be very small, thus the authority may use the "expected monetary value criterion". Such situations are sometimes referred to as those which have spread risk.

On the other hand, if the economic consequences of design flow exceedance are large compared with the scale of operations of the decision-making authority, then the selection of the design discharge can best be made with the use of "a modified expected monetary value criterion". Such circumstances occur, for instance, when an authority has an expensive asset such as a dam or a lake (in the case of the Putrajaya Corporation) which is apt to "fail" structurally by overtopping when its spillway capacity is exceeded or "fail" ecologically when excessive contaminants are discharged into the lake. Such situations are sometimes referred to as "having isolated risk". The use of an expected monetary value criterion in these situations must recognise the aversion of the decision-making authority to the risk of incurring large monetary losses and takes this into account when making the design flood selection decision.

4.2.2 Procedure for Expected Monetary Value Criterion

The appropriate expected monetary value criterion for choosing the discharge capacity of a hydraulic structure, once the decision to proceed with its construction has been made, is often to minimise the total discounted present worth of the expected costs. This is equivalent to making the total expected annual costs a minimum. The term "expected" is used in a statistical sense and means that the expected value of any cost or benefit is its weighted mean or weighted average value. The weights used to calculate the mean are the probabilities that the cost will assume different values during the life of the project. The total expected costs include the capital cost of the structure, its annual operation and maintenance costs and the economic costs of all exceedences.

In the case of stormwater drainage, the economic cost of exceedence of the design standard of the minor drainage system may be computed by assigning monetary values to inconveniences and flood damaged caused by the above-design events. Multiplying the costs associated with each probabilistic event with their respective annual exceedence probabilities then annualises these costs, ie.

$$\text{Annual Cost} = \sum C \cdot Pr \qquad - \qquad 4.1$$

where C is the flood damage and inconvenience cost associated with the occurrence of the event ;

Pr is the annual exceedence probability of the event

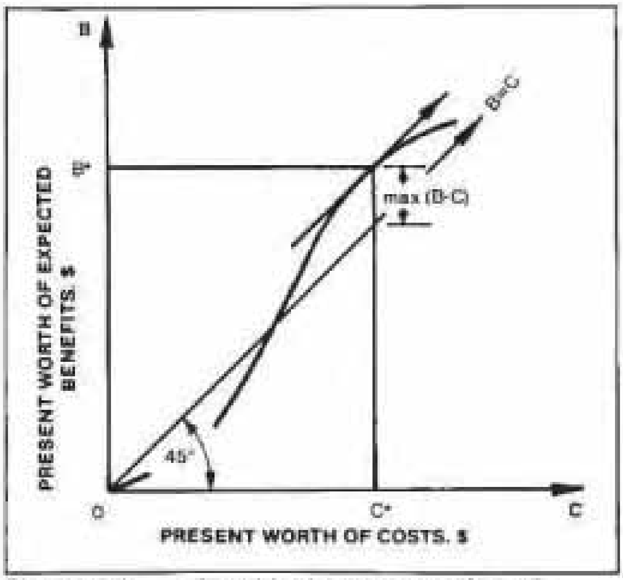

Figure 4.1 Graphical representation of economic criterion (ref. Inst. of Engrs, Aust., 1987)

The computation of flood damage and inconvenience costs is complicated in when other considerations such as water quality and ecological health of downstream aquatic ecosystems are included. It may be necessary to use other techniques such as a multiple objective approach under such circumstances. Discussions on multiple objective selection of design standards are contained in Section 4.2.4.

When using the expected monetary value criterion in selecting the design standard of a stormwater minor drainage system, the optimal design standard is that where the expected benefits to be derived from increasing the discharge capacity of the drainage system by a small increment is equal to the additional cost of provided that increment of increased discharge capacity. In other words, the best discharge capacity is that capacity at which the marginal expected benefit-cost ratio is equal to one or (1 +λ) when there are budgetary constraints, where λ is a small positive value.

If the relationship between benefits and costs for a project could be described by the cost-benefit curve shown in Figure 4.1, the best capacity would be that costing C* dollars (excluding costs of flood exceedences and counted to a present worth, if necessary), corresponding to the point (C*, B*) on the curve, where it can be see the tangent to the curve has a slope of one. Here the benefits are expected reductions in the costs of exceedences from those incurred by providing zero capacity. This means that, if the discount rate represents the opportunity cost of using capital on other projects or structures, the last dollar spent on the structure under consideration earns a rate of return at least equal to the rate of return obtainable on the other projects or structures.

Alternatively, the criterion may be considered as the discharge capacity of the drainage system at which, if the capacity were increased by the expenditure of one more dollar, the saving in the present worth of expected costs arising from exceedences would equal just one dollar. In each of these formulations both expected benefits and costs must be measured in the same units, either present worths or average annual values (Grant et al., 1982).

Consideration of these criteria will show that the two forms are equivalent, that is to say, the design at which the marginal benefit-cost ratio is unity, is the same as the design at which minimises total expected cost including costs of the capacity being exceeded. In the situation being considered, benefits are reductions in the costs incurred through the discharge capacity of the hydraulic structure being exceeded.

Using the expected monetary value criterion which the design discharge capacity of the structure should be such that the total expected costs are a minimum, then the total expected costs are made up of all or some following costs:

a) the capital cost of the structure;
b) the ordinary operation and maintenance costs of the structure;
c) the expected costs incurred by the occurrence of above-design events including the following:

    i. expected costs of injury and loss of human life;

    ii. expected repair costs; for example, scour at abutments and piers;

    iii. expected costs to replace the structure if it were destroyed;

    iv. expected costs incurred by interruption of service; for example, interruption of traffic flow;

    v. expected costs of accidents resulting from capacity exceedence; for example, cars being swept off culverts;

    vi. expected external costs which are costs not directly associated with the structure under consideration but which result from its performance; for example, pollution of receiving waters etc.

All of these costs must be expressed in the same units, either as a present worth at the same time datum or as an equivalent uniform annual cost. An appropriate discount rate must be chosen for both procedures together with an appropriate design life for the structure (Grant et al., 1982). Care should be taken not to double-count costs.

The procedure for choosing the design discharge will ordinarily require the estimation of the total expected costs for a number of different discharges and selecting the discharge which gives the lowest total expected costs. Interpolation procedures should be used where necessary.

Estimates of all costs associated with events, which exceed the capacity of the structure, must be made for each trial selection of design standard. The amount of effort that is devoted to the preparation of these estimates will depend on the importance of the structure being designed and the consequences of its "failure".

4.2.3 Procedure for Modified ExpectedMonetary Value Criterion

The procedure for this form of the problem is basically the same as with the expected monetary value criterion but with the following modification. First, the economic analysis is used to determine the optimal configuration of the hydraulic structure or series of hydraulic structures. Then, the decision maker may choose to modify this configuration in terms of the marginal or incremental costs and benefits involved and the authority's own perception of their decision environment and any other objectives they may have but are not explicitly expressed in the analysis.

This approach is, in essence, a simplification of the multi-objective approach discussed below. The form of the modification will depend on the nature of the problem.

4.2.4 Multiple Objective Selection of Design Standards

The determination of design capacity by minimising expected monetary costs, as described above, does no take into account desires of the decision-maker which are difficult to quantify in economic terms. Examples include the reduction of the risk of injury to members of the public, reduction in loss of human life, public environmental perception and expectations, etc. A multiple objective approach can be taken when dealing with these types of problem. The same approach can be extended to less quantitative assessments of "benefits" and "costs".

The utilisation of a multiple objective basis for selecting design standards can be illustrated as follows (Inst. of Engrs., Aust., 1987) :-

Suppose that the determination of the design capacity of a hydraulic structure is to be made with respect to an "economic" and to a "well-being" objective. Let the criterion for the economic objective be the minimisation of the expected annual monetary value of costs and let the criterion for the well-being objective be the minimisation of the expected annual number of lives lost. The procedure is then to evaluate the two criteria for a number of different flood sizes (or design capacities). The values calculated could be plotted as shown in Figure 4.2. While the best choice in terms of the economic objective may be the capacity which corresponds to point A with capital cost K_4, the decision maker after inspection of the calculated results may decide to choose the capacity which corresponds to point B with capital cost K_6. This shows that the designer is willing to pay the additional expect annual costs ΔC and additional capital cost (K_6 - K_4) to save the expected number of lives ΔL. Thus the designer imputes a value to human life in this particular instance.

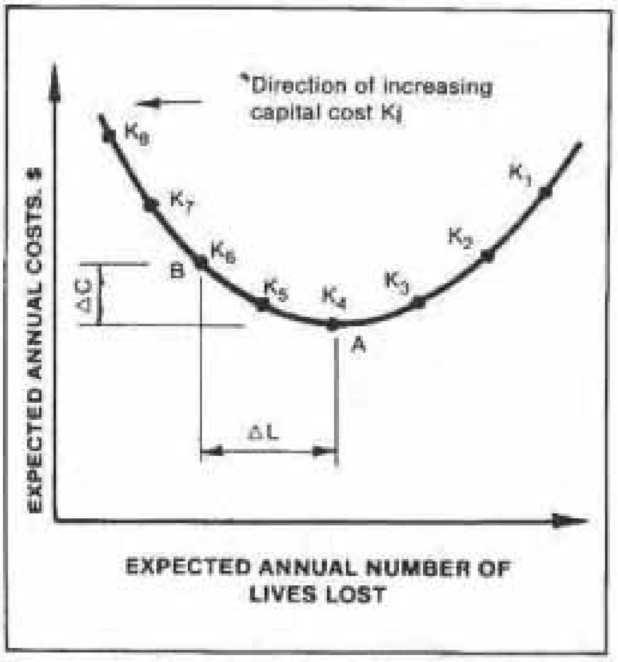

Figure 4.2 Conceptual relationship between capital cost of project, expected annual costs and expected annual number of lives lost (ref. Inst. of Engrs., Aust, 1987)

A similar approach in relation to water quality criteria may be adopted. In essence, an economic analysis is undertaken to provide the designer with an insight on the expected cost of the structure and the sensitivity of the cost to variation of subsequent criteria such as water quality, ecological health etc. An informed decision can then be made, often through negotiation with stakeholders on the appropriate departure from an economically optimum standard to achieve the goals of the other objectives.

4.3 JUDGEMENT BY EXPERIENCE

It might be necessary to select the magnitude or average recurrence interval of the design flood simply on the basis of a judgement. While this procedure is certainly one of last resort, it should be seen to be a valid procedure when used by engineers with sound experience in engineering hydrology, engineering economics and project costing. For situations involving a judgement, a sensitivity analysis for the design flood selection should always be made and the basis of selection clearly stated.

In the preparation of guidelines for flood estimation in Australian catchments, some 250 representative engineering bodies from all interests in the profession were surveyed by the Institution of Engineers, Australia (1983) for their choice of design standards for a range of

Table 4.1
Relative Frequency of Response (%) to survey of choice of design standards (ref. Inst. of Engrs., Aust., 1987)

Use Category	Percentage of Respondents — Design Average Recurrence Intervals (years)																	
	1	2	3	5	10	15	20	25	30	50	100	200	500	1000	5000	10000	PMF	Range*
Dams																		
Major Spillways											5.3		2.6	7.9		10.5	73.7	
Medium Spillways										5.4	10.8	5.4	2.7	5.4	2.7	8.1	59.5	
Minor Spillways				2.3				2.3		9.3	41.9	2.3		16.3		2.3	7.0	16.3
Flood Mitigation																		
Floodways						2.4	23.8			14.3	54.8						2.4	2.4
Flood Fringes				2.6			5.1			7.7	82.1							2.6
Bridges																		
Major Bridges & Culverts								1.4		23.9	60.6						1.4	12.7
Medium Bridges & Culverts					10.8		12.3	1.5	1.6	36.9	23.1							13.8
Minor Bridges & Culverts	1.4		1.4	13.9	20.8	1.4	26.4		1.4	9.7	2.8							20.8
Road Works																		
Rural & Catch Drains	3.5	3.5	3.5	29.8	15.8	1.8	7.0		1.8		8.8							24.6
Culverts with Causeways	10.8	5.4	21.6		10.8		18.9				2.7							29.7
Causeways with Culverts		5.4		8.1	13.5		10.8		2.7	10.8	24.3							24.3
Culverts without Causeways	3.7	3.7		11.1	14.8		14.8		3.7	7.4	18.5							22.2
Urban																		
Intense Business	1.2		3.7		8.8		27.5		3.7	30.0	25.0							
Business & Residential			1.2	12.0	28.9	1.2	20.5	1.2	8.4	12.0	14.5							
Other Residential	1.2	4.7	2.4	49.4	17.6		9.4			5.9	9.4							

*A range of values was given as opposed to a single value

hydraulic structures. The result of this is summarised in Table 4.1. As discussed in the publication of this result, IEAust (1987) points out that the survey and its summary reflect practice at the time the information was gathered and that the results of the survey should not be considered to have any intrinsic correctness. Nevertheless the results serve to provide a useful starting point for future analysis. Many of the contributing authorities did not consider a simple ARI event on its own but rather assessed the impact of a range of values.

4.4 URBAN STORMWATER SYSTEMS

4.4.1 General

The selection of the appropriate design standard for stormwater management structures need to consider a number of factors including the following:-

- the level of hydraulic performance required;
- construction and operating costs;
- maintenance requirements;
- safety;
- aesthetics;
- regional planning goals; and
- legal and statutory requirements.

In the past, hydraulic stormwater structures have often been design to meet one design criterion. The Institution of Engineers, Australia (1987) has recommended that stormwater management systems should ideally be designed for several performance levels, which may include:

- a maintenance requirement, (frequent event), related to a short design ARI, perhaps less than one year,

- a convenience or nuisance-reduction requirement, (infrequent event), possibly 1 to 10 years ARI;
- a flood damage prevention requirement, (severe or rare event), of about 100 years ARI; and
- a disaster management requirement, (extreme event), related to extreme events such as probable maximum floods.

The first two or three are relevant to street drainage, and all but the second to trunk drains.

In stormwater drainage network the capacities of the individual components are often different even though they may be designed for a single design ARI event. This is due to the use of standard pipe sizes and the common practice of ensuring that, along a given pipeline, the downstream pipes sizes are not to be smaller than upstream pipes leading to different amounts of excess capacity. This is especially true in the upper reaches of a piped drainage system where minimum pipe diameter criterion dominates. In these circumstances, a design ARI is mainly a means of setting a standard, rather than a complete description of system capacity. Selecting a single ARI design standard for a stormwater drainage network will often not result in all pipes reaching their discharge capacity in the same event owing to different sub-catchment sizes and critical storm durations.

The most commonly used design ARI values for street drainage systems are:

- 20 or 50 years for intensely-developed business, commercial and industrial areas;
- 10 years for other business, commercial and industrial areas and intensely-developed residential areas; and
- 5 years for other residential areas and open spaces.

A 100 year ARI criterion for administrative definition of flood-prone areas has been adopted by several governments, and the same standard has been widely accepted for channel capacities and detention basin performance for the design of the major drainage system. In recent years there has been debate over standards, particularly where different organisations are responsible for building and for maintaining works. A report published by the Australian Department of Housing and Construction (Scott and Furphy Consulting Group in association with Coopers & Lybrand Services, 1984) recommends standards based on performance, allowing flexibility in ways of achieving stated objectives and goals. Minimum prescribed standards for stormwater drainage are presented in this report, together with standards relating to roads, water supplies and other amenities. These were compared with higher levels of service in an analysis of comparative costs. It was concluded that minimum standards (including protection from major storm flooding) represented the most cost-effective level for provision of services. Many drainage authorities would disagree with this, supporting the higher standards currently in force because of their general acceptance by the public.

Adoption of the major/minor philosophy of street drainage described in Chapter 2 is likely to reduce design ARIs for pipe systems. Since specific allowances are made for overflows as part of the major drainage system, pipes can be designed on convenience requirements. It will not be necessary to provide extra pipe capacity as insurance against problems.

4.4.2 Selection of Average Recurrence Intervals

The selection of a suitable design ARI must be made with due considerations of local conditions and requirements. It may be appropriate for designers to vary the standards applied to different points in a drainage system, depending on the perceived risks of failure. For example, a low design might be applied at the upper end of a street drainage system, where overflows will cause little nuisance or damage. At a point further down, where flows from several drainage lines converge, a higher design ARI might be appropriate.

Benefit-cost analysis is recognised as the best way of selecting a design standard although it is often difficult to quantify the benefits associated with nuisance abatement and social well-being as they apply to street drainage. Grigg et al. (1976) assert that the difference between minor and major (or trunk) drainage systems is essentially between convenience and damage prevention systems.

Estimation of costs of alternative designs is very tedious if hand calculations are used, but is easily done in computer design procedures, which allow ample scope for sensitivity studies. In general, drainage system costs increase by roughly 10% for each doubling of design ARI, but for particular designs there are varying amounts of change. For example, increasing a standard from 5 to 10 year ARI may not result in any change in pipe sizes or costs in one case, but may involve large cost increases in another.

When considering costs, it is important not to omit maintenance costs, even though base information may be lacking. Possible trade-offs between construction and maintenance costs should be explored.

If comparative cost information is available for alternative designs, the procedure recommended in a U.K. National Water Council (1981) publication is appropriate. This involves comparison of costs with some "standard of performance" for each level of design. This standard can be characterised by flowrates at critical locations, anticipated damage or nuisance from overflows, or the frequency of occurrence of a particular nuisance.

For trunk drainage systems the emphasis shifts from convenience to prevention of flood damages. Benefits are easier to quantify as they now relate to flood damages. Regardless of the economic outcome, it may be necessary to select a design ARI which conforms to community standards, particularly those pertaining to flood plain management.

In addition to the ARI used for design, the performance of larger trunk drainage systems should be evaluated for extreme events up to the probable maximum floods. This is to ensure that systems will fail in a predictable and relatively safe manner in such events, although significant damages should be expected.

Standards may change. When a revised set of standards is introduced, they can be readily applied to new works after a transition period. However, there may be anomalies in connections to existing works built to older standards, and many existing drains may have insufficient capacities according to the new criteria. It is clearly not feasible to upgrade existing works in the short-term. The new standards should be taken as an objective to be pursued in long-term (say 20 to 50 year) renovation programmes.

4.4.3 Determination of Subsidiary Standards

In addition to standards which define the overall capacity of a drainage system, there are subsidiary standards relating to detailed aspects, for instance:

- the particular methods of design or analysis to be employed under different circumstances;
- minimum and maximum velocities and depths in pipes or channels;

- minimum allowable side-slopes and freeboards for open channels; and
- appropriate safety measures.

Some of these may relate to system operation at some given ARI (such as velocities and freeboards), while others are more general (such as sideslopes or scour protection).

It should be noted that there is a danger that excessive subsidiary standards, when applied rigidly and without understanding of the full circumstances, can lead to sub-optimal design. The designer should not overlook the overall performance objectives of the stormwater management system and that these objectives can often be achieved by a number of alternatives involving a combination of structural and non-structural management measures. For example, an overall performance goal for a street drainage system might be "to convey stormwater from streets and adjoining properties for storms up to a 2 year ARI without nuisance, and from storms up to a 100 year ARI without flooding of properties or other serious damage". Various combinations of underground pipes and surface drainage can achieve this performance objective. Allowable depths and velocities of surface flows over road surfaces can then be determined, keeping in mind the overall objective.

4.4.4 Design Standards for Stormwater Quality Treatment Measures

The selection of the appropriate design standard for stormwater quality treatment measures is by necessity different from that of stormwater drainage structures. Performance assessment of stormwater quality control measures involves consideration of the long-term, cumulative effects of stormwater pollution abatement. This involves computation of the effectiveness of these measures in the reduction of pollutant load (ie. the product of concentration and discharge) transported to receiving waters in addition to consideration of pollutant concentrations reduction.

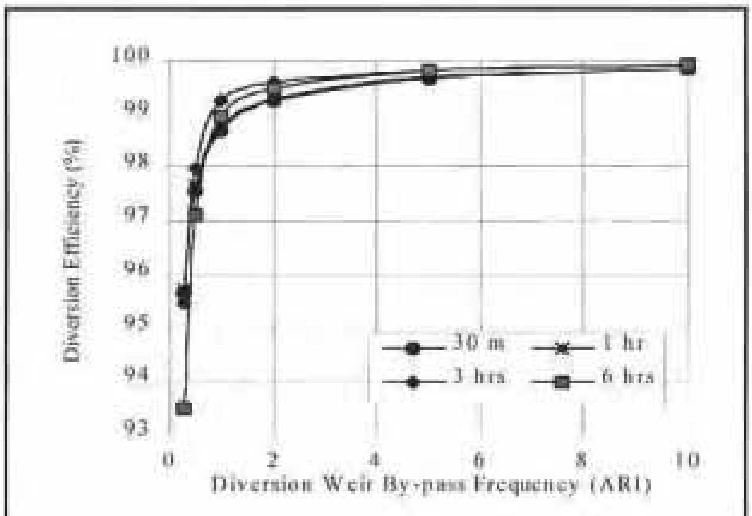

Figure 4.3 Trap Efficiencies Vs Design Bypass Frequency (ref. Wong et al., 1996)

In designing stormwater quality control measures, the emphasis is no longer on the efficient and rapid transfer of stormwater to the receiving waters. Instead, stormwater interception, detention/retardation and retention are the principal primary objectives and these mechanisms of stormwater treatment required excessively large and expensive structures if the appropriate design standards are not selected. The concept of treating the first flush is commonly adopted in practice to achieve a high level of cost effectiveness of the treatment measure. This can be justified by the fact that the majority of storm events with the potential to mobilise and transport urban pollutants to receiving waters are events of relatively low rainfall intensity. The selection of the design event for which the first flush is to be treated is highly varied in practice, ranging from 1 year ARI to 100 year ARI. This is often a reflection of inadequate appreciation of the relationship between probabilistic events and the volume of stormwater runoff generated by the large number of storm events with magnitudes less than the design probabilistic event. This relationship is highlighted by Wong et al. (1996) when they presented the overall percentage of the expected volume of the annual stormwater runoff treated by a gross pollutant trap against the design standard of the trap as shown in Figure 4.3.

In that investigation, Wong et al. (1996) undertook a continuous simulation study using 100 years of rainfall data for Melbourne to establish the relationship between volumetric treatment efficiency and the frequency at which the design discharge was exceeded. The volumetric treatment efficiency was defined as the overall expected volume of runoff (expressed as a percentage of the total expected runoff volume) which is conveyed into the gross pollutant trap at a rate that is lower than the design discharge of the trap. Simulations were carried out for catchments with critical storm durations of 0.5, 1, 3 and 6 hours. The results for each of these cases were found to be similar in that in excess of 93% of the expected annual runoff volume will be treated by the device designed for a 0.25 years ARI peak discharge. The corresponding volumetric treatment efficiency for a device designed for a 1 year ARI peak discharge was approximately 99%. The results are applicable for any type of hydraulic structures and clearly demonstrate that the design standard of these structures need not be set excessively high to gain significant benefits in the overall proportion of stormwater treated.

Similar curves were derived for major capital cities in Australia by Wong (1998) and are presented in Figure 4.4. The curves were derived for the case of a catchment with a time of concentration of 1 hr. The plot shows that all the capital cities considered tend to follow a similar relationship in which in excess of 99% volumetric treatment efficiency can be achieved by adopting a design standard of 1 year ARI. The results appear to be applicable across the various climatic regions of Australia, from the tropical region represented by the city of Darwin, to the temperate region represented by the city of Perth. It is not unreasonable to suggest that the relationship will provide sufficient guidance on the expected volumetric treatment efficiency for Malaysian climatic conditions.

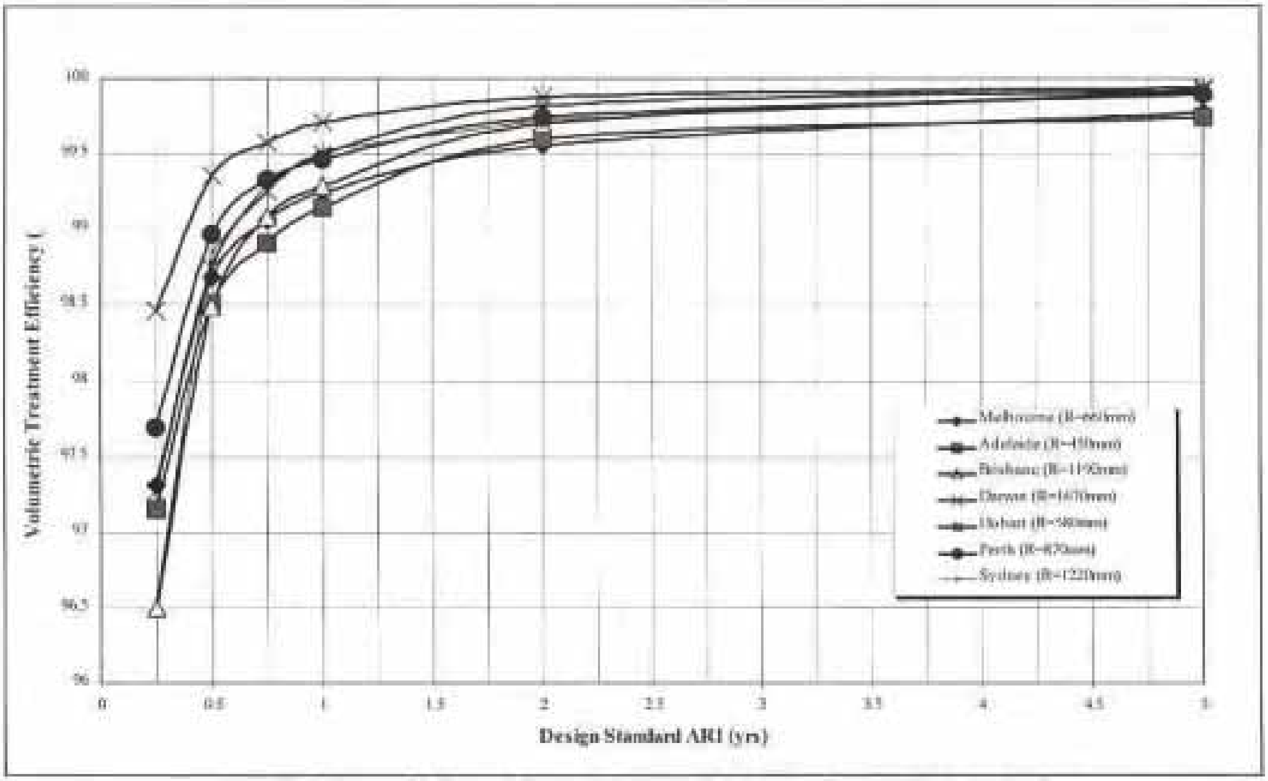

Figure 4.4 Volumetric Treatment Efficiency (Australian Cities)

With regards to stormwater quality, setting water quality targets depends on the ambient water quality standard and is therefore rather subjective. Common practice involves setting the following objectives:-

- Suspended Solids – 80% reduction of the typical urban load
- TN and TP – 45% reduction of the typical urban annual load
- Litter – 70% reduction of typical urban load larger than 5 mm; 100% removal of solids larger than 50 mm.

4.5 DESIGN STANDARDS FOR THE PUTRAJAYA PROJECT

4.5.1 Stormwater Drainage System

AGHD (1996) outlined the adopted design standard for the stormwater drainage system in the Putrajaya project as follows:-

Minor System

The minor stormwater drainage system in the Government and Central Business District is to be designed for the 100 year ARI event. The corresponding design standard for the Residential District is the 5 year ARI event.

Minor stormwater drainage systems in all non-green belt areas are to consist of underground pipes which are to be designed for the 100 year ARI event.

On-site detention and retention systems are to be designed to attenuate or infiltrate stormwater inflow such that the outflow from these systems for the 1 year ARI event is that under pre-development conditions.

Major System

The major drainage system (ie. designated overland flow paths, floodways, detention basin, etc) are to be design for the 100 year ARI event in all precincts.

4.5.2 Stormwater Quality Control Measures

Water quality objectives for the Putrajaya project are listed in Section 4.5.3. Gross pollutant and litter load of 50 mm or higher are to be removed entirely from the receiving waters. Gross pollutant traps are to be designed for the 1 year ARI Peak flow.

Prevention of oil discharge to the receiving due to spills and poor catchment practices during dry weather conditions. High source areas are to be identified and isolated and oil, grease and grit traps installed to serve these areas. These traps should have the capacity to store accidental oil spills typically expected and is to be designed for discharges up to the 1 year ARI peak discharge. Peak discharges as low as the 0.25 year ARI flow are considered acceptable in circumstances where the catchment area served is large (greater than 1 ha).

4.5.3 Putrajaya Lake Water Quality Guideline

Water quality criteria need to be formulated as part of an effort to develop a catchment management plan to protect and enhance the water quality in the Putrajaya Lake (Quek, 1998). Due to the recreational and aesthetics values associated with the Putrajaya Lake and the delicate nature of the lake and wetlands ecosystem, the proposed water quality criteria must meet the dual objectives of:

1. Recreational and aesthetics requirements, and
2. Protection of lake aquatic ecosystem.

The above criteria are defined by a set of physical, chemical, microbial and biological indicators of water quality. For management purposes, these criteria can be translated into water quality guidelines or water quality standards when these are legally enforced.

Recreational and Aesthetics Requirements
The recreational and aesthetics requirements can be divided into three categories as proposed by ANZECC (1992) in the publication entitled "*Australian Water Quality Guidelines for Fresh and Marine Water*":

1. Primary contact via sport activities in which a person comes into frequent direct contact with water, for example, swimming or surfing.
2. Secondary contact which involves less frequent body contact with the water, for example, boating or fishing, and
3. No body contact, for example, activities which stem from the visual recreational usage of the lake, for example, walking or relaxing near the lake to enjoy its natural beauty.

The applicability of various water quality guidelines to the three category waters is tabulated as shown in Table 4.2.

Table 4.2
Water Quality Criteria for Recreational and Aesthetics Requirements (ANZECC, 1992)

PARAMETER	PRIMARY CONTACT	SECONDARY CONTACT	NO BODY CONTACT
Microbiological	X	X	
Nuisance organisms #	X	X	X
Physical and Chemical:			
• Aesthetics	X	X	X
• Clarity	X	X	X
• Colour	X	X	X
• PH	X		
• Temperature	X		
• Toxic chemicals	X	X	
• Oil, debris	X	X	X

NB. # For example, algae

The Putrajaya Lake should be classified under the category for primary contact as it is the intention of the relevant authority to promote direct water contact sports like swimming in the lake. The water used for such activities should therefore be free from faecal contamination, pathogenic organisms, poor visibility and toxic chemicals in order to protect the health and safety of the user. In addition, for aesthetics reason the surface waters should be free from the following:

1. Floating debris, oil, grease and other objectionable matter.
2. Substances that produce undesirable colour, odour, taste or foaming.
3. Undesirable aquatic life such as "algal blooms", or dense growths of attached plants or insects.

Table 4.3 summarises the detailed water quality guidelines for water intended for recreational and aesthetics purposes.

Table 4.3
Summary of Water Quality Criteria for Recreational and Aesthetics Requirements (ANZECC, 1992)

PARAMETER	GUIDELINES
Microbiological*	The median bacterial content in fresh and marine waters taken over the bathing season should not exceed 150 faecal coliform organisms/100 ML or 35 enterococci organisms/100ML. Pathogenic free-living protozoans should be absent from bodies of fresh water.
Nuisance organisms	Macrophytes, phytoplankton scums, filamentous algal mats, sewage fungus, leeches etc should not be present in excessive amounts. Direct contact activities should be discouraged if algal levels of 15,000 – 20,000 cells/ML are present, depending on the algal species. Large numbers of midges and aquatic worms should also be avoided.
Physical and Chemical:	
• Visual clarity & color	To protect the aesthetic quality of a waterbody: • The natural visual clarity should not be reduced by more than 20%, • The natural hue of the water should not be change by more than 10 points on the Munsell Scale, • The natural reflectance of the water should not be changed by more than 50%. To protect the visual clarity of waters used for swimming, the horizontal sighting of a 200 mm diameter black disc should exceed 1.6 m.
• PH	The PH of the water should be within the range 5.0-9.0, assuming that the buffering capacity of the water is low near the extremes of the PH limits.
• Temperature	For prolonged exposure, temperatures should be in the range of 15-35 °C.
• Toxic chemicals	Water containing chemicals that are either toxic or irritating to the skin or mucous membranes are unsuitable for recreation. Toxic substances should not exceed levels given for untreated drinking waters.
• Surface films	Oil and petrochemicals should not be noticeable as a visible film on the water nor should they be detectable by odour.

Protection of Lake Aquatic Ecosystems

An aquatic ecosystem comprises the plant, animal and microbial communities that live in water and the physical environment and climate regime with which they interact. A healthy lake aquatic ecosystem is necessary to protect and enhance the recreational and aesthetic values associated with the lake and wetland system. The protection of an aquatic ecosystem is extremely complex as it involves many species of plants and animals, which all have their unique sensitivities and ecological requirements. In the case of the study area concerned, the following specific human activities can adversely impact upon the aquatic ecosystem:

1. Pollution caused by industrial, urban and agricultural activities.
2. Siltation and sedimentation from land clearance.
3. Nutrients (nitrogen and phosphorus) from fertilisers and detergent.
4. Diversion of flow in the river, and
5. Introduction of exotic species.

A set of water quality criteria needs to be formulated for the protection of the lake ecosystem. These may be broadly grouped as:

1. Physico-chemical, and
2. Biological indicators.

At present, the water quality indicators used are almost all physico-chemical and not biological. Due to the inherent variability of biological systems, there are marked differences in sensitivity of different ecosystems to particular pollutants and factors. ANZECC (1992) proposed the use of several biological guidelines as key indicators of the health of an aquatic ecosystem. The development of biological indicators for the protection of aquatic ecosystems is still a relatively new topic around the world. This is an aspect of the Putrajaya lake development which requires local scientific research.

The physico-chemical factors which are most important to the Putrajaya Lake are nutrients (nitrogen and phosphorus) and chlorophyll-a which are known to cause phytoplankton (algal blooms) in lakes and reservoirs. It is not possible to recommend a single set of nitrogen and phosphorus concentrations that will prevent the problem. Only through site-specific studies can the appropriate concentrations be determined for a particular ecosystem. ANZECC provided the following ranges of nutrient concentration as an indication of levels above which problems have been known to occur- depending upon a range of other factors:

- Total P 5-50 μg/l
- Total N 100-500 μg/l
- Chlorophyll-a 2-10 μg/l

Table 4.4 is a summary of the main water quality criteria for protection of lake ecosystem as proposed by ANZECC.

Table 4.4
Water Quality Criteria for the Protection of the Aquatic Ecosystems
(ANZECC, 1992)

PARAMETER	GUIDELINES
Physico-chemical:	
• Color & clarity	< 10% change in euphotic depth
• Dissolved oxygen	>6 mg/L
• Nutrients Chlorophyll-a	Total P 5-50 μg/L Total N 100-500 μg/L Chlorophyll-a 2-10 μg/L
• PH	6.5-9
• Salinity	<1000 mg/L
• Turbidity	<10% change in seasonal mean concentration
• Temperature	<2 °C increase
Toxicants (unit: μg/L)	
• Aluminium	<5 if PH<6.5, <100 if PH>6.5
• Ammonia	20-30
• Antimony	30
• Arsenic	50
• Beryllium	4
• Cadmium	0.2-2.0
• Chromium	10
• Copper	2-5
• Cyanide	5
• Iron	1000
• Lead	1-5
• Mercury	0.1
• Nickel	15-150
• Selenium	5
• Silver	0.1
• Sulfide	2
• Thallium	4
• Tin	0.008
• Zinc	5-50
Organic toxicants	Refer to Chapter 2 of ANZECC (1992)

Draft Putrajaya Lake Water Quality Guidelines

Based on the water quality criteria as discussed above, a draft guideline (Quek, 1998) has been prepared as part of a management plan for the Putrajaya Lake. The guideline is needed as this is the first time in Malaysia that an attempt has been made to protect and enhance the water quality in a large artificial inland water body. This task is given more urgency due to the presence of many known point sources of pollution upstream and the aesthetic and recreational values associated with the Putrajaya lake and wetlands. The *Draft Putrajaya Lake Water Quality Guidelines* is summarised as shown in Table 4.5.

It has been proposed to the *Putrajaya Catchment Management Committee* that the *Draft Putrajaya Lake Water Quality Guidelines* be reviewed in the proposed *Putrajaya Drainage and Sewerage Masterplan Study* with input from a multidisciplinary panel of experts.

Table 4.5
Draft Putrajaya Lake Water Quality Guidelines

PARAMETER	VALUE	PARAMETER	VALUE	PARAMETER	VALUE
BOD (mg/l)	3	Cr (IV) (mg/l)	0.05	SO_4 (mg/l)	200
COD (mg/l)	25	Cr (III) (mg/l)	-	S (mg/l)	0.05
DO (mg/l)	5-7	Cu (mg/l)	1	CO_2 (mg/l)	-
PH	6.5 - 9.0	Hardness(mg/l)	100	Gross - (Bql)	0.1
Colour (TUC)	150	Ca (mg/l)	-	Gross - (Bql)	1
Elect. Conductivity (umhos/cm)	1000	Mg (mg/l)	0.05	Ra - 226 (Bql)	+0.1
Floatables	NV	Na (mg/l)	-	Sr - 90 (Bql)	+0.1
Odour	NOO	K (mg/l)	-	CCE (µg/l)	500
Salinity (%)	1	Fe (mg/l)	0.3	MBAS/BAS (µg/l)	500
Taste	NOT	Pb (mg/l)	0.05	O & G (Mineral) (mg/l)	40; NF
Total Dissolved Solids (mg/l)	1000	Mn (mg/l)	0.1	O & G (Emulsified edible)(µg/l)	7000; NF
Total Suspended Solids (mg/l)	50	Hg (mg/l)	0.001	PCB (mg/l)	0.1
Temperature (C)	Normal 2	Ni (mg/l)	0.05	Phenol (µg/l)	10
Turbidity (NTU)	50	Se (mg/l)	0.01	Aldrin Dieldrin (µg/l)	0.02
Faecal Coliform (counts/100 ml)	150	Ag (mg/l)	0.05	BHC (µg/l)	2
		Sn (mg/l)	NR	Chlordane (µg/l)	0.08
		U (mg/l)	NR	t-DDT (µg/l)	0.1
Total Coliform (counts/100 ml)	5000	Zn (mg/l)	5	Endosulfan (µg/l)	10
Chlorophyll-a (µg/l)	10	B (mg/l)	1		
Total Nitrogen (mg/l)	0.5				
Total Phosphorous (mg/l)	0.05				
A1 (mg/l)	-	Cl (mg/l)	200	Heptachlor/	
As (mg/l)	0.05	Cl_2 (mg/l)	-	Epoxide (µg/l)	0.05
		CN (mg/l)	0.02	Lindane (µg/l)	2
Ba (mg/l)	1	F (mg/l)	1	2, 4-D (µg/l)	70
Cd (mg/l)	0.005	Silica (mg/l)	50.00	2, 4, 5-T (µg/l)	10
				2, 4, 5-TP (µg/l)	4
				Paraquat (µg/l)	10

Note:
NV - No visible floatable material or debris
NOO - No objectionable odour
NOT - No Objectionable taste
NR - No Recommendation
NF - Free from visible film, sheen, discolouration and deposits

It is emphasised that the *Draft Putrajaya Lake Water Quality Guidelines* is not the same as effluent discharge standard for waste water. Currently, there is no discharge standard in Malaysia suitable for the Putrajaya catchment. The Standards A and B by DOE (Environmental Act, 1986 revised) do not provide criteria to limit nutrients concentration for control of eutrophication in lake and is not suitable for the purpose of Putrajaya catchment water quality management.